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Faculty of Engineering

Department of Civil and Environmental Engineering

Concrete Building Detailed Design Project Design Report

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Submitted in fulfilment of the requirements for the MSc and the Diploma of Imperial College London

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Department of Civil and Environmental Engineering

Cluster: _____MSc Structure Cluster_____

Module:_____Research / Design Project - Structures 2021-2022_____

Assignment:_____Concrete Building Detailed Design Project_____

Assignment Setter: _____Robert Vollum_____

Submission Deadline:_____23 August 2022_____

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Content Page

Concrete building design

1 General introduction for the project

1.1 Project profile

Since the beginning of the 21st century, the number of skyscrapers has increased rapidly, and their height has continuously broken records. Almost all the top 20 tall buildings in the world adopt the plan view dimension decreasing along the height direction (abbreviated as reduced shape).
This phenomenon is not accidental but follows the natural law. Super tall buildings bear enormous vertical (gravity) and horizontal (wind load, equivalent horizontal forces, earthquake action, etc.) forces. The bending moment caused by the horizontal forces is directly proportional to the square of the building height, and the displacement caused by the horizontal force is directly proportional to the fourth square of the building height. For this reason, the high-rise building adopts a reduced body shape to reduce lateral forces effectively. It can fully utilize the strength and stiffness of the structural materials and improve the efficiency and economy of the structure.
Therefore, a plan for a 22-story Podium single tower structure will be developed in the waterfront redevelopment area of a regional city centre.
The structure consists of a one-story basement and two stories of retail space on the ground and first floors, and the floor area on the floors is1280m2 (do not include the expansion area for the slab). Meanwhile, glass curtain walls are chosen to make the model design with sound architectural aesthetics.
The residential tower, which has a studio, and one-and two-bedroom apartment, is raised from a podium deck on level 2.
One of the tower's faces slopes up to Level 18 is a garden level, and the two-story penthouse residences are located in a four-story building rising from the garden level.

1.2 Load condition

Permanent Loads

Table 1- 2 permanent load values

Precast concrete cladding, including glazing (kN/m^2)	Concrete density(Kn/m^3)	Finishes(KN/m^2)
2	25	1

Imposed loads

Table 1- 3 Imposed load values

Retail (kN/m^2)	Residential (kN/m^2)	Circulation (kN/m^2)	Podium deck (kN/m^2)	Garden deck (kN/m^2)	Roof (kN/m^2)
5	2.5	5	10	10	2.5

2 Conceptual design

2.1 Proposal 1

2.1.1 Project profile

Mentioned that using reduced plan dimension is an ideal design for high-rise buildings. This proposal introduces inclined columns in two directions,d, which can maximum approach the required architectural drawing.
At the retail level, most columns are spaced with 8×8m as 8m is the least allowable spacing except for the columns connected by inclined columns. For level 2, as the loads transferred by inclined columns are high, the transfer components' dimensions will be significant, and the load path will be longer; for structural efficiency and more service space, the columns under inclined columns are transferred to the ground directly. The detailed dimensions will be shown in a later chapter.

2.1.2 Column grid

For creating more space for service use in retail, and car-parking levels, the minimum spacing of columns will be 8×8, and the spacing in grid line B will be larger to provide a continuous load transfer for inclined columns above. To avoid cantilever slabs, the slab perimeter will be assigned columns in residential levels, furthermore, two orthogonal inclined columns will be designed along the slope of the structure
The discontinuous columns along the total height of the building will be assigned transfer components at the bottom.

2.1.3 Floor system

It is common in high-rise buildings to use flat slabs to provide more flexible service arrangements and design, more accessible construction, and better quality control. Therefore, flat slabs are applied in the proposal.

2.1.4 Transfer components

Transfer components are designed to transfer the loads in level 2 and Level 18 as the changes in columns layout. A beam wall may be designed for transfer components where deep depth requires, and lifts or staircase walls can be hidden.

2.1.5 Wall layout

The load-bearing core constructed from the basement to level 22 encased the access(lifts and staircase) for residential use, and the other core and walls are mainly for fire design purposes.
The lower core(11×4.5 m) is constructed from the basement to level 18, and a smaller upper core(4.5×6.75 m) is placed on that core; the upper core will be designed as Strut and Tie model as the specialty of its support.
In the basement, retaining walls will mainly be designed to resist lateral forces(soil and water pressures) and service use for basement.
For a more pleasing architectural outlook and sustainable use of light, cladding will be assigned around the perimeter of the building in retail and residential level rather than walls.

2.1.6 Architectural Appreciation for the whole model

The architectural material concrete combines both functionality and form. It permits efficient and successful implementation of the architectural concept. It can be engineered to respond to the building's structure, function, and aesthetic in order to create a successful and cohesive whole.
In order to obtain better light and a more transparent, modern design, non-load-bearing glass curtain walls are used for the structure's perimeter.
Meanwhile, the sea-side high-rise building in the regional city centre enjoys a vibrant environment and high architectural aesthetics.
A rendering model for the building, in reality, is shown in Figure 2- 1

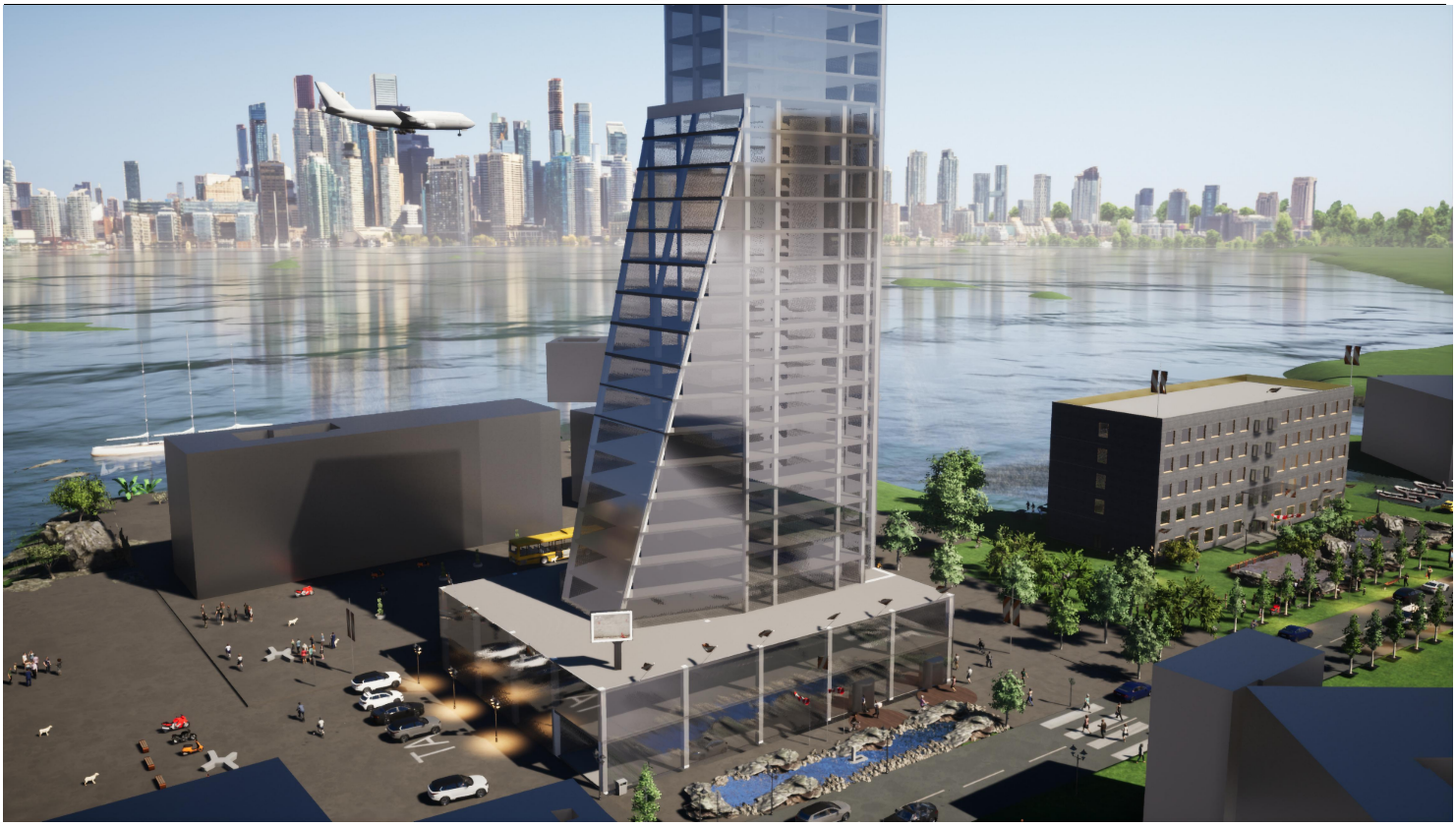


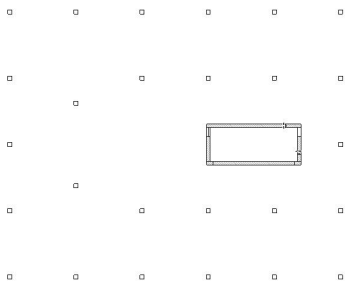
Figure 2- 1 The rendering model for proposal 1 from northwest view(Twinmotion)

2.1.7 Stability

It is essential to check that the whole building and its various elements are structurally stable.
Two stability criteria need to be considered.

2.1.7.1 Lateral stability-resit bending and torsion

In this design, the lateral forces should consider wind loads, lateral loads due to geometric imperfection, and accidental loads.
For resisting those, the main stability system Figure 2- 2



Shear walls are arranged in the plan so that the shear centre coincides with the resultant of the overturning forces. This layout can provide bending moments in both orthogonal directions and limit torsional behaviour due to eccentricity.

Figure 2- 2 Lateral stability system

The use of shear walls assumes that the floor will act as a horizontal diaphragm to transfer loads to the shear walls. This can generally be assumed for in-situ concrete buildings, and no other checks are necessary. However, the inclined column will induce additional pull and push force; therefore, the check for the slab in tension is essential to ensure the action can successfully transfer to the core.

2.1.7.2 Uplift

As it shows, two orthogonal direction stability resistance for the multi-storey building is achieved by designing the core-frame structure.
Because the groundwater level is 1m below ground level, the water pressure may cause uplift(buoyancy) of the structure. The foundation design will ensure sufficient resistance to the uplift forces and measures to avoid uplift during construction.

2.2 Proposal 2

2.2.1 Project profile

Inspired by the structure at the White City campus, Imperial College, walking columns are applied to deal with the slope. Another main difference with proposal 1 is the floor system. Proposal 2 use the traditional beam-slab floor system.

2.2.2 Column grid

The distribution of columns in retail levels is same with proposal 1, as it's an ideal distribution considering both minimum spacing requirement and load transfer.

From level 3 to level 18, the inclined columns are replaced by walking columns along the diagonal lines. The first stage of walking columns are rise from level 2 to level 5, the second stage of the columns is from level 5 to level 8, the third stage is from level 8 to level 12, the forth stage is from level 12 to level 18.

2.2.3 Floor system and building height

As shown in Figure 2-3 beam-to-slab floor is applied in the design, the red beams are transfer beams as there is column layout change in level 2 as well as level 18.

In beam-slab system, try to make the arrangement of columns and beams to be rectangular.

2.2.4 Wall layout

The wall layout is also same as proposal 1.

2.2.5 Architectural Appreciation for whole model

The whole model(without slab and facade for a better view to beam and column system) is shown in Figure 2-3

Figure 2-3 The rendering model for proposal 2 from northwest view(Twinmotion)

Figure 2-4 Similar walking column structure proposal 2 (Photoed by Yibin, at white city campus, imperial college)

In Figure 2-3, even though the latter facade will soothe the outlook of the slope, it is still not pleased as the proposal. Furthermore, unlike the inclined columns in proposal 1, which try to approach the slope requirements infinitely, proposal 2 sacrifices many service areas as the slope region is divided by a series of vertical walking columns.

2.3 Comparison for both proposals

2.3.1 Comparison for column system

The design difficulty is the eccentric vertical load at level 18 for inclined columns will produce additional push and pull forces and transfer thought slabs to the central core.
The stress will be high stress concentration in connection, additional pull and push force in slab(slab will be in tension) and additional shear forces transfer to shear core.
On the other hand, the mobilization with slabs and core system will Increase structural efficiency in the use of structural elements.

Table 2- 1 Column system comparisons between proposal 1 and 2

Items for comparison between walking columns and inclined columns	Proposal 1	Proposal 2
	Inclined columns	Walking columns
Maximizing the bending force arm of the lateral force resistant structure	√	×
Maximize the use of space inside the building to increase service area	√	×
Save for construction materials and frames rather than more transfer components	√	×
Shorter load path to the bottom, more efficient structure system	√	×
Additional pull and push force(slabs will be in tension and additional compression forces)	√	×
Additional shear forces to shear wall	√	×
High stress concentration in connection	√	×
Repetitive but easy to design	×	√

2.3.2 Comparison in floor system

Table 2- 2 Floor system comparisons between proposal 1 and 2

Items for comparison in floor system	Proposal 1	Proposal 2
	Flat slab	beam-and-slab
More clear space	√	×
More flexible in column and room distribution	√	×
Requires less framework	√	×
Fast construction	√	×
Concentration of reinforcement at the nodes of beams and columns, complicated laps and anchorages, concrete prone to honeycomb, difficult construction	×	√
Easier HVAC and plumbing system	√	×
a better look and better light diffusion	√	×
More friendly for long span slab design		
Utilization of drop panels may impede greater mechanical ducting.	√	×
More material efficiency when floor span is large	×	√
Clear load path	×	√
Easier for design	×	√

The lighting permeability FEA analysis for flat slab and beam-and-slab is presented in section 10.1.

2.3.3 Build-ability comparison

Typically, how the designer conceives and details the structure has a greater impact on its buildability than the material used. The following criteria can determine the buildability of a concrete frame:

- **Simplicity for installation** - streamlines the formwork, falsework, layout, and ceiling installation.Repetition of design members - permits the re-use of formwork, expedites the establishment of processes, minimizes the learning curve and training needs, and enhances safety.

- **Standardization** - permits factory production and high-quality finishes, accelerates construction,enables broken components to be rapidly changed, and enables pieces to be selected/approved before final installation.
- **Rationalization of reinforcement** - accelerates construction, allows prefabrication, and decreases time spent on details.
- **Prefabrication** - the ratio of components that can be precast or prefabricated.
- **Using in-situ concrete** - permits late design modifications, offers durability for prefabricated systems and is suitable for foundations.

Table 2- 3 Build-ability comparison between proposal 1 and proposal 2

	Proposal 1	Proposal 2
Simplicity for installation	installed directly on the underside of the flat slab	bending the M & E services to avoid the beams
Repetition of design elements	High repetition except for inclined columns	High repetition except for different dimensions for many transfer components
Standardization	Flat slab will facilitate the use of big table frameworks to increase productivity	The beams and slabs are quite locally designeds and construction
Rationalization of reinforcement	Utilizing prefabricated welded mesh reduces the time required to install flat slabs. This mesh is available in conventional sizes and enables improved quality control in flat slab buildings	The mesh and reinforcement arrangement can not be prefabricated
Using in-situ concrete	yes	yes
The flexibility of Room layout	Placement of partition walls is permitted anywhere. Use of flat slab permits omission of false ceiling and finishing of slab soffit with skim coating.	For beam-slab system, false ceiling and finishing slab soffit with coating can not be omitted. And beams need to be designed under every partition

These permit the incorporation of standard structural elements and prefabricated parts into the design to facilitate building. This construction procedure makes the structure more accessible, reduces the number of site employees, and boosts site productivity, increasing the likelihood of achieving a higher buildable score.

2.3.4 Robustness comparison

Building classification

Localized accidental damage is inevitable; nonetheless, the disproportional collapse of structures must be avoided.

It was necessary to design structures not to sustain damage disproportionate to the initial impact of an accident.

According to table 1, buildings can be classified into 4 classes with different treatments to meet robustness requirements. The required design is 22 levels residential building, which is beyond the limit for residential buildings (max 15 storeys) in class 2B. Hence this project is classified as class 3 (the worst condition), which requires a systematic risk assessment.

For flat slab, Eurocode 2, Part 1-1 (Clause 9.4.1 (3)) has a similar requirement to amendment 3 of BS 8110:1997. It stipulates that horizontal ties must connect "directly and robustly" with the vertical structure. It also states that this is often accomplished by ensuring that two bottom bars in each direction pass directly between the column reinforcement. (British Standard Institution, 2005)

Benefits from in-situ building

Generally, a well-designed and well-detailed cast-in-place structure will meet the comprehensive tying criteria.

Avoid elements whose failure might result in the collapse of more than a small portion of the neighbouring building. If this is not practicable, alternate load pathways must be developed, or the piece in concern must be reinforced.
Both proposals are in-situ concrete. Consequently, adding additional reinforcement to a building is not necessary to maintain its robustness. Standard detailing of reinforcement guarantees that the ties are securely fastened. Meanwhile, they are also very robust due to their monolithic nature.
At the same time, by using a high grade of concrete (50 MPa), the slab resistance for punching shear is very high.
Therefore, the flat slab can provide a reliable alternate load path even if one column or one-panel slab fails.
Particular emphasis should be paid to transfer structures in proposals 1 and 2, where specific elements may become "key elements,' and alternate load pathways should be considered.
Hence, in proposal 2, the walking column system with too many transfer components may make many components critical simultaneously, whereas a reliable alternative load path cannot be provided.
Therefore, proposal 1 will have much higher robustness than proposal 2.
(Brooker, October 2008)

2.3.5 Cost comparison

The mean differences that contribute the cost difference are the cost between inclined columns and more vertical columns(more transfer components) and the cost between flat slab and traditional slabs.
For inclined columns, the design and construction cost for special components will be higher, such as the slabs in tension
A flat slab construction is less costly than a conventional slab building. Traditional slab building is 15.8 percent more costly than flat slab construction. Compared to conventional slab structures, flat slab structures are the best option for high-rise construction. The lighter weight of the materials utilised in a flat-slab construction minimises transportation and pre-building expenses.
In general, the cost of flat slab constructions is less than that of conventional slab structures. This is because concrete floor slabs in flat-slab structures require fewer cuts and joints. These floors may be poured in one continuous operation, which minimises labour expenses in comparison to cutting individual slabs and assembling them. Flat slab structures also require fewer concrete layers overall, resulting in substantial savings throughout the life of the structure.
Due to the longevity advantages of flat slab buildings, the overall cost of these structures is also reduced. Traditional slab constructions require substantial maintenance to preserve their look and safety. Because they are composed of big panels that cannot be easily disassembled, they tend to show their age more quickly than flat slab structures. Once they meet age-appropriate maintenance schedules, however, newer, well-maintained traditional structures might appear identical to new flat slab structures.

