

POLITECNICO DI MILANO



**VI Facoltà di Ingegneria Edile - Architettura
Polo Regionale di Lecco**

Master Science of Architectural Engineering 2012-2014

‘Lung Tsun Stone Bridge’ Archaeology Museum

Hong Kong, China



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ABSTRACT

This thesis is based on an international competition which intends to create suitable space, ambience and environment to facilitate the in-situ preservation of the remnants of Lung Tsun Stone Bridge as well as amplification of public awareness of different approaches in preservation, exhibition and interpretation of heritage resources.

The site is located in the north apron area of Kai Tak Redevelopment in Kowloon, Hong Kong whose overall landscape has been dominated by the former Kai Tak Airport site for decades. The history of the Kowloon City dates back to late Sung Dynasty, the coastline reclamation imprints, and remarkable aviation plan in Kai Tak area. A thorough study to the characteristics of this region is rather important for us to understand the authenticity of Kowloon Culture and to further identify the functionality of our design. On the other hand, within the planning zone of Kai Tak Redevelopment since 2007, our site is involved in the largest governmental development in Hong Kong. It will be in between schools, public rental buildings, multi-purpose sports complex and metro link during the next four years. The opportunities and restrictions have set a challenge for us to integrate the newly discovered bridge remnants into the development plan.

As a result, a garden overlapping the remnants and a museum suspending over it is our solution. At first, Fengshui is particularly studied to transform the ancient Chinese philosophical message into a modern language, regarding the relationship between the garden and the museum, the harmony between the museum and the surrounding context, and most importantly the link of people and surrounding environment.

In the urban level, the garden shall meet the requirements of 'ambience and environment of in-situ preservation and appreciation'. In an addition, a landscape garden provides a place of relaxation and decompression of citizens and visitors in this densely populated city, as well as a culture buffering zone between old Kowloon City and new Kai Tak area.

In the architecture level, the final form shows our respect to the concept of 'Bridge' as a tool of connection. It shall connect the past, present and future in the museum's function of interpreting the history, while it also connects the eastern architecture philosophy to the western construction skills. It is raised above the ground-plane, floating off the surface, allowing visitors to approach from different directions. The garden undulates beneath the structure, leading visitors into and out of the museum.

In the structural level, both regular steel frame structure and truss system are applied to support the double museum volume, namely ground-touching west wing and cantilever east wing. The design minimalizes the disturbance to the ruins with a structure floats over it. It captures a direct view from the nearby entrance piazza as a culture gateway.

In the technological level, façade texture, lights and indoor comfort are considered to assist the visitors relate the museum and garden from inside out while have them enjoy the time to appreciate the education, relaxation or just simple catch-up.

In the end, we believe our design is a satisfactory result.

ASTRATTO

Questa tesi é basata su una competizione internazionale che ha quale scopo di creare spazi idonei, atmosfera e ambienti per facilitare la preservazione in loco dei resti del ponte di pietra Lung Tsun, come pure sensibilizzare l'opinione pubblica sui vari approcci in preservazione, esposizione ed interpretazione di risorse del patrimonio culturale.

Il sito si trova a nord della zona di riqualificazione di Kai Tak a Kowloon, HKG, il cui paesaggio è stato dominato dall'ex aeroporto Kai Tak per decenni. La storia di Kowloon City ha inizio con la dinastia Sung, con l'avvio della bonifica della zona costiera, nonché con l'imponente piano aviaro nella zona di Kai Tak. Un approfondito studio delle caratteristiche di questa regione è piuttosto importante per capire l'autenticità della cultura di Kowloon e per meglio identificare la funzionalità del nostro design. D'altra parte, nella zona pianificatoria di Kai Tak dal 2007 il nostro sito è coinvolto nel più grande sviluppo governativo di HKG. Si situerà tra scuole, uffici pubblici, complessi sportivi poli-funzionali e raccordi della metropolitana nei prossimi quattro anni. Le opportunità e restrizioni hanno rappresentato per noi una sfida nell'integrare nel piano di sviluppo i resti del ponte di recente scoperti.

Di conseguenza, la nostra soluzione è consistita in un giardino che si sovrappone alle rovine, ed un museo sospeso sopra di esse. In principio, il Fengshui è stato particolarmente studiato per trasformare l'antico messaggio filosofico cinese in una lingua moderna, rispetto alla relazione tra il giardino e il museo, l'armonia tra il museo e il contesto circostante e, in primis, il legame tra la gente e l'ambiente circostante.

A livello urbano, il giardino deve adempiere ai requisiti di "atmosfera e ambiente di preservazione e apprezzamento in loco". Inoltre, un giardino paesaggistico offre un luogo di rilassamento e decompressione per cittadini e visitatori in questa città densamente popolata, come pure una zona di cuscino culturale tra la vecchia Kowloon City e la nuova area Kai Tak.

A livello architettonico, la forma finale dimostra il nostro rispetto del concetto di "ponte" come strumento di collegamento. Dovrà collegare il passato, il presente e il futuro nella funzione del museo di interpretare la storia, ed allo stesso tempo collegherà la filosofia dell'architettura orientale con le tecniche di costruzione occidentali. È sollevato sopra il terreno, fluttuando sulla superficie, rendendo possibile ai visitatori di approcciare il ponte da varie direzioni. Il giardino ondeggia sotto la struttura, conducendo i visitatori dentro e fuori il museo.

A livello strutturale, sia la regolare struttura con telaio in acciaio che il traliccio sono usati per sostenere il doppio del volume del museo, ossia l'ala ovest a terra e l'ala est sospesa. Il design minimizza il disturbo alle rovine con una struttura flutuante sopra di esse. Cattura una vista diretta dall'adiacente piazza d'entrata quale passaggio culturale.

A livello tecnologico, la struttura della facciata, le luci e il comfort interno sono considerati per aiutare i visitatori a relazionare con museo e giardino dall'interno mentre hanno la possibilità di godersi ed apprezzare l'educazione, rilassarsi o semplicemente socializzare.

Per finire, crediamo che il nostro design porti ad un risultato soddisfacente.

CHAPTER 1

COMPETITION INTRODUCTION

1.0 Competition Introduction

1.1 Competition Brief

1.1.1 The Competition

The Competition invites qualified professionals in the fields of architecture, engineering, landscape architecture, urban design, planning and surveying and the general public to contribute innovation concepts on the design for the Preservation Corridor for the Lung Tsun Stone Bridge Remnants (the Preservation Corridor) for reference and future implementation by the Government of the Hong Kong Special Administrative Region (HKSARG).

1.1.2 The Background

The Lung Tsun Stone Bridge (the Bridge) is a unique ‘heritage’ of Hong Kong in terms of its historical significance in relation to the socio-economic development of the Kowloon City District.

The Bridge was first built between 1873 and 1875 with a total length of about 200 metres to serve as landing steps to the Kowloon Walled City for both the Qing Dynasty government officials and local public. At the landward end of the Bridge, there was a two-storey pavilion known as the “Pavilion for Greeting Officials” (the Pavilion). A timber extension of about 80 metres was added in 1892, which was later on replaced by a concrete pier structure in 1910. Subsequently, a section of the Bridge ashore and the Pavilion were buried to make way for the development of Kai Tak Bund in 1924 while the remaining part was used as a pier for ferry services between Hong Kong Island, Hung Hom and Kowloon City until 1930s. Subsequently, the rest of the Bridge was demolished and buried under the reclaimed land accommodating the then military airport in 1942 during the Japanese occupation period in World War II; and eventually the development and expansion of the Hong Kong International Airport until 1998.

In April 2008, the remnants of the Bridge were first unearthed during an archaeological investigation under the Environmental Impact Assessment (EIA) study for the Kai Tak Development.

In the light of the historical significance of the Bridge, a two-stage public engagement exercise was carried out between June 2010 and April 2011 to foster public awareness and build consensus with the public for the preservation of the Bridge remnants. To take heed of public opinion and allow for the greater flexibility in the interpretation approach, provisions have been made in the latest approved Kai Tak Outline Zoning Plan (Plan No. S/K22/4) for a 30 metre wide preservation corridor with an ‘Open Space(3)’ zoning to facilitate in-site preservation of the Bridge remnants. The Preservation Corridor is also intended for the provision of outdoor open-air public space for leisure and/or recreational uses serving the needs of local residents as well as the general public.

In the quest for design excellence, the Preservation Corridor is subject to the competition to meet the public aspiration for the collection of design ideas and concepts from a wider spectrum of participants

and amplify the public awareness of different approaches in preservation, exhibition and interpretation of heritage resources.

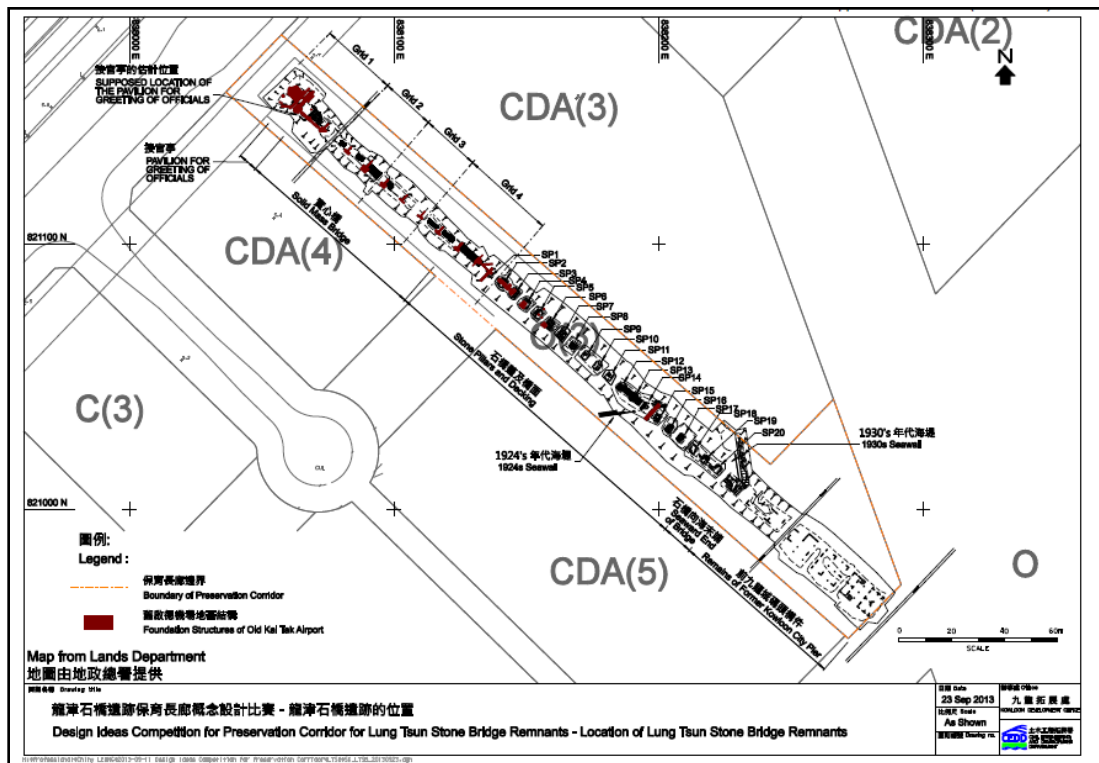
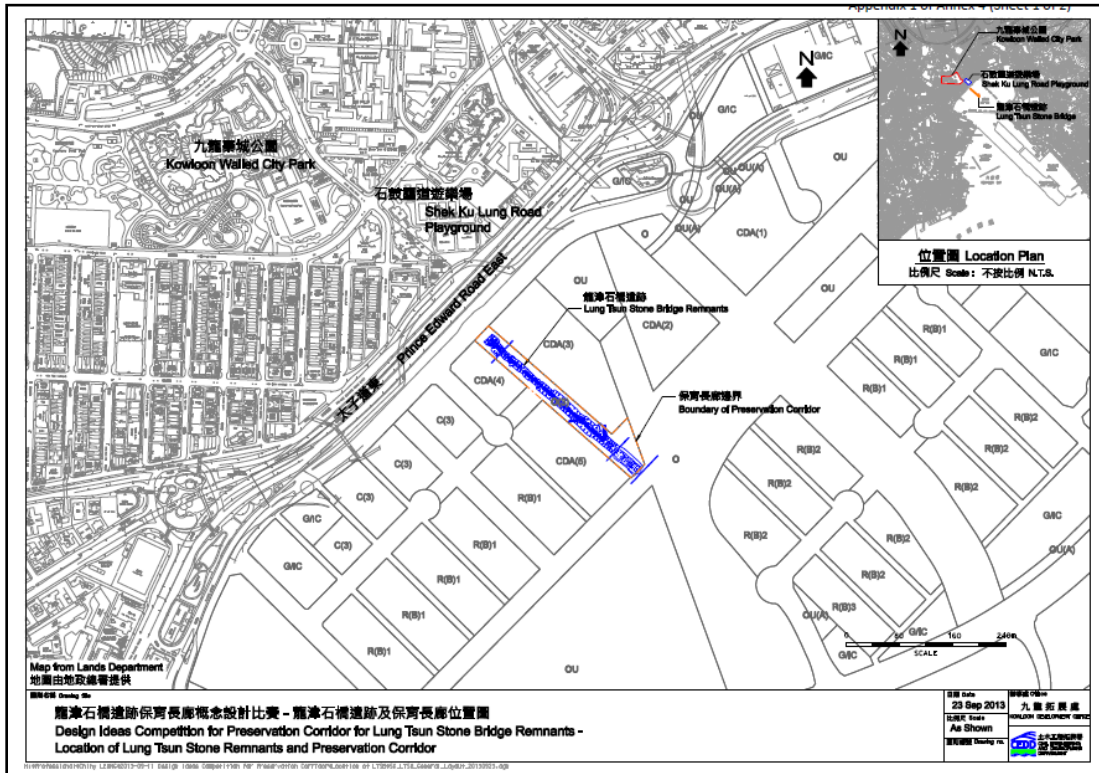


Figure 1-1 Competition Locations



Figure 1-2 North Facing of the Excavation Site

1.1.3 Objectives of the Competition

The objective of the Competition is to collect innovative concepts for creating suitable space, ambience and environment to facilitate in-situ preservation and appreciation of the Bridge remnants. The scope of the design competition also includes ancillary facilities to facilitate exhibition and interpretation of the Bridge remnants, the integration of the Preservation Corridor with the surrounding development and open space network, and the pedestrian connectivity. It encourages public participation in the design process. The design concepts should achieve the followings:

- Innovation, creativity, originality and identity to facilitate preservation, exhibition and interpretation of the Bridge remnants
- Appropriate ambience and setting to harmonize with the neighborhood developments, through the application of suitable urban design principles; and
- A theme for the Preservation Corridor for future design developments of the facilities.

CHAPTER 2

HONG KONG AND THE SITE

2.0 Hong Kong and the Site

2.1 Hong Kong

2.1.1 Geography



Figure 2-1 World map (Hong Kong)



Figure 2-2 Hong Kong city map

Location: Eastern Asia, in the southeast coast of China

Area: 1,104 km²(land), 1,650 km²(sea)

Land boundaries: city of Shenzhen in Guangdong Province

Coastline: total 733km

Islands: three main islands namely New Territories, Kowloon Island and Hong Kong Island with 263 islands

Climate: subtropical and monsoonal with cold dry winters and hot wet summers

Terrain: hilly and mountainous with steep slopes

Population: 7,155 million (2012)

Language: Cantonese, Mandarin and English

On 1st July 1997, Hong Kong was handed over to the People's Republic of China by the United Kingdom. The old Legislative Council was replaced by the Provisional Legislative Council elected by a selection committee whose members were appointed by the PRC government.

2.1.2 History and Development

Starting out as a finishing village, salt production site and trading ground, Hong Kong later evolved into a military port of strategic importance and eventually an international financial centre that has the world's 6th highest GDP per capita, supporting 33% of the foreign capital flows into China.

From 1800s to 1930s was the Colonial Hong Kong Era. By the early 19th century, the British Empire trade was heavily dependent upon the importation of tea from China. While the British exported to China luxury items like clocks and watches, there remained an overwhelming imbalance in trade.

Date	Treaty	Result
20 Jan 1841	Convention of Cheunpee	Preliminary cession of Hong Kong island to the United Kingdom
29 Aug 1842	Treaty of Nanjing	Cession of Hong Kong Island, founded as a crown colony of the United Kingdom
18 Oct 1860	Convention of Beijing	Cession of Kowloon (south of Boundary Street)
1 July 1898	Second Convention of Beijing	Lease of the New Territories (Including New Kowloon)

Table 2-1 History Brief of Hong Kong at 1800-1900

From 23 Dec 1941 to 15 Aug 1945, Hong Kong was occupied by Japan. The period, called '3 years and 8 months' halted the economy. The British, Canadians, Indians and the Hong Kong Volunteer Defence Forces resisted the Japanese invasion commanded by Sakai Takashi which started on 08 Dec 1941, eight hour after the attack on Pearl Harbor. By the end of the war in 1945, Hong Kong had been liberated by joint British and Chinese troops. The population of Hong Kong had shrunk to 600,000; less than half of the pre-war population of 1.6 million due to scarcity of food and emigration. The communist revolution in China in 1949 led to another population boom in Hong Kong. Thousands of refugees emigrated from mainland China to Hong Kong, and made it an important entrepot until the United Nations ordered a trade embargo on mainland China due to the Korean War.

After the Second World War, Britain chose to keep Hong Kong for strategic reasons. The political and institutional system made only minimal changes due to the political instability in Mainland China at that time which caused an influx of mainland residents to Hong Kong. In 1950s, Hong Kong transformed from a territory of entrepot trade to one of industry and manufacturing while 1960s is considered a turning point for Hong Kong's economy. The construction business was also revamped with new detailed guidelines for the first time since World War II. During 1970s, the opening of the mainland Chinese market and rising salaries drove many manufacturers north. High life expectancy, literacy, per-capita income and other socioeconomic measures attest to Hong Kong's achievements over the last four decades of the 20th Century. Higher income also led to the introduction of the first private housing estates with Taikoo Shing. The period saw a boom in residential high rises, many of the people's homes became part of Hong Kong's skyline and scenery.

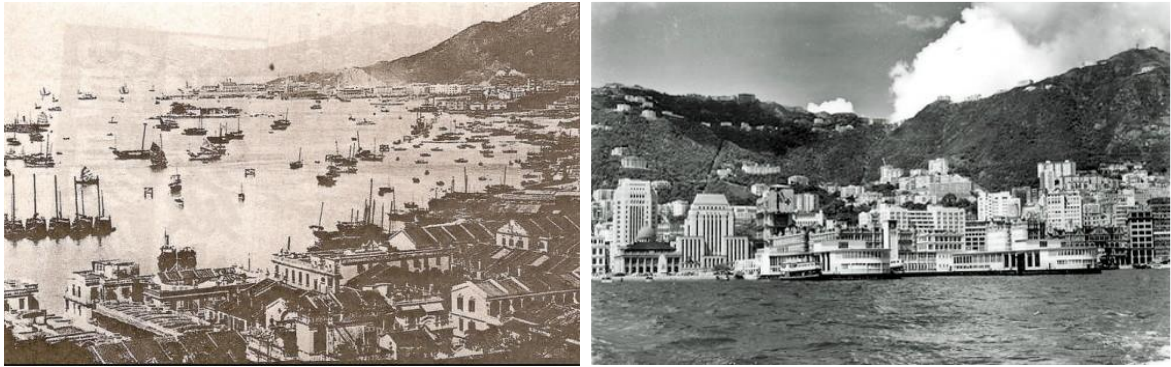


Figure 2-3 Hong Kong 1888 (left), 1960 (right)

In 1982, the British Prime Minister, Margaret Thatcher, hoped that the increasing openness of the PRC government and the economic reform in the mainland would allow the continuation of British rule. The resulting meeting led to the signing of Sino-British Joint Declaration and the proposal of the One Country, Two Systems concept by Deng Xiaoping. On 4 April 1990, the Hong Kong Basic Law was officially accepted as the mini-constitution of the Hong Kong SAR after the handover. In July 1992, Chris Patten was appointed as the last British Governor of Hong Kong.

On 1 July 1997 Hong Kong was handed over to the People's Republic of China by the United Kingdom. The old Legislative Council, elected under Chris Patten's reforms, was replaced by the Provisional Legislative Council elected by a selection committee whose members were appointed by the PRC government. Tung Chee Hwa, elected in December by a selection committee with members appointed by the PRC government, assumed duty as the first Chief Executive of Hong Kong.

The new millennium signaled a series of events, including SARS outbreak in 2003, Bird Flu Pandemic in 2009. Hong Kong's skyline has continued to evolve, with two new skyscrapers dominating, the 415 meter tall, 88 storey Two International Finance Centre and the 484 meter tall, 118 storey International Commerce Centre in West Kowloon



Figure 2-4 Hong Kong 1989 (left), 2013 (right),

2.2 The Site

2.2.1 Location

The site (30x300m) nowadays locates at south east of Kowloon Island, north apron area in Kai Tak Development Outline Zoning Plan, close to old Kowloon Centre-Kowloon City and looking to the old Kai Tak Airport Runway.

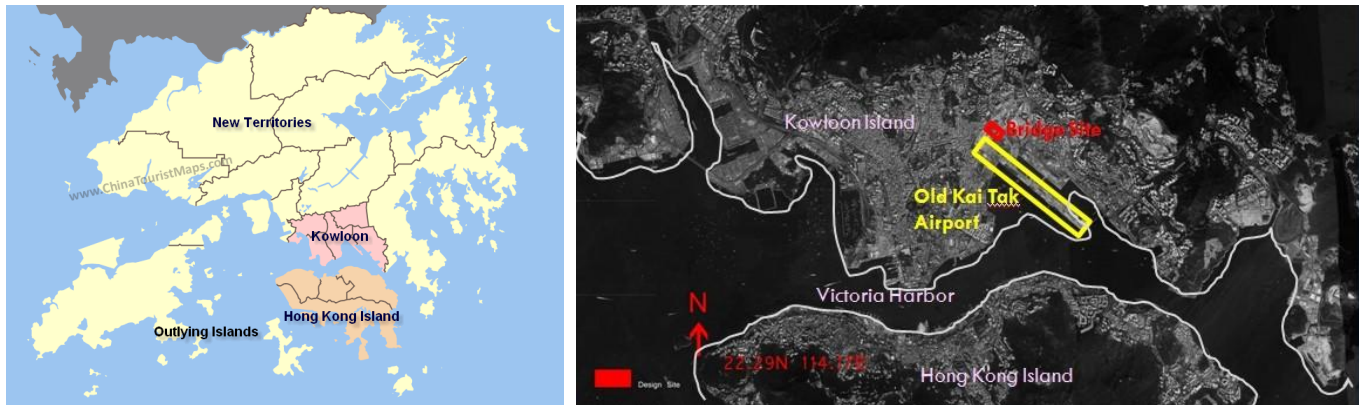


Figure 2-5 Scale of Bridge Site to Kai Tak Airport



Figure 2-6 Size and Direction inside Kai Tak Outline Zone Plan at the north apron area

2.2.2 History and Development

Following contents are mainly drawn from the Interpretation Principles and Guidelines for Lung Tsun Stone Bridge provided by Civil Engineering and Development Department, HKSAR.

Constructed from 1873 to 1875, Lung Tsun Stone Bridge (龙津石桥, LTSB) which was also known as Longin Bridge was a landing pier in the area of Kowloon City. It linked the East Gate of Kowloon Walled City (九龙寨城, KWC) and Kowloon Street (九龙街) to the coast of Kowloon Bay (九龙湾) from the late 19th century to early 20th century.

The Bridge that was physically connected to Lung Tsun Pavilion (龙津亭, also known as Pavilion for Greeting Officials) and the concrete Kowloon City Pier (The Pier) had undergone extension, reconstruction, maintenance and repair, demolition, and burial. LTSB was partially covered in the 1920s, and completely covered in the 1940s. Its site had also been used for different purposes. In 2008, remains of the Bridge were unearthed.

As a landing pier facilitating water transport, LTSB witnessed the evolution of commercial activities in Kowloon City, change in political power of Hong Kong, and the flourishing trade of opium and gambling. Its burial for Kai Tak Airport development also reflected the emergence of Hong Kong as an international city.

2.2.2.1 The Past of Kowloon City

● Settlement

Being a plain surrounding by hills and flowing streams, the area of present day Kowloon City had been well-developed for its waterway business. It is believed that from Han (漢) (206 BC- 220AD) to Southern Song (宋) (1127-1279) Dynasty the region was once dominated by salt production and it is believed that the surrounding areas were demarcated as forbidden zone to prevent smuggling. It was not until the late 12th century that village settlements appeared in the area.

By the mid 14th century Ng, Lee and Chan clans settled down in Kowloon City area. The establishment of a Tin Hau Temple reflected that regional identity had been developed. One of the oldest villages in Kowloon Peninsula, Nga Tsin Wai (衙前圍) was believed to be developed jointly by the three clans.

From 1662 to 1669, Qing(清) (1644-1911) government implemented the Great Evacuation to resist the beat back of Ming(1368-1644) Dynasty loyalists, which resulted in severe economic downturn of villages in Kowloon. After the lifting of the evacuation in 1669, the Qing (清) government encouraged villagers especially Hakka people, to set up villages along coastal areas. Dramatic changes were witnessed in Kowloon that many villages, including Kowloon Tsai, Kowloon Tong, Mongkok, Shum Shui Po, Cheung Sha Wan, Hung Hom and Tsim Sha Wai etc., were established in less than a century. In the 19th century, there were more than twenty villages established in Kowloon City area.

Village alliances such as Kowloon seven yeuk (九龍七約) and six heung (六鄉) were formed to solve internal problems among villages as well as external matters such as pirate defence. The former was

dated back to the first Dajiao (打醮) after Great Evacuation.

- **Kowloon Street**

'Kowloon Street' was not a street as indicated by its name, but an area / market between the foot of Bak Hok Mountain (白鶴山) to the coast of Kowloon Bay, deposited with mixed use buildings for shops and houses. It was concededly the most important location for commercial activities in Kowloon City area whose development was strongly related to its strategic location next to Kowloon Bay, a place with flat land and natural bay ideal for boat parking. The area was well connected to other areas by road and water transport network.

Early history of the Street could not be retrieved as it was not a market endorsed by the imperial government. In the late 18th century, the Street developed into a mature commercial district with a substantial concentration of shops. In 1846, a few hundreds of shops and houses were found in the area implying its prosperity. Its population was much bigger than other places on Hong Kong Island.

Before the mid 19th century, most trading activities at 'Kowloon Street' were performed by fishermen and residents outside Kowloon Peninsula. By the late 19th century restaurants and entertainment venues such as teahouses, wine shops, tobacco stores, opium dens, and gambling dens started to emerge. By then new markets had been formed in Yau Ma Tei (油麻地) and Shum Shui (深水) etc., but Kowloon Street was still the most significant market attracting business from the New Territories, Huishou (惠州) and Tamsui (淡水) etc. for its better prices. The flourishing commercial activities such as trading, fishing, and smuggling of opium, salt and commodities had undoubtedly created a basis for the conversion of an original pier into LTSB in 1873.



Figure 2-7 Shops surrounding Kowloon Street area at 1900s



Figure 2-8 Plan of Kowloon Street showing urban texture back in 1920s

- **Kowloon Walled City**

After the loss in the First Opium War (1840-42), Hong Kong Island was ceded to Britain under the Treaty of Nanking 《南京條約》 in 1842. Forts at Tsim Sha Tsui and Kwun Chung providing maritime defence were forced to be demolished and bombarded respectively. To strengthen the defence of Kowloon Peninsula, KWC was constructed between 25th November, 1846 and 31st May, 1847. It was a walled garrison-city that measured about 6.5 acres for the stationing of civil and military officials. Enclosed by massive stone walls with six watchtowers and four gates, Yamen, military buildings, gunpowder and ordnance magazines were deposited within KWC. It was physically separated from Kowloon Street and had no business activity.

The signing of Convention of Peking 《北京條約》 in 1860 after the loss in the Second Opium War (1856-60) extended Britain's colony –Kowloon (south of present day Boundary Street) was ceded to Britain. KWC located to the north of Boundary Street was not involved.

In 1898, under The Convention Between Great Britain and China Respecting an Extension of Hong Kong Territory 《中英展拓香港界址專條》 the area north of present day Boundary Street to south of Sham Chun River and surrounding islands were leased to Britain except KWC and the area of existing pier (LTSB) for the convenience of Chinese men-of-war, merchant and passenger vessels, which might come and go and lie there at their pleasure; and for the convenience of movement of the officials and people.

However, as the colonial government encountered strong resistance to its occupation in the New Territories in early 1899, the Governor Sir Henry Blake arranged a flag hoisting ceremony and delivered a speech to the elders of the villages of the Kowloon area on the 17th of April, in front of the

Chinese Customs Station, which was only a few yards from the beach outside KWC. On the 19th of May British troops were sent to force all Chinese civil and military officials to vacate from KWC. Subsequently the Order-in-Council was issued to legalize British jurisdiction in KWC in December. Despite the fact that the issue of LTSB was not mentioned, the colonial government repaired LTSB in 1900.



Figure 2-9 British Army in KWC at 1899



Figure 2-10 Kowloon City Development Scheme in 1920s with Red Highlighted Kowloon Walled City

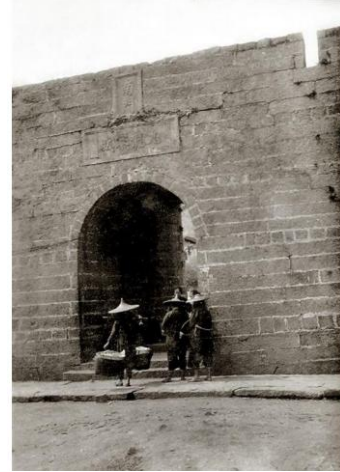


Figure 2-11 KWC at the end of 1800s (left), South Gate of KWC at 1920s (right)

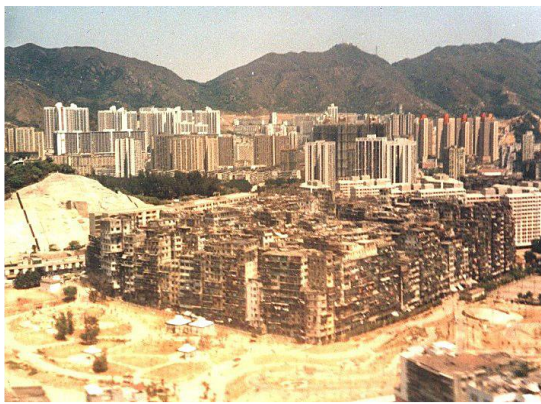


Figure 2-12 KWC in 1990

In 1987, with the agreement of the Chinese authorities, the decision was taken to clear the area and build a park on the site which would incorporate as many remaining features as possible of the original buildings and other features. The main discoveries were two carved granite plaques from the original South Gate, which had been the main entrance, one bearing the characters for "South Gate" and the other "Kowloon Walled City". The design of the park was inspired by the Jiangnan garden style of the early Qing Dynasty. Construction began in May 1994, and completed in August 1995. The park covers 31,000 square metres and is divided into eight scenic zones with individual characteristics but blending into the overall design.

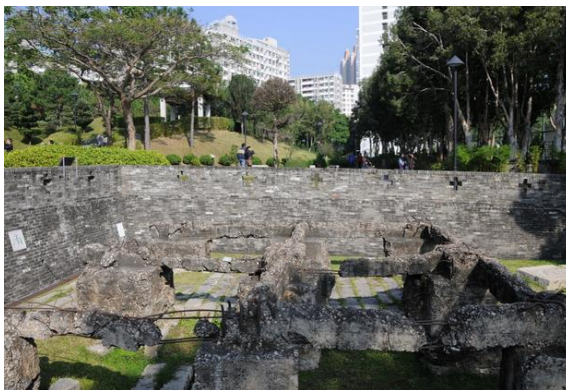


Figure 2-13 South Gate Remnants at Kowloon Walled City Park (left) and bird eye's view (right)

- **Opium and Customs**

After the end of the Second Opium War, Qing(清) imperial government was forced to sign a series of unequal treaties with Western powers. The terms included the legitimization of the trade of opium. Since then, opium smuggling using Hong Kong as the base drastically increased. Almost all the opium delivered to China came from Hong Kong. According to the treaties the import of opium would incur both import tax (進口稅) at trading ports controlled by Western powers and regional tax(地方稅) to the imperial government. In order to avoid import tax, smugglers opted for water transport from Hong Kong by wooden boats to non-trading ports.

In 1868, the imperial government established six Chinese Customs Stations (the Customs) at the east and west side of the boundary of Kowloon and Macau in order to collect regional tax from the wooden boats. The imperial government also ordered customs force vessels to block the smuggling. In 1871, the imperial government setup Chinese Customs Stations in Guangdong waters to impose customs tax (常關稅). Four of the Customs were found at the outbound of Hong Kong including Kowloon City. It is believed that the predecessor of LHSB and LHSB itself had served Chinese Customs force vessels in blocking opium smuggling.



Figure 2-14 Opium Smokers

- **Gambling**

Despite the enforcement of “An Ordinance for the Suppression of Public Gaming in the Colony of Hong Kong” in 1844 by the colonial government, illegal gambling dens were found everywhere. In 1867, the enactment of “The Order and Cleanliness Ordinance” was passed to control gambling by legitimizing it. In 1872 when public gambling was prohibited again, gambling dens moved to KWC that was then under Chinese jurisdiction to continue their operation.

The flourishing gambling business in KWC increased the use of water transport to Kowloon City, with LHSB as the landing pier.

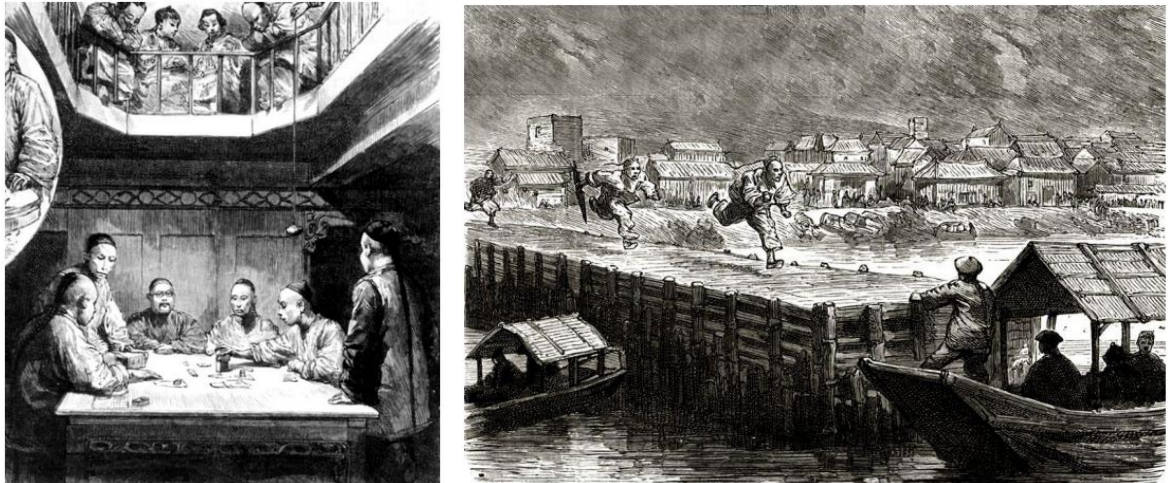


Figure 2-15 Comics of Gambling Dens in 1880s (left) and Gamblers on LTSB (right)

- **Kai Tak Airport**

In October 1910, an 85-meter long new concrete structure (Kowloon City Pier, the Pier) was constructed to replace the 1892 timber extension of LTSB. The latter ended in a “T” shape on plan was financed by fund raising led by Lok Sin Tong. In 1916, Kai Tack Land Development Company (KTLDC) owned by Sir Ho Kai and Mr Au Tak commenced reclamation at Kowloon Bay for Kai Tack Bund. A residential development scheme whose first stage development was completed in 1924 buried the northern portion of the LTSB. The Pavilion was demolished in 1920s for the construction of new roads.

In view of the rapid urban development of Kowloon Peninsula, the Hong Kong colonial government established Town Planning Committee in 1922 and issued Town Planning Scheme in the same year. The scheme created the grid morphology of rectangular blocks of buildings served by parallel streets. The residential development at Kai Tack Bund was the pioneer example of the grid system, followed by others coastal areas in Kowloon Peninsula.

Following the bankruptcy of KTLDC the Hong Kong colonial government took over the development project and designated the eastern part of the reclamation as the airfield for Royal Air Force (RAF) in 1925. In the same year the first recorded flight from Kai Tack took place. Kai Tak Airfield was constructed between 1927 and 1930. The first control tower and a hangar were built in 1935. The landing of the first commercial passenger flight in 1936 began its history as a civilian airport for public use.

Kowloon City Pier was reconstructed in ca.1932, and between 1936 and 1937. Before the Japanese Occupation, the remaining stone structures of LTSB and the new concrete structures still served as a landing pier in the Victoria Harbour.

Soon after the Japanese capture of Hong Kong in 1941, the new Japanese military government modified facilities of the airfield for their planes. This led to the demolition of tenement buildings and shop houses in Kai Tack Bund and eradication of villages in today’s San Po Kong. Debris from the site clearance together with rocks from the stone walls of KWC, Sacred Hill, Hammer Hill and Po Kong Village fung shui hill were used as fill materials for airfield extension. From 1943 to 1945 the Japanese

built two new runways and used the airport solely for military purpose. The remaining parts of LTSB and the Pier structure were then entirely covered.

After the Second World War, in 1945 the colonial government re-opened the Japanese military airport as an RAF airfield, and in 1946 the civilian airport resumed its operation. In 1947, the unsatisfactory seafront of the Japanese reclamation was reinforced by a newly constructed seawall across its front.

In 1954, the colonial government approved a master plan for Kai Tak Airport (the Airport) development. In 1958, a new northwest / southeast heading 2529 meter runway was constructed on a promontory into Kowloon Bay. It was also the year when the name Hong Kong International Airport (HKIA) was officially adopted.

The Airport continually expanded to cater for the ever-increasing capacity. The Terminal Building was extended five times, twice for the runway, and at least three times for the apron. It is noted that the works relating to the former Terminal Building and associated facilities has disturbed the underlying LTSB from 1950s to 1990s.

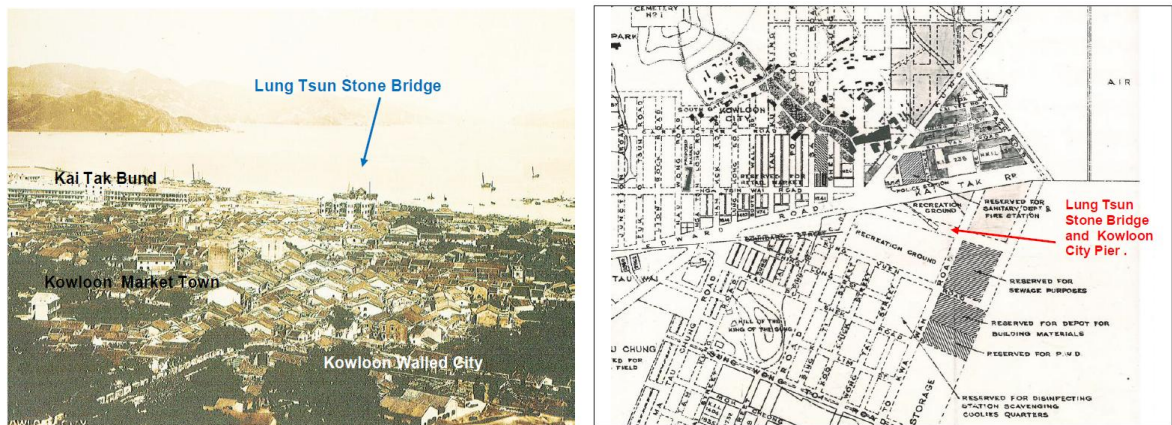


Figure 2-16 LTSB and Kowloon City Pier in 1910 (left), in 1930 (right)

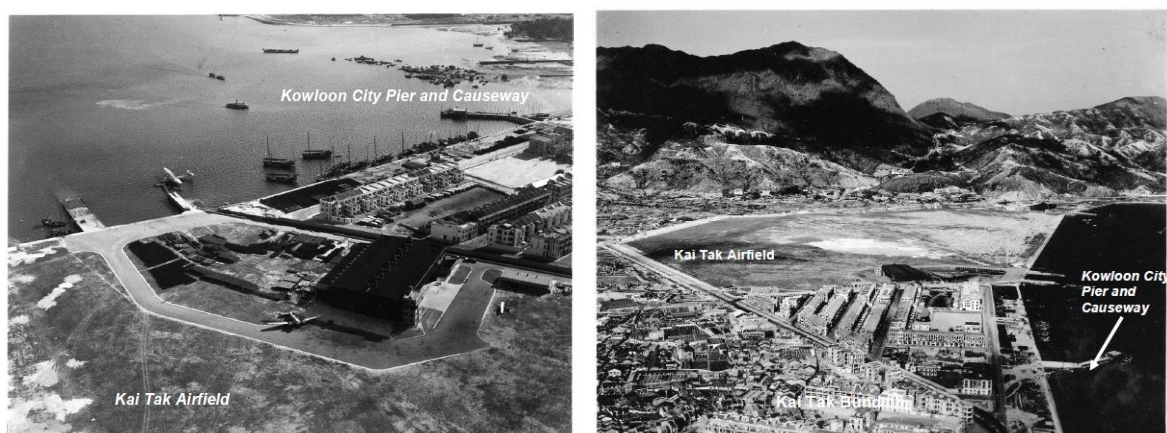


Figure 2-17 Kai Tak Airfield in 1930s

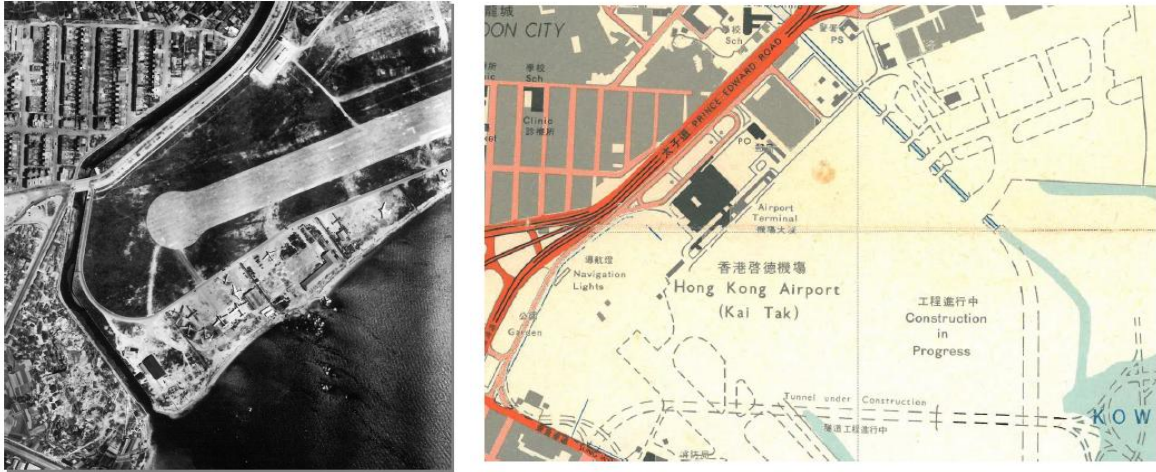


Figure 2-18 Kai Tak Airport Ariel Photo in 1954 (left), map in 1973 (right)

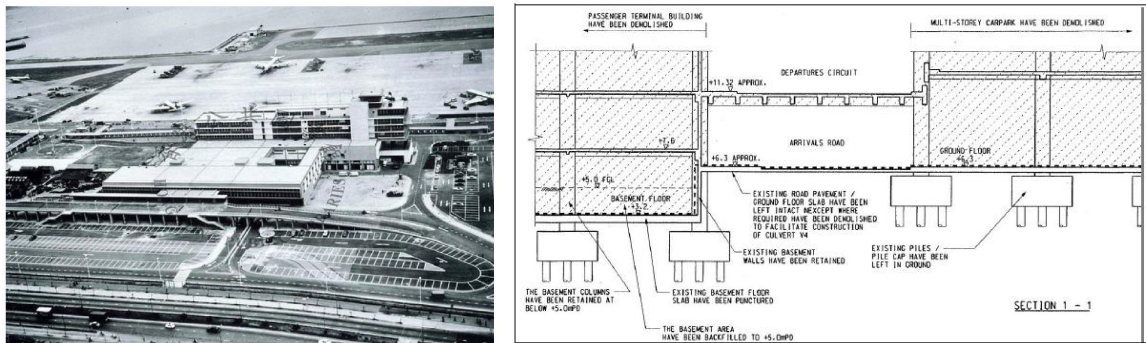


Figure 2-19 Kai Tak Airport Terminal Building in 1960 (left),
Section of Pile Cap of Terminal Building Basement (right)

The growth of Hong Kong also put a strain on the airport's capacity. Its usage was close to, and for some time exceeded, the designed capacity. The airport was designed to handle 24 million passengers per year, but in 1996, Kai Tak handled 29.5 million passengers, plus 1.56 million tonnes of freight, making it the third busiest airport in the world in terms of international passenger traffic, and busiest in terms of international cargo throughput. Moreover, clearance requirements for aircraft takeoffs and landings made it necessary to limit the height of buildings that could be built in Kowloon. While Kai Tak was initially located far away from residential areas, the expansion of both residential areas and the airport resulted in Kai Tak being close to residential areas. This caused serious noise pollution for nearby residents. A night curfew from midnight to about 6:30 in the early morning also hindered operations.

As a result, in the late 1980s, the Hong Kong Government began searching for alternative locations for a new airport in Hong Kong to replace the aging airport. After deliberating on a number of locations, including the south side of Hong Kong Island, the government decided to build the airport on the island of Chek Lap Kok off Lantau Island. A huge number of resources were mobilized to build this new airport, part of the ten programs in Hong Kong's Airport Core Programme.

The new airport officially opened on 6 July 1998. All essential airport supplies and vehicles that were left in the old airport for operation (some of the non-essential ones had already been transported to the new airport) were transported to Chek Lap Kok in one early morning with a single massive move.

On 6 July 1998 at 01:28, after the last aircraft departed for Chek Lap Kok, Kai Tak was finally retired as an airport. A small ceremony celebrating the end of the airport was held inside the control tower after the last flight took off. Richard Siegel, then director of civil aviation of Hong Kong, gave a brief speech, ending with the words "Goodbye Kai Tak, and thank you", before dimming the lights briefly and then turning them off.



Figure 2-20 Kai Tak Airport in Operation



Figure 2-21 The Closure of KT Airport in 1998

The Table 2-2 below summaries the historical events relating to the site.

Year	Events
Song(宋) (960-1279)	Present day Kowloon City was a part of Koon Fu Cheung (官富場), one of the salt yards (鹽場) governed by Chinese officials where trade of salt also took place
Yuan (元) (1271-1368)	「宋王臺」 (Sung Wong Toi) wordings were craved on a stone at the south of Kowloon City to commemorate the arrival of Duanzong(端宗) and Weiwang(衛王)
Qing (清) (1644-1911)	In 1668 first fortified when a signal station was established In 1810 a small fort (九龍寨炮台) was built at the head of the beach
1840-42	First Opium War
1842	Signing of Treaty of Nanking 《南京條約》 on the 29th of August, Hong Kong Island was ceded to Britain
1846-47	Between 25th November 1846 and 31st May 1847, a walled garrison-city (later known as Kowloon Walled City (KWC)) was constructed for maritime defence
1847	Completion of Yamen (衙門) within KWC and establishment of the Longjin Free School
1854	KWC was captured by rebels during the Taiping Rebellion (太平天國)
1856-60	Second Opium War
1860	Signing of Convention of Peking 《北京條約》 on the 18th of Oct, Kowloon (south of present day Boundary Street) was ceded to Britain
1860s	Hong Kong became the centre of opium smuggling
1871	Setting up of Chinese Customs Station in Kowloon City for opium tax collection
1872	Re-prohibition of public gambling
1873-75	Construction of LTSB
1892	Construction of timber extension of LTSB, ending with “T” shape on plan. Lok Sin Tong played a leading role in the fund raising
1898	Signing of The Convention Between Great Britain and China Respecting an Extension of Hong Kong Territory 《中英展拓香港界址專條》 on the 9th of June, KWC and LTSB remained under Chinese jurisdiction
1899	Sir Henry Blake’s flag hoisting ceremony and speech on the 17th of April, in front of the Chinese Customs Station, outside KWC Chinese civil and military officials were forced to vacate from KWC on the 19th of May
1900	Repairing of LTSB by the Hong Kong colonial government
1910	Erection of a new concrete pier in Kowloon City (Kowloon City Pier, the Pier) in October to replace the timber extension of 1892
1916-24	First stage Kai Tack reclamation at Kowloon Bay to develop Kai Tack Bund, a residential housing scheme by Kai Tack Land Development Company(KTLDC)
1920s	Demolition of Lung Tsun Pavilion and burial of northern part of LTSB for the reclamation
1925	Kai Tack reclamation used as Royal Air Force (RAF) airfield with the first recorded flight
1927-30	Construction of Kai Tak Airfield
ca.1932	Reconstruction of the Pier with a wooden shelter

Year	Events
1933	Construction of a causeway joining the seaward end of LTSB
1935	Completion of the first control tower and a hangar in Kai Tak
1936	Landing of the first commercial passenger flight in Kai Tak
1936-37	Reconstruction of the Pier with a ramp linking LTSB
1941-45	The walls of KWC were modified for the extension of Lung Tsun River (龍津河) (present day Kai Tak Nullah (啟德明渠) or Kai Tak River (啟德河))
1942	Part of LTSB and the Pier were buried under the airfield during further reclamation of Kowloon Bay
1943-45	Construction of airport runways during the Japanese Occupation period covering the remaining LTSB and the Pier
1943	The walls of KWC were demolished during Japanese Occupation to provide fill material for Kai Tak Airfield extension. Sung Wong Toi was also demolished
1945	Japanese Occupation ended on the 15th of August. RAF airfield was re-opened
1946	Civilian airport re-opened
1954	Approval of master plan for the development of Kai Tak Airport on the 16th of June
1957	Completion of reclamation for Kai Tak Airport
1958	Construction of a new runway on a promontory into Kowloon Bay. The name Hong Kong International Airport (HKIA) was officially adopted for Kai Tak Airport
1959	Completion of expansion of HKIA
1962	Completion of a passenger terminal building (PTB) in HKIA
1965	Demolition of the old Kai Tak Terminal Building
1974	Completion of Airport Runway extension
1975	Extension of Airport runway to 3,390 meters to meet the long haul flight requirements
1981	Completion of Stage 4 development of the PTB, HKIA
1984-88	Stage 5 development of the PTB, HKIA
1987-94	Kowloon Walled City was demolished
1995	Kowloon Walled City Park was opened on the site of Kowloon Walled City
1998	Relocation of HKIA from Kai Tak to Chek Lap Kok

Table 2-2 Timeline showing the development of the historical events related to the Site

2.2.2.2 The Past of LTSB

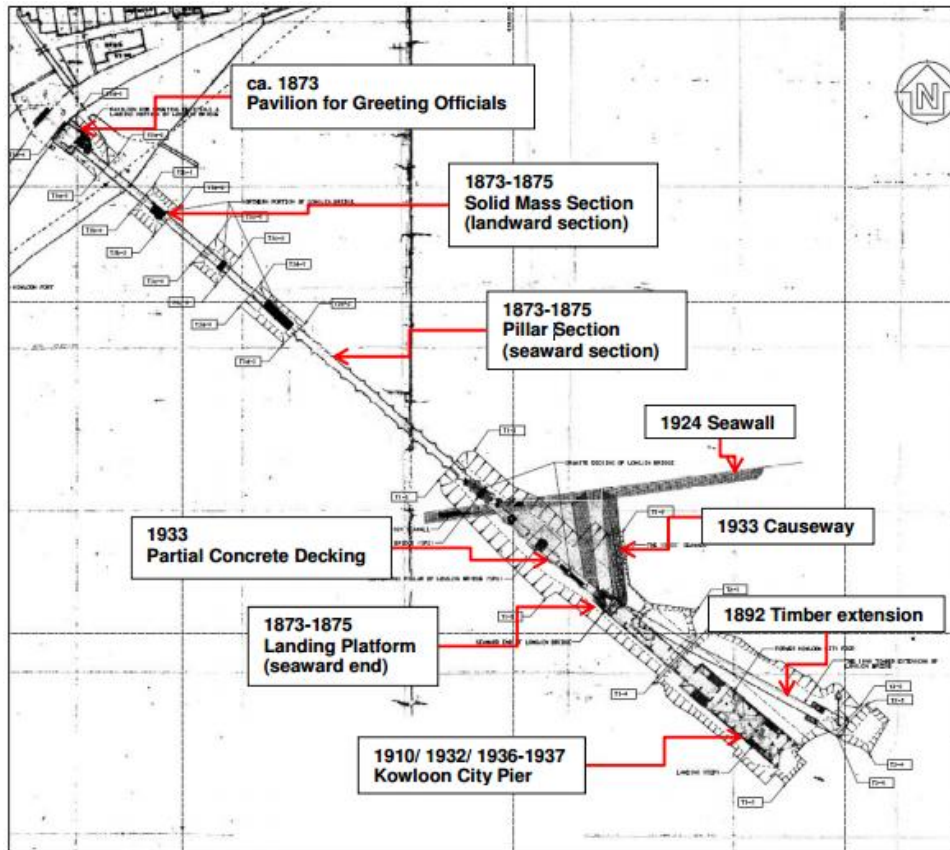


Figure 2-22 General layout plan of LTSB excavation overlaid with historical maps

LTSB and the Pavilion were prominent landmarks on the Kowloon coastline in the late 19th century.

- **The Pavilion**

It marked the landward end of LTSB. It was used by the local elders to greet new officials to the yamen (Chinese court) in KWC. A stone lintel of 1873 inscribed with two large Chinese characters 龙津 (Lung Tsun) was placed at its main entrance but no direct evidence was available to confirm its year of construction. It was a two-storey building constructed of bricks and stones with Chinese traditional roof system. It was 8 meters wide and 7 meters long.



Figure 2-23 Pavilion for Greeting Officials in 1898 (left), in 1910s (right)



Figure 2-24 Southern entrance in 1900s (left), Northern entrance, ca. the 1910s (right)

- **The Bridge**

The LTSB built between 1873 and 1875 was around 200 meters long. It was composed of three major sections namely:

- (1) Solid Mass Section (landward section),
- (2) Pillar Section (seaward section)
- (3) Land Platform (seaward end)



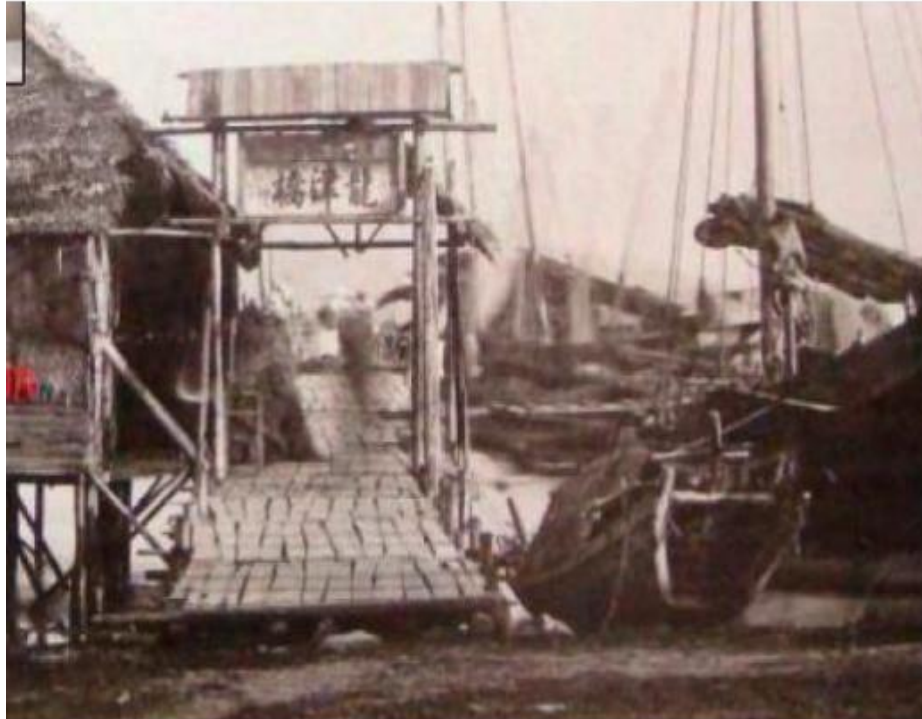


Figure 2-25 Photo Record of LSTB

The bridge was extended and reconstructed for several times

- (1) As the beach silted up and LSTB was not suitable for cross harbor ferries during low tides, in 1892 a timber extension in a slightly different orientations was added. Similar to the seaward end, it also ended with a 'T' shape on plan
- (2) In 1910, the timber extension was modified by the Public Works Department
- (3) In ca.1932 a new replacement concrete structure namely the Kowloon City Pier with a wooden shelter was constructed
- (4) Between 1936 and 1937 the extension was reconstructed. It was the last modification of LSTB. A ramp was built between the northern portion of the Pier and LSTB. The Pier was supported by forty-seven concrete pillars with or without inclined pillars.

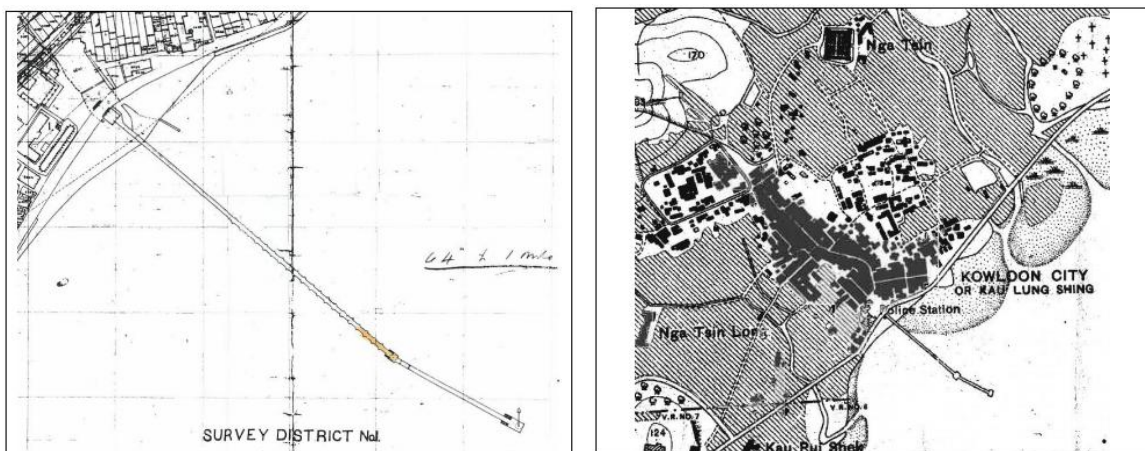


Figure 2-26 Survey Map of LSTB IN 1900s (left), prior to reclamation in 1904 (right)



Figure 2-27 Kowloon City Pier in 1932

- **A seawall**

It was built in 1924 to reinforce the LTSB during the development of Kai Tak Bund. It was later modified in the 1930s.

- **Causeway**

A causeway was added to the seaward end in 1933.

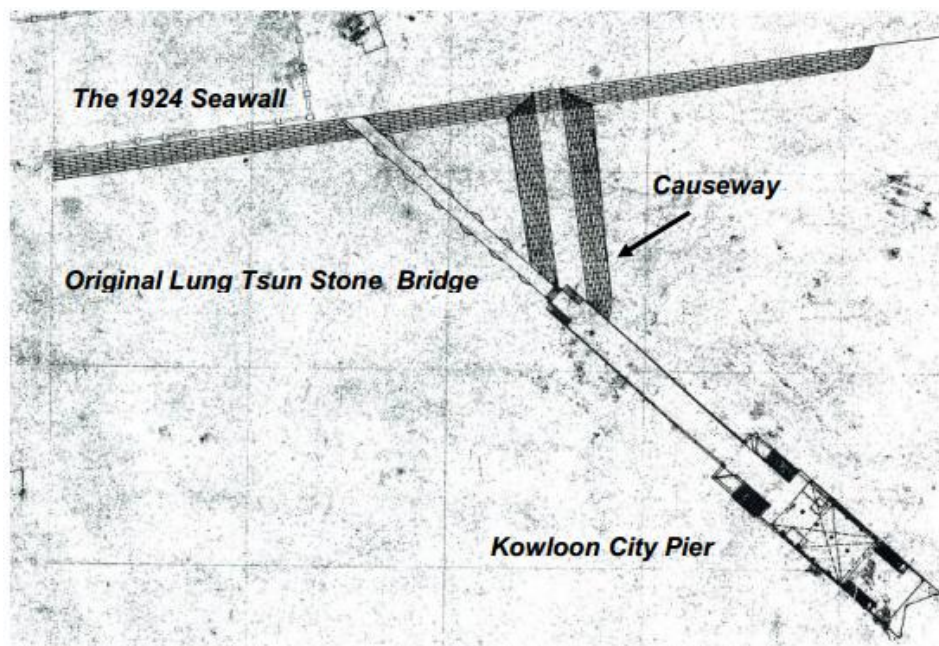


Figure 2-28 Seawall, Causeway and the Pier in 1930s

The Pavilion, LTSB, Pier, Seawall and Causeway disappeared from sight either by demolition or burial. In 1920s, the Pavilion was demolished for the construction of new roads and the northern portion of LTSB was buried for Kai Tak Bund Development. The remaining parts of the LTSB and the Pier was buried in the 1940s during the construction of Japanese military airport, the predecessor of Kai Tak Airport.

Table 2-3 below shows the development of LTSB and Ancillary Structures.

Year	Development of LTSB and Ancillary Structures
ca. 1873	Construction of Lung Tsun Pavilion
1873-1875	Construction of Lung Tsun Stone Bridge
1892	Addition of a wooden extension in different orientation to the seaward end of LTSB
1900	Repairing of LTSB by the Hong Kong colonial government
1910	Erection of a new concrete pier
1916	Renewing reinforced concrete beams and repairing masonry piers
1920s	Partial burial of LTSB by Kai Tack Bund, demolition of the Pavilion to make way for the construction of new roads
1921-1922	Maintenance
1923	The decking of the concrete pier was badly damaged and general repairs were carried out.
1924	Construction of Seawall
1925-1927	Maintenance
1930	Rewiring, navigation light and general repair
1931	Maintenance
ca. 1932	Reconstruction of the Pier
1933	Construction of Causeway to the seaward end of LTSB
1934	Raising level of causeway to the Pier
1936-1937	Reconstruction of the Pier
1940s	Complete burial of the site by Japanese Army to extend the Kai Tak airfield for Japanese military airport

Table 2-3 Timeline of showing the development of LTSB and Ancillary Structures

2.2.2.3 The Present of LTSB Site

● Kai Tak Development

A Feasibility Study was completed in 1998 followed by a revised scheme in 2001 based on the public concerns over harbor reclamation. As a result, Government in 2004 commissioned the Kai Tak Planning Review with “zero reclamation” as the starting point to carry out an extensive 3-stage Public Participation Program, arriving at a Preliminary Outline Development Plan (PODP). After rounds of public consultation and with the guidance of the Town Planning Board, the statutory Kai Tak Outline Zoning Plan (Kai Tak OZP) was formulated in 2007 with subsequent changes made in 2009 and 2012.

Environmental Impact Assessment (EIA) Report was submitted under the Environmental Impact Assessment Ordinance (EIAO), which demonstrates that Kai Tak Development (KTD) will comply with environmental standards and legislation after mitigation measures for the construction and operation stages are implemented. The EIA Report was approved on 4 March 2009.

Kai Tak Development is a huge and highly complex development project spanning over 320 hectares with the largest available land fronting Victoria Harbour. It offers opportunities to bring the harbor to the people, provide quality living environment for around 90 000 residents, as well as revitalize all of the surrounding districts such as Kowloon City, Wong Tai Sin and Kwun Tong. What’s more, KTD seeks to practice sustainable development and cultivate a comprehensive network of parks and gardens for everyone to enjoy. The planning vision of KTD is to develop “a distinguished, vibrant, attractive and people-oriented community by the Victoria Harbour”.



Figure 2-29 Division of Kai Tak Development Area (KTD)

- **The Archaeological Finds of LTSB**



Figure 2-30 Photo of Excavation site, 2009

Following contents are mainly drawn from the full excavation report provided by Civil Engineering and Development Department, HKSAR.

In 2008, an archaeological investigation was conducted in the EIA study for KTD. Remains of the southern section of LTSB were first discovered. Following the recommendation of the EIA report, a further investigation was completed in February 2009. Several sections of the northern part of LTSB as well as the three foundation walls of the Pavilion were found at a level below the basement floor of the former Terminal Building. Forty-seven broken concrete supporting pillars and landing steps of the Pier were also discovered at the open area of former North Apron.

In 2012, a total of four girds and one trenches (which was divided into three bays, namely as Trench 1a, 1b and 1c) were excavated along the footprint of LTSB. A full archaeological excavation was carried out to unearth all the remaining features of LTSB and the ancillary structures along its entire length of 224 meters, including Pavilion of Greeting Officials, Solid Mass Section, 20 Stone Pillars, Partial Concrete Decking, Seawall, and Landing Platform. A total of 42 pieces of blue and white porcelain sherds, total of 2 pieces of glass sherds and 1 glass medicine bottle and total of 2 storage ware sherds are found which can be dated back to late Qing Dynasty to early Republican Period. The LTSB features were exposed after the excavation in 2008 and 2009. The loose fragments at the surface of the remains, such as (a) individual or small granite blocks, and (b) concrete or sandy mortar was affected by soil movement, storm water, surface runoff, ground water changes, and the growth of vegetation etc. After the 2012 excavation study the whole length of the exposed features was backfilled to the current ground level for proper protection.