2.3.6 Reason for choosing proposal 1 as detailed design plan

By comparison, proposal 1 will offer

- Better serviceability --flexible partition arrangement, flat space, higher clear height and column arrangement and permeable light.
- More economical--simplicity of installing M & E services, fast and simple framework on site, saving the cost of making ceilings
- Advantages in structural design--flexibility of column arrangement(traditional beam-slab system will much prefer rectangular arrangement while flat slab is going to more flexible for column arrangement. In-site construction of slab will offer more reliable second load path that with a better robustness. Moreover, less transfer components in proposal 1 will provide a more direct load path with good structural efficiency
- Architectural aspect-- more flat and smooth ceiling view, Heat insulation, sound insulation and fire corrosion resistance

3 Architectural detailed design

3.1 Ramp design

A two-way ramp design

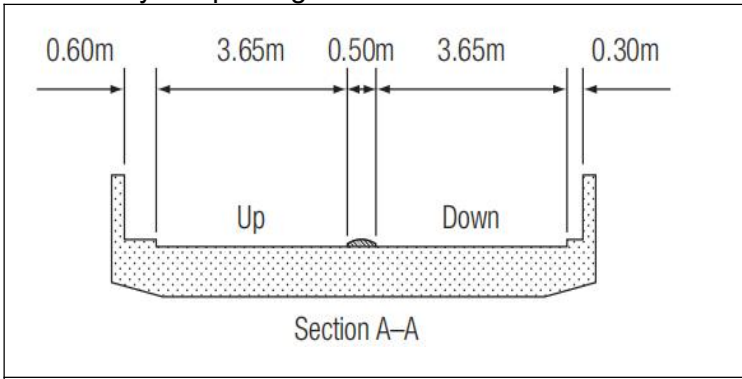


Figure 3- 1 Example for 1 two-way ramp

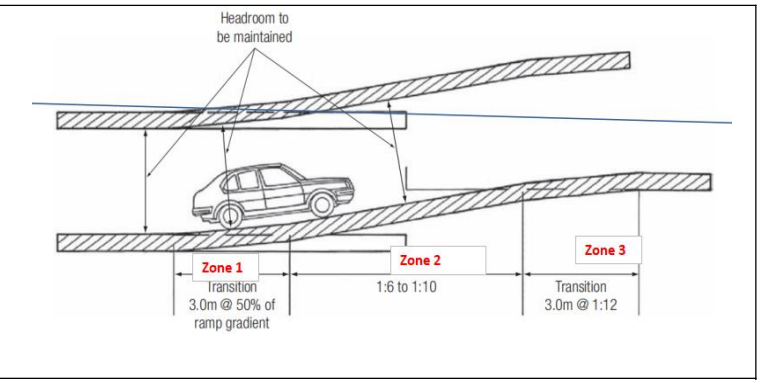


Figure 3- 2 Schematic diagram of ramp

(The Institution of Structural Engineers, March 2011)

For the ramp design, to minimize the ramp length to provide more space for car parking, the maximum allowable ramp gradient needs to be identified as the maximum allowable gradient is controlled by basement level height. The optimized values for design are shown below:

Table 3- 1 Ramp design parameters

Width of additional central margin (m)	0.5	m
Width for straight approach(up)	3	m
Width for straight approach(down)	3	m
Side clearance	0.3	m
total width	7.1	m

	Gradient	Length(m)	Rise(m)
Zone 1	1:17	3	0.1755
Zone 2	1:8.5	22	2.5745
Zone 3	1:12	3	0.25
Total		28	3

The detailed car parking dimensions and layout will be displayed in drawings.

3.2 Access design

The design layout in level 2 is shown in Figure 3- 3

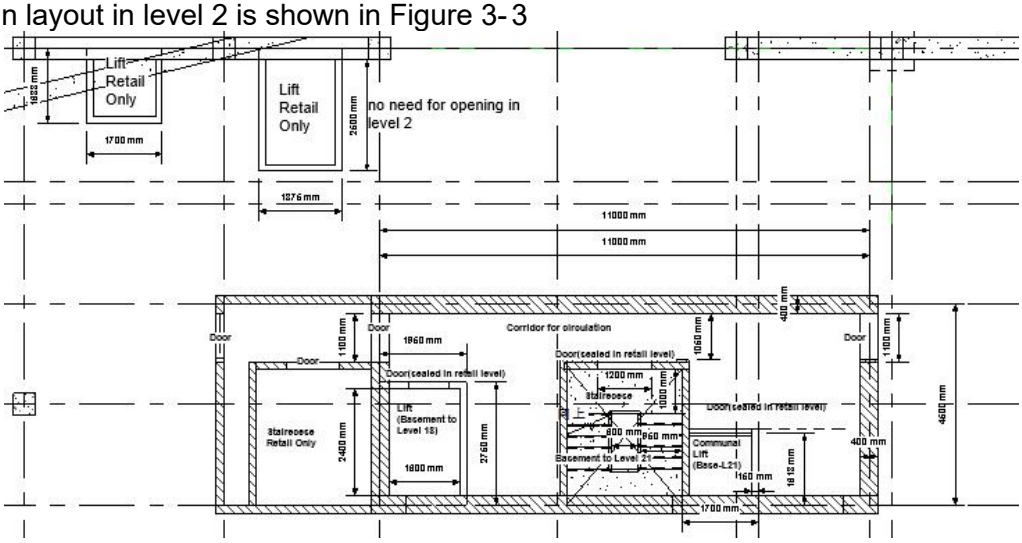


Figure 3- 3 The access design layout in level 2

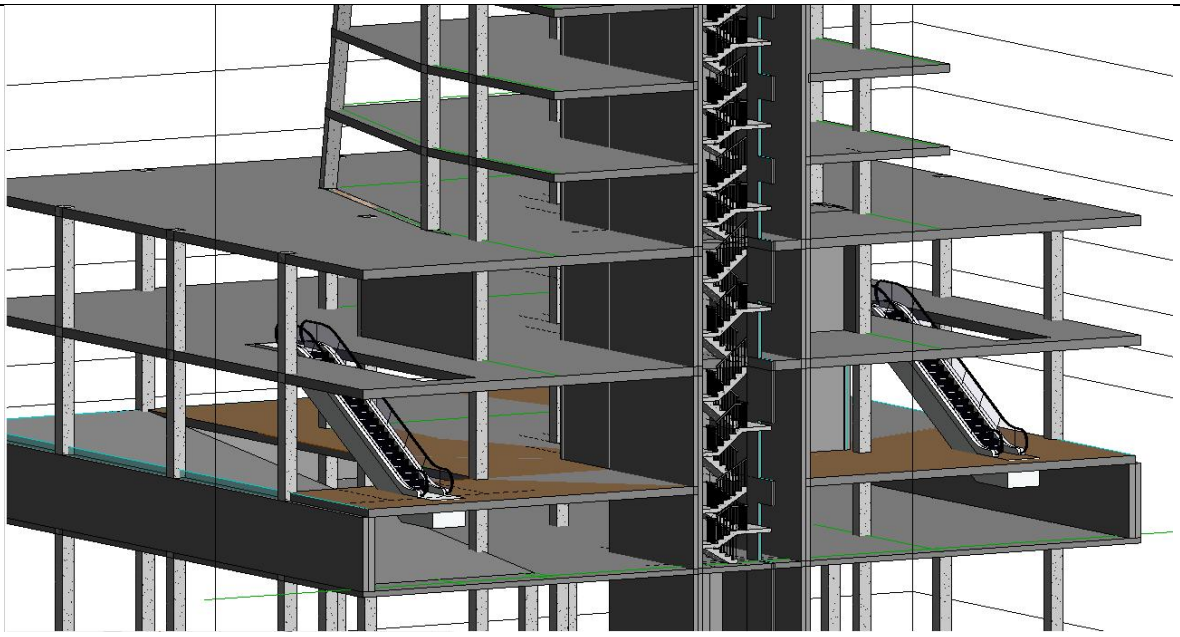


Figure 3-4 3D profile model for access design (Revit)

3.2.1 Lifts design

For residents, two lifts (one for fire-fighting and one for regular use) are arranged in the central core (in Figure 3-3). They are only provided for basement and residential floors.
The communal lift is a fire lift designed for 8 people with 150mm wall thickness from the base to level 21, while another lift is from the basement to level 18(in Figure 3-3) .
Another lift is a normal one and is designed for 13 people.
For lifts only in retail levels, two lifts (one for the fire lift) are designed near the transfer components, which are designed as beam walls as shown in the plan view drawing. One side of the walls for lifts can be hidden in the beam walls and can provide more service areas.

3.2.2 Escalator design

Two escalators with 30°are provided only in retail level as shown in level 1 plan view and Figure 3-4.

3.2.3 The corridor design

The minimum width for corridor is 1050mm and have two 1100mm wide doors on the two sides providing for residents to travel through those access (in Figure 3-3).

4 Material specification

Because the project is a high-rise building, the high axial stress requires high requirements for concrete. Therefore, most of the components use grade 50 concrete.
Meanwhile, high-stress concentration requires even higher grade concrete for unique designs in this project(e.g., inclined columns and the point of connection between upper and lower walls).
Moreover, as some of the components are exposed in particular environments such as groundwater, to extend structural durability, the particular type of concrete which can resist water-proofing, the freeze-thaw and water leaking need to be applied.
Furthermore, achieving zero carbon aim in this era is more environmental-friendly concrete.

Exposure condition

The site is flat, 0.5km from the sea.
BS 8500 covers the concrete selection and concrete cover to suit design exposure.

Table 4-1 Exposure class considerations

any external concrete (e.g. Edge columns, walls, and beams)	XS1	Corrosion induced by chloride from sea water
internal concrete components (e.g., internal columns and	X0	No risk of corrosion or acctack

beams)		

Material information for components

Table 4- 2 Material information for conponents

	Concrete grade fck (MPa)	Steel grade fyk MPa	Exposure class	Content of concrete (for sustainability)
External columns	50	500	XS1	EcoPlus concretes
Internal columns	50	500	X0	EcoPlus concretes
walls(internal)	50	500	X0	EcoPlus concretes
walls(basement)	30	500	XS1	Water proof, corrosion resistance
transfer beams	50	500	X0	EcoPlus concretes
beam-walls	50	500	X0	EcoPlus concretes
slabs	50	500	X0	EcoPlus concretes
Foundation	50	500	XS1	Water proof, corrosion resistance

Consideration for sustainability

EcoPlus concretes will be used in this project considering to meet zero carbon dioxide goal in uk.
Hanson Regen GGBS (ground granulated blast furnace slag) is used in EcoPlus concretes as a partial substitute for Portland cement (CEM I) in the composition.
GGBS is a byproduct of the iron industry, and its production consumes less than a third as much energy and emits less than ten percent as much CO2 as CEM I. In addition, it does not necessitate the extraction of new resources and prevents the disposal of slag in landfills.
Moreover, the amount of GGBS in concrete for different components may be different, to meet the requirement of BS EN 15167-1
40-60% ggbs by mass of cement in structural elements (cement type IIA).
70-75% ggbs by mass of cement in foundations (cement type IIB).
(the concrete centre, April 2010)

5 Structural design

The report and calculation sheet present a detailed validation of the structural viability of proposal 1.
Detailed codified design elements include vertical and inclined columns, flat slabs, transfer components(transfer beam and beam walls), and shear walls in proposal 1.
Only static analysis and calculation will be implemented in this project. Dynamic analysis such as earthquake and wind turbulence actions will not be considered.
Detailed codified procedures to calculate actions and design introduced in Eurocode and National Annex for those elements will mainly be presented in the calculation sheet.
Many important assumptions and advanced analysis (FEA) about inclined columns, flat slabs and shear are implemented in this design. This report will demonstrate key results, findings, and comparisons among those elements.

5.1 Regular vertical columns

5.1.1 Design codes

The detailed design for columns strictly follows the procedures in EC2 and Uk National AnneX and them are braced columns (assume all horizontal forces are undertaken by shear core).

5.1.2 For fire design

Concrete is naturally resistant to fire. It is incombustible and has a low heat transfer rate, making it a highly effective fire barrier.
Considering 2 hours of fire resistance, simplified and tabular methods for determining the fire resistance of columns are used. According to EC2, Part 1-2: Structural fire design, not only minimum dimension of cover and nominal axis distance can be designed, but the fire resistance can also be validated based on methods A, and B introduced in the code.

5.1.3 Design for bi-axial bending

The maximum design stress happens in the combination of maximum axial forces and bi-axial bending moment. Therefore, in the calculation, the check for compression stress will take the axial load at the bottom of the column and the maximum bending moment on a particular floor.

5.1.4 Load arrangement and load combinations

Alternate-span-loaded and all-span-loaded cases are applied to identified the maximum moment
And ultimate limit state load combination (1.35×Gk+1.5×Qk) is applied for maximum axial load calculation.

5.2 Analysis and design for inclined columns

When a column is under eccentric vertical loads at its top and bottom, push and pull horizontal forces will be produced to balance the moment induced by the eccentric vertical loads. In the core-frame structure, the horizontal forces will transfer to the central core, as it is assumed the structure is braced by the core (the core takes all horizontal forces). Meanwhile, as the structure is unsymmetrical about the y-y axis of the core, the push and pull horizontal forces will impart additional shear forces to the shear wall. Simplified force analysis are shown in Figure 5-1 and Figure 5-2. (JIANG Han, 2022)

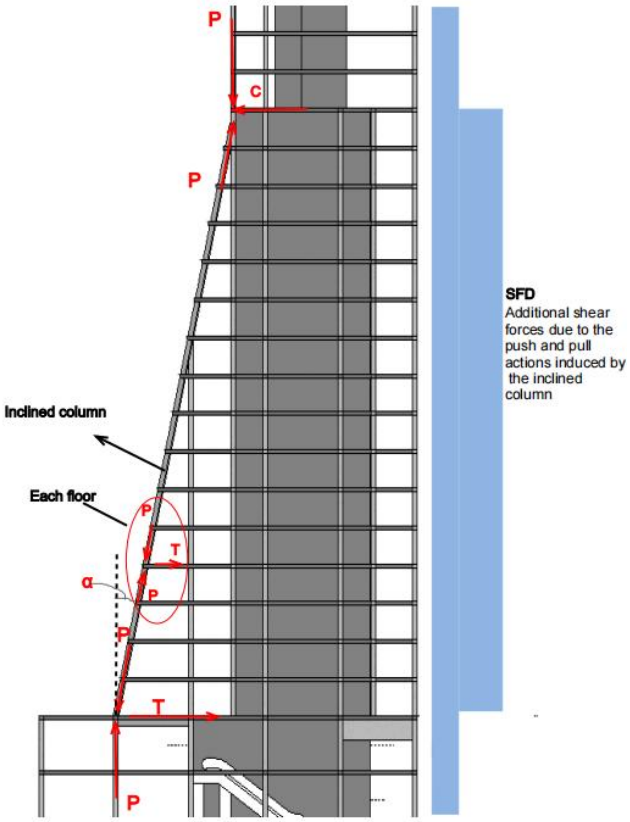


Figure 5-1 W-E direction(Y) α≈ 75°

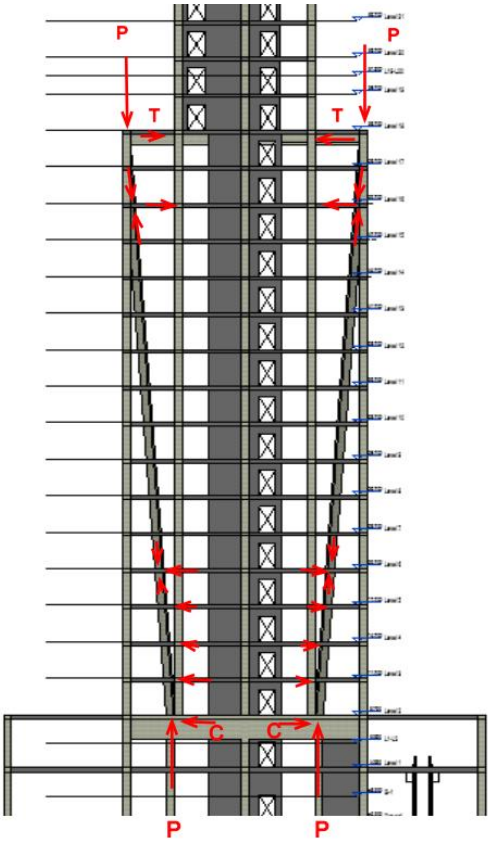


Figure 5-2 N-S direction（X） α≈ 86.33°

The columns are inclined in two orthogonal directions, therefore, the simplified force analysis figures in two directions are shown at LHS.

Note*:The inclined column angle is α, axial force is P, compression force is C and Tension force is T.

5.2.1 Design for the bi-axial bending inclined columns

The design procedures for inclined column are the same with the normal one.

5.2.2 Slab design for push and pull forces

The essay introduced two methods to identify stress in high-rise structure under horizontal load, one is by averaging the top and bottom stresses outputted from ETABS, and another method is by eliminating the bending stiffness in all directions.
To validate FEA results, hand calculation based on force equilibrium is provided.

FEA Methods

In ETABS, the output stress results for shell elements(slab) are the stresses at the top (σ_S^T) and bottom(σ_S^B).

The flexural stresses need to be excluded within them to get the normal stress only induced mainly by the inclined columns' eccentric loads.
In order to separate the in-plane axial normal stress induced by in-plane axial force from the output of the ETABS program, the following two FEA methods can be used

FEA method 1

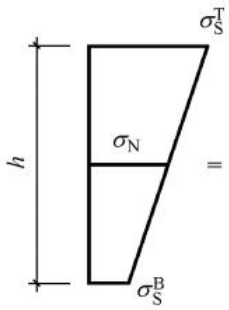


Figure 5-3 Top and bottom stresses shown in ETABS

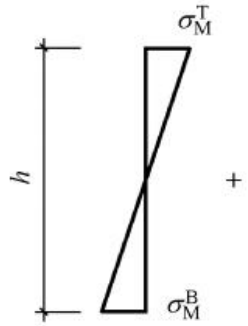


Figure 5-4 Flexural stress induced by vertical loads (designed in flat slabs)

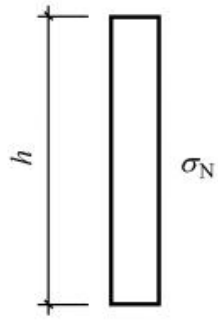


Figure 5-5 Remaining normal stress

The constant normal stress (in Figure 5-5) along the slab's height (excluding the flexural stresses) is obtained by averaging the normal stress (in Figure 5-3) output results of the top surface and bottom surface of the floor.

FEA method 2

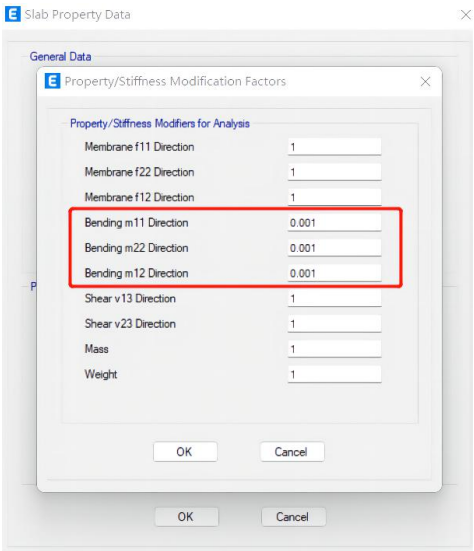


Figure 5-6 FEA setting

By setting a negligible bending stiffness of the floor—that is, ignoring the out-of-plane stiffness of the slabs—the normal stress results of the top surface and bottom surface of the floor output by ETABS are close to the actual axial normal stress of the floor.
The advantage of the second method is that the axial stress of the floor can be obtained directly from the results of the program output, which is intuitive and convenient.

(Wei Lian, 2017)

5.2.2.1 Hand calculation method

The slab in level 2 near the inclined column (in Figure 5-1) will be investigated as an example.
According to the force equilibrium in Figure 5-1, the tension force T in the slab at level 2 can be calculated as force equilibrium: T = P×cosα.
The normal stresses calculated by the FEA and hand calculation methods are presented in Figure 5-7.

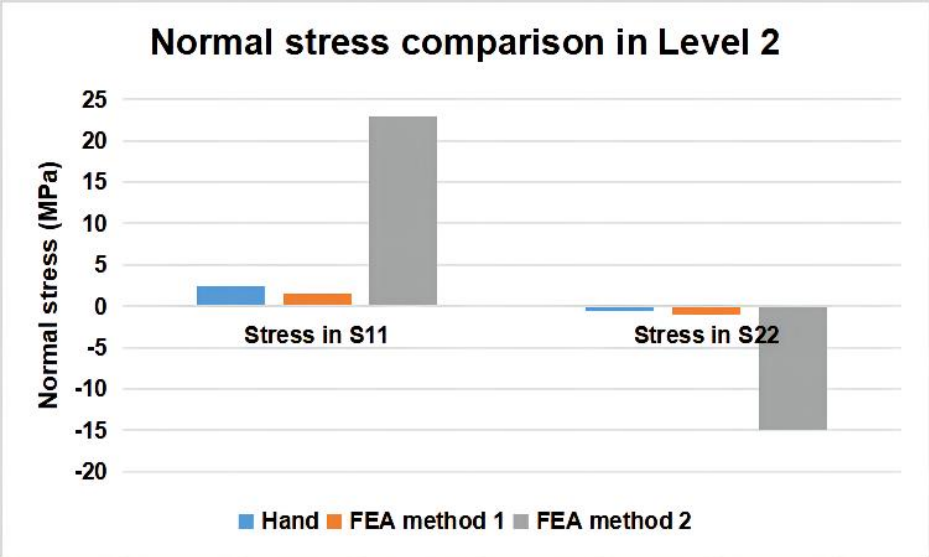


Figure 5- 7 slab normal stress comparisons in level 2

It can be found that the results from FEA method 1 and hand calculation match well, which proves the methods are reliable to be used in the design.

Even the normal stresses in FEA method 2 are higher than the other methods, which shows more conservative.
However, the results deviate a lot, are not reasonable, and cannot be applied in design.
FEA method 2 may be only suitable in the beam-to-slab system rather than the flat slab system.

In the design, the additional reinforcement to resist normal stress can be placed in the middle of the slab (flexural reinforcements will be placed at the top and bottom).
It is worth noting that the FEA contour (shown in the calculation report) shows the tension flow in slabs induced by inclined columns not only flows to the core, which is mainly designed to resist horizontal loads, but also to neighbouring columns. This is not desirable as the column design did not consider the additional horizontal forces. In order to maximally guide the tension flow to the core, the direction of the additional design of reinforcement in slabs is directly from the inclined columns to the position of the core tube.

5.2.3 additional shear forces to core design consideration

The predominant deformation mechanism in a shear wall is flexure, not shear. As with the flexural mode shape, the deformation mode of shear walls has a maximum slope at the top and a minimum slope at the bottom. A shear wall's structural behaviour mirrors that of a cantilever beam.
Therefore, this project does not consider the design for the additional shear force for the core.

5.2.4 Nodes stress check for inclined columns at level 2 and level 18

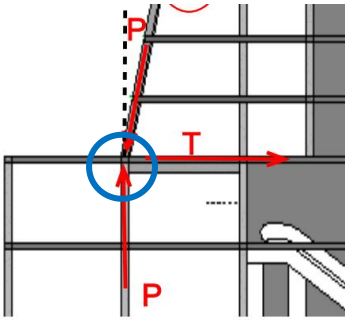


Figure 5- 8 Node 1

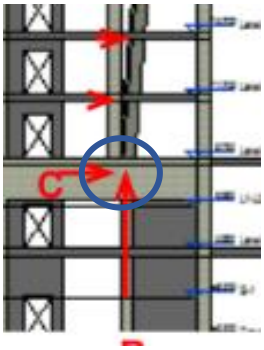


Figure 5- 9 Node 2

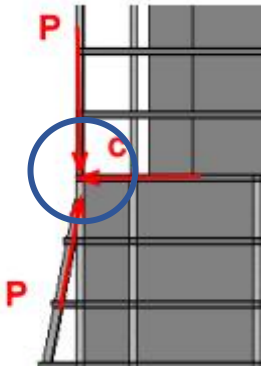


Figure 5- 10 Node 3

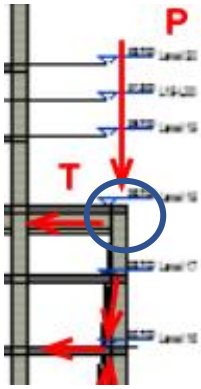


Figure 5- 11 Node 4

As the stress will be concentrated on those nodes between vertical and inclined columns, the check for the nodes shown in Figure 5- 8 to Figure 5- 11 is important.
EC2 provides the methods to check different types of nodes; they can be identified as: Node 1-CCT, Node 2-CCC, Node 3-CCC and Node 4-CCT, and the compressive stresses in those nodes should satisfy the concrete compressive stress resistance. For nodes in tension, additional reinforcements should be provided.

5.3 Flat slabs

5.3.1 General introduction and analysis

For flat slab design, the detailed calculation and analysis will only be provided at level 2, which is the most critical and complex.
The determination of critical design parameters (e.g., moment and shear force) will be defined by the coefficient method, 2D EFM, and 3D FEA models.
The design of flexural moments and punching shear stress for the flab slab between columns will be done by hand calculation and validated by FEA results.
However, the design of shear stress and flexural moments around the shear core will be based on the FEA method.
The detailed design calculations will strictly follow the codes presented in the calculation sheet.

5.3.2 Compare EFM, coefficient method and FEA results of moments

Three methods will be applied to calculate flexural moments in the slabs: Equivalent Frame Model, Coefficient method and FEA in the 3D model.
The detailed EFM assumptions, illustration figures, and calculations are presented in the calculation report.
The details of FEA analysis and comparisons between 2D EFM and 3D FEA models will be provided here.
As the results from the FEA model, including all floors, are not accurate, the flat slab analysis model for level 2 is separately constructed, as shown in Figure 5-12.
The flexural moments and shear stress comparisons among the three methods are only limited in the results around the columns because the flexural moments and shear stress around the core can not be appropriately calculated by both the coefficient method and EFM.

5.3.2.1 FEA for the moments in strips for flat slabs at level 2

Software settings

the strips are manually set in the flat slabs in x and y directions (the pink lines in Figure 5-13), and the moment per meter can be obtained as shown in Figure 5-18 and Figure 5-19.

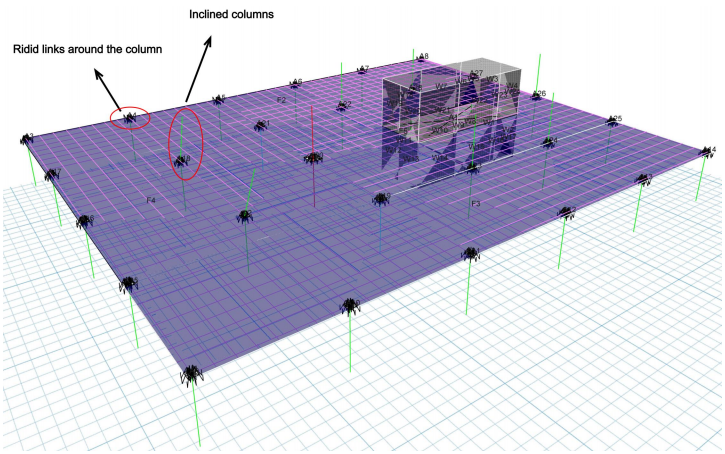


Figure 5-12 FEA model at Level 2 (ETABS)

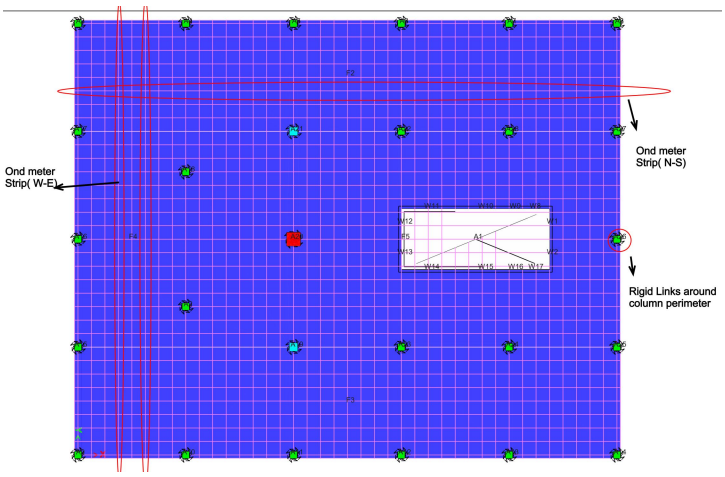


Figure 5-13 Plan view FEA model at Level 2 (ETABS)

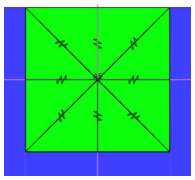


Figure 5-14 Rigid links

As the definition of element connection is the point-to-point between slab and columns, false stress concentration will happen around columns. To deal with this problem, rigid links are introduced to spread the stress around the columns (shown in Figure 5-14 and also the red marked circle in Figure 5-12). Point restraints are connected to the nodes of shell elements around the column perimeter.

The boundary condition for columns are assumed fixed at their far ends.

FEA results

the bending moment contours and moment strips in 2 orthogonal directions(M11,M22) and torsion for single load case at ULS are shown below

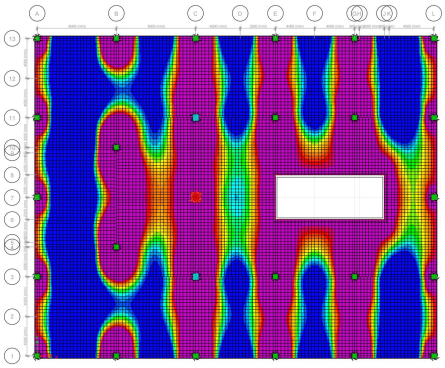


Figure 5- 15Moment could for M11 Level 2

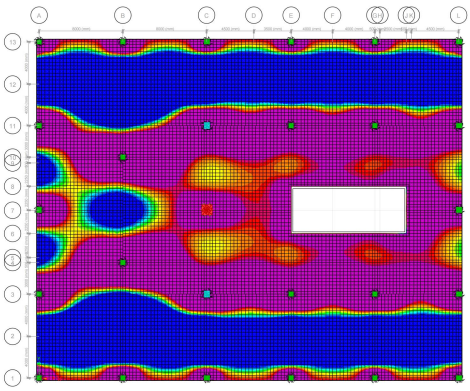


Figure 5- 16 Moment could for M22 Level 2

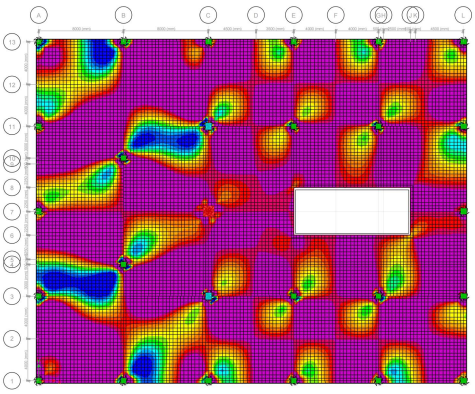


Figure 5- 17 Moment could for M12 Level 2

Conservatively, the design hogging and sagging moments should be the moment in direction plus torsion(M11+M12 and M22+M12), however, the torsion in slabs is hard to calculated by hand, for the purpose of comparison, the design moments will only consider in x and y direction.
In hand calculation, the slab moments will be calculated as Kn.m/m, therefore, as the per meter strips had already set in the slabs, the moment per m in strips will be shown in Figure 5- 18 in x direction and Figure 5- 19 in y direction.

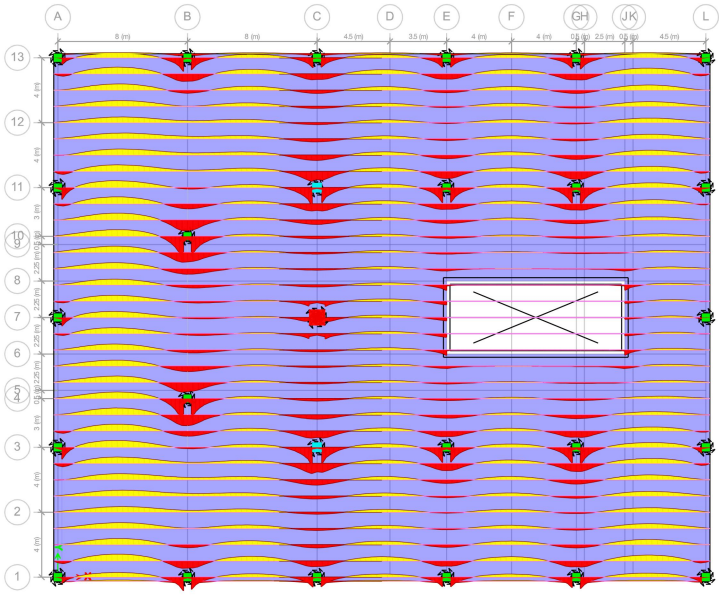


Figure 5- 18 Moments in per meter strip (x direction)

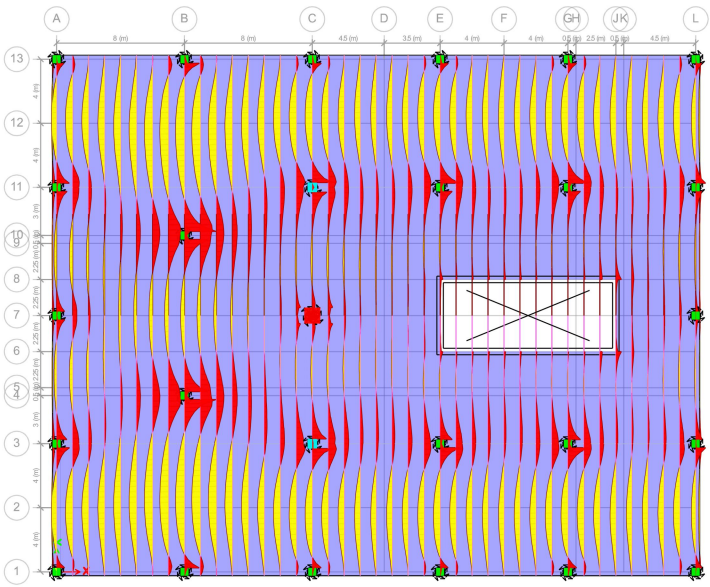


Figure 5- 19 Moments in per meter strip (y direction)

Parametric analysis for moments between coefficient, EFM and FEA methods

As the slab has been divided into the column and middle strips, the width for the strips has been defined according to code (e.g. For Grid A, assumed bay width is 4.25m, the corresponding column strip width= 2.125 m, and middle strip width is 2.125 m)
The coefficient method and EFM can calculate the maximum moment per meter, and the maximum moment per meter can also be extracted from FEM in the required width region.
Therefore, the critical moment in specific width region and Grid among the three methods can be compared and validated if they are reasonable and close enough.
The moment analysis is based on grid lines in two orthogonal directions.
For the grid line with irregular columns distribution (e.g. Grid line 7 and Grid (10+11)), assumptions for EFM are appropriately introduced in each grid line, the maximum hogging and sagging moments in column and middle strip. The moment results in columns and middle strips in selected grid lines calculated by the coefficient, EFM and FEA methods are plotted in Figure 5- 20 to Figure 5- 23(critical grids in Level 2 as an example).