Excavation Plan and Section

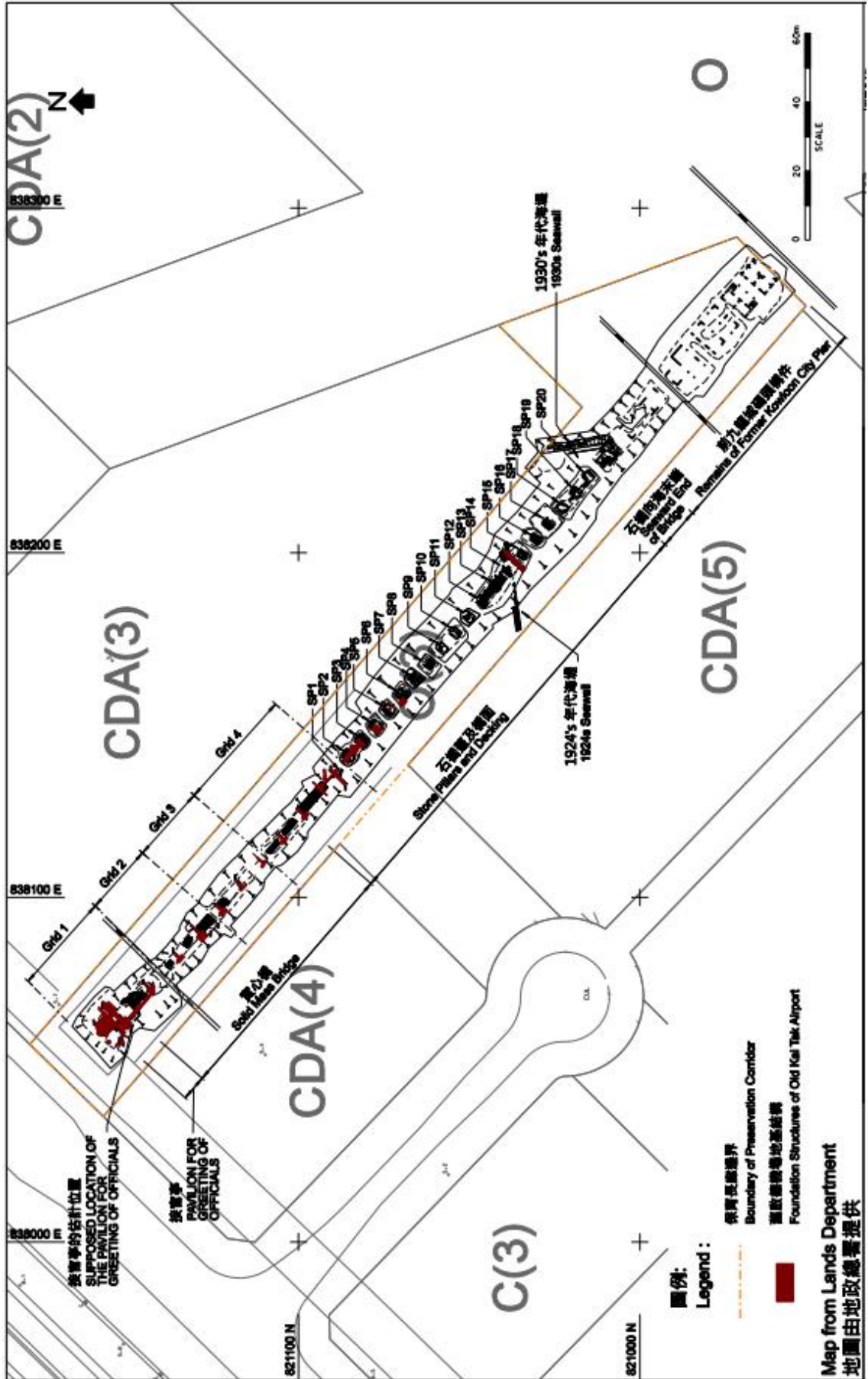


Figure 2-31 Ruins Distribution Map in Plan

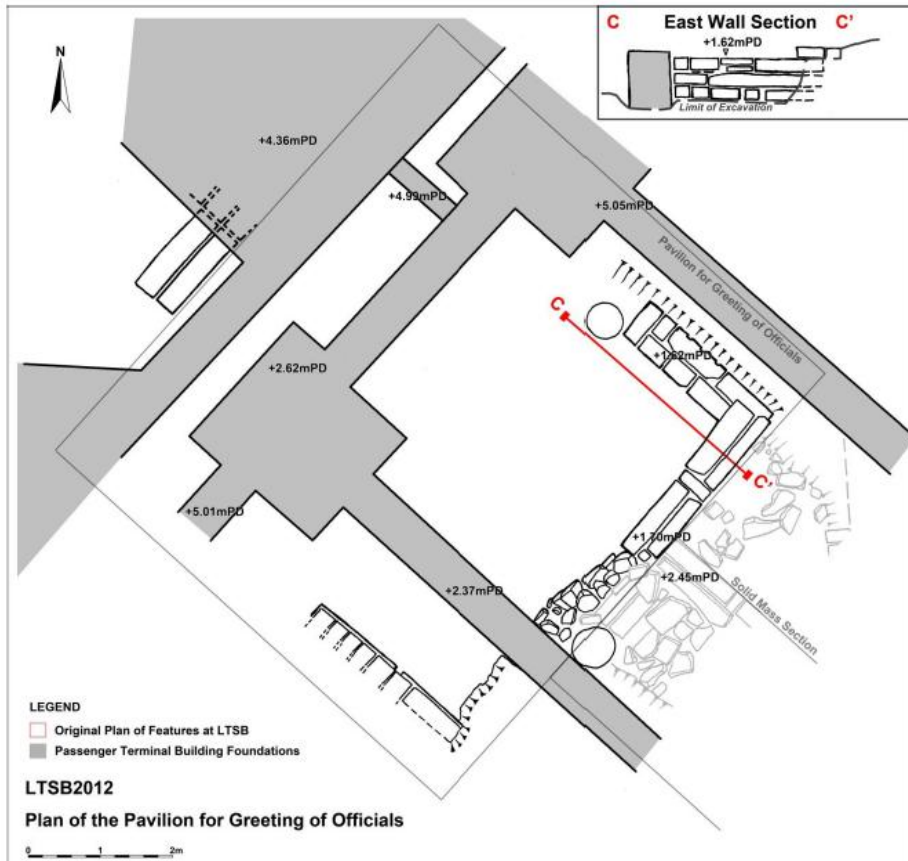


Figure 2-32 Plan Record of test pit for Pavilion, Grid 1, Grid 2

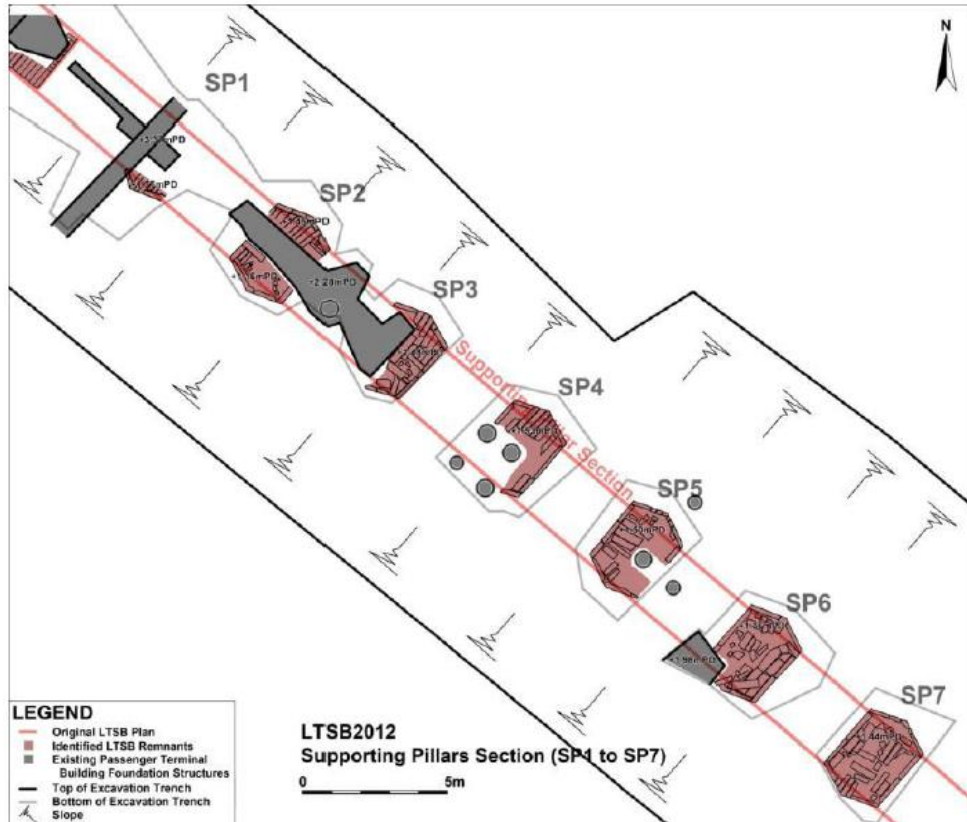
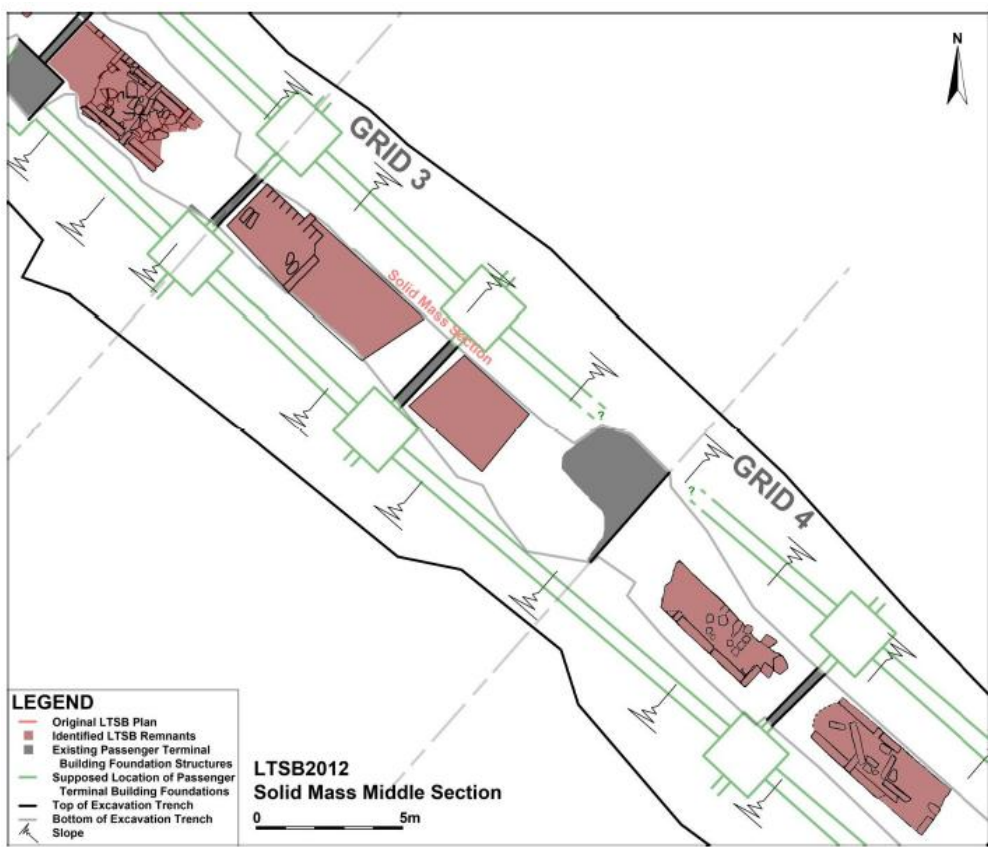


Figure 2-33 Plan Record of test pit for Grid 3, Grid 4, SP 1-7

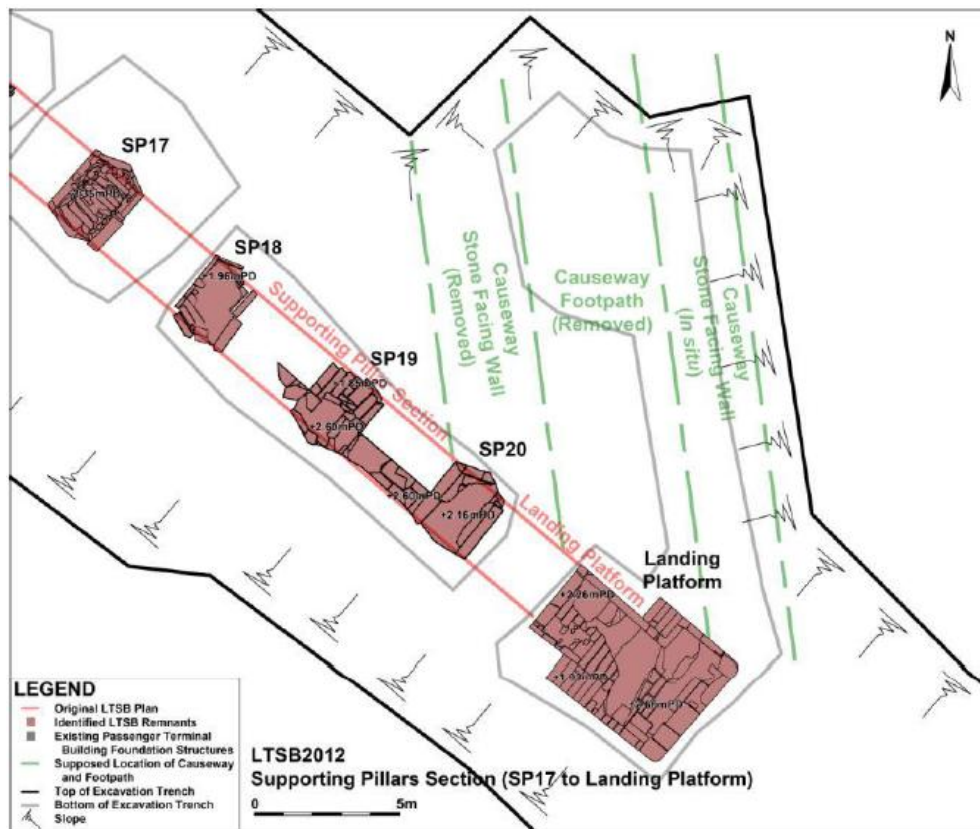
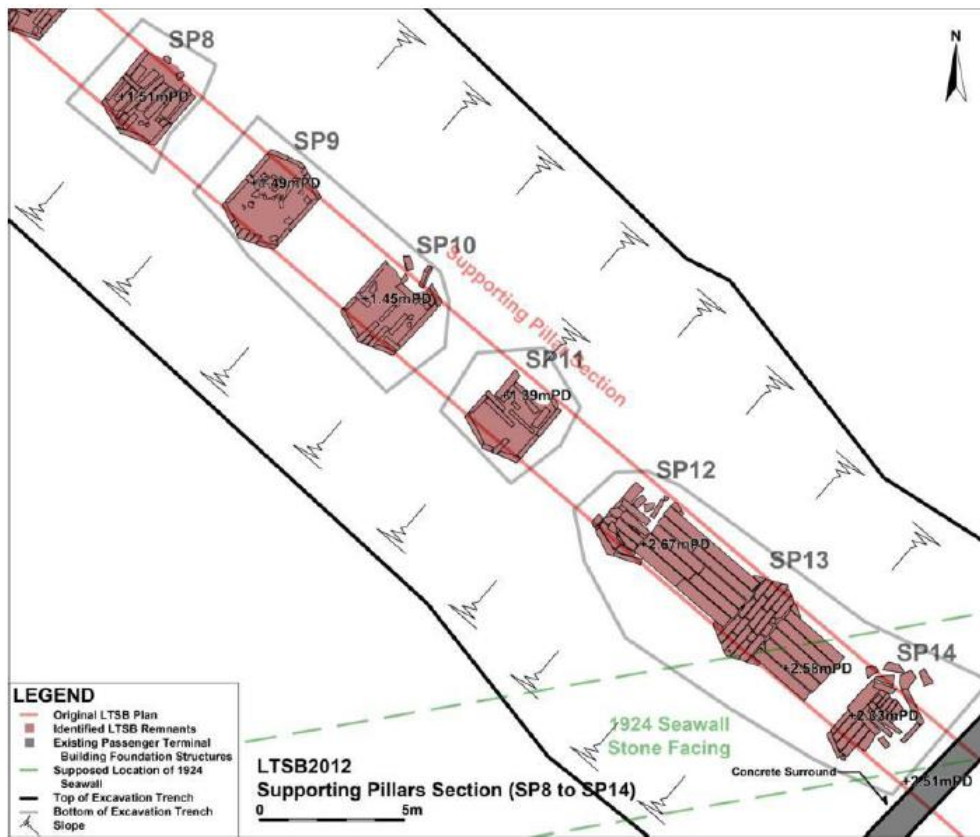


Figure 2-34 Plan record of test pit for SP8-20, Platform

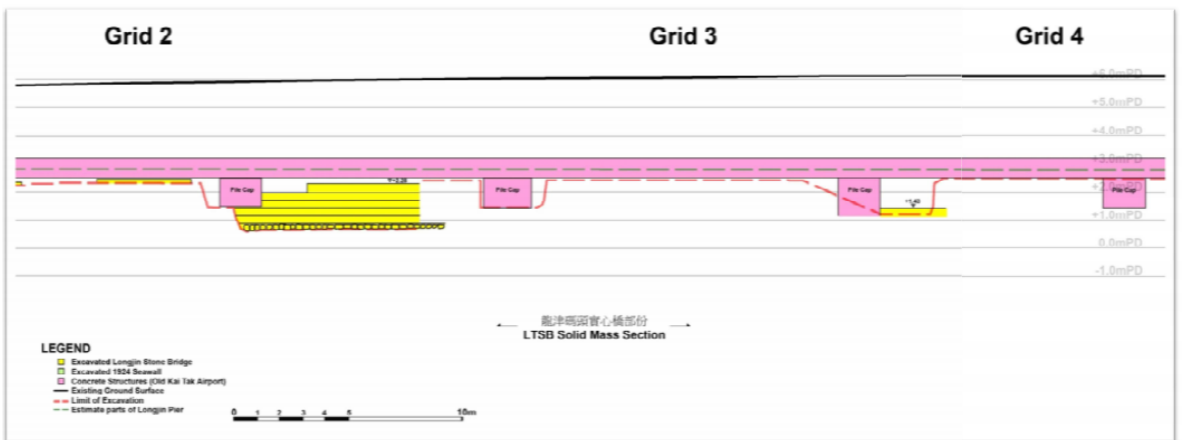
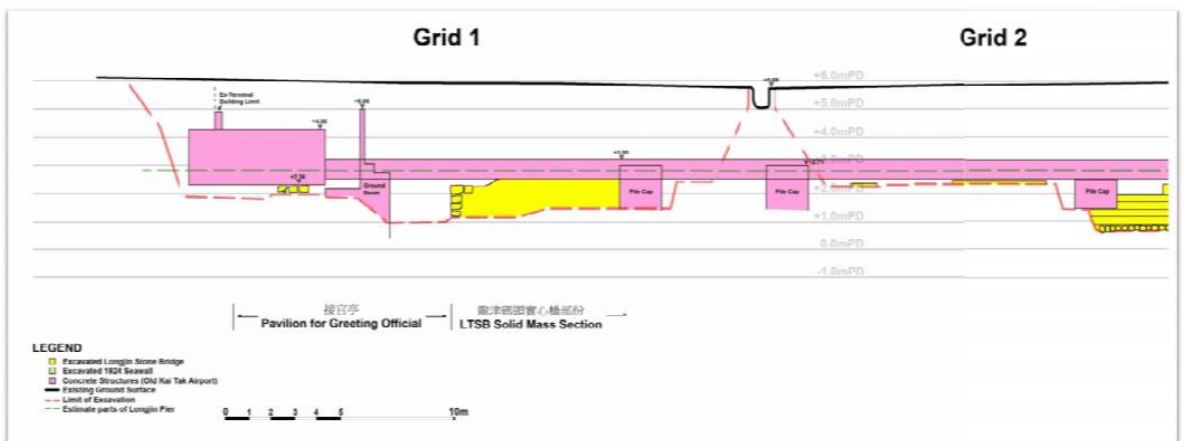
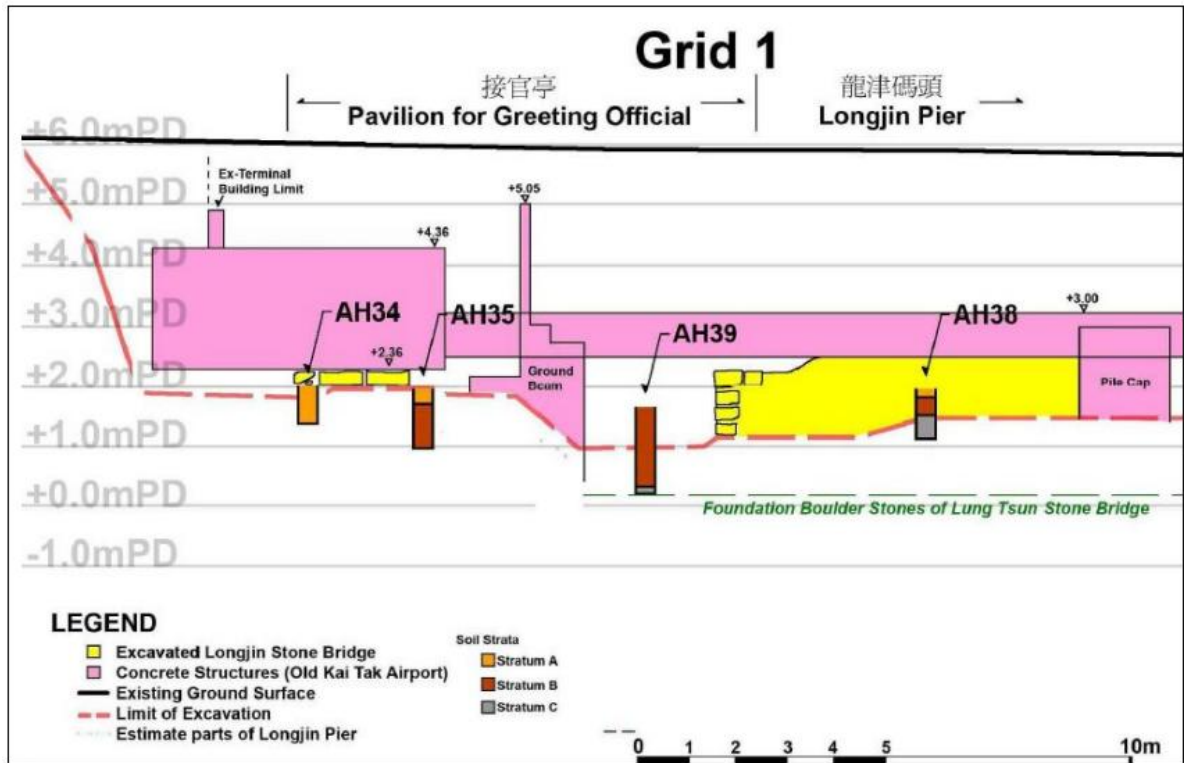


Figure 2-35 Section record of test pit for Pavilion, Grid 1-4

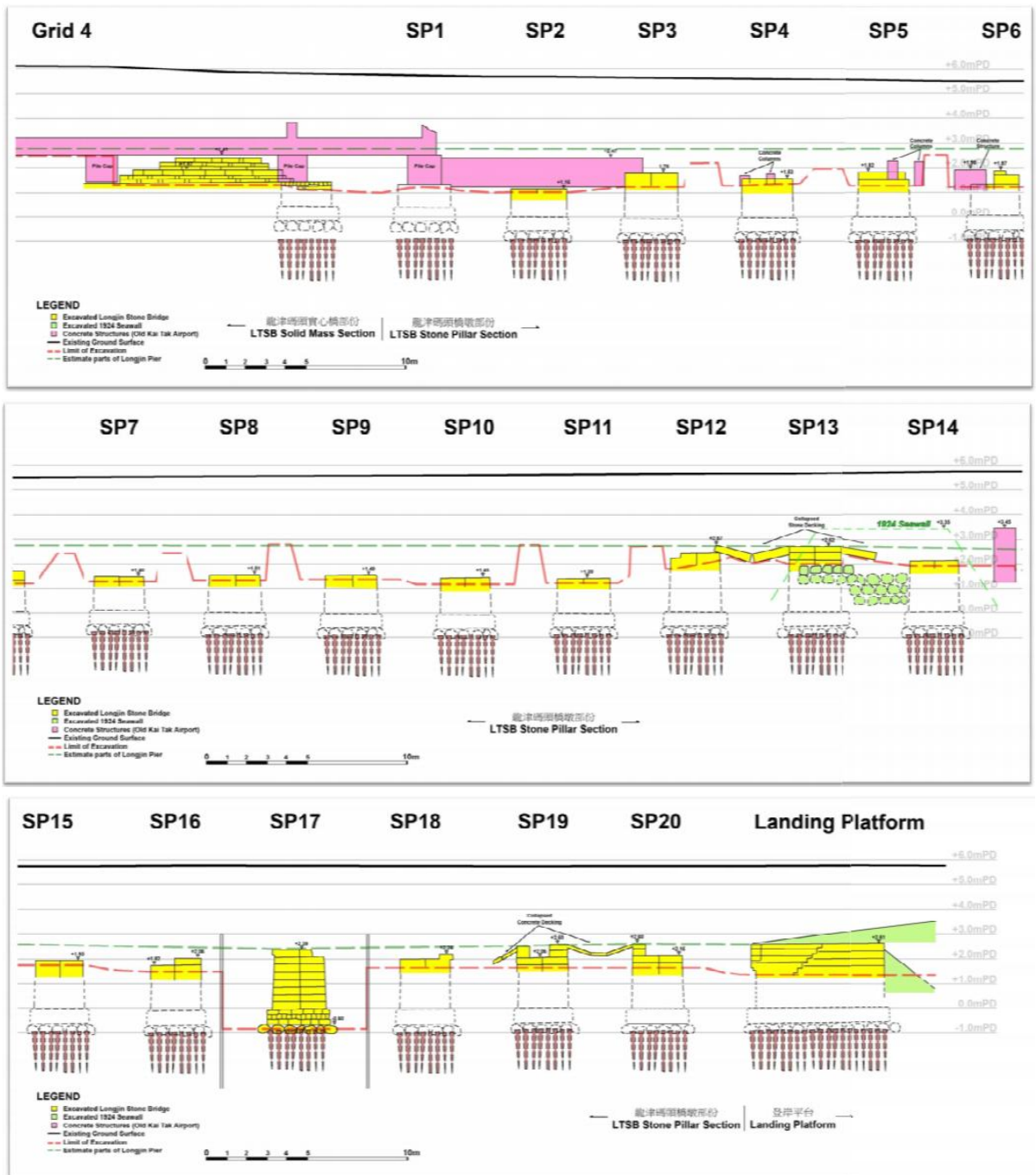


Figure 2-36 Section record of test pit for SP1-20 and Platform

Item Size Data

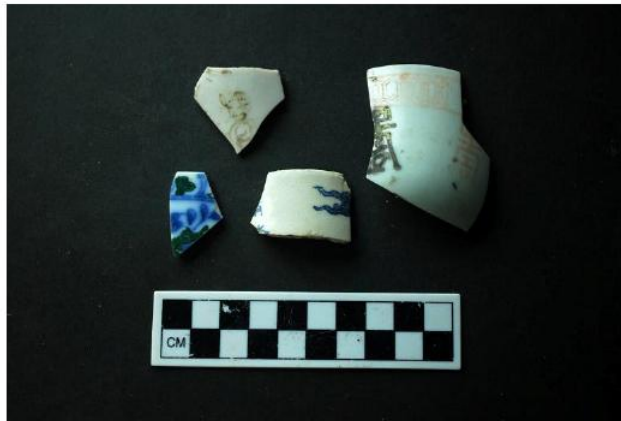
Item	Size (length x width) in m	Uppermost of Layer (mPD)	Notes
Solid Mass in Grid 1 (Pavilion)	7.0x8.0	+ 2.49	
Solid Mass in Grid 2	2.4x1.2	+ 2.54	
Solid Mass in Grid 3	2.4x2.2	+ 1.89	
Solid Mass in Grid 4	4.3x2.1	+ 2.26	NW
	3.4x1.2	+ 1.85	SE
SP1	2.8x1.0	+ 1.77	
SP2	2.6x3.8	+ 1.45	
SP3	2.6x3.8	+ 1.76	
SP4	2.6x3.7	+ 1.53	
SP5	2.6x3.7	+ 1.82	
SP6	2.6x3.7	+ 1.87	
SP7	2.7x3.5	+ 1.44	
SP8	2.6x3.5	+ 1.51	
SP9	2.6x3.8	+ 1.49	
SP10	2.6x3.4	+ 1.45	
SP11	2.6x2.8	+ 1.39	
SP12	2.6x3.6	+ 2.37	
SP13	2.6x3.6	+ 2.67	
SP14	2.0x3.0	+ 2.03	
SP15	2.2x2.8	+ 1.90	
SP16	2.3x3.1	+ 2.30	
SP17	2.5x3.5	+ 2.35	
SP18	2.5x3.2	+ 1.94	
SP19	2.5x3.3	+ 1.85	
SP20	2.6x3.2	+ 2.16	
Landing Platform	7.0x2.9	+ 2.66	

Table 2-4 Size and Uppermost Level Record of Solid Mass Section, SP 1-20, Landing Platform

Typical Photo Records



Figure 2-37 Typical Photo Records of excavation site



BF Plate 26 L3, Grid 4. Blue and White and Enamel Colours Porcelain Cup Sherds.



BF Plate 27 L3, Grid 4. Opium Storage Ware.



BF Plate 20 L3, Grid 4. Glass medicine bottle .

Figure 2-38 Typical Artefacts photo records

Archaeological Significance

Based on the historical and archaeological background, seven remnant items were classified into high and low significance levels. Four high significance Items will be discussed following.

Remnant Items	Significance Levels	Current Condition
1) Pavilion for Greeting Officials and solid mass section	High	The pavilion has been demolished except for its wall foundation stones and footing slabs.
2) SP12 and SP13 with original decking	High	The decking remains on SP12 and SP13 are the only surviving decking components of LTSB. A truncation is noted in the middle on each of the granite slabs of this part of the decking at roughly similar length.
3) Stone Pillars SP17	High	The SP17 is the only almost-complete excavated stone pillar in site.
4) Old Kowloon City Pier	High	The seawall should be retained in-situ without observable disturbance.
5) 1924 Kai Tak Bund Seawall	Low	The seawall should be retained in-situ without observable disturbance.
6) 1933 Causeway	Low	The seawall should be retained in-situ without observable disturbance.

Table 2-5 Significance Table

- 1) The Pavilion was a two-storey building constructed of bricks and stones with Chinese traditional roof system. It was 8 meters wide and 7 meters long.

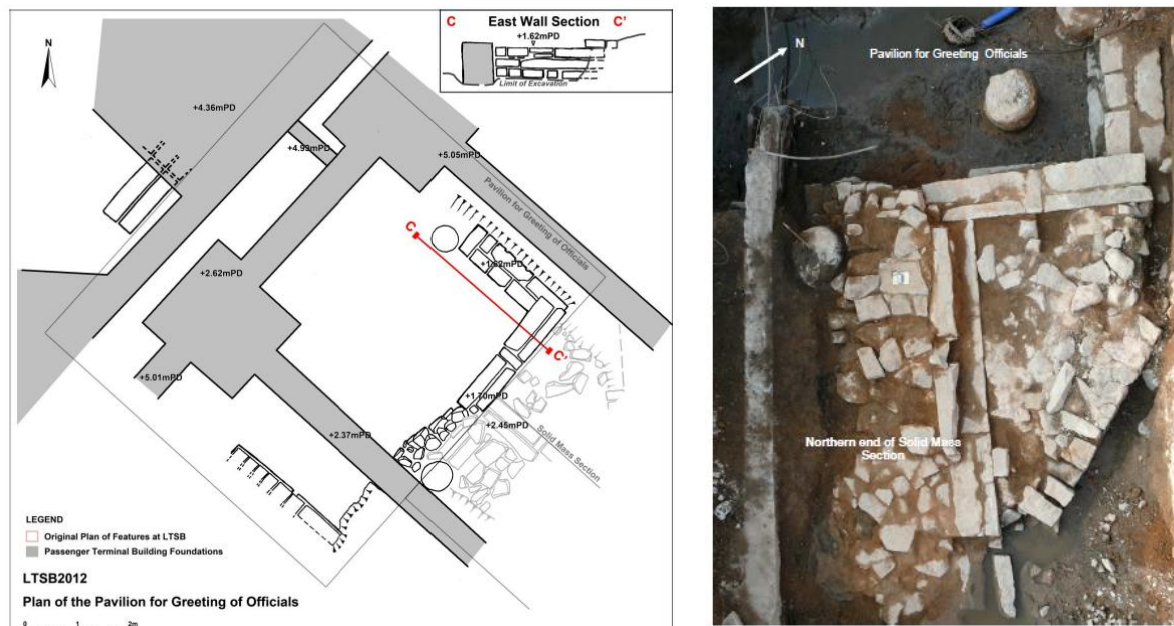


Figure 2-39 Plan of pavilion and detail photo

The solid mass section next to the Pavilion ruins was in the form of a solid mass with its walls constructed of granite blocks. Each span of its decking was composed of nine longitudinal granite slabs placed in parallel with the centerline of LTSB.



Figure 2-40 Solid Mass Section photo

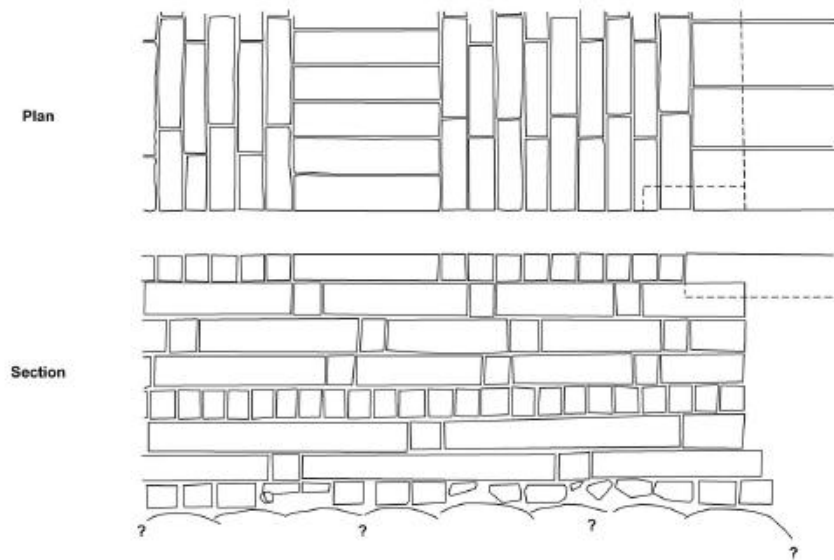


Figure 2-41 Reconstruction of Solid Mass

2) Decking on SP12 and SP13

The original decking remains on SP12 and SP13 formed the topmost part of the pillar. It laid in a direction parallel to the alignment of LTSB itself. The decking consists of five rows of rectangular granite slabs with a length of 6m. The two ends of the decking were embedded into the south eastern side of SP12 and north western side of SP13 to a depth of about 30cm.



Figure 2-42 Photos of Decking on SP12 and SP13

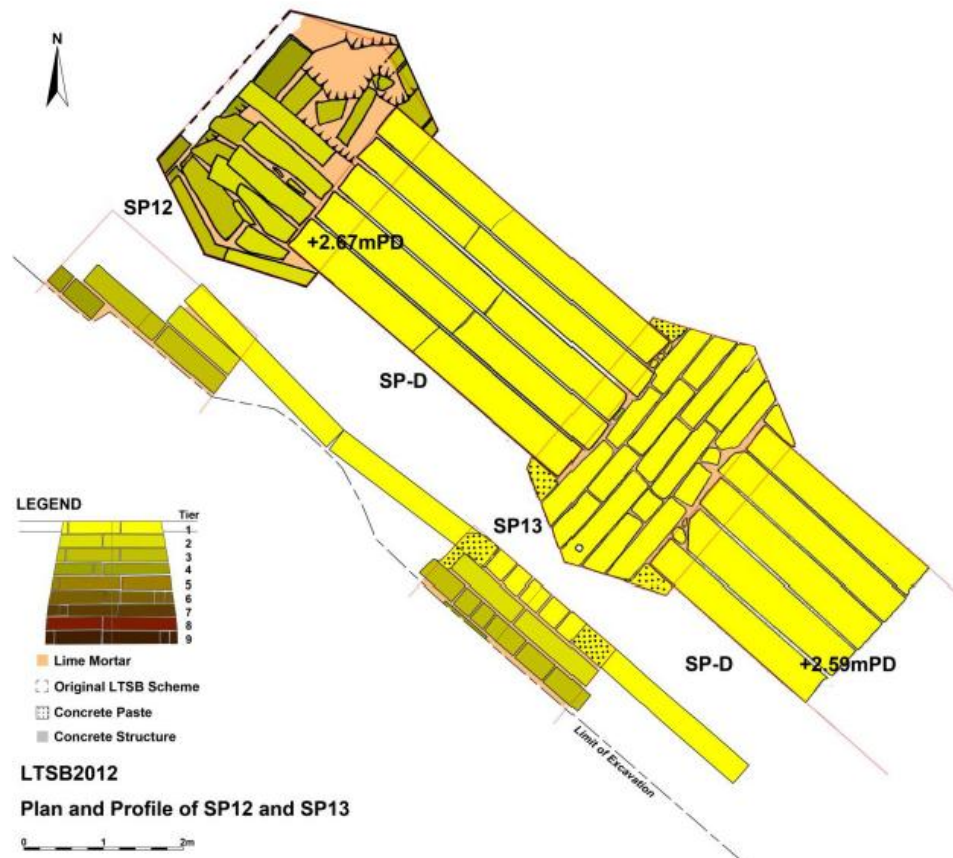


Figure 2-43 Profile of the decking

3) SP17

As the only almost-complete profile of stone pillar, SP17 consist of two parts of different construction methods. The top part consists of rectangular granite slabs that lays in alternating style, with reddish yellow lime paste acted as gluing agent of the slabs. The plan shape of the stone pillar is in regular skewed hexagon. The bottom part consists of boulders of over 50cm in size. These boulders acted as the foundation stones to support the stone pillar above. Comparison made to similar construction works of river bridge, it is believed that timber posts were possible vertically arranged under pillar footing slabs to support the pillar above.



Figure 2-44 SP17 Photos

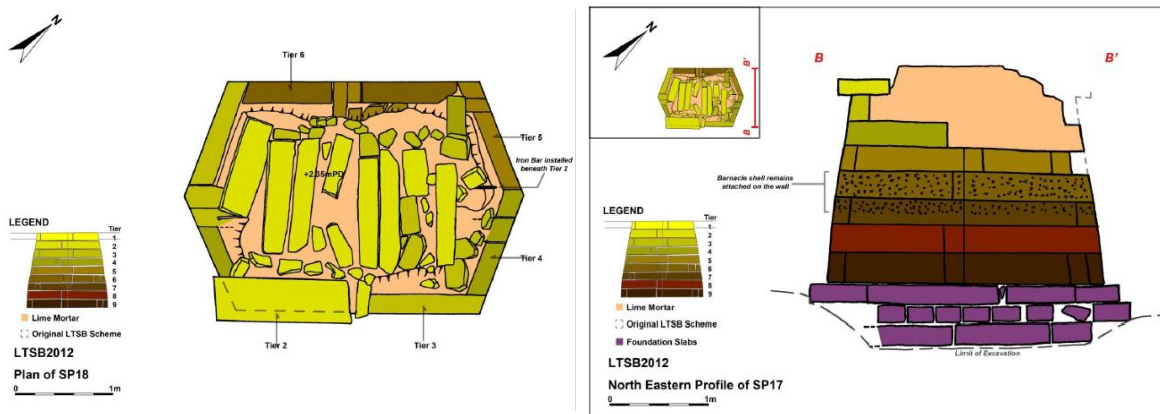


Figure 2-45 SP17 profiles

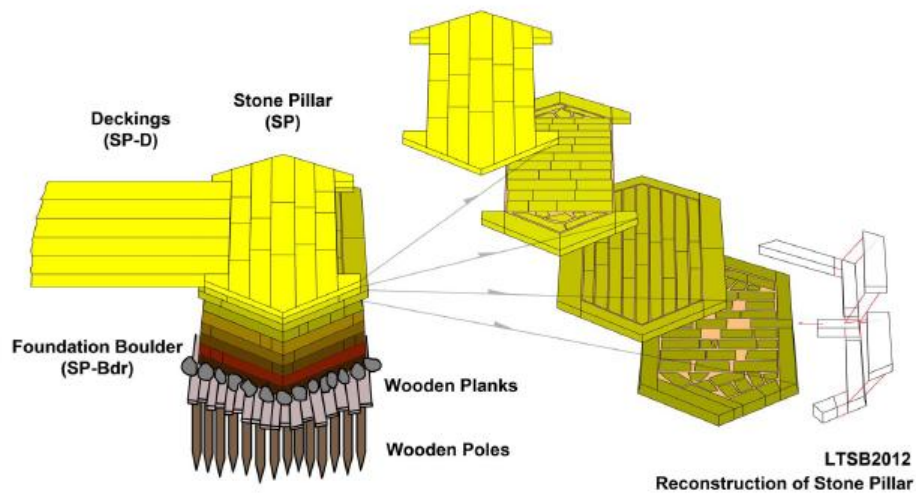


Figure 2-46 Reconstruction of Stone Pillar

4) Kowloon City Pier

In ca.1932 a new replacement concrete structure namely 'The Kowloon City Pier' (the Pier) with a wooden shelter was constructed.

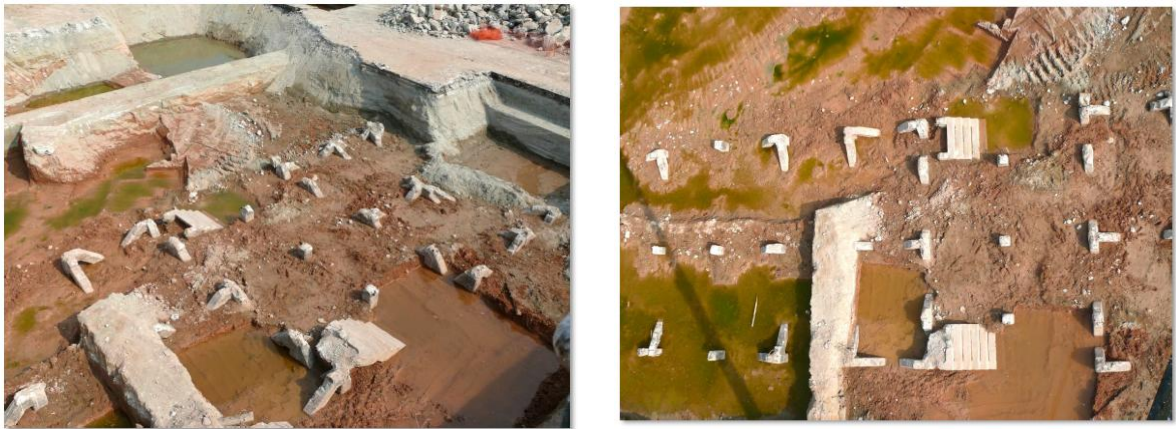


Figure 2-47 Photo of Kowloon City Pier

2.2.3 Conclusion

- Four Nodes

These four nodes with high significance can not only define the division inside the remnants site, but also provide a typically well-preserved showcase for better understanding on the bridge structure.

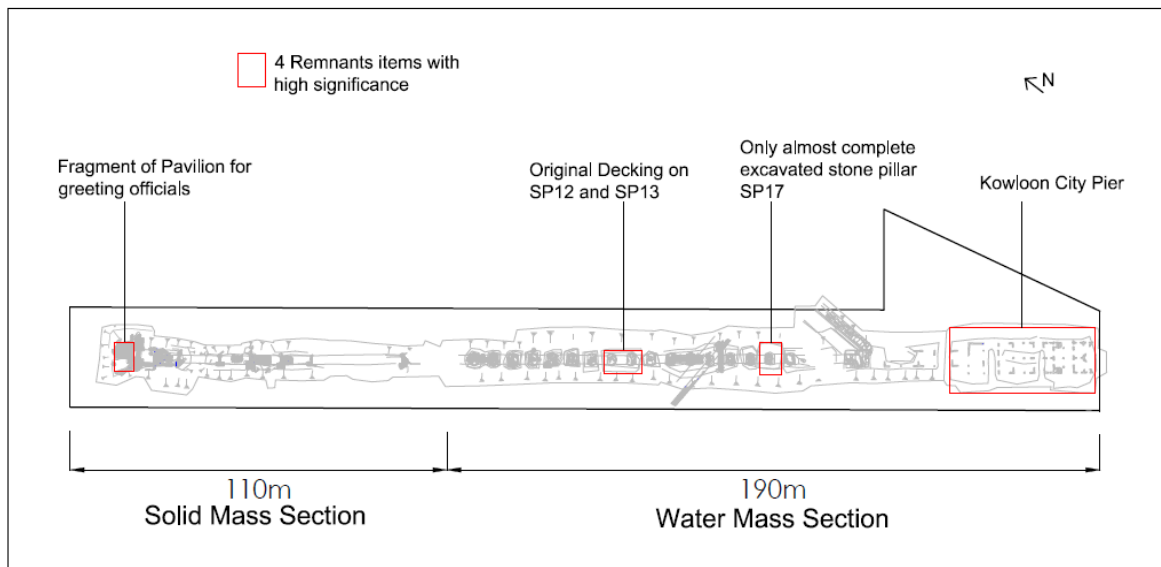


Figure 2-48 Four remnants items with high significance

Excavation Typology

The finding on excavation typology can help to generalize the remnants geometry or further design process.

In plan, the solid mass section and supporting pillars have the similar excavation area in plan. Moreover, each supporting pillar have similar excavation size, (approx . 2.6m in length x 3.7m in width).

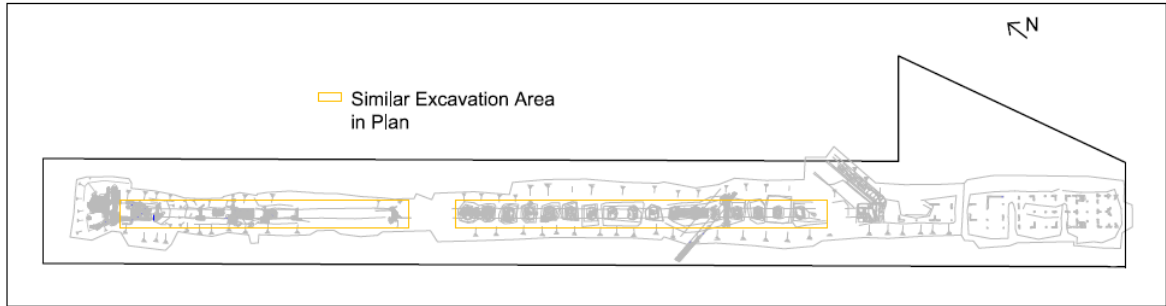


Figure 2-49 Typology of excavation site in plan

Figure 2-35 & 2-36 (Section record of test pit for Pavilion, Grid 1-4, supporting pillars and platform) shows the vertical typology of the current excavation site. Figure 2-50 below is a simple illustration that indicates the volumes of remnants items above the excavation limit. It shows that except SP12, 13 and 17, most of the items are exposed within similar height above the excavation level but different slopes. Decking on SP12 and SP13 locates 1.1m above the general limit, while SP17 has -0.8mPD bottom level and +2.39mPD top level, 3.2m in level difference.

Generally, the typology is considered as 'gradually low down from north end to south end' and 'steep at west while gentle at east'

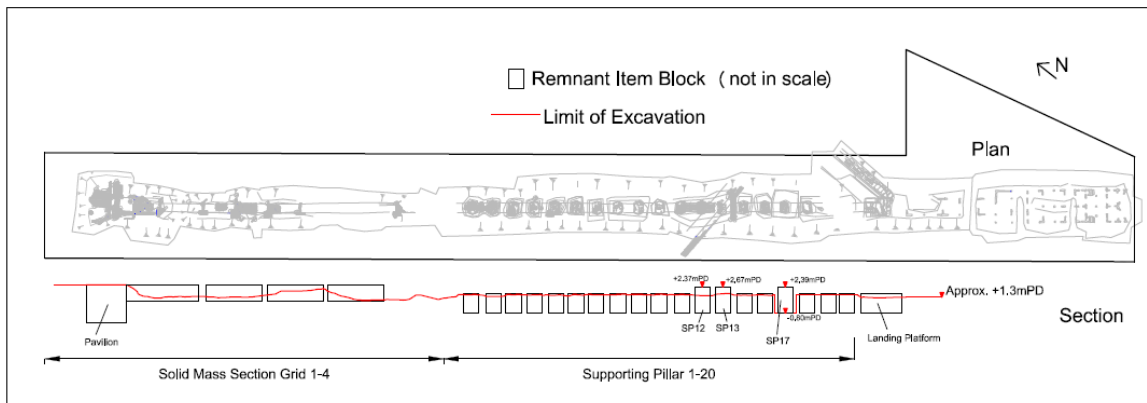


Figure 2-50 Typology of excavation site in Section

CHAPTER 3

URBAN ANALYSIS

3.0 Urban Analysis

3.1 Urban Analysis in Hong Kong

3.1.1 Traffic System

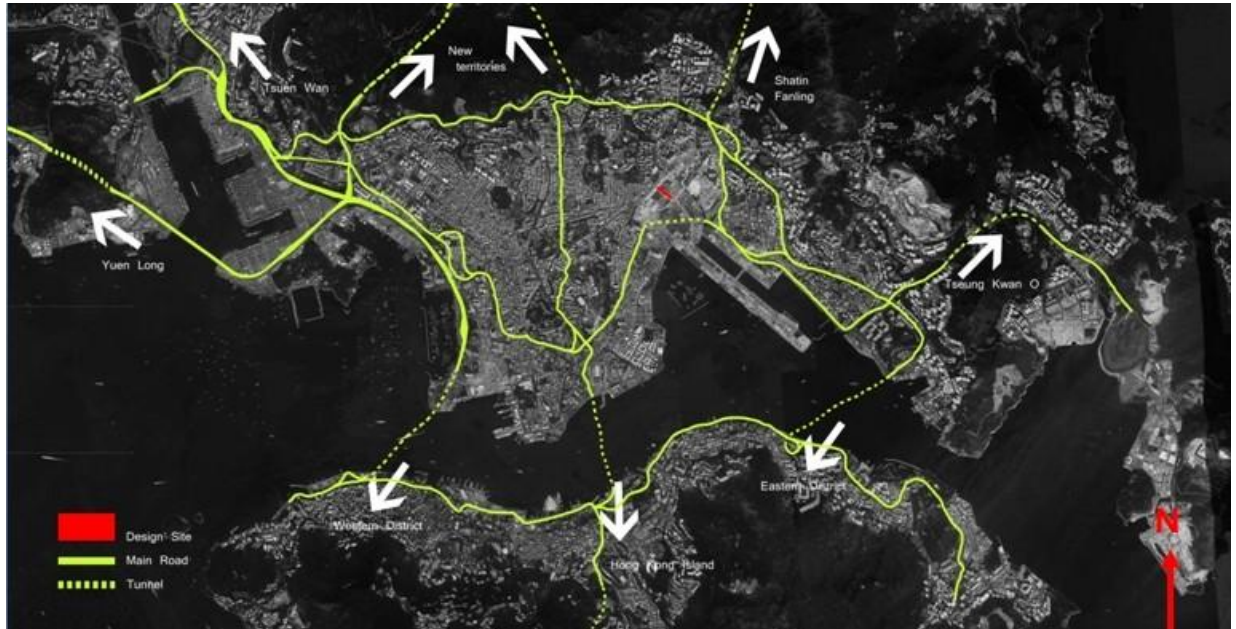


Figure 3-1 Main Road and Tunnel

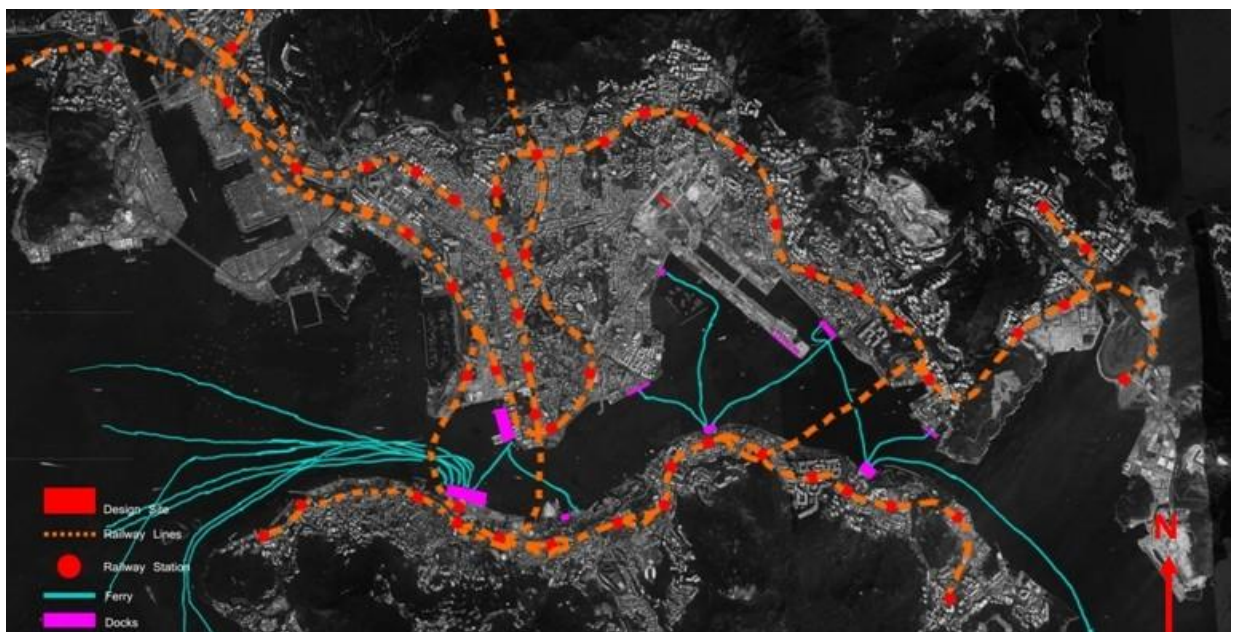


Figure 3-2 Railway and Ferry

Hong Kong has a highly developed and sophisticated transport network, including rail, road, maritime and air, etc. There are total 10 lines and 83 railway station n Mass Transit Railway (MTR). 27 ferry services are serving outlying islands, new towns and inner-Victoria Harbor.

3.1.2 Urban Texture



Figure 3-3 City Blocks

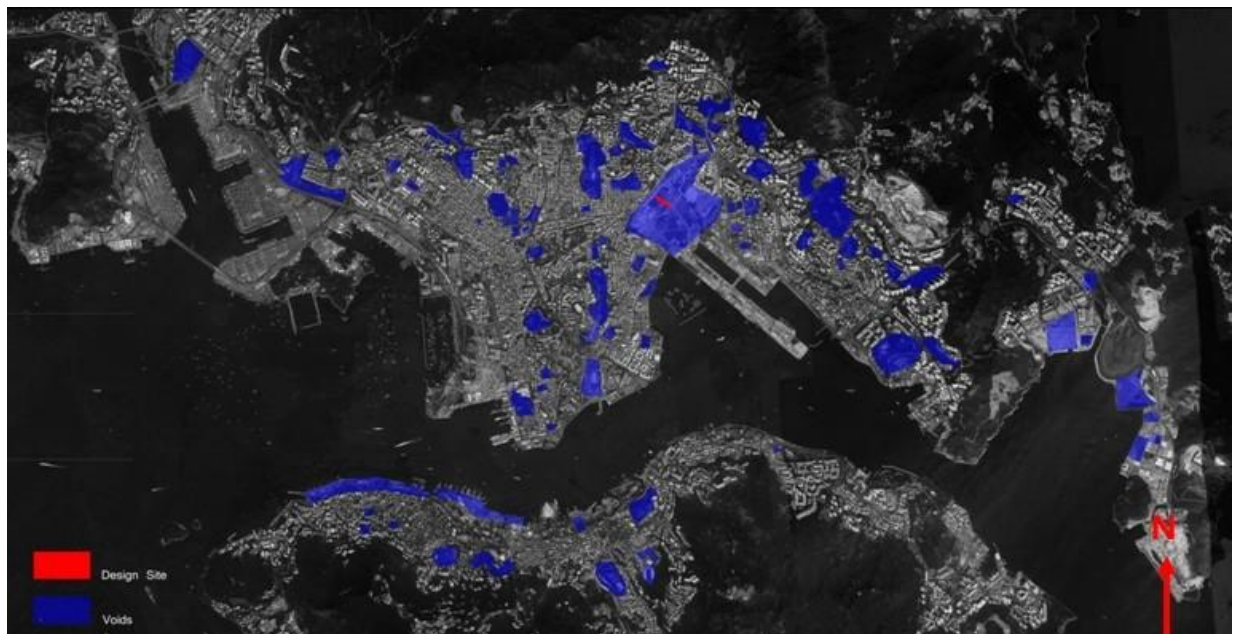


Figure 3-4 City Void

Hong Kong is one of the world cities which have the highest dense urban texture. The human development index has reached to 0.898 at 2011. The research shows that though the land area is limited, construction work is still carried out in every single unused space (void) apart from green space required.

3.1.3 Land Use

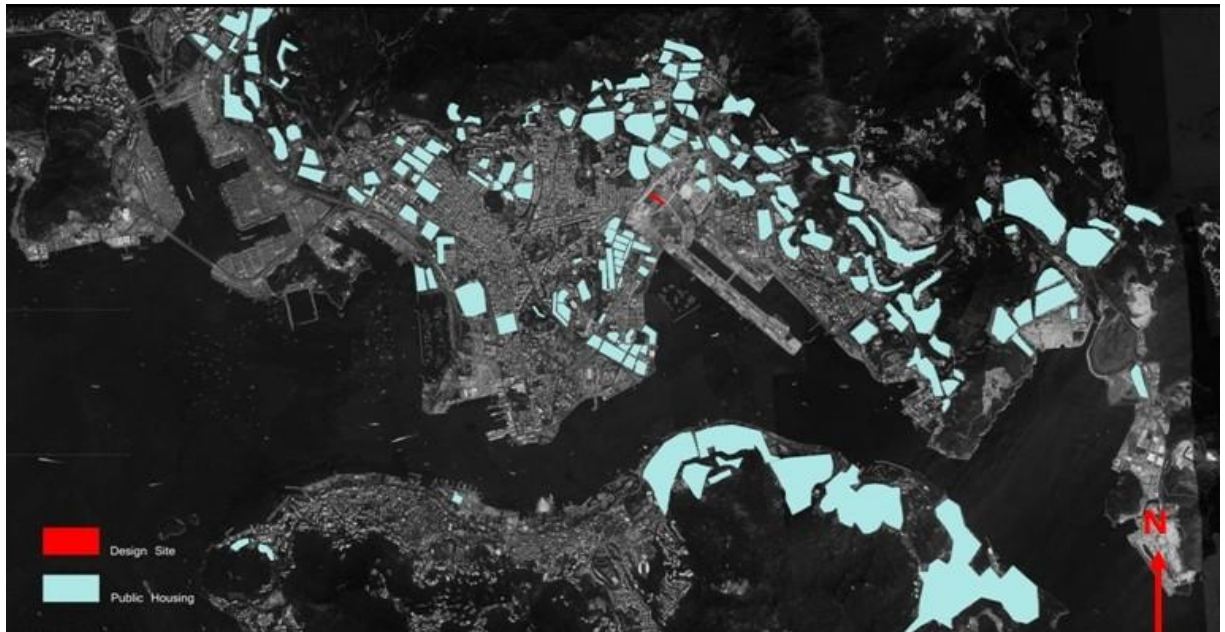


Figure 3-5 Public Housing

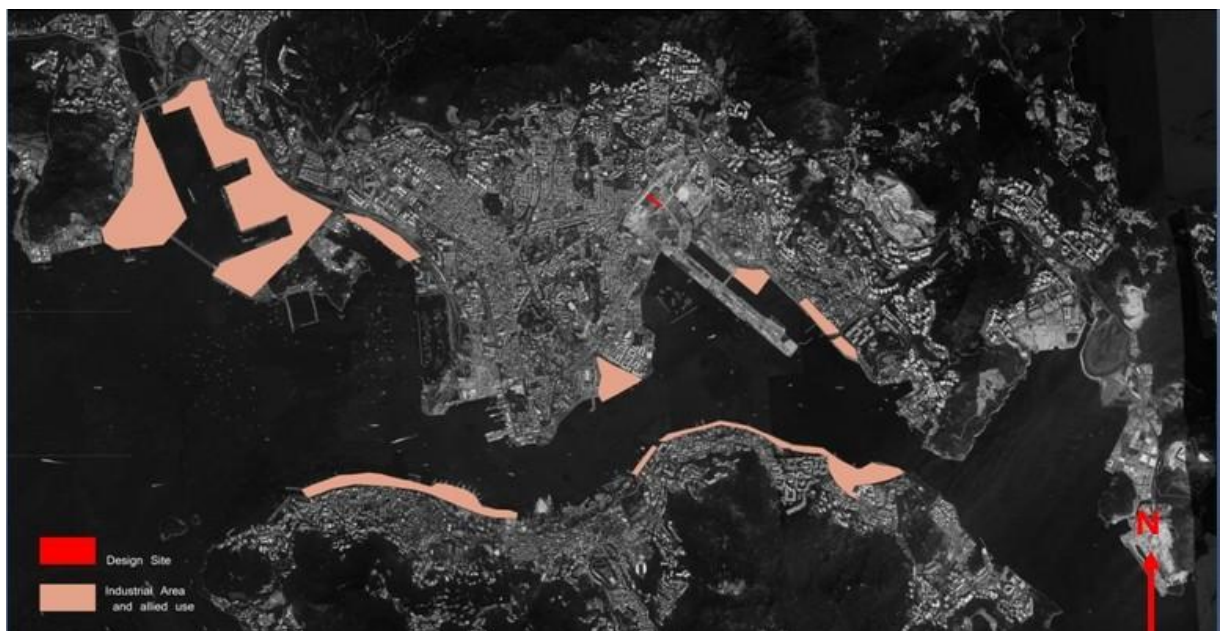


Figure 3-6 Industrial Area

Public housing in Hong Kong is a set of mass housing programmes through which the Government of Hong Kong provides affordable housing for lower-income residents. It is a major component of housing in Hong Kong, with nearly half of the population now residing in some form of public housing. To provide readily available premises to meet Hong Kong's changing economic and social needs, the Government announced in October 2009 an optimisation scheme with a package of measures to optimise the use of existing industrial buildings through encouraging redevelopment of industrial buildings situated in non-industrial zones and conversion of entire existing industrial buildings.

3.2 Urban Analysis near Site



Figure 3-7 Green Space and Water Element near site

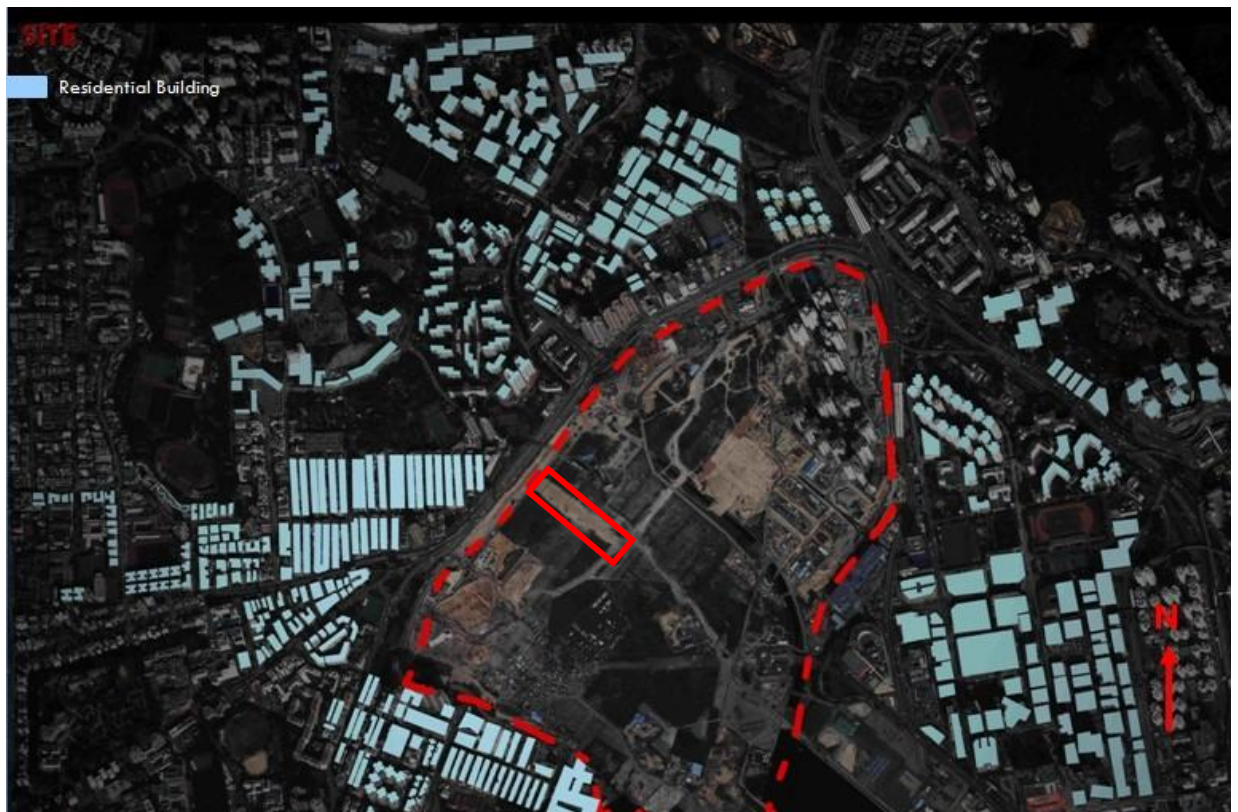


Figure 3-8 Residential Building near site

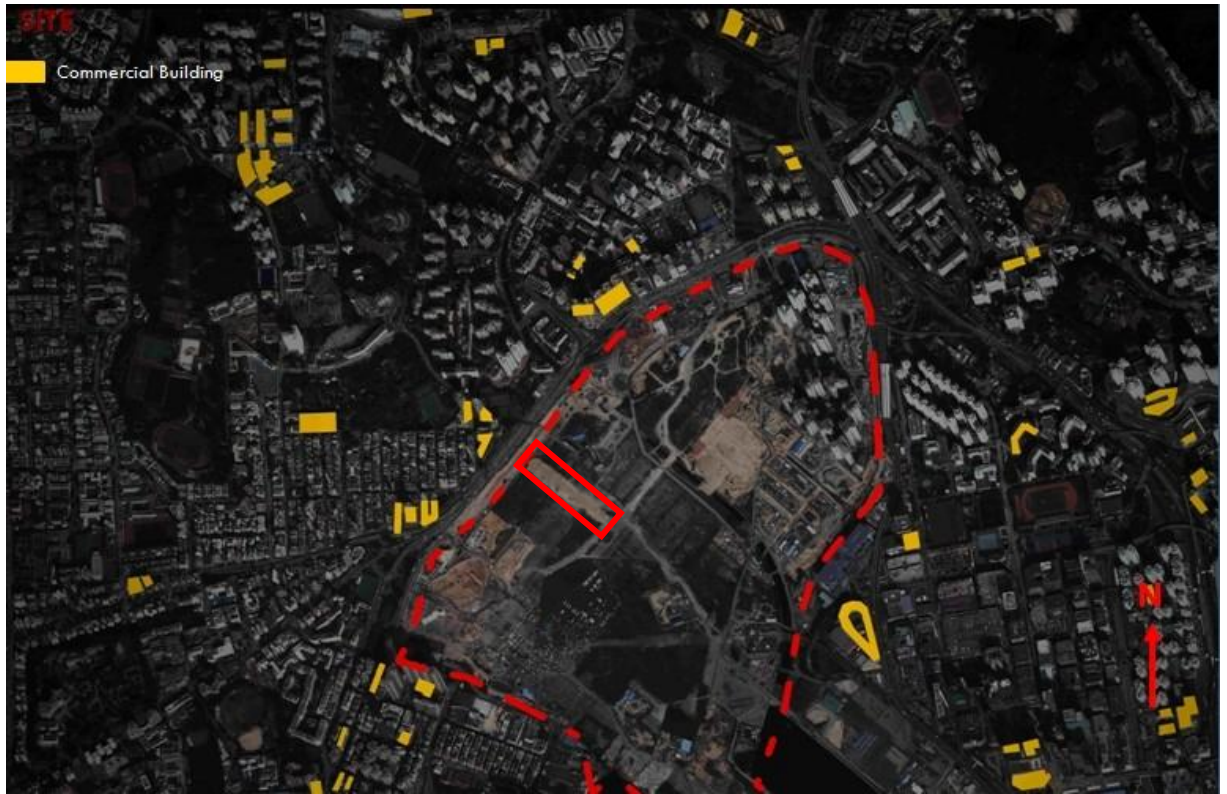


Figure 3-9 Commercial Building near site

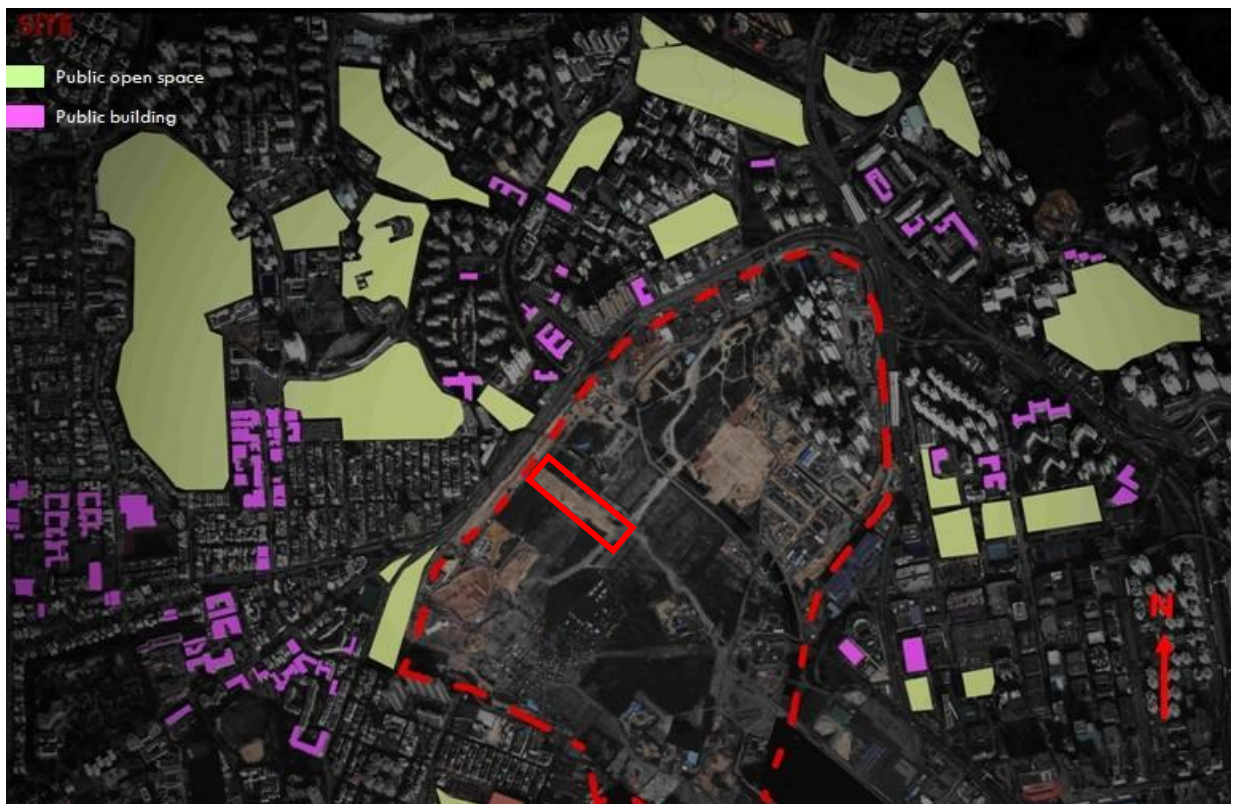


Figure 3-10 open space and Public Building near site

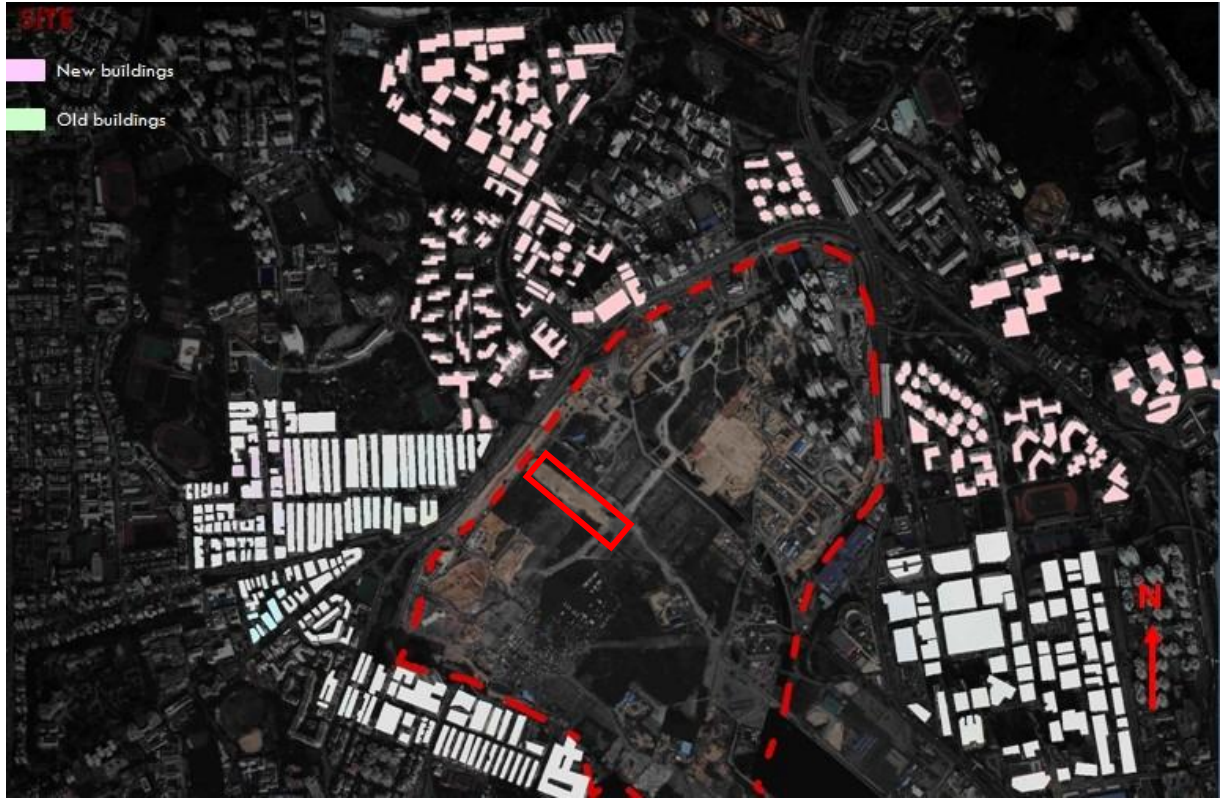


Figure 3-11 New Building and Old Building near site

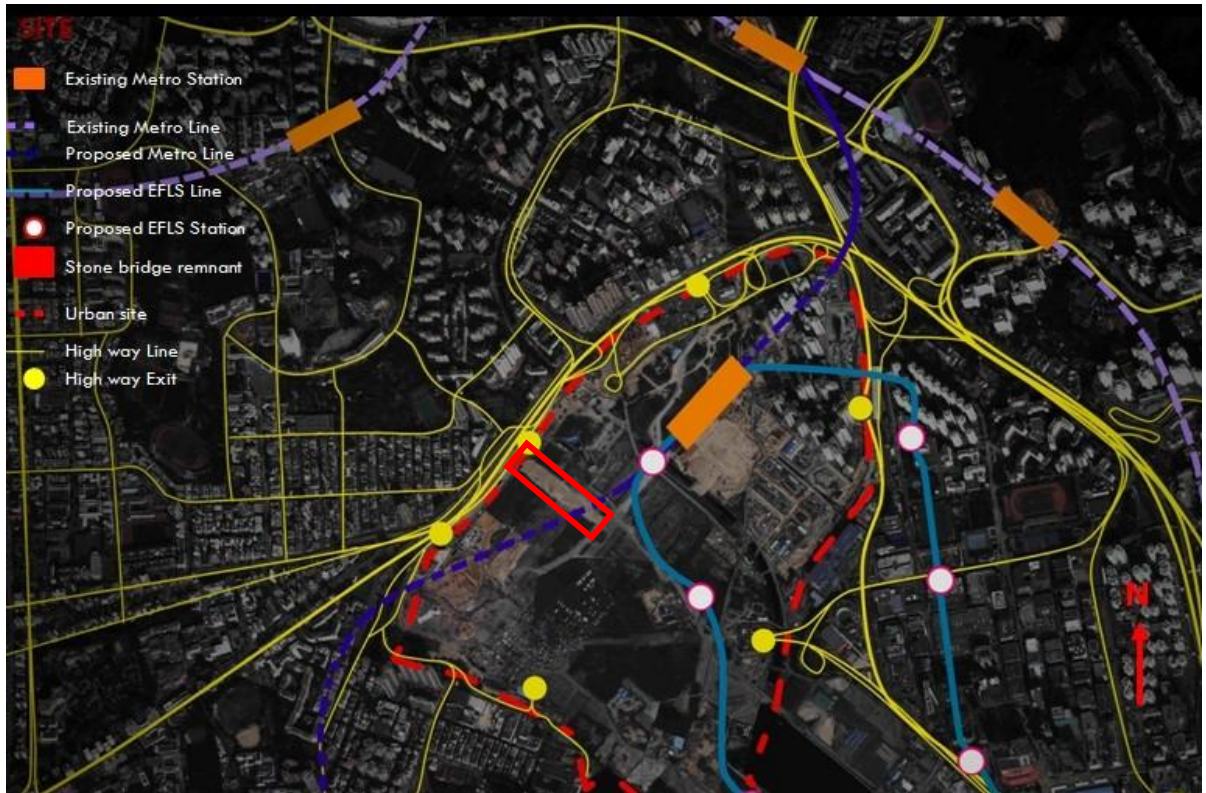


Figure 3-12 Transport system near site.

Six railway stations locate in 10 minutes of driving distance to the site. (Stations include: Kowloon Tong, Lok Fu, Wong Tai Sin, Diamond Hill, Ngau Chi Wan and Kowloon Bay) No direct railway line can reach the site in a walking distance less than 10 minutes. However, a new Shatin-Central line has been proposed and now under construction. Kai Tak Station will be built inside Kai Tak Area locating 200m away from the site. This station will also serve for Environmental-friendly linkage system, a inner-Kai Tak elevated rail system. 6 highway exits located around the site with 5 EFLS stops.

3.3 SWOT Analysis

- **Strength:**

- 1) Good location connecting old Kowloon City and New Kai Tak Area, and also connecting Kowloon Island and Hong Kong Island through piers
- 2) Comprehensive transport system in the neighborhood, including tram, underground metro and developing light linkage system
- 3) Significant historical and cultural value upon the archaeological findings
- 4) Great public attention and participation on the Kai Tak redevelopment project
- 5) Existing heritage sites are in 20 minutes walking distance. (Kowloon Walled City Park, Song Wong Toi Heritage Park)
- 6) Have a good landscape view to Lion Rock, and close to waterfront promenade.

- **Weakness:**

- 1) Urban environment is not homogenous or well-organized due to the old Kowloon city
- 2) Poor pedestrian connection between old Kowloon City and the site
- 3) Lack of cultural node in the new complex hub
- 4) Lack of hot tourism spots in the neighborhood to attract visitors

- **Opportunities:**

- 1) Integration into the comprehensive plan for Kai Tak redevelopment which is to be the new heritage, culture, leisure and tourism hub in Kowloon Island
- 2) Good opportunity to illustrate the history of Lung Tsun Stone Bridge, colony's territory extension and Kai Tak airport development, for educational and tourism purposes, therefore to build a new historical nodes in Kowloon island

- **Threats:**

- 1) Extreme weather (typhoon) may bring bad effect to the square area.
- 2) The construction work to the surrounding developing area may cause damage onto the remnants structure.

3.4 Goals, Objectives & Actions

Regarding the goals set from competition brief, further objectives and actions are set coordinately before design process.

Goals	Objectives	Actions
To create suitable space, ambience and environment to facilitate in-situ preservation and appreciation of the Bridge remnants	Plan an in-situ garden to have a place of relaxation and decompression from the intense commercial district while have the ruins' cellar floor untouched for observation	<ol style="list-style-type: none"> 1) Focus on in-situ interpretation of four nodes with high significance 2) Transform the particular typology of the excavation site into a suitable garden typology 3) Transform the style of Chinese garden into a modern garden 4) Create a linkage of the neighborhood transportation through the garden pathway.
To facilitate exhibition and interpretation of the Bridge remnants, the integration of the Preservation Corridor with the surrounding development and open space network, and the pedestrian connectivity.	Plan a museum to create a new culture node in the area for meeting, education, visiting, transportation and most importantly the interpretation of Bridge remnants.	<ol style="list-style-type: none"> 1) Provide exhibition space for interpretation of the bridge, including its history and related architecture etc. 2) Provide public space for educational and leisure purpose. 3) Provide a research office for further study on LSTB or related archaeological subject.

Table 3-1 Goals, Objectives and Actions

3.5 Feng Shui 风水

Before getting start of garden and architecture design, additional to urban analysis, Feng Shui (metaphysics) is considered in concept design. It is traditional Chinese philosophical system of harmonizing everyone with surrounding environment. The term 'Feng Shui' literally translates 'Wind-Water' in English.

3.5.1 Fengshui for Gardens

Taoism had a strong influence on the classical garden. After the Han Dynasty (206 BCE – 220 CE), gardens were frequently constructed as retreats for government officials who had lost their posts or who wanted to escape the pressures and corruption of court life in the capital. They chose to pursue the Taoist ideals of disengagement from worldly concerns. For followers of Taoism, enlightenment could be reached by contemplation of the unity of creation, in which order and harmony are inherent to the natural world. The gardens were intended to evoke the idyllic feeling of wandering through a natural landscape, to feel closer to the ancient way of life, and to appreciate the harmony between man and nature.

In Taoism, rocks and water were opposites, Yin and Yang, but they complemented and completed one another. Rocks were solid but water could wear away rock.