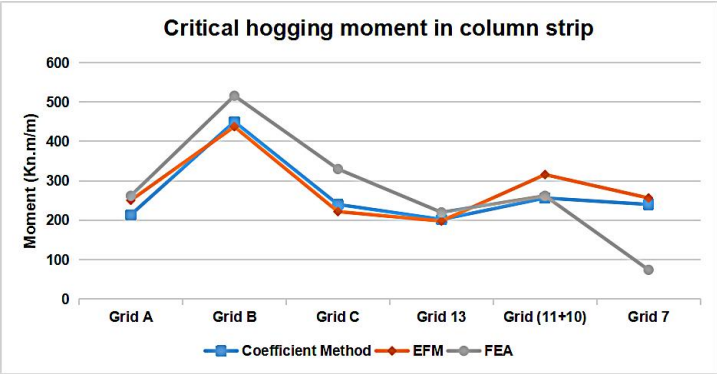


Figure 5- 20 The critical hogging moment in column strip in critical grid lines at level 2

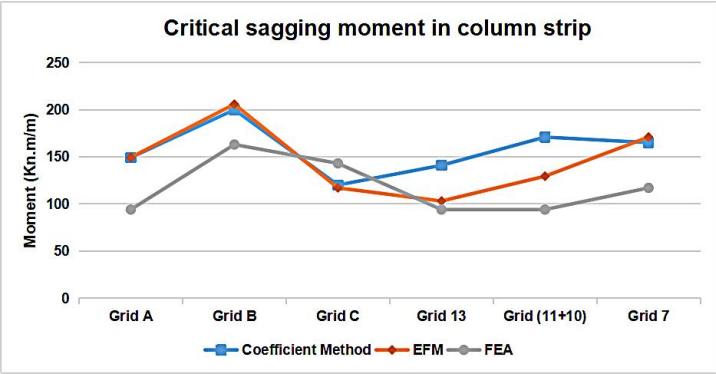


Figure 5- 21 The critical sagging moment in column strip in critical grid lines at level 2

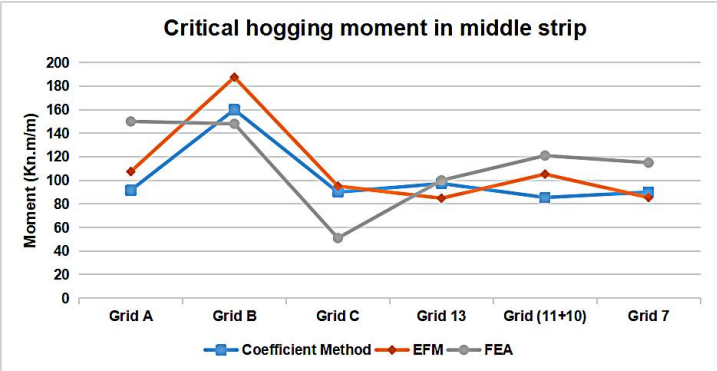


Figure 5- 22 The critical hogging moment in middle strip in critical grid lines at level 2

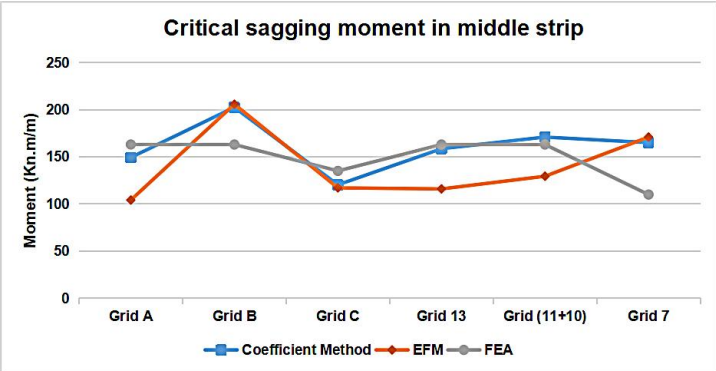


Figure 5- 23 The critical sagging moment in middle strip in critical grid lines at level 2

By comparison in those line charts, the main trend for moments from the three methods in typical grid lines is the same, especially for coefficient and EFM results.
Even though most cases show that the moments from coefficient and EFM are more conservative than that from the FEA model, the cases in which FEA results are higher than the coefficient method and EFM can not be neglected to ensure the structure's safety.
Therefore, the three methods will be implemented for the design moment results to identify the critical one for designing a flat slab.
It is worth noting that the coefficient method and EFM results are almost the same, but the results from the FEA model have a high deviation with them in grid line 7. it shows that the EFM can be upgraded to a better model.
It is worth noticing that the moments in the contours around the core are much smaller than the moments between columns. Therefore, in the grid lines E, H,6, and 8, the moments around cores are not critical to use in flexural reinforcement design.

5.3.2.2 For shear force results analysis

Shear force analysis also relies on FEA and hand calculation results.
For design simplicity, only the most critical edge and internal columns will be designed and applied in Level 2, which is the edge columns in two orthogonal directions, C2 and C7, corner column C1, and internal columns C12 and C14.
The shear force from EFM in two orthogonal directions will not be accurate enough because the assumption for the bay width will only consider for bending moment rather than shear force and the shear force will not be conservative enough. Therefore, the loading area will be specifically identified to calculate shear stress by hand.
Regarding the FEA result, the maximum shear forces V_{max} can be output with the unit of Kn/m in ETABS. They need to divide by average effective depth to get shear stress.
The maximum shear force contour and the line chart to compare shear stress between hand calculation and FEA are shown below.

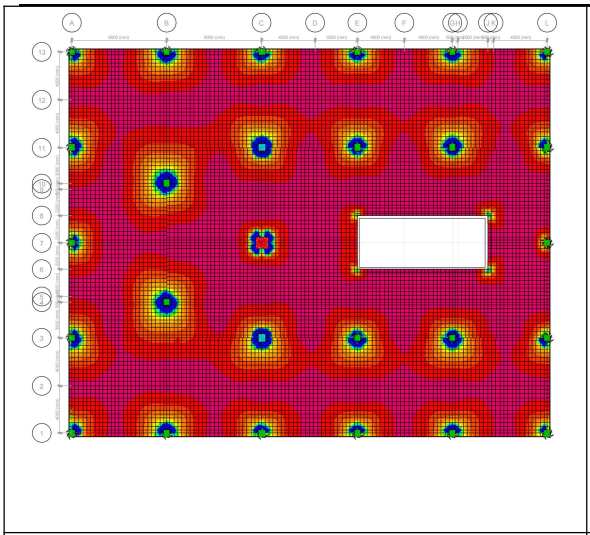


Figure 5-24 V_{max} in Level 2

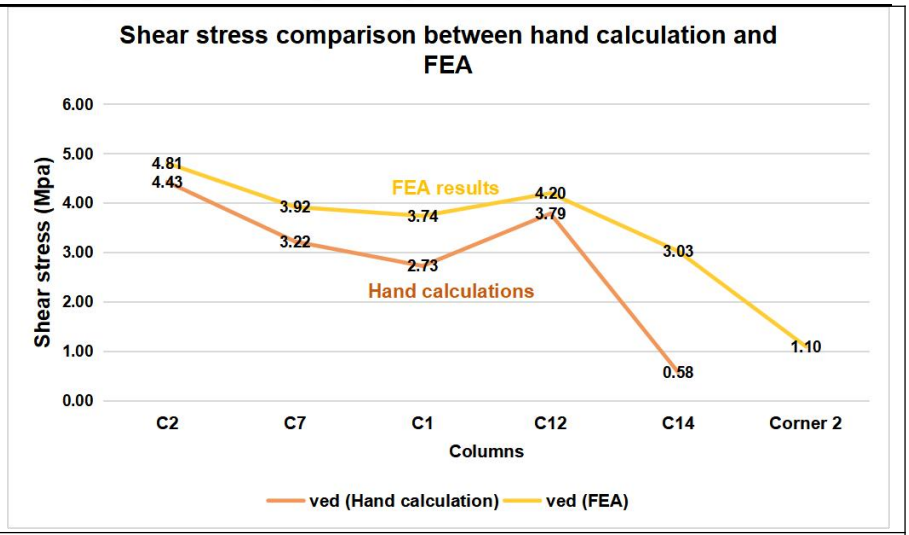


Figure 5-25 Shear stress comparison between hand calculations and FEA

From this contour (Figure 5-24), it can be found that the maximum shear force around the column happens in C12 (1382Kn/m), while the minimum happens around C15(277Kn/m). However, after converting the shear force into shear stress, the maximum stress is around corner column 2(4.81MPa), as the column perimeter is smaller. The shear forces around the shear core are very small. As checked in the calculation sheet, the shear resistance provided by concrete and flexural resistance reinforcements is enough to resist punching shear.
By comparison in the line chart, the shear stress results from the FEA model are always higher than the stress from hand calculation which shows that FEA model results are more conservative to be applied in calculation. The difference between hand calculation and FEA results is slight except for column 14. The error between the two methods for C2, C7, C1, and C12 are in the range of 7%~27%, which is acceptable, while the error for column 14 is 81%. This may be caused by the incorrectly automatic mesh in ETABS and cause stress concentration in this range.
At the same time, the shear stress around wall corners is much smaller than the stress around the columns as the wall provides continuous support and can spread shear stress.

5.3.3 Flat slab design between columns

5.3.3.1 Flexural reinforcement design

The flat slab is designed without heads. In regular flat slab design, after identifying the values of design moment and shear force, the flexural reinforcement requirement and arrangement in middle and column strips under sagging and hogging moments can be calculated, and the requirements for punching shear.
Besides that, specific detailing requirements, such as 50% tension reinforcement, must be placed within the width equal to 0.125 of the panel width on either side of the column.
The detailed codified calculation for flat slab between columns is presented in the calculation report.

5.3.3.2 Reinforcement design to resist punching shear

The design will provide a detailed calculation for the columns with different dimensions and particular positions (edge corner and internal columns).
The calculation will check the necessity of providing punching shear reinforcement in the perimeter around the column, 2d from the face of column u1, and the perimeter at which punching shear links are no longer required. The detailed arrangement can refer to drawings.

5.3.4 Flat slab design around cores

5.3.4.1 Design moment and shear force determination

In this project, shear walls are positioned around elevator shafts and staircases and around the shear wall core in reinforced concrete buildings.
As the support for slabs on the core is continuous, the coefficient method introduced in the code can not be applied in this case and EFM. As a result, the determination of the design moment in the slab depends on the FEA model. As the model has been validated, the FEA results are reasonable and can be used.

5.3.4.2 Flexural reinforcement design

As in grid (10+11), the reinforcement design in the middle strip region has already been covered in this area which means the design bay width for the flat slab near the core is overlapped with the design in grid line (10+11).

Therefore, we only need to check whether the designed reinforcement in the middle strip region is enough to resist the moment in this direction.

Table 5- 1 The moment comparison between slab around columns and around the core(x direction)

Region	$M_{ed,sagging,max}$ (Kn.m/m)	$M_{ed,hogging,max}$ (Kn.m/m)	$M_{xy,max}$ torsion (Kn.m/m)	$M_{ed,sagging}+M_{ed,xy}$ (Kn.m/m)	$M_{ed,hogging}+M_{ed,xy}$ (Kn.m/m)
Middle width(Grid 10+11)	171	85.45	16.415	187.415	101.865
Bay width (Grid 8)	41	11	16.415	57.415	27.415

By inspection, the moment in the bay width of the core (Grid 8) is far smaller than the moment in the middle strip in the grid line (10+11). Therefore, no additional flexural reinforcement in x direction needs to be designed.

In Y direction

The y direction is similar to in x direction, and the bending moment near the core is much smaller than near the columns. Meanwhile, grid line C is more critical than grid lines E and H. For this reason, the reinforcement designed in grid line C can be applied in grid lines E and H.

5.3.4.3 Reinforcement design for resisting punching shear

The shear stress along the walls is much smaller than the stress around wall corners. Hence, the shear resistance needs to be checked in the corner region. Furthermore, according to the contour, the stress will only be concentrated in a limited region. Hence, the corner can be simplified to design as a 400×400mm corner column.

5.4 Transfer components

5.4.1 General introduction and analysis

As the spacing requirements in retail level and car parking and the layout change is garden level, column grid will change between level 2 and level 3 and level 18 and level 19.

The high-rise building, which means the loads transferred to level 2 will be pretty high; for resisting bending and deflection, the transfer components' depth tends to be high. However, too high a depth of transfer beam may look not pleased. Hence, for transfer components which require high depth, they can be designed as Beam walls by using Strut and Tie Model (Figure 5- 26).

For the transfer components with comparatively shallower depth, they will be designed as transfer beams (Figure 5- 27).

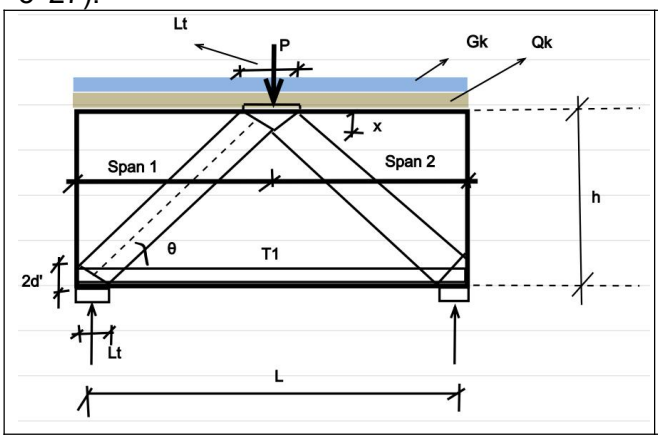


Figure 5- 26 Beam wall (STM)

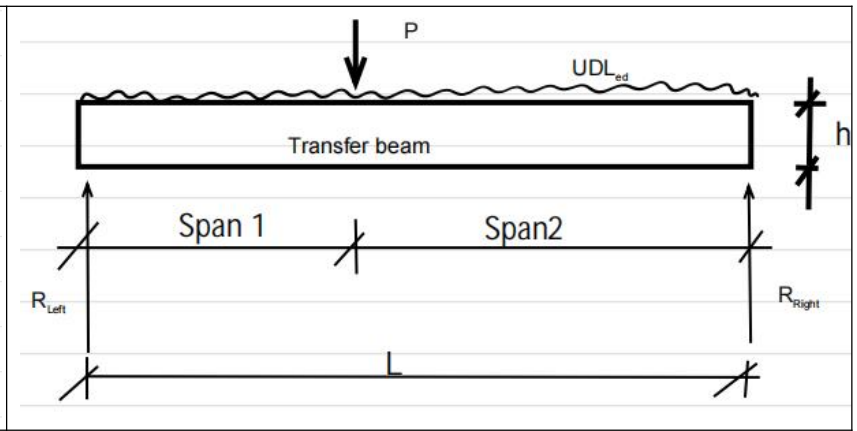


Figure 5- 27 Transfer Beams (Beam)

5.5 Wind pressure calculation

For the high-rise building, not only the horizontal force lever arm will increase along the height of the building, but also the pressure will increase accordingly (e.g. considering the wind fractions with the ground and the other buildings), which may lead to wind forces becoming one of the main parameters of structural design.
Wind load is a very random factor, and the actions of the building are very complicated.
By inspection, it can be found that the wind loading area from the west-east is more significant than the area from the north-south direction. Therefore, the west-east direction is the critical one considering the wind load. However, for the design, it can be found that the second-moment area in the W-E direction is smaller than in the N-S direction. Hence, the stresses induced by two directions need to be investigated.

5.5.1 Define the divisions of wind pressure

Code defines that the wind pressure varies along the height of a building for rectangular buildings. However, the building in this project has a complicated shape which can not be defined in the code. Hence, some assumptions have to be made to consider the geometry changes.
The building width b will need to be defined; the most conservative way is to define the b as the most extended length. However, in this case, b=40m in W-E and b= 32m in N-S are too conservative as the range of height of the width is only 12.7% of the total height of the building.
Therefore, it was defined as below.

In the West-East direction

Table 5- 2 Height of zones determination (W-E)

Width of the building in another direction	d	=	16	m
Conservatively consider the width of the building	b	=	32	m
the total height of the building	h_{tot}	=	68.7	m
As it's the case that	h_{tot}>2b	=	64	m
Therefore, it's the case that 3 zones for different wind pressure				

Zone No.	Range in Height m		
1	0	~	32
2	32		36.7
3	36.7		68.7

The geometry of the wind surface area is divided into 5 zones, and wind pressure has 3 zones with different values. Hence, 5 UDLs can be calculated accordingly (in Figure 5- 28 and Figure 5- 29).
UDL= corresponding wind pressure×equivalent loading length in defined geometry region
The detailed calculation for wind forces is presented in the calculation report.

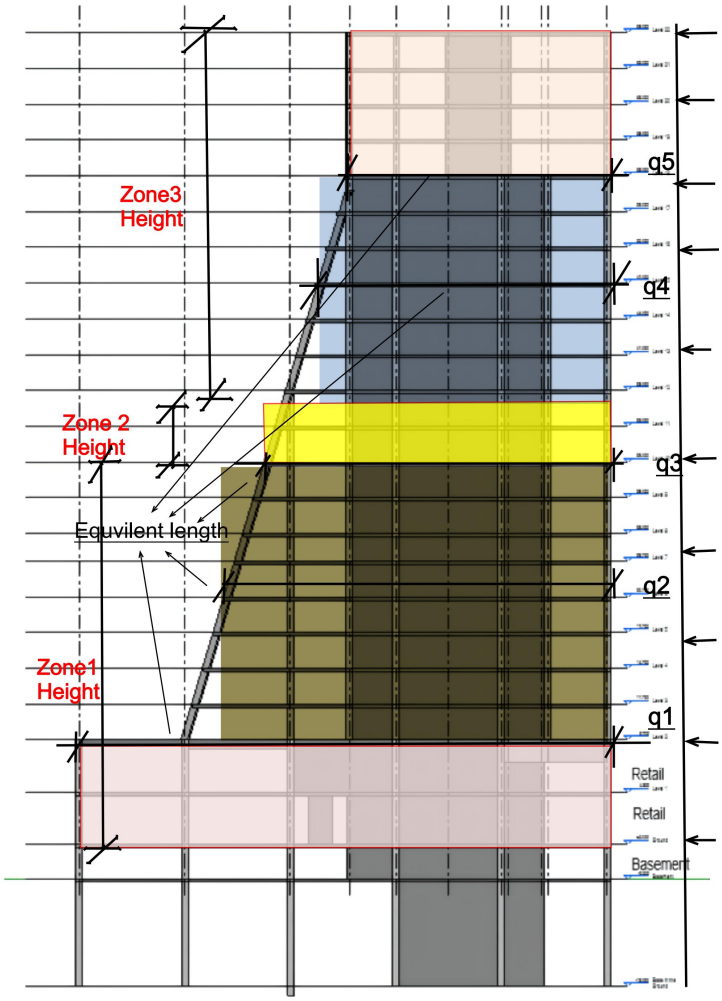


Figure 5- 28 Pressure zones and equivalent wind surface(W-E)

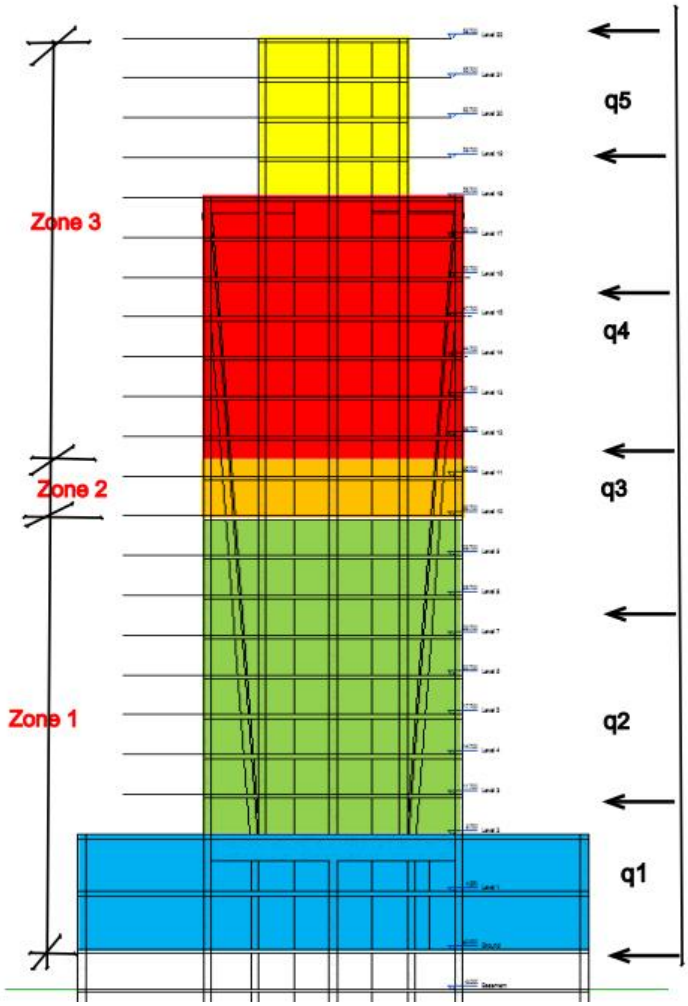


Figure 5- 29 Pressure zones and equivalent wind surface(N-S)

In the North-South direction

Table 5- 3 Height of zones determination (N-S)

Width of the building in another direction	d	=	32	m
Conservatively consider the width of the building	b	=	16	m
the total height of the building	h_{tot}	=	68.7	m
As it's the case that	h_{tot}>2b	=	32	m
Therefore, it's the case that 3 zones for different wind pressure				

Zone No.	Range in Height m		
1	0	~	16
2	16		52.7
3	52.7		68.7

6 Retaining walls design

Retaining walls are constructed to bind soils between two distinct elevations. A retaining wall is then predominantly subjected to lateral actions from the retained soil in addition to any other surcharge. Many retaining walls are of the cantilever form, but it is also common to encounter walls that are laterally restricted at the top, such as a basement retaining wall supported laterally by an elevated floor slab.

In brief, the ground conditions are shown in Table 6- 1.

6.1 Ground conditions

Table 6- 1 Ground information

Description	Depths below ground level	Soil Data	Notes
Made Ground	Ground-level to 3m		Sulfates found in ACEC class AC-3s should

			be taken
Dense sands and gravel	Below 3m	SPT 'N' blows = 40 φ' = 40°	

The ground water level should be taken as 1m below ground level.

6.2 Structural analysis and assumptions

In this project, the retaining walls in the basement are assumed to be pinned at the top and fixed at the bottom. Regarding actions, two types of actions will mainly focus on the earth pressure (static soil pressure, active and passive earth pressure) and hydro-static pressure.
For simplicity of hand calculation, active and passive earth pressure will not be considered as they need to consider the movement of the backfill.
The basement wall is 3m in height from the foundation to ground level. Therefore, the influence of groundwater should be considered, which is the increased soil pressure(saturated soil will have a higher bulk density than dry soil) and hydrostatic water pressure. The detailed load conditions are shown in figure*** and will be used to calculate moments and shear forces to design flexural and shear reinforcements (detailed calculations are provided in the calculation report).

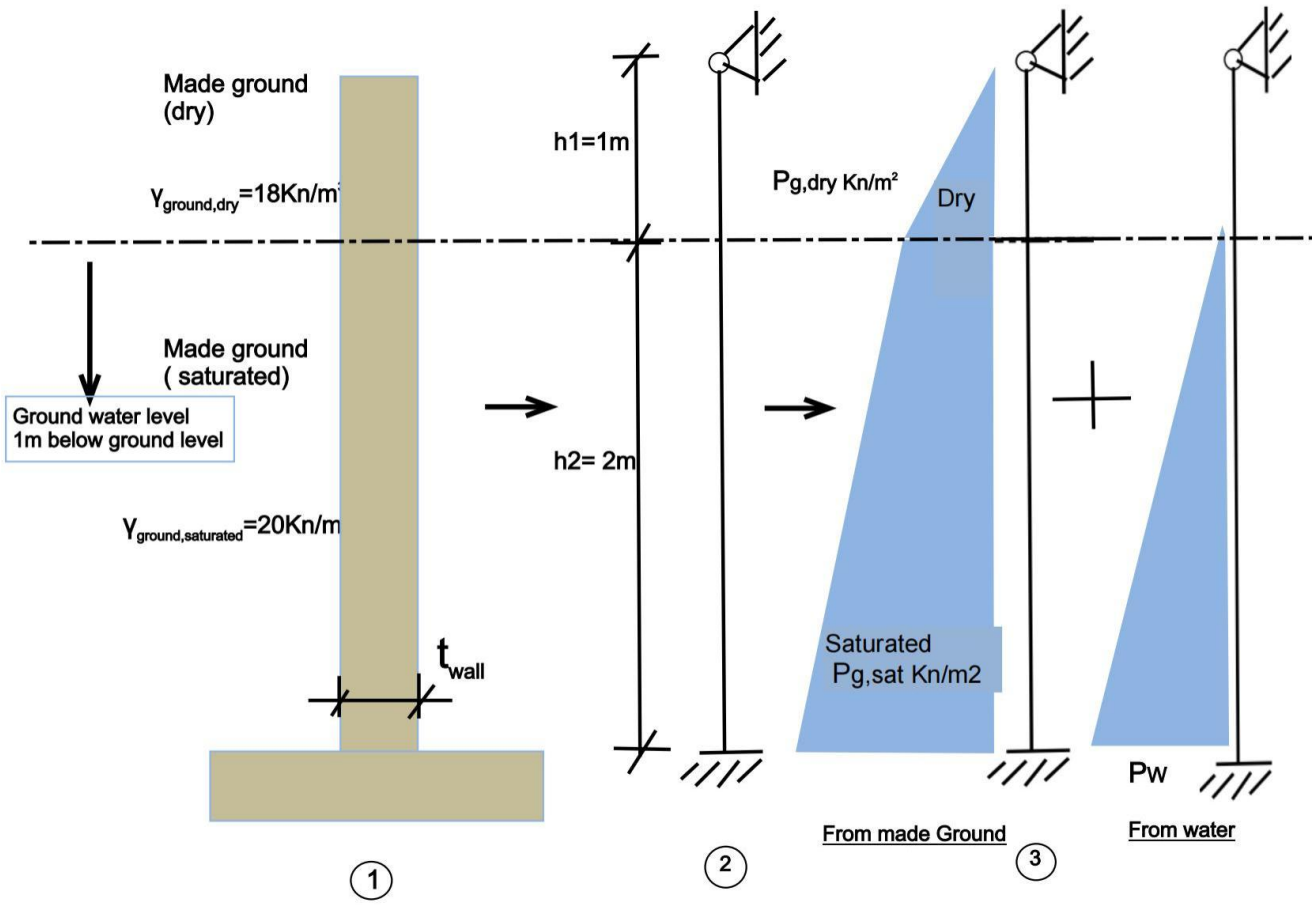


Figure 6- 1 Boundary condition and actions on the basement wall

6.3 Crack control considerations

In basements and retaining structures, leakage "through" cracks that traverse the entire thickness of the member is the key concern. Because of this, crack control becomes essential in constructing basement walls, particularly in groundwater environments. At the serviceability limit, crack widths must be limited to a required level.

Meanwhile, edge restraint will also happen in basement walls as the fresh concrete wall pours on the hardened foundations. For this reason, cracking due to edge restraint and early thermal effect must be checked in this project.

7 Shear core design and investigation (Research component)

The shear core design is mainly based on shear wall design procedures to check whether deflection and drifts in each level satisfy SLS requirements.

$$\text{Allowable deflection or drift} = \frac{h}{500} \quad \text{(h for total building height in deflection and storey height for drift)}$$

Meanwhile, the extreme fibre flexural compressive stress $f = \frac{N}{A} \pm \frac{M}{Z}$ under critical load combinations needs to be calculated to design section dimensions, concrete grades and reinforcement.

The suitable structure model and assumptions are essential to the design parameters influencing the design's safety and efficiency.
Therefore, three parts will be investigated and discussed as shown below:

- Part 1-Rectangular hollow section design assumption validation

For RHS equivalent cantilever model validation, the results in the 2D hand calculation model will be compared with the 3D FEA model (assume core takes all horizontal load only).

- Part 2-Mesh setting for wall deflection and drift analysis
- Part 3-Efficiency study between core only and wall-frame models

Frame action is non-negligible in deflection resistance and influences the other design parameters, especially in tall buildings with widespread columns.
For this reason, core wall and wall-frame models will be compared in their efficiency to resist bending action and deflection.

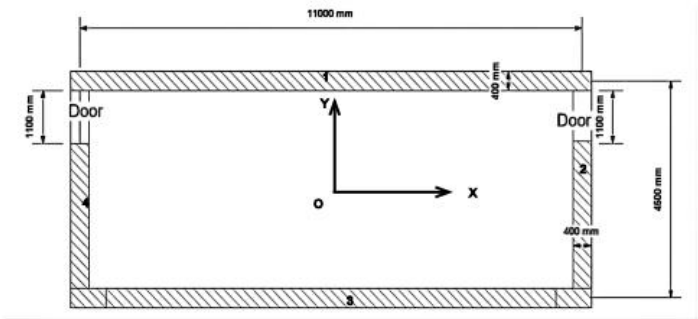
- Part 4-Investigation about how in-plane slab stiffness influences core and core-frame behaviour

In the common design, we may want to design a shear wall braced structure specifically; the designers usually try to change the stiffness of slabs to mock the condition that all loads are taken by the shear walls(slabs).Therefore, how to set the parameters is approaching enough to the case that only the core takes all of the lateral force to brace the system.
At the same time, how the deflection, drifts and the overturning moment change with the change of stiffness will be investigated.

7.1 Part 1- RHS cantilever beam model validation

7.1.1 Propose for shear core design

In a frame-core system, as the bending resistance stiffness in the core is much higher than that of columns, the common method for designing a shear core is to assume all lateral forces are taken by the core only.



As the core is composed of four directions shear walls, analysis and design for individual wall element may not suitable for shear core.
Therefore, the design and calculations for the shear wall can be proposed as an equivalent open rectangular hollow section(RHS).
The RHS core will act as a cantilever beam with different stiffness in different heights and directions (as the layout and dimensions for the upper core are different to that of the lower core).

Figure 7- 1 Equivalent RHS for shear core

In 2D frame calculation, a simplified RHS cantilever will undertake 5 different uniforms distributed load as calculated in wind force part.
For deflection, SLS is applied, and deflection at each floor can be calculated. The drift is the difference between the displacement of different floors, which must also meet the requirements.
Even though the loading area in the W-E direction is more significant than in the N-S direction, the stiffness of cores in W-E is less than in the N-S direction. For this reason, the deflection and drift checks in both directions are necessary.
The details for hand calculations are presented in the calculation sheet.

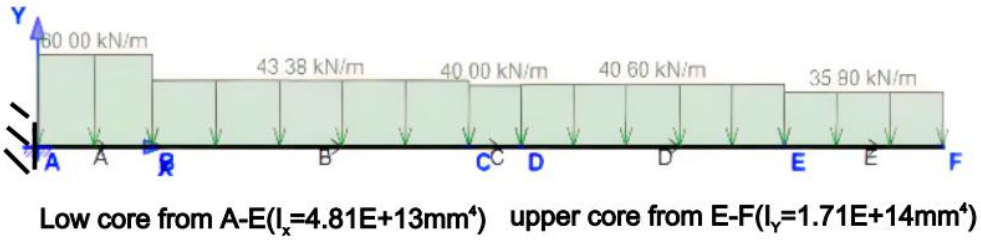
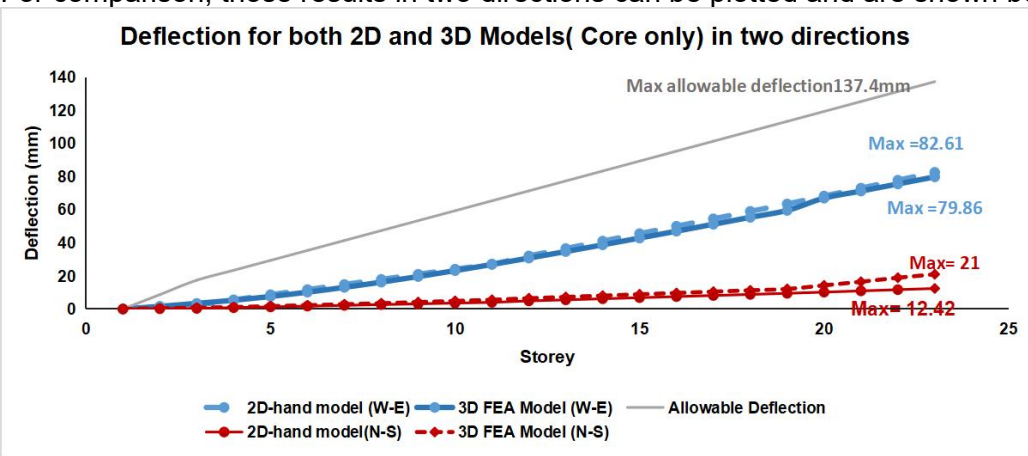


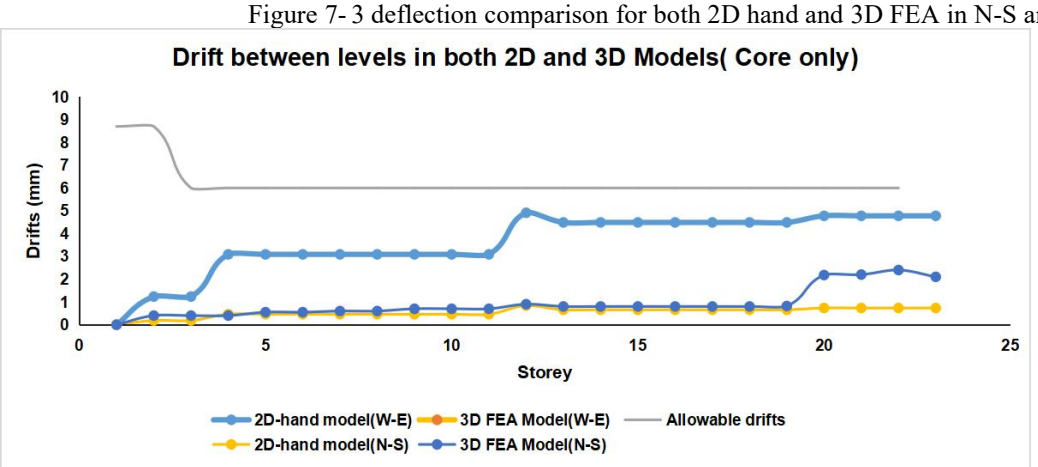
Figure 7-2 2D Equivalent cantilever beam

For comparison, those results in two directions can be plotted and are shown below



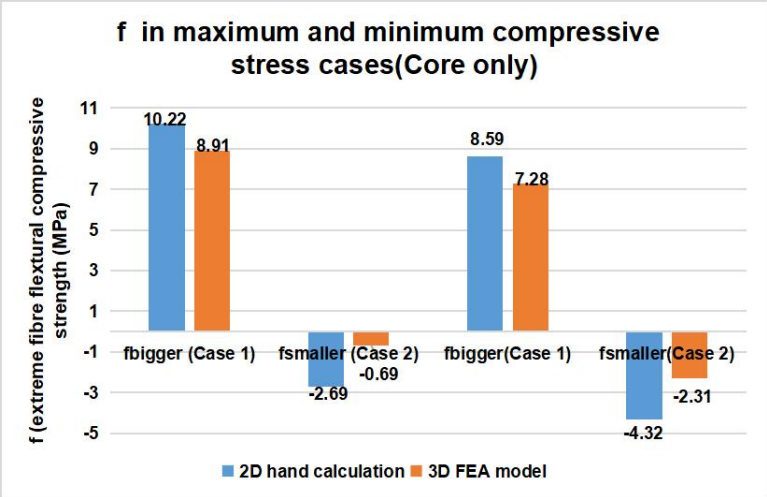
The deflection (Figure 7-3) in 2D hand calculations and the 3D FEA model are almost the same.
The deflection in the W-E direction is approximately 4 times that in the N-S direction.
All deflections satisfy the SLS requirement.
Therefore, in the later design and discussion. W-E direction will be mainly investigated.

Figure 7-3 deflection comparison for both 2D hand and 3D FEA in N-S and S-W direction



All drifts at LHS are within the limit of the SLS requirement, and it can be found that the deflection check for this project is controlled by drift which approaches the limit value.