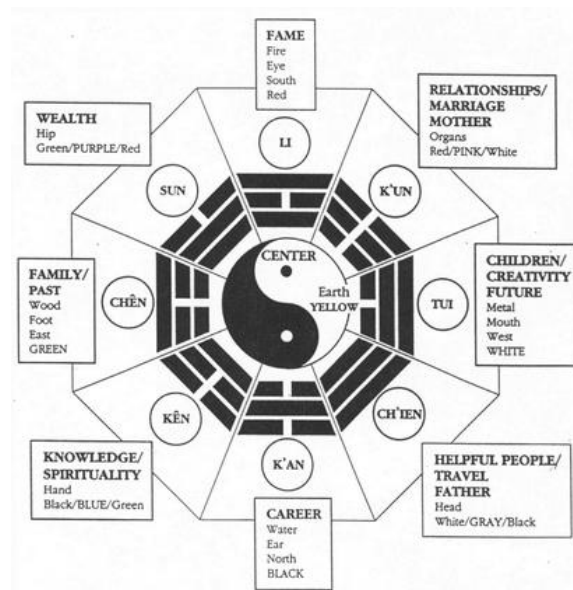


Figure 3-13 Taoism symbol

The winding paths and zig-zag galleries bridges that led visitors from one garden scene to another also had a message. They illustrated a Chinese proverb, "By detours, access to secrets".

According to the landscape historian and architect Che Bing Chiu, every garden was "a quest for paradise of a lost world, of a utopian universe. The scholar's garden participated in this quest; on the one hand the quest for the home of the Immortals, on the other hand the search for the world of the golden age so dear to the heart of the scholar."



Figure 3-14 Bird eye's view to a typical Chinese Garden

3.5.2 Feng Shui for Architecture

Fengshui practice discusses architecture in metaphoric terms of 'invisible force' that bind the universe, earth and humanity together, as 'Qi' 气. There are three principles, including the unity of human beings with nature, the balance of Yin and Yang, and the attraction and repulsion of five elements – metal, wood, water, fire and earth. These principles are set up to help people pursue good fortune and avoid disaster, thus improving people's living standard.



Figure 3-15 Hong Cun Village, Huang Shan(left), Forbidden City, Beijing (right)

According to Feng Shui, the proper positioning of the central axis is important for buildings. And the central axis should run from north to south. A node must locate at traffic intersection to integrate the Qi of the neighborhood to boost the development. It shall also have its entrance orienting towards South with its body lays at North. The main volume shall always have a backyard to collect its own Qi. And the volume shall also follow the rules of harmony. To the main building volume, a void shall be left blank to have a balance.

3.6 Location of Garden and Museum

Yin and Yang can be understood as contrast of solid and void as shown in Figure 3-16 below. Regarding the special geology of the site, the long and narrow remnant site is a void space. A solid museum volume shall be introduced for a need of contrast.

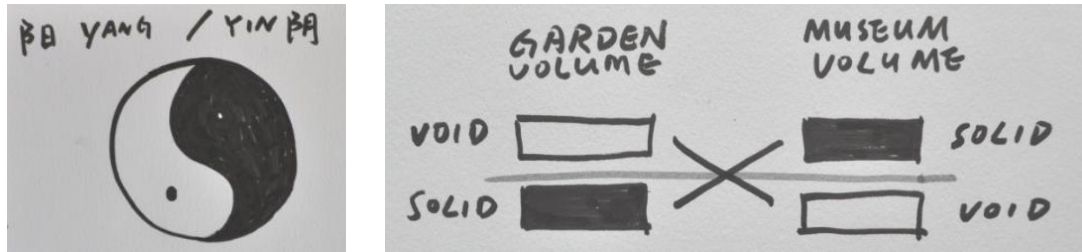


Figure 3-16 Yin and Yang Contrast

Museum plays a role of main building with garden as its courtyard. The two work together to collect Qi. The central axis should be carefully positioned to have a best orientation shown in Figure 3-17 below. Shaping of garden and museum will be explained more in next two chapters.

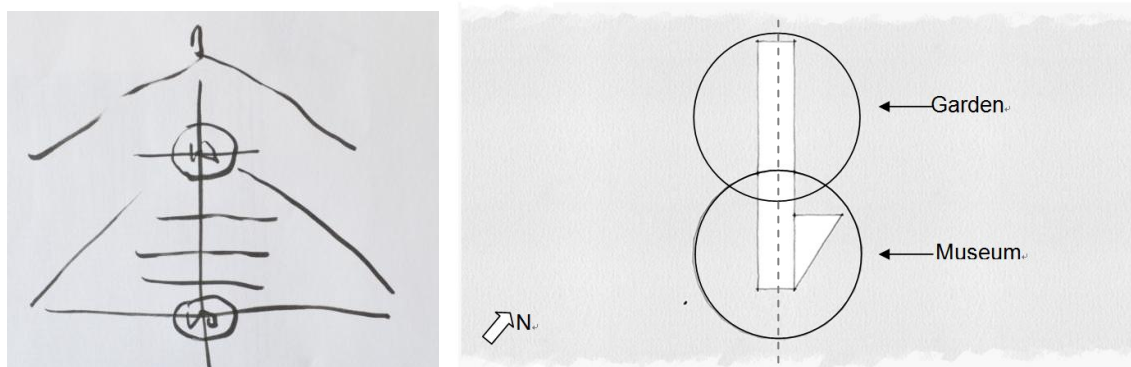


Figure 3-17 Positioning garden and museum

CHAPTER 4
GARDEN DESIGN

4.0 Garden Design

To build a garden in Hong Kong, the idea of Chinese gardens first comes up to our mind. The style dates back to Han Dynasty (206BC) and is a best place for visitors to achieve inner peace while enjoying moving in its creative landscape.

Therefore a background research on Chinese gardens was first conducted to have a better understanding to the garden design and philosophy in China, as well as to create a concept of adopting traditional style into a modern outlook under same philosophy.

4.1 Study of Chinese Gardens

4.1.1 Design of Classical Chinese Garden

The Chinese garden is a landscape garden style which has evolved over three thousand years. It includes both the vast gardens of the Chinese emperors and members of the Imperial Family, built for pleasure and to impress, and the more intimate gardens created by scholars, poets, former government officials, soldiers and merchants, made for reflection and escape from the outside world. They create an idealized miniature landscape, which is meant to express the harmony that should exist between man and nature.

A typical Chinese garden is enclosed by walls and includes one or more ponds, rock works, trees and flowers, and an assortment of halls and pavilions within the garden, connected by winding paths and zig-zag galleries. By moving from structure to structure, visitors can view a series of carefully composed scenes, unrolling like a scroll of landscape paintings.¹

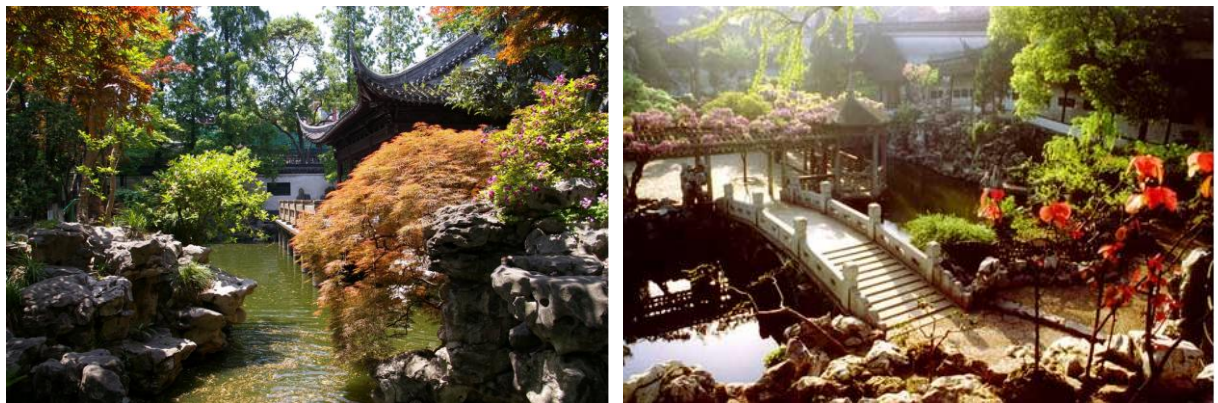


Figure 4-1 Yuyuan Garden 豫园 in Shanghai, 1559(left),
The Lion Forest 狮子林 in Suzhou, 1342 (right)



Figure 4-2 Humble Administrator's Garden 拙政园 in Suzhou, 1506-1521(left),
The Lingering Garden 留园 in Suzhou, 1593(right)

A Chinese garden was not meant to be seen all at once; the plan of a classical Chinese garden presented the visitor with a series of perfectly composed and framed glimpses of scenery; a view of a pond, or of a rock, or a grove of bamboo, a blossoming tree, or a view of a distant mountain peak or a pagoda. The 16th century Chinese writer and philosopher Ji Cheng instructed garden builders to 'hide the vulgar and the common as far as the eye can see, and include the excellent and the splendid'. The classical garden was surrounded by a wall, usually painted white, which served as a pure backdrop for the flowers and trees. A pond of water was usually located in the center. Many structures, large and small, were arranged around the pond. In the garden described by Ji Cheng above, the structures occupied two-thirds of the hectare, while the garden itself occupied the other third. In a scholar garden the central building was usually a library or study, connected by galleries with other pavilions which served as observation points of the garden features. These structures also helped divide the garden into individual scenes or landscapes. The other essential elements of a scholar garden were plants, trees, and rocks, all carefully composed into small perfect landscapes. Scholar gardens also often used what was called "borrowed" scenery (借景 *jiējǐng*) ; where unexpected views of scenery outside the garden, such as mountain peaks, seemed to be an extension of the garden itself.



Figure 4-3 Painted map of the Master of Nets Garden begun in 1140, renovated in 1736-1796

4.1.2 Garden Architecture

Chinese gardens are filled with architecture: halls, pavilions, temples, galleries, bridges, kiosks, and towers, occupying a large part of the space. The Humble Administrator's Garden in Suzhou has forty-eight structures, including a residence, several halls for family gatherings and entertainment, eighteen pavilions for viewing different features of the garden, and an assortment of towers, galleries, and bridges, all designed for seeing different parts of the gardens from different point of view.



The garden is filled with small pavilions, (ting 亭) which are designed for providing shelter from the sun or rain, for contemplating a scene, reciting a poem, taking advantage of a breeze, or simply resting. Pavilions are sometimes attached to the wall of another building or sometimes stood by themselves at view points of the garden, by a pond or at the top of a hill.

Figure 4-4 Pavilion for viewing the rock garden at the Prince Gong Mansion in Beijing, 1777



The garden structures are not designed to dominate the landscape, but to be in harmony with it. Gardens also often feature two-story towers (lou 楼), usually at the edge of the garden, with a lower story made of stone and a whitewashed upper story, two-thirds the height of the ground floor, which provided a view from above of certain parts of the garden or the distant scenery.

Figure 4-5 Jianshan Tower for viewing the garden at Humble Administrator's Garden in Suzhou, 1506-1521



Galleries (lang 廊) are narrow covered corridors which connect the buildings, protect the visitors from the rain and sun, and also help divide the garden into different sections. These galleries rarely straight, they zigzag or are serpentine, following the wall of the garden, the edge of the pond, or climbing the hill of the rock garden. They have small windows, sometimes round or in off geometric shapes, to give glimpses of the garden or scenery to those passing through.

Figure 4-6 Gallery at Humble Administrator's Garden in Suzhou, 1506-1521



Bridges (qiao 桥) are another common feature of the Chinese garden. Like the galleries, they are rarely straight, but zigzag or arch over the ponds, suggesting the bridge of rural China, and providing view points of the garden. Bridges are often built from rough timber or stone-slab raised pathways.

Figure 4-7 Zig-zag bridge in the Humble Administrator's Garden illustrates the proverb, 'by detours, access to secrets'



The artificial mountain (jiashan) or rock garden is an integral element of Chinese classical gardens. The mountain peak was a symbol of virtue, stability and endurance in the philosophy, of Confucius and in the I Ching. A mountain peak on an island was also a central part of the legend of the Isles of the Immortals, and thus became a central element in many classical gardens.

Figure 4-8 A scholar rock from Lake Taihu in Beijing Botanical Garden

4.2 Concept

4.2.1 Garden Access



Figure 4-9 Sketch for Garden Access

As shown in the figure 4-9, within the outline zone of Kai Tak Development (KTD), the major axis is orthogonal to the bridge site. In addition, it is important to exploit the visual setting of metro station and entrance piazza and accentuate the orientation of urban axis.

Therefore four accesses are designed, as shown in figure 4.10.

Accesses 1 and 2 are designed at the location of the pavilion remnants and Kowloon City Pier remnants which sit namely at North and South end of the site. They direct the linkage between old Kowloon City and Kai Tak Development area.

Access 3 is to connect the bridge site and entrance piazza, further linking to metro station.

Access 4 is to have an easy accessibility between the complex buildings at two sides of the bridge site along existing axis.

4.2.2 Garden Texture

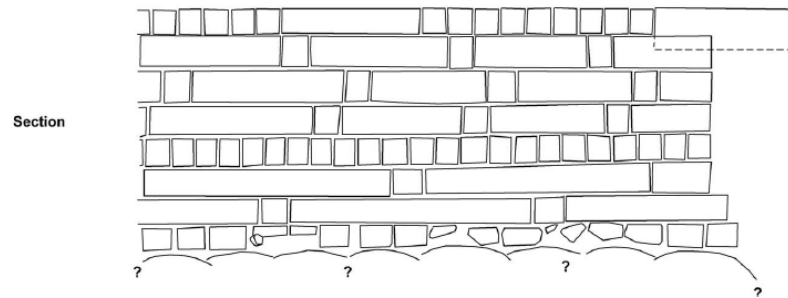


Figure 4-10 Construction Method of Alternating cross-straight piling

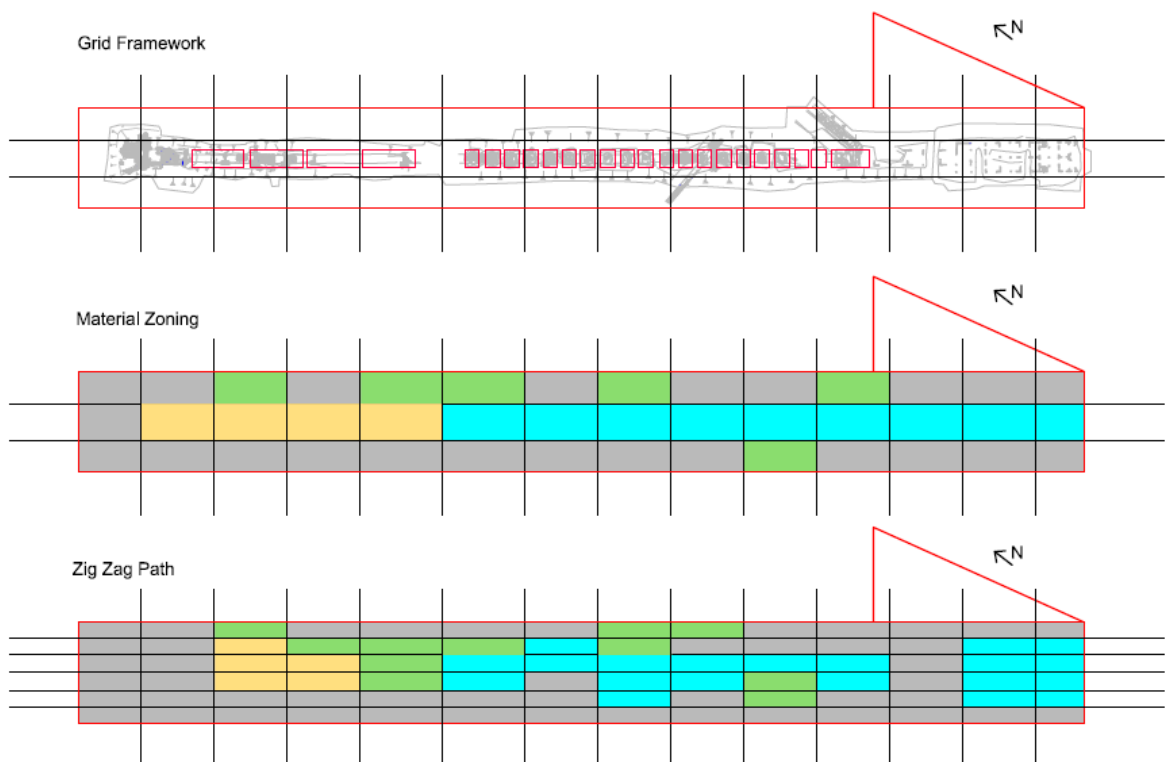


Figure 4-11 Grid Transformation

The solid mass items and supporting pillars were simulated as identical blocks due to similar size. Inspired by the construction method of Alternating cross-straight piling, the bridge site is divided into grids net. To separate Solid Mass section and Water Mass section, different garden textures are filled up in central grids while surrounded by additional textures in different height.

4.2.3 Garden Materials

To create different visiting experiences, four patterns of pathway are chosen, namely pebble pavement, stone pavement, lawn and water.



Figure 4-12 Pebble Pavement

Pebble is only used in solid mass section to decorate the broken pieces in solid mass section.



Figure 4-13 Stone Pavement

Stone pavement is the pavement of main walk path. Rectangular stone is preferred to match the textures of stone pillar remnants so to create a harmonious sense in geometry design.



Figure 4-14 lawn

Lawn is selected to be the buffer material between different walking planes. Lawn slope can gently lead visitor into different areas.



Figure 4-15 Water

Water will only appear in water mass section. It can also reflect the surrounding landscape and architecture.

4.2.4 Garden Typology

As suggested in the conservation plan, remnant items shall stay untouched without observable disturbance. The existence of remnant appears like the 'artificial rocks' in traditional Chinese gardens.

Thanks to the long span of the site, it is necessary to create changing viewing perspectives vertically. Based on the analysis in Chapter 2, the excavations site has shown different degrees of exposure above cellar ground level which is 'gradually low down from north end to south end' and 'steep at west while gentle at east'. The garden typology is designed to suit the current site typology. Garden elements are designed when needed to fit the typology.

Please refer to the Figure in the following page for Garden Typology Map.

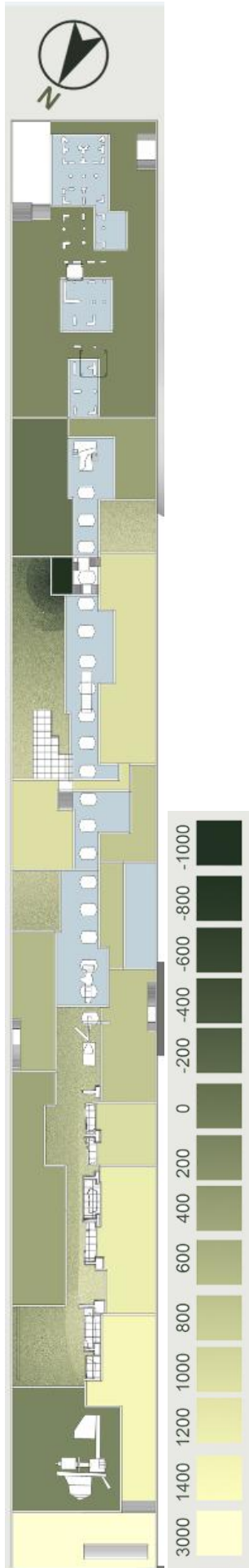


Figure 4-16 Garden Typology Map

4.2.5 Garden Elements.

4.2.5.1 Street Level Bridge

The garden is first divided into Solid Mass Section and Water Mass Section by a street level bridge. It is also to create easy accessibility between complex buildings at site's sides.

4.2.5.2 North Start

The North Start at Solid Mass Section originally locates pavilion for greeting officials which has been mostly destructed by construction works of former Kai Tak Airport Terminal Buildings. The remaining foundation pieces are left untouched with a wavy staircase links ground floor to cellar floor as North Start, which bridges a vertical distance of 3.6m.

4.2.5.3 Solid Mass Section

Besides pavilion remnants, the solid mass section contains two other major features, a) relatively large portion of reclamation fill soil and concrete surface for Kai Tak Bund development, b) pieces of tiers of granite blocks of bridge. The tier fragments are preserved with glass cover on top while reclamation concrete surface stays exposed. To have a better observation to the ruins, the east side of the ruins was laid with pebble pavement while the west side was laid with stone pavement. It can not only create a visual experience on due to height difference, but also allow visitors to have closer observation to the tier pieces of granite bridge blocks. Lawn serves as slope to lead people into next area (Water Mass Section).

4.2.5.4 Water Mass Section

The excavation of 20 supporting pillars is designed to stay exposed above water pond. A stone bridge is designed for cross above water mass section it buffers both sides' level difference as well as a small view deck towards 'decking on SP12 and SP13'. Moreover, Two staircases are built aside SP17 from stone pavement level (+1m) into SP17 bottom level (-1m). A shaded pocket space with plants and seating areas are provided for rest.

4.2.5.5 Bridge Birdge & Pavilion

An 80m zig-zag bridge is designed to link garden and museum volume at vertical distance of 7.8m. It not only transits indoor construction into outdoor landscape, but also convergent from narrow visual scale into wider. On the other hand, the horizontal zig-zag and the vertical ramp element can help visitors have changing perspective to the garden. In height of +3m, the ramp shares same plane with the cover of pavilion (10mx 12m). This pavilion works as a rain shelter with resting seated provided for visitors. The bottom of the ramp is connected to the lawn area.

4.2.5.6 SP17 stairs

Due to the special typology around SP17, two staircases are designed around it for better observation.

4.2.5.7 South Start

At south end of the site, the remnants of Kowloon City Pier are mostly beneath museum volume and preserved with water surrounding in same surface. There are two big staircases along the end wall to guide visitors into garden from museum volume and street level, as South Start of the garden tour.

Garden Elements

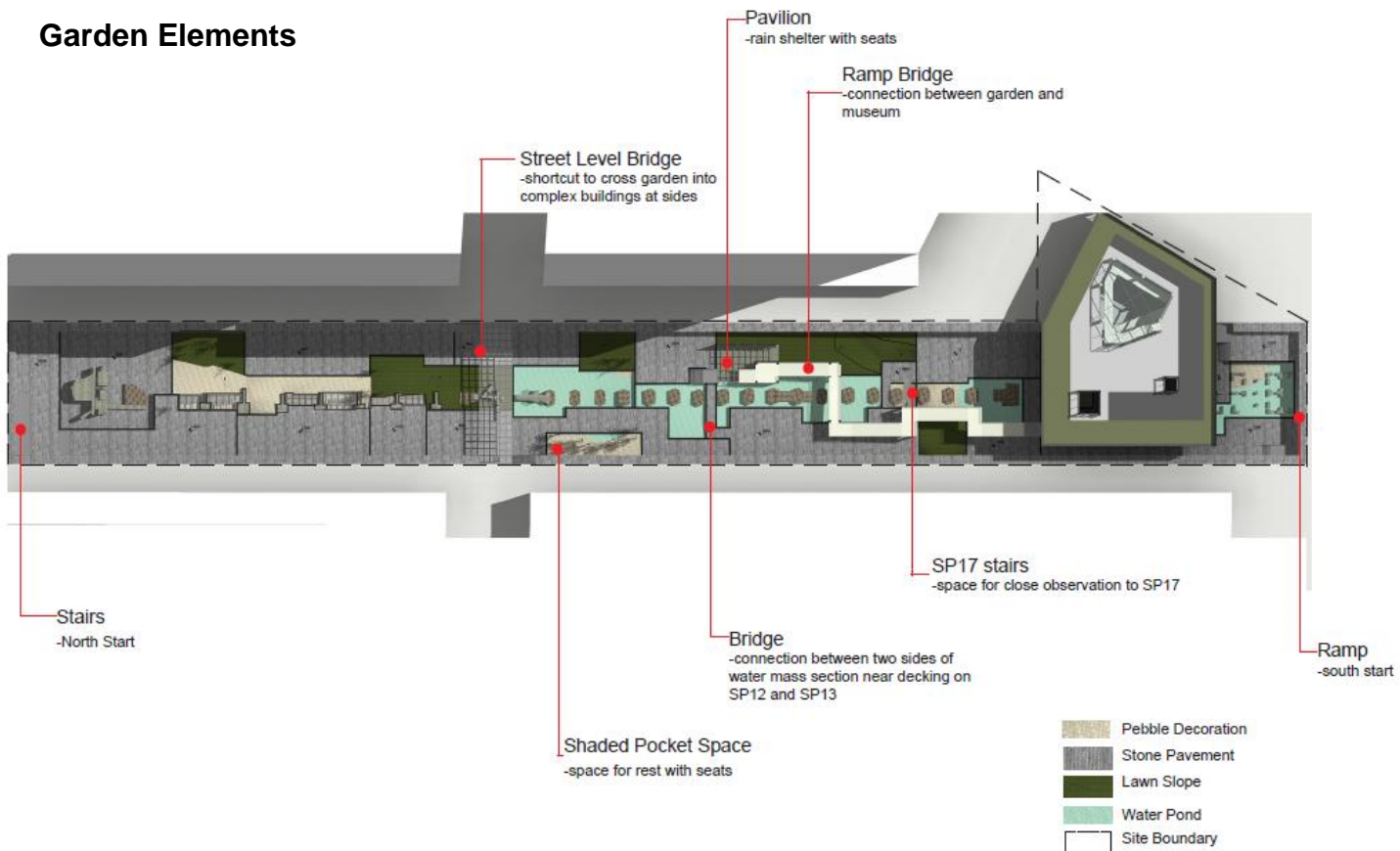


Figure 4-17 Garden Plan with Elements

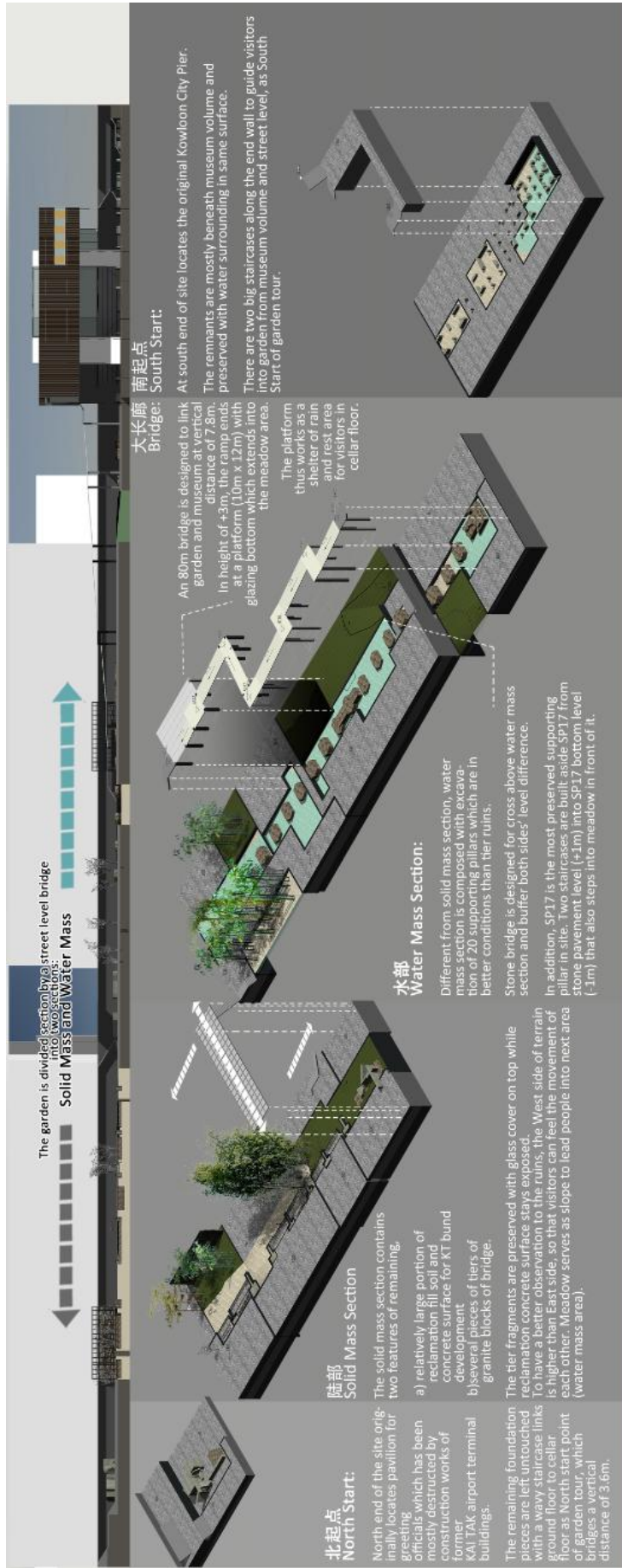


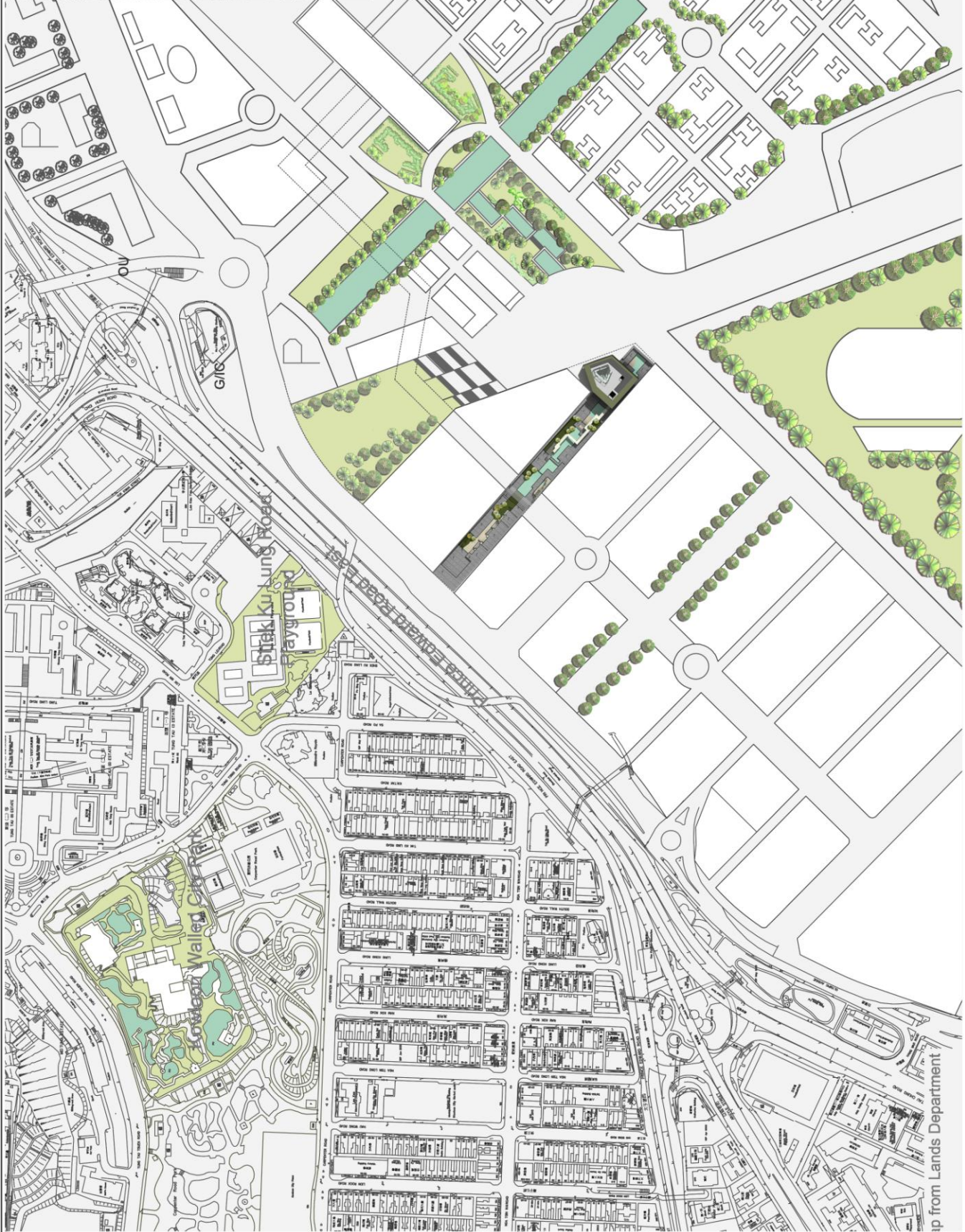
Figure 4-18 Elements in Perspectives

4.3 Garden Drawings

4.3.1 Masterplan



位置圖 Location Plan
 比例尺 Scale: 不按比例 N.T.S.



Drawing Scale
 1:1000

Drawing Code
 Masterplan



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Drawing Code
Garden plan

Drawing Scale
1:500

4.3.2 Impression





CHAPTER 5
ARCHITECTURE DESIGN

5.0 Architecture Design

5.1 Case Study

5.1.1 Museum Architecture

'Nothing can get around the realization that museum buildings as 'cathedrals of today' and museums as institutes of art relate to two very different worlds. The mechanism of supply and demand operates here as well. What has brought about the museum boom? Whether museums can really – or indeed exclusively – claim to account for the throng of visitors and the thirst for art is at least questionable.

Museums as institutions have trimmed the old traditional view of their educational role in order to adapt themselves to present-day habits in dealing with culture as consumption...At any rate, art is still at the heart of it. And when you look closer, you notice that a lot of what is said to be fundamental to the building of museums concerns one of the oldest functions of museums – preserving, and security measures. That applies not just to the Acropolis or Stonehenge, where problems have to be solved at an acute stage...

'Museums have become one of the most desirable commissions for architects to win, now that cities have discovered museums as a marketing factor. A spectacular museum building possesses supra-regional attraction, and in the best cases assures the city both a distinctive emblem and a city-centre function. Rundown or peripheral parts of cities can be revived by a museum and linked to the rest of the city – examples that spring to mind are Frank O.Gehry's Guggenheim Museum in Bilbao or Herzog & de Meuron's Tate Modern in London...The architecture of the museum itself, the building anchored in its urban context together with its functional room for the administration, must function in the dialogue between building and art, architecture and users (museum staff and visitors). In the end, it is therefore the experience of every individual visitor to the museum that decides whether the definition of the space in which he finds himself and sees cultural artefacts is a success or a failure. The future of a museum lies in proving itself as a complex space for new, permanent experiences and not just for lavish but short-term spectacles.'

(Werner Oechslin, Thierry Greub, *Museums in the 21st Century – Concepts, Projects, Buildings*, Art Center Basel



Figure 5-1 Tate Modern and Expansion, London, Herzog & de Meuron

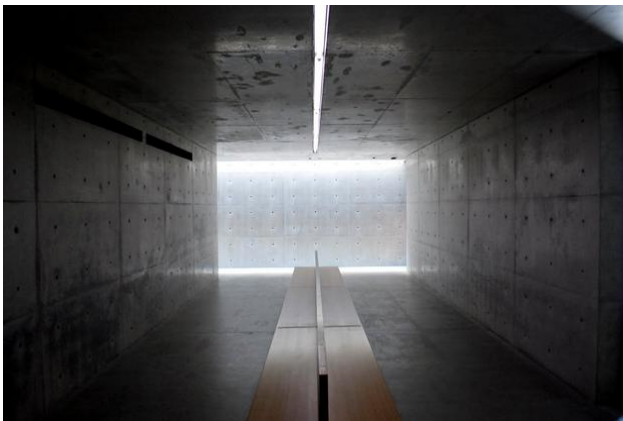


Figure 5-2 Chichu Art Museum in Kagama, Japan, by Tadao Ando

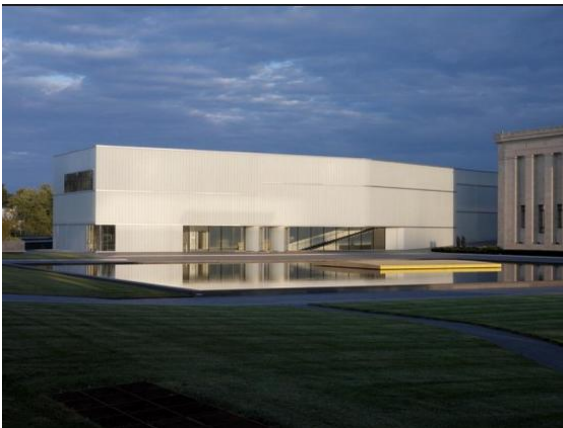


Figure 5-3 Nelson-Atkins Museum of Art Expansion in Kansas, USA , by Steven Holl (Left) ,
University of Michigan Museum of Art Expansion in Ann Arbor, USA, by Allied Works Architecture
(right)

5.1.2 Archaeology Museum

Archaeology museums specialize in the display of archaeological artifacts. Many are in the open air,. Others display artifacts found in archaeological sites inside buildings. This type of museum has also developed a 'museum-without-walls' through a series of underwater wreck trails.

Two specific archaeology museums and one museum in China were studied before the design work.

5.1.2.1 The New Acropolis Museum in Athens, Greece, by Bernard Tschumi Architects

The archaeological excavation site over which the new museum in Athens seems to float is south of the Acropolis, with a direct view of the Parthenon high above, and only a stone's throw away from the Theatre of Dionysus at the foot of the hill...The excavation site that was already there was to be affected as little as possible and it was necessary to follow strict earthquake protection guidelines and rigid local regulations for new buildings in this historical context. A design was needed that would bring together, within the arrangement of a central new building, the parts of the Parthenon frieze that are still in Greece and many other exhibits that are present still distributed among the little museum by the Acropolis, the National Museum and various storage facilities.

(Frank R. Werner Impressive time-space architecture: Bernard Tschumi's New Acropolis Museum in Athens, *Museums in the 21st Century – Concepts, Projects, Buildings*, Art Center Basel)



Figure 5-4 Perspective views with reference to the Parthenon, excavations (left),
Site Plan (right)

The only way to do justice to the location and the collection was with minimalistic simplicity and a complete lack of constructive-structural ambiguity...The building's spine is formed by a pathway through the exhibition, manifesting itself spatially, by analogy with the dramaturgy of the exhibits on show, in three building materials: concrete, marble and glass. The cubature of the new building is also tripartite. Here, the excavation site as the 'cellar floor' has been left largely untouched. All that has happened is that columns have been inserted intermittently, and these support the actual main floor of the new museum. And the entrance lobby, the temporary exhibition area and ancillary rooms have also been accommodated in an archaeologically exceptionable corner. The excavation zone is enclosed only by honeycomb-like wall grilles and sun-protection walls on the periphery.



Figure 5-5 Entrance (left), Main Lobby (right)

The main volume above, also accessible from the outside by open-air steps, has two storeys, and a mezzanine floor has been added. It contains all the major exhibition galleries from the Archaic Period to the Hellenistic-Roman epoch, and additionally accommodates lecture theatres, light wells, access systems, infrastructure facilities and a café terrace. All this is topped by a largely glazed pavilion, strictly rectangular in conception. It is linked directly with the Parthenon in terms of sightlines, and presents its world-famous frieze fragments.

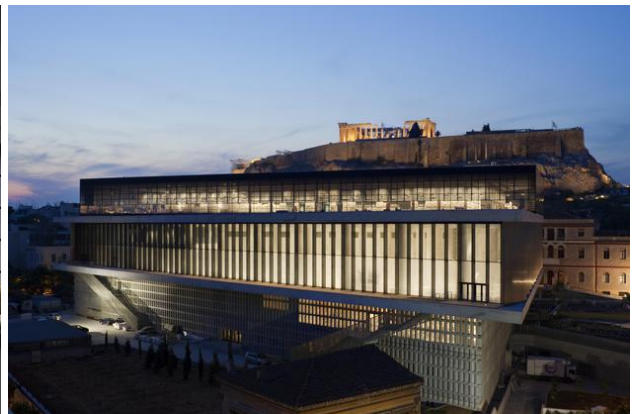
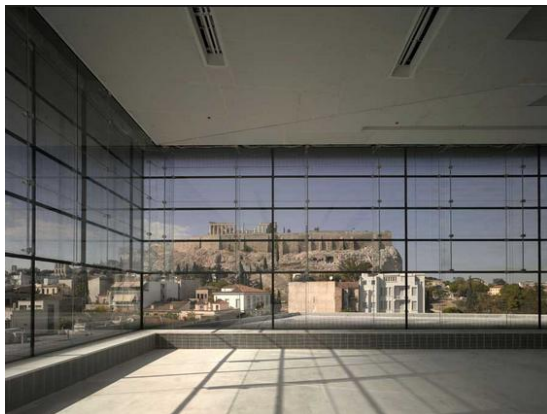


Figure 5-6 View to Parthenon from inside (left), night view of the museum (right)

5.1.2.2 Varus (Teutoburg) Battlefield, Kalkriese Museum and Park in Bramsche, Germany, by Gigon/Guyon Architects

The museum and landscape park at Kalkriese is the first museum in Europe built on a battlefield that together with its park has found a way to present the place, its history and its archaeological finds by suggestion rather than statement. The answers that architects came up with consist of making what is now a wholly inconspicuous battlefield into a specific place, using visual symbols and abstract means, while presenting the historical periods as superimposed layers and getting the terrain and archaeology to tell a convincing story.



Figure 5-7 Proposed surfaces in the Kalkriese-Niewedder Depression with Roman finds (as at August 2005)

The Kalkriese battlefield site comprises a visitor centre, the museum, three pavilions in the park and interventions in the landscape. The museum at the entrance to the park catches the eye from a distance. The L-shaped steel skeleton design, using double-T beams clad with massive 18-square-metre, oxidized, mild-steel plates, floats on slender stelae without damaging the historic terrain. The hall-like black box of the single storey structure contains the 600-square-metre museum and a lecture hall, while on the narrow northern side the 40-metre-high lookout tower gives the sort of overview of the battlefield a commander would have. The museum contains everything worth knowing about the battle, with a critical look at the archaeological evidence, but also casts an eye over the history of the Hermann (Armin) cult.

...The purpose of these light-hearted artistic interventions is to hone the eye for the landscape (in the 'seeing' pavilion it is crammed into a spherical lens and turned upside down), sharpen the perception (the 'hearing' pavilion is equipped with a horn-shaped listening trumpet that extends outside) and link the time layers together (the 'asking' pavilion has nine video monitors showing pictures of current theatres of war worldwide).



Figure 5-8 Kalkriese Museum (left), the 'hearing' pavilion (right)

Thus the textual sources, the archaeological finds, our conceptions of the Teutoburg ambush and battles in general, architecture and landscaping come together to turn a historic battlefield into a field of associations, a 'place of remembrance'. Over and beyond all myths and transfigurations, the museum and the authentic landscape (the only true 'exhibit') become not only an archaeological museum of a wholly new type but just as much a geographically specific paradigm and parable.

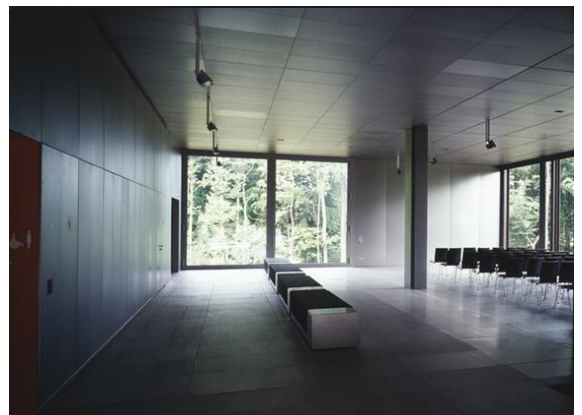


Figure 5-9 Steel plates: Romans route of march (left), Museum Lobby(right)

5.1.2.3 Museum in China - Guangdong Museum in Guangzhou, China by Rocco Yim

The Guangdong Museum is one of four major cultural landmark buildings for the new financial hub in Zhujiang Xincheng (Pearl River New Town) of Guangzhou. The five-storey museum has a total floor area of approximately 67,000 square metres.



Figure 5-10 Guangdong Museum

Conceived as an *Objet d'Art* at a monumental scale, an allegory to the impeccably and intricately sculpted antique Chinese artefact, such as a lacquer box, an ivory ball, a jade bowl or a bronze tank, which collects and reflects treasures of the times. The new museum is not only designed to house a great variety of fascinating objects of treasure, it is also in itself designed as a treasured object of great fascination that contemplates to become an identifiable cultural icon, giving the visitors a memorable tour and experience of the local provincial history and traditional wisdom as well as contributing to the appreciation and enhancement of cultural identity of the city.

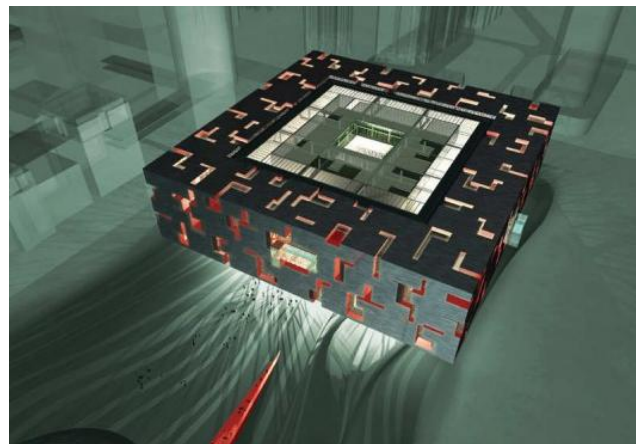
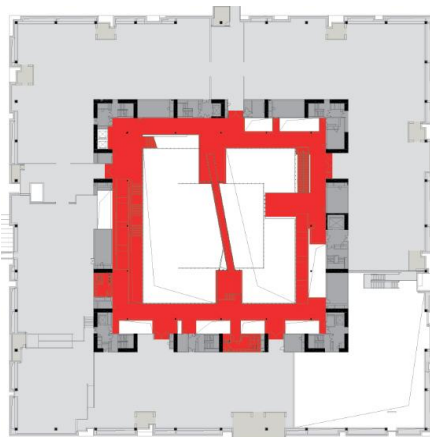


Figure 5-11 Plan (left) and night view(right)

The overall treatment of the main façade is also based on the analogy of ivory ball. Using materials such as aluminium panels, fritted glass and GRC panels, each elevation is uniquely designed with different geometric voids recessed into the building mass. In order to achieve a smooth transition between the museum and the adjoining landscape, an undulating landscape deck is introduced underneath the elevated museum box, metaphorically symbolizing a silk cloth unwrapping a much treasured piece of artwork.



Figure 5-12 Façade Details

The Museum's spatial arrangement takes its inspiration from the legendary concentric ivory balls carving. Each carving slices through the box and reveals different layers and varying degrees of transparency within the interiors, forming interesting spatial patterns and luring visitors through its exhibits inside. The interweaving of interior space pockets also reveal the intricate relationship between the visual and physical connections and separation of the atrium corridor, the individual exhibition halls and the back-of-house service areas. This deliberate arrangement not only reinforces the clarity and coherence of the treasure-box concept, but also allow flexibility in planning and operation of all the exhibition spaces. In addition, each of the main exhibition halls are punctured with random alcoves of dynamic spatial geometry. Filled with natural light and served as visual breakouts to the outside, they are also transitions between the exhibition halls which offer visitors intimate and well-balanced resting spaces.

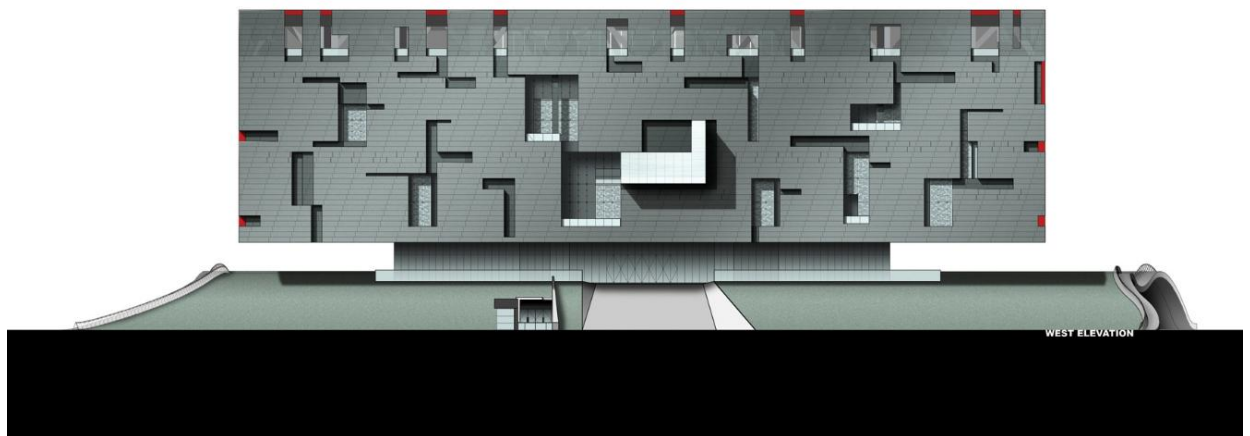


Figure 5-13 Elevation

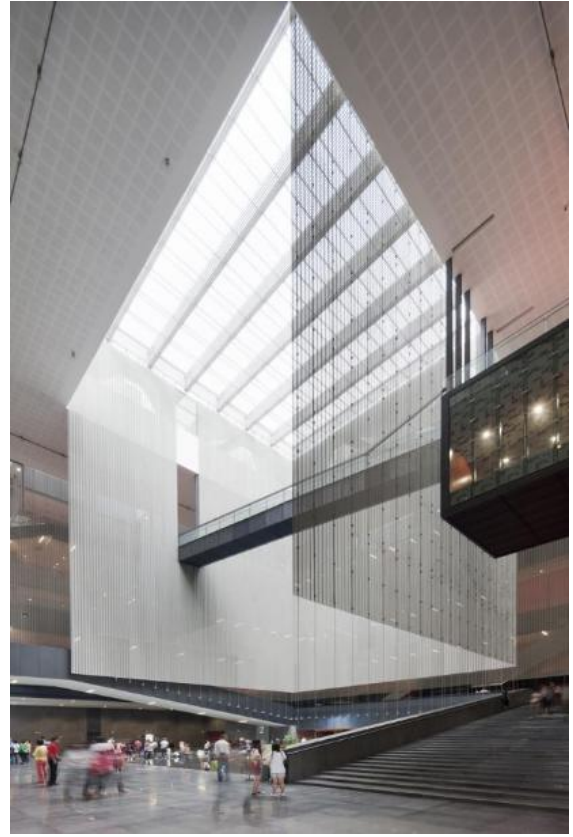
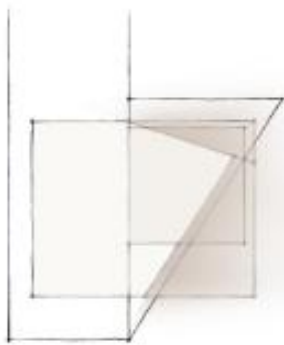


Figure 5-14 Interior of Guangdong Museum

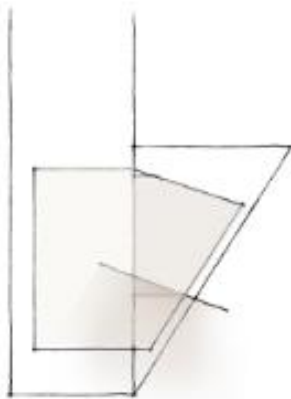
5.2 Museum Concept

Generally an archaeology museum shall function as a compact jewel box that conceals the history meaning. Particularly in our case, the museum should evoke the space and people from the surviving fragment of granite pillars (the treasure displayed outdoor). LTSB has been main entrance for Kowloon City that connecting the seaward and landward geologically. It also demonstrates the development of Hong Kong from 1800s to 1900s as a connection of culture and history. Taking the concept of 'bridge', a museum model was gradually formed to highlight its 'connecting' form.

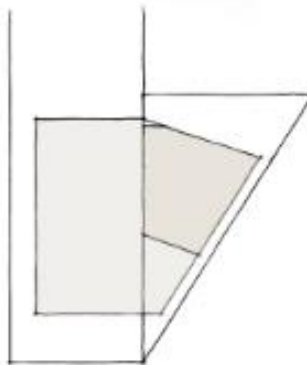
5.2.1 Shaping



The shape of the museum stays in humble and gentle morphology when at the same time fits in the existing urban context.

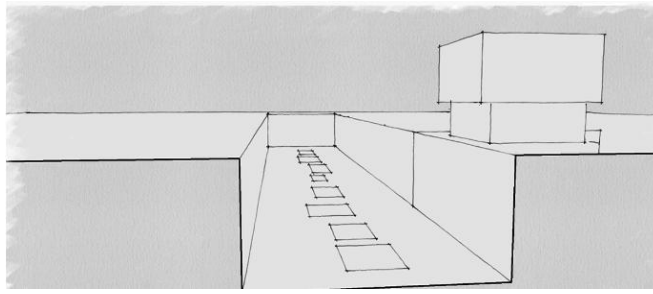


The entrance at lower floor faces South and the main body lays at North.

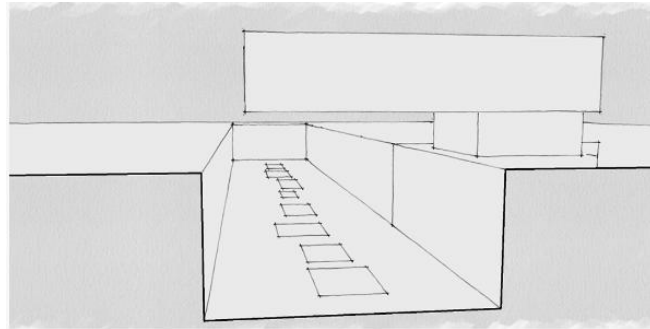


Space at corner is left blank for the balance of solid and void.

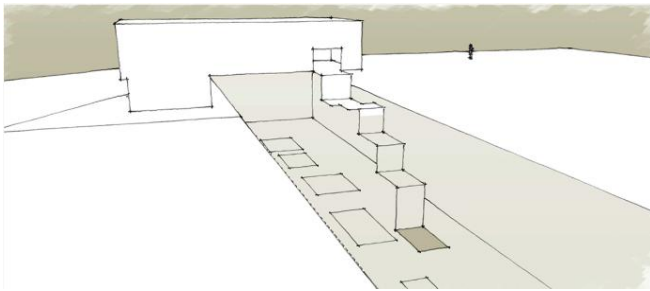
Figure 5-15 Horizontal Shaping



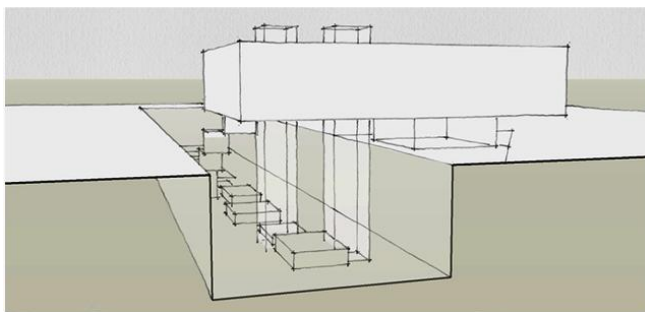
To achieve harmony of contrast, the museum volume goes solid where as the bridge remnant is at void cellar floor. Moreover, the museum has a stable linkage while the routes at bridge garden are instable zig-zag.



The suspended structure is introduced to lay over cellar floor so to create an overlook the garden at central axis and to accentuate the spine of the whole area. It represents the 'connecting bridge' concept



Between museum and the garden, a bridge is to offer views of the garden as one steps out and a further linkage between indoor and outdoor.



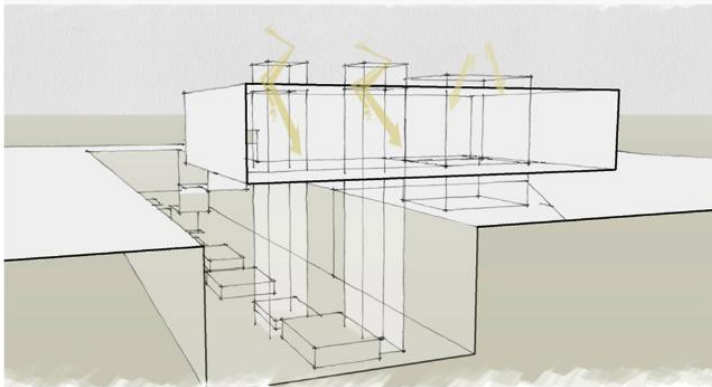
Two supporting structure is to simulate the in-situ supporting pillar and to stabilize the museum volume.

Figure 5-16 Vertical Shaping

The volume is raised above the ground-plane, floating off the surface, allowing visitors to approach from different directions. The garden undulates beneath the structure, leading visitors into and out of the museum.

Therefore the final form shows our respect to the concept of 'Bridge' as a tool of connection. The museum shall contain the archaeological value that connects the past, present and future in its function of interpreting the history, while it also connects the eastern architecture philosophy to the western construction skills.

5.2.2 Strategy



The two supporting structure is transformed into light tube to introduce natural light in the suspended floor.

The penetrating atrium as museum spine is to import skylight through both upper and lower floors.

Figure 5-17 Natural Light Strategy

5.2.3 Façade Texture



Inspired by the Piling Method of supporting pillars, longitudinal board was placed in parallel for museum façade.

Openings are made where needed.

Figure 5-18 Sketch 1

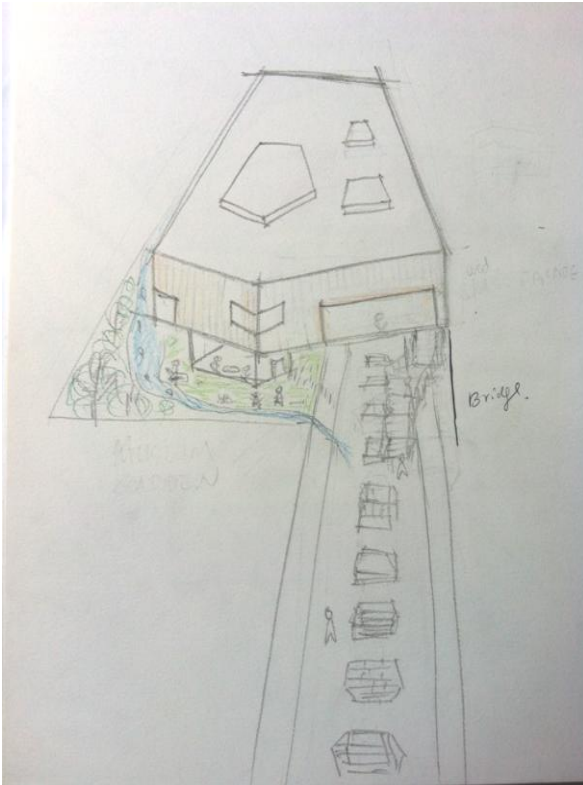


Figure 5-19 Sketch 2

To maintain the Qi balance from the void excavation site, the museum is designed to be a solid volume. The upper floor is mostly with solid-looking texture while the lower floor stays light. The appearance will be a floating modern architecture that defends and interprets historical ruins.

5.2.4 Volume Transformation

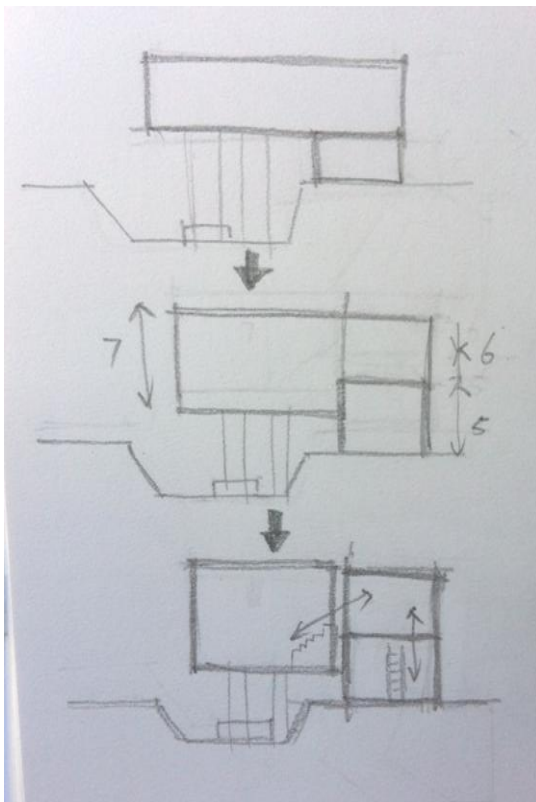


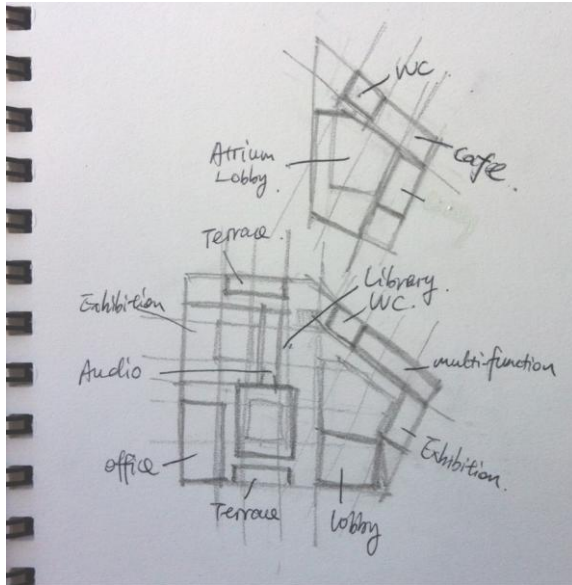
Figure 5-20 Sketch 3

To create a more lively interaction between museum and garden, the particular structure that floats above the site is dragged down to have its floor slab closer to the ruins.

Thus a floating 1-floor block and a stable 2-floor block are created.

Staircases and lifts are the vertical linkage in circulation connection.

5.2.5 Function Hierarchy



In lower floor, the space have different functions, mainly lobby, shops, café and toilets

In upper floors, the spaces have functions including exhibition room, audio rooms, multi-functional rooms, library, viewing terrace, staff office, etc.

Figure 5-21 Sketch 4

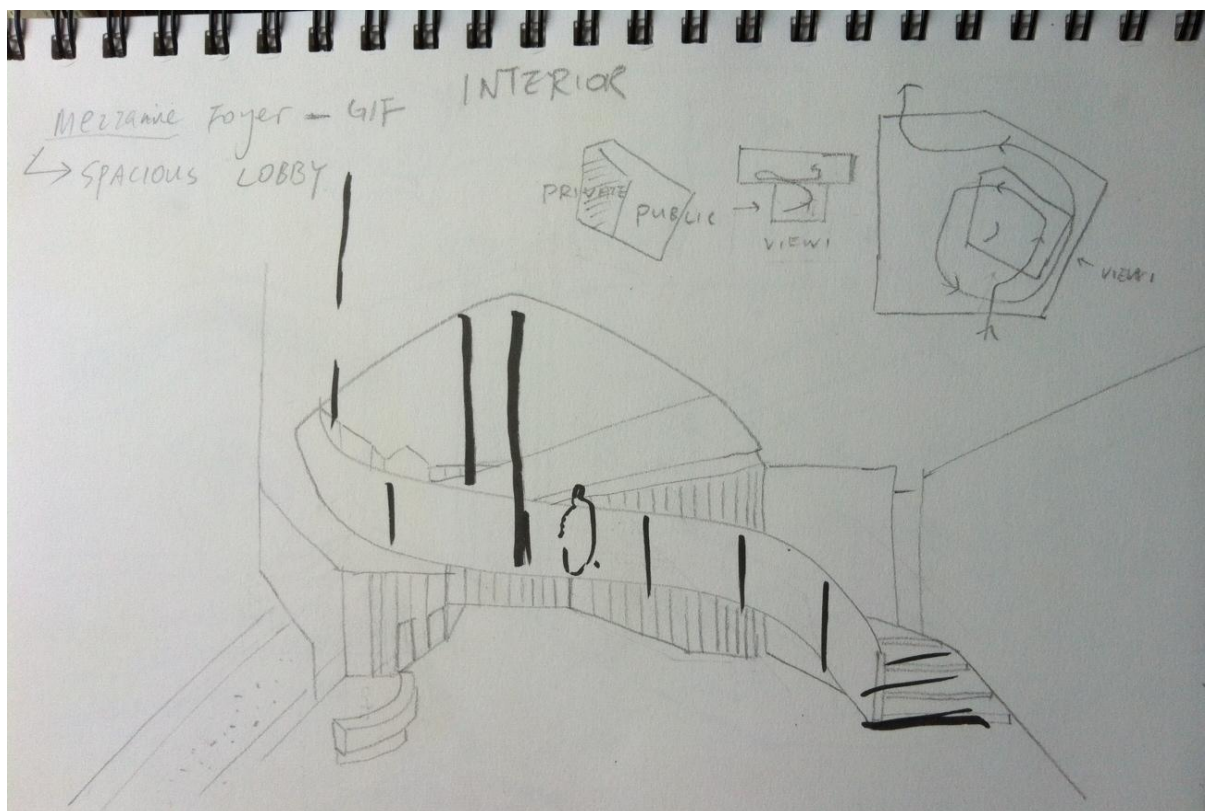


Figure 5-22 Sketch 5 (lobby impression)



Figure 5-23 Plan in Function

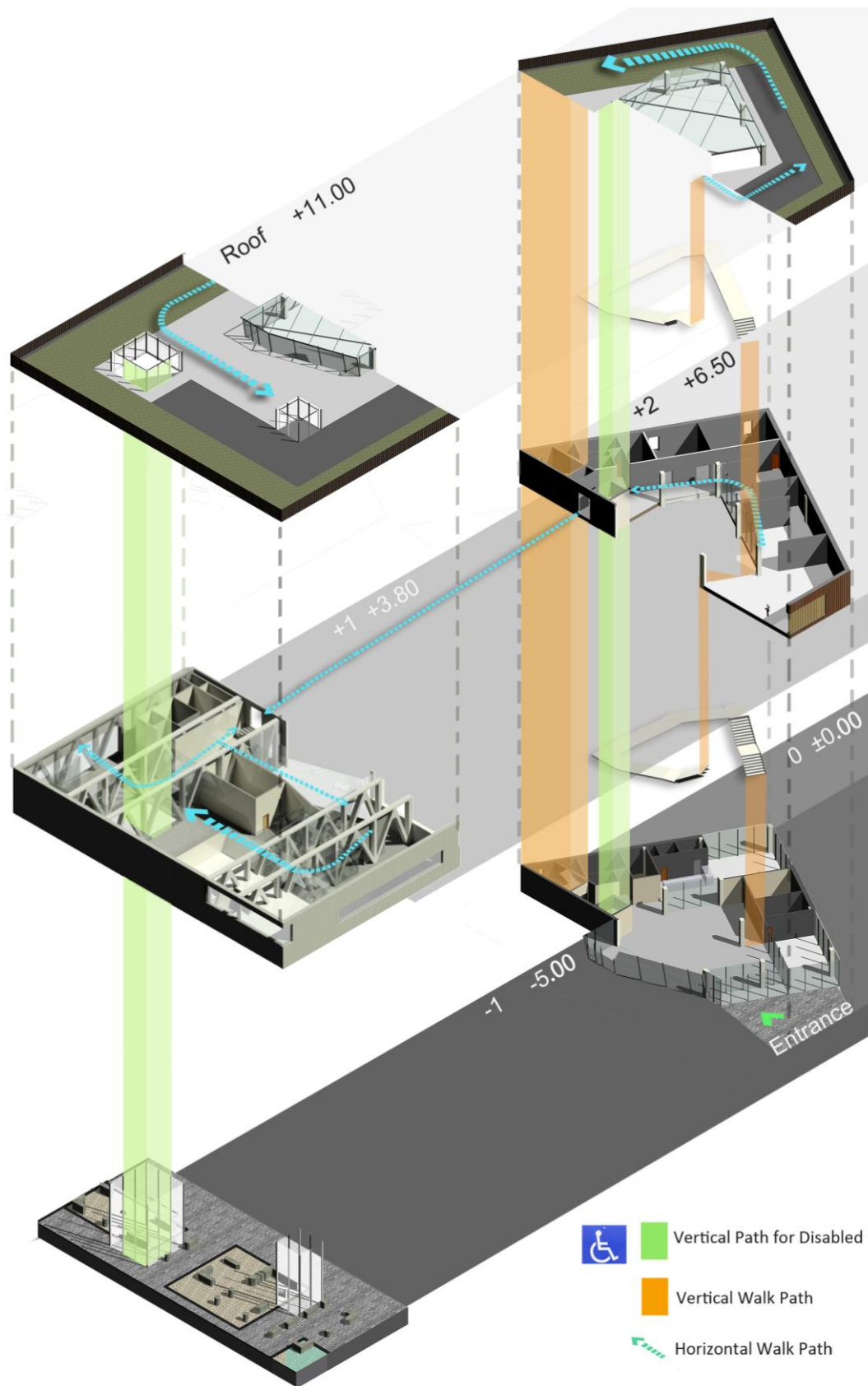
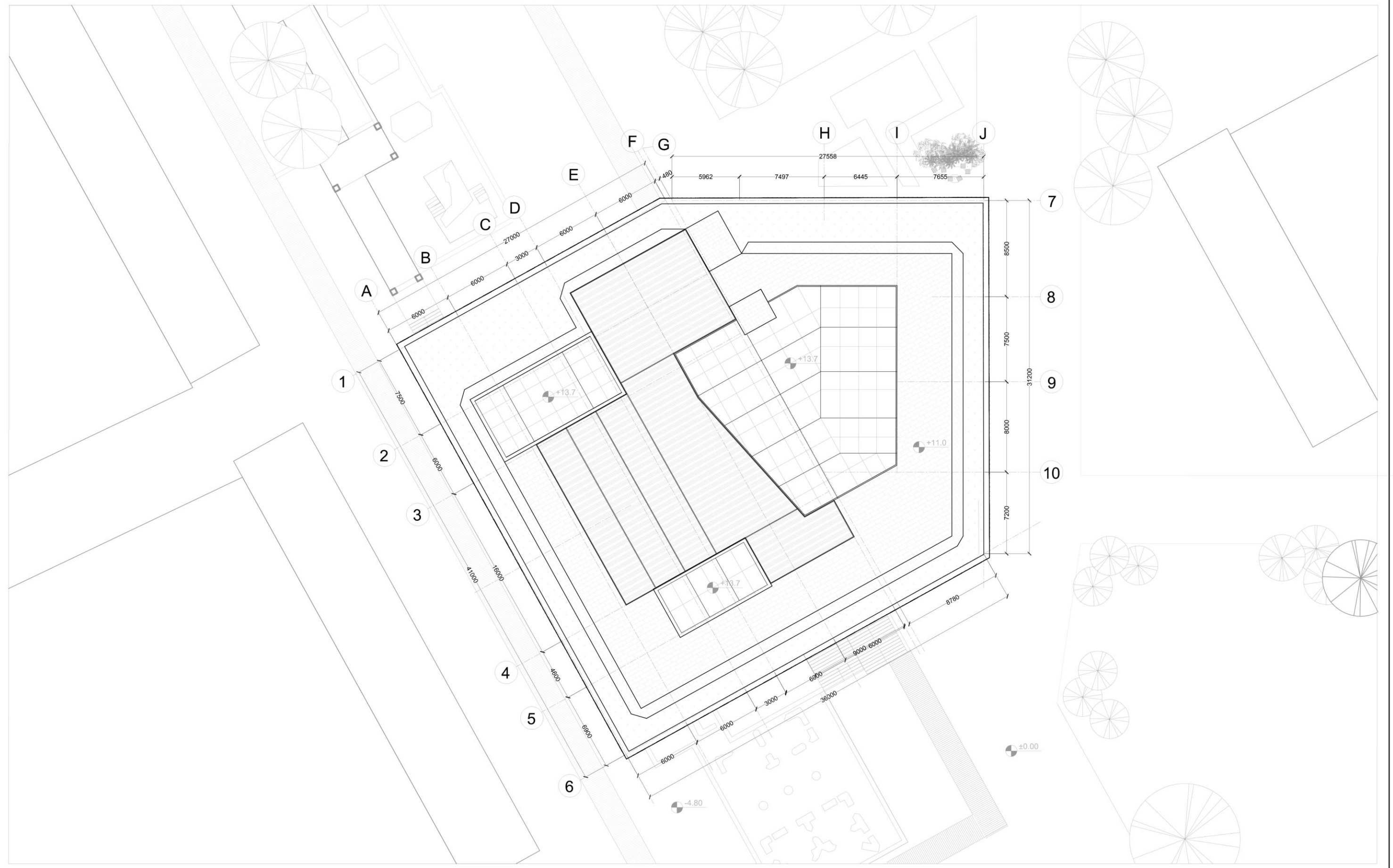