Figure 7-4 drifts comparison for both 2D hand and 3D FEA in N-S and S-W direction



Two extreme load cases are considered:

- Case 1-1.5Wk+1.35Gk+1.5 ×0.7Qk(maximum compressive stress)
- Case 2- 1.5Wk+1.0Gk(minimum compressive stress)

The extreme fibre flexural and compressive stresses f are similar, while the 3D FEA model is smaller than the 2D hand calculation. This is because the assumed loading area for hand calculation is more significant than the automatically calculated axial load on the core. (for conservative consideration.)
*Note:"+" in compression, "-" in tension

Figure 7-5 the stresses f in maximum and minimum compressive stress cases(core only)

The results(Figure 7-5) justify that assuming the shear wall core structure acting like an RHS cantilever beam is an ideal way to do 2D hand calculation for core in terms of deflection, drifts, and overturning moments.
It is worth noting that the reason that cantilever deflection can not completely match the standard cantilever deform shape and the drift are not uniform is that the core's UDLs will change according to the change of geometry and height(details for wind load calculations are presented in calculation report).

7.2 Part 2- Mesh setting for wall deflection and drift analysis

Mesh size is always an essential parameter for FEA analysis which will influence the accuracy of results and efficiency of computer operation. Typically, a finer mesh size will increase the accuracy of results but decrease the speed of running and burden for the computer.
In this part, a serious of FEA core models with different magnitude of mesh sizes are modeled in ETABS to find how the mesh size interacts with the results of deflection and drifts.

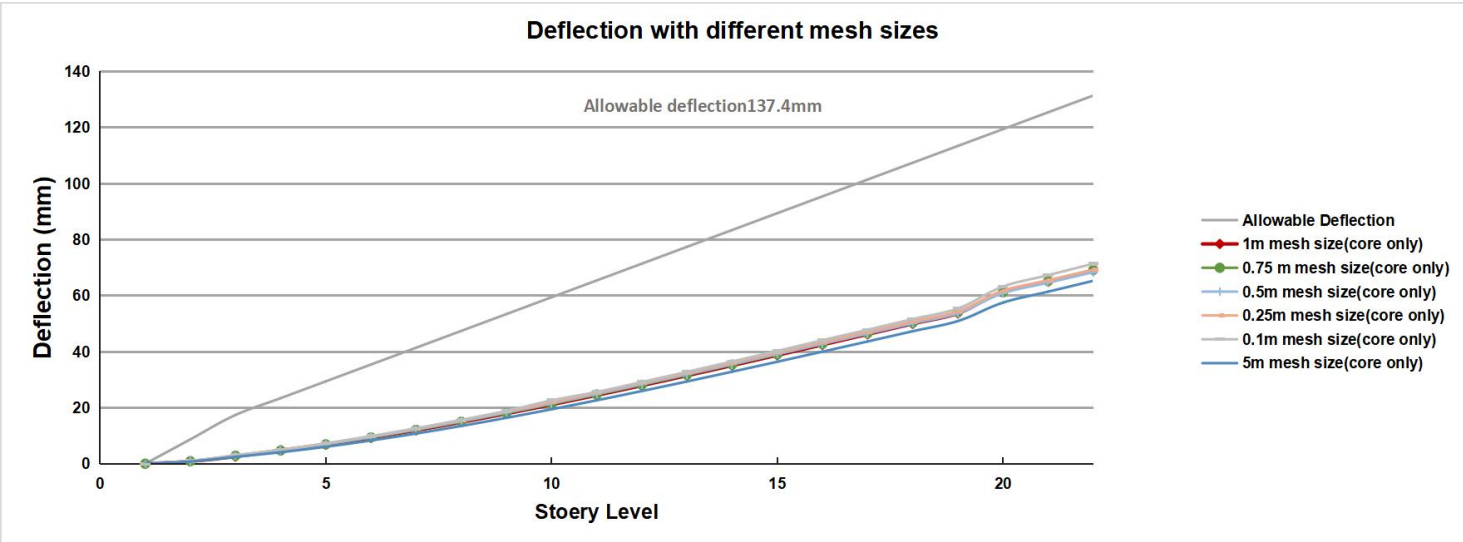


Figure 7-6 Deflection with different mesh sizes for core only structure (mesh size: 0.1, 0.25, 0.5, 0.75, 1 and 5m)

As Figure 7-6 shows, the overall trend and magnitudes of deflection results are the same with the different mesh sizes (5m, 1m, 0.75m,0.5m, 0,25m and 0.1 m). Although the deflection from level 15 to level 22 may fluctuate, this may be caused by the errors in the manual reading of the results from the FEA model.

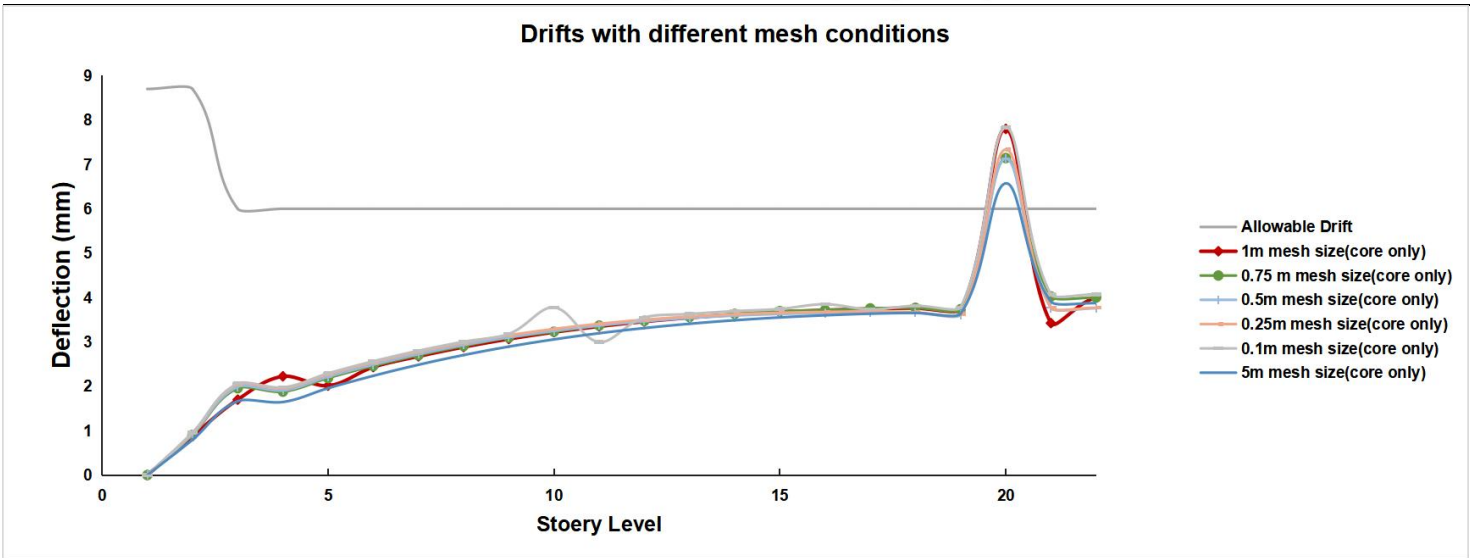


Figure 7- 7 Drifts with different mesh sizes for core only structure (mesh size: 0.1, 0.25, 0.5, 0.75, 1 and 5m)

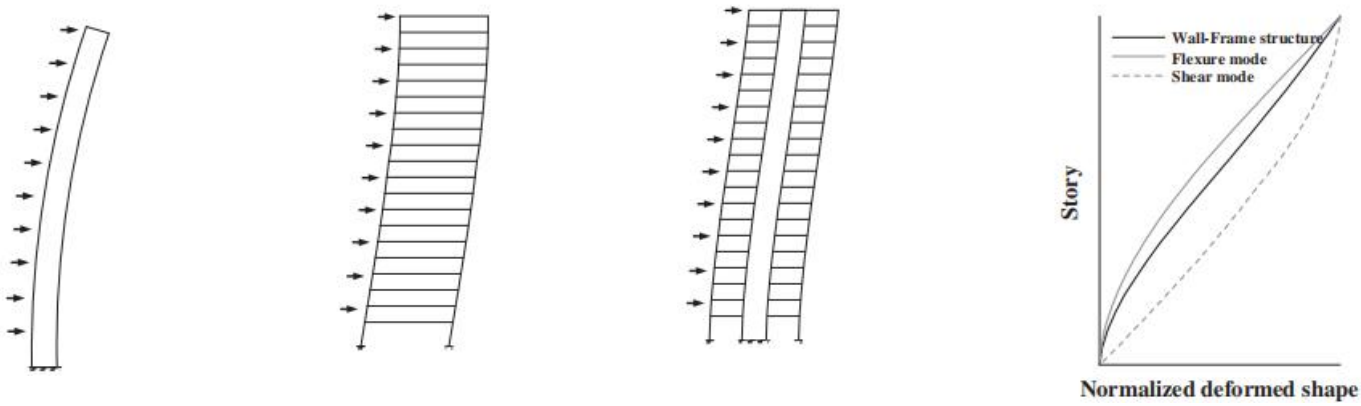
The drifts (Figure 7- 7) in all models in every storey are almost the same, while there is a sharp drift at level 19, which does not satisfy the allowable drift. The upper and lower core connections cause the sudden drift. (In the 2D model, the core transfer is assumed to be continuous, which is why there is no sharp drift in the 2D hand calculation). This may rely on special treatment and design for the layout change of the core.

7.3 Part 3-Efficiency study between core only and wall-frame models

7.3.1 General discussion and analysis

However, the reality is that the shear walls and the frame jointly withstand lateral forces as the outside slabs and columns will also provide lateral force resistance, shear force resistance, and stiffness for the structure.
The shear wall generally exhibits bending behaviour, and the shear frame deforms. As illustrated in Figure 7- 8, the deformed shape of the shear wall–structural frame systems is therefore defined by the interaction between the shear wall in bending mode and the frame in shear mode.

The bending mode in the core wall is a standard deformed cantilever form, it has a single curvature, and as the story progresses, its lateral displacement grows. In contrast, the shear mode exhibits a double curvature, and substantial displacements are seen in the middle stories. Moreover, the more columns and the wider the distribution of columns are, the more the frame action can not be ignored.
As seen in Figure 7- 8, the distorted shape of a shear wall–structural frame system occurs between bending and shear modes. To create an effective equivalent analytical model of the shear wall–structural frame systems, it is necessary to account for both shear and bending modes.
Therefore, not only will the deflection results from hand calculations by the 2D simplified model and 3D core FEA model results be checked, but they will also compare with the results in the 3D core-frame FEA model.
(Yong-Koo Park, 2014)



(a)core wall (b) frame (c) wall-frame Deformed shape

Figure 7- 8 Deform shape for core wall, frame and wall-frame structures

(Yong-Koo Park, 2014)

7.3.2 Efficiency comparison between core and core-frame structure

In ETABS, both 3D core and core-frame are modelled, the deflection contours are shown in Figure 7- 9and Figure 7- 10

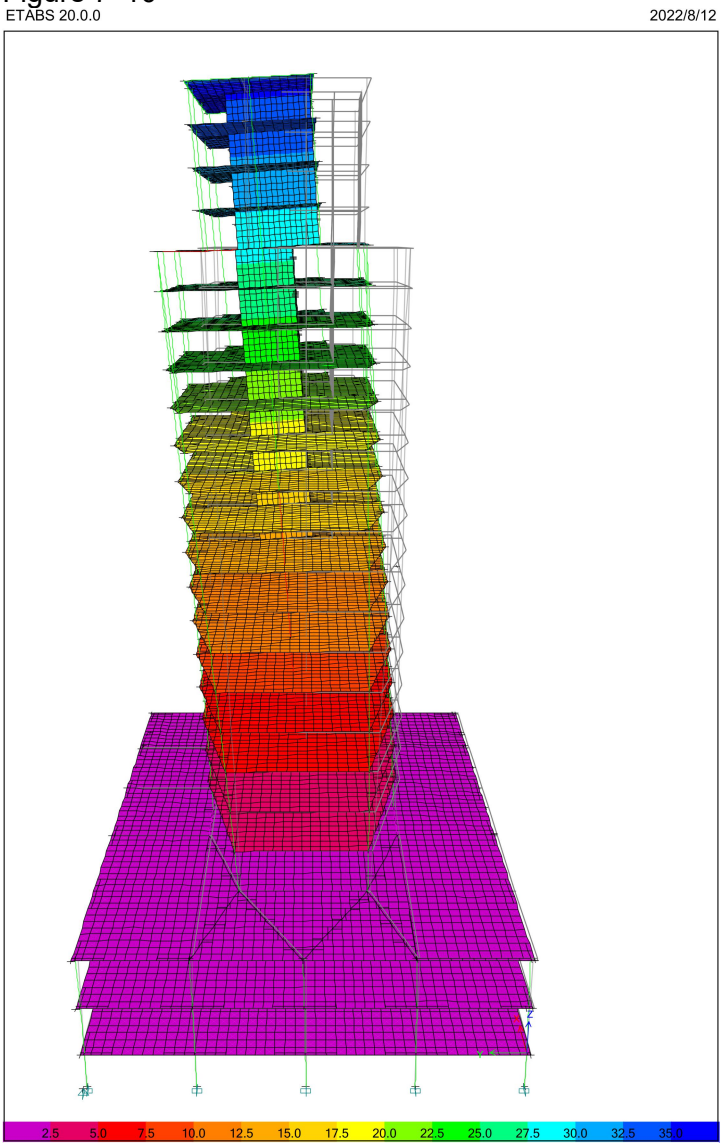


Figure 7- 9 Deflection contour for core-frame FEA model (W-E)

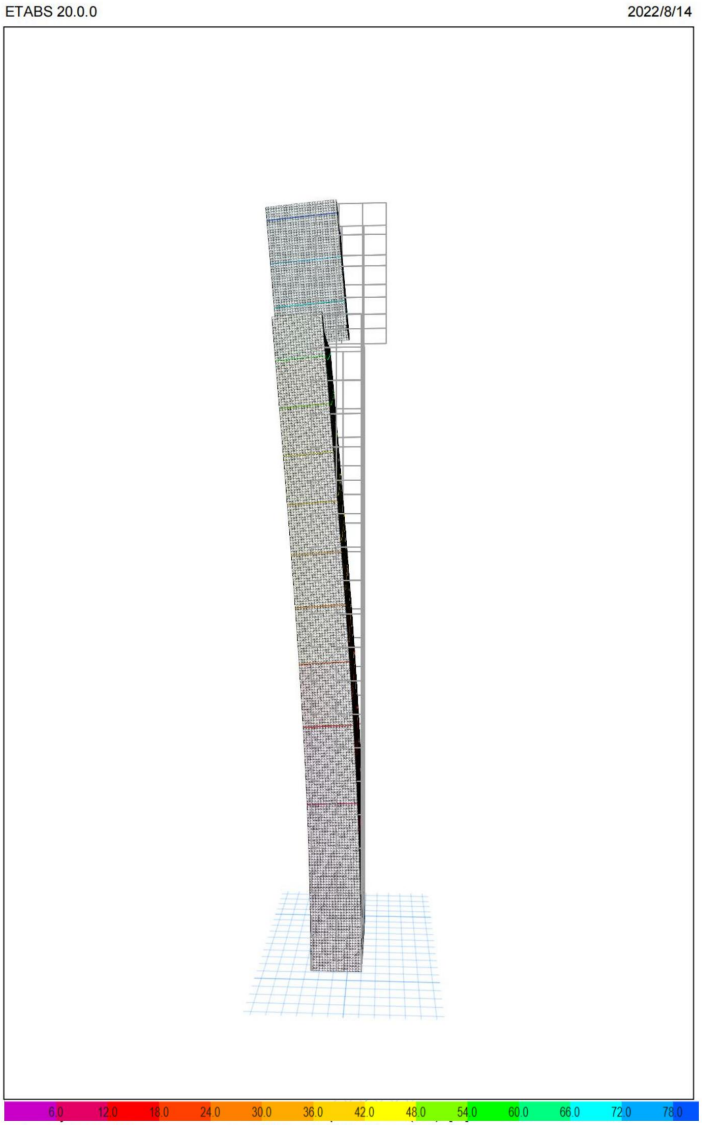


Figure 7- 10 Deflection contour for core FEA model (W-E)

Meanwhile, the axial load for each assigned piers(shear walls) induced by factored wind load and notional horizontal loads can be output. Then the extreme fiber flexural, compressive stresses f can be calculated accordingly.

Table 7- 1 Design parameters comparison between the core and the frame-core models

		Case 1		Case 2		$Deflection_{max}$	$Drift_{max}$
		f_{bigger}(MPa)	$f_{smaller}$(MPa)	f_{bigger}(MPa)	$f_{smaller}$(MPa)	(mm)	(mm)
Core only	2D hand	10.22	-2.69	8.59	-4.32	82.61	4.78
	FEA	8.91	-0.69	7.28	-2.31	79.86	4.28
Core-frame	FEA(without stiffness reduction)	4.82	2.74	3.19	1.11	35.48	1.72

Efficiency(%) (Compared with results in Core-frame and 2D hand Core only	**-53%**	-202%	-253%	**-126%**	**-57%**	**-64%**

As shown in Table 7- 1, by considering the frame-core action, the design compressive, tension stress, and control parameter(deflection and drifts) can be reduced on a large scale in this project(53% in design compressive stress, 57% in deflection and 64% in drift), which is more economical and structurally efficient.

7.4 Part 4-Investigation about how in-plane slab stiffness influences core and core-frame behaviour

7.4.1 FEA setting

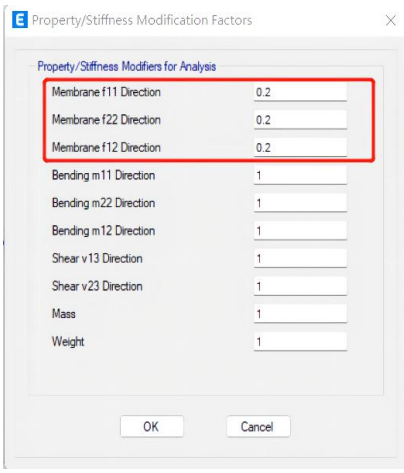


Figure 7- 11 The setting for reducing slab in-plane stiffness in FEA model

One way to simulate the lateral force only taken by core structure is to build an individual core model. However, the simulation may also be done by reducing slab stiffness to change the interactions between frame and core.