Figure 5-24 Circulation Map

5.3 Drawings

5.3.1 Plan

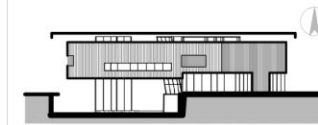


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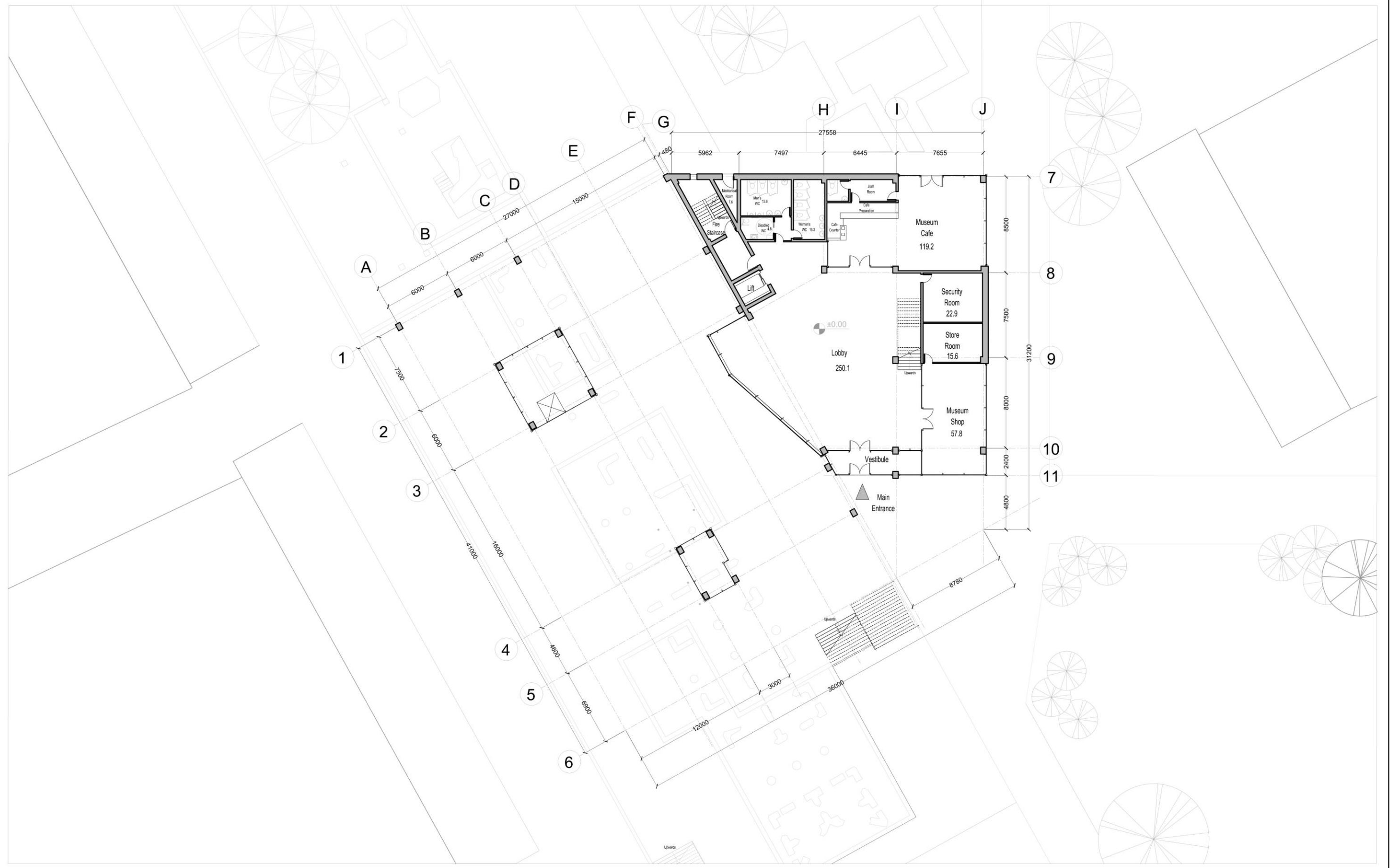
PROJECT
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Drawing Code
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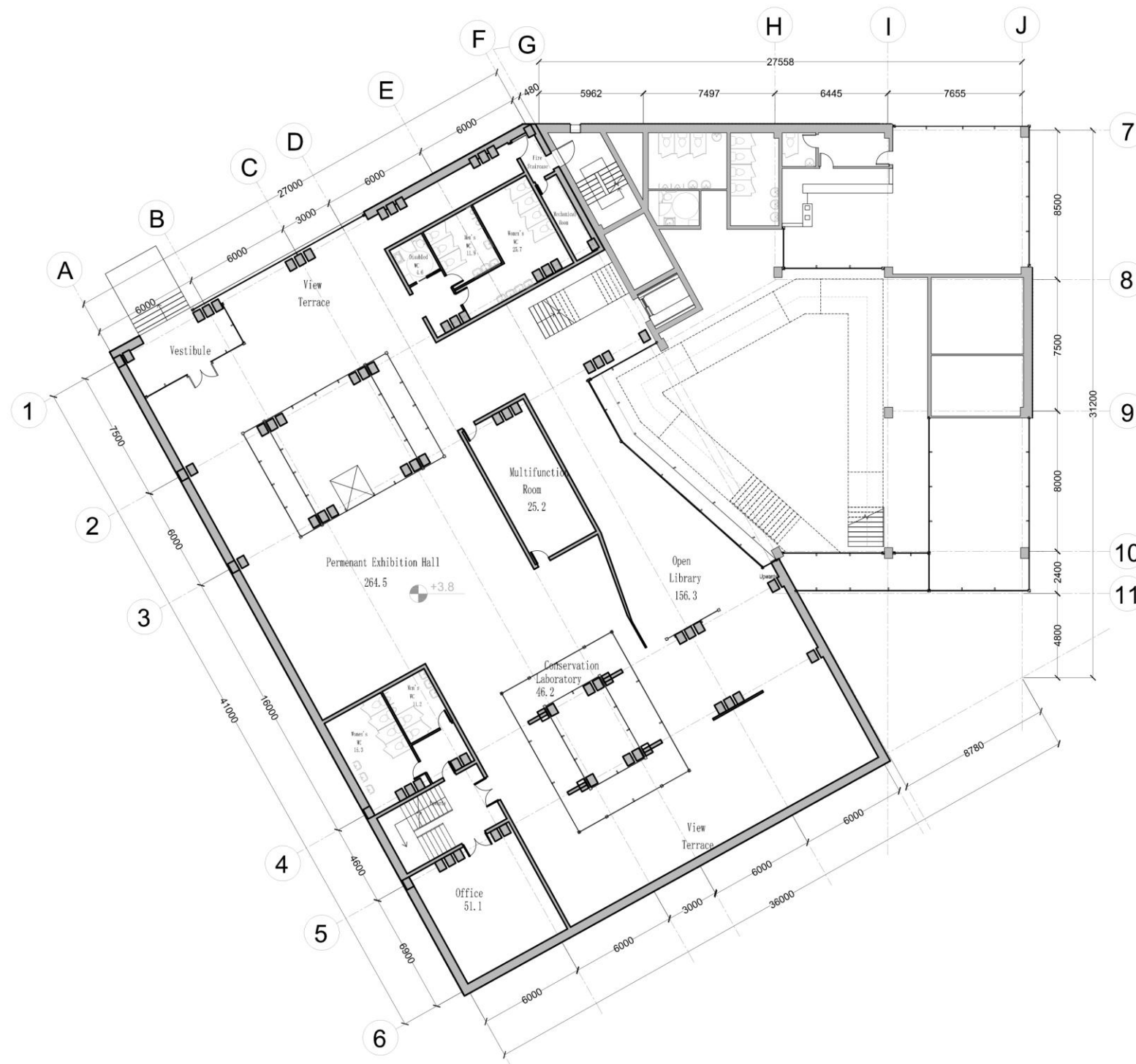
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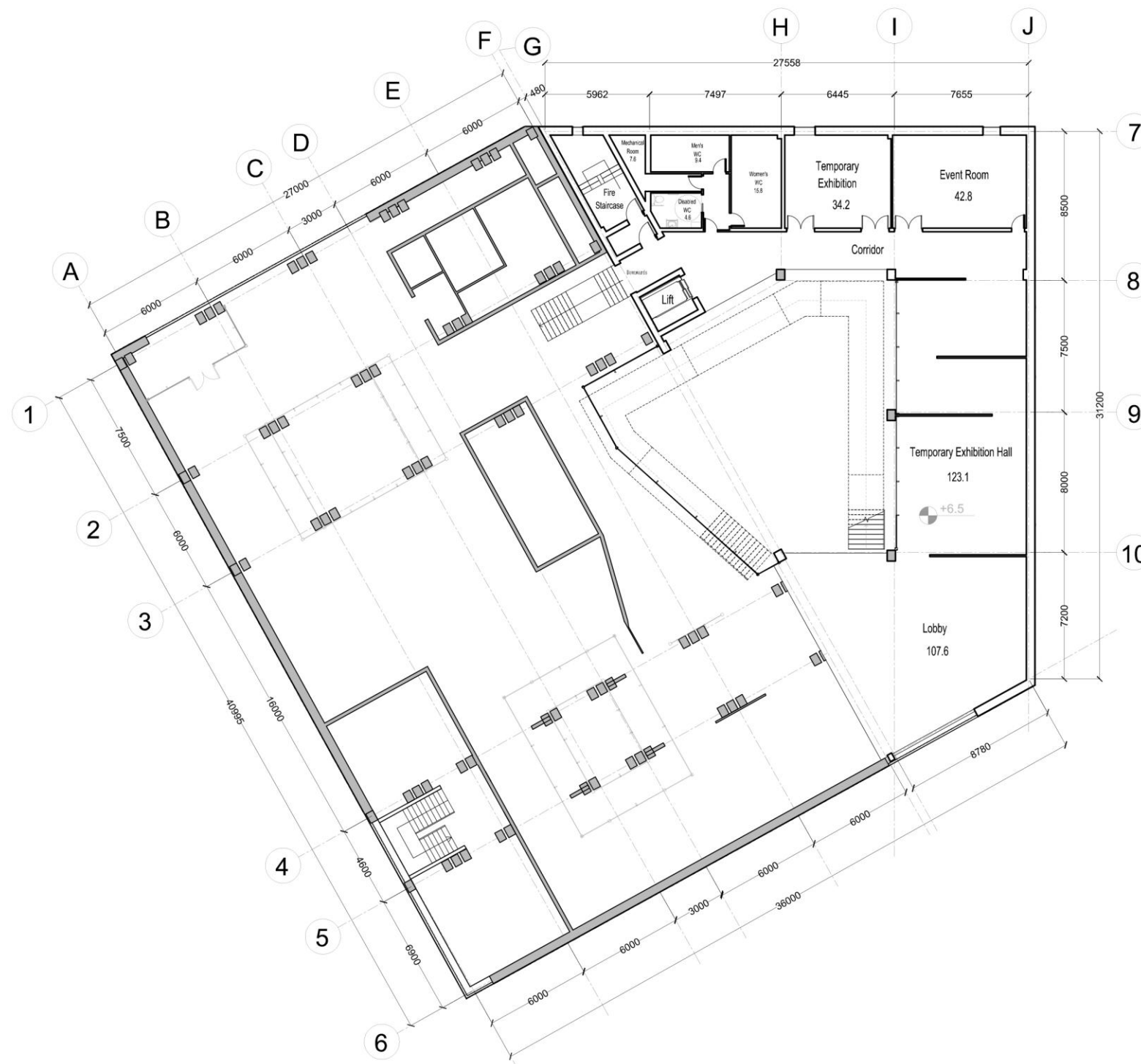
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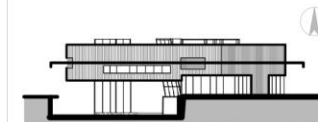


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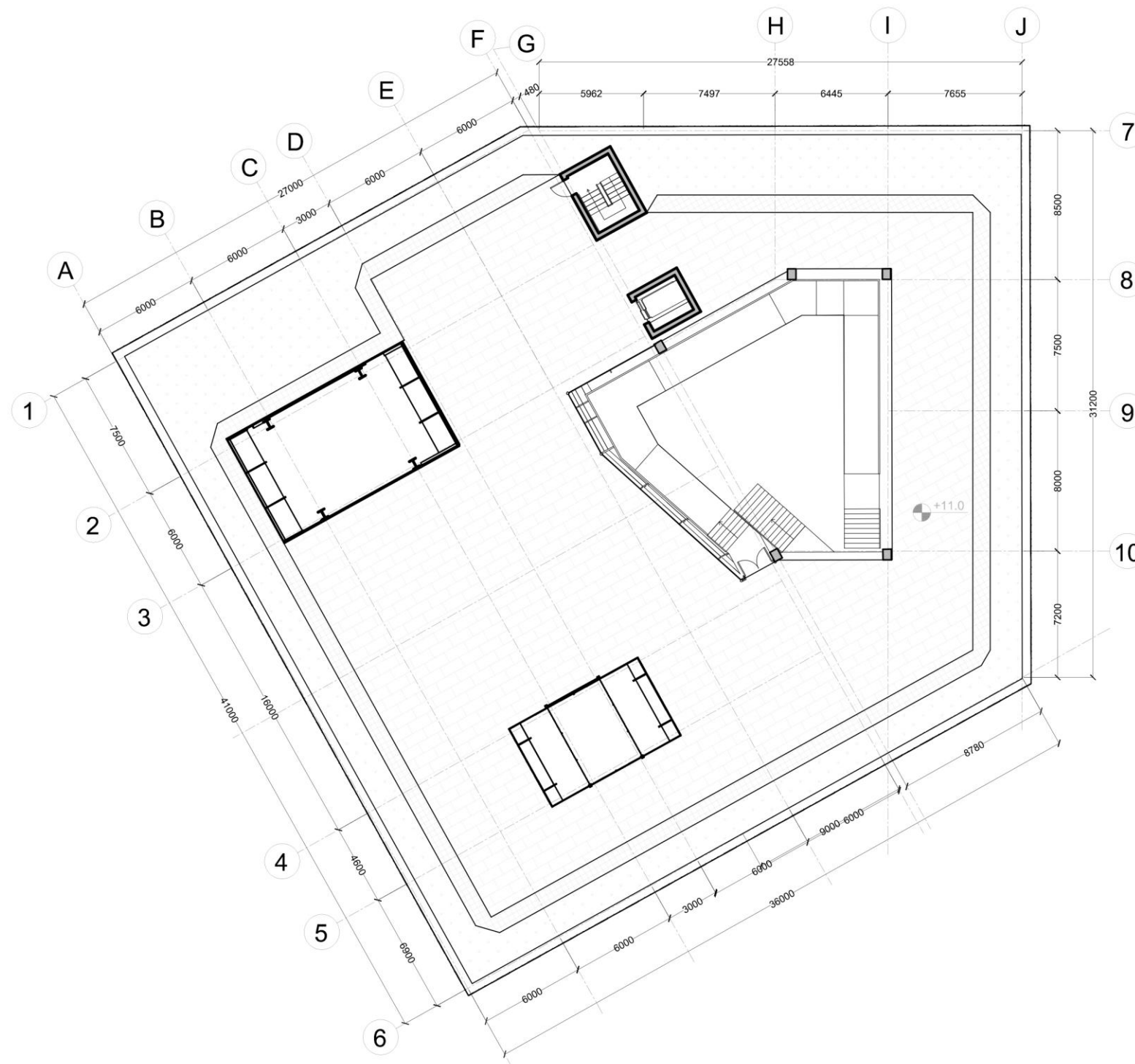
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5.3.2 Elevation



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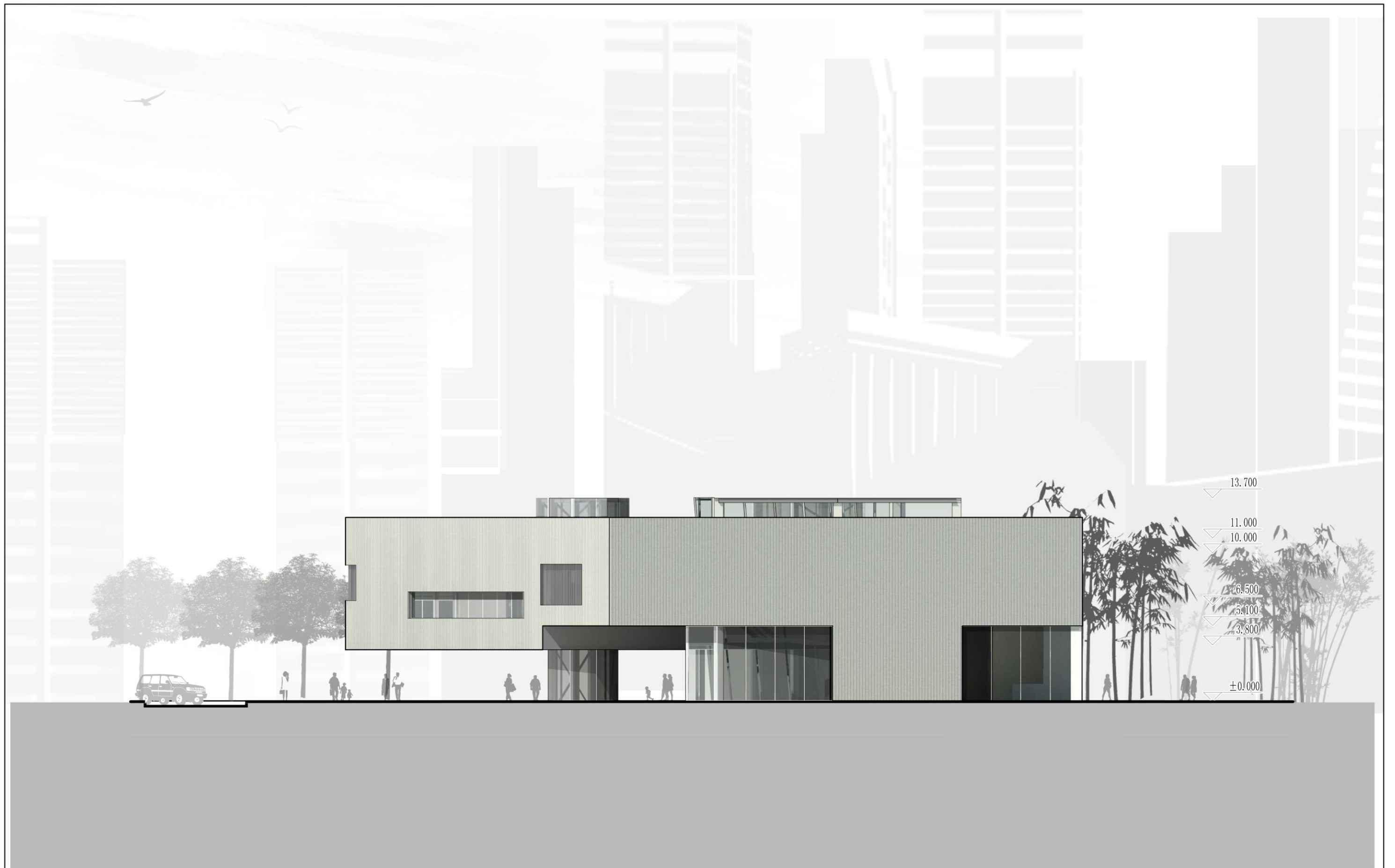
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5.3.3 Section



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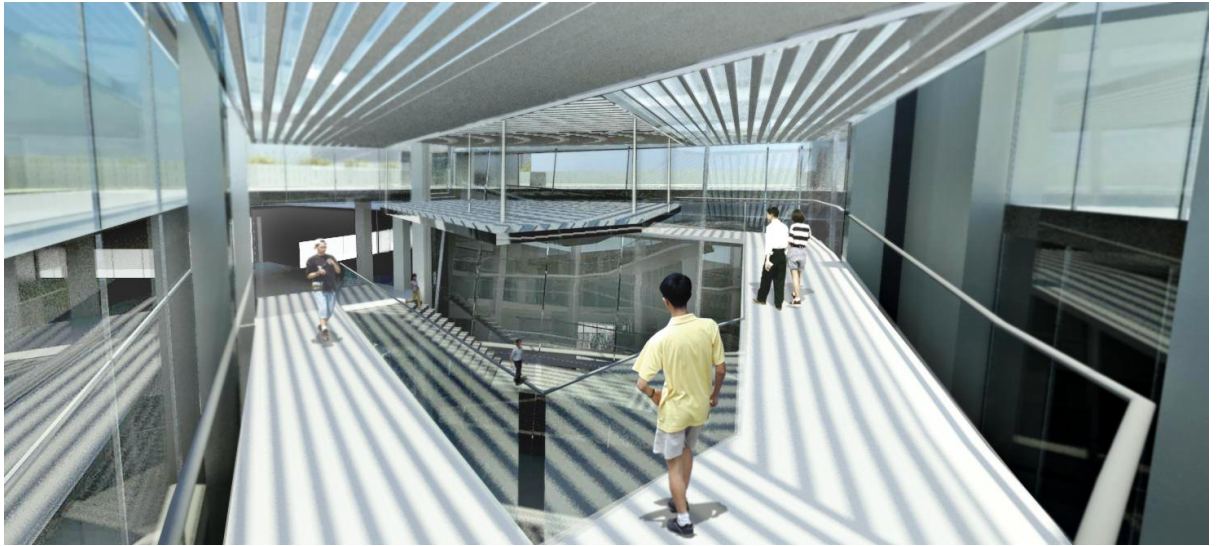
5.3.4 Impression











CHAPTER 6
STRUCTURE DESIGN

6.0 Structure Design

6.1 Introduction

After the museum shape and volume is determined, components materials, climate and site condition etc have been considered carefully before the design of museum structure. Finally, two different structure systems are designed.

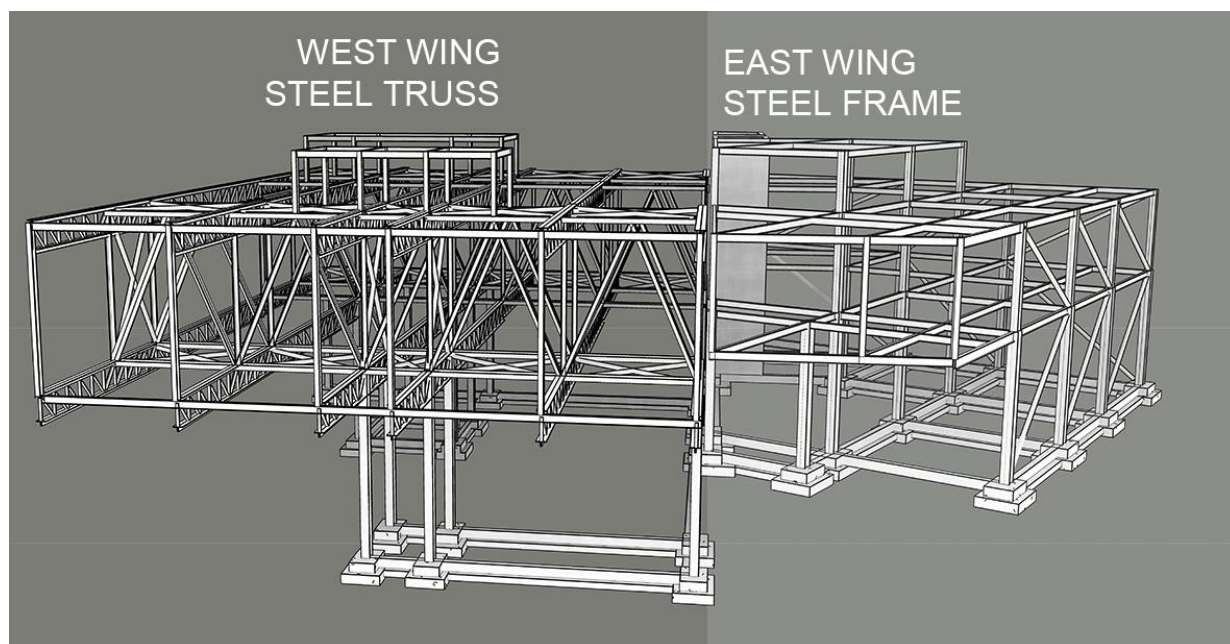


Figure 6-1 3D model of the overall structure

At the East Wing, regular steel frame structure is used to express the solid and stable volume of the two floors that rise above the ground plane.

At the West Wing, the volume floats off the ruin surface so to create a crossing 'bridge' structure whereas to minimize the interruption to the ruins beneath. Because of the large-span cantilever situation, two column-structures are necessary to support the loading by being connected with concrete foundation system. To have more flexibility in the space distribution, the large-area created in between these two structures shall stay clear. Thus it needs another support system other than applying more columns. According to the specific requirement of West Wing, truss system is eventually selected.

Please refer to the structural drawings attached in Section 6.5.

All the analysis and estimations are based on the Euro Codes:

- 1) Eurocode (EN 1990): Basis of structural design
- 2) Eurocode 1 (EN 1991): Actions on structures
- 3) Eurocode 3 (EN 1993): Design of steel structures

4) Eurocode 4 (EN 1994): Design of composite steel and concrete structures

6.2 Actions on structure

Generally four types of load need to be analyzed, namely **Dead loads (G)**: the self-weight (including finishes), shall be calculated from dimensions of the structures and the density of the materials used; **Imposed loads (Q)**: shall produce the most adverse effect; **Wind loads (W)** and **Snow loads**. In Hong Kong, it is not necessary to consider structure action regarding snow load.

Therefore, three types of load, (G,Q and W) will be explained in details in following pages.

6.2.1 Dead loads

6.2.1.1 External Wall

Ventilated External Wall				
No.	Layer	Thickness	Density	Weight
		(mm)	(kg/m ³)	(kN/m ²)
1	Marly Etemit FC' Fiber Cement Cladding	12	350	0.041
2	Vertical Cladding Rail Substructure	26	-	-
3	Ventilation Cavity	30	1.3	0.000
4	Plastic Vapour Barrier	5	1250	0.061
5	Extruded Rigid Polystyrene Insulation	100	40	0.039
6	Fiber Cement Board	8	700	0.055
7	Rockwool Insulation	100	38	0.037
8	Vapour Barrier	3	1250	0.037
9	Double Plasterboard	24	950	0.323
Total				0.594

Table 6-2 Layers and Density of Ventilated External Wall

Floor No.	Height	Weight of External Wall (kN/m)	
	(m)	without Bracing	with Bracing
G/F	6.5	3.86	4.0
1/F	7.2	4.28	4.5
2/F	4.5	2.67	3.0

Table 6-3 Weight of Ventilated External Wall on Each Floor

6.2.1.2 Floors

1/F Typical Floor				
No.	Layer	Thickness	Density	Weight
		(mm)	(kg/m ³)	(kN/m ²)
1	Recycled Carpet Tile and Adhesive Layer	10	950	0.093
2	Screed Layer	30	950	0.279
3	Separation Layer PET	5	1250	0.061
4	Extruded Polystyrene Insulation	100	40	0.034
5	Trickle Protection Layer	10	100	0.010
6	RC Layer and Galvanized Steel Deck	150	2300	2.781
7	Extruded Polystyrene Insulation	100	38	0.032
8	Trickle Protection Layer	10	100	0.010
9	Waterproofing Membrane	5	1250	0.061
10	Fibercement Board	12	350	0.041
Total				3.40

Table 6-4 Layers and Density of 1/F typical floor

2/F Typical Floor				
No.	Layer	Thickness	Density	Weight
		(mm)	(kg/m ³)	(kN/m ²)
1	Recycled Carpet Tile and Adhesive Layer	10	950	0.093
2	Screed Layer	30	950	0.279
3	Separation Layer PET	5	1250	0.060
4	Extruded Polystyrene Insulation	100	40	0.035
5	Trickle Protection Layer	5	100	0.005
6	RC Layer and Galvanized Steel Deck	150	2300	2.875
7	Rockwool Insulation	30	12	0.004
8	Fire Resistant Plasterboard	12	950	0.110
Total				3.46

Table 6-5 Layers and Density of 2/F typical floor

3.5 kN/m² is taken as dead load for both floor element for further calculation

Roof				
No.	Layer	Thickness	Density	Weight
		(mm)	(kg/m ³)	(kN/m ²)
1	Recycled Polished Concrete	20	950	0.015
2	Screed Layer	22	950	0.205
3	Separation Layer PET	2	1250	0.025
4	Extruded Polystyrene Insulation	60	40	0.024
5	RC Concrete Slab	150	2300	3.181
7	Extruded Polystyrene Insulation	100	38	0.037
8	Acoustic Insulation	40	12	0.005
9	Separation Layer PET	2	1250	0.025
10	Fire Resistant Plasterboard	24	12	0.003
Total				3.52

Table 6-6 Layers and Density of Roof

3.5 kN/m² is taken as dead load for roof element for further calculation

6.2.2 Imposed loads

According to Section 6 *Imposed Loads on buildings*, EN1991-1-1:2002, Imposed loads shall be classified as variable free actions. They are those arising from occupancy, including

- Normal use by persons;
- Furniture and moveable objects
- Vehicles;
- Anticipating rare events.

The imposed loads specified are modeled by uniformly distributed loads, line loads, or concentrated loads or combinations of these loads. Only uniformly distributed load is taken here for load simulation.

Table 6-3 below shows the category of floor/roof use that provided in EN1991-1-1:2002, and the value that selected for our project.

Category	Specific Use	Sub-Category	Example	q_k (kN/m ²) (uniformly distributed load)	
				Characteristic Range	Value Used in our Project
B	Office Area			2.0-3.0	3.0
C	Area where people may congregate	C1: Areas with tables	schools, café, receptions, etc	2.0-3.0	3.0
		C2: Areas with fixed seats	conference rooms, lecture halls, etc	3.0-4.0	3.0
		C3: Area without obstacles for moving people	museum, exhibition rooms	4.0-5.0	5.0
Roof- I	Roof accessible with occupancy according to Categories A to G				2.0

Table 6-7 Categories of use and values for Imposed Load,
According to EN 1991-1-1:2002 Table 6.1, 6.2 and 6.9

Assuming the even distribution of areas under Category C, an average value $q_k=4$ kN/m² is taken for calculation.

6.2.3 Wind loads

Two dominant factors shape the extreme wind load in Hong Kong. One is the exposure to severe typhoons while the other is the protection by one of the most sheltered natural harbors in the world. These two factors tend to interact each other greatly.

EN1991-1-4 gives guidance on the determination of natural wind actions for the structural design of building and civil engineering works for each of the loaded areas under consideration.

6.2.3.1 Fundamental Value of the Basic Wind Velocity $v_{b,0}$

$v_{b,0}$ is the characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separation of at least 20 obstacle heights.

According to UK National Annex to Eurocode 1, $v_{b,0}$ value is adjusted for altitude and topography in the UK NA to EN 1991-1-4.

According to Section 2.6, Code of Practice on Wind Effects in Hong Kong 2004, this altitude factor however is not included in the Code due to the geographical nature of Hong Kong.

At Section 1.6, Code of Practice on Wind Effects in Hong Kong 2004, based on the analysis and other sources of published information, the Code has adopted reference hourly mean wind speeds of 65.2m/s at a height of 90m above mean sea level in Waglan Island...it can be seen that the adopted values have demonstrate the expected level of confidence for design purpose.

Hence $v_{b,0} = 65.2\text{m/s}$

6.2.3.2 The Basic Wind Velocity v_b

$$V_b = C_{dir} \times C_{season} \times V_{b,0}$$

Where

v_b is the basic wind velocity, defined as a function of wind direction and time of year at 10 m above ground of terrain category II,

$v_{b,0}$ is the fundamental value of the basic wind velocity = 46.9m/s

C_{dir} is the directional factor =1.0 (recommended by National Annex)

C_{season} is the season factor =1.0 (recommended by National Annex)

NB: The value for C_{dir} and C_{season} are recommended by EN 1991-1-4 is 1.

Hence $v_b = 65.2\text{m/s}$

6.2.3.3 The Mean Wind Velocity $v_{m(z)}$

At a height z above the terrain depends on the terrain roughness and orography and on the basic wind velocity, v_b , and should be determined as

$$v_{m(z)} = c_r(z) \times c_o(z) \times v_b$$

Where

$c_o(z)$ is the orography factor, taken as 1.0 unless otherwise specified.

$c_r(z)$ is the roughness factor, accounts for the variability of the mean wind velocity at the site of the structure due to the height above ground level and the ground roughness of the terrain upwind of the structure in the wind direction considered.

The recommended procedure for the determination of $c_r(z)$ is based on a logarithmic velocity profile and is given as:

$$c_r(z) = k_r \times \ln(z / z_0) \text{ for } z_{\min} \leq z \leq z_{\max}$$

$$c_r(z) = c_r(z_{\min}) \text{ for } z \leq z_{\min}$$

Where,

z_0 is the roughness length

k_r is the terrain factor depending on the roughness length z_0

$$k_r = 0.19 \times (z_0 / z_{0,II})^{0.07}$$

Where

$z_{0,II} = 0.05$ m (refer to the Table 6-4 below)

z_{\min} is the minimum height (refer to the Table 6-4 below)

z_{\max} is to be taken as 200 m, unless otherwise specified in the National Annex

Terrain Category	z_0 (m)	z_{\min} (m)
0 Sea or coastal area exposed to the open sea	0.003	1
I Lakes or flat and horizontal area with negligible vegetation and without obstacles	0.01	1
II Area with low vegetation such as grass and isolated obstacles(trees, buildings) with separations of at least 20 obstacle heights	0.05	2
III Area with regular cover of vegetation or building or with isolated obstacles with separations of maximum 20 obstacle heights(such as villages, suburban terrain, permanent forest)	0.3	5
IV Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15m	1	10

Table 6-8 Terrain Categories and terrain parameters, drawn from Table 4.1, Eurocode 1991-1-4

Our site is located in Hong Kong, so the terrain condition belongs to category IV.
Hence, $z_0 = 1\text{m}$ and $z_{\min} = 10\text{ m}$

$$k_r = 0.19 \times (1 / 0.05)^{0.07} = 0.234$$

$$c_r(z_{\min}) = k_r \times \ln(z/z_0) = 0.234 \times \ln 10 = 0.539$$

$$c_r(z) = 0.234 \times \ln(z)$$

Since the minimum level of the building is 6.5 m high, it can be concluded that $z_{\min} < z_i < z_{\max}$ for all levels of i .

Where

i is the number of the levels.

6.2.3.4 The Turbulence Intensity $I_v(z)$

$$I_v(z) = \frac{k_I}{c_0(z) \ln(z/z_0)} \quad \text{for } z_{\min} < z < z_{\max}$$

$$I_v = I_v(z_{\min}) \quad \text{for } z < z_{\min}$$

Where,

k_I is the turbulence factor = 1.0 (recommended by EN 1991-1-4)

6.2.3.5 The Peak Velocity Pressure $q_p(z)$

At height z , this includes mean and short-term velocity fluctuations, as

$$q_p(z) = [1+7 I_v(z)] \frac{1}{2} \rho v_m(z)^2 = [1+7 I_v(z)] \frac{1}{2} \rho c_r(z)^2 c_o(z)^2 v_b^2 = c_e(z) q_b$$

Where,

ρ is the air density. The recommended value is 1.25 kg/m^3 .

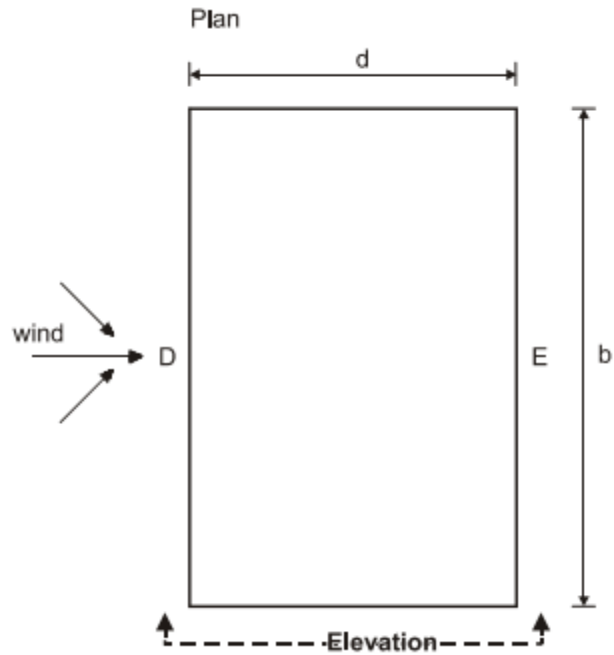


Figure 6-2 Dimension of Building Measurement

Where z_e is the reference height for windward walls of rectangular plan buildings

where

h is the height of the building, =13.7m

b is the crosswind dimension =41m/52m (depends on wind direction)

$e=b$ or $2h$ which is smaller

It's obvious that for whatever wind direction. $h < b$, so $z_e = h = 13.7m$.

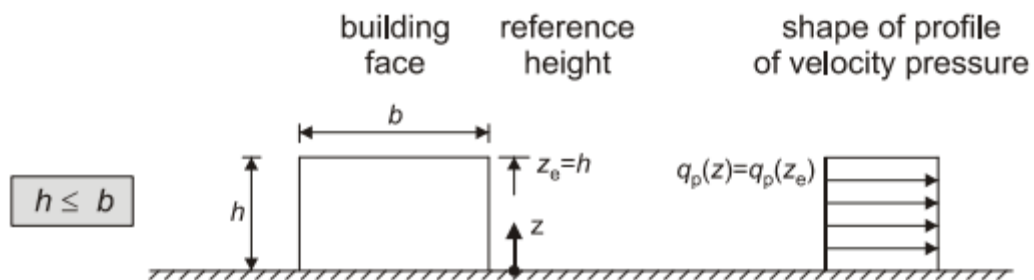


Figure 6-3 Reference height z_e

Hence $q_p(z) = q_p(z_e) = q_p(h)$

$c_r(z) = 0.234 \times \ln(13.7) = 0.612$

$I_v(z) = 1/(\ln(13.7)) = 0.382$

$v_m(z) = 0.612 \times 1 \times 65.2m/s = 40m/s$

Levels	Height $z(m)$	$c_r(z)$	$I_v(z)$	$v_m(z)$ (m/s)	$q_p(z)$ (N/m ²)
3 floors	13.7	0.612	0.382	40	618

Table 6-9 Calculation of Peak Velocity Pressure

6.2.3.6 The Pressure Coefficient, c_{pe}

For external wall, the external pressure coefficients c_{pe} for buildings and parts of buildings depend on the size of the loaded area A , which is the area of the structure that produces the wind action in the section to be calculated. The external pressure coefficients are given for loaded areas A of 1 m^2 and 10 m^2 in the tables for the appropriate building configurations as $c_{pe,1}$, for local coefficients, and $c_{pe,10}$, for overall coefficients, respectively.

Since the pressure coefficients for vertical walls and flat roof vary through the wall and roof surface, the calculation is made considering geometry of the structure, the aspect ratio (h/d) and wind direction.

$$c_{pe} = c_{pe,1}, \text{ for } A \leq 1 \text{ m}^2$$

$$c_{pe} = c_{pe,1} - (c_{pe,10} - c_{pe,1}) \log_{10} A, \text{ for } 1 < A < 10 \text{ m}^2$$

$$c_{pe} = c_{pe,10}, \text{ for } A > 10 \text{ m}^2$$

The reference heights, z_e , for windward walls of rectangular plan buildings depend on the aspect ratio h/b . The parameters h and b for each kinds of velocity pressure profile is as follows:

The area is greater $A > 10 \text{ m}^2$ in all the cases hence the wind pressure on the surfaces is calculated from the peak velocity pressure $q_p(z)$, and external pressure coefficient to be used will be $c_{pe,10}$.

For wind in prevailing direction, $e = 2 \times 13.7 = 27.4 \text{ m}$, $e < d$, $h/d = 13.7/52 = 0.27$, so the c_{pe} for Zone A is -1.2, Zone B is -0.8, Zone C is -0.5, Zone D windward wall is 0.8, Zone E leeward wall is -0.5.

Elevation for $e < d$

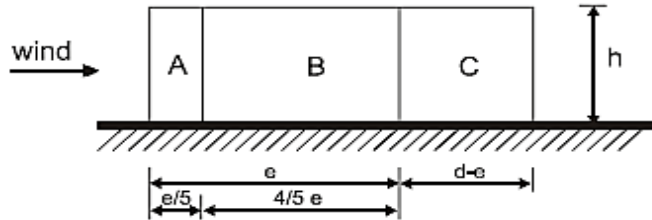


Figure 6-4 Key for vertical walls (According to Figure 7.5 in EN 1991-1-1-4)

Zone	A		B		C		D		E	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
h/d										
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
$\leq 0,25$	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

Table 6-15 Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings (According to Table 7.1 in EN1991-1-1-4)

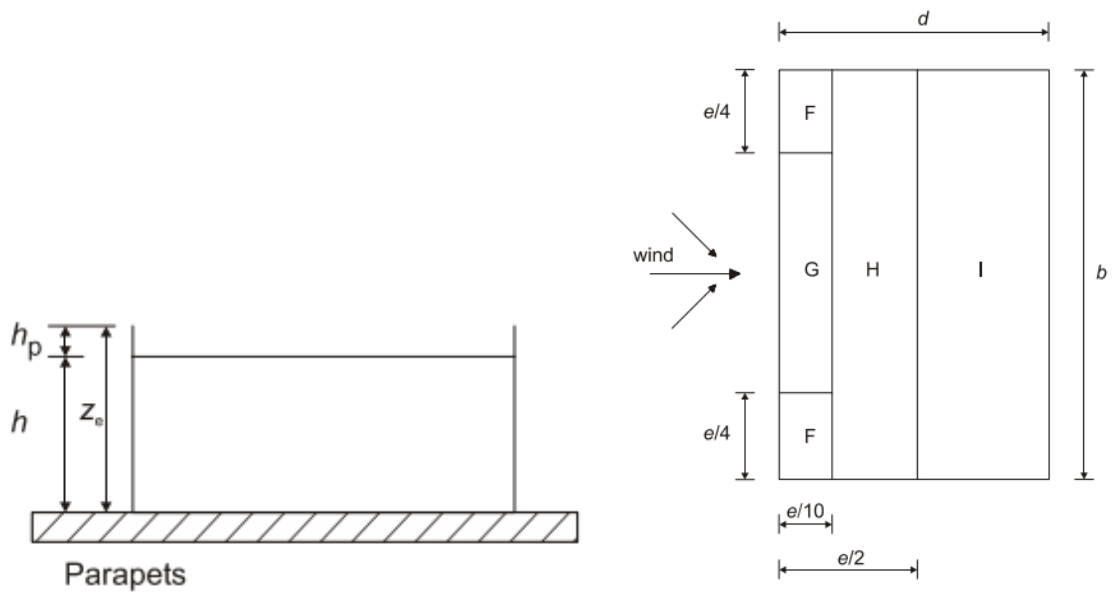


Figure 6-5 Key for Flat Roof (According to figure 7.6 in EN 1991-1-1-4)

Roof type		Zone							
		F		G		H		I	
		$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
Sharp eaves		-1,8	-2,5	-1,2	-2,0	-0,7	-1,2	+0,2	-0,2
With Parapets	$h_p/h=0,025$	-1,6	-2,2	-1,1	-1,8	-0,7	-1,2	+0,2	-0,2
		-1,4	-2,0	-0,9	-1,6	-0,7	-1,2	+0,2	-0,2
	$h_p/h=0,05$	-1,2	-1,8	-0,8	-1,4	-0,7	-1,2	+0,2	-0,2
		-1,2	-1,8	-0,8	-1,4	-0,7	-1,2	+0,2	-0,2

Table 6-11 External pressure coefficients for flat roofs (According to Table 7.2 in EN 1991-1-1-4)

For our building, parapet wall is 1.4m, so the c_{pe} for Zone F is -1.2, Zone G is -0.8, Zone H is -0.7, Zone I is 0.2 and -0.2.

6.2.3.7 Wind Load

The wind force, F_w acting on a structure or a structural element may be determined by vectorial summation of the forces acting on their reference surfaces:

$$F_w = c_s \times c_d \times \sum_i \times w_{ei} \times A_i$$

where

c_s c_d is the structural factor (separated into a size factor c_s and a dynamic factor c_d) is taken as 1,0 as recommended for framed buildings which have structural walls and which are less than 100 m high and whose height is less than 4 times the in-wind depth.

A_i is the reference area of the structure or structural element

In cases where the wind force on building structures is determined by application of the pressure coefficients c_{pe} on windward and leeward side (zones D and E) of the building simultaneously, the lack of correlation of wind pressures between the windward and leeward side may have to be taken into account. For buildings with $h/d \leq 1$, the resulting force is multiplied by 0,85.

Details of the calculation of the wind pressure and wind load at reference heights are reported in Table 6-11, accordingly to the position of zones on each façade shown in Figure 6-6.

Envelop	Zone	$q_p(z)$, (N/m ²)	$c_{pe,10}$	We (N/m ²)	Area (m ²)	F (kN)
Façade 1	A	618	-1.2	-742	75	-56
	B		-0.8	-495	300	-148
	C		-0.5	-309	186	-58
Façade 2	D	618	0.8	495	493	244
Façade 3	A	541	-1.2	-649	75	-49
	B		-0.8	-433	300	-130
	C		-0.5	-270	52	-14
	D	300	0.8	240	427	102
Façade 4	A	300	-1.2	-360	75	-27
	B		-0.8	-240	300	-72
	C		-0.5	-150	3	0.5
	E	541	-0.5	-270	378	-102
Façade 5	E	618	-0.5	-309	366	-113
Roof	F	618	-1.2	-742	44	-33
	G		-0.8	-495	66	-33
	H		-0.7	-433	448	-194
	I		0.2/-0.2	124	1165	144

Table 6-12 Wind Pressure and Wind Load on each facade

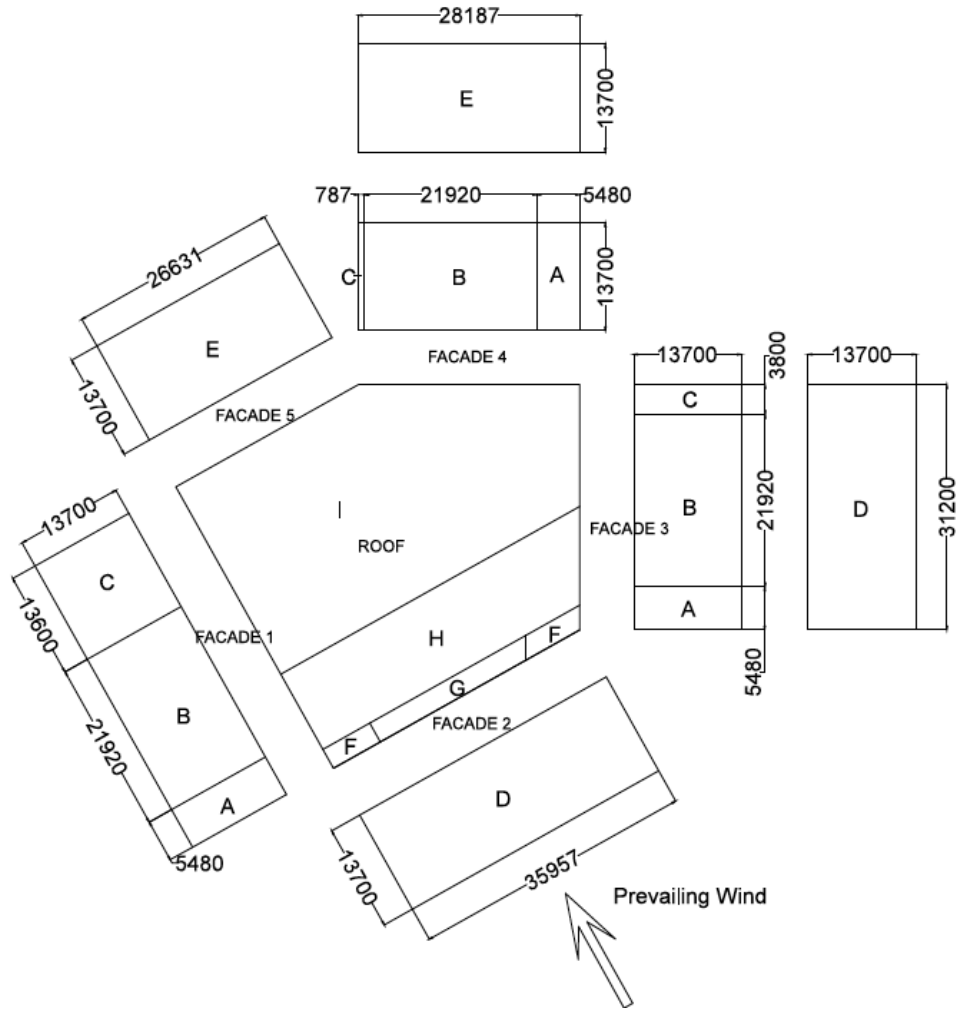


Figure 6-6 Layout of the facades and the zones position (dimension in mm)

6.3 East Wing Steel Beam-Column Frame System Verification

In this Section 6.3, verifying the steel beam-column frame on the east wing is carried out. A pin frame which is under the most unfavorable situation is checked.

For this frame, we checked the composite slab, secondary beam B1, primary beam B2, internal column C1, edge column C2, vertical wind bracing, the connection between secondary beam B1 with primary beam B2, the connection between primary beam B2 with internal column C1 and the base of column C1.

The frame we checked is shown in the Figure 6-7 below.

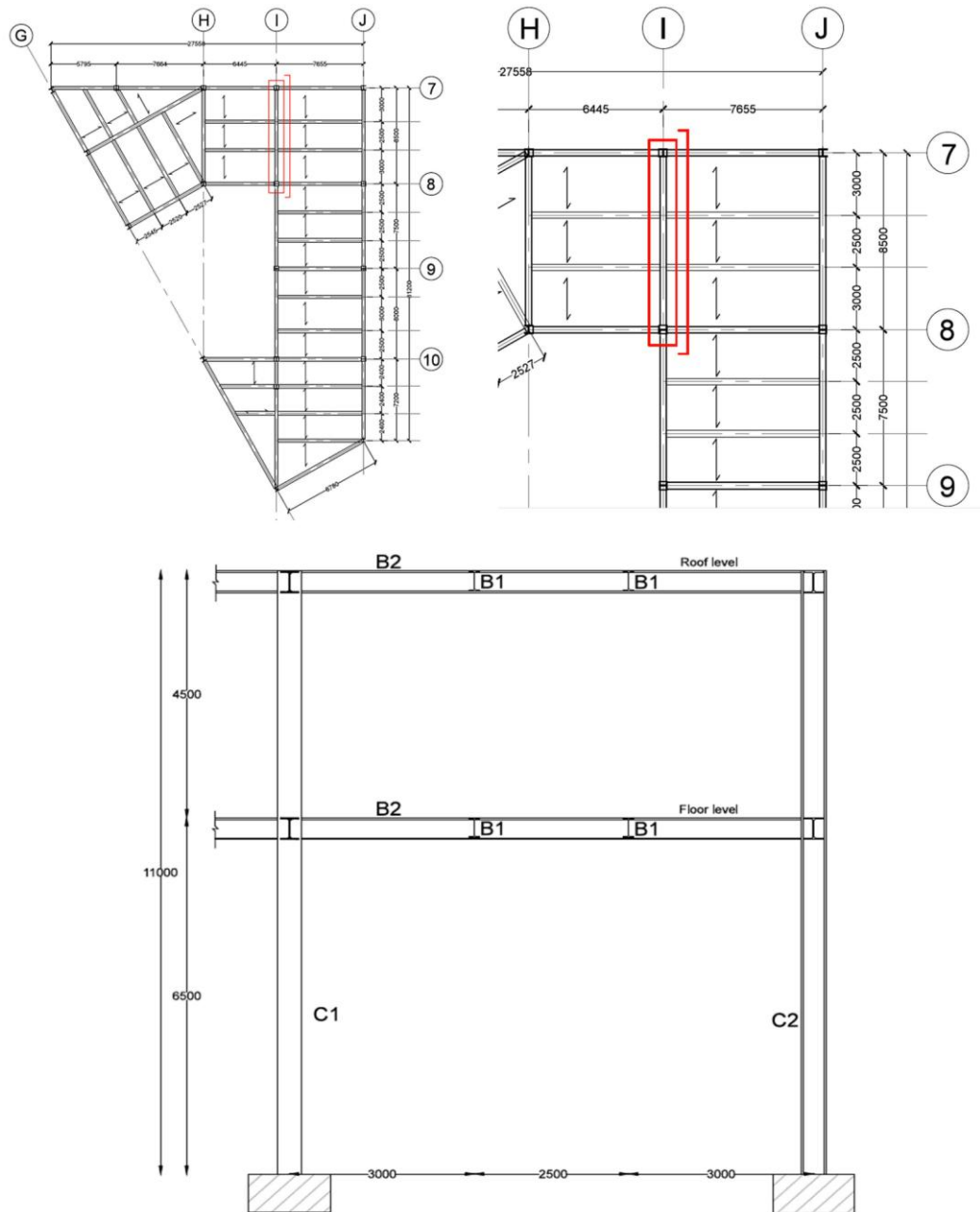


Figure 6-7 Frame under verification

6.3.1 Composite slab design

The slabs are supported by pairs of secondary beams or by one secondary beam and one edge primary beam. It means they are single direction slabs. Figure 6-8 demonstrates the organization of the composite slabs, and the slab with the maximum span of 3m that will be verified (highlighted in red).

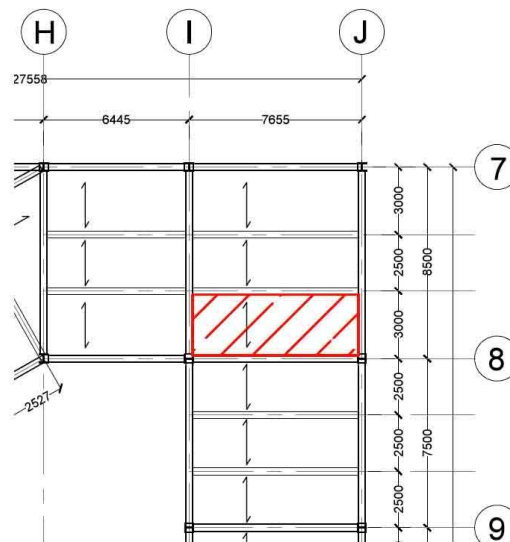
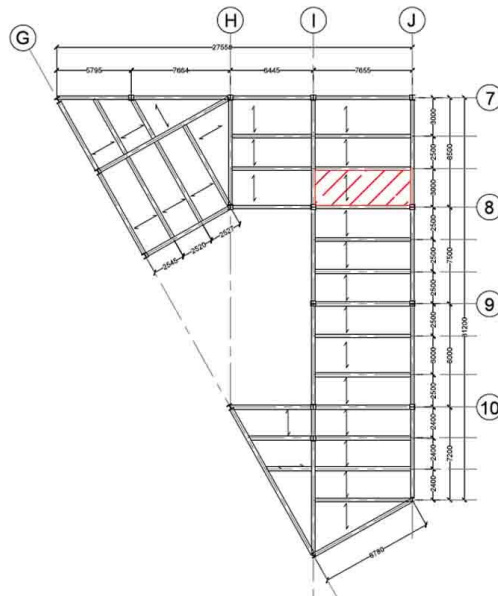


Figure 6-8 Organization of slabs (dimension in mm)

6.3.1.1 Proposed Floor Slab Properties

In our project, we choose ComFlor[®] 80, 150mm thick as floor and roof slab. Table 6-12 below shows the information of the deck.

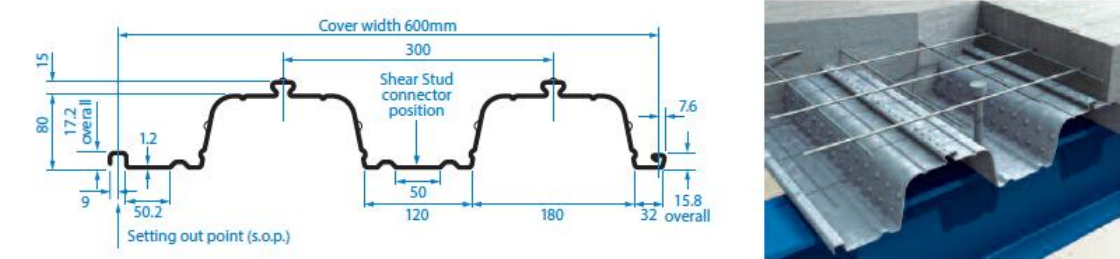
	Properties	Value
Composites slab	Span	$l = 3.0$ m (most unfavorable)
	Total depth of slab	$h = 150$ mm
	Thickness of Corus profiled steel sheeting CF80	$t = 1.2$ mm
	Depth of profile	$h_p = 80$ mm
	Effective cross-sectional area of the profile	$A_{pe} = 1871$ mm ² /m
	Second moment of area of the profile	$I_p = 245$ cm ⁴ /m
	Yield strength of the profiled deck	$f_{yp} = 350$ N/mm ²
	Height of neutral axis above soffit	47.6 mm
	Depth of concrete above profile	$h_c = 115$ mm
	Width of the bottom of the trough	$b_{bot} = 120$ mm
	Width of the top of the trough	$b_{top} = 170$ mm
	Design value of bending resistance (sagging)	$M_{Rd} = 22.2$ kNm/m
Shear connectors	Diameter	$d = 25.4$ mm
	Overall height before welding	$h_{sc} = 155.5$ mm
	Height after welding	150 mm
Concrete	Normal concrete strength class C25/30 density (reinforced)	24 kN/m ³ (wet) 23.5 kN/m ³ (dry)
	Cylinder strength	$f_{ck} = 25$ N/mm ²
	Secant modulus of elasticity	$E_{cm} = 31$ N/mm ²
		

Table 6-13 Data of ComFlor[®] 80 composite slab (From Tatasteel[®])

6.3.1.2 Loads

Table 6-13 below shows the calculation of the dead loads and imposed loads on composite slab during construction stage and composite stage.

Concrete weight				
	Unit volume m^3/m^2	Density kN/m^3	Weight kN/m^2	
Wet	0.106	24	2.51	
Dry	0.106	23.5	2.45	
Dead Loads				
	Construction stage kN/m^2		Composite stage kN/m^2	
	Steel deck	0.15	Concrete slab	2.45
			Steel deck	0.15
			Ceiling and covering	0.9
Total		0.15		3.5
Imposed Loads				
	Construction stage kN/m^2		Composite stage kN/m^2	
	Construction loading outside the working area	0.75	Imposed floor load for museum area	4
	Construction loading inside the working area	0.75		
	Concrete slab	2.51		
Total		4.01		4

Table 6-14 Loads on composite slab

In conclusion,

Dead loads at construction stage is $g_k=0.15\text{kN}/\text{m}^2$ while at composite stage is $g_k=3.5\text{kN}/\text{m}^2$

Imposed loads at construction stage is $q_k=4.01\text{kN}/\text{m}^2$ while at composite stage is $q_k=4\text{kN}/\text{m}^2$

6.3.1.3 Ultimate Limit State (ULS) verification

Table 6-14 shows the calculation of loads combination of the composite slab under Ultimate Limit State during construction stage and composite stage, as well as the bending and shearing reactions from the slab.

Loads combination at ULS			
Design value of combined actions $F_d = \xi Y_G g_k + Y_Q q_k$			
Partial factor		Construction stage kN/m^2	Composite stage kN/m^2
$Y_G = 1.35$	Dead loads	$g_k=0.15$	$g_k=3.5$
$Y_Q = 1.5$	Imposed loads	$q_k=4.01$	$q_k=4$
$\xi = 0.925$	Total	$F_d=6.2$	$F_d=10.37$
Reactions at ULS (per meter width of the steel deck)			
The design bending moment $M_{Ed} = \frac{F_d l^2}{8}$ The design shear force $V_{Ed} = \frac{F_d l}{2}$			
	Construction stage	Composite stage	
$M_{Ed} \text{ kNm/m}$	6.98	11.67	
$V_{Ed} \text{ kN/m}$	9.3	15.56	

Table 6-15 Loads combination and reactions from composite slab under ULS

In conclusion,

Design bending moment at construction stage is $M_{Ed}=6.98 \text{ kNm/m}$ while at composite stage is $M_{Ed}=11.67 \text{ kNm/m}$

Design shear force at construction stage is $V_{Ed}=9.3 \text{ kN/m}$ while at composite stage is $V_{Ed}=15.56 \text{ kN/m}$

Table 6-15 shows the calculation of resistance of steel deck and concrete slab

Partial factors for resistance	Design values of material strength	
Structural steel $Y_{MO} = 1.0$	Steel deck $f_{yp,d} = \frac{f_{yp}}{Y_{MO}}$	Concrete $f_{cd} = \alpha_{cc} \times f_{ck} / Y_C$
Concrete $Y_C = 1.5$		
$\alpha_{cc} = 0.85$	350 N/mm^2	14.2 N/mm^2
Reinforcement $Y_s = 1.15$		
Longitudinal shear $Y_{vs} = 1.25$		

Table 6-16 Resistance of material under ULS

- **Verification at the Construction Stage**

Bending resistance

$$\frac{M_{Ed}}{M_{Rd}} = \frac{6.98}{22.2} = 0.31 < 1.0$$

Where $M_{Rd} = 22.2$ kNm/m is given by Table 6-12

Therefore the bending moment resistance at the construction stage is satisfactory.

- **Verification at the Composite Stage**

Bending resistance – location of plastic neutral axis (pna)

The maximum compressive resistance per meter of the concrete above the sheeting assuming the pna is below the solid part of the slab is determined as:

$$N_c = f_{cd}A_c = 14.2 \times 80 \times 1000 \times 10^{-3} = 1136 \text{ kN/m}$$

Max tensile resistance per meter of the profiled steel sheet is determined as:

$$N_p = f_{yp,d}A_p = 350 \times 1871 \times 10^{-3} = 655 \text{ kN/m}$$

As $N_p < N_c$ the neutral axis lies above the profiled sheeting.

Therefore the sagging bending moment resistance should be determined from the stress distribution shown in the Figure 6-9.

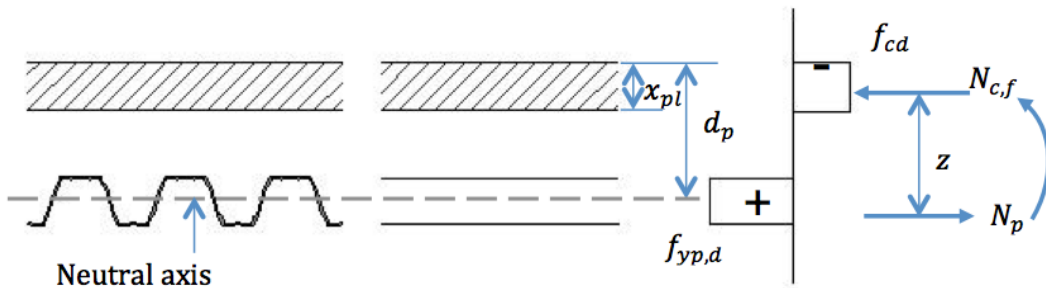


Figure 6-9 Reaction diagram of composite slab

The depth of concrete in compression is:

$$x_{pl} = \frac{A_p f_{yp,d}}{b f_{cd}}$$

Where:

b: is the width of the floor slab being considered, here $b = 1000$ mm

$$x_{pl} = \frac{1871 \times 350}{1000 \times 14.2} = 46.1 \text{ mm}$$

Bending resistance – full shear connection

For full shear connection, the design moment resistance is:

$$M_{pl,Rd} = A_p f_{yd} (d_p - x_{pl}/2)$$

$d_p = h$ - depth from soffit to neutral axis of sheeting

$$d_p = 150 - 47.6 = 102.4 \text{ mm}$$

The plastic bending resistance per meter width of the slab is:

$$M_{pl,Rd} = 1871 \times 350 \times (102.4 - 46.1/2) \times 10^{-6} = 52 \text{ kNm/m}$$

$$\frac{M_{Ed}}{M_{pl,Rd}} = \frac{11.67}{52} = 0.22 < 1.0$$

Therefore the bending moment resistance for full shear connection is satisfactory.

Longitudinal shear resistance: m-k method

$$V_{l,Rd} = \frac{bd_p}{\gamma_{vs}} \left(\frac{mA_p}{bl_s} + k \right)$$

m and k are design values obtained from the manufacturer. For the ComFlor80 steel deck the following values are obtained from the manufacturer.

$$m = 620 \text{ N/mm}^2$$

$$k = 0.19 \text{ N/mm}^2$$

For a uniform load applied to the whole span length;

$$l_s = \frac{l}{4} = \frac{3000}{4} = 750 \text{ mm}$$

$$V_{Rd} = \left[\frac{1000 \times 102.4}{1.25} \times \left(\frac{620 \times 1871}{1000 \times 750} + 0.19 \right) \right] \times 10^{-3} = 142.3 \text{ kN/m}$$

$$V_{Ed} = 15.56 \text{ kN/m}$$

The design shear resistance must not be less than the maximum design vertical shear.

$$\frac{V_{Ed}}{V_{Rd}} = \frac{15.56}{142.3} = 0.11 < 1.0$$

Therefore the design resistance to longitudinal shear is satisfactory.

Design vertical shear resistance

$$V_{v,Rd} = (v_{min} + k_q \sigma_{cp}) b_s d_p$$

Although in reality the slab is continuous, it is normally convenient to have it simply supported.

Therefore, the beneficial effect of the hogging moments at the supports is neglected,

such that $\sigma_{cp} = 0$. Hence

$$V_{v,Rd} = v_{min} b_s d_p$$

The recommended value of v_{min} is

$$v_{min} = 0.035 k^{3/2} f_{ck}^{1/2}$$

$$\text{Where } k = 1 + \sqrt{200/d_p} \leq 2.0$$

$$1 + \sqrt{200/102.4} = 2.40, \text{ so } k = 2.0$$

$$v_{\min} = 0.035 \times 2^{3/2} \times 25^{1/2} = 0.49 \text{ N/mm}^2$$

$$V_{v, Rd} = 0.49 \times 1 \times 102.4 = 50.2 \text{ N/m} > V_{Ed} = 15.56 \text{ kN/m}$$

Therefore the vertical shear resistance is satisfactory.

In conclusion, all design checks of the composite slab in the ultimate limit state are satisfactory.

6.3.1.4 Serviceability Limit State (SLS) verification

- **Construction Stage Deflections**

At serviceability, loading = 0.15 + 2.51 = 2.66 kN/m²

$$\bar{\delta}_s = \frac{5F_d l^4}{384EI} = \frac{5 \times 2.66 \times 3^4}{384 \times 210 \times 245 \times 10^6} \times 10^6 = 5.45 \text{ mm}$$

As this is less than 10% of the slab depth (15 mm), the effects of the additional concrete may be ignored in the design of the steel sheeting.

$$\bar{\delta}_{s, \max} = l/180 \text{ but } 20 \text{ mm max where the loads from ponding are ignored.}$$

$$\bar{\delta}_{s, \max} = 3000/180 = 16.7 \text{ mm, OK}$$

Composite Stage Deflections

At serviceability, loading = 3.5 + 4 = 7.5 kN/m²

$$\bar{\delta}_s = \frac{5F_d l^4}{384EI} = \frac{5 \times 7.5 \times 3^4}{384 \times 210 \times 245 \times 10^6} \times 10^6 = 15.4 \text{ mm}$$

$$\bar{\delta}_{s, \max} = l/180 \text{ but } 20 \text{ mm max where the loads from ponding are ignored.}$$

$$\bar{\delta}_{s, \max} = 3000/180 = 16.7 \text{ mm}$$

Hence the result is satisfactory.

6.3.2 Secondary beam design

The secondary beam (B1) shown in Figure 6-10 is simply supported at both ends and is fully restrained along its length.

For the loading shown, design the beam in S275.

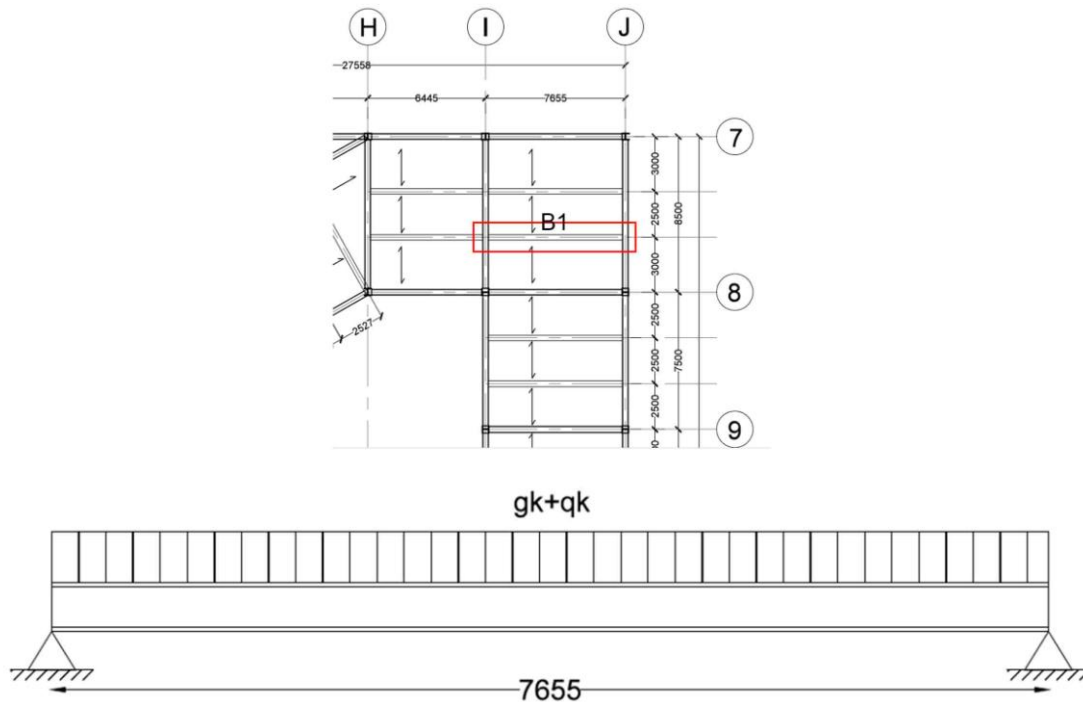


Figure 6-10 Beam B1 fully restrained along length (dimension in mm)

6.3.2.1 Loads

Characteristic values of loads:

Dead load: $g_k = 3.5 \text{ kN/m}^2 \times 3 \text{ m} = 10.5 \text{ kN/m}$

Imposed load: $q_k = 4 \text{ kN/m}^2 \times 3 \text{ m} = 12 \text{ kN/m}$

Design values $F_d = \xi \gamma_G g_k + \gamma_Q q_k = 0.925 \times 1.35 \times 10.5 + 1.5 \times 12 = 31.1 \text{ kN/m}$

Reactions:

$$\text{Design moment } M_{Ed} = \frac{F_d l^2}{8} = \frac{31.1 \times 7.655^2}{8} = 227.8 \text{ kNm}$$

$$\text{Design shear force } V_{Ed} = \frac{F_d l}{2} = \frac{31.1 \times 7.655}{2} = 119 \text{ kN}$$

6.3.2.2 Proposed beam section selection

To determine the section size, assume that the flange thickness is less than 40mm so that the design strength is 275 N/mm², and that the section is class 1 or 2.

$$M_{Sd} \leq M_{c,Rd}$$

Hence, for class 2 section, $M_{c,Rd} = M_{y,Ed} = W_{pl,y} f_y / \gamma_{M0}$

$$W_{pl,y} \geq \frac{M_{y,Ed} \gamma_{M0}}{f_y} = \frac{227.8 \times 10^3 \times 1.0}{275} = 829 \text{ cm}^3$$

Try section IPE360, $W_{pl,y} = 1020 \text{ cm}^3$

IPE360			
Geometry			
h = 360 mm		Section properties	
b = 170 mm		Axis y	Axis z
t _f = 12.7 mm		I _y = 1.63E+8 mm ⁴	I _z = 1.04E+7 mm ⁴
t _w = 8 mm		W _{y1} = 9.04E+5 mm ³	W _{z1} = 1.23E+5 mm ³
r ₁ = 18 mm		W _{y,pl} = 1.02E+6 mm ³	W _{z,pl} = 1.91E+5 mm ³
y _s = 85 mm		i _y = 150 mm	i _z = 37.9 mm
d = 298.6 mm		S _y = 5.10E+5 mm ³	S _z = 9.55E+4 mm ³
A = 7270 mm ²		Warping and buckling	
A _L = 1.35 m ² .m ⁻¹		I _w = 3.14E+11 mm ⁶	I _t = 3.80E+5 mm ⁴
G = 57.1 kg.m ⁻¹		i _w = 42.6 mm	i _{pc} = 154 mm

Table 6-17 section properties

Source: http://www.statictools.eu/profil_IPE.php?profil=IPE360&act=zobraz&je=0&lang=EN

Classification of cross-section

As a simply supported beam is not required to have any plastic rotation capacity (only one hinge required), it is sufficient to ensure that the section is at least class 2 to develop the plastic moment resistance.

Flange buckling, $y_s/t_f \leq 11\epsilon$

Where

y_s : half the width of the flange

t_f : flange thickness

$$\epsilon = \sqrt{(235/f_y)} = \sqrt{(235/275)} = 0.924$$

For this section the limit is $11\epsilon = 0.924 \times 11 = 10.2$

$$y_s/t_f = 85/12.7 = 6.69 < 11\epsilon = 10.2$$

Web bulking, $d/t_w \leq 83\epsilon = 0.924 \times 83 = 76.7$

Where

d: the depth between roots

t_w : the web thickness

$$d/t_w = 298.6/8 = 37.33 < 83\epsilon = 76.7$$

Hence, the section is at least class 2

- **Ultimate Limit State (ULS) verification**

Shear on web

The shear resistance of the web must be checked.

$$V_{Sd} \leq V_{pl,Rd}$$

The design plastic shear resistance of the web is given by:

$$V_{pl,Rd} = A_v \frac{f_y / \sqrt{3}}{\gamma_{M0}}$$

For rolled I and H section loaded parallel to the web,

$$\text{Shear area } A_v = 1.04 h t_w$$

And the partial safety factor $\gamma_{M0} = 1.05$

$$\text{Hence } V_{pl,Rd} = \frac{1.04 h t_w f_y / \sqrt{3}}{\gamma_{M0}} = \frac{1.04 \times 360 \times 8 \times 275}{\sqrt{3} \times 1.05 \times 10^3} = 453 \text{ kN}$$

This is greater than the shear on the section $V_{Ed} = 119 \text{ kN}$

As this beam has partial depth end-plates, a local shear check is required on the web of the beam where it is welded to the end-plate.

$$V_{pl,Rd} = A_v \frac{f_y / \sqrt{3}}{\gamma_{M0}}$$

Where

$$A_v = t_w d$$

d: depth of end-plate = 250mm

$$\text{Hence } V_{pl,Rd} = \frac{8 \times 250 \times 275}{\sqrt{3} \times 1.05 \times 10^3} = 302 \text{ kN}$$

This is greater than the shear on the section $V_{Ed} = 119 \text{ kN}$

- **Serviceability Limit State (SLS) verification**

Eurocode 3 requires that the deflection of the beam be checked under the following serviceability loading conditions:

- 1) Imposed loads
- 2) Dead loads + Imposed loads

Figure 6-11 below shows the vertical deflections to be considered.

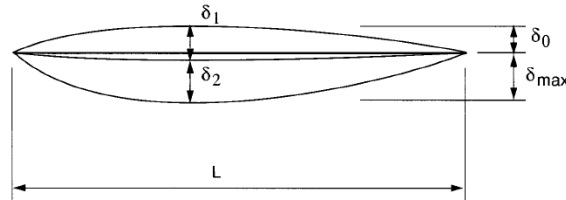


Figure 6-11 Vertical deflections

$\bar{\delta}_0$: the precamber

$\bar{\delta}_1$: the deflection due to dead loads

$\bar{\delta}_2$: the deflection due to imposed loads

$\bar{\delta}_{max}$: the total deflection caused by dead loads and imposed loads without precamber

Deflection checks are based on the serviceability loading

For a uniform load

$$\bar{\delta} = \frac{5}{384} \times \frac{F_k L^3}{EI_y}$$

Where

F_k : the total load = q_k or $(q_k + g_k)$ as appropriate

L: the span, 7.655m

E: the modulus of elasticity (210000 N/mm²)

I_y : the second moment of area about the major axis (y-y)

For unit load of 1 kN/m

$$\bar{\delta} = \frac{5}{384} \times \frac{F_k L^3}{EI_y} = \frac{5}{384} \times \frac{10^3 \times 7.655 \times 7655^3}{210000 \times 16300 \times 10^4} = 1.3 \text{ mm}$$

The deflection limits are L/250 for $\bar{\delta}_{max}$, and L/350 for $\bar{\delta}_2$.

The calculated deflections shown in Table 6-17 are less than the limits, so no precamber is required.

	Calculated deflection (mm)	Deflection limit (mm)
$\bar{\delta}_1$ dead loads	$1.3 \times 10.5 = 13.65$	
$\bar{\delta}_2$ imposed loads	$1.3 \times 12 = 15.6$	$L/350 = 21.9$
$\bar{\delta}_{max}$	29.25	$L/250 = 30.6$

Table 6-18 Calculated and limiting deflections

6.3.3 Primary beam design

The primary beam (B2) shown in figure 6-12 is laterally restrained at the ends and at the points of application of load.

The beam is designed in S275.

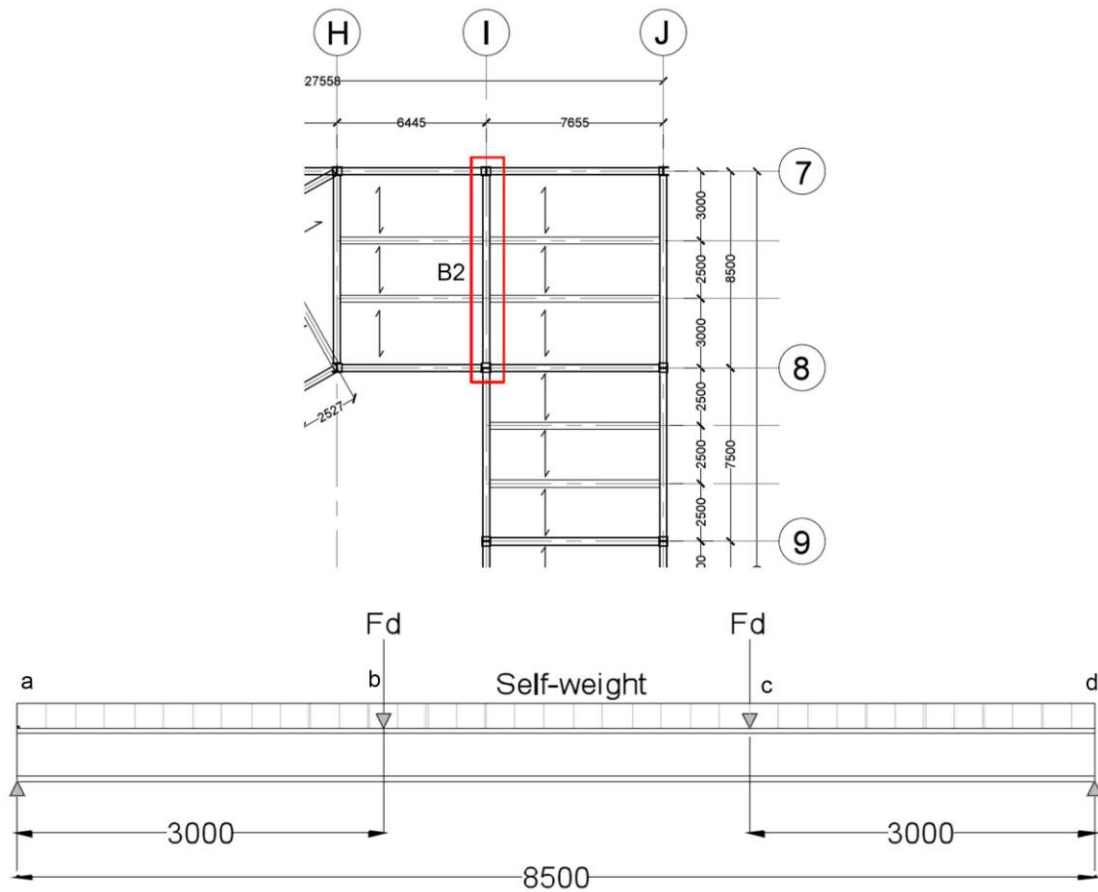


Figure 6-12 Beam B2 restrained at load points (dimension in mm)

6.3.3.1 Loads

The point loads are taken as the end reactions from beams B1.

Permanent action at point Dead load:

$$g_{k,1} = 3.5 \text{ kN/m}^2 \times (3+2.5) \text{ m} / 2 \times (7.655+6.445) \text{ m} / 2 = 73.7 \text{ kN}$$

Variable action at point Imposed load:

$$q_{k,1} = 4 \text{ kN/m}^2 \times (3+2.5) \text{ m} / 2 \times (7.655+6.445) \text{ m} / 2 = 84.2 \text{ kN}$$

For self-weight of beam B2 and casting, $g_{k,2} = 155 \text{ kg/m} \times 9.8 \text{ N/kg} \times 8.5 \text{ m} = 13 \text{ kN}$

Point loads

$$\text{Design values } F_{d1} = \xi \gamma_G g_k + \gamma_Q q_k = 0.925 \times 1.35 \times 73.7 + 1.5 \times 84.2 = 218 \text{ kN}$$

Self-weight

The self-weight of the beam and casting is assumed uniformly distributed along the full length of the beam.

$$F_{d2} = 0.925 \times 1.35 \times 13 = 16 \text{ kN}$$

Reactions:

Design shear force

$$V_{Sd} = F_{d1} + F_{d2} / 2 = 218 + 16 / 2 = 226 \text{ kN (at the supports)}$$

Design moment

Moment at mid-span (maximum)

$$M_{Sd} = 218 \times 3 + 16 \times 8.5 / 8 = 671 \text{ kNm}$$

Moment at load point

$$M_{Sd} = 226 \times 3 + 16 / 8.5 \times 3^2 / 2 = 662.5 \text{ kNm}$$

6.3.3.2 Proposed beam section selection

We assume that a rolled universal beam will be used and that the flanges will be less than 40mm thick. For grade S275 steel, $f_y=275\text{N/mm}^2$. Because the beam is unrestrained between the point loads, the design resistance ($M_{c,Rd}$) of the section will be reduced by lateral torsional buckling. The final section, allowing for the buckling resistance moment being less than the full resistance moment of the section, would have to be determined from experience. Here, the bending strength (f_b) can be assumed to be about 240N/mm , for preliminary sizing.

The plastic modulus required, $W_{pl} = 671 \times 10^3 / 240 = 2796 \text{ cm}^3$

Try section HEB360, $W_{pl,y} = 3240 \text{ cm}^3$

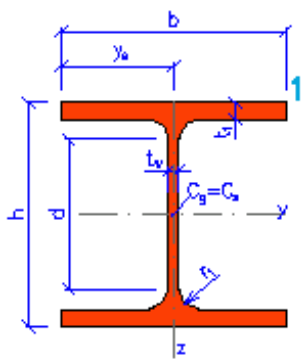
HE400B			
Geometry			
h = 400 mm		Section properties	
b = 300 mm		Axis y	Axis z
$t_f = 24 \text{ mm}$		$I_y = 5.77\text{E}+8 \text{ mm}^4$	$I_z = 1.08\text{E}+8 \text{ mm}^4$
$t_w = 13.5 \text{ mm}$		$W_{y1} = 2.88\text{E}+6 \text{ mm}^3$	$W_{z1} = 7.21\text{E}+5 \text{ mm}^3$
$r_1 = 27 \text{ mm}$		$W_{y,pl} = 3.24\text{E}+6 \text{ mm}^3$	$W_{z,pl} = 1.10\text{E}+6 \text{ mm}^3$
$y_s = 150 \text{ mm}$		$i_y = 171 \text{ mm}$	$i_z = 74 \text{ mm}$
d = 298 mm		$S_y = 1.62\text{E}+6 \text{ mm}^3$	$S_z = 5.50\text{E}+5 \text{ mm}^3$
$A = 1.98\text{E}+4 \text{ mm}^2$		Warping and buckling	
$A_L = 1.93 \text{ m}^2 \cdot \text{m}^{-1}$		$I_w = 3.82\text{E}+12 \text{ mm}^6$	$I_t = 3.60\text{E}+6 \text{ mm}^4$
$G = 155 \text{ kg} \cdot \text{m}^{-1}$		$i_w = 74.7 \text{ mm}$	$i_{pc} = 186 \text{ mm}$

Table 6-19 HEB400 section properties

Source: http://www.statictools.eu/profil_HEB.php?profil=HE400B&act=zobraz&lang=EN&je=0

- **Classification of cross-section**

As a simply supported beam is not required to have any plastic rotation capacity (only one hinge required), it is sufficient to ensure that the section is at least class 2 to develop the plastic moment resistance.

Flange buckling, $y_s/t_f \leq 11\epsilon$

Where

y_s : half the width of the flange

t_f : flange thickness

$$\epsilon = \sqrt{(235/f_y)} = \sqrt{(235/275)} = 0.924$$

For this section the limit is $11\epsilon = 0.924 \times 11 = 10.2$

$$y_s / t_f = 150/24 = 6.25 < 11\epsilon = 10.2$$

Web bulking, $d/t_w \leq 83\epsilon = 0.924 \times 83 = 76.7$

Where

d : the depth between roots

t_w : the web thickness

$$d/t_w = 298/13.5 = 22 < 83\epsilon = 76.7$$

Hence, the section is at least class 2

6.3.3.3 Ultimate Limit State (ULS) verification

- **Design buckling resistance moment**

The design buckling resistance moment of a laterally unrestrained beam is given by the following equation:

$$M_{b,Rd} = \chi_{LT} \beta_W W_{pl,y} f_y / \gamma_{M1}$$

Where χ_{LT} is the reduction factor for lateral-torsional buckling.

In this case, the central segment b-c is subjected to uniform moment, which is the most severe condition, so only b-c is checked.

For segment b-c

The value of λ_{LT} can be determined using

$$\lambda_{LT} = \frac{L/i_{LT}}{(C_1)^{0.5} \left[1 + \frac{(L/i_{LT})^2}{25.66} \right]^{0.25}}$$

Where

L : the length between b and c

I_z : the second moment of area about the z-z axis = 10800 cm^4

I_w : the warping constant = 3.82 dm^6

$W_{pl,y}$: the plastic modulus about the y-y axis = 3240 cm^3

I_t : the torsion constant = 360cm^4

C_1 : the correction factor for the effects of any change of moment along the length L

Between the point b and point c the moment is approximately constant, therefore C_1 may be taken as 1.0

$$\alpha_{LT} = (I_w / I_t)^{0.5} = (3.82 \times 10^6 / 360)^{0.5} = 103\text{cm}$$

$$i_{LT} = (I_z I_w / W_{pl,y}^2)^{0.25} = (10800 \times 3.82 \times 10^6 / 3240^2)^{0.25} = 7.9\text{cm}$$

$$\lambda_{LT} = \frac{L/i_{LT}}{(C_1)^{0.5} \left[1 + \frac{(L/\alpha_{LT})^2}{25.66}\right]^{0.25}} = \frac{250/7.9}{\left[1 + \frac{(250/103)^2}{25.66}\right]^{0.25}} = 30.1$$

The non-dimensional slenderness is given by:

$$\bar{\lambda}_{LT} = (\lambda_{LT} / \lambda_1) (\beta_W)^{0.5}$$

Where

$$\lambda_1 = 93.9\epsilon = 93.9 \times \sqrt{(235/f_y)} = 86.8$$

$\beta_W = 1$ for class 1 or class 2 sections

$$\bar{\lambda}_{LT} = 30.1/86.8 = 0.35$$

For rolled I section, buckling curve a, shown in the figure 6-13 should be used.

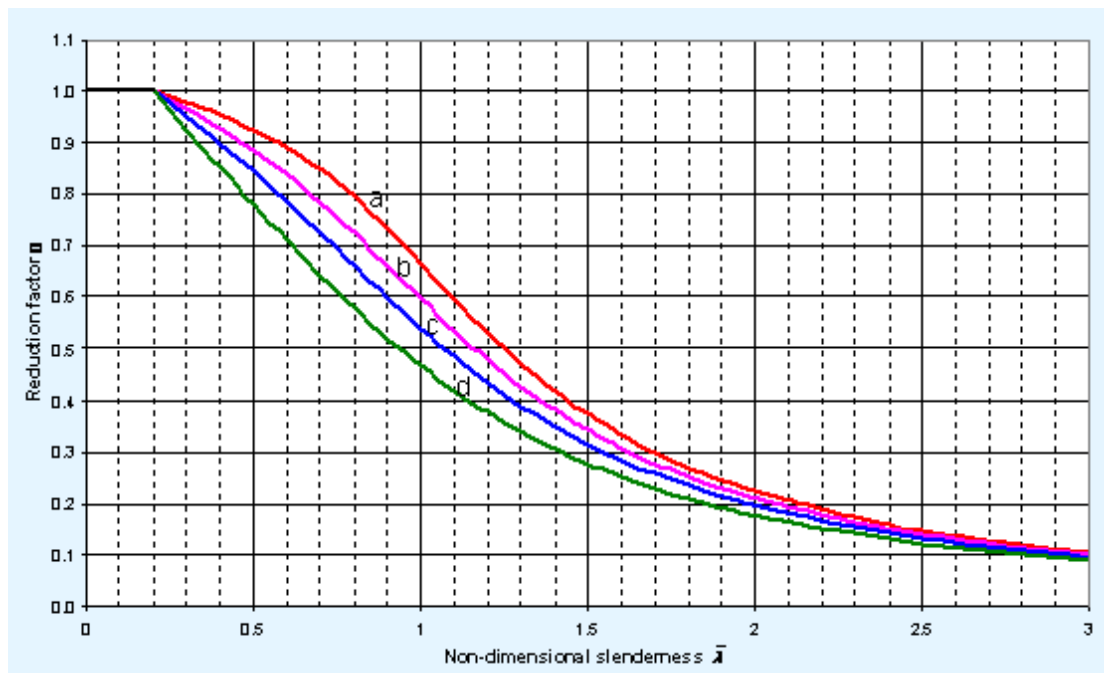


Figure 6-13 Buckling curves for χ (According to EN 1993-1-1, Table 6.4)

Hence we can get $\chi_{LT} = 0.97$

The design buckling resistance moment for segment b-c is:

$$M_{b,Rd} = \chi_{LT} \beta_W W_{pl,y} f_y / \gamma_{M1} = 0.97 \times 1.0 \times 3240 \times 275 / 1.05 / 10^3 = 823\text{kNm}$$

This is greater than the design moment (671 kNm) between b and c point. So it is satisfied.

- **Shear on web**

In all cases where there point loads on members it is prudent to check for the effect of shear. The following check should be carried out:

Shear at point loads, $V_{sd} = 226 - 16/8.5 \times 3 = 220.4$ kN

The design shear resistance for a rolled I section is:

$$V_{pl,Rd} = \frac{1.04ht_w(f_y/\sqrt{3})}{Y_{M0}}$$

$$V_{pl,Rd} = \frac{1.04 \times 400 \times 13.5(275/\sqrt{3})}{1.05 \times 10^3} = 849 \text{ kN}$$

$V_{sd} = 226$ kN (at the supports) $< \frac{V_{pl,Rd}}{2}$, so no reduction in moment resistance due to shear in the web is necessary.

6.3.3.4 Serviceability Limit State (SLS) verification

- **Deflection check**

In this case self-weight deflection is small and may be neglected. The point-load deflection can be considered by calculation the deflection from unit loads and then multiplying by the applied loads. Note that the serviceability loads are used for deflection checks.

For two points loads on a beam the maximum deflection is given by:

$$\delta = \frac{F_k a}{24EI_y} (3L^2 - 4a^2)$$

Where

F_k : the value of one point load

L : the span

a : the distance from the support to the adjacent point load

For this beam the unit load deflection is:

$$\delta = \frac{1 \times 3000}{24 \times 210000 \times 57700 \times 10^4} (3 \times 8500^2 - 4 \times 3000^2) = 1.865 \times 10^{-4} \text{ mm}$$

	Calculated deflection (mm)	Deflection limit (mm)
δ_1 dead loads	$1.865 \times 10^{-4} \times 73.7 \times 10^3 = 13.74$	
δ_2 imposed loads	$1.865 \times 10^{-4} \times 84.2 \times 10^3 = 15.7$	$L/350 = 24.3$
δ_{max}	29.44	$L/250 = 34$

Table 6-20 Calculated and limiting deflections

Both limits are greater than the sum of the deflections, so the result is satisfactory.

6.3.4 Internal column design

The internal column (C1) is shown in figure 6-14. It bears the loads from a roof and one floor and designed in S275.

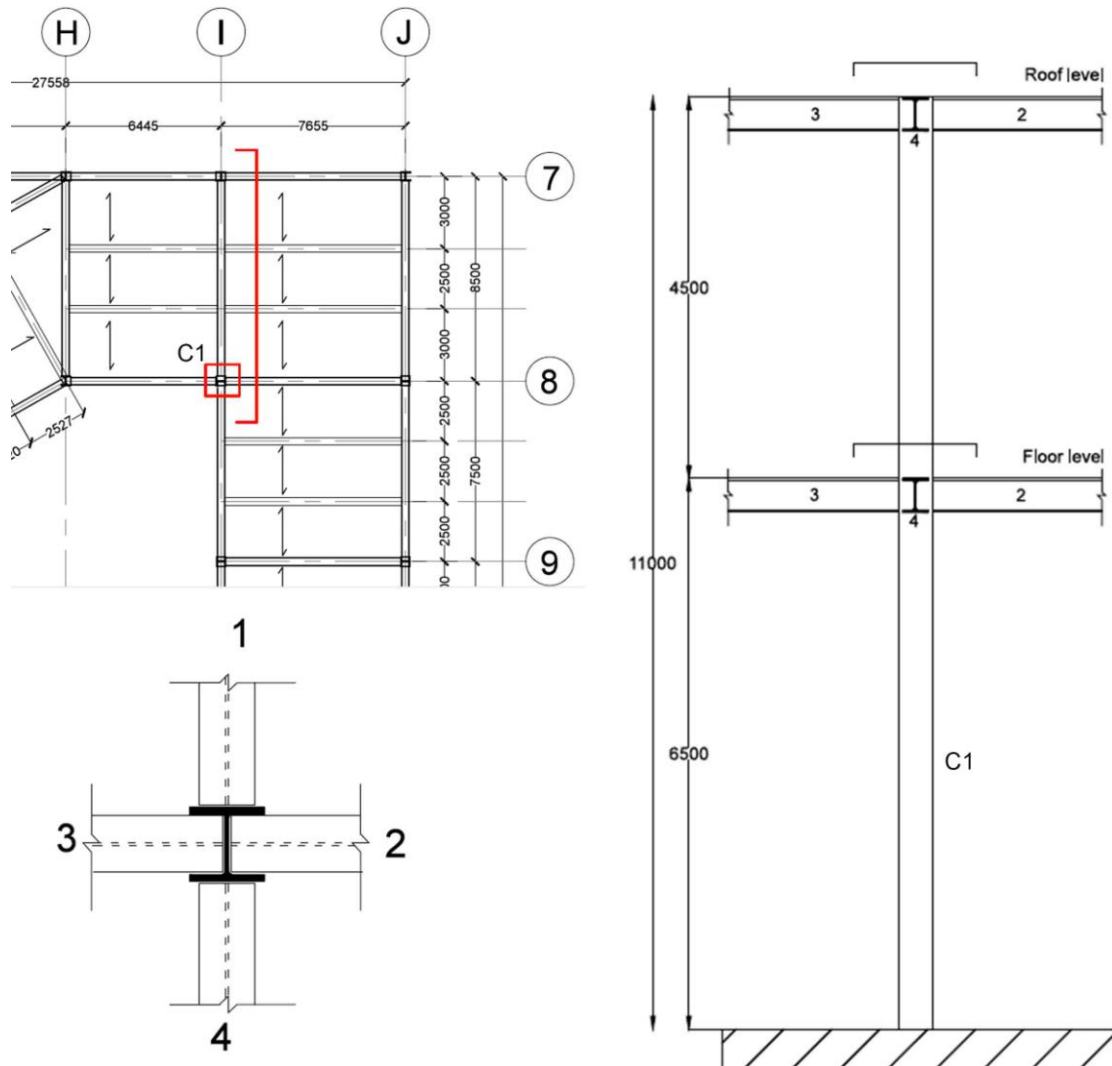


Figure 6-14 Internal Column C1 (dimension in mm)

6.3.4.1 Loads

Here is the loading calculation of the column

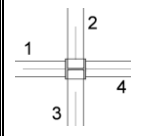
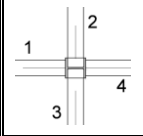
	Beam	Dead load	Imposed load	Design axial compression
	1	33.8	14.6	64.1
	2	73.7	27.6	133.4
	3	70	24.8	124.6
	4	73.7	26.9	132.4
Total		251.2	93.9	454.5
	1	33.8	38.7	100.3
	2	73.7	90.7	227.8
	3	70	82.3	210.9
	4	73.7	90	227.2
Total		251.2	301.7	766.2
Total		502.4	395.6	1220.7

Table 6-21 Loading for Column C1 (kN)

Design values of axial compression

$$N_{sd} = \xi Y_G g_k + Y_Q q_k = 0.925 \times 1.35 \times 502.4 + 1.5 \times 395.6 = 1220.7 \text{ kN}$$

Take into consideration of the effect of unbalanced loading from beam 1 and beam 4 on the column, for these two create the most critical situation.

Eccentricity is to be taken as 100mm from face of column.

$$e = h/2 + 100 \text{ mm} = 200 + 100 = 300 \text{ mm (we choose section HD400} \times 287)$$

Design moment:

At floor level:

$$M_{f,y,Ed} = 300 \times (227.2 - 100.3) = 38 \text{ kNm}$$

The moment is distributed between the column lengths in proportion to their bending stiffness (EI/L), unless the ratio of the stiffness does not exceed 1.5, in which case the moment is divided equally. As the ratio of the column stiffness is less than 1.5, the design moment is therefore:

$$M_{y,Ed} = 38 \times 0.5 = 19 \text{ kNm}$$

6.3.4.2 Proposed beam section selection

Try section HD400 × 287, $W_{pl,y} = 5810 \text{ cm}^3$

HD400x287			
Geometry			
h = 393 mm		Section properties	
b = 399 mm		Axis y	Axis z
$t_f = 36.6 \text{ mm}$		$I_y = 9.97\text{E}+8 \text{ mm}^4$	$I_z = 3.88\text{E}+8 \text{ mm}^4$
$t_w = 22.6 \text{ mm}$		$W_{y1} = 5.07\text{E}+6 \text{ mm}^3$	$W_{z1} = 1.94\text{E}+6 \text{ mm}^3$
$r_1 = 15 \text{ mm}$		$W_{y,pl} = 5.81\text{E}+6 \text{ mm}^3$	$W_{z,pl} = 2.96\text{E}+6 \text{ mm}^3$
$y_s = 199.5 \text{ mm}$		$i_y = 165 \text{ mm}$	$i_z = 102.9 \text{ mm}$
d = 290 mm		$S_y = 2.91\text{E}+6 \text{ mm}^3$	$S_z = 1.48\text{E}+6 \text{ mm}^3$
$A = 3.66\text{E}+4 \text{ mm}^2$		Warping and buckling	
$A_L = 2.31 \text{ m}^2 \cdot \text{m}^{-1}$		$I_w = 1.23\text{E}+13 \text{ mm}^6$	$I_t = 1.46\text{E}+7 \text{ mm}^4$
$G = 287 \text{ kg} \cdot \text{m}^{-1}$		$i_w = 94.24 \text{ mm}$	$i_{pc} = 194.4 \text{ mm}$

Table 6-22 HD400 × 287 section properties

Source: http://www.staticstools.eu/profil_HD.php?tlac&profil=HD400x287&lang=EN&je=0

- **Classification of cross-section**

This section is design to withstand small moments in addition to axial force.

Flange (under compression), the limiting value of c/t_f for class1 of a rolled section is $y_s/t_f \leq 10\epsilon$

Where

y_s : half the width of the flange

t_f : flange thickness

$$\epsilon = \sqrt{(235/f_y)} = \sqrt{(235/275)} = 0.924$$

$$y_s/t_f = 199.5/36.6=5.45 < 10\epsilon =9.24$$

Web (under bending and compression), the limiting value of d/t_w for class1 of a rolled section is $d/t_w \leq 33\epsilon = 0.924 \times 33 =30.5$

Where

d: the depth between roots

t_w : the web thickness

$$d/t_w = 290/22.6=12.8 < 33\epsilon =30.5$$

hence, the section is class 1

6.3.4.3 Ultimate Limit State (ULS) verification

- **Resistance of cross-section**

For a class 1 section without bolt holes, the reduced design plastic resistance moment, allowing for axial force, is:

$$M_{N_y,Rd} = \frac{M_{ply,Rd}(1-n)}{1-0.5a}$$

Where

$$n = N_{sd}/N_{pl,Rd}$$

$$a = (A-2bt_f)/A \quad \text{but } a \leq 0.5$$

To calculate $M_{ply,Rd}$

For bending about one axis, the design moment of resistance is

$$M_{ply,Rd} = W_{pl,y} f_y / \gamma_{M0} = 5810 \times 10^3 \times 275 / 1.05 / 10^6 = 1521 \text{ kNm}$$

To calculate n and a

Applied axial force, $N_{sd} = 1220.7 \text{ kN}$

For a member under axial compression, the design plastic resistance of the cross-section is:

$$N_{ply,Rd} = A f_y / \gamma_{M0} = 36600 \times 275 / 1.05 / 10^3 = 9585 \text{ kN} > N_{sd} = 1220.7 \text{ kN}$$

$$n = 1220.7 / 9585 = 0.13$$

$$a = 0.2$$

$$M_{N_y,Rd} = \frac{M_{ply,Rd}(1-n)}{1-0.5a} = 1521 \times (1-0.13) / (1-0.5 \times 0.2) = 1191 \text{ kNm} > M_{y,Ed} = 19 \text{ kNm}$$

Hence the result is satisfactory.

6.3.4.4 Serviceability Limit State (SLS) verification

- **Flexural buckling**

A class 1 member under moment about the major axis only, should satisfy the following:

$$\frac{N_{Sd}}{\chi_{\min} A f_y / \gamma_{M1}} + \frac{k_y M_{y,Sd}}{W_{pl,y} f_y / \gamma_{M1}} \leq 1$$

χ_{\min} is the lesser of χ_y and χ_z , the reduction factors for y-y and z-z axes respectively.

Assuming that the connection between the column and the primary beam are effectively pinned, then the slenderness about the y-y axis is:

$$\lambda_y = l / i_y = 6500 / 165 = 39.4$$

$$\lambda_1 = \pi (E / f_y)^{0.5} = 93.9 \epsilon = 86.8$$

$$\bar{\lambda}_y = \lambda_y / \lambda_1 \beta_A^{0.5} = 0.45$$

Use buckling curve b (Figure 6-13)

$$\chi_y = 0.922$$

$$\lambda_z = l / i_z = 6500 / 102.9 = 63.2$$

$$\lambda_1 = \pi (E / f_y)^{0.5} = 93.9 \epsilon = 86.8$$

$$\bar{\lambda}_z = \lambda_z / \lambda_1 \beta_A^{0.5} = 0.73$$

Use buckling curve c (Figure 6-13)

$$\chi_{\min} = \chi_z = 0.718$$

$$k_y = 1 - \frac{\mu_y N_{Sd}}{\chi_y A f_y} \text{ but } k_y \leq 1.5$$

$$\text{Where } \mu_y = \bar{\lambda}_y (2\beta_{My} - 4) + \frac{W_{pl,y} - W_{el,y}}{W_{el,y}}$$

$$\beta_{My} = 1.1 \mu_y = 0.45 (2 \times 1.1 - 4) + \frac{5810 - 5070}{5070} = -0.65$$

$$k_y = 1 - \frac{-0.65 \times 1220.1 \times 10^3}{0.922 \times 36600 \times 275} = 1.1 < 1.5$$

$$\frac{N_{Sd}}{\chi_{\min} A f_y / \gamma_{M1}} + \frac{k_y M_{y,Sd}}{W_{pl,y} f_y / \gamma_{M1}} = \frac{1220.1 \times 10^3}{0.718 \times 36600 \times 275 / 1.05} + \frac{1.1 \times 19 \times 10^6}{5810 \times 10^3 \times 275 / 1.05} \leq 1$$

Hence the result is satisfactory for flexural buckling.

Lateral torsional buckling

A class 1 section should satisfy the following:

$$\frac{N_{Sd}}{\chi_z A f_y / \gamma_{M1}} + \frac{k_{LT} M_{y,Sd}}{\chi_{LT} W_{pl,y} f_y / \gamma_{M1}} \leq 1$$

$$k_{LT} = 1 - \frac{\mu_{LT} N_{Sd}}{\chi_z A f_y} \text{ but } k_{LT} \leq 1$$

$$\text{Where } \mu_{LT} = 0.15 \bar{\lambda}_z \beta_{M,LT} - 0.15$$

$$\beta_{M,LT} = 1.1$$

$$\mu_{LT} = 0.15 \times 0.73 \times 1.1 - 0.15 = -0.03$$

$$k_{LT} = 1 - \frac{-0.03 \times 1220.1 \times 10^3}{0.718 \times 36600 \times 275} = 1.027 > 1$$

$$k_{LT} = 1$$

To calculate χ_{LT}

$$\bar{\lambda}_{LT} = \lambda_{LT} / \lambda_1 \beta_W^{0.5}$$

$$\lambda_{LT} = \frac{L / i_{LT}}{C_1^{0.5} \left[1 + \frac{(L / i_{LT})^2}{25.66} \right]^{0.25}}$$

$$\lambda_{LT} = \frac{6500 / 109}{\left[1 + \frac{(6500 / 109)^2}{25.66} \right]^{0.25}} = 45.5$$

$$\bar{\lambda}_{LT} = \lambda_{LT} / \lambda_1 \beta_W^{0.5} = 45.5 / 86.8 = 0.52$$

Use buckling curve a (Figure 6-13)

$$\chi_{LT} = 0.893$$

$$\frac{N_{Sd}}{\chi_z A f_y / \gamma_{M1}} + \frac{k_{LT} M_{y,Sd}}{\chi_{LT} W_{pl,y} f_y / \gamma_{M1}} = \frac{1220.1}{0.718 \times 36600 \times 275 / 1.05} + \frac{1.0 \times 19 \times 10^6}{0.893 \times 5810 \times 10^3 \times 275 / 1.05} \leq 1$$

Hence it is satisfactory for lateral torsional buckling.

6.3.5 Edge column design

The edge column (C2) shown in figure 6-15 bears the loads from a roof and one floor. The column is designed in S275.

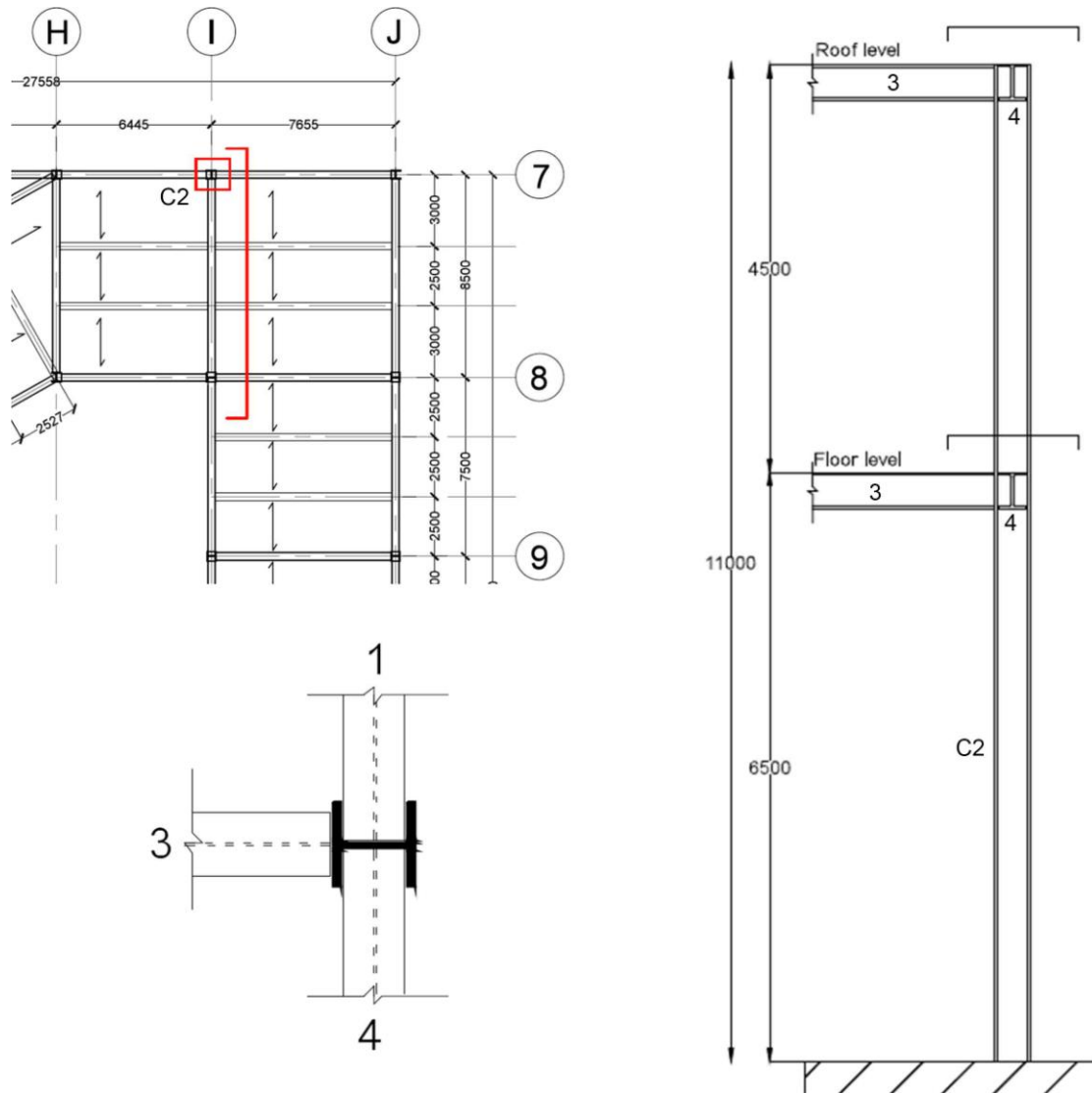


Figure 6-15 Edge Column C2 (dimension in mm)

6.3.5.1 Loads

Here is the loading calculation of the column

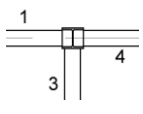
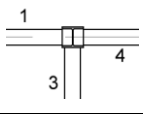
	Beam	Dead load	Imposed load	Design axial compression
	1	16.9	9.8	35.8
	2	0	0	0
	3	70	24.8	124.6
	4	20.1	11.6	42.5
Total		107	46.2	202.9
	1	16.9	29.1	64.7
	2	0	0	0
	3	70	82.3	210.9
	4	20.1	28.8	68.3
Total		107	140.2	343.9
Total		214	186.4	546.8

Table 6-23 Loading for Column C2 (kN)

Design values of axial compression

$$N_{sd} = \xi Y_G g_k + Y_Q q_k = 546.8 \text{ kN}$$

Takeing into consideration of the effect of unbalanced loading from beam 3 on the column, it creates the most critical situation.

Eccentricity is to be taken as 100mm from face of column.

$$e = h/2 + 100 \text{ mm} = 200 + 100 = 300 \text{ mm (we choose section HD400} \times 287)$$

Design moment:

At floor level:

$$M_{f,y,Ed} = 300 \times 210.9 = 63.3 \text{ kNm}$$

The moment is distributed between the column lengths in proportion to their bending stiffness (EI/L), unless the ratio of the stiffness does not exceed 1.5, in which case the moment is divided equally. As the ratio of the column stiffness is less than 1.5, the design moment is therefore:

$$M_{y,Ed} = 63.3 \times 0.5 = 31.7 \text{ kNm}$$

6.3.5.2 Ultimate Limit State (ULS) verification

- **Resistance of cross-section**

For a class 1 section without bolt holes, the reduced design plastic resistance moment, allowing for axial force, is:

$$M_{N_y,Rd} = \frac{M_{ply,Rd}(1-n)}{1-0.5a}$$

Where

$$n = N_{sd}/N_{pl,Rd}$$

$$a = (A-2bt_f)/A \quad \text{but } a \leq 0.5$$

To calculate $M_{ply,Rd}$

For bending about one axis, the design moment of resistance is

$$M_{ply,Rd} = W_{pl,y} f_y / \gamma_{M0} = 5810 \times 10^3 \times 275 / 1.05 / 10^6 = 1521 \text{ kNm}$$

To calculate n and a

Applied axial force, $N_{sd} = 546.8 \text{ kN}$

For a member under axial compression, the design plastic resistance of the cross-section is:

$$N_{ply,Rd} = A f_y / \gamma_{M0} = 36600 \times 275 / 1.05 / 10^3 = 9585 \text{ kN} > N_{sd} = 546.8 \text{ kN}$$

$$n = 546.8 \text{ kN} / 9585 = 0.06$$

$$a = 0.2$$

$$M_{N_y,Rd} = \frac{M_{ply,Rd}(1-n)}{1-0.5a} = 1521 \times (1-0.06) / (1-0.5 \times 0.2) = 1588 \text{ kNm} > M_{y,Ed} = 31.7 \text{ kNm}$$

Hence the result is satisfactory.

6.3.5.3 Serviceability Limit State (SLS) verification

- **Flexural buckling**

A class 1 member under moment about the major axis only, should satisfy the following:

$$\frac{N_{Sd}}{\chi_{min} A f_y / \gamma_{M1}} + \frac{k_y M_{y,Sd}}{W_{pl,y} f_y / \gamma_{M1}} \leq 1$$

χ_{min} is the lesser of χ_y and χ_z , the reduction factors for y-y and z-z axes respectively.

Assuming that the connection between the column and the primary beam are effectively pinned, then the slenderness about the y-y axis is:

$$\lambda_y = l / i_y = 6500 / 165 = 39.4$$

$$\lambda_1 = \pi (E / f_y)^{0.5} = 93.9 \epsilon = 86.8$$

$$\bar{\lambda}_y = \lambda_y / \lambda_1 \beta_A^{0.5} = 0.45$$

Use buckling curve b (Figure 6-13)

$$\chi_y = 0.922$$

$$\lambda_z = l / i_z = 6500 / 102.9 = 63.2$$

$$\lambda_1 = \pi (E / f_y)^{0.5} = 93.9 \epsilon = 86.8$$

$$\bar{\lambda}_z = \lambda_z / \lambda_1 \beta_A^{0.5} = 0.73$$

Use buckling curve c (Figure 6-13)

$$\chi_{min} = \chi_z = 0.718$$

$$k_y = 1 - \frac{\mu_y N_{Sd}}{\chi_y A f_y} \text{ but } k_y \leq 1.5$$

$$\text{Where } \mu_y = \bar{\lambda}_y (2\beta_{My} - 4) + \frac{W_{pl,y} - W_{el,y}}{W_{el,y}}$$

$$\beta_{My} = 1.1 \mu_y = 0.45 (2 \times 1.1 - 4) + \frac{5810 - 5070}{5070} = -0.65$$

$$k_y = 1 - \frac{-0.65 \times 546.8 \times 10^3}{0.922 \times 36600 \times 275} = 1.05 < 1.5$$

$$\frac{N_{Sd}}{\chi_{min} A f_y / \gamma_{M1}} + \frac{k_y M_{y,Sd}}{W_{pl,y} f_y / \gamma_{M1}} = \frac{546.8 \times 10^3}{0.718 \times 36600 \times 275 / 1.05} + \frac{1.1 \times 31.7 \times 10^6}{5810 \times 10^3 \times 275 / 1.05} \leq 1$$

Hence the result is satisfactory for flexural buckling.

- **Lateral torsional buckling**

A class 1 section should satisfy the following:

$$\frac{N_{Sd}}{\chi_z A f_y / Y_{M1}} + \frac{k_{LT} M_{y,Sd}}{\chi_{LT} W_{pl,y} f_y / Y_{M1}} \leq 1$$

$$k_{LT} = 1 - \frac{\mu_{LT} N_{Sd}}{\chi_z A f_y} \text{ but } k_{LT} \leq 1$$

$$\text{Where } \mu_{LT} = 0.15 \bar{\lambda}_z \beta_{M,LT} - 0.15$$

$$\beta_{M,LT} = 1.1$$

$$\mu_{LT} = 0.15 \times 0.73 \times 1.1 - 0.15 = -0.03$$

$$k_{LT} = 1 - \frac{-0.03 \times 546.8 \times 10^3}{0.718 \times 36600 \times 275} = 1.013 > 1$$

$$k_{LT} = 1$$

To calculate χ_{LT}

$$\bar{\lambda}_{LT} = \lambda_{LT} / \lambda_1 \beta_W^{0.5}$$

$$\lambda_{LT} = \frac{L / i_{LT}}{C_1^{0.5} \left[1 + \frac{(L / \alpha_{LT})^2}{25.66} \right]^{0.25}}$$

$$\lambda_{LT} = \frac{6500 / 109}{\left[1 + \frac{(6500 / 918)^2}{25.66} \right]^{0.25}} = 45.5$$

$$\bar{\lambda}_{LT} = \lambda_{LT} / \lambda_1 \beta_W^{0.5} = 45.5 / 86.8 = 0.52$$

Refer to buckling curve (Figure 6-13)

$$\chi_{LT} = 0.893$$

$$\frac{N_{Sd}}{\chi_z A f_y / Y_{M1}} + \frac{k_{LT} M_{y,Sd}}{\chi_{LT} W_{pl,y} f_y / Y_{M1}} = \frac{546.8}{0.718 \times 36600 \times 275 / 1.05} + \frac{1.0 \times 31.7 \times 10^6}{0.893 \times 5810 \times 10^3 \times 275 / 1.05} \leq 1$$

Hence the result is satisfactory for lateral torsional buckling.

6.3.6 Bracing design

Figure 6-16 shows the layout of the typical vertical bracing.

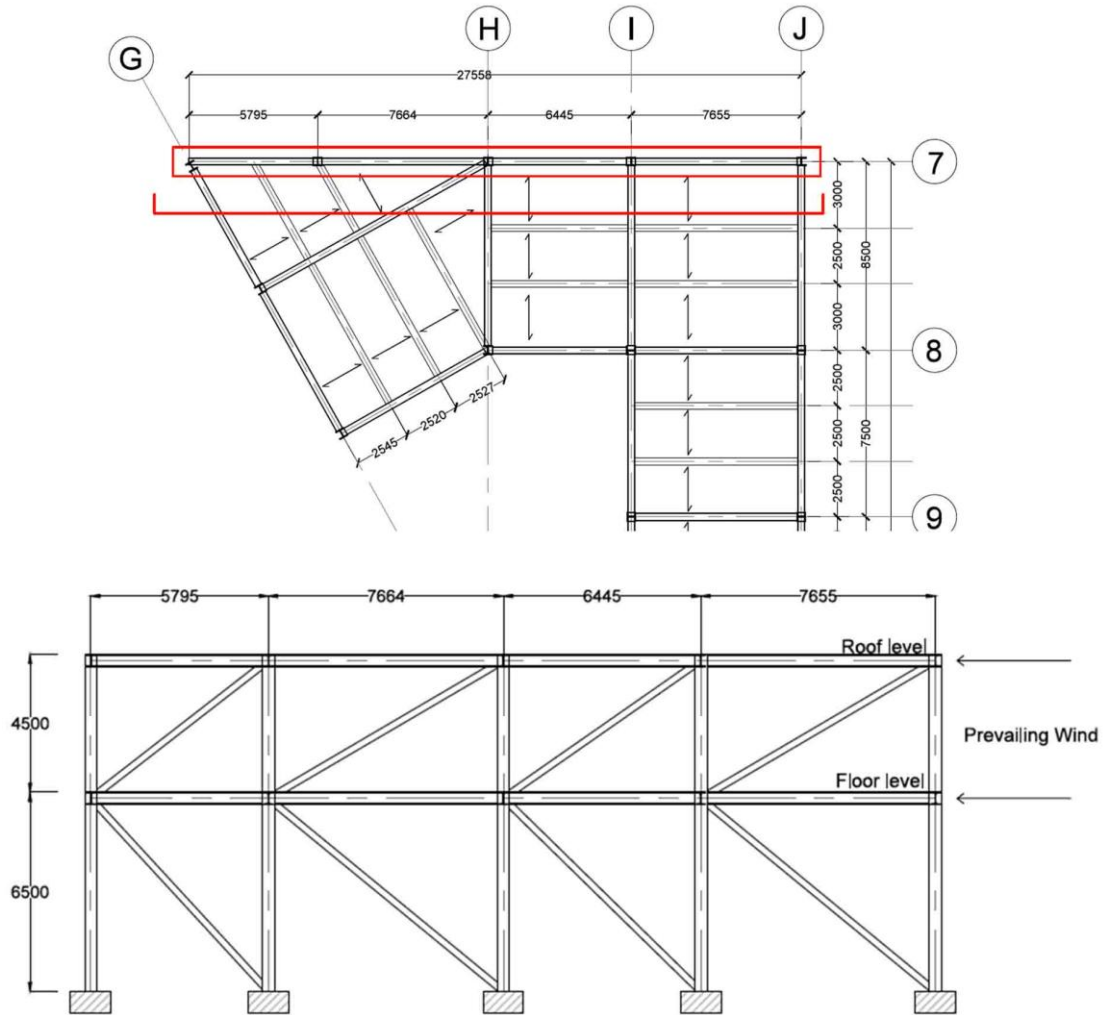


Figure 6-16 Vertical bracing (dimension in mm)

6.3.6.1 Loads

- **Forces in the bracing system**

Total overall wind load on the selected frame, according to the calculation in section 6.2.3

$$F_w = 201.5 \text{ kN}$$

Design wind load at ULS

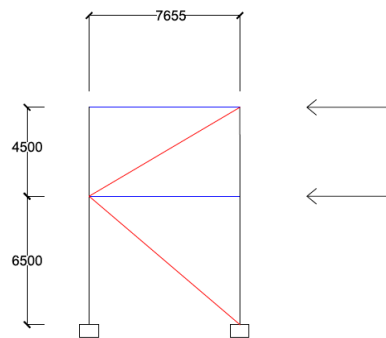
With wind as the leading variable action, the design wind load per braced bay is:

$$F_{Ed} = 1.5 \times 201.5 = 302.2 \text{ kN}$$

Distributing this total horizontal load as point loads at roof and floor levels, in proportion to the story heights:

$$\text{Roof level } \frac{4.5}{11} \times 302.2 = 123.6 \text{ kN}$$

$$\text{Floor level } \frac{6.5}{11} \times 302.2 = 178.6 \text{ kN}$$



- **Equivalent horizontal forces**

With wind as the leading variable action, the design values of the combined floor and roof actions are:

Design value for combined roof actions

$$= 0.925 \times 1.35 \times 3.5 + 1.5 \times 1.0 = 5.87 \text{ kN/m}^2$$

Design value for combined floor actions

$$= 0.925 \times 1.35 \times 3.5 + 1.5 \times 0.7 \times 4.0 = 8.57 \text{ kN/m}^2$$

$$\text{Total roof load} = 5.87 \times 608.5 = 3572 \text{ kN}$$

$$\text{Total floor load} = 8.57 \times 608.5 = 5215 \text{ kN}$$

Equivalent horizontal forces for each bracing system are:

$$\text{Roof level} = \frac{3572}{200} \times 0.5 = 8.9 \text{ kN}$$

$$\text{Floor level} = \frac{5215}{200} \times 0.5 = 13 \text{ kN}$$

The equivalent horizontal forces have been based on $\phi_0 = 1/200$.

Horizontal forces at floor level

Horizontal design force due to wind = 178.6 kN

Horizontal design force due to equivalent horizontal loads = 13 kN

Total horizontal design force per bracing system = 178.6 + 13 = 191.6 kN

Hand calculations can be carried out to find the member forces. Simply resolving forces horizontally at ground level is sufficient to calculate the force in the lowest (most highly loaded) bracing member, as shown in figure.

Horizontal component of force in bracing member = 191.6 kN

$$\text{Vertical component of force in bracing member} = \frac{191.6}{7.655} \times 6.5 = 162.7 \text{ kN}$$

Axial force in bracing

$$N = \sqrt{191.6^2 + 162.7^2} = 251.4 \text{ kN}$$

Partial factors for resistance

$$Y_{MO} = 1.0$$

$$Y_{M1} = 1.0$$

$$Y_{M2} = 1.25 \text{ (for bolts and welds)}$$

6.3.6.2 Proposed section selection

CHS 219.1x10 mm thick Circular Hollow Section (CHS), grade S275 is selected

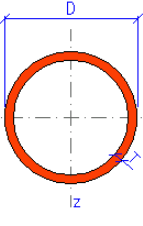
CHS 219.1x10		Section Properties
		$I_y = I_z = 3.60E+7 \text{ mm}^4$
Geometry		$W_{y,el} = W_{z,el} = 3.28E+5 \text{ mm}^3$
D = 219.1 mm		$W_{y,pl} = W_{z,pl} = 4.38E+5 \text{ mm}^3$
T = 10 mm		$i_y = i_z = 74 \text{ mm}$
A = 6570 mm ²		Warping and buckling
$A_L = 0.688 \text{ m}^2 \cdot \text{m}^{-1}$		$I_t = 7.20E+7 \text{ mm}^4$
G = 51.6 kg·m ⁻¹		$C_t = 6.57E+5 \text{ mm}^3$

Table 6-24 HD400 × 287 section properties

Source: http://www.staticstools.eu/profil_CHS.php?tlac&profil=CHS 219.1&lang=EN&je=0

● **Section classification**

$$d/t \leq 50\epsilon^2$$

$$\epsilon = (235/f_y)^{0.5}, f_y = 275 \text{ N/mm}^2, \epsilon = 0.92$$

$$d/t \leq 50\epsilon^2 = 50 \times 0.92^2 = 42.7$$

Since $d/t = 219.1/10 = 21.9 < 42.7$, the section is Class 1 for axial compression

6.3.6.3 Ultimate Limit State (ULS) verification

- **Cross sectional resistance to axial compression**

Basic requirement $\frac{N_{Ed}}{N_{Rd}} \leq 1.0$

N_{Ed} is the design value of the applied axial force

$$N_{Ed} = 251.4 \text{ kN}$$

$N_{c,Rd}$ is the design resistance of the cross-section for uniform compression

$$N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{MO}}$$

$$N_{c,Rd} = \frac{6570 \times 275}{1.0} \times 10^{-3} = 1806 \text{ kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{251.4}{1806} = 0.14 < 1.0$$

Therefore, the resistance of the cross section is adequate.

Tensile resistance

When the wind is applied in the opposite direction, the bracing member considered above will be loaded in tension. By inspection, the tensile capacity is equal to the cross-sectional resistance, 1806kN, >251.4 kN. Hence the result is satisfactory.

6.3.6.4 Serviceability Limit State (SLS) verification

- **Flexural buckling**

For a uniform member under axial compression the basic requirement is:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0$$

$N_{b,Rd}$ is the design buckling resistance and is determined from:

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$$

χ is the reduction factor for buckling and can be determined from (Figure 6-13)

For hot finished CHS in grade S275 steel use buckling curve 'a'.

For flexural buckling the slenderness is determined from:

$$\lambda = \sqrt{\frac{A f_y}{N_{cr}}} = \left(\frac{l_{cr}}{i}\right) \left(\frac{1}{\lambda_1}\right)$$

Where

L_{cr} is the buckling length

As the bracing member is pinned at both ends, conservatively take:

$$L_{cr} = L = \sqrt{7655^2 + 6500^2} = 10042 \text{ mm}$$

$$\lambda_1 = 93.9\epsilon$$

$$\epsilon = \sqrt{\frac{236}{f_y}} = \sqrt{\frac{235}{275}} = 0.92$$

$$\lambda_1 = 93.9 \times 0.92 = 76.1$$

$$\lambda = \left(\frac{10042}{74}\right) \times \left(\frac{1}{76.1}\right) = 1.78$$

For $\lambda = 1.78$ and buckling curve 'a'

$$\chi = 0.28$$

Therefore,

$$N_{b,Rd} = \frac{0.28 \times 65.7 \times 10^2 \times 275}{1.0} \times 10^{-3} = 653 \text{ kN}$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{251.4}{653} = 0.38 \leq 1.0$$

Therefore, the flexural buckling resistance of the section is satisfactory.

6.3.7 Connections design

Two connection design will be discussed in this section, namely the connection between a pair of secondary beam and the primary beam and the connection between primary beam and the column, which are under the most critical situations.

6.3.7.1 Beam to beam connection (B1-B2)

- **Initial design information**

For a secondary beam the design ultimate reaction is:

$$V_{Ed} = 119 \text{ kN}$$

The connection is to be nominally pinned so that deformations of the secondary beam can occur without inducing significant moments in the primary beams.

- **Basic details**

To achieve the required flexibility, the connection will be detailed in the following way shown in Figure 6-17

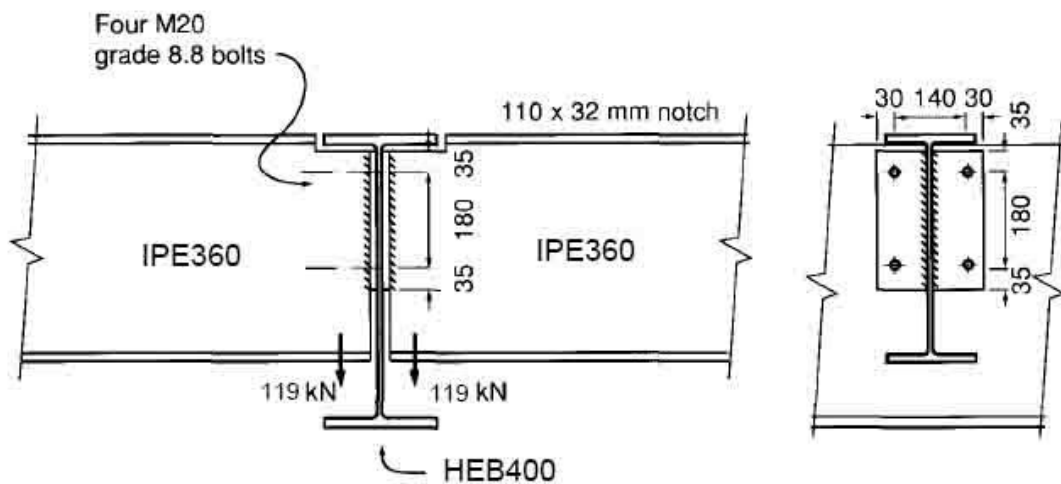


Figure 6-17 Primary beam to secondary beam connection (dimension in mm)

The proposed end-plate is 200mm wide, 250mm deep and 8mm thick. The dimensions of the plate have been selected to comply with the recommended minimum bolt gauge of 140mm and a plate depth greater than 0.6 of the depth of the supported beam.

It is envisaged that a four-bolt connection will be adequate. Using an end distance of 35mm, the bolt pitch is 180mm.

Minimum end distance, $e_1 = 1.2d_0 = 1.2 \times 22 = 26.4 \text{ mm}$, $i.e. < 35 \text{ mm}$

Normal edge distance, $e_2 = 1.5d_0 = 1.5 \times 22 = 33 \text{ mm}$, $i.e. > 30 \text{ mm}$

Although the edge distance is less than normal, it is acceptable provided that the bearing resistance is reduced accordingly and that the edge distance is not less than the minimum value of $1.2d_0$

Assumed notch depth, $d_c=32\text{mm}$

Assumed notch length, $c=110\text{mm}$

- **Shear resistance of the bolt group**

Treating the force on each shear plane separately:

Shear per bolt $=119/4=29.8\text{kN}$

Shear resistance of bolt, $F_{v,Rd} = \frac{0.6f_{ub}A_s}{Y_{Mb}}$

Where

Y_{Mb} is the material factor = 1.35

f_{ub} is the ultimate tensile strength of the bolt = 800N/mm^2

A_s is the tensile stress area of the bolt = 245mm^2

$$F_{v,Rd} = \frac{0.6f_{ub}A_s}{Y_{Mb}} = \frac{0.6 \times 800 \times 245}{1.35 \times 10^3} = 87.1\text{kN/plane} > 29.8\text{kN}$$

Hence the result is satisfactory.

- **Shear resistance of the end-plate**

The reduction in shear resistance resulting from the presence of the fasteners can be ignored if:

$$A_{v,ner}/A_v \geq f_y/f_u$$

$$\frac{A_{v,ner}}{A_v} = \frac{(2 \times 250 \times 8) - (4 \times 8 \times 22)}{2 \times 250 \times 8} = 0.824 \geq \frac{f_y}{f_u} = \frac{275}{430} = 0.64$$

Hence the presence of the holes can be ignored and the gross area of the plate can be used.

Design plastic shear resistance

$$V_{pl,Rd} = \frac{f_y A_s}{\sqrt{3} Y_{M0}} = \frac{275 \times 4 \times 10^3}{\sqrt{3} \times 1.05 \times 10^3} = 604.8\text{kN} > 119\text{kN}$$

So the plates are adequate in shear.

- **End-plate weld resistance**

The end-plate is connected to the beam web by two full depth fillet welds.

$$\text{Fillet weld shear strength, } f_{vw,d} = \frac{f_u}{\sqrt{3} \beta_w Y_{Mw}} = \frac{430}{0.85 \times 1.35 \times \sqrt{3}} = 216\text{N/mm}^2$$

Total length of weld $= 250 \times 2 = 500\text{mm}$

Resistance required/mm $= 119 \times 10^3 / 500 = 238\text{N/mm}$

Design resistance, $F_{vw,d} = a f_{vw,d}$

So throat thickness required, $a \geq 238/216 = 1.1\text{mm}$

Leg length required $=a/0.7 = 1.57\text{mm}$
 So a minimum 6 mm fillet weld is used.

- **Bearing resistance of supporting beam**

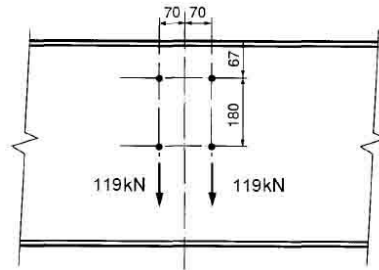


Figure 6-18 Primary beam web bolts holes (dimension in mm)

It shows that the bearing of the primary beam web is more critical than the bearing resistance of the secondary beam end-plates.

A detailed appraisal of the calculation of limiting bearing stress is presented under beam to column connection design. The limiting values stated here have been extracted from that calculation:

$$f_{b,Rd} = 796 \text{ kN/mm}^2 \text{ (limited bearing deformation)}$$

The actual bearing stress:

$$f_{b,Sd} = \frac{2V_{Sd}}{4dt_w} = \frac{2 \times 119 \times 10^3}{4 \times 20 \times 13.5} = 220.4 \text{ N/mm}^2 < f_{b,Rd}$$

Hence the result is satisfactory

- **Summary**

The final design of B1-B2 connection is grade 8.8 bolts and a 200x250x8 mm grade S275 end-plate welded to the beam with a full-length 6 mm fillet weld (each side).

6.3.7.2 Beam to column connection (B2-C2)

- **Initial design information**

For the connection between the primary beam B2 and an edge column at the ground floor level, the design ultimate beam reaction

$$V_{Sd} = 226 \text{ kN}$$

The connection is to be nominally pinned so that rotation of the beam can occur without inducing significant moments in the column.

- **Basic details**

To achieve the required flexibility, the connection is shown as figure 6-19 below.

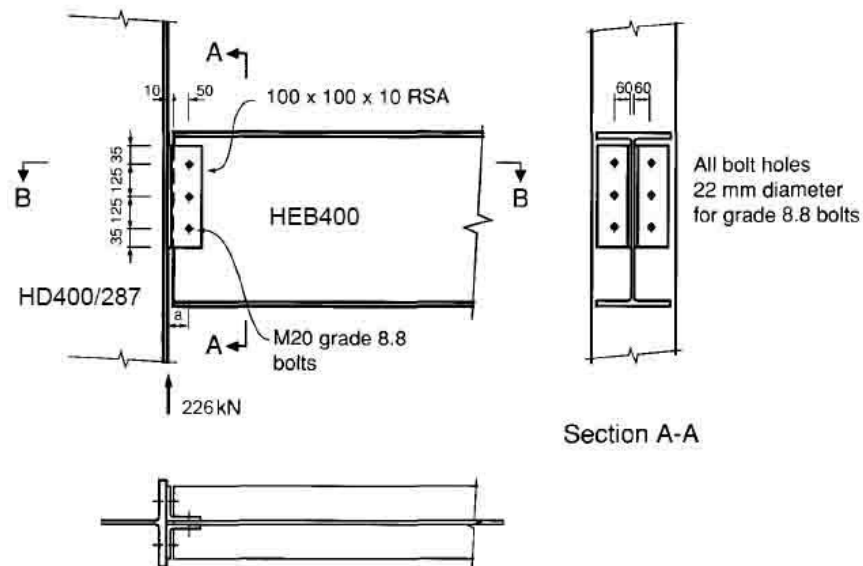


Figure 6-19 Primary beam to column connection (dimension in mm)

- **Check adequacy of bolts to the beam web**

The design shear force is the vector sum of vertical and horizontal components. The horizontal components is that due to a moment arising from the support reaction acting through an eccentricity

$$F_{v,Sd} = (F_v^2 + F_m^2)^{0.5}$$

Where

F_v is the vertical shear component per beam web bolt $F_v = V_{Sd}/3 = 75.4 \text{ kN}$

F_m is the horizontal shear component per beam web bolt due to the reaction eccentricity $F_m = V_{Sd}a/Z_b$ (for the outermost bolt)

Z_b is the elastic modulus of the bolt group $= \frac{n(n+1)}{6} p$

p is the bolt pitch

n is the number of bolts

$$F_m = \frac{V_{Sd}a}{Z_b} = \frac{6V_{Sd}a}{n(n+1)p} = \frac{6 \times 226 \times 60}{3 \times 4 \times 125} = 54.2 \text{ kN}$$

$$\text{So } F_{v,Sd} = (F_v^2 + F_m^2)^{0.5} = (75.4^2 + 54.2^2)^{0.5} = 92.9 \text{ kN}$$

Shear resistance of bolt per shear plane:

$$F_{v,Rd} = \frac{0.6f_{ub}A_s}{\gamma_{Mb}} = 87.1 \text{ kN/plane}$$

For double shear, bolt resistance = $87.1 \times 2 = 174.2 \text{ kN} > 92.9 \text{ kN}$

So the shear resistance of the bolts is adequate.

- **Shear rupture resistance**

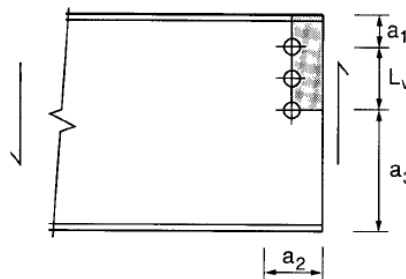


Figure 6-20 Primary beam web bolts holes (dimension in mm)

This check is applied to ensure that there is sufficient material remaining in the web after allowing for the presence of bolt holes.

Shear resistance is

$$V_{eff,Rd} = \left(\frac{f_y}{\sqrt{3}} \right) A_{v,eff} / \gamma_{M0}$$

Where

$$\gamma_{M0} = 1.05$$

$$A_{v,eff} = tL_{v,eff}$$

$$L_{v,eff} = L_v + L_1 + L_2 \quad (\text{but } \leq L_3)$$

$$L_v = 250 \text{ mm}$$

$$L_1 = 70 \text{ mm}$$

$$L_2 = (a_2 - k d_{o,t})(f_u/f_y) = (50 - 0.5 \times 22) \times (430/275) = 61 \text{ mm}$$

$$L_3 = L_v + a_1 + a_3 = 250 + 70 + 216.7 = 536.7 \text{ mm, but this should not be greater than}$$

$$L_3 = (L_v + a_1 + a_3 - n d_{o,v})(f_u/f_y) = (536.7 - 3 \times 22) \times (430/275) = 736 \text{ mm}$$

$$\text{So } L_{v,eff} = L_v + L_1 + L_2 = 250 + 70 + 61 = 381 \text{ mm}$$

$$V_{eff,Rd} = \left(\frac{f_y}{\sqrt{3}} \right) A_{v,eff} / \gamma_{M0} = (275/\sqrt{3}) \times 13.5 \times 381 / 1.05 / 10^3 = 778 \text{ kN} > V_{Sd} = 226 \text{ kN}$$

Hence the result is satisfactory.

- **Bearing resistance of bolts**

Bearing resistance:

$$F_{b,Rd} = \frac{2.5\alpha f_u dt}{Y_{Mb}}$$

Where

$$Y_{Mb} = 1.35$$

$$t = 13.5 \text{ mm}$$

$$d = 20 \text{ mm}$$

α is the lesser of $\frac{e_1}{3d_0}$, $(\frac{p_1}{3d_0} - \frac{1}{4})$, $\frac{f_{ub}}{f_u}$ or 1.0

p_1 is the bolt pitch = 125 mm

e_1 distance of a bolt from a free edge parallel to the applied load = $92.9 / 54.2 \times 50 = 85.7 \text{ mm}$

$$\frac{e_1}{3d_0} = 1.30 \quad (\frac{p_1}{3d_0} - \frac{1}{4}) = 1.64 \quad \frac{f_{ub}}{f_u} = 1.86 \quad \text{so minimum value is } \alpha = 1.0$$

Ultimate bearing strength

$$F_{b,Rd} = \frac{2.5\alpha f_u dt}{Y_{Mb}} = \frac{2.5 \times 1 \times 430 \times 20 \times 13.5}{1.35} = 214.9 \text{ kN} > F_v = 75.4 \text{ kN}$$

Hence the result is satisfactory.

6.3.8 Column base-plate design

The design of base-plate for internal column C1 is discussed in this section 6.3.8.

- **Initial design information**

Column section: HD400 × 287

Material strength: Steel grade S275	$f_y=275 \text{ N/mm}^2$
Concrete grade C35/45	$f_{ck}=35 \text{ N/mm}^2$ (cylinder/cube strength)
Grout	$f_{gk,cube}=12 \text{ N/mm}^2$

Loadings:

Characteristic axial loads on column C1:

$$N_{sd} = 1220.7 \text{ kN}$$

Characteristic shear loads resulting from beams connected to column C1:

- **Axial resistance**

Column flange thickness, $t_f = 36.6 \text{ mm}$

The thickness of the base plate should not be less than the thickness of the column flange. Use a base-plate, thickness $t > t_f$, choose 40mm (to be verified).

Determine the maximum potential effective bearing width, c , of the plate.

$$c = t \frac{f_y}{3f_j \gamma_{M0}}$$

Where the bearing strength:

$$f_j = \beta_j k_j f_{cd}$$

$$f_{cd} = f_{ck} / \gamma_c = 35 / 1.5 = 23 \text{ N/mm}^2$$

$$\beta_j = 0.67, \text{ with } f_{gk,cube} > 0.2 f_{ck,cube}, \text{ grout thickness} < 0.2 \times 550$$

$$k_j \text{ is the concentration factor} = \left(\frac{a_1 b_1}{ab} \right)^{0.5}$$

Where $a=b=650 \text{ mm}$ (to be verified).

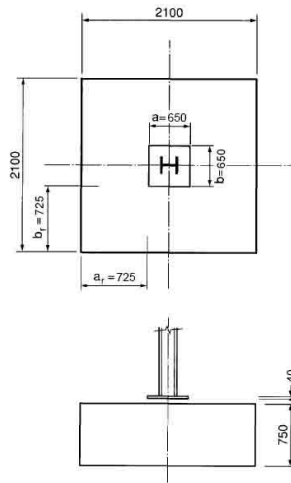


Figure 6-21 Proposed layout of the column base-plate (dimension in mm)

A 650mm base-plate has been selected to provide adequate room for locating the holding-down bolts.

a_1 = the smaller of:

$$a + 2a_r = 650 + 2 \times 725 = 2100 \text{ mm}$$

$$5a = 5 \times 650 = 3250 \text{ mm}$$

$$a + h = 650 + 750 = 1400$$

So $a_1 = 1400 \text{ mm}$

As the foundation and concentrically placed base-plate are both square, it is evident that

$$b_1 = a_1 = 1400 \text{ mm}$$

$$k_j = \left(\frac{a_1 b_1}{ab} \right)^{0.5} = \left(\frac{1400 \times 1400}{650 \times 650} \right)^{0.5} = 2.15$$

$$f_j = \beta_j k_j f_{cd} = 0.67 \times 2.15 \times 23 = 33.1 \text{ N/mm}^2$$

$$c = t \frac{f_y}{3f_j \gamma_{M0}} = 40 \left(\frac{275}{3 \times 33.1 \times 1.05} \right)^{0.5} = 65 \text{ mm}$$

Figure 6-22 shows that the effective area of the base-plate based on a cantilever projection of 65mm from the column.

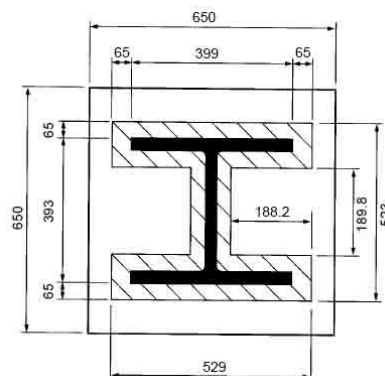


Figure 6-22 Effective area of the column base-plate (dimension in mm)

Effective area, $A_{\text{eff}} = 529 \times 523 - 2 \times 188.2 \times 189.8 = 205.2 \times 10^3 \text{ mm}^2$

Design bearing pressure = $N_{\text{sd}}/A_{\text{eff}} = 1220.7 \times 10^3 / 205.2 \times 10^3 = 6 \text{ N/mm}^2 < f_j = 33.1 \text{ N/mm}^2$

Hence the result is satisfactory.

- **Shear resistance**

Where the applied shear force is less than 20% of the applied vertical load, no special provisions are necessary for the transfer of the shear load from the base-plate to the foundation.

$N_{\text{sd}} = 1220.7 \text{ kN}$, $N_{\text{sd}}/5 = 1220.7/5 = 244.1 \text{ kN} > V_{\text{sd}} = 85 \text{ kN}$

Satisfactory

- **Determine the plate dimensions**

Minimum width of plate required = $b + 2c = 529 \text{ mm}$

Minimum depth of plate required = $d + 2c = 523 \text{ mm}$

A dimension of 650mm is needed to accommodate the holding-down bolts, so use a 650x650x40 mm thick grade 275 base-plate.

- **Welding requirements**

The plate and column are in tight bearing. There is no need to check the ability of the weld to transmit axial loads. However, sufficient weld should be used along the web of the section to allow the safe transfer of the applied shear force. In the case of the flanges, a full length weld will be provided on the outer face.

$V_{\text{sd}} = 85 \text{ kN}$

Weld shear strength:

$$f_{\text{vw,d}} = \frac{f_u}{\sqrt{3}\beta_w \gamma_{\text{Mw}}} = \frac{430}{0.85 \times 1.35 \times \sqrt{3}} = 216 \text{ N/mm}^2$$

An 8 mm fillet weld is selected:

Throat thickness, $a = 0.7 \times 8 = 5.6 \text{ mm}$

Resistance of weld/mm:

$$F_{\text{vw,d}} = a f_{\text{vw,d}} = 216 \times 5.6 = 1210 \text{ N/mm}^2$$

Length of weld required = $107.1 \times 10^3 / 1210 = 88.5 \text{ mm}$

We use a 100 mm run of 8 mm weld on each side of the column web.

Total weld run = 200 mm > 88.5 mm

Hence the result is satisfactory

- **Design summary**

The column base should be arranged as follow shown in Figure 6-23.

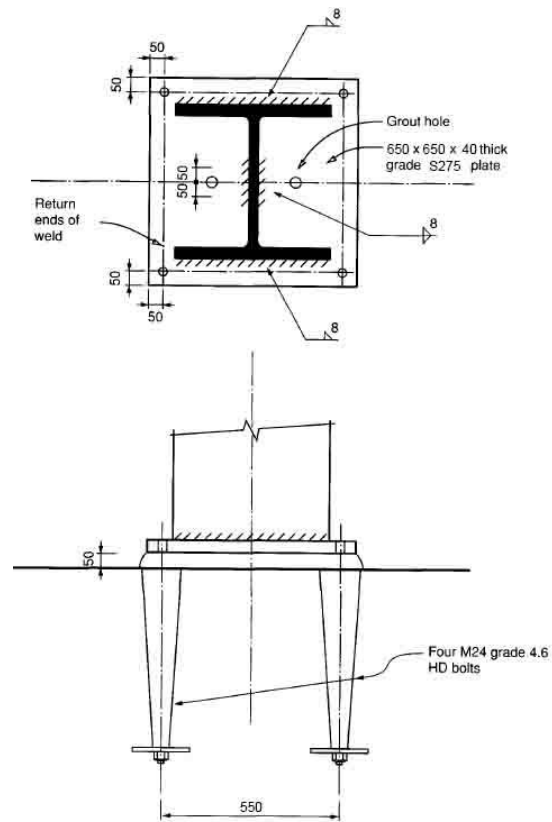


Figure 6-23 Layout of the column base-plate (dimension in mm)

6.4 West Wing Truss System Verification

In order to fulfill the need of the long free span and cantilever in the West Wing, a truss system with 5 main truss is applied. It supports between the roof and floor, and connects on the upper/lower chord by means of secondary trusses.

Each main truss has a span of 27m and a height of 7m, and is supported at the tops of the columns (there is no moment transmission between the truss and the column).

General transverse stability of the building is provided by fixity of the columns at ground level; longitudinal stability is provided by a system of roof bracings and braced bays in the walls.

Please Refer to Drawing S102 at Section 6.5 Structure Drawings.

6.4.1 Load

The truss secondary beams and main trusses are designed with the concentrated load on the node of chord.

For secondary truss, the concentrated load is generated by the purlins which support the floor/roof slabs and is connected to the node of upper chord of truss.

For the main truss, the concentrated load is subjected to the node of both upper chord and lower chord, generated at the supporting ends of the secondary trusses.

The concentrated loads that correspond to the combination of actions, determined with respect to the ultimate limit state (ULS) according to EN 1990, are used for the verification of resistance and instability of the section.

For the secondary truss, the most critical cases with span of 16m at 1st floor level are verified as ST-C for the secondary trusses at axis C (Please refer to Drawing S102)

For the main truss, the most critical cases with cantilever span of 12m are verified as MT-4 for the main truss at axis 4. (Please refer to Drawing S102)

6.4.2 Secondary truss design

6.4.2.1 General dimension and section property

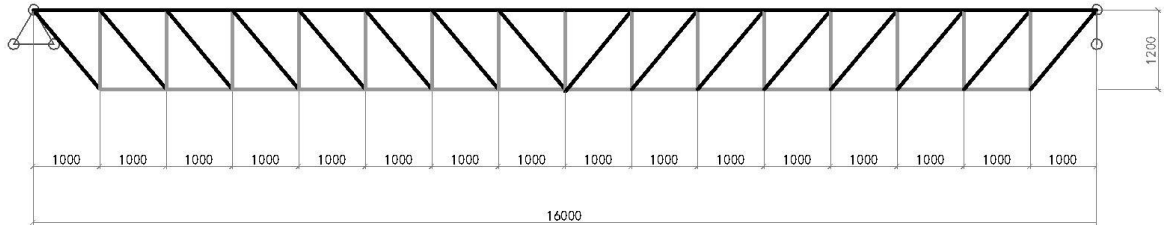


Figure 6-24 Truss secondary beam

As shown in the Figure 6-24 above, the truss chords are parallel and are made up of IPE 330 profiles with the webs horizontal, two L150x150x15 angles for diagonals, the posts are single angles L100x100x10.