In ETBAS, the bending stiffness M11, M22, and M12 will mainly influence flexural behaviour identified as in-plane stiffness, while the membrane stiffness f11,f22, and f12 are in-plane stiffness influence horizontal load transfer.

To investigate how in-plane slab stiffness interacts with core and core-frame behaviour, we will analyse how design parameter(deflection, drifts and critical stress in walls) changes with the change of in-plane stiffness.

The 3D model for the whole structure is constructed in ETABS. The membrane stiffness f11, f22, and f12 with factors 0.3, 0.2, 0.1, 0.001 will be input and investigated.

Theoretically, suppose the slab is strong enough to transfer horizontal forces, which means the in-plane stiffness is high. In that case, columns can be well-mobilized to resist lateral forces, providing lateral stiffness and sharing the resistance to overturning moment. With the decrease in in-plane slabs' stiffness, the interaction between the core and the columns will decrease, and the core-frame structure will be more likely to behave as a core structure rather than a core-frame structure.

7.4.2 Deflection for core and core-frame model with different stiffness factors

The deflection results with different stiffness factors in each level will be manually recorded. For comparison, the deflection for core only conditions will also be plotted, the deflection values for core and core frame FEA models with different stiffness factors are shown in Figure 7- 12.

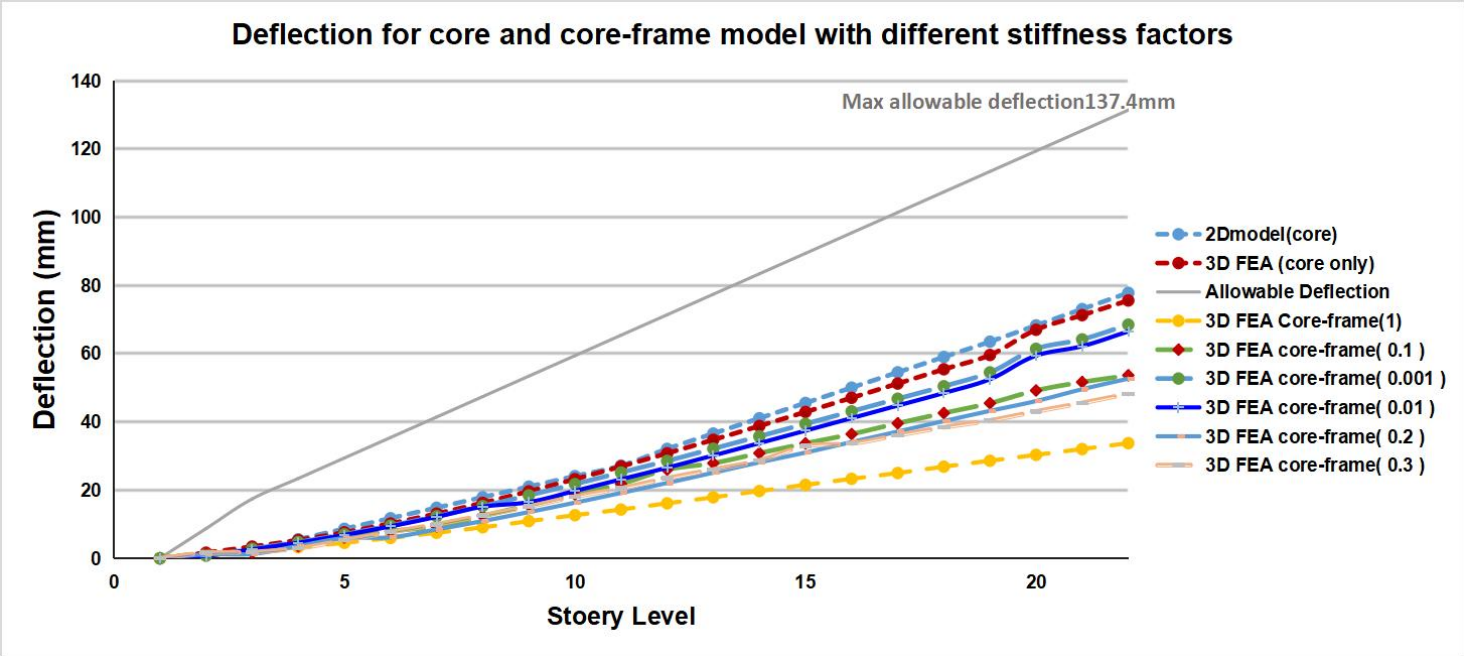


Figure 7- 12 Deflection for core and core-frame FEA with different stiffness factors for flat slabs

It can be found that the overall trend for deflection is the same. The maximum deflection condition will be only core to take all the horizontal load. The minimum deflection condition will be slabs with total stiffness(stiffness factor=1). The model results are well matched with the theoretical prediction that as the stiffness factor decreases from 1 to 0.001, the deflection in all floors will increase accordingly. When the stiffness is small enough, the deflection results in core-frame structure will infinitely approach the results in core structure only.

7.4.3 Drift for different stiffness factors for core-frame models

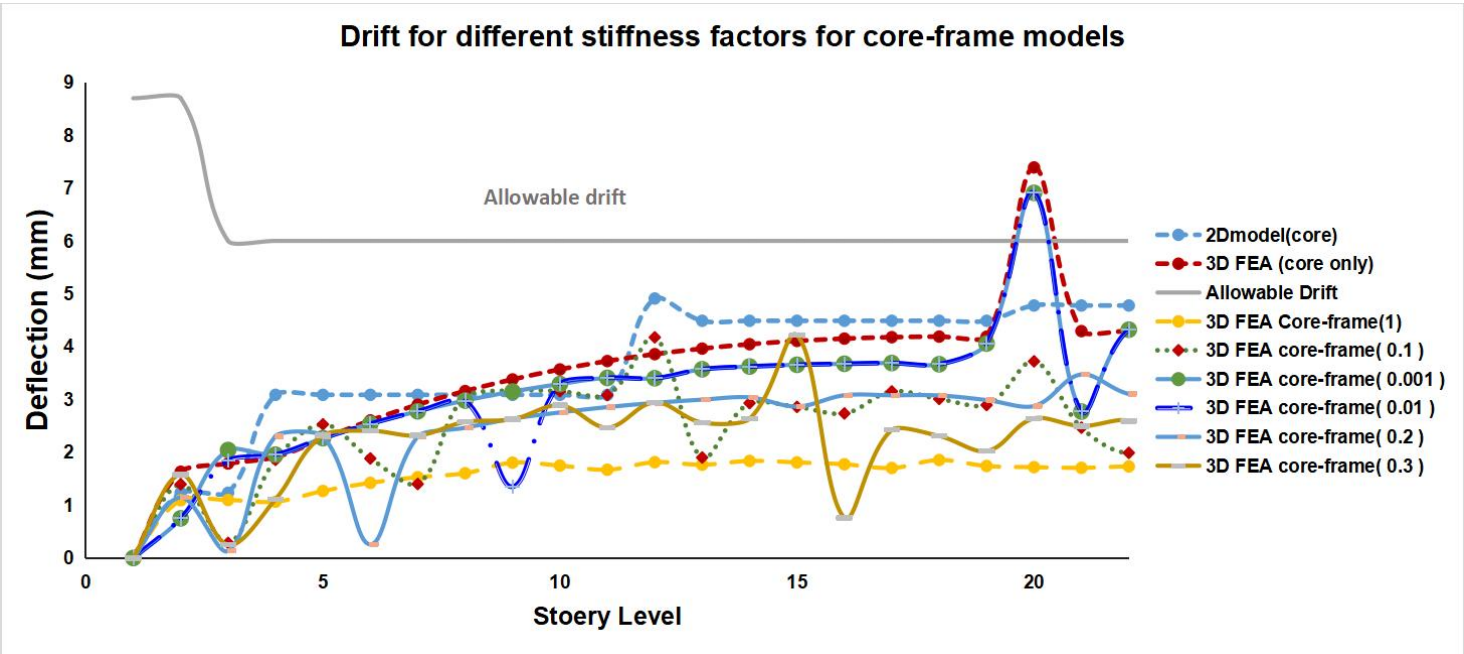


Figure 7- 13 Drifts for core and core-frame FEA with different stiffness factors for flat slabs

The results of drifts are quite unstable and unpredictable, and most drifts satisfy the allowable drift on every floor except the one on level 19. One reason contributing to that is the manual error. Another reason is that the horizontal slabs will give lateral action(pull and push). The frame and slab layout dimensions vary along the height of the building (which means very complex stiffness conditions), and the frame actions to the core are non-linear. For this reason, drifts will be unpredictable and have a non-linear behaviour.

7.4.4 f for different stiffness factors in core-frame FEA model

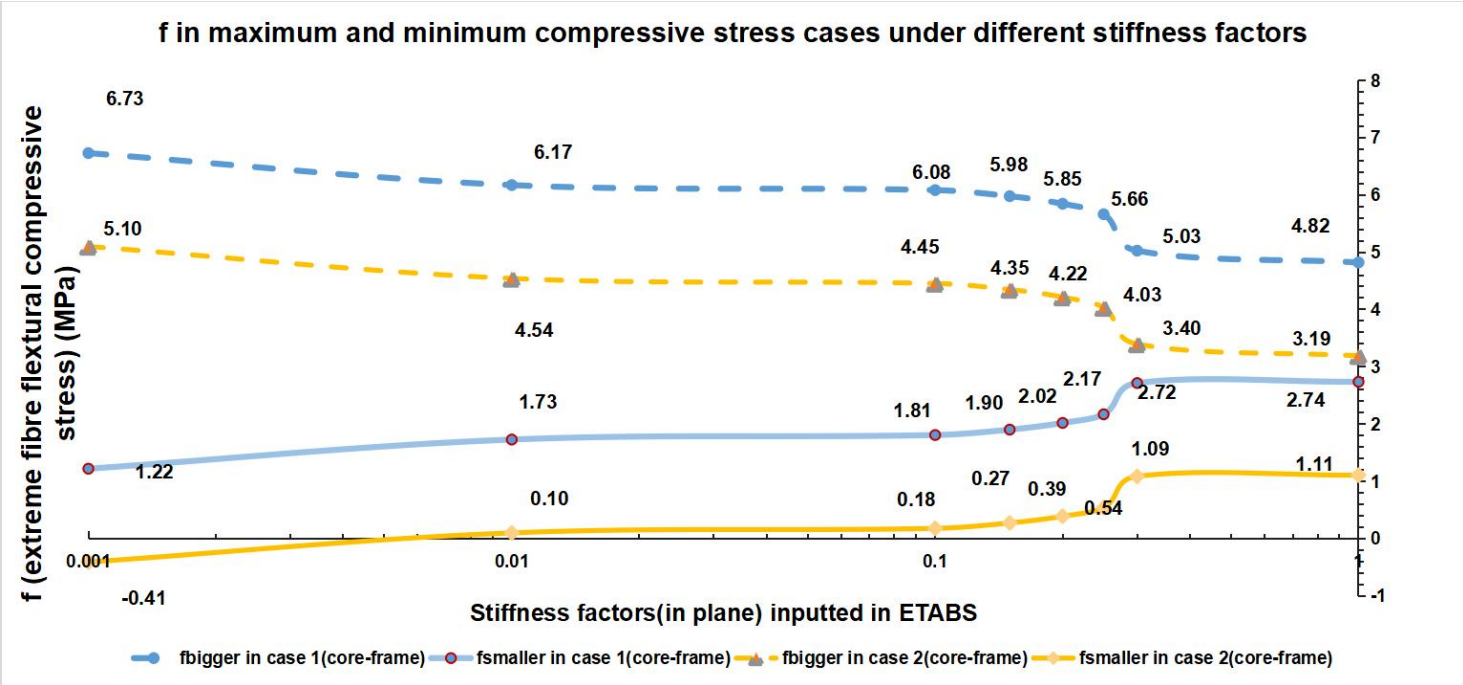


Figure 7- 14 Extreme fibre flexural compressive stresses f for different stiffness factors in core-frame FEA model

As predicted before, with the increase of the stiffness factors, the f_{bigger} will decrease, and the $f_{smaller}$ will increase in the maximum and minimum compressive stress load cases as predicted before. One thing worth noting is that the stresses vary more rapidly when the stiffness factor increases from 0.001 to 0.3 than when the factor increases from 0.3 to 1. Which means the stress changes is very sensitive when stiffness in the region from 0.001 to 0.3.

7.5 Conclusion

In the research part, the first part demonstrated that the design shear core as an RHS cantilever beam is reasonable and well-matched with the 3D FEA model; wind blow from the N-S direction is the critical direction for shear wall design. The deflection and drifts in each level satisfy the SLS requirement.
The second part shows that the mesh sizes within 5m do not influence deflection and drifts.
The third part shows that by mobilizing columns and slabs in this system (considered a core-frame structure), the design compressive stress(f_{bigger} in case 1) can be reduced by 53%, and the design minimum stress case($f_{smaller}$ in case 2) can be reduced by 126%(in compression after considering a core-frame structure, no need for additional design reinforcement to resist tension). Moreover, the deflection and drifts can be reduced by 57% and 64%, respectively. Less concrete and reinforcement are needed for designing a shear core. At the same time, more elements are mobilized, which means less cost and structural efficiency.
Part 4 shows that by reducing in-plane stiffness factors for slabs in the whole structural model, the deflection will infinitely approach the deflection of the core-only model. Drift would be unstable and unpredictable in core-frame structure caused by the lateral elements, which will provide lateral forces, stiffness, and manual reading error. Moreover, as expected, by mobilizing columns, the overturning moment induced by horizontal forces in the core will decrease, and the stress change will be more sensitive to stiffness changes in the region(0.001-0.3) than the stiffness changes in the region (0.3-1).

8 Foundation design

A raft foundation is preferred for a multi-storey building with a basement.
Advantages of raft foundation
For commercial buildings, the loads are much larger. The raft foundation not only spreads the stress to the ground but also solves the problem of overlapping for individual wall footing design.

The design of the basement floor slab and foundation can be combined, and construction time and cost can be saved.
Moreover, raft foundations can prevent differential settlement, which happens when the ground surface of a structure settles at various rates, thereby reducing cracking and other more significant issues.

The general parameters for raft foundation design

The buried depth should be higher than 1/12 of the height of the building from the ground. Therefore, take the buried depth as 6 m.
A slab raft foundation is applied as it is easy to construct and benefits the height of the building.
The thickness of the foundation should be calculated based on the reaction from the base.
The plan view size of the raft should be determined according to the foundation's bearing capacity, the superstructure's layout, and the load distribution.
The concrete grade of the foundation is 50 MPa with a high permeability grade.
Raft reinforcement rate is generally 0.5%-1.0%, as appropriate. Smaller diameters with a closely spaced arrangement are recommended to provide better bending and crack resistance behaviour. Hence, double layer reinforcements with a 16 mm diameter and 200 300 mm spacing will be applied.

Matters needing attention during construction

It should be noted that raft foundation construction must be handled properly to against edge erosion. They are ineffective if the building's weight is concentrated at a single location, but this is uncommon in residential construction and so not usually a cause for worry.
The foundation design needs to ensure sufficient resistance to the uplift forces and take measures to avoid uplift during construction.

9 Construction procedure and method statement

General contents of construction procedure

- Preliminaries

The site set-up should be outlined; particular issues to consider are:Interfaces with the public, Interfaces with adjacent properties, Site logistics, Handling of materials, Plant and equipment to be provided, e.g.cranes

- Construction survey scheme

Building plan control,Personnel organization,Equipment preparation, Vertical control, Elevation control

- Foundation pit earthwork excavation project
- Artificial hole - in - place pile and bolt - shotcrete support engineering
- Reinforcing bar engineering Steel bar cutting processing, binding, Wall reinforcement binding, binding for frame columns, Flat bar binding, Stair steel binding, after pouring belt and tower crane reinforcement treatment, measures to control deviation of rebar
- Framework engineering: Template layout, primary and secondary braces and bolt selection, Template system design(columns, beams, staircase, flat slabs, walls, after pouring), construction for frameworks(e.g. for columns stringing, Leveling, positioning, processing or pee-assembling column models, installing column molds, installing tie rods or diagonal braces, correcting perpendicularity, check)
- Concrete engineering works: Construction organization, concrete pouring, construction joint setting and treatment, post-pouring belt construction, concrete maintenance and protection
- Waterproof engineering →Construction of scaffolding→Internal walls engineering→Roofing construction→ Doors and Windows project→Finishing engineering→Other decoration, HVAC and other sub-projects

Safety construction considerations

- For construction safety, the following suggestions are made:
- Reduce the requirement to work at height.
- Reduce the requirement to work in excavations and confined spaces, e.g., minimise the depths of foundations.
- Avoid the specification of materials that are difficult to handle (e.g., long lengths of 40 mm diameter reinforcing bar).
- Clearly convey the lateral stability requirements for the permanent works. Avoid the specification of techniques that generate noise.
- Minimise the use of techniques that cause vibration. Consider the depth of the groundwater in the design.

- Reduce the risk of contact with contaminants, e.g., specify a thorough site investigation if there is a risk of contamination identified in a desk study.

10 Sustainability

Sustainability is not only about decreasing environmental effects but also about balancing social, economic, and environmental costs and benefits over time.
Generally speaking, to increase sustainability, passive and positive measures will be considered.
The passive measure means the method of reducing energy consumption, while the positive way is to increase clean energy generation.
The embodied carbon dioxide content (i.e., the carbon dioxide created during the structure's creation and construction) is negligible compared to the carbon dioxide produced over the building's lifetime by its heating and lighting systems. The longer a building lasts, its carbon dioxide emissions become less important.
Therefore, increasing the efficiency of the structure's lighting and HVAC(heating, ventilation, and air conditioning) systems is the key to reducing energy consumption.
This part will investigate two passive measures(structural design for floor system and architectural design for double-layer glass curtain) to increase the efficient use of sunlight to reduce reliance on artificial lighting and one positive measure (solar panels) to utilise clean energy to achieve the goal of sustainability.