The pairs of angles which make up the section of a diagonal are joined by battens, to ensure combined action with respect to buckling between the truss nodes. To be efficient, battens must therefore prevent local slip of one angle in relation to the other.

Each chord is fabricated in two pieces. The diagonals and posts are bolted at their two ends to vertical gusset plates, which are welded to the horizontal webs of the IPE 330 chords.

This secondary truss is supported by the main truss, for which the web is perpendicular to the plane of the secondary truss

6.4.2.2 Loads and Internal Actions

The concentrated loads which are subjected to the node of the upper chord of truss beam is showed in Figure 6.25 below. It is generated by the floor/roof purlin.

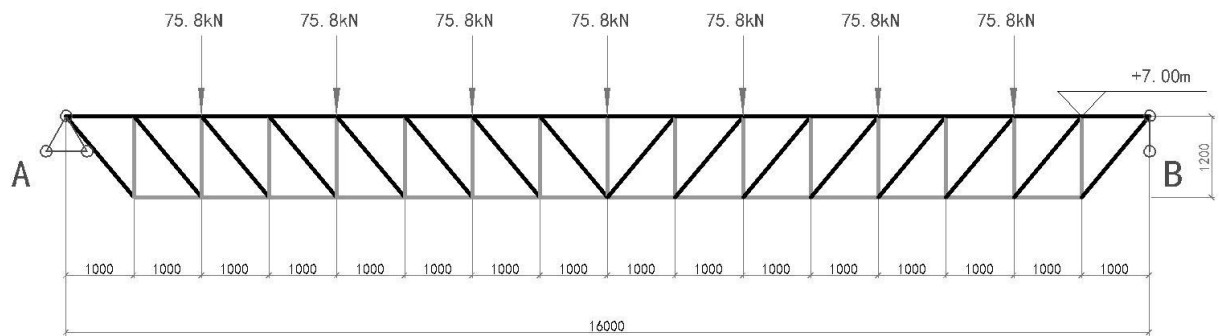


Figure 6-25 Secondary truss at roof level--Loads

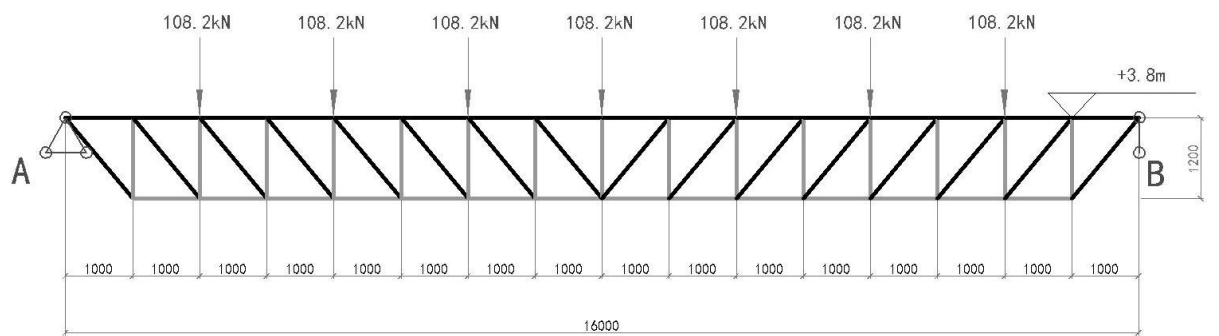


Figure 6-26 Secondary truss at 1/F level--Loads

The internal action of all the elements are thus shown as below:

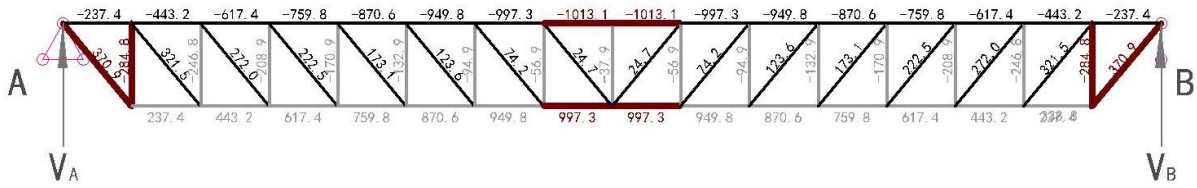


Figure 6-27 Secondary truss at roof level --internal actions and reactions

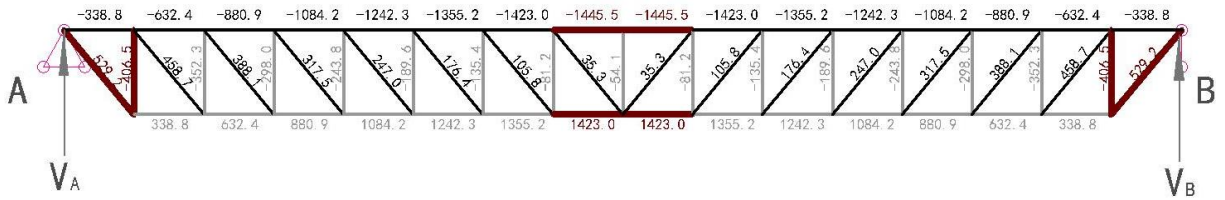


Figure 6-28 Secondary truss at 1/F level--internal actions and reactions

Four elements under critical condition are selected for analysis of their internal action. (Highlighted in red in Figure 6.27 & 28 above)

Hence we have,

Truss Element	Roof Level	1/F Level
	(kN)	(kN)
Upper Chord in Compression	-1013.1	-1445.5
Lower Chord in Tension	997.3	1423.0
Diagonal in Compression	284.8	-406.5
Diagonal in Tension	307.9	529.2

Table 6-25 Four truss elements under analysis

Due to the symmetrical structure and symmetrically loaded, the vertical reactions at two supports are calculated as:

Reaction	Roof Level	1/F Level
$V_A=V_B$	$75.8 \times 7/2 = 265.3\text{kN}$	$108.2 \times 7/2 = 378.7\text{kN}$

Table 6-26 Vertical reaction in Roof and 1/F level

As the internal action calculated above, the secondary truss at 1st floor level is more critical, its verification will be listed in the following Section.

6.4.2.3 Verification of members

Only the verification of upper chord in compression will be discussed in this section

Because the upper chord member lays adjacent to mid span, the normal compression force calculated under gravity ULS loads is greatest and shall be equal to”

$$N_{Ed} = -1445.5 \text{ kN}$$

Cross-section properties

For an IPE 330 with horizontal web (steel grade S355)

Total Section Area, A			Effective Section Area, A _{eff}
62.6cm ²	I _y =11770 cm ⁴	I _x =788 cm ⁴	60.95cm ²
Notes: 1) Class of cross section is determined according to EN 1993-1-1, Table 5.2, sheet 2 2) Effective Properties of cross section is determined according to EN 1993-1-5, Table 4.1			

Table 6-27 Cross-section properties

Resistance of cross-section

According to EN 1993-1-1, Section 6.2.4,

$$N_{c,Rd} = \frac{A_{eff} f_y}{\gamma_{M0}} = \frac{6095 \times 0.355}{1.0} = 2164 \text{ kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{1445.5}{2164} = 0.667 < 1$$

Hence the result is satisfactory.

Buckling resistance of member

In the plane of the truss

i.e. about the weak axis of the cross-section (EN 993-1-1 Section 6.3.1):

The buckling length of the upper chord member is equal to 90% of the system length (EN 1993-1-1 Appendix B, .B.1.1):

$$L_{cr,z} = 0.9 \times 1000 = 900 \text{ mm}$$

The elastic critical force is:

$$N_{cr,z} = \frac{\pi^2 E I_z}{l_z^2} = \frac{\pi^2 \times 21000 \times 788}{90^2} = 20161 \text{ kN}$$

The slenderness is given by:

$$\lambda_z = \sqrt{\frac{A_{eff} f_y}{N_{cr,z}}} = \sqrt{\frac{6095 \times 0.355}{20161}} = 0.33$$

The buckling curve is curve **b** (Please refer to figure 6-13 in the previous section, or EN 1993-1-1 Table 6.2)

So the imperfection factor is:

$$\alpha=0.34$$

$$\Phi_z=0.5 \times (1 + \alpha(\lambda_z - 0.2) + \lambda_z^2) = 0.58$$

$$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \lambda_z^2}} = 0.94$$

The design buckling resistance is then:

$$N_{b,z,Rd} = \frac{\chi_z A_{eff,y} f_y}{\gamma_{M0}} = \frac{0.94 \times 6095 \times 0.355}{1.0} = 2034 \text{ kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{1445.5}{2034} = 0.71 < 1$$

Hence the result is satisfactory.

Out of the plane of the truss,

i.e. about the strong axis of the cross-section (EN 1993-1-1 Section 6.3.1)

The lateral supports of the upper chord are composed of purlins at 2000 mm intervals.

The normal compression force is almost constant between lateral supports

There is therefore no need to use a method which allows for non-uniform force.

The elastic critical force is:

$$N_{cr,y} = \frac{\pi^2 E I_z}{l_y^2} = \frac{\pi^2 \times 21000 \times 788}{200^2} = 4082 \text{ kN}$$

The slenderness is given by:

$$\lambda_y = \sqrt{\frac{A_{eff,y} f_y}{N_{cr,y}}} = \sqrt{\frac{6095 \times 0.355}{4082}} = 0.53$$

The buckling curve is curve **a** (EN 1993-1-1 Table 6.2), and the imperfection factor is:

$$\alpha=0.21$$

$$\Phi_y=0.5 \times (1 + \alpha(\lambda_y - 0.2) + \lambda_y^2) = 0.68$$

$$\chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \lambda_y^2}} = 0.90$$

The design buckling resistance is then:

$$N_{b,z,Rd} = \frac{\chi_z A_{eff,y} f_y}{\gamma_{M0}} = \frac{0.9 \times 6095 \times 0.355}{1.0} = 1947 \text{ kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{1445.5}{1947} = 0.74 < 1$$

Hence the result is satisfactory.

6.4.2.4 Verification of global deflection

An estimate can be made for the deflection of the truss beam by calculating that for an equivalent beam, under the same loading. In order to do this, the classic approach is to use elementary beam theory, giving the equivalent beam a second moment of area equal to:

$$I = \sum_{i=1}^2 A_{ch,i} d_i^2$$

where

$A_{ch,i}$ is the section area of the chord i

d_i is the distance from the centroid of both chords to the centroid of the chord i .

In the case of this secondary truss,

$$A_{ch,1} = A_{ch,2} = 62.6 \text{ cm}^2$$

$$d_1 = d_2 = 60 \text{ cm}$$

$$I = \sum_{i=1}^2 A_{ch,i} d_i^2 = 62.6 \times 60^2 \times 2 = 225360 \text{ cm}^4$$

Deflection checks are based on the serviceability loading

For a uniform load

$$\delta = \frac{5}{384} \times \frac{F_k L^3}{EI_y}$$

Where

F_k : the total load = q_k or $(q_k + g_k)$ as appropriate

L : the span, 7.655m

E : the modulus of elasticity (210000 N/mm²)

I_y : the second moment of area about the major axis (y-y)

For load of 54kN/m

$$\delta = \frac{5}{384} \times \frac{F_k L^3}{EI_y} = \frac{5}{384} \times \frac{54.1 \times 10^3 \times 16000^3}{210000 \times 225360 \times 10^4} = 5.9 \text{ mm} < \frac{16000}{250} = 64 \text{ mm}$$

Hence the result is satisfactory.

6.4.3 Main Truss Design

6.4.3.1 General dimension and section property

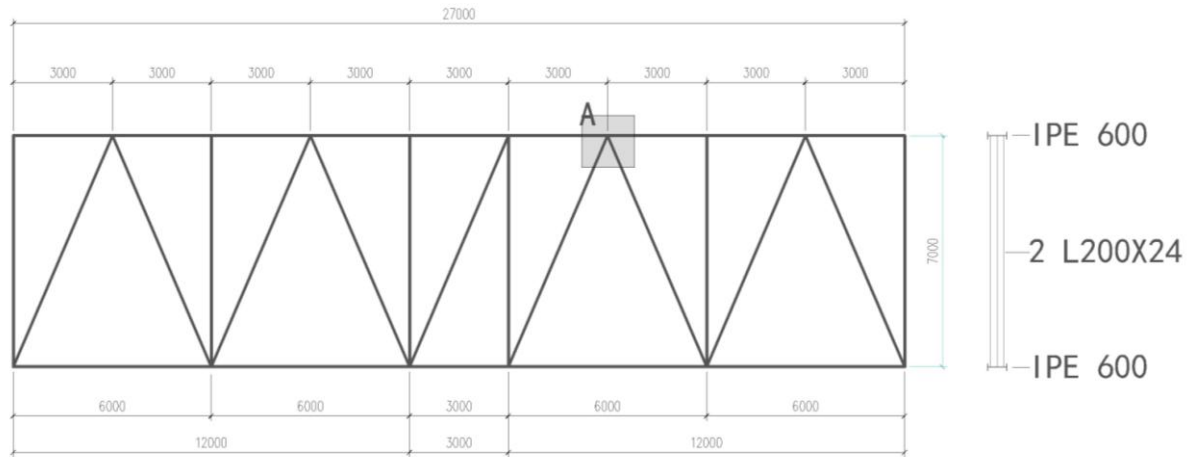
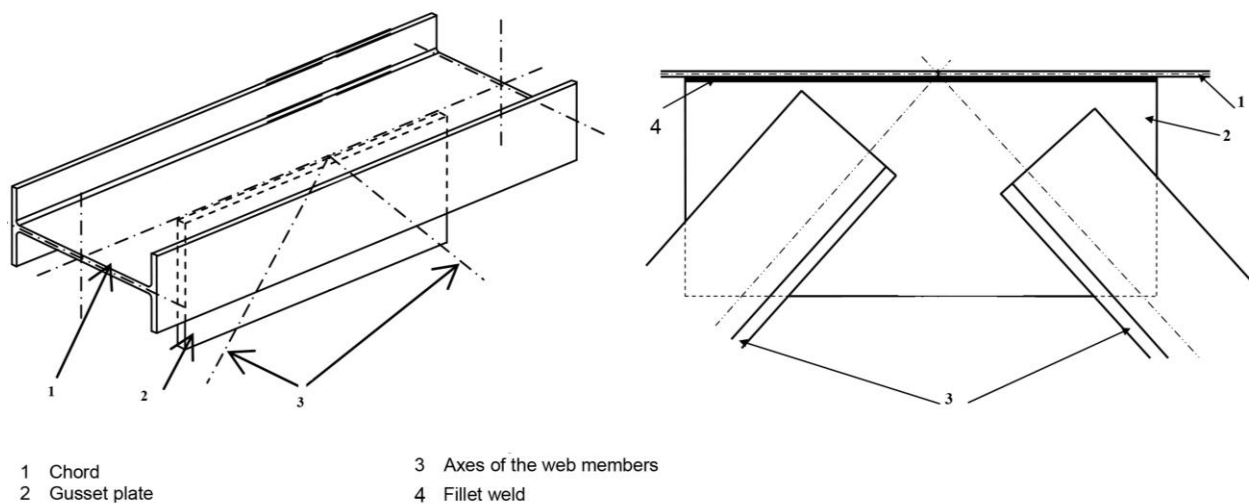


Figure 6-29 Main truss MT-C

As illustrated in the Figure 6-29 above, the truss chords are parallel to each other and are made up of IPE 600 profiles with the horizontal webs. The diagonals and posts are made of twinned angles: two L200x24 angles

The pairs of angles which make up the section of a diagonal are joined by battens, to ensure combined action with respect to buckling between the truss nodes. To be efficient, battens must therefore prevent local slip of one angle in relation to the other.

Each chord is fabricated in two pieces. The diagonals and posts are bolted at their two ends to vertical gusset plates, which are themselves welded to the horizontal webs of the IPE 600 chords.



- 1 Chord
- 2 Gusset plate
- 3 Axes of the web members
- 4 Fillet weld

Figure 6-30 Typical upper chord K joint at point A

6.4.3.2 Loads and internal actions

As discussed earlier, the concentrated loads generated by the secondary trusses at roof/1/F level are subjected to the node of upper/lower chord of main truss. The loads on MT-C are showed in Figure 6-31 below.

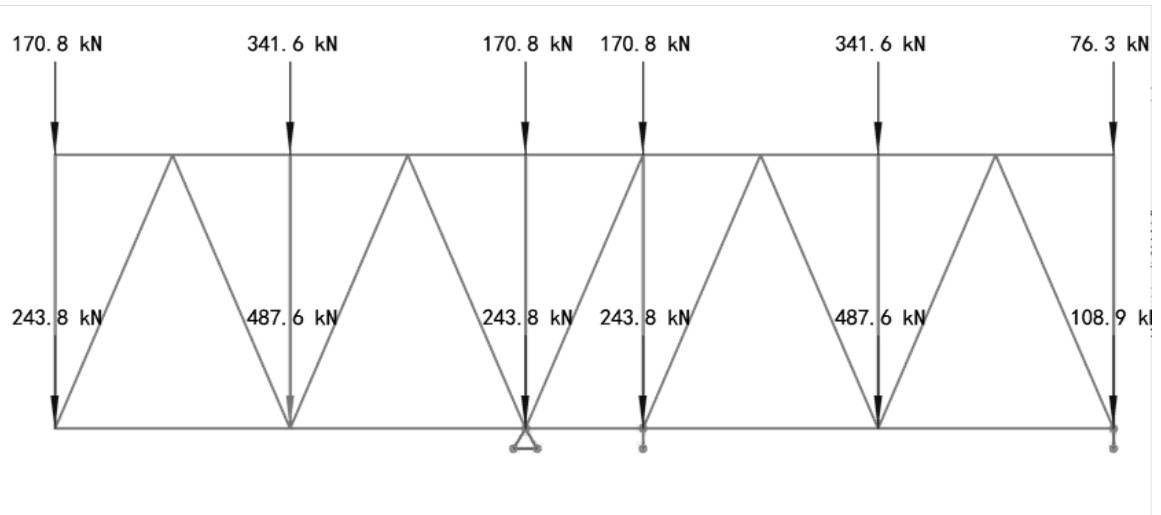


Figure 6-31 Main truss --loads

Thus we get the internal action of all the elements shown as below.

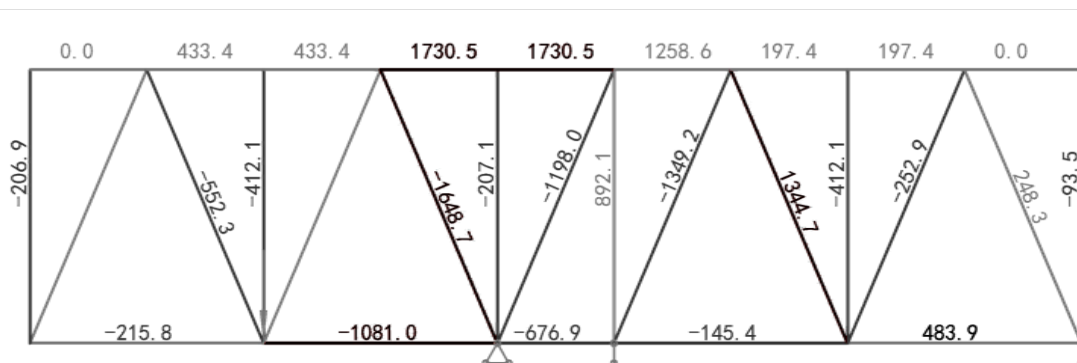


Figure 6-32 Main truss--internal actions and reactions

Four elements under critical condition are selected for analysis of their internal action. (Highlighted in red in Figure 6-31 above) Hence we have,

Truss Element	(kN)
Upper Chord in Tension	1730.5
Lower Chord in Compression	-1081.0
Diagonal in Compression	-1648.7
Diagonal in Tension	1344.7

Table 6-28 Four truss elements under verification

6.4.3.3 Verification of Members

- **Verification of lower chord in compression**

Compression force calculated under gravity ULS loads is greatest and equal to:

$$N_{Ed} = -1081.0 \text{ kN}$$

Cross-section properties

For an IPE 600 with horizontal web (steel grade S355)

Total Section Area
$A = 156 \text{ cm}^2$
$I_y = 92080 \text{ cm}^4$
$I_x = 3387 \text{ cm}^4$

Table 6-29 Cross section properties

Classification of cross-section

The material parameter:

for steel S335, $\epsilon = 0.81$

The cross-section can be classified in uniform compression,

The compressed flanges are classified as outstand flanges (EN 1993-1-1 Table 5.2, Sheet 2):

$$\frac{c}{t} = \frac{(220-12)/2}{19} = 5.47 < 9\epsilon = 7.29$$

The flange is Class 1.

The web is classified as an internal compressed part (EN 1993-1-1 Table 5.2, Sheet 1):

$$\frac{c}{t} = \frac{600-38}{12} = 46.83 > 42\epsilon = 34.02$$

The web is Class 4.

Effective properties of the cross-section

The effective area A_{eff} is calculated for pure compression.

The flanges are Class 1, so it is fully effective.

The effective width of the web is evaluated according to EN 1993-1-5 Table 4.1:

$$\Psi = 1 \Rightarrow k_{\sigma} = 4$$

$$\lambda_p = \frac{c/t}{93.9\epsilon\sqrt{k_{\sigma}}} = \frac{46.83}{93.9 \times 0.81 \times \sqrt{4}} = \text{he effective area of the section is:}$$

$$A_{eff} = 15600 - (562 - 562 \times 0.78) \times 12 = 14116 \text{ mm}^2$$

Resistance of cross-section

In compression (EN 1993-1-1 Section 6.2.4):

$$N_{c,Rd} = \frac{A_{eff,y} f_y}{\gamma_{M0}} = \frac{14116 \times 0.355}{1.0} = 5011.2 \text{ kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{1081}{5011.2} = 0.22 < 1$$

Hence the result is satisfactory.

Buckling resistance of member

In the plane of the truss, i.e. about the weak axis of the cross-section (EN 993-1-1 Section 6.3.1):

The buckling length of the lower chord member is equal to 90% of the system length (EN 1993-1-1 Appendix B, B.1.1):

$$L_{cr,z} = 0.9 \times 6000 = 5400 \text{ mm}$$

The elastic critical force is:

$$N_{cr,z} = \frac{\pi^2 E I_z}{l_z^2} = \frac{\pi^2 \times 21000 \times 3387}{540^2} = 2405 \text{ kN}$$

The slenderness is given by:

$$\lambda_z = \sqrt{\frac{A_{eff,y} f_y}{N_{cr,z}}} = \sqrt{\frac{14116 \times 0.355}{2405}} = 1.44$$

The buckling curve to use is curve **b** (Please refer to Figure 6-13, or EN 1993-1-1 Table 6.2), and the imperfection factor is:

$$\alpha = 0.34$$

$$\Phi_z = 0.5 \times (1 + \alpha(\lambda_z - 0.2) + \lambda_z^2) = 1.7$$

$$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \lambda_z^2}} = 0.4$$

The design buckling resistance is then:

$$N_{b,z,Rd} = \frac{\chi_z A_{eff,y} f_y}{\gamma_{M0}} = \frac{0.4 \times 14116 \times 0.355}{1.0} = 2004 \text{ kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{1081}{2004} = 0.54 < 1$$

Hence the result is satisfactory.

Out of the plane of the truss, i.e. about the strong axis of the cross-section (EN 1993-1-1 Section 6.3.1)

The lateral supports of the upper chord are composed of purlins at 2000 mm intervals.

The normal compression force is almost constant between lateral supports.

There is therefore no need to use a method which allows for non-uniform force.

The elastic critical force is:

$$N_{cr,y} = \frac{\pi^2 EI_z}{l_y^2} = \frac{\pi^2 \times 21000 \times 92080}{600^2} = 53012 \text{ kN}$$

The slenderness is given by:

$$\lambda_y = \sqrt{\frac{A_{eff} f_y}{N_{cr,y}}} = \sqrt{\frac{14116 \times 0.355}{53012}} = 0.31$$

The buckling curve is curve **a** (EN 1993-1-1 Table 6.2), and the imperfection factor is:

$$\alpha = 0.21$$

$$\Phi_y = 0.5 \times (1 + \alpha(\lambda_y - 0.2) + \lambda_y^2) = 0.56$$

$$\chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \lambda_y^2}} = 0.96$$

The design buckling resistance is then:

$$N_{b,z,Rd} = \frac{\chi_z A_{eff,y}}{Y_{M0}} = \frac{0.96 \times 14116 \times 0.355}{1.0} = 4810 \text{ kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{1081}{4810} = 0.23 < 1$$

Hence the result is satisfactory.

- **Verification of Diagonal in compression (double angles L200x24)**

The diagonal in compression is verified as follow:

Given the results shown above:

$$N_{Ed} = -1648.7 \text{ kN}$$

Total Section Area
$A=181 \text{ cm}^2$
$I_x= 6676.4 \text{ cm}^4$
$I_y=9058.9 \text{ cm}^4$

Table 6-30 Cross section properties

Resistance of cross-section

In compression (EN 1993-1-1 Section 6.2.4):

$$N_{c,Rd} = \frac{A_{fy}}{Y_{M0}} = \frac{18100 \times 0.355}{1.0} = 6425.5 \text{ kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{1648.7}{6425.5} = 0.26 < 1$$

Hence the result is satisfactory.

Buckling resistance of member

In the plane of the truss, i.e. about the weak axis of the cross-section (EN 993-1-1 Section 6.3.1):

The buckling length of the lower chord member is equal to 90% of the system length (EN 1993-1-1 Appendix B, B.1.1):

$$L_{cr,z} = 0.9 \times 7615 = 6854 \text{ mm}$$

The elastic critical force is:

$$N_{cr,z} = \frac{\pi^2 EI_z}{L_z^2} = \frac{\pi^2 \times 21000 \times 6676.4}{685.4^2} = 2945.5 \text{ kN}$$

The slenderness is given by:

$$\lambda_z = \sqrt{\frac{A_{eff} f_y}{N_{cr,z}}} = \sqrt{\frac{18100 \times 0.355}{2945.5}} = 1.47$$

The buckling curve to use is curve **b** (Please refer to figure 6-13, or EN 1993-1-1 Table 6.2), and the imperfection factor is:

$$\alpha = 0.34$$

$$\Phi_z = 0.5 \times (1 + \alpha(\lambda_z - 0.2) + \lambda_z^2) = 1.7$$

$$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \lambda_z^2}} = 0.4$$

The design buckling resistance is then:

$$N_{b,z,Rd} = \frac{\chi_z A_{eff,y}}{Y_{M0}} = \frac{0.4 \times 18100 \times 0.355}{1.0} = 2570.2 \text{ kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{1648.7}{2570.2} = 0.64 < 1$$

Hence the result is satisfactory.

Out of the plane of the truss, i.e. about the strong axis of the cross-section (EN 1993-1-1 Section 6.3.1)

The lateral supports of the upper chord are composed of purlins at 2000 mm intervals.

The normal compression force is almost constant between lateral supports.

There is therefore no need to use a method which allows for non-uniform force.

The elastic critical force is:

$$N_{cr,y} = \frac{\pi^2 EI_z}{l_y^2} = \frac{\pi^2 \times 21000 \times 9058.9}{761.5^2} = 3237 \text{ kN}$$

The slenderness is given by:

$$\lambda_y = \sqrt{\frac{A_{eff,y}}{N_{cr,y}}} = \sqrt{\frac{18100 \times 0.355}{3237}} = 1.41$$

The buckling curve is curve **b** (EN 1993-1-1 Table 6.2), and the imperfection factor is:

$$\alpha = 0.34$$

$$\Phi_y = 0.5 \times (1 + \alpha(\lambda_y - 0.2) + \lambda_y^2) = 1.69$$

$$\chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \lambda_y^2}} = 0.37$$

The design buckling resistance is then:

$$N_{b,z,Rd} = \frac{\chi_z A_{eff,y}}{Y_{M0}} = \frac{0.37 \times 18100 \times 0.355}{1.0} = 2439 \text{ kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{1648.7}{2439} = 0.68 < 1$$

Hence the result is satisfactory.

- **Verification of global deflection**

An estimate can be made for the deflection of the truss by calculating that for an equivalent beam, under the same loading. In order to do this, the classic approach is to use elementary beam theory, giving the equivalent beam a second moment of area equal to:

$$\square I = \sum_{i=1}^2 A_{ch,i} d_i^2$$

where

$A_{ch,i}$ is the section area of the chord i

d_i is the distance from the centroid of both chords to the centroid of the chord i .

In the case of this main truss,

$$A_{ch,1} = A_{ch,2} = 156 \text{ cm}^2$$

$$d_1 = d_2 = 350 \text{ cm}$$

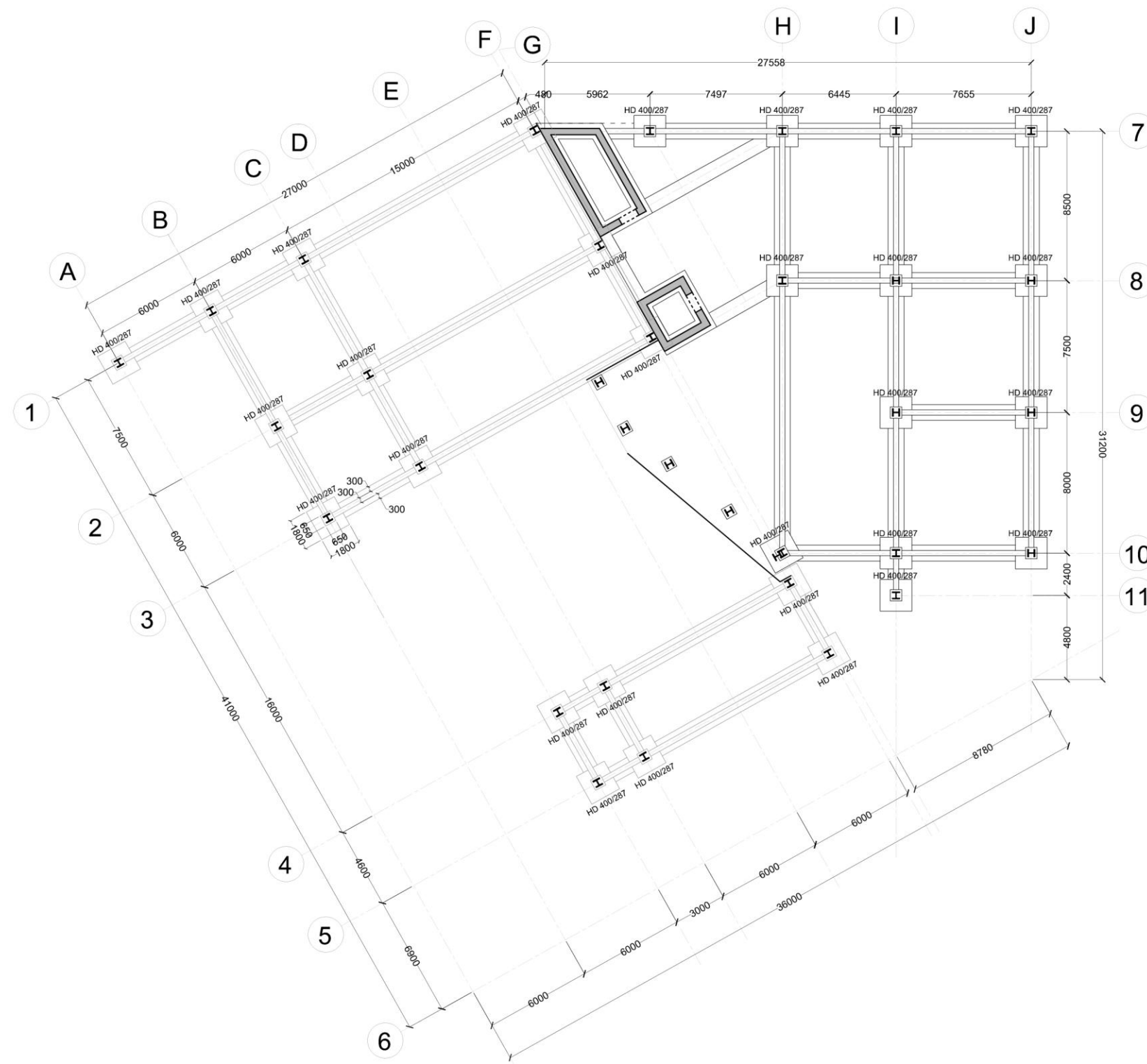
$$\square I = \sum_{i=1}^2 A_{ch,i} d_i^2 = 62.6 \times 60^2 \times 2 = 3822 \times 10^4 \text{ cm}^4$$

The vertical deflection at the suspension span (12 m) is determined as:

$$W = \frac{l^3 F}{30 E I_y} = \frac{12000^3 \times 276 \times 10^3}{30 \times 210000 \times 3822 \times 10^8} = 0.198 \text{ mm} < \frac{12000}{180} = 66.7 \text{ mm}$$

Hence the result is satisfactory.

6.5 Structure Drawings



Final Submission
26-11-2014

POLITECNICO DI MILANO
MASTER OF SCIENCE IN ARCHITECTURE ENGINEERING
GRADUATION THESIS

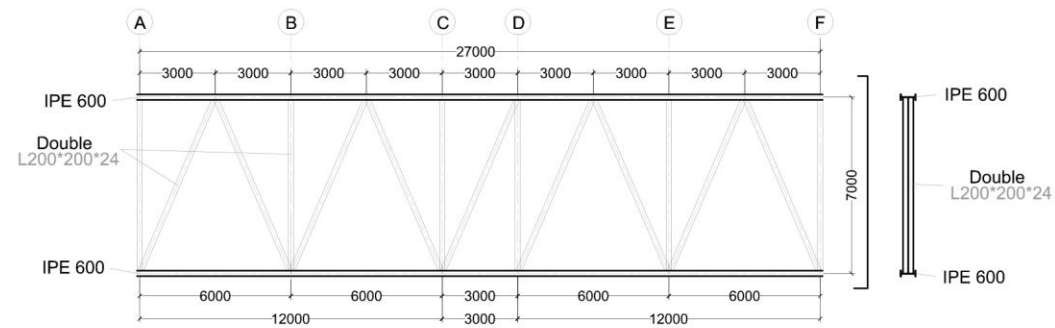
PROJECT
LUNG TSUN STONE BRIDGE ARCHAEOLOGY MUSEUM
HONG KONG - CHINA

AUTHORS
SHUFEI CHEN
MENGYANG LIN
JIE YANG

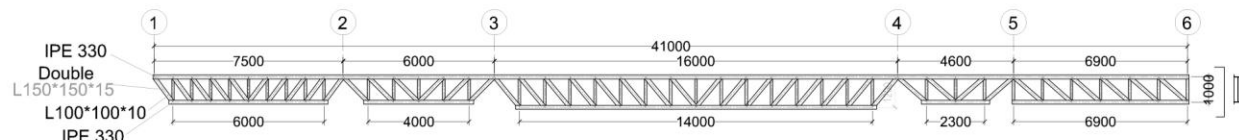


Drawing Code
Structure - Base Plan
S 101

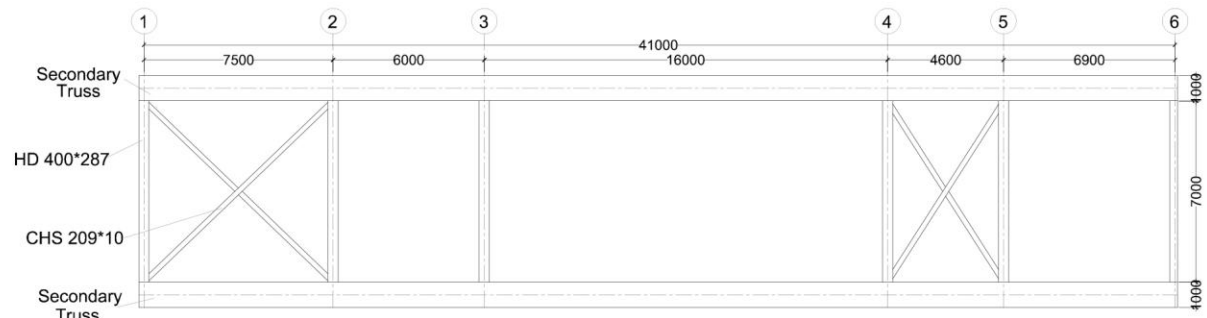
Drawing Scale
1: 300



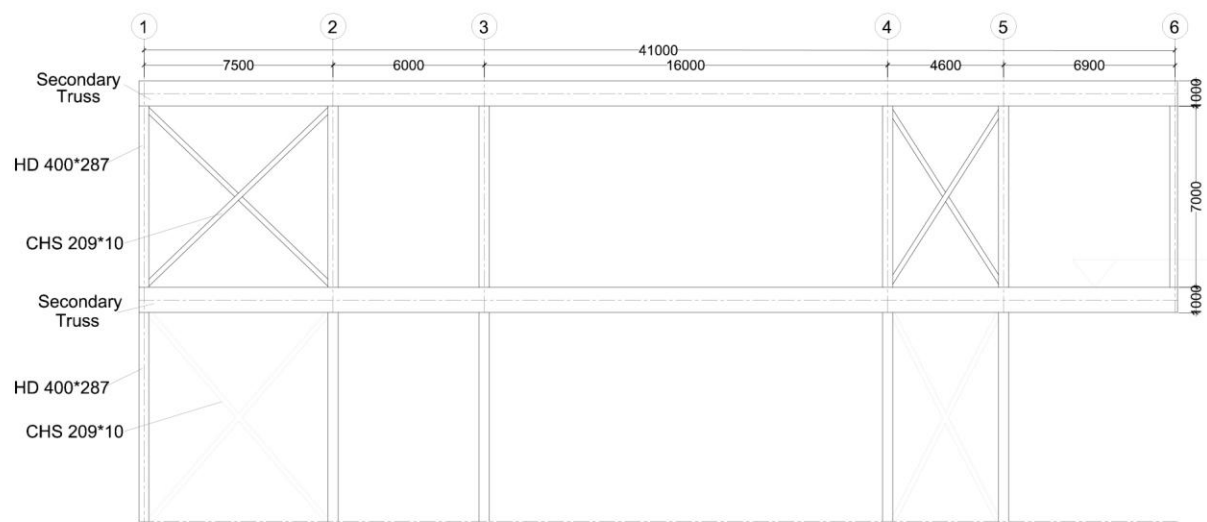
Main Truss 1:250



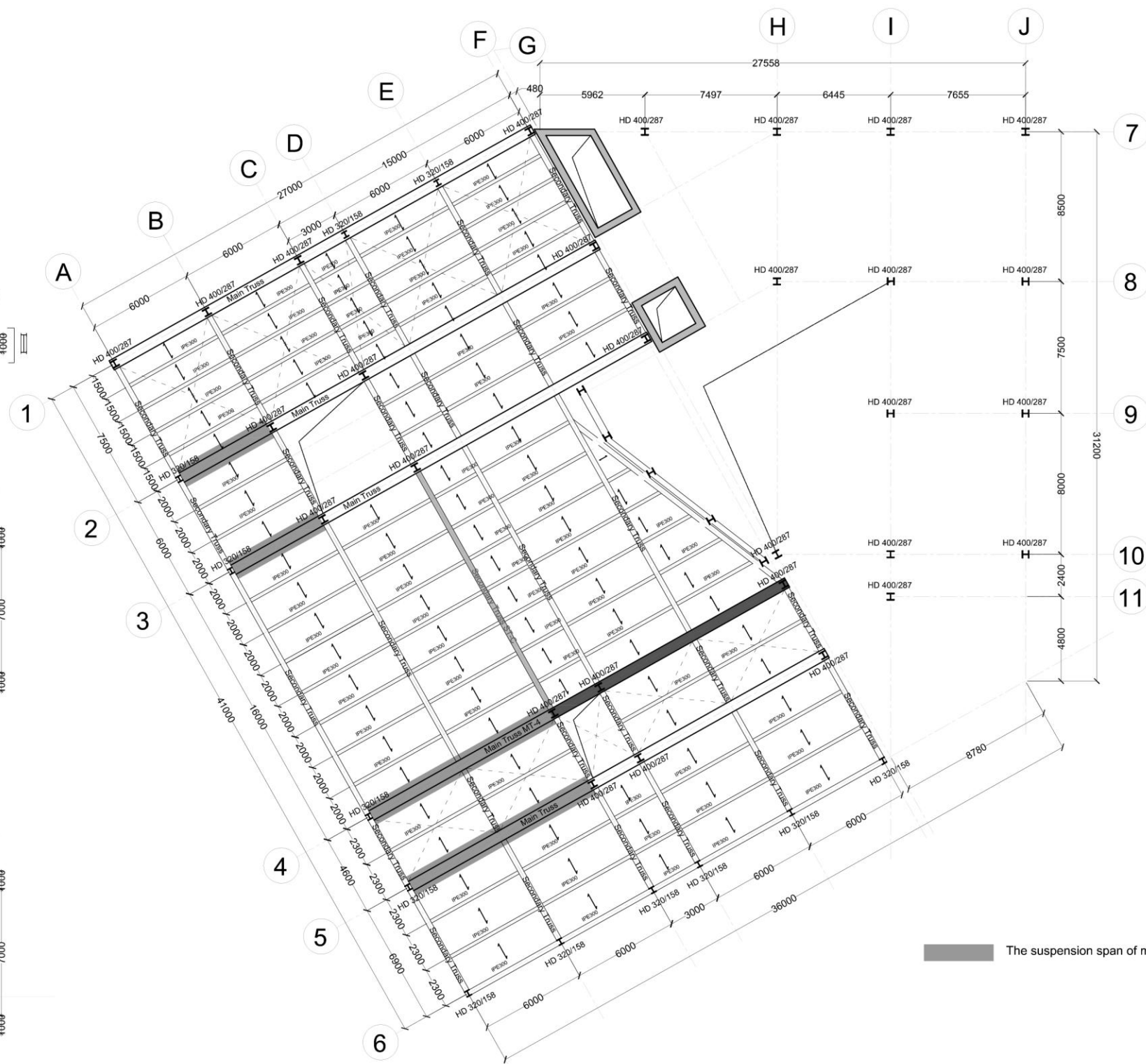
Secondary Truss 1:250



Vertical Bracing at axis A/F 1:250



Vertical Bracing at Axis C 1:250



The suspension span of main truss



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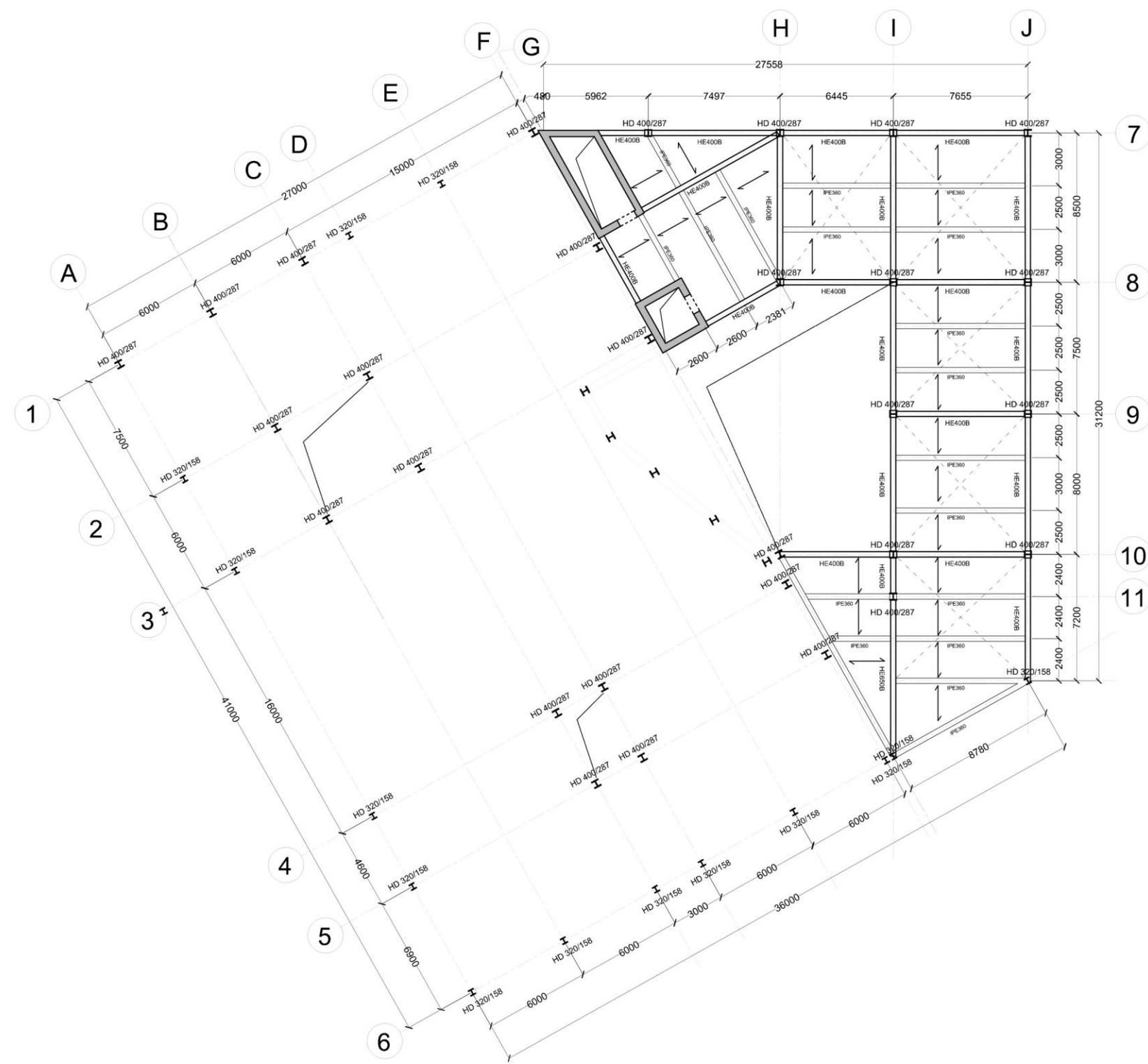
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AUTHORS
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MENGYANG LIN
JIE YANG



Drawing Code
Structure - 1 Floor Plan
S 102

Drawing Scale
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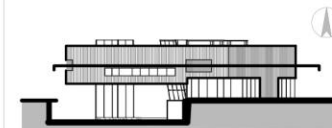


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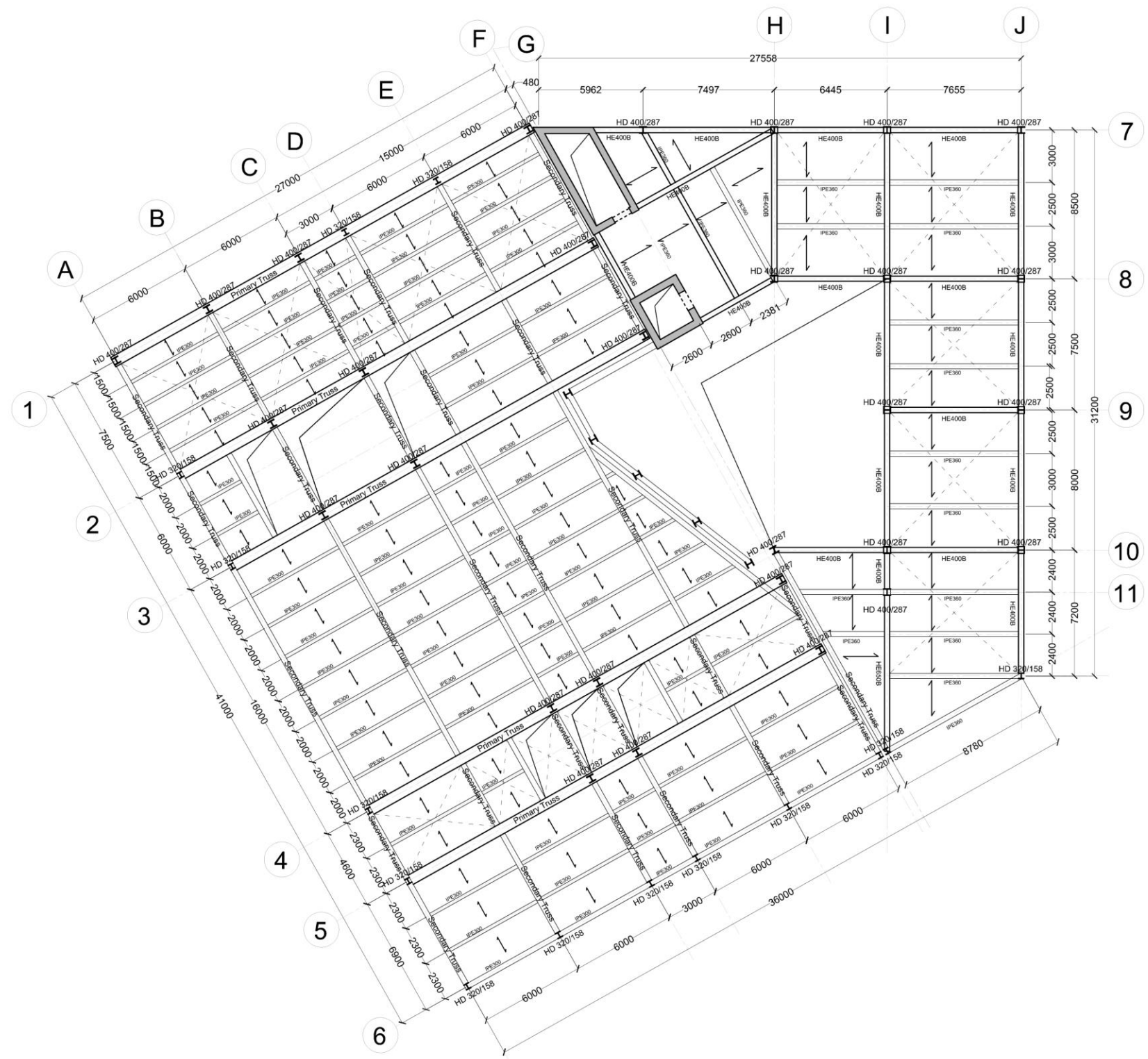
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Drawing Code
Structure - 2 Floor Plan
S 103

Drawing Scale
1: 300



Final Submission
26-11-2014

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GRADUATION THESIS

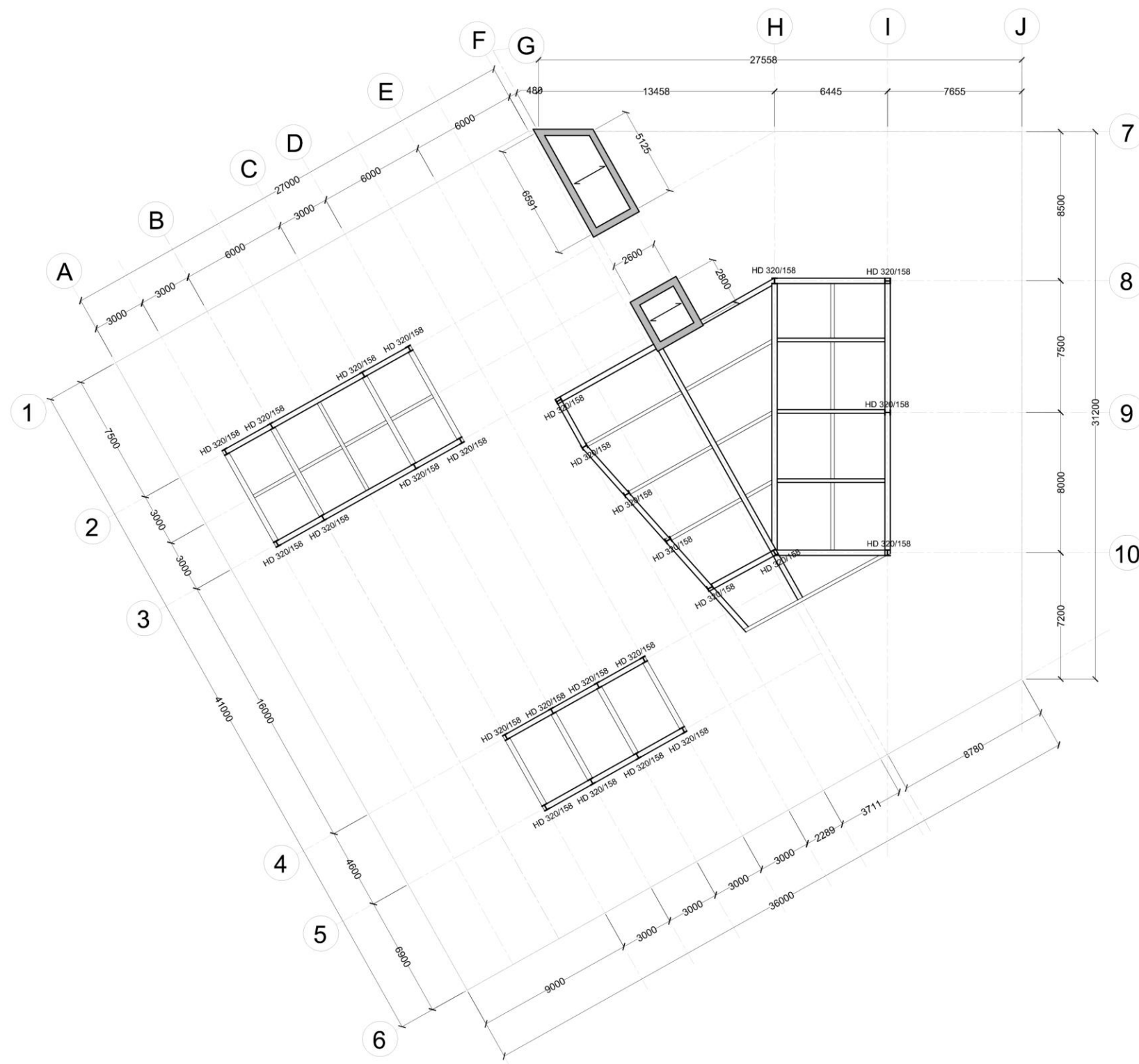
PROJECT
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MENGYANG LIN
JIE YANG



Drawing Code
Structure - Roof Plan
S 104

Drawing Scale
1: 300

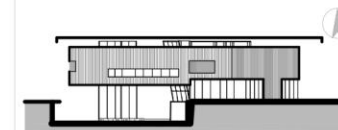


Final Submission
26-11-2014

POLITECNICO DI MILANO
MASTER OF SCIENCE IN ARCHITECTURE ENGINEERING
GRADUATION THESIS

PROJECT
LUNG TSUN STONE BRIDGE ARCHAEOLOGY MUSEUM
HONG KONG - CHINA

AUTHORS
SHUFEI CHEN
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JIE YANG



Drawing Code
Structure - Top Plan
S 105

Drawing Scale
1:300

CHAPTER 7

TECHNOLOGICAL DESIGN

7.0 Technological Design

7.1 Climate Analysis

7.1.1 Hong Kong Climate Data

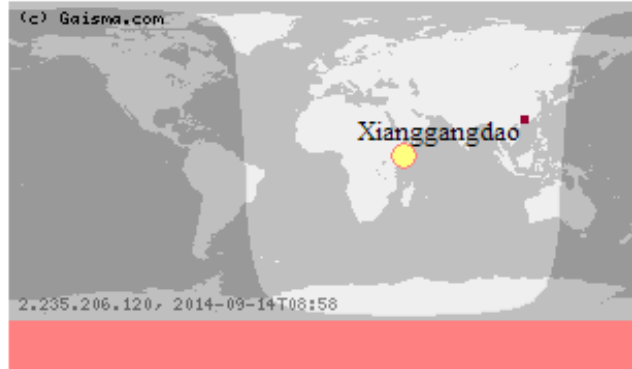


Figure 7-1 Hong Kong in World Map

Latitude: 22°15'N

Longitude: 114°10'E

Time zone: UTC +8 hours

The climate of Hong Kong is monsoon-influenced humid subtropical climate. Hong Kong has four seasons: a cool and dry winter, unstable and wet spring, hot and humid summer, and warm and pleasant autumn. The city is affected by both cool northeast monsoons and warm maritime airstreams.



Spring (March-May)



Summer (June-September)



Autumn (September-December)



Winter (December-February)

Figure 7-2 Four seasons in Hong Kong

7.1.1.1 Temperature

Following graph and data are drawn from Local Analysis in Ecotect and Climate-Consultant ULCA.

Hong Kong has very hot summer days. Summer is the most critical season in Hong Kong. As the Climate Consultant indicates, from June to September, the Dry-Bulb Temperature can reach 36 °C with average value of 28 °C. Relative Humidity is higher than comfort level of 60% all year long, particularly high for rain season from June to August.

On average, about 30 tropical cyclones form in the western North Pacific or China Seas every year, and about half of the tem reach typhoon strength (maximum winds of 118 kilometers per hour or more).

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
DbT(°C)	16	16	19	22	26	27	28	28	27	25	21	17
RH(%)	74	80	82	84	81	84	79	81	80	72	69	66

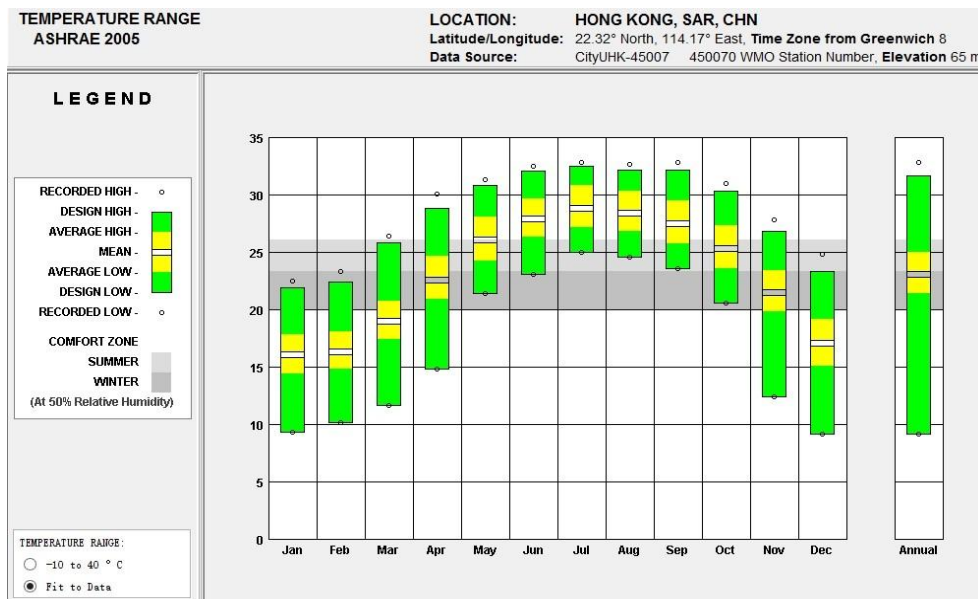


Figure 7-3 Temperature Data in Hong Kong

7.1.1.2 Wind

In Hong Kong, wind direction is reported from North East most time of the year, except at July and August from South West. Wind speed remains from 2 to 3m/s, but can reach 11m/s at extreme time (typhoon). Typhoon usually occur at July and September.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Direction(°)	90	90	100	90	60	90	250	240	80	90	90	90
Speed(m/s)	2	3	2	3	2	3	3	2	3	3	2	2

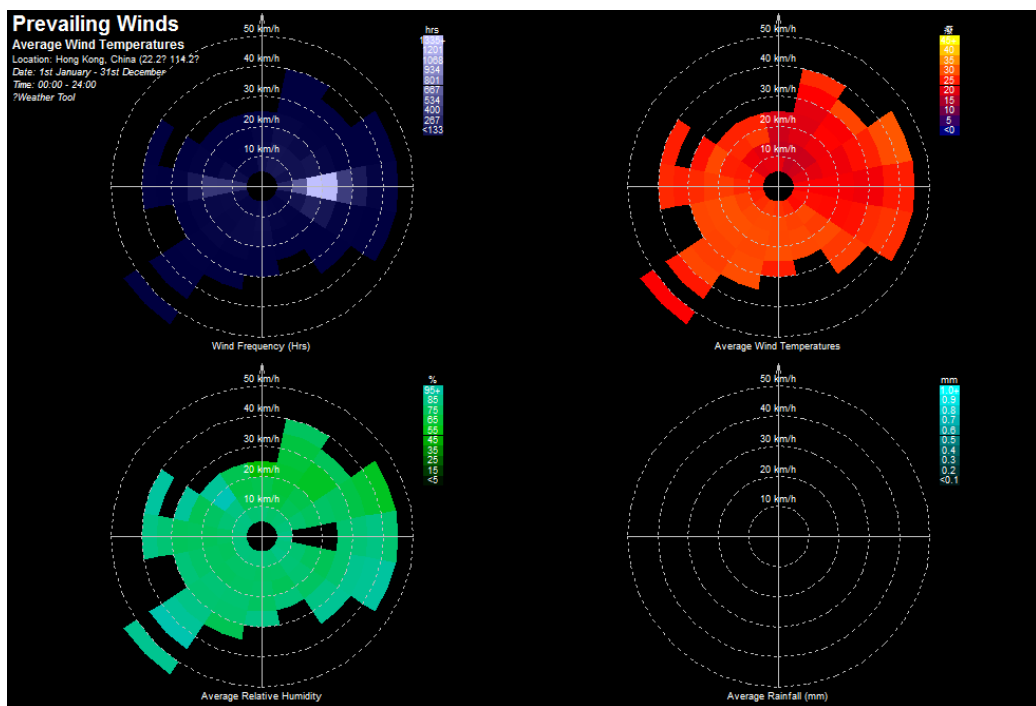


Figure 7-4 Wind Data in Hong Kong

7.1.1.3 Solar

It has been reported solar energy has a big potential for building energy use in Hong Kong. Annual average horizontal solar irradiance can reach 1290 kWh/m². The solar energy can reach peak from July to October.

Avg daily total	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Global horiz rad (wh/m ²)	2538	2691	2906	3370	3670	3855	4925	4331	3999	4048	3450	3056
Diffuse rad (wh/m ²)	1533	1763	1921	2325	2269	2375	2407	2381	2263	2016	1787	1659

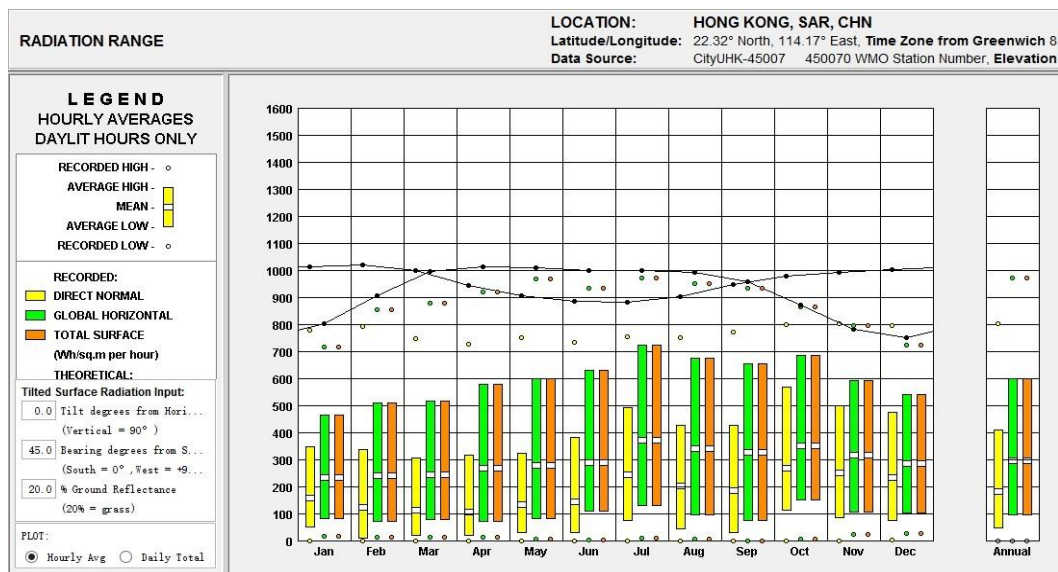


Figure 7-5 Solar Radiation Data in Hong Kong

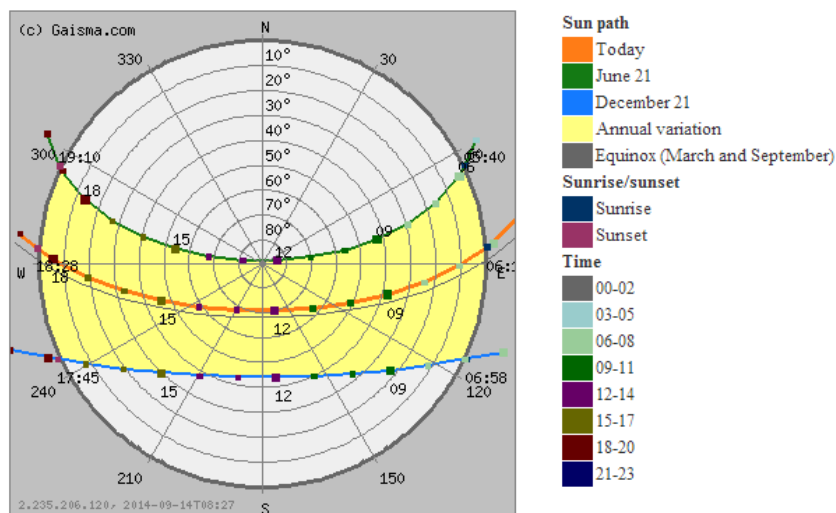
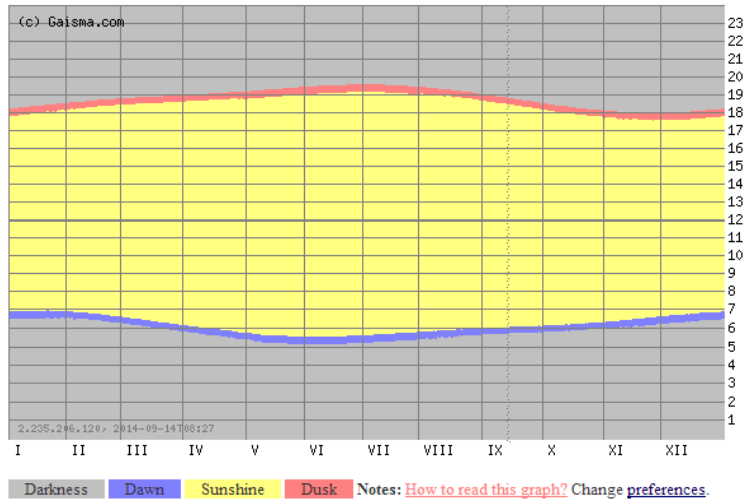


Figure 7-6 Solar Path



Date	Sunrise	Sunset	Length	Change	Dawn	Dusk	Length	Change
Today	06:10	18:28	12:18		05:47	18:51	13:04	
+1 day	06:10	18:27	12:17	00:01 shorter	05:48	18:50	13:02	00:02 shorter
+1 week	06:12	18:21	12:09	00:09 shorter	05:49	18:44	12:55	00:09 shorter
+2 weeks	06:14	18:14	12:00	00:18 shorter	05:51	18:37	12:46	00:18 shorter
+1 month	06:19	18:00	11:41	00:37 shorter	05:57	18:22	12:25	00:39 shorter
+2 months	06:34	17:41	11:07	01:11 shorter	06:11	18:04	11:53	01:11 shorter
+3 months	06:54	17:41	10:47	01:31 shorter	06:29	18:06	11:37	01:27 shorter
+6 months	06:33	18:32	11:59	00:19 shorter	06:11	18:55	12:44	00:20 shorter

Figure 7-7 Dawn and Dusk Time

Average monthly hours of sunshine over the year

This is the monthly total of sunhours



Figure 7-8 Average monthly hours of sunshine over the year

7.1.1.4 Rainfall

Hong Kong suffers from showers and thunderstorms from May to September. It might reach 380mm at peak amount.

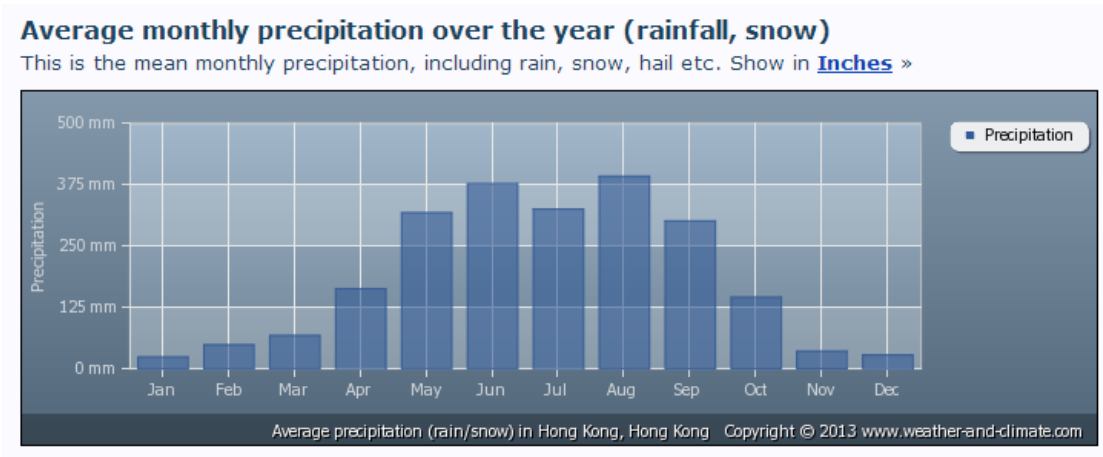


Figure 7-9 Average monthly precipitations over the year

7.1.1.5 Comfort Zone

During summer we notice that temperature and humidity has surpassed the comfort band. It indicates that HVAC system is necessary and important to help the space achieve comfort level.

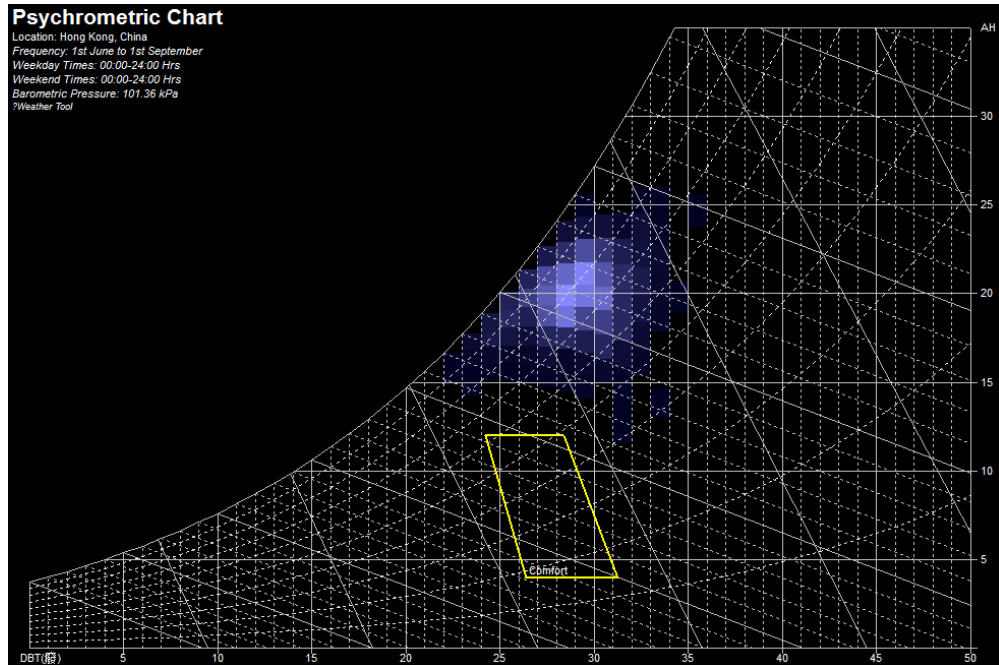


Figure 7-10 Comfort Zone for summer time

During winter the graph shows most of the temperature and humidity lays inside comfort band. Heating system might be considered regarding different aspects.

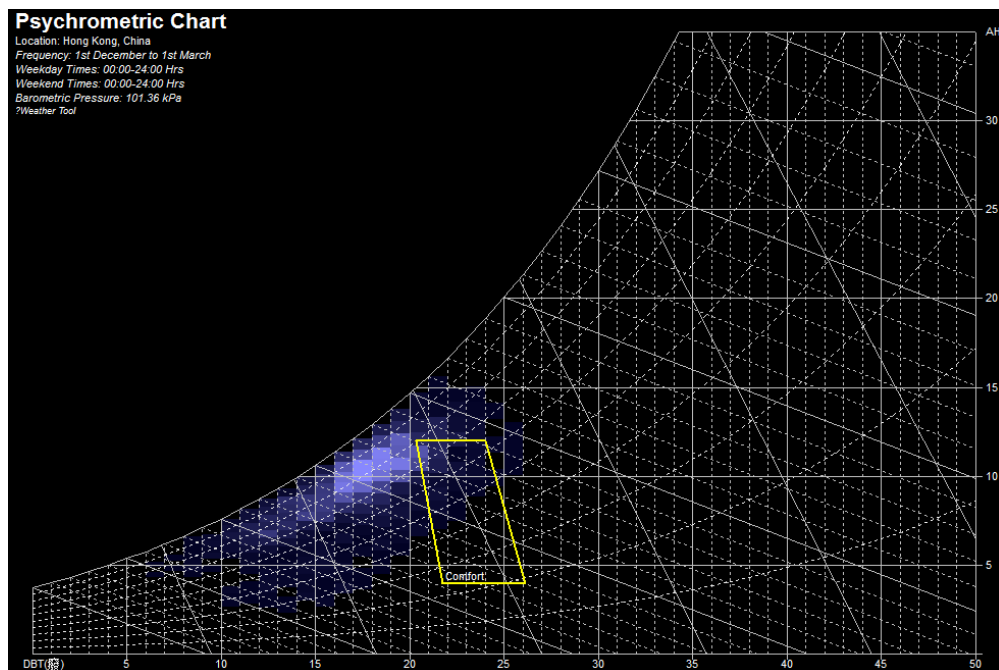


Figure 7-11 Comfort Zone for winter time

7.1.2 Site Climate

Due to the small distance (10m) between our site boundary and surrounding designated high-rise commercial buildings, the main scope of Ecotect analysis lays on the shadow and daylight analysis.

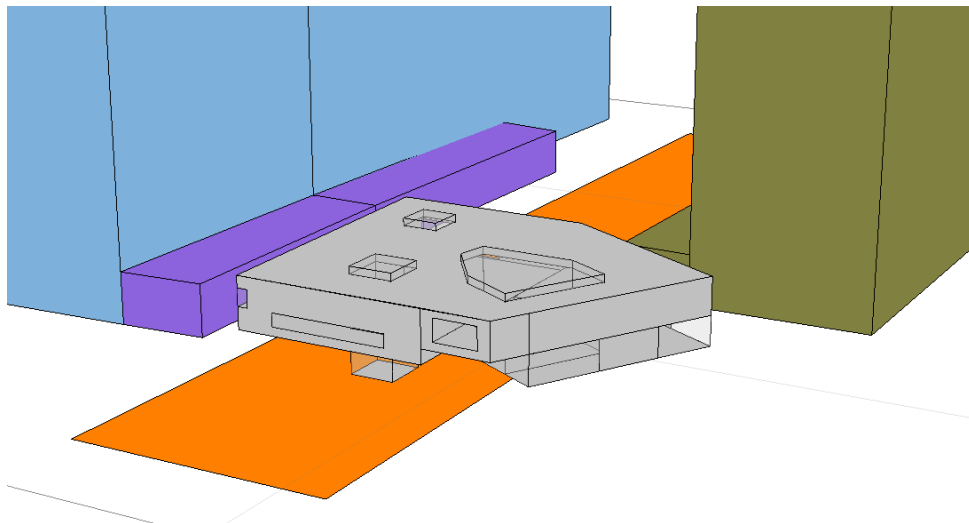
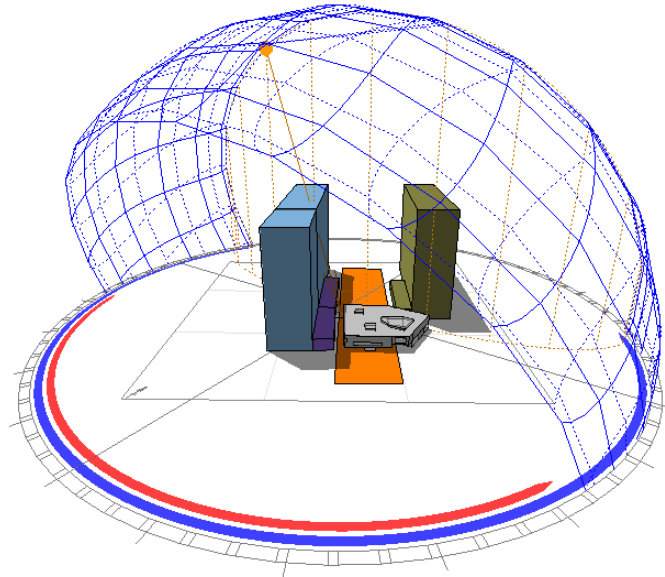


Figure 7-12 Ecotect Modeling

7.1.2.1 Sun Path & Shadow Analysis

Sun path is specified for four specific times during a year, namely winter solstice, summer solstice, vernal equinox and autumnal equinox.

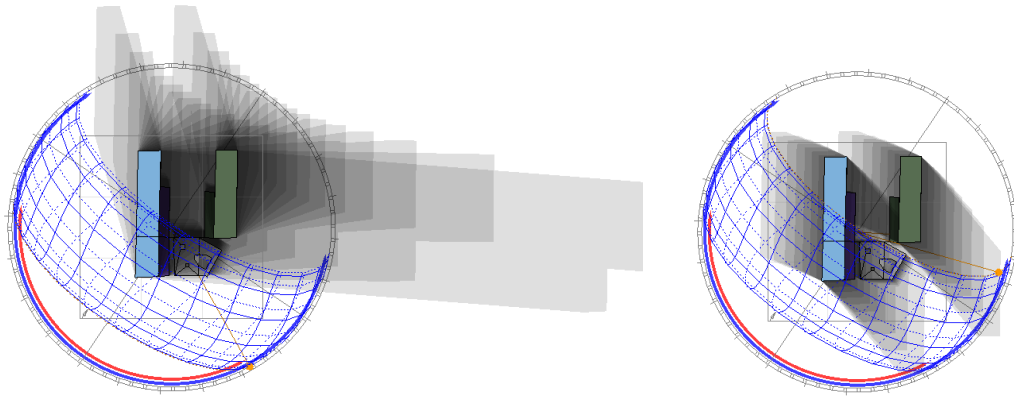


Figure 7-13 Sun path for winter solstice (left) and summer solstice (right)

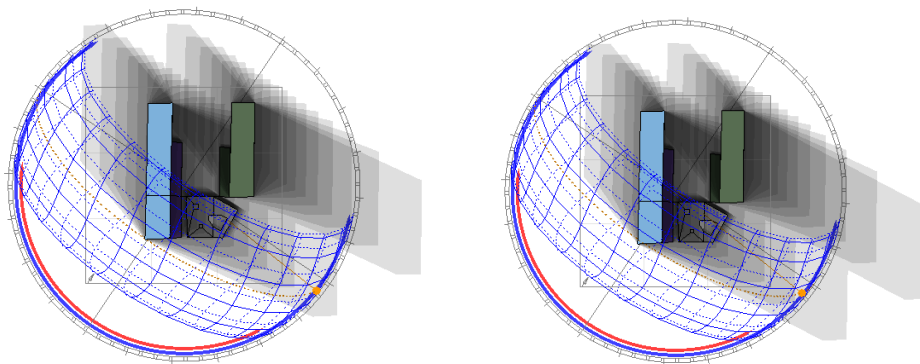


Figure 7-14 Sun path for vernal equinox (left) and autumn equinox (right)

At winter solstice day, it shows that most of museum volume is under the shadow of surrounding buildings. Insufficient direct solar exposure claims adequate insulation demand of the building to stay warm inside, regarding the absence of heating system.

At summer solstice day- the longest day in a hot summer, the sun reaches its highest position at the sky. The large span of skylight roof has a big chance to be exposed under sunlight. This fact requires a suitable action to protect the building from solar heat gain while meanwhile achieve good daylighting. The graph above shows the similarity of sun path situation at vernal equinox and autumn equinox.

7.1.2.2 Global Shadow Analysis

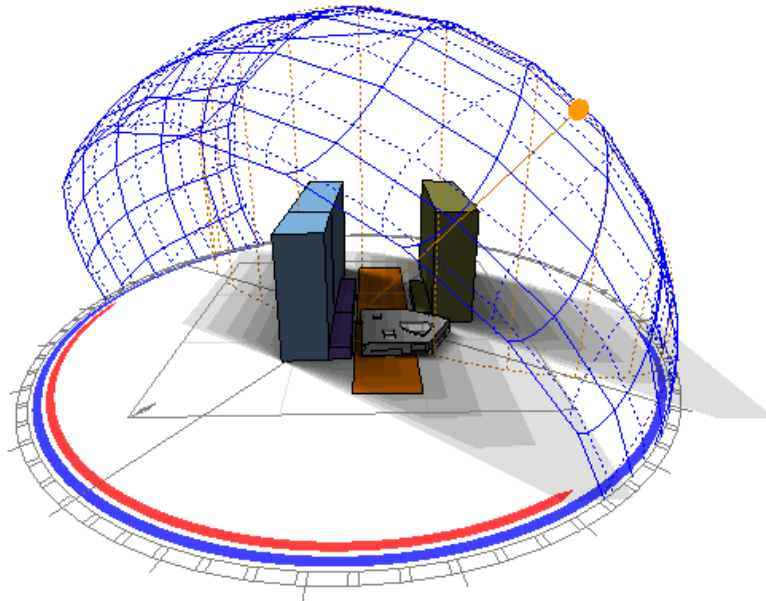


Figure 7-15 Shadow Analysis at hottest day 15-August

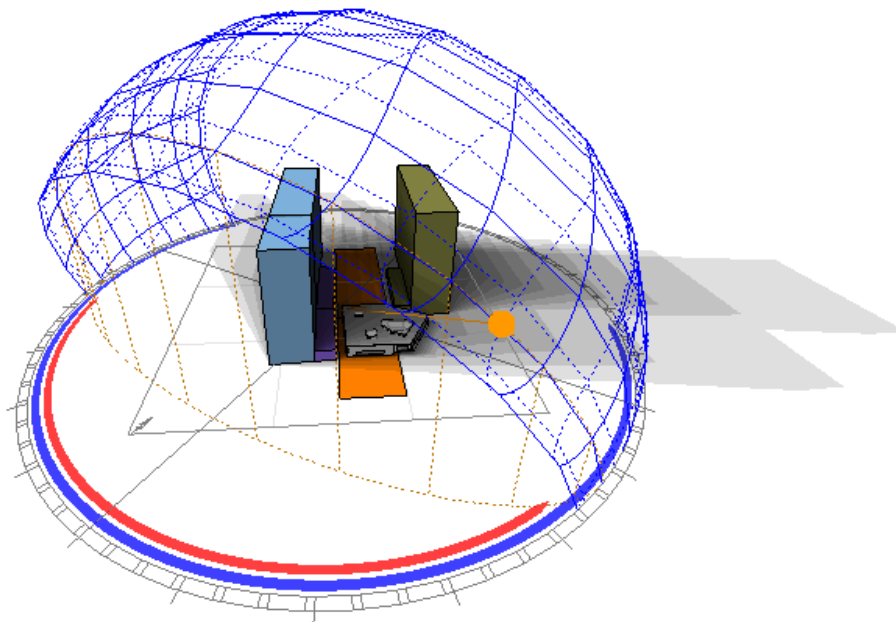
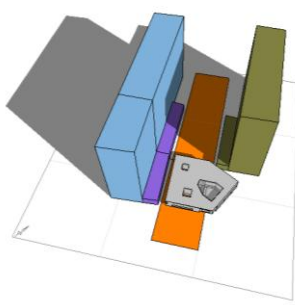


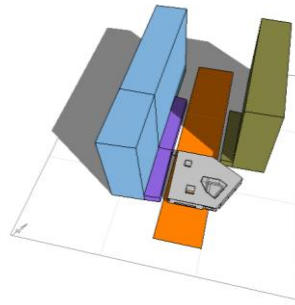
Figure 7-16 Shadow Analysis at coldest day 01-February

Global shadow position is analyzed to determine the location of transparent fenestration.

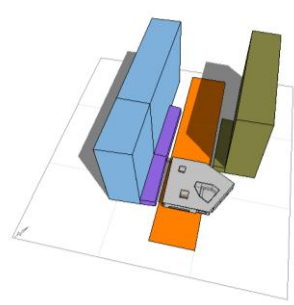
Working hour (8:00 – 18:00) at both hottest day (15-August) and coldest day (01-February) were chosen to do the analysis.



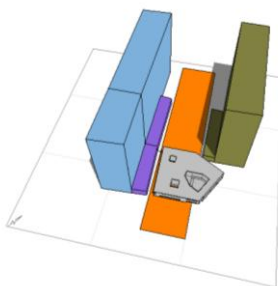
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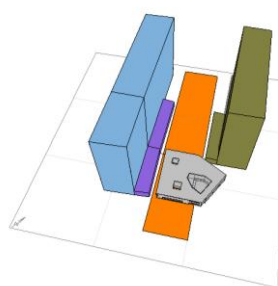
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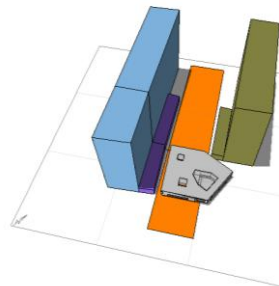
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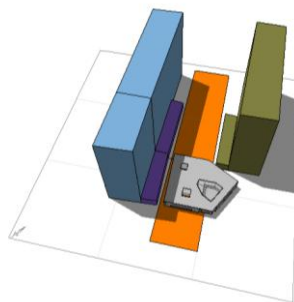
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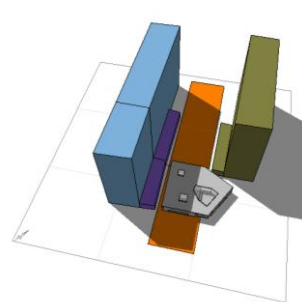
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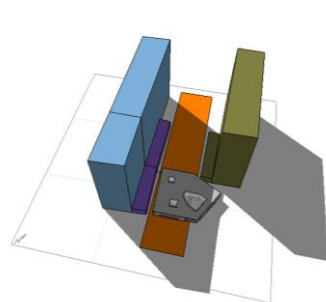
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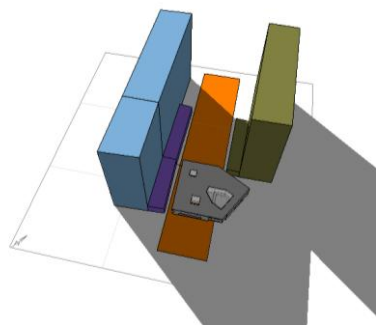
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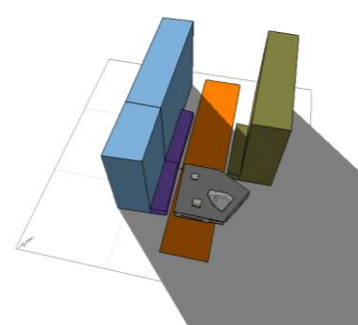
15:00



16:00



17:00



18:00

Figure 7-17 Shadow Display at hottest day 15-August

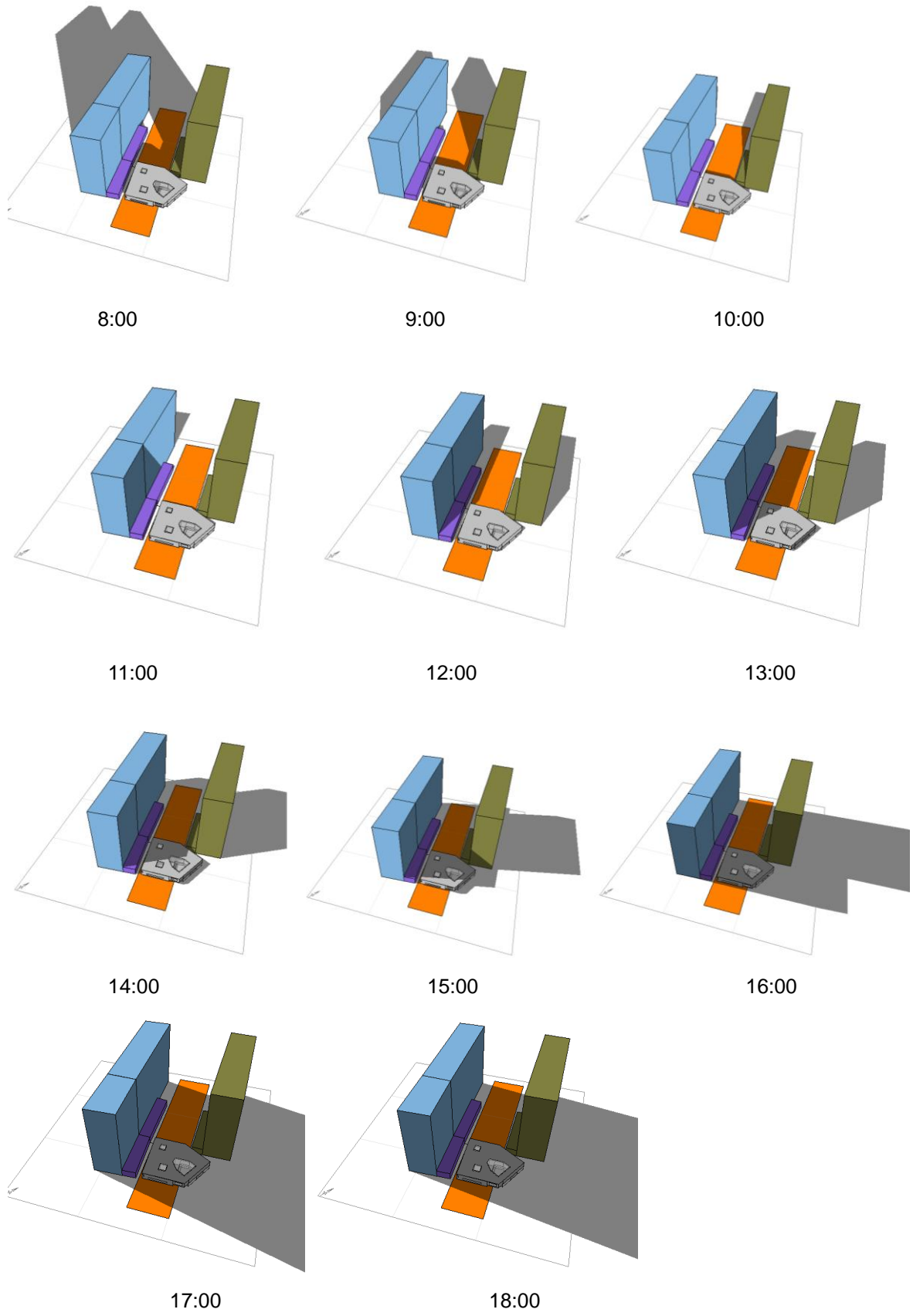


Figure 7-18 Shadow Display at coldest day 01-February

● **Conclusion:**

At hottest day, from 8:00 to 13:00, the museum is exposed at sunlight, large windows at north and south façade of the building shall have necessary shading device to prevent it from over-heating, as well as the big span horizontal rooftop. At coldest day, from 14:00, the shadow from surrounding buildings gradually covers the museum. The upper floor of the building shall get sufficient solar radiation for 4 hours.

7.1.2.3 Daylight Analysis

Daylight factor:

The ratio of internal light level to external light level and is defined as follows:

$$DF = (E_i / E_o) \times 100\%$$

where, E_i = illuminance due to daylight at a point on the indoors working plane,

E_o = simultaneous outdoor illuminance on a horizontal plane from an unobstructed hemisphere of overcast sky.

We use Ecotect program to simulate the day lighting condition. Several parameters were set before performance as follows:

- 1) Sky illuminance were calculated from the latitude of Hong Kong: 11500 lux
- 2) Window clearance: average 0.90

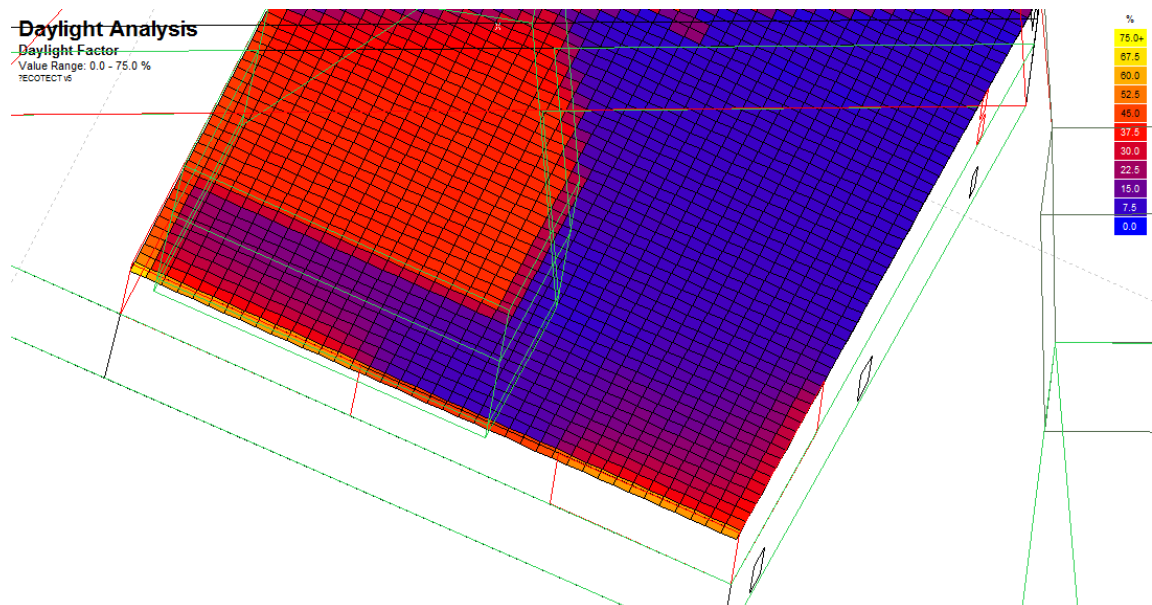


Figure 7-19 Daylight Factor for G/F (lobby and museum café)

The graph shows at G/F, the daylight factor value range from 3.1% to 70% while the atrium area reaches average 50%. Thanks to the skylight and use of glazing façade and lack of obstruction at East

side of the volume, the natural light penetrates into the floor area.

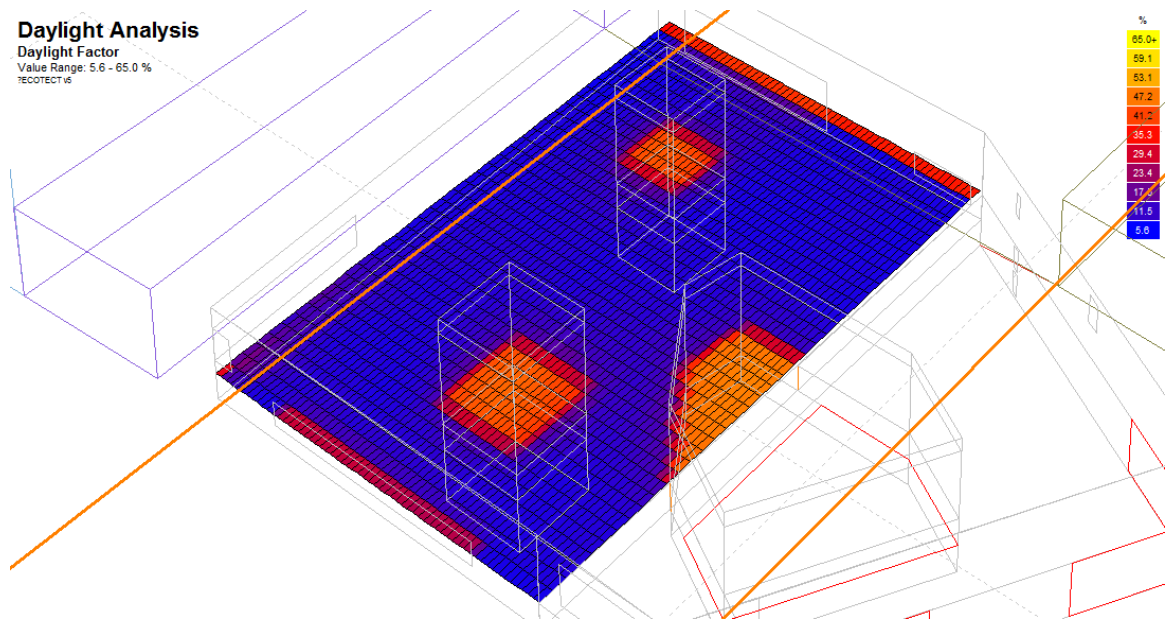


Figure 7-20 Daylight Factor for 1/F (Exhibition, Office and Open Lib)

The graph shows at 1/F, the daylight factor value range from 5.6% to 65% while area around skylight system is average 50%. The natural light depends on mostly the skylight, and the glass window at both viewing terraces at North and South side.

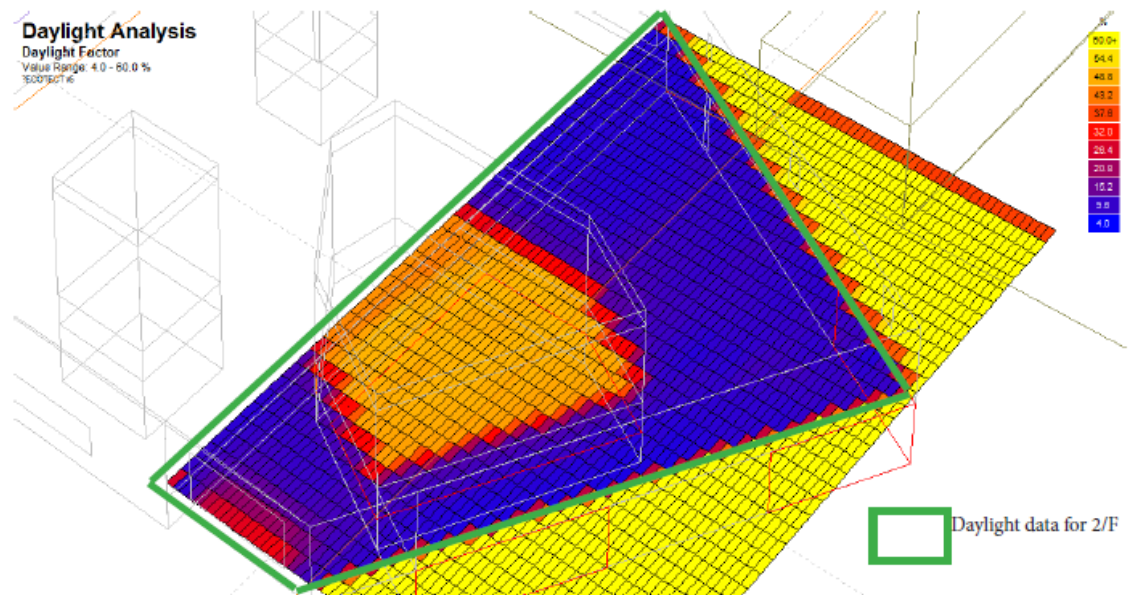


Figure 7-21 Daylight Factor for 2/F (Exhibition and Multifunction Rooms)

The graph shows at 2/F, the daylight factor value range from 4% to 60% while the corridor at 2/F can reach average daylight factor value at 50%. The window at South side brings into natural light while most of it comes from atrium skylight.

7.2 Technical Solutions

7.2.1 Ventilated Façade

Hong Kong is always hot and humid in summer. In the rain screen system, the outer panel deflects rain and solar heat away from the building. The ventilation space allows air to freely circulate behind the panel, creating a wall ventilated and comfortable inner building. A ventilated façade system has minimal thickness and weight, yet it offers maximum performance. The construction principle avoids cold bridge, eliminating condensation and mould growth.

Fiber cement is a composite material made of sand, cement and cellulose fibers. Compared to wooden cladding, fiber cement is not susceptible to termites or rot. It is a non-combustible material and requires little maintenance after installation.

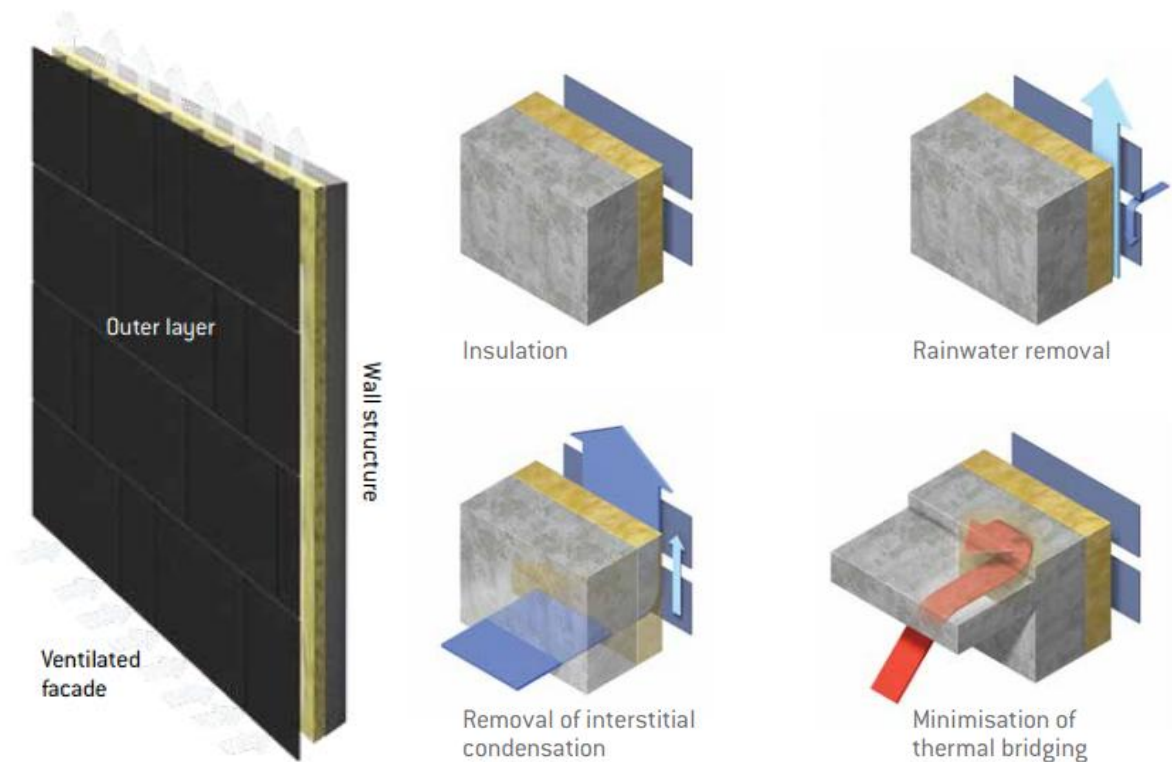


Figure 7-22 Ventilated Facades

7.2.2 Green Roof

Green roofs can make towards creating sustainable living in high-density cities, like Hong Kong. It can not only provide urban dwellers with amenity and recreational space essential for healthy living, but also can filter out fine air particles through vegetation so to improve air quality.

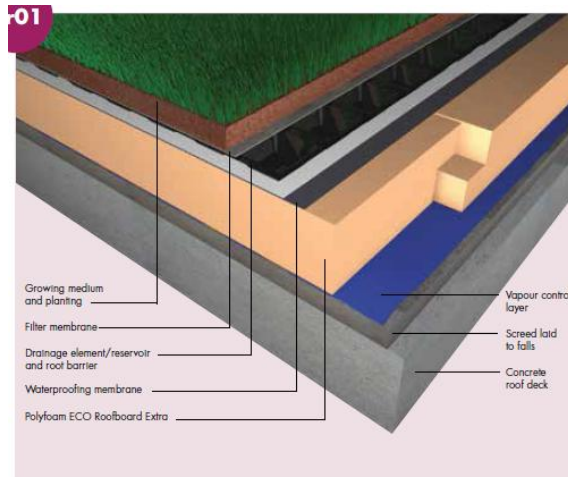


Figure 7-23 Green Roof

7.2.3 Glazing Wall

Curtain wall façades are constituted by an auxiliary structure where infill panels are inserted. The ground floor will be mostly glazing to create a floating sensation.

Double glazing is formed with two panes of glass separated by air gap. It offers more insulation from conductive and convective heat transfer.

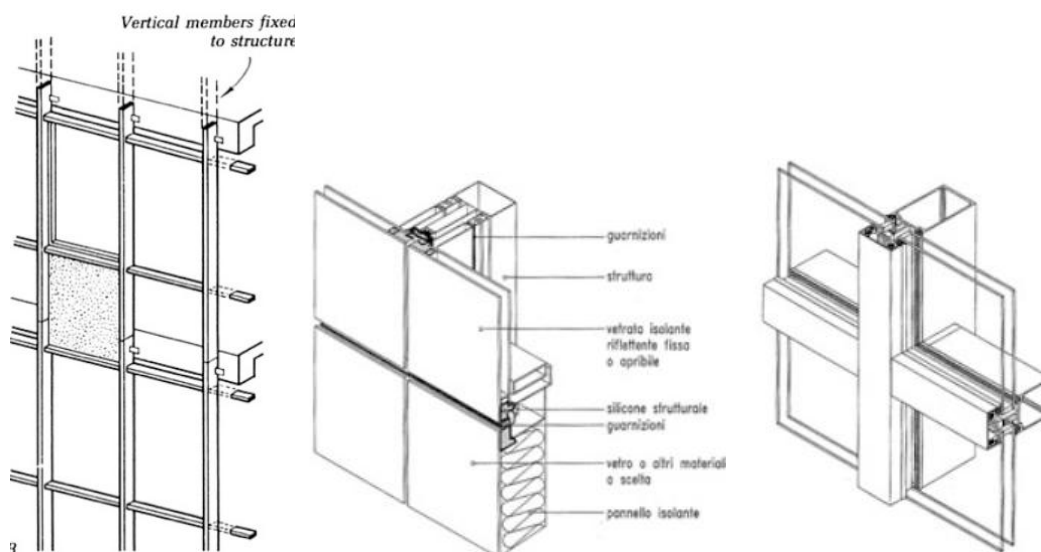


Figure 7-24 Glazing Wall

7.2.4 Skylight

Two forms of skylight system are designed to ensure the adequate amount of natural light for visitors to have a good appreciation to the artifacts.

In east wing of two-floor atrium, fiber cement louvers hang on the top floor beam beneath the glazing frame. It allows the large atrium to be daylit through the glazing cover while prevented the space to be overheated.

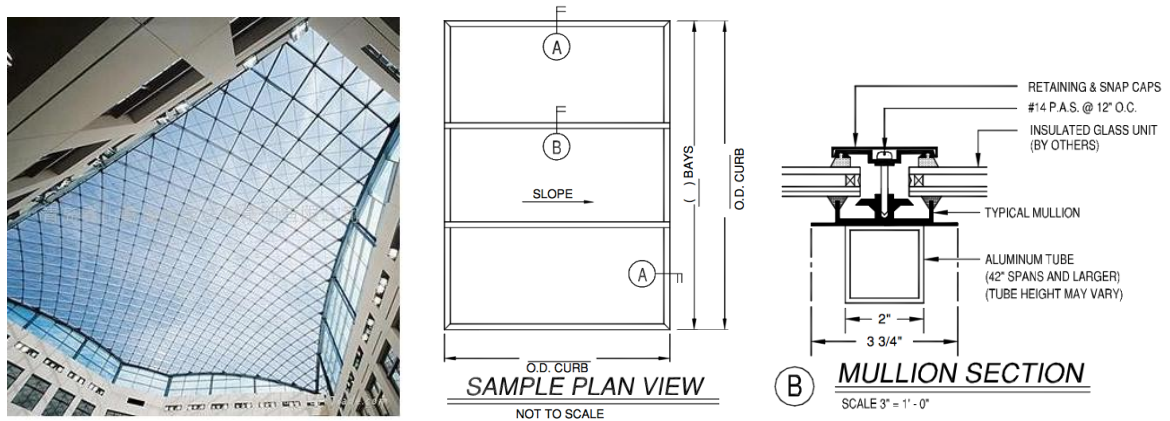


Figure 7-25 Atrium skylight system

At west wing, transparent light guide with prism guide film is designed to introduce the natural light inside the suspending floor and deep to ruins beneath it.

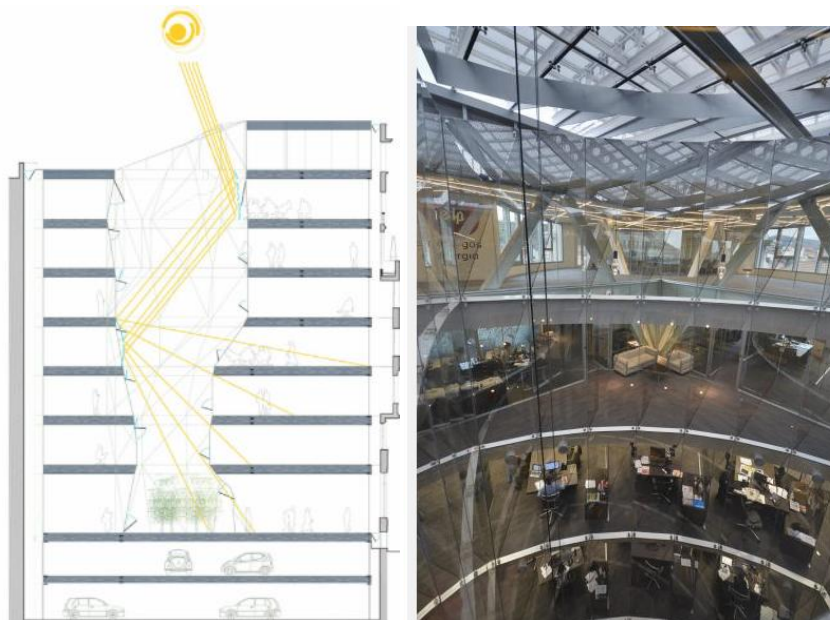


Figure 7-26 Transparent hollow light guides

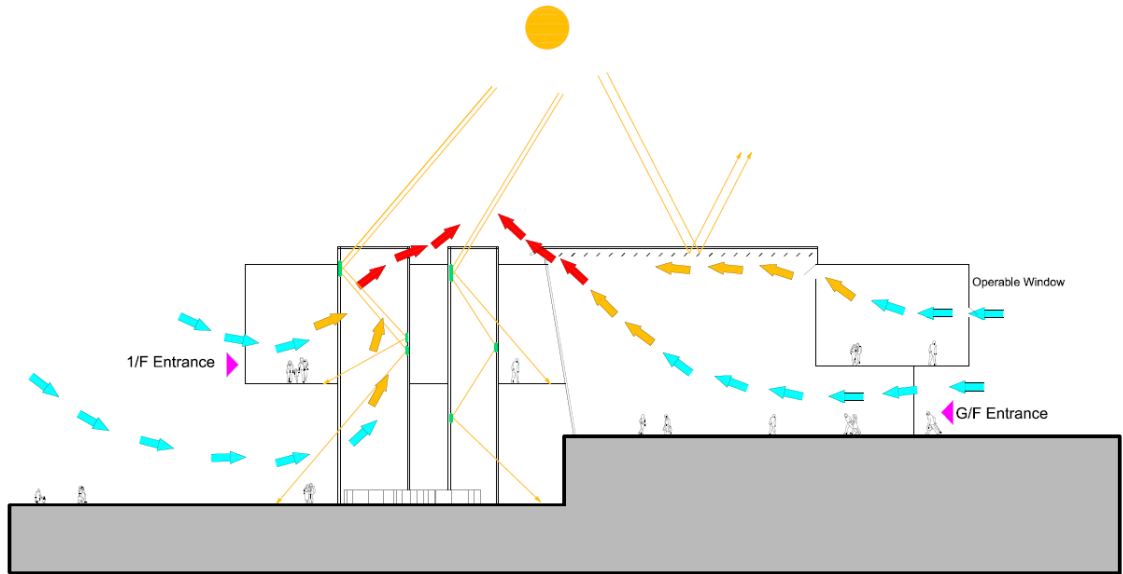


Figure 7-27 Scheme on Daylight and Natural Ventilation in mild seasons

Thus the atrium and skylight can provide daylight and natural ventilation.

7.2.5 Suspended Ceiling

Suspended ceiling is to used to cover duct facilities for air conditioning system.

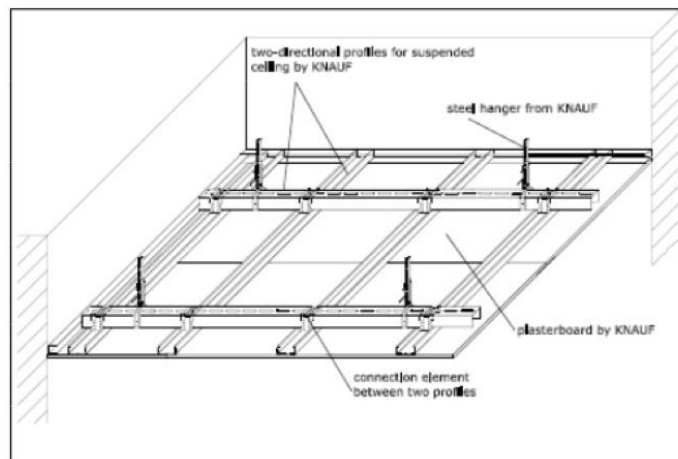


Figure 7-28 Suspended Ceiling

7.2.6 Elements Data

A U value is a measure of heat loss in a building element such as a wall, floor or roof. It can also be referred to as an 'overall heat transfer co-efficient' and measures how well parts of a building transfer heat. This means that the higher the U value the worse the thermal performance of the building envelope. A low U value usually indicates high levels of insulation. They are useful as it is a way of predicting the composite behavior of an entire building element rather than relying on the properties of individual materials.

The U value shall be calculated as:

$$U = \frac{1}{\frac{1}{h_i} + \sum_j \frac{d_j}{\lambda_j} + \frac{1}{h_e}}$$

Where:

hi= internal convective coefficient in m²K/W

he= external convective coefficient in m²K/W

d= total thickness of the component in m

λ= thermal conductivity in W/m k

Thus U value for each building element is calculated as follows.

7.2.6.1 Vertical Elements

Ventilated External Wall				
No.	Layer	λ	d	R
		(W/m k)	(mm)	(m ² k/W)
1	Marly Etemit FC' Fiber Cement Cladding	16.00	12	0.0008
2	Vertical Cladding Rail Substructure	-	26	-
3	Ventilation Cavity	5.20	30	0.0058
4	Plastic Vapour Barrier	0.16	5	0.0313
5	Extruded Rigid Polystyrene Insulation	0.04	100	2.5000
6	Fiber Cement Board	16.00	8	0.0005
7	Rockwool Insulation	0.04	100	2.5000
8	Vapour Barrier	0.50	3	0.0060
9	Double Plasterboard	0.16	24	0.1500
			ΣR	5.1943
			1/hi	0.13
			1/he	0.04
			U(W/m ² k)	0.186

Internal Wall Dry/Dry - 150				
No.	Layer	λ	d	R
		(W/m k)	(mm)	(m ² k/W)
1	Double Plasterboard	0.16	24	0.1500
2	Rockwool Insulation	0.04	100	2.5000
3	Double Plasterboard	0.16	24	0.1500
			ΣR	2.8000
			1/hi	0.13
			1/he	0.04
			U(W/m ² k)	0.337

Internal Wall Dry/Dry - 100				
No.	Layer	λ	d	R
		(W/m k)	(mm)	(m ² k/W)
1	Double Plasterboard	0.16	24	0.1500
2	Rockwool Insulation	0.04	50	1.2500
3	Double Plasterboard	0.16	24	0.1500
			ΣR	1.5500
			1/h _i	0.13
			1/h _e	0.04
			U(W/m ² k)	0.581

Internal Wall Wet/Dry - 150				
No.	Layer	λ	d	R
		(W/m k)	(mm)	(m ² k/W)
1	Ceramic Tile with Adhesive	0.84	8	0.0095
2	Backer Board	0.16	13	0.0813
3	Vapour Barrier	0.50	5	0.0100
4	Rockwool Insulation	0.04	100	2.5000
5	Double Plasterboard	0.16	24	0.1500
			ΣR	2.7508
			1/h _i	0.13
			1/h _e	0.04
			U(W/m ² k)	0.342

Internal Wall Wet/Wet - 100				
No.	Layer	λ	d	R
		(W/m k)	(mm)	(m ² kW)
1	Ceramic Tile with Adhesive	0.84	8	0.0095
2	Backer Board	0.16	13	0.0813
3	Rockwool Insulation	0.04	100	2.5000
4	Backer Board	0.16	13	0.0813
5	Ceramic Tile with Adhesive	0.84	8	0.0095
			ΣR	2.6815
			1/h _i	0.13
			1/h _e	0.04
			U(W/m ² k)	0.351

7.2.6.2 Horizontal Elements

Ground Floor				
No	Layer	λ	d	R
		(W/m k)	(mm)	(m ² kW)
1	Recycled Carpet Tile and Adhesive Layer	0.055	10	0.1818
2	Screed Layer	0.750	30	0.0400
3	Separation Layer PET	0.270	5	0.0185
4	Extruded Polystyrene Insulation	0.035	150	4.2857
5	Waterproofing/Separating Membrane	0.270	5	0.0185
6	Reinforced Concrete	0.420	120	0.2857
7	Gravel	-	-	-
			ΣR	4.8303
			1/h _i	0.13
			1/h _e	0.04
			U(W/m ² k)	0.200

1/F Typical Floor				
No	Layer	λ	d	R
		(W/m k)	(mm)	(m ² k/W)
1	Recycled Carpet Tile and Adhesive Layer	0.055	10	0.1818
2	Screed Layer	0.750	30	0.0400
3	Separation Layer PET	0.270	5	0.0185
4	Extruded Polystyrene Insulation	0.035	100	2.8571
5	Trickle Protection Layer	0.270	10	0.0370
6	RC Layer and Galvanized Steel Deck	0.420	150	0.3571
7	Steel Beam	-	-	-
8	Extruded Polystyrene Insulation	0.035	100	2.8571
9	Trickle Protection Layer	0.270	10	0.0370
10	Waterproofing Membrane	0.270	5	0.0185
11	Fibercement Board	0.160	12	0.0750
			ΣR	6.4794
			1/h _i	0.13
			1/h _e	0.04
			U(W/m ² k)	0.150

2/F Typical Floor

No	Layer	λ	d	R
		(W/m k)	(mm)	(m ² k/W)
1	Recycled Carpet Tile and Adhesive Layer	0.055	10	0.1818
2	Screed Layer	0.750	30	0.0400
3	Separation Layer PET	0.270	5	0.0185
4	Extruded Polystyrene Insulation	0.035	100	2.8571
5	Trickle Protection Layer	0.270	5	0.0185
6	RC Layer and Galvanized Steel Deck	0.420	150	0.3571
7	Steel Beam	-	-	-
8	Service Gap	5.200	500	0.0962
9	Knauf' Aluminium Suspended Ceiling Hanger	-	-	-
10	Rockwool Insulation	0.035	30	0.8571
11	Fire Resistant Plasterboard	0.160	12	0.0750
			ΣR	4.5014
			1/h _i	0.13
			1/h _e	0.04
			U(W/m ² k)	0.214

Green Roof				
No	Layer	λ	d	R
		(W/m k)	(mm)	(m ² k/W)
1	Growing Medium and Planting			
2	Filter Membrane			
3	Drainage Element/Reservior and Root Barrier			
4	Polyfoam Slimline Membrane	0.270	10	0.0370
5	Polyfoam Eco Roofboard Extra/Super	0.032	200	6.2500
6	Mastic Asphalt Waterproof Membrane on Isolating Layer			
7	Screed Laid to Falls	0.750	63	0.0840
8	Concrete Roof Deck	0.420	150	0.3571
9	Steel Beam			
10	Service Gap			
11	Knauf' Aluminium Suspended Ceiling Hanger			
12	Rockwool Insulation	0.035	30	0.8571
13	Fire Resistant Board Layer	0.160	12	0.0750
			ΣR	7.6603
			1/h _i	0.13
			1/h _e	0.04
			U(W/m ² k)	0.128

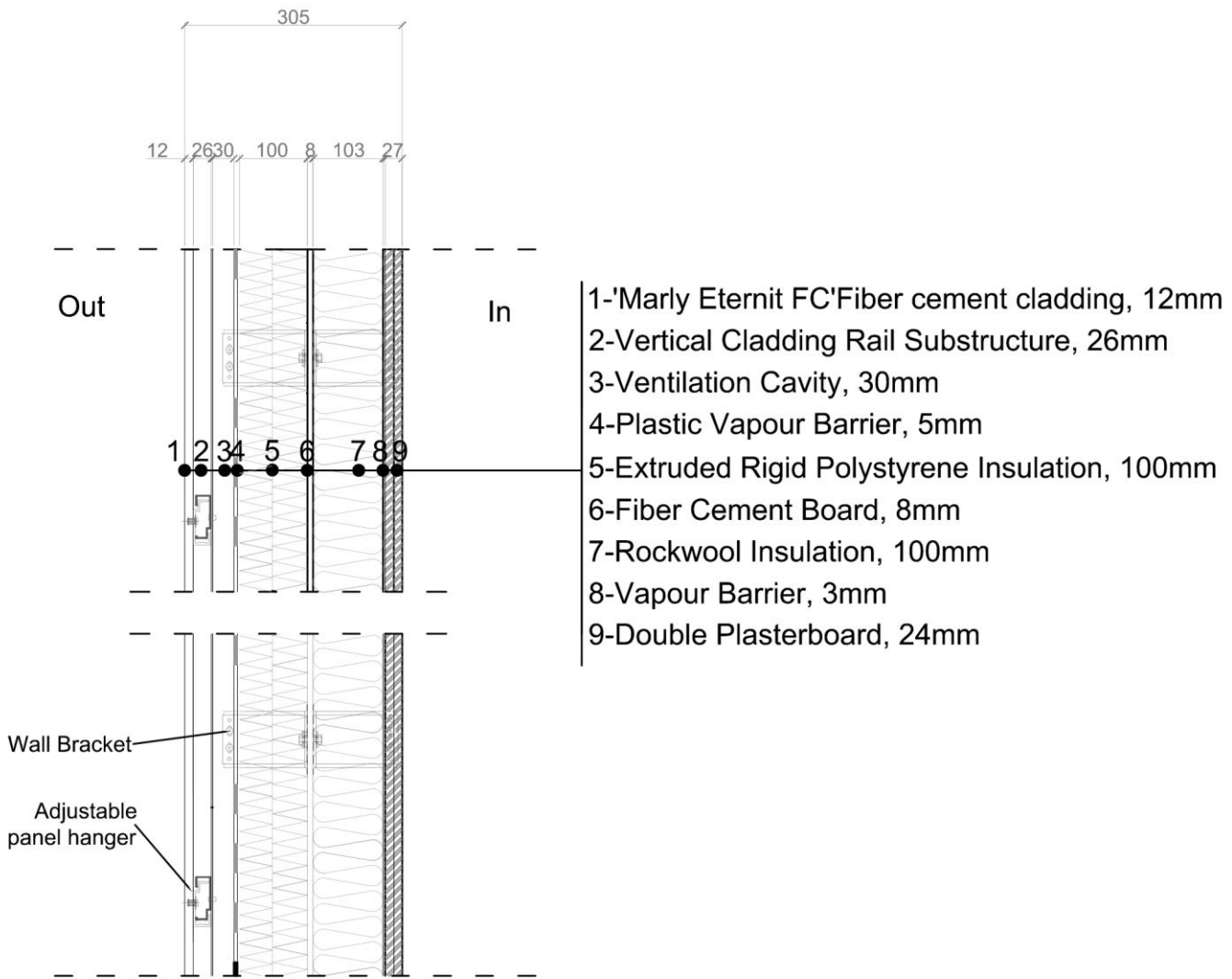
- **Summary**

Component		U-value (W/m ² K)
Vertical	Ventilated External Wall	0.19
	Schuco Glazing Façade	0.80
	Schuco Window	0.90
	Internal Wall Dry/Dry-150	0.34
	Internal Wall Dry/Dry-100	0.58
	Internal Wall Dry/Wet-150	0.34
	Internal Wall Wet/Wet-150	0.35
Horizontal	Green Roof	0.13
	2/F	0.21
	1/F	0.15
	G/F	0.20

7.2.7 Detail Drawings

Ventilated External Wall (section)

U-value=0.186 w/m2K



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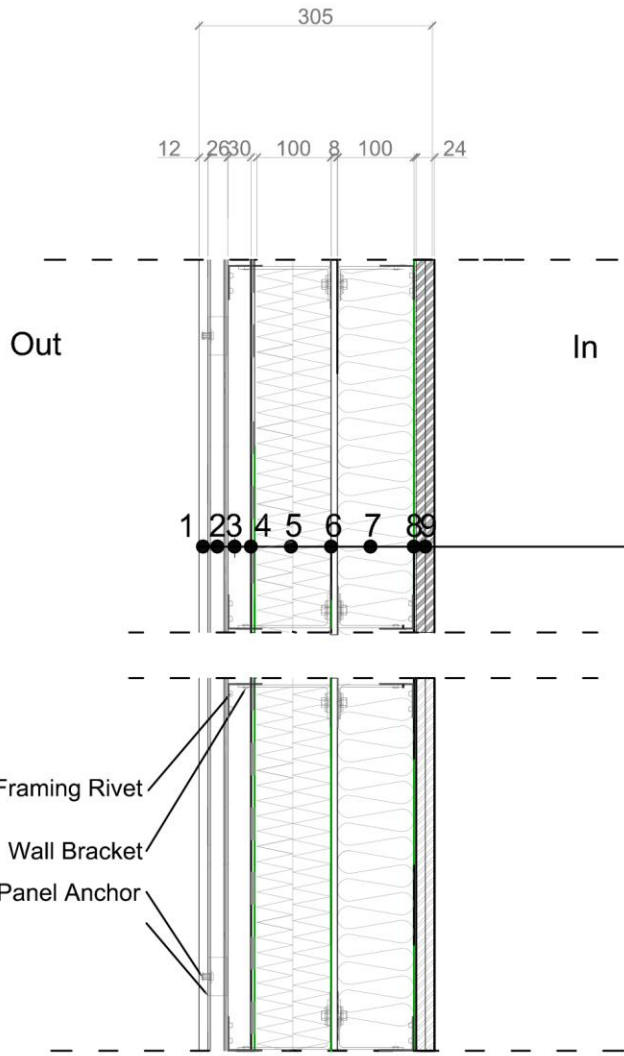
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 LUNG TSUN STONE BRIDGE ARCHAEOLOGY MUSEUM
 HONG KONG - CHINA

Drawing Code
 Technology - Material - External wall
 T 101


Drawing Scale
 1: 10

Ventilated External Wall (Plan)

U-value=0.186 w/m²K

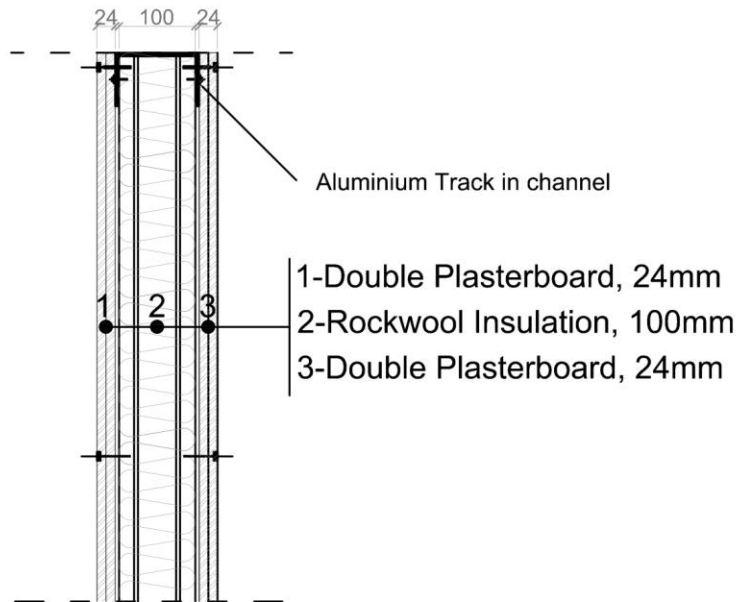


- 1-'Marly Eternit FC' Fiber cement cladding, 12mm
- 2-Vertical Cladding Rail Substructure, 26mm
- 3-Ventilation Cavity, 30mm
- 4-Plastic Vapour Barrier, 5mm
- 5-Extruded Rigid Polystyrene Insulation, 100mm
- 6-Fiber Cement Board, 8mm
- 7-Rockwool Insulation, 100mm
- 8-Vapour Barrier, 3mm
- 9-Double Plasterboard, 24mm

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<p>Final Submission 26-11-2014</p>	<p>PROJECT LUNG TSUN STONE BRIDGE ARCHAEOLOGY MUSEUM HONG KONG - CHINA</p>		<p>Drawing Code Technology - Material - External wall T 102</p> <p>Drawing Scale 1: 10</p>

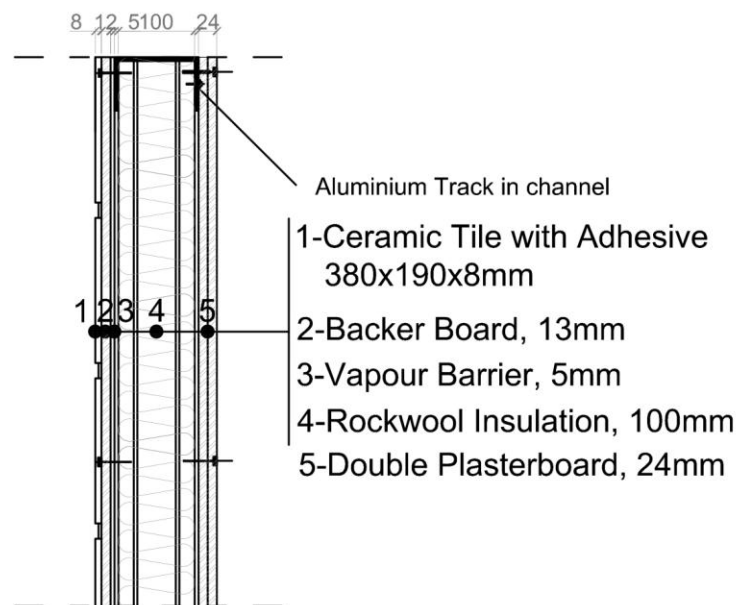
Internal Wall Dry/Dry -150 (Section)

U-value=0.337 w/m²K



Internal Wall Wet/Dry -150 (Section)

U-value=0.342 w/m²K



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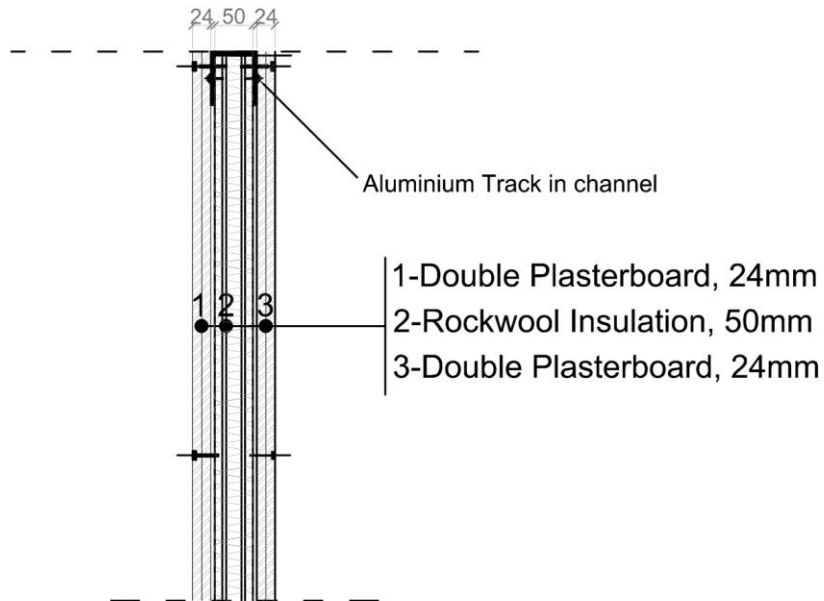
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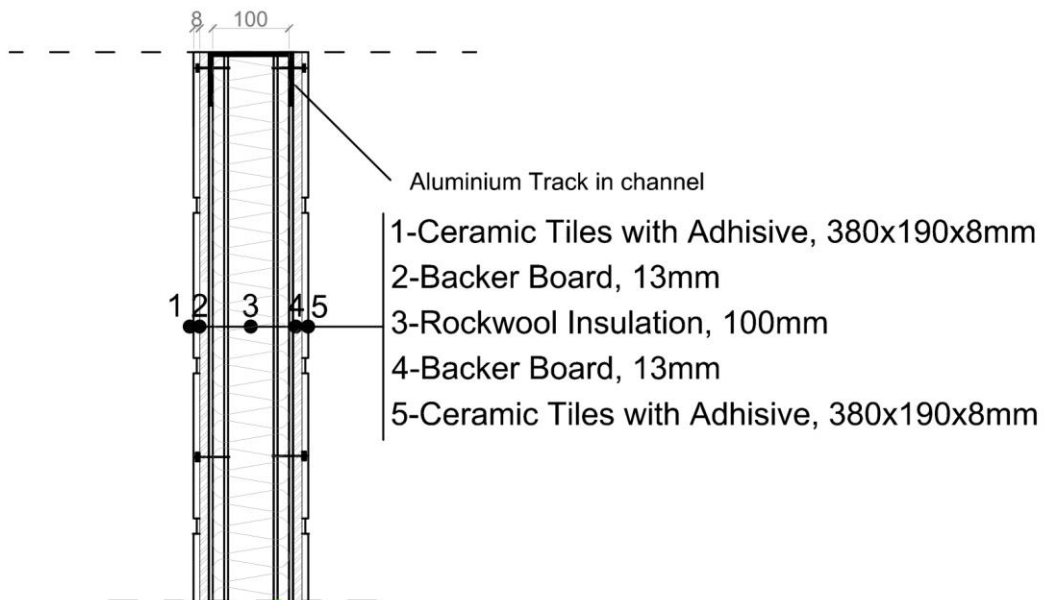
Drawing Code
Technology - Material - Internal wall
T 103

Drawing Scale
1: 10

Internal Wall Dry/Dry -100 (Section)
 U-value=0.581 w/m²K



Internal Wall Wet/Wet -150 (Section)
 U-value=0.351 w/m²K



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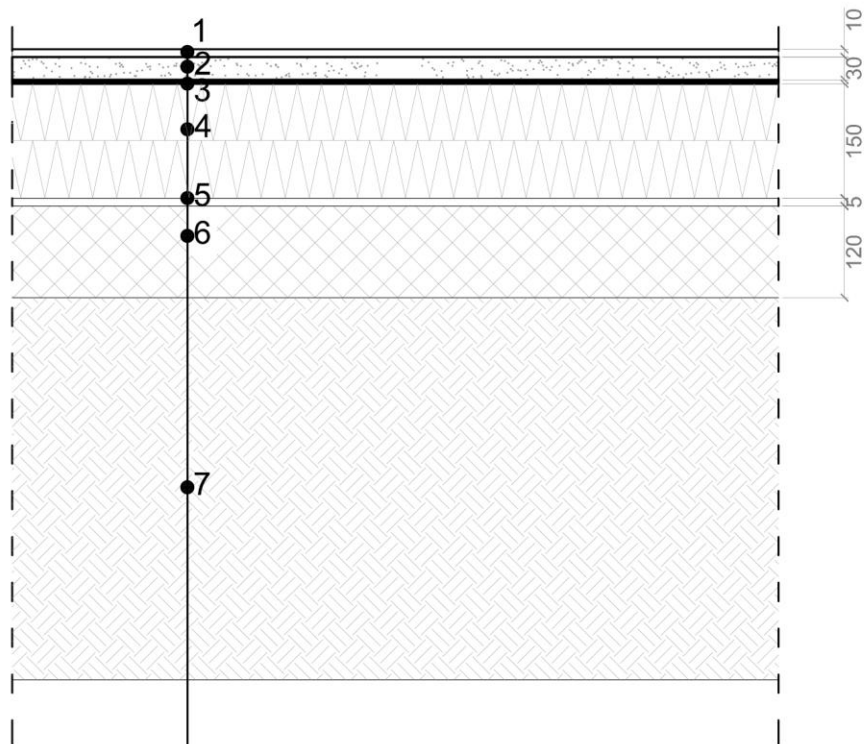
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 Tecnology - Material - Internal wall
 T 104

Drawing Scale
 1: 10

Ground Floor (Section)
 U-value=0.200w/m2K



- 1-Recycled Carpet Tile and Adhesive Layer, 10mm
- 2-Screed Layer, 30mm
- 3-Separation Layer PET, 5mm
- 4-Extruded Polystyrene Insulation, 150mm
- 5-Waterproofing/Separating Membrane, 5mm
- 6-Reinforced Concrete, 120mm
- 7-Earth



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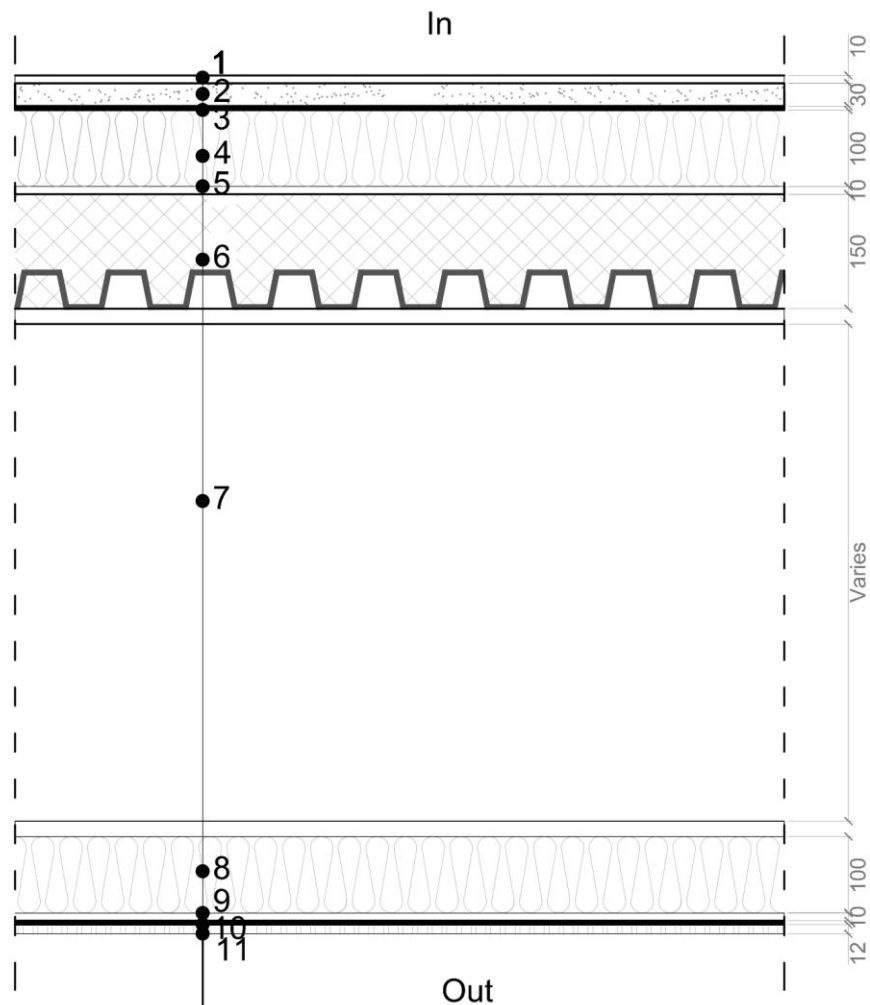
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Drawing Code
 Technology - Material - Ground Floor
 T 201

Drawing Scale
 1: 10

1/F Floor (Section)
 U-value=0.15 w/m²K



- 1-Recycled Carpet Tile and Adhesive Layer, 10mm
- 2-Screed Layer, 30mm
- 3-Separation Layer PET, 5mm
- 4-Extruded Polystyrene Insulation, 100mm
- 5-Trickle Protection Layer, 10mm
- 6-RC Layer and Galvanized Steel deck, 150mm
- 7-Steel Beam
- 8-Extruded Polystyrene Insulation, 100mm
- 9-Tricle Protection Layer, 10mm
- 10-Vapour Barrier, 5mm
- 11-Fibercement Board, 12mm



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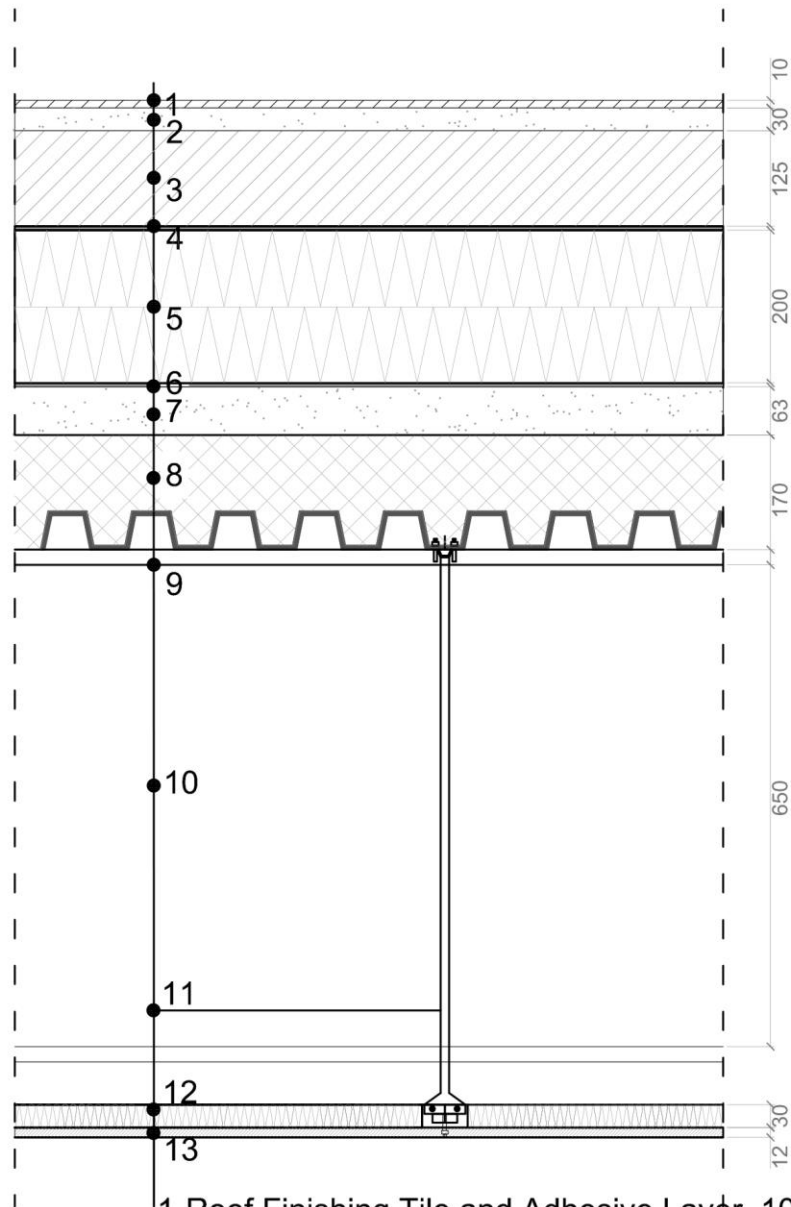


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Drawing Code
 Tecnology - Material - 1 Floor
 T 202

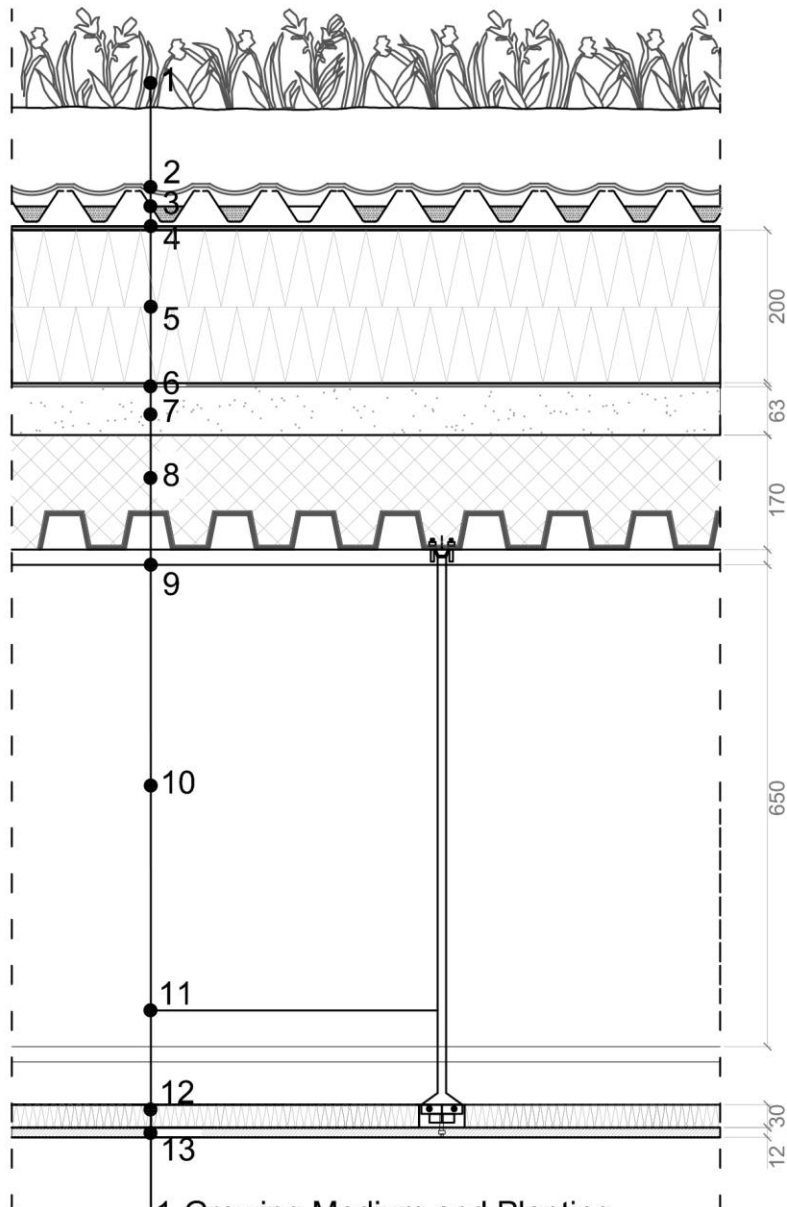
Drawing Scale
 1: 10



- 1-Roof Finishing Tile and Adhesive Layer, 10mm
- 2-Screed Layer, 30mm
- 3-Foamglass sloping board, 125mm
- 4-Polyfoam Slimline Membrane
- 5-Polyfoam Eco Roofboard Extra/Super, 200m, U-value=0.16w/m2k
- 6-Mastic Asphalt Waterproof Membrane on Isolating Layer
- 7-Screed Laid to Falls, 63mm
- 8-Concrete Roof Deck, 150mm
- 9-Steel Beam
- 10-Service Gap
- 11-'Knauf' Aluminium Suspended Ceiling Hanger
- 12-Rock Wool Insulation, 30mm
- 13-Fire Resistant Board Layer, 12mm

Roof (Section)
 U-value=0.128 w/m2K

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- 1-Growing Medium and Planting
- 2-Filter Membrane
- 3-Drainage Element/Reservoir and Root Barrier
- 4-Polyfoam Slimline Membrane
- 5-Polyfoam Eco Roofboard Extra/Super, 200m, U-value=0.16w/m2k
- 6-Mastic Asphalt Waterproof Membrane on Isolating Layer
- 7-Screed Laid to Falls, 63mm
- 8-Concrete Roof Deck, 150mm
- 9-Steel Beam
- 10-Service Gap
- 11-'Knauf' Aluminium Suspended Ceiling Hanger
- 12-Rock Wool Insulation, 30mm
- 13-Fire Resistant Board Layer, 12mm