10.1 Lighting comparison between flat slab and traditional beam-to-slab system

The first part compares indoor lighting between beam-less structures and beam structures.
The main difference between proposals 1 and 2 is the floor system, and proposal 1 applied a flat slab while proposal 2 applied proposal 2. A flat slab with beams can offer permeable light, which means less light loss through obstacles.
Through the comparison of indoor lighting simulation data and figures of the two models, it can be proved that the natural lighting of beam-less structures is better than that of beam structures, and the power loss generated by lighting is lower, which is more energy-saving.
The information for sunlight is assumed the building location in Haikou, China, a seaside and regional city centre.
The light illumination analysis contours for beam-less and with beam floor under natural lighting are shown in Figure 10- 1 and Figure 10- 2.

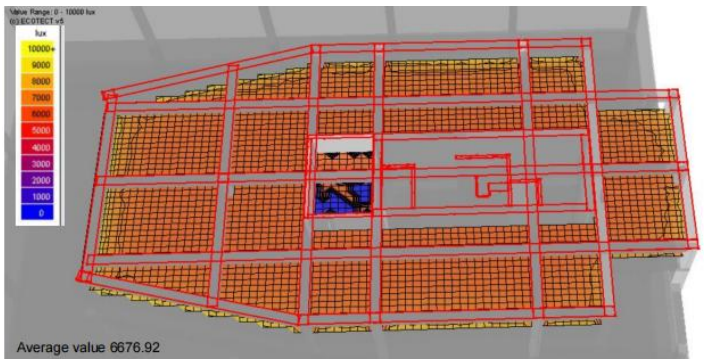


Figure 10- 1 Lighting contour for slab with beams floor

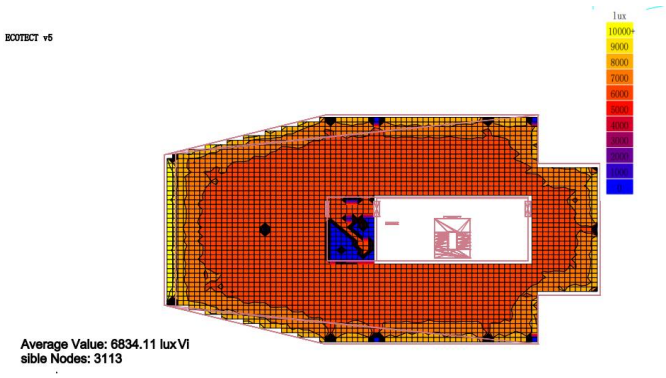


Figure 10- 2 Lighting contour for flat slab floor

Average values for illumination	
Flat slab	Slab with beams
Lux(lighting unit)	Lux
6834.11	6276.92
Efficiency(%)	**8.15%**

By comparison, using a flat slab, the efficiency of natural lighting can be increased by 8.15%.

10.2 HVAC energy comparison between double and single-layer glass curtains

The second passive measure is by setting double layers of glass curtains to reduce the heating, ventilation and air conditioning(HVAC) energy consumption.
Air is an ideal medium for heat insulation and sound insulation by setting the structure of double glazing to reduce the speed of indoor and outdoor temperature exchange to maintain a relatively stable temperature and achieve energy savings.
In this part, by simulating floor 3 as a case, the energy consumption difference between the HVAC with a single-layer glass curtain and a double-layer glass curtain is obtained to obtain the energy-saving efficiency of the double curtain wall adopted in this proposal, and then prove the sustainability of the building project..

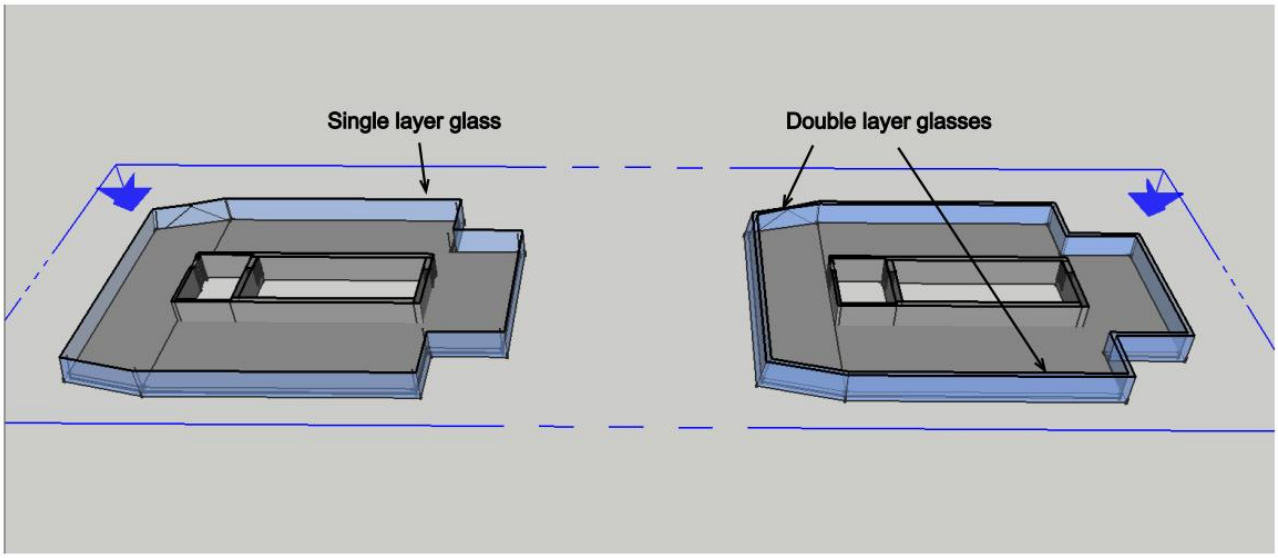


Figure 10-3 Single- and double-layers glasses （cladding) models in SU

After the draw, the models in SU are input into Autodesk Ecotect. By setting the HVAC system to maintain the indoor temperature between 18℃ ~ 26℃, the energy consumption for heating and cooling data can be obtained as shown in Figure 10-3 for the single-layer glass model and the double-layer glass model.

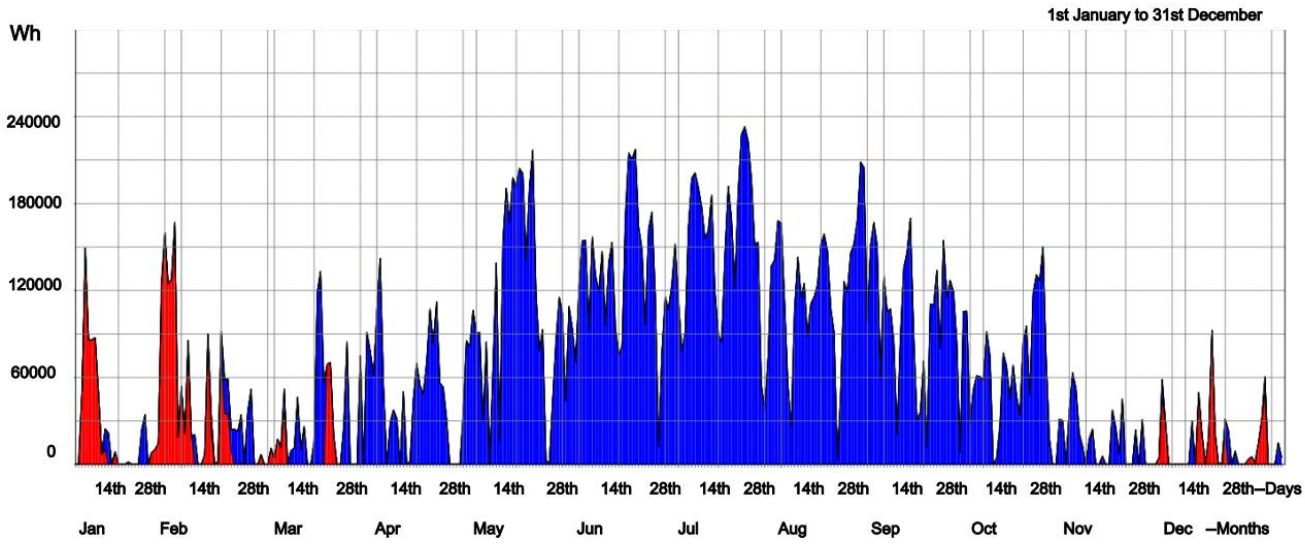


Figure 10-4 Energy consumption of sir-conditioning cooling and heating (double-layer glass)

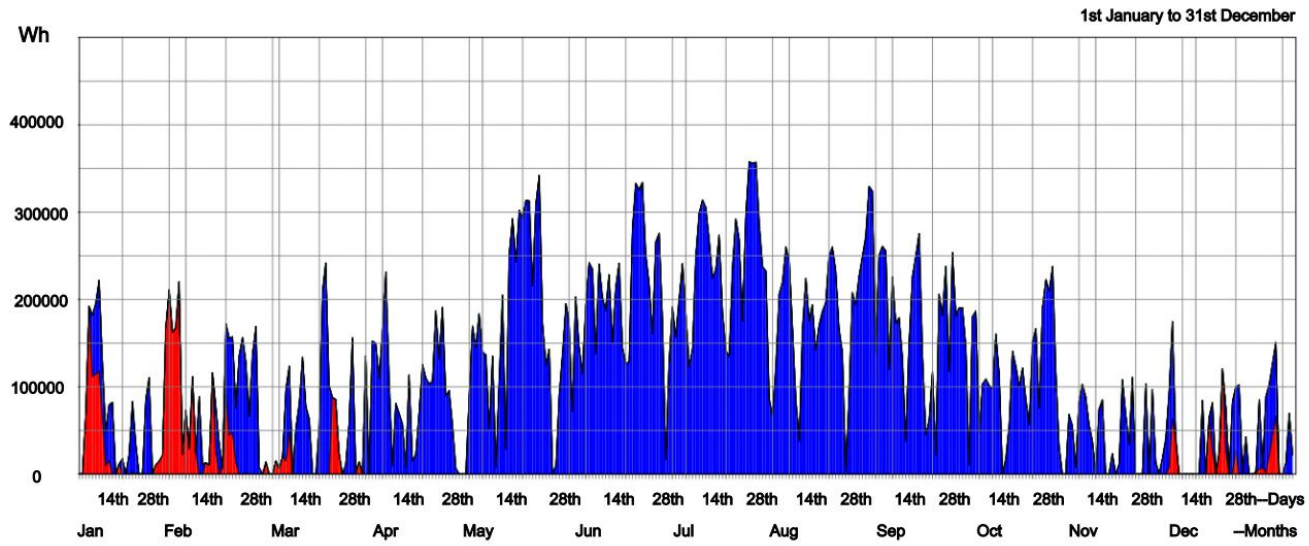


Figure 10- 5 Energy consumption of sir-conditioning cooling and heating (single layer glass)

By comparing Figure 10- 4 and Figure 10- 5, the overall energy consumption for HVAC in the double-layer model is much lower than that in the single-layer model. The peak values for HVAC use are between July and August, which are 220000Wh in the double-layer model and 350000 in the single-layer model.

10.3 Parametric analysis by replacing the glass curtain wall at the slop with solar panels

Generally speaking, lighting from east-to-west directions is sufficient for service use. In contrast, over-natural lighting will lead to many problems, such as an increase in the burden of the cooling system and light pollution. A positive measure that not only mitigates the excessive lighting intensity but can also generate clean energy is applied in this project, which is by replacing the sloping facade with solar panels (shown in Figure 10- 6).

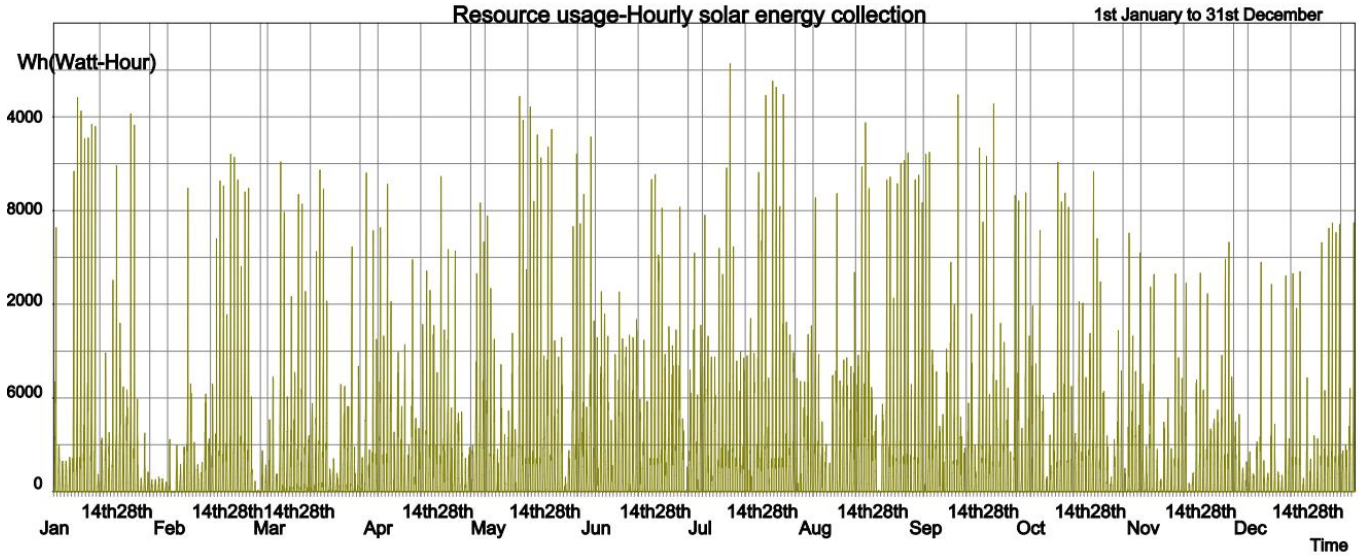


Figure 10- 6 Hourly solar energy collection from the solar panels from 1st January to 31st December

Through the simulation, the electricity generation every day and every month can be obtained,
Total area of solar panels to generate electricity is 795.984m^2, EG- Electricity Generation, Wh (Watt-hour)

Table 10- 1

Month	Jan Wh	Feb Wh	Mar Wh	Apr Wh	May Wh	Jun Wh	Jul Wh	Aug Wh	Sep Wh	Oct Wh	Nov Wh	Dec Wh
EG	2E+06	2E+06	3E+06	3E+06	5E+06	4E+06	5E+06	4E+06	4E+06	3E+06	2E+06	2E+06
Sum of electricity generation									=	**40926628**		wh

It shows that by utilizing solar energy in the slop area, 40926628 wh of electricity can be generated, which is completely clean energy.

10.4 Conclusion

This chapter has already statistically justified the three mitigating measures to increase sustainability to benefit the environment.
Using a flat slab can increase the use of light due to the sound light permeability of a flat slab and can reduce reliance on artificial light by 8%. The energy consumption of the HVAC system can be largely reduced by using a double-layer glass curtain. Finally, the solar panels on the slope's surface can alleviate the excessive light and generate lots of cleaning energy from sunlight.
Therefore, the energy use efficiency in this design is extraordinary and can not only alleviate energy pollution but also provide clean energy for the building.

10.5 A letter to client

Dear Client,

We have received your request to design a secondary exit path in case of fire from the car-parking level to level 17 for safety concerns.
As we agreed, the detailed design required in the contract has already been finished.
However, we are still very happy to outline how the structural design may be changed and how much additional construction and redesign costs will be according to your demand. And we hope it will be helpful for your decision.
As the additional staircase will be designed from the basement to level 17, if we want to minimize the reduced area for residential use, the ideal location for the other staircase is located on the right-hand side of the communal lift. The stairs for fire have to be encased in walls. Therefore, the staircase walls can share the core wall on the right-hand side and part of the lift's wall on the left-hand side. However, the core wall at RHS of the staircase needs to be moved 500mm, because the remaining width in the original core is not enough. Residents can safely egress to the second egress through the corridor, as shown in the Figure 10-7.

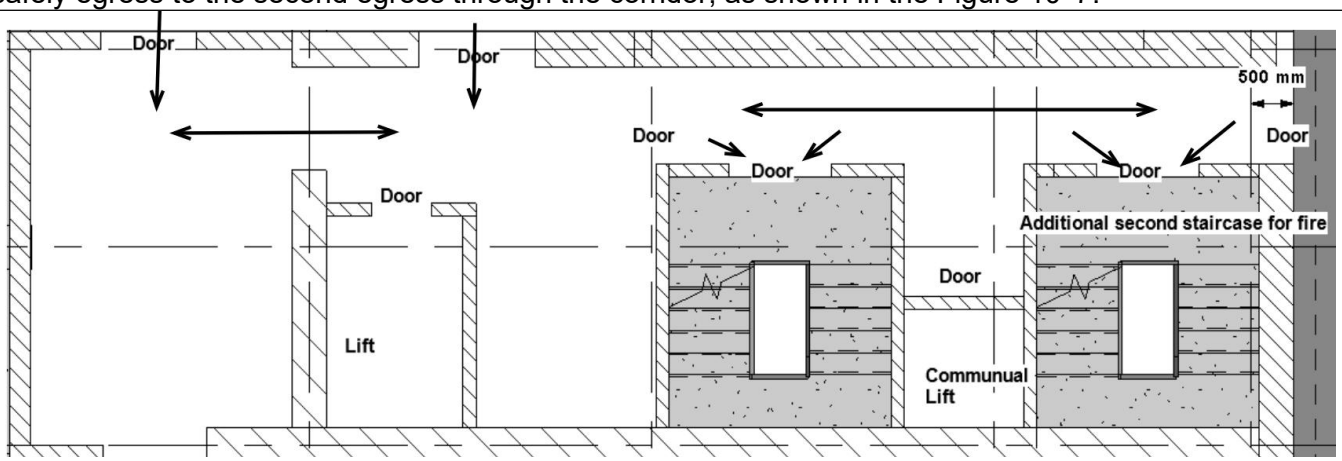


Figure 10-7 The design for secondary staircase for fire

The additional construction fee will be approximately 2500 pounds with a 2.25m^2 sacrifice of the residential area on every floor.
If you are happy with this scheme, feel free to contact us, and we can make another contract for the detailed design and structural validation for the second staircase.

Sincere,
Yibin
Structural engineer
Imperial design company limited

11 Reference

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