Green Roof (Section)
U-value=0.128 w/m2K



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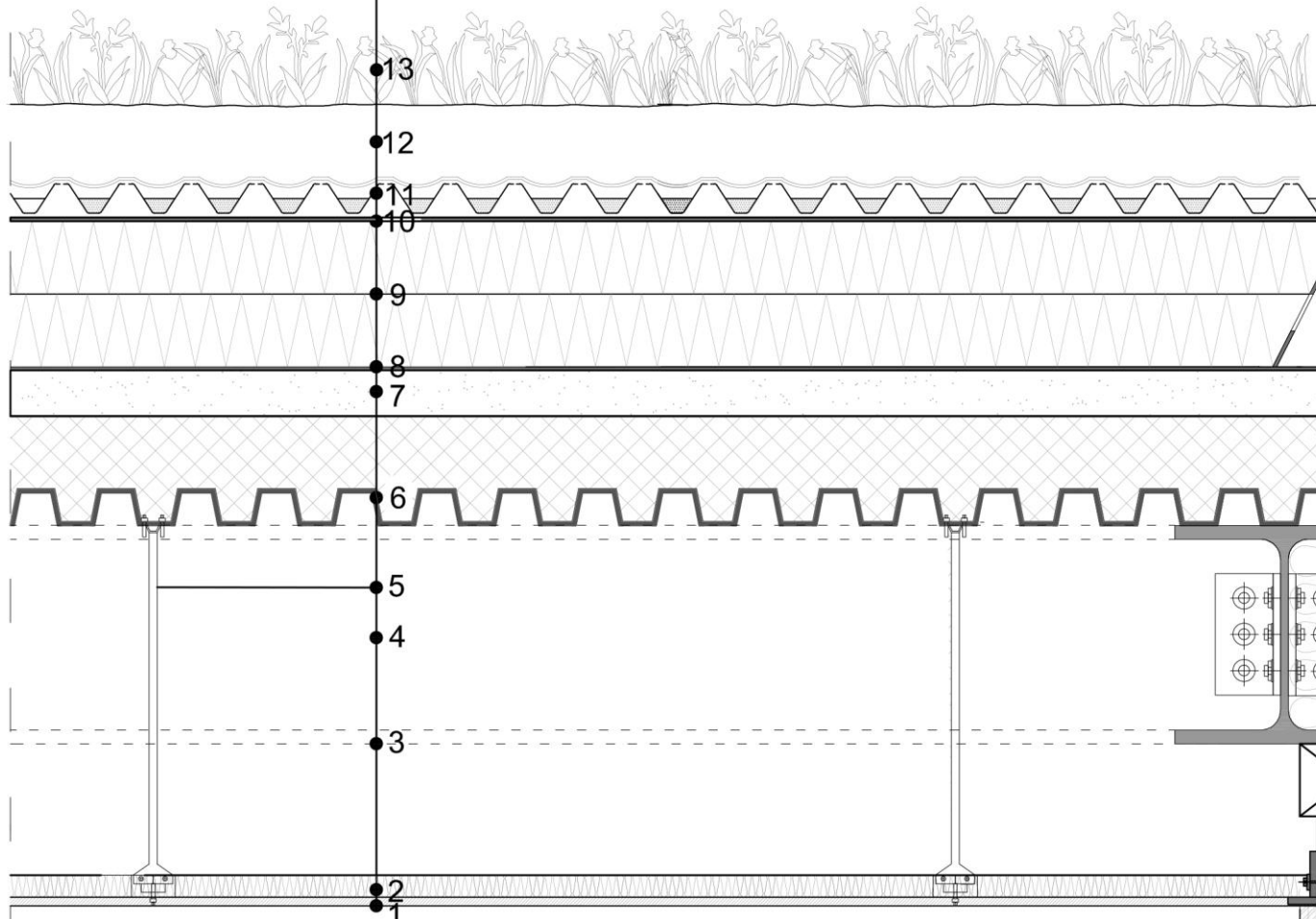
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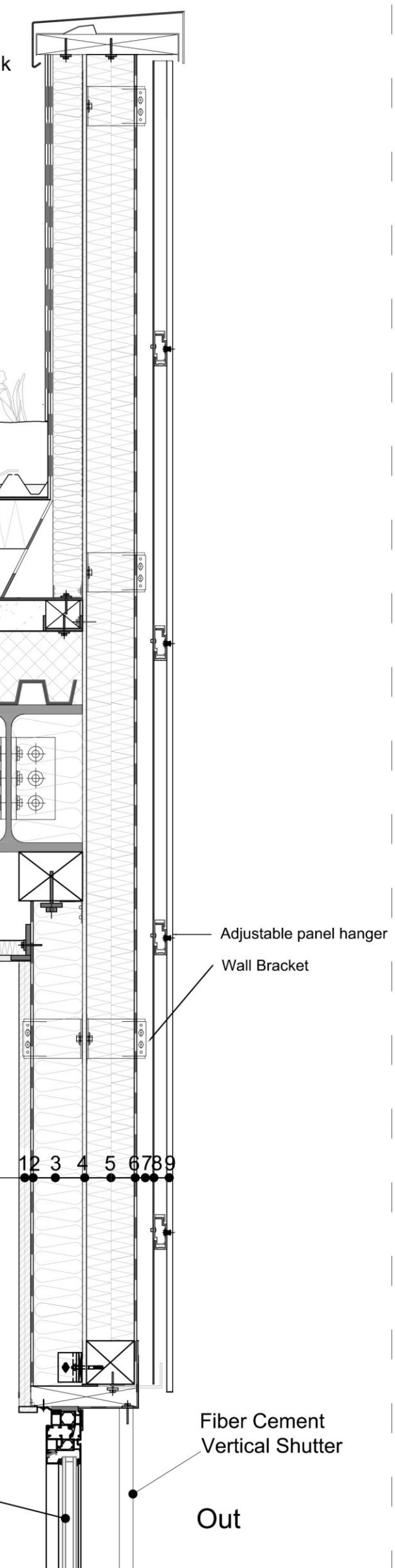
Drawing Code
 Tecnology - Material - Green Roof
 T 302

Drawing Scale
 1: 10

- 13-Growing Medium and Planting
- 12-Filter Membrane
- 11-Drainage Element/Reservoir and Root Barrier
- 10-Polyfoam Slimline Membrane
- 9-Polyfoam Eco Roofboard Extra/Super, 200mm, U-value=0.16w/m2k
- 8-Mastic Asphalt Waterproof Membrane on Isolating Layer
- 7-Screed Laid to Falls, 63mm
- 6-Concrete Roof Deck, 150mm
- 5-'Knauf' Aluminium Suspended Ceiling Hanger
- 4-Service Gap
- 3-Steel Beam
- 2-Rock Wool Insulation, 30mm
- 1-Fire Resistant Board Layer, 12mm



- 1-Double Plasterboard, 24mm
- 2-Vapour Barrier Layer, 3mm
- 3-Rockwool Insulation, 100mm
- 4-Fiber Cement Board, 8mm
- 5-Extruded Rigid Polystyrene Insulation, 100mm
- 6-Plastic Vapour Barrier, 3mm
- 7-Ventilation Cavity, 30mm
- 8-Vertical Cladding Rail Substructure, 26mm
- 9-'Marly Eternit FC' Fiber cement cladding, 12mm



Schuco Facade
VISS-TVS
U-value=0.80 w/m2K

In

Fiber Cement
Vertical Shutter

Out

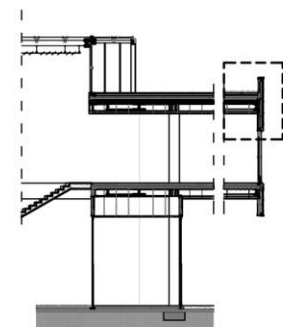


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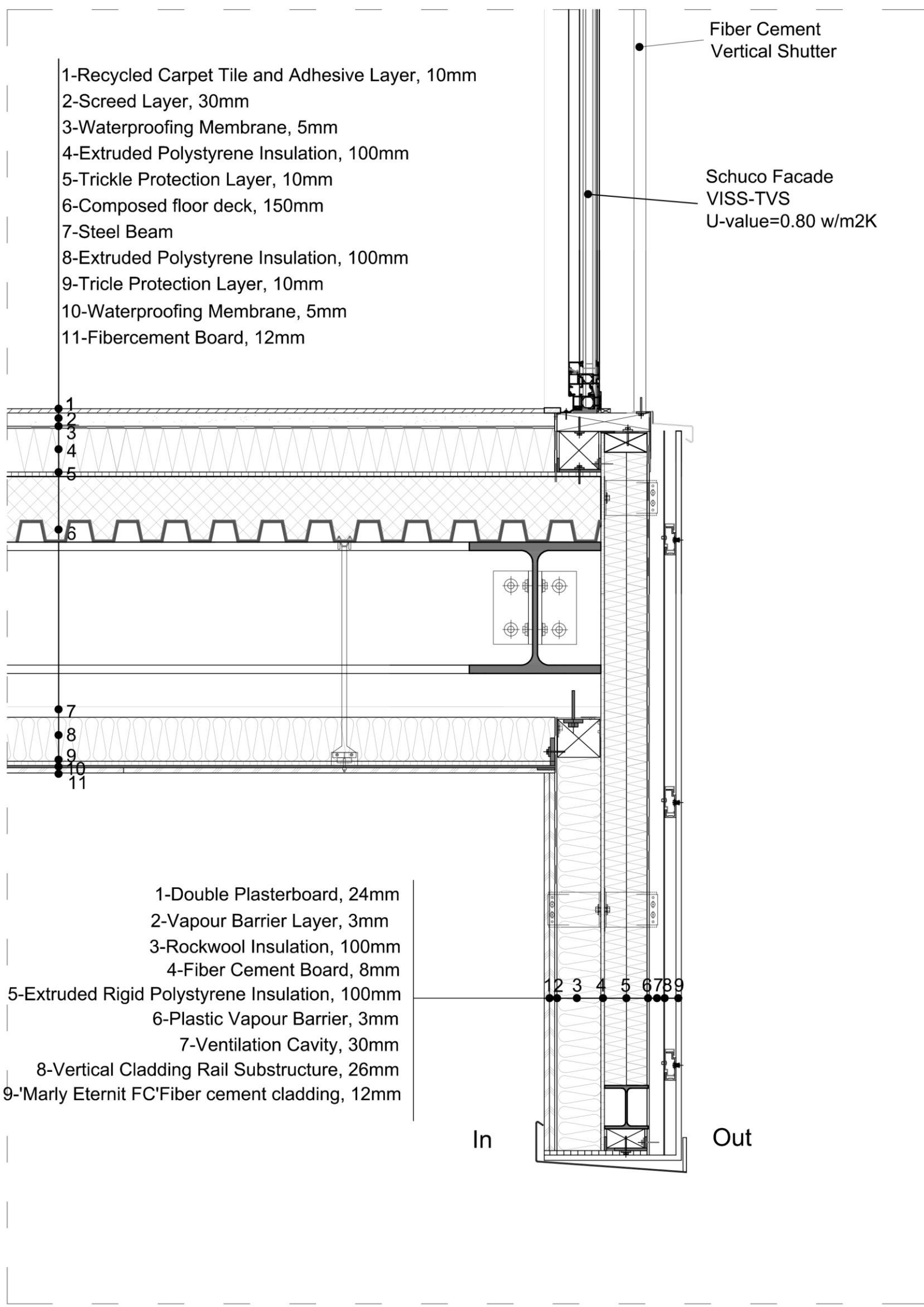
PROJECT
LUNG TSUN STONE BRIDGE ARCHAEOLOGY MUSEUM
HONG KONG - CHINA

AUTHORS
SHUFEI CHEN
MENGYANG LIN
JIE YANG



Drawing Code
Technology - Joint 1
T 401

Drawing Scale
1: 10



- 1-Recycled Carpet Tile and Adhesive Layer, 10mm
- 2-Screed Layer, 30mm
- 3-Waterproofing Membrane, 5mm
- 4-Extruded Polystyrene Insulation, 100mm
- 5-Trickle Protection Layer, 10mm
- 6-Composed floor deck, 150mm
- 7-Steel Beam
- 8-Extruded Polystyrene Insulation, 100mm
- 9-Tricle Protection Layer, 10mm
- 10-Waterproofing Membrane, 5mm
- 11-Fibercement Board, 12mm

Fiber Cement Vertical Shutter

Schuco Facade VISS-TVS
U-value=0.80 w/m2K

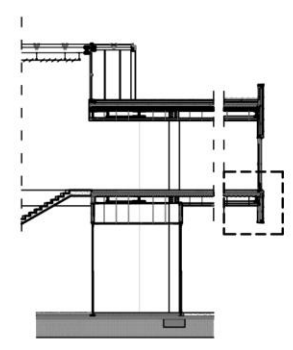
- 1-Double Plasterboard, 24mm
- 2-Vapour Barrier Layer, 3mm
- 3-Rockwool Insulation, 100mm
- 4-Fiber Cement Board, 8mm
- 5-Extruded Rigid Polystyrene Insulation, 100mm
- 6-Plastic Vapour Barrier, 3mm
- 7-Ventilation Cavity, 30mm
- 8-Vertical Cladding Rail Substructure, 26mm
- 9-'Marly Eternit FC' Fiber cement cladding, 12mm

In Out



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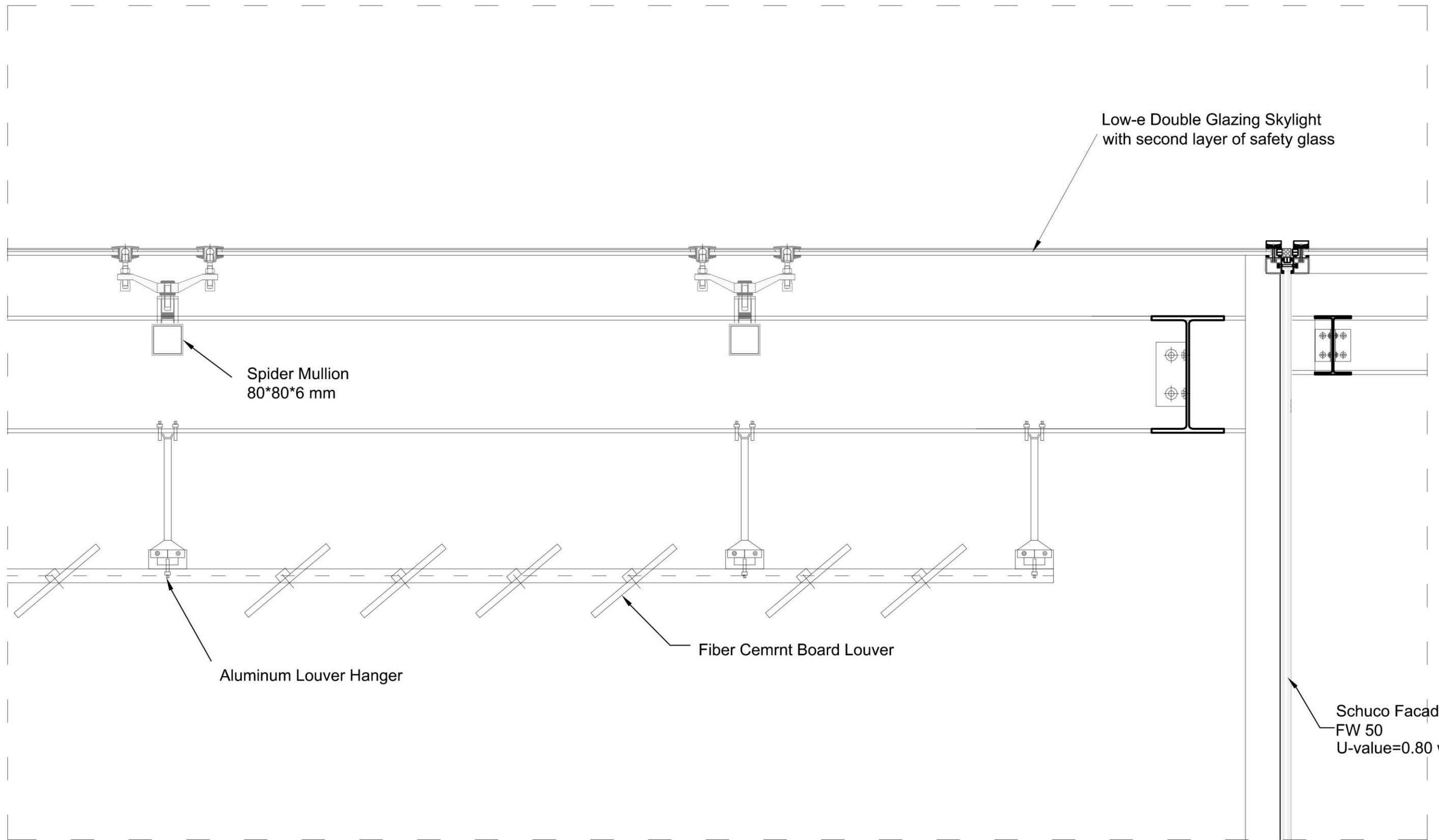


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Drawing Scale
1: 10



Low-e Double Glazing Skylight
with second layer of safety glass

Spider Mullion
80*80*6 mm

Aluminum Louver Hanger

Fiber Cemrnt Board Louver

Schuco Facade
FW 50
U-value=0.80 w/m2K

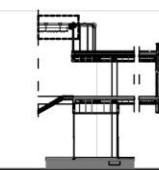


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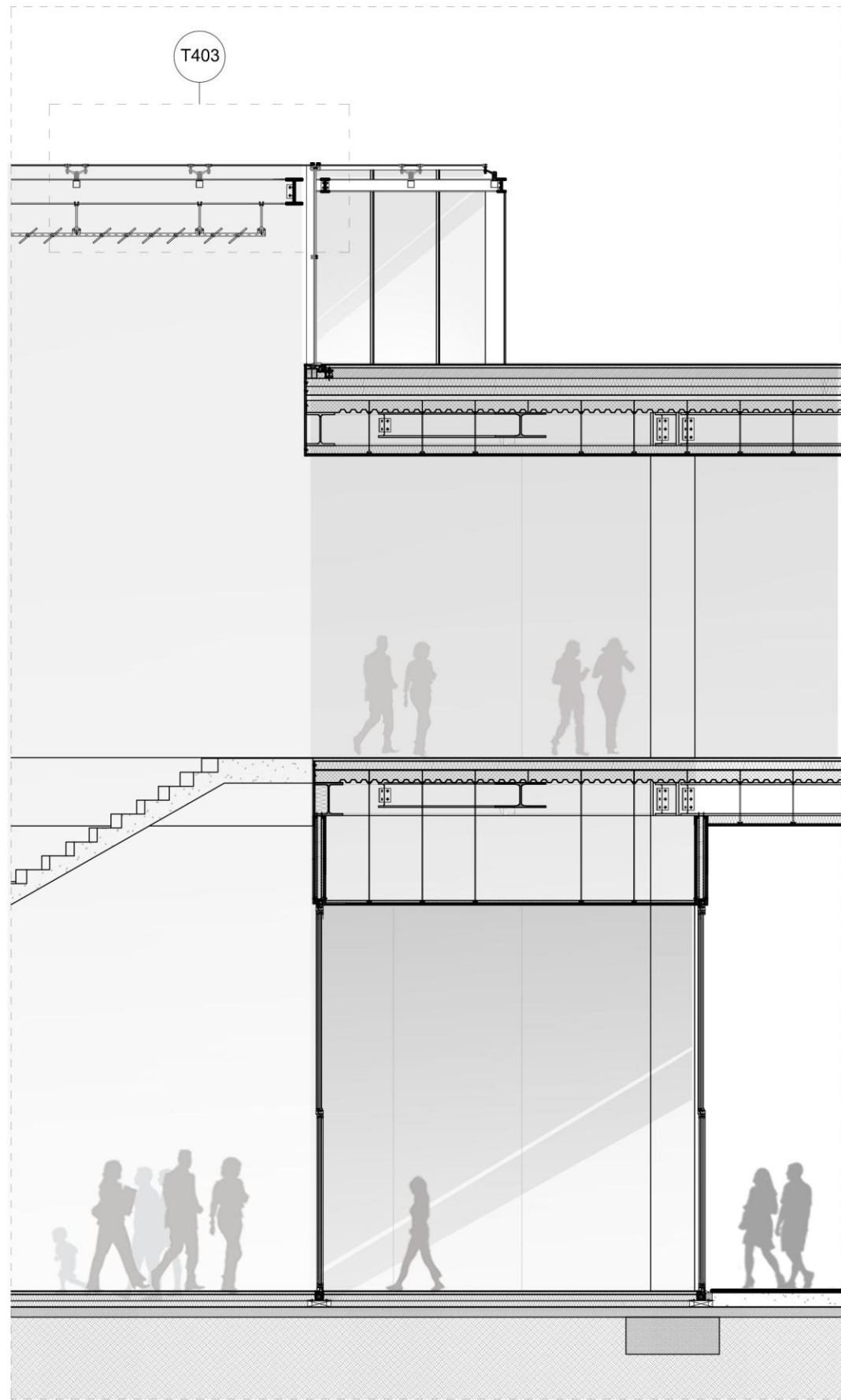
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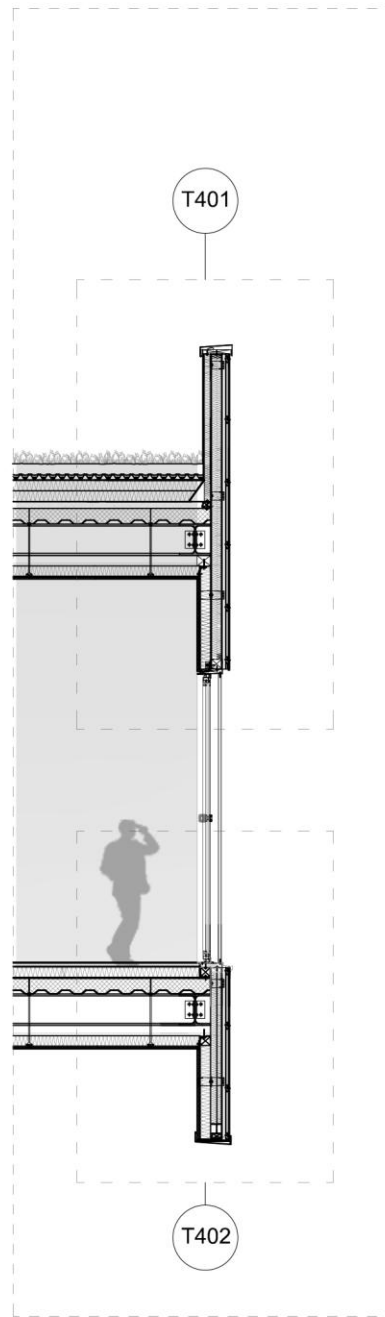


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Drawing Scale
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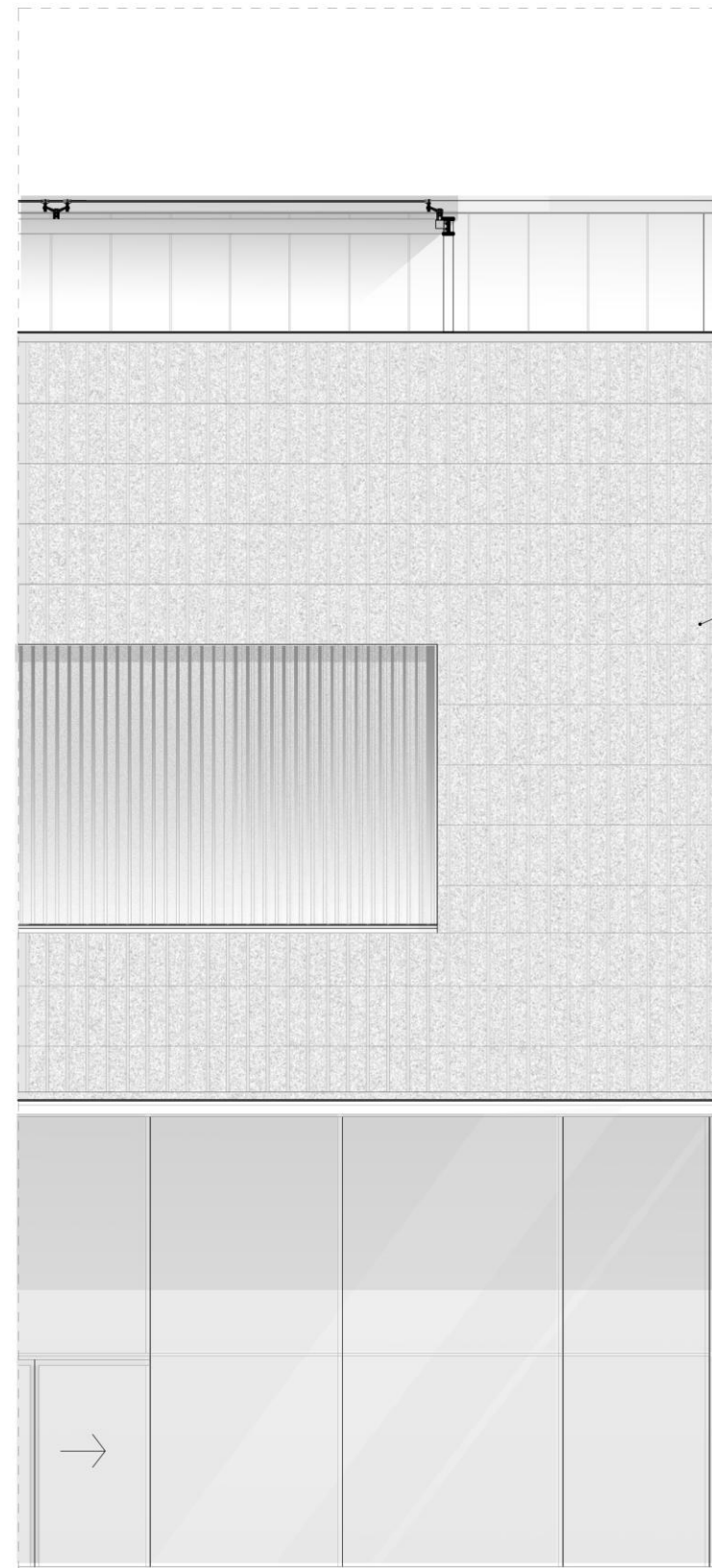


T403



T401

T402



*Marly Eternit FC Fiber cement cladding 600mmX1400mm



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Technology - Blowup
A 501

Drawing Scale
1: 100

7.3 Cooling Load Estimation

The objective of an HVAC (Heating, Ventilating, and Air-conditioning) system is to control the temperature, humidity, air movement, and air cleanliness, normally with mechanical means, to achieve thermal comfort. Load estimation is to calculate the peak design load (cooling/heating), estimate likely plant/equipment capacity or size, as well as form the basis for building energy analysis. In Hong Kong, cooling load estimation is our goal to achieve before HVAC design process.

7.3.1 Design Condition

From <Code of Practice for Energy Efficiency of Building Services Installation 2012> issued by Hong Kong Electrical and Mechanical Services Department, the design condition for both indoor and outdoor at summer time is recommended as below.

Summer Condition	Indoor	Outdoor
Temperature	23°C	33°C
Relative Humidity	50%	80%

Table 7-1 Design Condition

7.3.2 Cooling Load Components

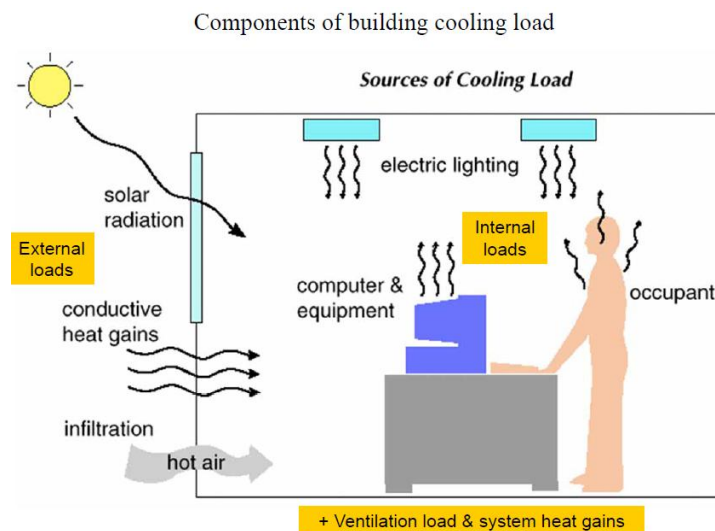


Figure 7-29 Components of Building Cooling Load

Cooling load is constituted of three major parts, namely external heat gain, internal heat gain and infiltration and ventilation heat gain.

- External
 - 1) Solar gain through fenestration Q_1
 - 2) Conduction heat gain through fenestration Q_2
 - 3) Conduction heat gain through roof and walls Q_3
 - 4) Conduction heat gain through partitions Q_4
- Internal
 - 5) Electrical lighting Q_5
 - 6) People Q_6
- Infiltration and Ventilation
 - 7-1) Air leakage and moisture migration Q_{7-1}
 - 7-2) Outdoor ventilation air Q_{7-2}

There are several different methods for cooling load calculation.

Here we choose CLTD/SCL/CLF method by ASHRAE, where CLTD= Cooling load temperature difference, SCL= solar cooling load, and CLF= cooling load factor.

A CLTD study was conducted about parameters need for calculation regarding Hong Kong Buildings by Hong Kong City University. The data are drawn from the study and will be presented in the appendix pages.

A cooling peak load will be discussed in this section at default time 12:00pm, 15-August.

To better understand the calculation, the museum is divided into 5 vertical facades as shown below.

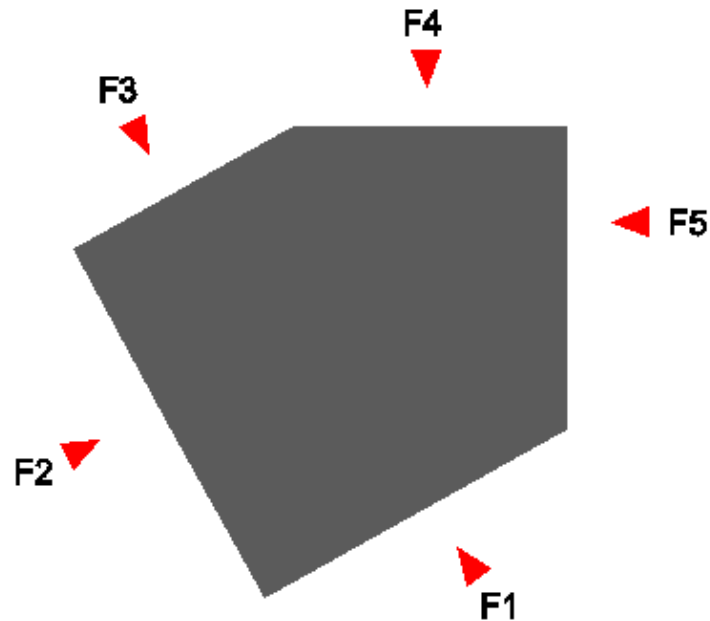


Figure 7-30 Five Facades

- Solar gain through fenestration Q1

$$Q1 = Q_{fes} * CLF$$

$$Q_{fes} = (SC) (A_s * \max SHGF + A_{sh} * \max SHGF_{sh})$$

Where SC=shading coefficient (Please refer to Appendix 1)

A_s = unshaded area of window glass

A_{sh} =shaded area of window glass

$\max SHGF$ = maximum solar heat gain factor for window glass(Please refer to Appendix 2)

$\max SHGF_{sh}$ =maximum solar heat gain factor for shaded area on window (Appendix 3)

CLF= Cooling load factor (Please refer to Appendix 4)

Components	Orientation	Unshaded Window		Shaded Window		Total	Shading Coefficient	Qfes	Cooling Load Factor	Q1 (W)	
		Area (m ²)	Max SHGF (W/m ²)	Area (m ²)	Max SHGF (W/m ²)	Area (m ²)					
F1	S-W	78.1	391	19.1	142	97.2	0.54	17955	0.38	6823	
F2	S-E	140.0	391	29.3	142	169.3	0.54	31806	0.49	15585	
F3	N-E	71.0	565	7.8	142	78.8	0.54	22260	0.27	6010	
F4	N	47.4	147	0.0	142	47.4	0.54	3763	0.89	3349	
F5	E	90.6	671	0.0	142	90.6	0.54	32828	0.27	8864	
Horizontal	-	42.0	877	232.5	142	274.5	0.54	37718	0.85	32061	
Total											72691W

Table 7-2 Calculation of Solar gain through fenestration

- Conduction heat gain through fenestration Q2

$$Q2=A U (\text{CLTD})$$

where

A = area,

U= heat transfer coefficient (W/m²k)

CLTD=Cooling Load Temperature Difference (Please refer to Figure 7-35)

Components	Orientation	Glazing	U-value (W/m ² k)	Cooling Load Temperature Difference CLTD (°C)	Q2(W)
		Area (m ²)			
F1	S-W	97.2	0.85	5	413
F2	S-E	169.3	0.85	5	720
F3	N-E	78.8	0.85	5	335
F4	N	47.4	0.85	5	201
F5	E	90.6	0.85	5	385
Horizontal	-	274.5	0.90	5	1235
Total					3289 W

Table 7-3 Calculation of Conduction heat gain through fenestration

- Conduction heat gain through roof and walls Q3

$$Q3=A U (\text{CLTD})$$

where

A = area,

U= heat transfer coefficient ($\text{W}/\text{m}^2\text{k}$)

CLTD=Cooling Load Temperature Difference (Please refer to Appendix 6)

Components	Orientation	Wall	U-value ($\text{W}/\text{m}^2\text{k}$)	Cooling Load Temperature Difference CLTD ($^{\circ}\text{C}$)	Q3(W)
		Area (m^2)			
F1	S-W	190.0	0.19	5	181
F2	S-E	264.7	0.19	5	251
F3	N-E	125.1	0.19	15	357
F4	N	288.5	0.19	11	603
F5	E	270.3	0.19	12	616
Roof	-	1514.2	0.130	15	2953
Total					4960W

Table 7-4 Calculation of Conduction heat gain through Roof and Walls

- Conduction heat gain through partitions Q4
 $Q4 = A U (t_{\text{adjacent}} - t_{\text{inside}})$

Components	Floor Slab	U-value (W/m ² k)	tadjacent	tinside	(tadjacent - tinside)	Q4(W)
	Area (m ²)		(°C)	(°C)	(°C)	
1/F	1120.2	0.15	33.0	23	10	1680
G/F	583.4	0.20	29.9	23	6.9	805
Total						2485 W

Table 7-5 Calculation of Conduction heat gain through Partitions

- Electrical Lighting Q5

$$Q5 = \text{Lighting Power in Watt} \times \text{Ful} \times \text{Fsa} \times \text{CLF}$$

Where

Lighting Power = Lighting Power Density x Room Area (Please refer to Appendix 7)

Ful = usage factor for each hour of the day =0 when all lights are off

=1 when max. design number lights are on

Fsa= service allowance factor or multiplier

CLF= Cooling load factor (Please refer to Appendix 8)

Floor	Function	Illuminance (lx)	Area (m ²)	Lighting Power Density (Watts/sf)	Lighting Power Required (Watts)	Ful	Fsa	CLF	Q5 (W)
		(BS EN12464-1:2002)		(Table 9.6.1-ASHRAE 90.1-2007)					
G/F	Men's WC	100	13.6	0.9	131.7	1	0.8	0.79	83.2
	Women's WC	100	15.2	0.9	147.2	1	0.8	0.79	93.0
	Disabled WC	100	4.6	0.9	44.5	1	0.8	0.79	28.2
	Museum Café	200	119.2	1.2	1539.1	0.5	0.8	0.79	486.4
	Security Room	200	22.9	1.1	271.0	1	0.8	0.79	171.3
	Store Room	100	15.8	0.8	136.0	0.5	0.8	0.79	43.0
	Museum Shop	200	57.8	0.6	373.2	0.5	0.8	0.79	117.9
	Lobby	200	250.1	0.6	1614.6	0.5	0.8	0.79	510.2
1/F	Men's WC	100	25.4	0.9	246.0	1	0.8	0.79	155.5
	Women's WC	100	31.5	0.9	305.0	1	0.8	0.79	192.8
	Disabled WC	100	4.6	0.9	44.5	1	0.8	0.79	28.2
	Terrace 1	200	76.5	0.5	411.6	0.5	0.8	0.79	130.1
	Multifunction Room	300	42.5	1.3	594.5	0.5	0.8	0.79	187.9

	Open Library	300	156.3	1.2	2018.1	0.5	0.8	0.79	637.7
	Laboratory	500	35.9	1.4	540.8	1	0.9	0.79	384.5
	Terrace 2	200	47.3	0.5	254.5	0.5	0.8	0.79	80.4
	Office	500	51	1.1	603.6	1	0.8	0.79	381.5
	Corridor	100	134	0.5	720.9	0.5	0.8	0.79	227.8
	Exhibition Hall	300	264.5	1.0	2846.0	0.5	0.9	0.79	1011.8
2/F	Men's WC	100	9.4	0.9	91.0	1	0.8	0.79	57.5
	Women's WC	100	15.8	0.9	153.0	1	0.8	0.79	96.7
	Disabled WC	100	4.6	0.9	44.5	1	0.8	0.79	28.2
	Workshop	300	34.2	1.3	478.4	0.5	0.8	0.79	151.2
	Multifunction Room	300	42.8	1.3	598.7	0.5	0.8	0.79	189.2
	Corridor	100	61.1	0.5	328.7	0.5	0.8	0.79	103.9
	Exhibition Hall	300	123.1	1.0	1324.6	0.5	0.9	0.79	470.9
Total									6048.7 W

Table 7-6 Calculation of Cooling Load for Electrical Lighting

- People Q6
 $Q6-s=n * SHG * CLF$
 $Q6-l=n * LHG$
 where

SHG=Sensible heat gain

(Please refer to Appendix 10)

LHG=Latent heat gain

(Please refer to Appendix 10)

Floor	Function	Area	Occupancy Factor for floor area per person (Appendix 9)	n (Number of people)	Heat Gain per person (W)		CLF	Q6-s	Q6-l	Q6(W)
			(Table B1 of Code of Practice for Fire Safety in Buildings 2011)		Sensible Heat	Latent Heat				
G/F	Museum Café	119.2	2	119	75	95	0.9	8046	11324	19370
	Museum Shop	57.8	3	9	75	75	0.9	585	650	1235
	Lobby	250.1	2	75	90	95	0.9	6077	7128	13205
1/F	Terrace 1	76.5	2	38	90	95	0.9	3098	3634	6732
	Multifunction Room	42.5	1	21	75	75	0.9	1434	1594	3028
	Open Library	156.3	2	16	65	55	0.9	914	860	1774
	Laboratory	35.9	2	9	90	95	0.9	727	853	1580
	Terrace 2	47.3	2	24	90	95	0.9	1916	2247	4162
	Office	51	9	3	75	75	0.9	172	191	363
	Exhibition Hall	264.5	3	106	90	95	0.9	8570	10051	18621
2/F	Workshop	34.2	2	17	90	95	0.9	1385	1625	3010
	Multifunction Room	42.8	1	21	75	75	0.9	1445	1605	3050
	Exhibition Hall	123.1	2	49	90	95	0.9	3988	4678	8666
Total									84796 W	

Table 7-7 Calculation of Heat from People

- Infiltration and Ventilation Q7

$$Q7=Q7-1+Q7-2$$

$$Q7-1=1.23 * \text{air flow rate} * (t_o-t_i)$$

$$Q7-2=3010 * \text{air flow rate} * (w_o-w_i)$$

Floor	Function	Area (m ²)	Occupant Density (per 100m ²)	n (Number of people)	Outdoor air rate (CFM/person)	Combined Outdoor air rate (l/s/person)	t _i -t _o	w _o -w _i	Q7-1	Q7-2	Q7
G/F	Museum Café	119.2	100	119	9	5	10	0.011	6598	17760	24358
	Museum Shop	57.8	15	9	16	8	10	0.011	853	2297	3150
	Lobby	250.1	30	75	10	5	10	0.011	4614	12421	17036
1/F	Terrace 1	76.5	50	38	6	3	10	0.011	1411	3799	5211
	Multifunction Room	42.5	50	21	6	3	10	0.011	784	2111	2895
	Open Library	156.3	10	16	17	9	10	0.011	1634	4399	6033
	Laboratory	35.9	25	9	17	9	10	0.011	938	2526	3464
	Terrace 2	47.3	50	24	6	3	10	0.011	873	2349	3222
	Office	51	5	3	17	9	10	0.011	267	718	984
	Exhibition Hall	264.5	40	106	11	6	10	0.011	7157	19267	26424
2/F	Workshop	34.2	50	17	6	3	10	0.011	631	1699	2330
	Multifunction Room	42.8	50	21	6	3	10	0.011	790	2126	2915
	Exhibition Hall	123.1	40	49	11	6	10	0.011	3331	8967	12298
Total										110319W	

Table 7-8 Calculation of Heat of Infiltration and Ventilation

$$Q= Q1+Q2+Q3+Q4+Q5+Q6+Q7 = 284 \text{ kW}$$

Therefore, peak cooling load is thus estimated for the 12:00pm in hottest day, 15-August is 284kW.

7.3.3 HVAC System

7.3.3.1 Geothermal Heat Pump

In Hong Kong, electrical energy consumption for air conditioning of buildings is very high. Since most central air conditioning systems are air-cooled, there has been much concern about the need for appropriate energy conservation strategies and solutions for the 'heat island effect' due to heat dissipation from the air conditioning condensers of the crowded buildings.

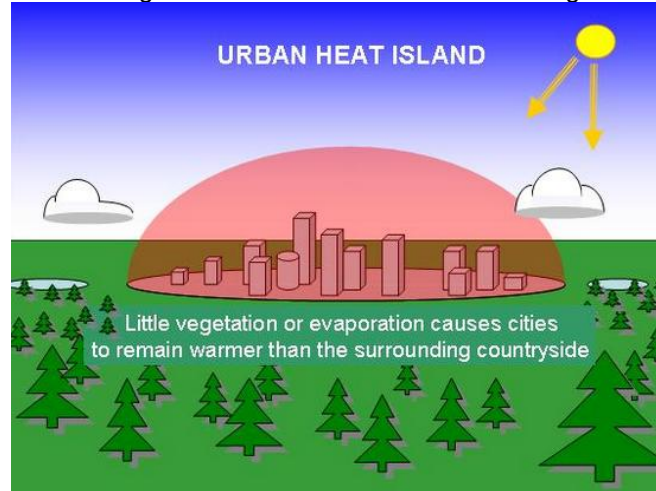


Figure 7-31 Urban Heat Island Effect

Recently, the Hong Kong government has initiated a pioneering project for the Wet Land Garden in the New Territories in which a geothermal heat pump system is employed for central cooling and heating of the associated buildings. From the results of the simulation studies based on the assumed geological properties of the site and the local weather conditions of Hong Kong, it is concluded that it is feasible to adopt the use of geothermal heat pump systems for cooling and heating in Hong Kong...It is envisaged that it will have a high potential for application in Hong Kong, given the current concern about the accumulation of rejected heat from air conditioning systems resulting in 'heat islands', especially in the urban areas. Geothermal heat pump systems rely on the earth for heat rejection or heat absorption in order to achieve space cooling or heating.

(LAM & WONG, *Geothermal Heat Pump System for Air Conditioning in Hong Kong*, Proceedings World Geothermal Congress 2005)

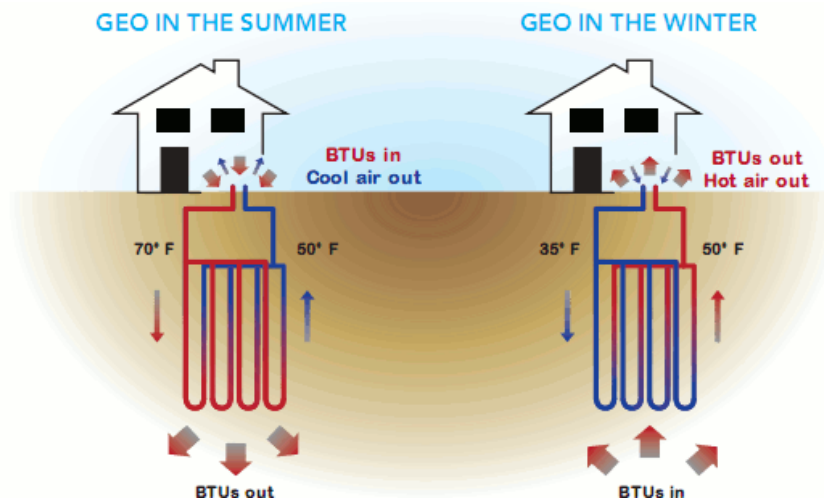


Figure 7-32 Working Principle of Geothermal Heat Pump

A geothermal heat pump transforms the earth energy into useful energy to heat and cool buildings. It provides low temperature heat by extracting it from the ground or a body of water and provides cooling by reversing this process. Its principle application is space heating and cooling, though many also

supply hot water, such as for domestic use. It can even be used to maintain the integrity of building foundations in permafrost conditions, by keeping them frozen through the summer.

Since a GSHP (ground source heat pump) does not directly create any combustion products and because it draws additional free energy from the ground, it can actually produce more energy than it uses. Because of this, GSHP efficiencies routinely average 200 to 500% over a season. GSHP systems are more efficient than air-source heat pumps, which exchange heat with the outside air, due to the stable, moderate temperature of the ground. They are also more efficient than conventional heating and air-conditioning technologies, and typically have lower maintenance costs. They require less space, especially when a liquid building loop replaces voluminous air ducts, and are not prone to vandalism like conventional rooftop units. Peak electricity consumption during cooling season is lower than with conventional air-conditioning, so utility demand charges may be also reduced.

One geothermal system has three major components:

- a heat pump,
- an earth connection
- And an interior heating or cooling distribution system.

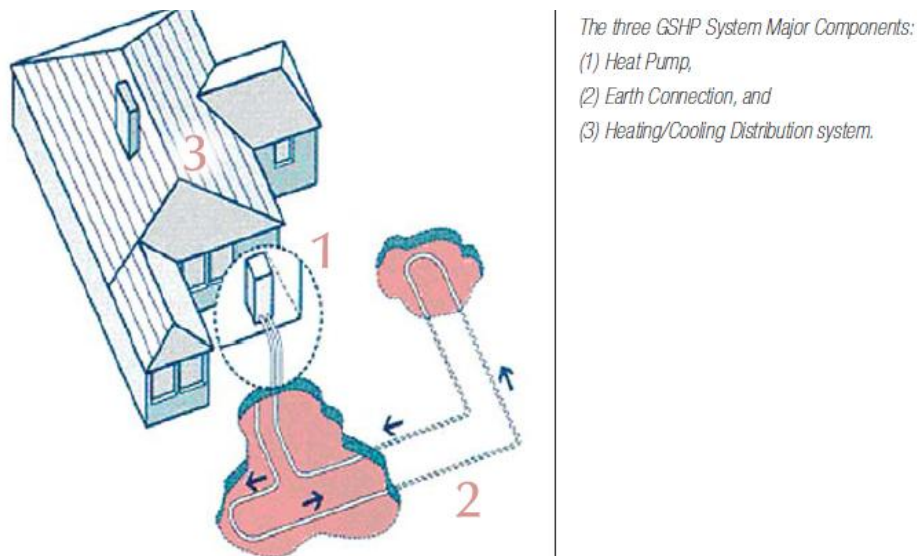


Figure 7-33 The three GSHP system major components

- The heat pump transfers the heat between the heating/cooling distribution system and the earth connection. It is the basic building block of the GSHP system. The water-to-air designation indicates that the fluid carrying heat to and from the earth connection is water or a water/antifreeze mix and that the heat distribution system inside the building relies on hot or cool air.

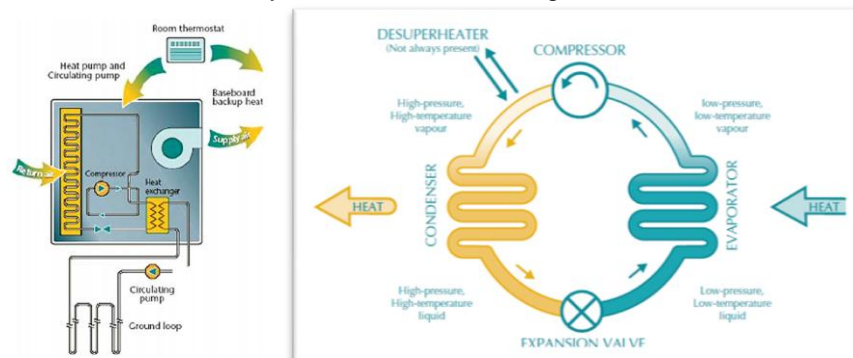
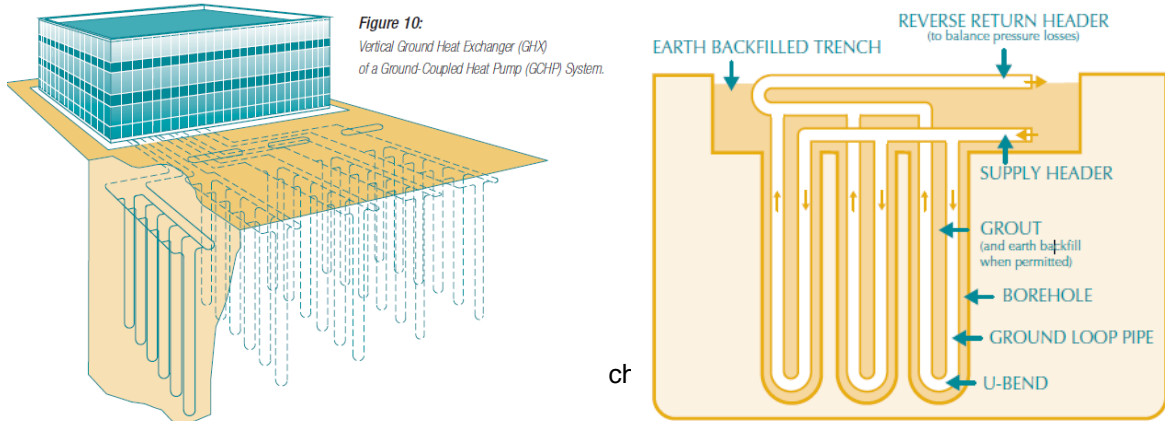


Figure 7-34 Typical Heat Pump Unit (left) and refrigeration cycle in heating mode(right)

- Vertical loops can be used when the land area is limited. It can also minimize the disturbance to existing landscaping. Because the ground temperature is steady year round below the surface, vertical ground heat exchangers are more efficient than horizontal ones, which may experience seasonal temperature fluctuations. The boreholes, 45 to 150m in depth, are drilled by rigs normally used for drilling wells. They contain either one or two or four loops of pipe with a U-bend at the bottom. After the pipe is inserted, the hole is backfilled and grouted.



- The heating/cooling distribution system delivers the conditioned air to the building. The various types of air delivery systems that can be used are well documented (ASHRAE, 1992) and consist mainly of air ducts, diffusers, fresh air supply systems and control components.

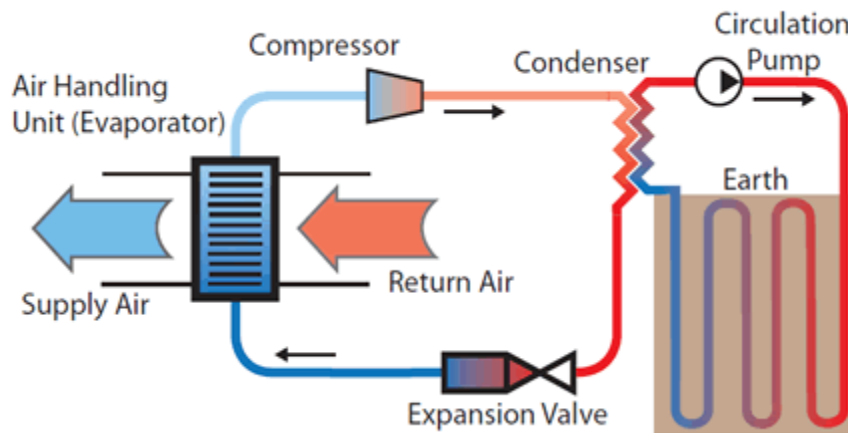


Figure 7-36 Typical circulation of cooling mode

7.3.3.2 System Sizing

The measure for the efficiency of a heat pump system is the Seasonal Performance Factor (SPF), which is the average Coefficient of Performance (COP) in a given plant over a year or a heating/cooling season.

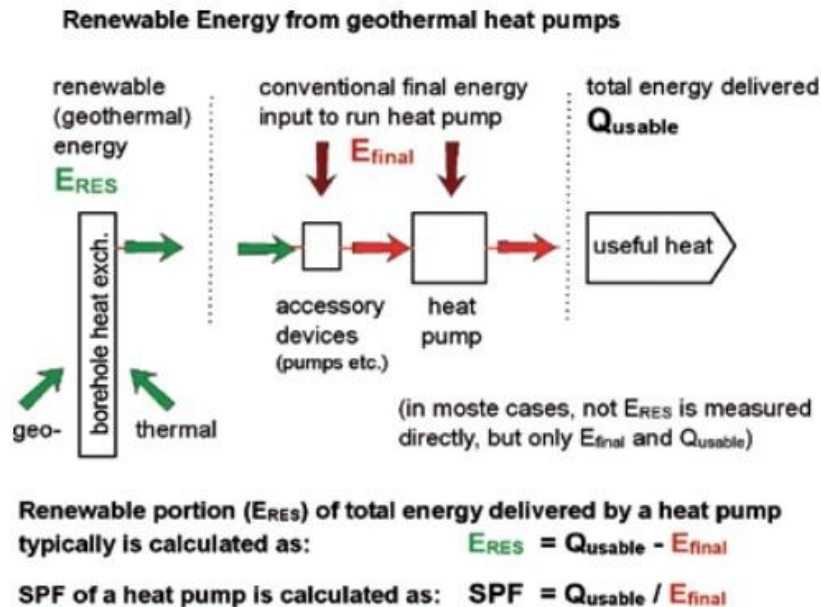


Figure 7-37 Energy flow in a geothermal heat pump system

Within the new directive 2009/28/EC on Renewable Energies, the amount of geothermal energy captured by heat pumps to be considered energy from renewable sources E_{res} , shall be calculated in accordance with the following formula:

$$E_{res} = Q_{usable} \times (1 - 1/SPF)$$

Where Q_{usable} = the estimated total usable heat delivered by heat pumps

SPF = estimated average seasonal performance factor for chosen heat pumps

Here we assume $SPF = 3.5$, $Q_{usable} = 284 \text{ kW}$.

Hence we get $E_{res} = 202 \text{ kW}$

$$E_{res} = 4.182 \text{ (J/gK)} \times m \times \Delta T$$

Where m = water flow rate (g/s)

ΔT = Temperature difference between inlet and outlet water = 5K

So we get water flow rate = 13.6 kg/s

Ground heat exchanger sizing is concerned mainly with the determination of heat exchanger length.

The required geothermal heat exchanger length L_c based on cooling requirements:

(Chapter of Ground-Source Heat Pump Project Analysis, *Clear Energy Project Analysis: RETScreenEngineering & Cases Textbook*, 2005 Canada)

$$L_c = q_{d,cool} \left[\frac{\frac{(COP_c + 1)}{COP_c} (R_p + R_s F_c)}{T_{ewt,max} - T_{g,max}} \right]$$

Where COP_c = design cooling coefficient of performance of heat pump system = 5

$q_{d,cooling}$ = design cooling load = 284 kW

R_p = pipe thermal resistance = 1/0.3 mK/W (for polyethylene tubing having inner radius of 17.3 mm and a centre to centre half distance of 25.4 mm.)

R_s = soil thermal resistance = 1/2.6 mK/W (LAM&WONG, 2005)

F_c = part load factor for cooling = 0.3

$T_{w,max}$ = maximum design entering water temperature = 92.2 °C

$T_{g,max}$ = maximum undisturbed ground temperature = 25 °C

So we get $L_c = 4.5$ km

The drilling depth in the site is assumed to be 100 m.

A double-U in one loop is designed to install. (four pipes in total per hole)

Hence, total 10 bore holes are needed to satisfy the cooling load demand.

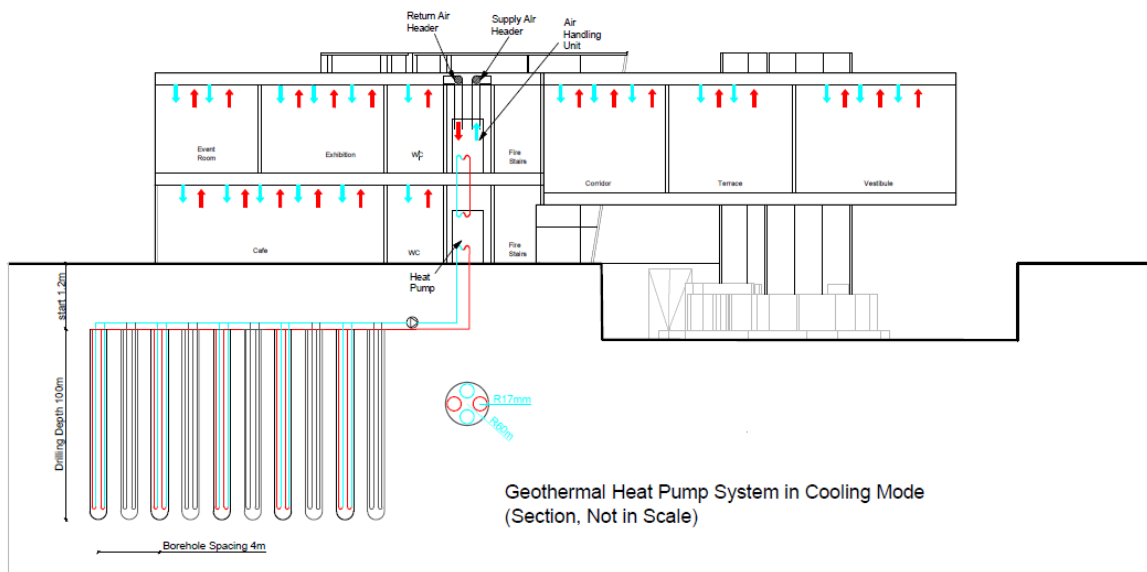


Figure 7-38 Working Scheme of Geothermal Heat Pump

CONCLUSION

Hong Kong is a city where East meets west every minute in every corner. Here the essence of traditional Chinese culture is best preserved whereas each facet of this literally-translated 'fragrant harbor' is greatly influenced by the west. The unique identity can be found in the city's architecture. The remarkable Bank of China Tower by I.M. Pei would not be able to stand until the plan had bypassed the convention of consulting with Feng Shui masters. Yet the business model in Hong Kong restricts the number of cultural projects and freedom of design. By understanding it as one opportunity to challenge ourselves, we decided to choose this archaeology project in Hong Kong.

Rocco Yim used to say that, 'The good thing about architecture is that it asks for solutions to some practical problems'. The very first problem that occurred to us was to define a concept and a form to the given site. Research and case study were carried out about the site history, archaeological typology, conservation methods, undergoing developing strategy and interpretation principles, etc. The concept was then gradually outlined, leading to the final form of one garden and one museum.

The different pathways along with landscapes were designed in the bridge garden to create a pedestrian connection and a relaxing experience to appreciate the bridge ruins. It was a subject not only about urban design, but also about garden design. Chinese gardens were studied to understand the eastern philosophy of landscape design. On the other hand, the garden was divided into two sections so to have a visual awareness of the history timeline.

To reflect the original basis of the archaeological site - a bridge, the form of the museum were shaped into a crossing structure floating over orthogonally the ruin planes. We designed the viewing terrace that gives an overlook of the ruins and the exhibition hall next to it with display of features in a way of telling a convincing story about the bridge. The upper envelope follows the morphology of bridge decking which is formed of rectangular board placed parallel. The whole look indicates the floating solid upper mass as an eye-catcher.

The structure of the building is a fusion as well, made of regular steel beam-column framework and truss system. Truss system is the key to maintain the form of crossing bridge. Elements under critical conditions were carefully verified. Upon the climate condition of the city, light well and skylight atrium were designed to attract natural light and natural ventilation for mild seasons while geothermal heat pump is used for air-conditioning system during hot summer.

Overall, solutions reached us progressively along with more and more practical problems in each step, from urban analysis to form determination, from local site study to global development strategy, from case study to element extraction and from architecture shaping to structure and technology detailing. We had integrated our knowledge and past experience into the application, while at the same time seeking for a better result in new subjects. It has been rather exciting for us to slowly unveil the final project until it is vividly presented here in this report. But what we have learned from this project has gone beyond the pages.



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APPENDIX

This appendix is made for data reference of cooling load calculation in Section 7.3.2. It is referred from Cooling Loading Calculation Study conducted by Hong Kong City University. For more information, please refer to <http://personal.cityu.edu.hk/~bsapplec/cooling.htm> ..

Nominal thickness, mm	Solar transmission	Shading coefficient						
			Venetian		Roller shade, opaque		Draperies, light	
			Medium	Light	Dark	White	Openb	Closedb
Clear	3-12	0.78 -	0.64	0.55	0.59	0.25	0.65	0.45
Heat-absorbing	3-6	0.46	0.57	0.53	0.45	0.3	0.49	0.38
Heat-absorbing	10	0.34	0.54	0.52	0.4	0.28		
Reflective								
SCa=0.30			0.25	0.23			0.23	0.21
SCa=0.40			0.33	0.29			0.33	0.28
SCa=0.50			0.42	0.38			0.41	0.34
SCa=0.60			0.5	0.44			0.49	0.38
Insulating glass:								
Clear out-clear	6	0.8	0.57	0.51	0.6	0.25	0.56	0.42
SCa=0.84								
Heat absorbing	6	0.56	0.39	0.36	0.4	0.22	0.43	0.35
SCa=0.55								
Reflective								
SCa=0.20	6	0.8	0.19	0.18			0.18	0.16
SCa=0.30			0.27	0.26			0.27	0.25
SCa=0.40			0.34	0.33			0.36	0.29

Appendix 1 Shading Coefficient for Window Glasses with or without Indoor Shading Device

Month	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept	Oct.	Nov.	Dec.
SHGFsh, W/m ²	98	107	114	126	137	142	142	133	117	107	101	95

Appendix 2 Maximum Solar Heat Gain Factor of Shaded Area

Month	Maximum solar heat gain factor for 22 degree north latitude, W/m2					
	North	North-east / north-west	East / west	South-east / south-west-	South	Horizontal
January.	88	140	617	789	696	704
February.	97	265	704	759	578	808
March.	107	404	743	663	398	882
April	119	513	719	516	210	899
May	142	572	687	404	139	892
June	180	589	666	355	134	880
July	147	565	671	391	140	877
August	123	502	694	496	223	879
September	112	388	705	639	392	854
October	100	262	676	735	563	792
November	88	142	606	786	686	699
December	84	101	579	790	730	657

Appendix 3 Maximum Solar Heat Gain Factor for Sunit Glass on Average Cloudness Days

Solar time, hour	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
North	0.08	0.07	0.06	0.06	0.07	0.73	0.66	0.65	0.73	0.8	0.86	0.89	0.89	0.86	0.82	0.75	0.78	0.91	0.24	0.18	0.15	0.13	0.11	0.1
North-east	0.03	0.02	0.02	0.02	0.02	0.56	0.76	0.74	0.58	0.37	0.29	0.27	0.26	0.24	0.22	0.2	0.16	0.12	0.06	0.05	0.04	0.04	0.03	0.03
East	0.03	0.02	0.02	0.02	0.02	0.47	0.72	0.8	0.76	0.62	0.41	0.27	0.24	0.22	0.2	0.17	0.14	0.11	0.06	0.05	0.05	0.04	0.03	0.03
South-east	0.03	0.03	0.02	0.02	0.02	0.3	0.57	0.74	0.81	0.79	0.68	0.49	0.33	0.28	0.25	0.22	0.18	0.13	0.08	0.07	0.06	0.05	0.04	0.04
South	0.04	0.04	0.03	0.03	0.03	0.09	0.16	0.23	0.38	0.58	0.75	0.83	0.8	0.68	0.5	0.35	0.27	0.19	0.11	0.09	0.08	0.07	0.06	0.05
South-west	0.05	0.05	0.04	0.04	0.03	0.07	0.11	0.14	0.16	0.19	0.22	0.38	0.59	0.75	0.81	0.81	0.69	0.45	0.16	0.12	0.1	0.09	0.07	0.06
West	0.05	0.05	0.04	0.04	0.03	0.06	0.09	0.11	0.13	0.15	0.16	0.17	0.31	0.53	0.72	0.82	0.81	0.61	0.16	0.12	0.1	0.08	0.07	0.06
North-west	0.05	0.04	0.04	0.03	0.03	0.07	0.11	0.14	0.17	0.19	0.2	0.21	0.22	0.3	0.52	0.73	0.82	0.69	0.16	0.12	0.1	0.08	0.07	0.06
Horizontal	0.06	0.05	0.04	0.04	0.03	0.12	0.27	0.44	0.59	0.72	0.81	0.85	0.85	0.81	0.71	0.58	0.42	0.25	0.14	0.12	0.1	0.08	0.07	0.06

Appendix 4 Cooling Load Factor for Window Glass with Indoor Shading Devices

Solar time, hour	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
CLTD,(°C)	1	0	-1	-1	-1	-1	-1	0	1	2	4	5	7	7	8	8	7	7	6	4	3	2	2	1

Note: The values are calculated for an inside temperature (Ti) of 25.5°C and outdoor daily mean temperature (Tom) of 29.4°C.

Appendix 5 Cooling Load Temperature Difference for Conduction through Window Glass

Solar time, hour	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
Roof	14	12	10	8	7	5	4	4	6	8	11	15	18	22	25	28	29	30	29	27	24	21	19	16
External wall	8	7	7	6	5	4	3	3	3	3	4	4	5	6	6	7	8	9	10	11	11	10	10	9
North	9	8	7	6	5	5	4	4	6	8	10	11	12	13	13	13	14	14	14	13	13	12	11	10
North-east	11	10	8	7	6	5	5	5	7	10	13	15	17	18	18	18	18	18	17	17	16	15	13	12
East	11	10	9	7	6	5	5	5	5	7	10	12	14	16	17	18	18	18	17	17	16	15	14	12
South-east	11	10	8	7	6	5	4	4	3	3	4	5	7	9	11	13	15	16	16	16	15	14	13	12
South	15	14	12	10	9	8	6	5	5	4	4	5	5	7	9	12	15	18	20	21	21	20	19	17
South-west	17	15	13	12	10	9	7	6	5	5	5	5	6	6	8	10	12	17	10	11	12	11	11	19
West	14	12	11	9	8	7	6	5	4	4	4	4	5	6	7	8	10	12	15	17	18	17	16	15
North-west																								

Note: The values are calculated for an inside temperature (Ti) of 25.5°C and outdoor daily mean temperature (Tom) of 29.4°C.

Appendix 6 Cooling Load Temperature Difference for Roof and External Walls

**TABLE 9.6.1 Lighting Power Densities Using the Space-by-Space Method
ANSI/ASHRAE/IESNA Standard 90.1-2007**

In cases where both a common space type and a building-specific type are listed, the building specific space type shall apply.

Common Space Types	LPD, watts/sf	Building-Specific Space Types	LPD, watts/sf
Office – Enclosed	1.1	Gymnasium/Exercise Center	
Office – Open Plan	1.1	Playing Area	1.4
Conference/Meeting/Multipurpose	1.3	Exercise Area	0.9
Classroom/Lecture/Training	1.4	Courthouse/Police Station/Penitentiary	
For Penitentiary	1.3	Courtroom	1.9
Lobby	1.3	Confinement Cells	0.9
For Hotel	1.1	Judges' Chambers	1.3
For Performing Arts Theater	3.3	Fire Stations	
For Motion Picture Theater	1.1	Engine Room	0.8
Audience/Seating Area	0.9	Sleeping Quarters	0.3
For Gymnasium	0.4	Post Office – Sorting Area	1.2
For Exercise Center	0.3	Convention Center – Exhibit Space	1.3
For Convention Center	0.7	Library	
For Penitentiary	0.7	Card File and Cataloging	1.1
For Religious Buildings	1.7	Stacks	1.7
For Sports Arena	0.4	Reading Area	1.2
For Performing Arts Theater	2.6	Hospital	
For Motion Picture Theater	1.2	Emergency	2.7
For Transportation	0.5	Recovery	0.8
Atrium – First Three Floors	0.6	Nurses' Station	1.0
Atrium – Each Additional Floor	0.2	Exam/Treatment	1.5
Lounge/Recreation	1.2	Pharmacy	1.2
For Hospital	0.8	Patient Room	0.7
Dining Area	0.9	Operating Room	2.2
For Penitentiary	1.3	Nursery	0.6
For Hotel	1.3	Medical Supply	1.4
For Motel	1.2	Physical Therapy	0.9
For Bar Lounge/Leisure Dining	1.4	Radiology	0.4
For Family Dining	2.1	Laundry – Washing	0.6
Food Preparation	1.2	Automotive – Service/Repair	0.7
Laboratory	1.4	Manufacturing	
Restrooms	0.9	Low Bay (<25 ft Floor to Ceiling Height)	1.2
Dressing/Locker/Fitting Room	0.6	High Bay (≥25 ft Floor to Ceiling Height)	1.7
Corridor/Transition	0.5	Detailed Manufacturing	2.1
For Hospital	1.0	Equipment Room	1.2
For Manufacturing Facility	0.5	Control Room	0.5
Stairs – Active	0.6	Hotel/Motel Guest Rooms	1.1
Active Storage	0.8	Dormitory – Living Quarters	1.1
For Hospital	0.9	Museum	
Inactive Storage	0.3	General Exhibition	1.0
For Museum	0.8	Restoration	1.7
Electrical/Mechanical	1.5	Bank/Office – Banking Activity Area	1.5
Workshop	1.9	Religious Buildings	
Sales Area	1.7	Worship Pulpit/Choir	2.4
		Fellowship Hall	0.9
		Retail	
		Sales Area	1.7
		Mall Concourse	1.7
		Sports Arena	
		Ring Sports Area	2.7
		Court Sports Area	2.3
		Indoor Playing Field	1.4
		Warehouse	
		Fine Material Storage	1.4
		Medium/Bulky Material Storage	0.9
		Parking Garage – Garage Area	0.2
		Transportation	
		Airport – Concourse	0.6
		Air/Train/Bus – Baggage Area	1.0
		Terminal – Ticket Counter	1.5

Appendix 7 Lighting Power Density

Total	Time of Day											→
Hours	8	9	10	11	12	13	14	15	16	17	18	19
Space	Number of hours after entry into space											→
↓	0	1	2	3	4	5	6	7	8	9	10	0
8		0.66	0.72	0.76	0.79	0.81	0.83	0.85	0.86	0.25	0.20	
10		0.68	0.74	0.77	0.80	0.82	0.84	0.86	0.87	0.88	0.90	
12		0.70	0.75	0.79	0.81	0.83	0.85	0.87	0.88	0.89	0.90	
14		0.72	0.77	0.81	0.83	0.85	0.86	0.88	0.89	0.90	0.91	
16		0.75	0.80	0.83	0.85	0.87	0.88	0.89	0.90	0.91	0.92	

Appendix 8 Cooling Load Factor for Lighting Analysis in different hour per day

Section 2 – Provisions of Means of Escape

Subsection B4 – Assessment of Occupant Capacity

Clause B4.1

As a guide to assessing the requirements on means of escape, the following Table B1 should be used as the basis for calculating the occupant capacity of a building or part of a building.

Table B1: Assessment of Occupant Capacity

Use Classification	Type of Accommodation	Occupancy Factor (usable floor area in m ² per person) or otherwise as specified
1b	Flats:	
	<ul style="list-style-type: none"> - with corridor or balcony access having five or more flats on each floor served by each staircase - flats not covered by the above 	<p>4.5</p> <p>9</p>
1c	Tenement houses	3
2	Boarding houses, hostels, hotels, motels, guesthouses	Number of bedspaces
	Dormitories	3
3a	Day care centres, nurseries, child care centres	4
	Hospitals (areas other than the patient care areas)	9
	Patient care areas	Number of bedspaces
3b	Detention and Correctional Centres	Number of bedspaces
4a	Offices	9
	<ul style="list-style-type: none"> - Board rooms, conference rooms, function rooms 	10
	<ul style="list-style-type: none"> - Staff rooms 	9
4b	Retail shops / Department Stores (including arcade and common areas)	
	Basement, G/F, 1/F & 2/F	3
	3 rd floor & above	4.5

Use Classification	Type of Accommodation	Occupancy Factor (usable floor area in m ² per person) or otherwise as specified
	Markets, supermarkets, showrooms, jewellery and goldsmith shops, pawn shops and money changers	2
	Café, restaurants, dining areas, lounges, bars and pubs	1
	Banking halls (areas accessible to the public)	0.5
	Betting halls (areas accessible to the public)	0.5
	Places where public information or service counters are provided (areas accessible to the public)	0.5
5a	Art galleries, exhibition areas, museums	2
	Cinemas:	
	Seating areas	Number of seats
	Foyer areas	0.5
	Dance floors	0.75
	Sports Stadia	
	standing	0.5
	removable seating	0.5
	fixed seating	Number of seats
	bench seating	450mm/person
Indoor sports facilities:		
Sports / activity areas	10	
standing	0.5	
removable seating	0.5	
fixed seating	Number of seats	
bench seating	450mm/person	
Theatres:		
Seating areas	Number of seats	
Foyer areas	0.5	
5b	Libraries	2
	Reading rooms, study rooms	1
	Classrooms of school not covered by the Education Ordinance, lecture rooms	2 or number of seats

Use Classification	Type of Accommodation	Occupancy Factor (usable floor area in m ² per person) or otherwise as specified
5c	Transport facilities like passenger terminals, railway stations, etc.	Based on actual design and layout
5d	Public halls, assembly halls, conference halls removable seating fixed seating	0.5 Number of seats
	Gymnasia	3
	Swimming Pool	3
	Columbaria	2
	Viewing galleries	0.5
6a	Commercial Laundries	10
	Commercial Laboratories	10
	Factories / Workshops	4.5
	Commercial Kitchens	4.5
6b	Warehouses	30
6c	Storage, manufacturing of hazardous/ dangerous goods premises	30
7	Carparks	30
8	Plant rooms, switch rooms, transformer rooms, etc.	30

Notes:

1. The occupant capacity in a single user specialised industrial workplace will be determined by the Commissioner for Labour according to the specialised trade process proposed.
2. For any use not specified in this Table, the Building Authority should determine the factor to be used.
3. The Building Authority recognises actual counting as a reliable way to establish the occupant capacity of a building.
4. The occupant capacity in Karaoke Establishments should refer to "A Guide to Application for Karaoke Establishment Permits in Restaurant" issued by the Food and Environmental Hygiene Department.
5. The usable floor area for assessing the occupant capacity in the swimming pool in Use Classification 5d refers to the water surface area of the swimming pool.
6. For Use Classification 8, the net floor area should be used in applying the occupancy factor. If the net floor area of a room does not exceed 100m², the occupant capacity is considered to be zero.

Appendix 9 Occupancy Factor for Floor Area per Person from Code of Practice for Fire Safety in Building 2011

Activity	Total heat, W		Sensible heat, W	Latent heat, W
	Adult, male	Adjusted		
Seated at rest	115	100	60	40
Seated, very light work, writing	140	120	65	55
Seated, eating	150	170b	75	95
Seated, light work, typing,	185	150	75	75
Standing, light work or walking	235	185	90	95
Light bench work	255	230	100	130
Light machine work	305	305	100	205
Heavy work	470	470	165	305
Moderate dancing	400	375	120	255
Athletics	585	525	185	340

Appendix 10 Heat Gain from Occupants at Various Activities

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