Structural Analysis: Gas Filling Room Building

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Abstract

This document contains the structural analysis report of a design of Gas Filling Room Building in Smelter Ferronickel Kolaka. This document is intended for a portfolio to demonstrate the competency and experience of the author. We do not intend to share sensitive information in the project, and hence, we do not disclose as much sensitive information as possible.

Contents

1 GENERAL

1.1 Introduction

This document covers the calculation and design of Gas Filling Room for Oxygen Station in Smelter Ferronickel Kolaka. The basis for the design is a data input from our partner who conduct the design for the functionality purpose of the building. And the guidelines for the design are to be mentioned in the following section. The main building components of Gas Filling Room are the main structure of the building and the blast-resisting walls.

1.2 Codes and Standards

The following codes and standards are legitimate for the use in this calculation and design:

A. **Codes**

SNI 1726:2019	Tata Cara Perencanaan Ketahanan Gempa untuk Struktur Bangunan Gedung dan Nongedung
SNI 1727:2020	Beban Desain Minimum dan Kriteria Terkait untuk Bangunan Gedung dan Struktur Lain
SNI 2847:2019	Persyaratan Beton Structural untuk Bangunan Gedung dan Penjelasan
SNI 1729:2020	Spesifikasi untuk Bangunan Gedung Baja Struktural
SNI 2052:2017	Baja Tulangan Beton
SNI 8460:2017	Persyaratan Perancangan Geoteknik
JIS G3101 SS400	Spesification of Steel Material
ASCE 7-16	Minimum Design Loads and Associated Criteria for Buildings and Other Structures
ACI 381-19	Building Code Requirements for Structural Concrete
ASTM-A615	American Society for Testing Material: Specification for Carbon Steel
AISC 341-16	Seismic Provisions for Structural Steel Buildings

1.3 Units of Measurements

The units of measurements used in this document are the International System of Units, known as SI (*Système International*).

1.4 Material Specifications

Table 1: Material Specifications

Material	Property	Notation	Measure	Unit
Concrete Structure	Compressive Strength	f_c'	29.00	MPa
Leveling Concrete	Compressive Strength	f_c'	12.00	MPa
Reinforcing Deformed Bar	Yield Strength	f_y	420	MPa
Reinforcing Deformed Bar	Tensile Strength	f_u	525	MPa

1.5 Unit Weights

Table 2: Unit weights

Material	Measure	Unit
Reinforced concrete	23.50	kN/m^3
Leveling concrete	21.60	kN/m^3
Steel	76.99	kN/m^3
Water	9.81	kN/m^3

2 METHODOLOGY

In this design, there will be 2 major components of the building which will be analyzed separately. These components are the main structure and the blast-resisting walls. The sequences in conducting the analysis for each structure are presented as follows.

Algorithm 1 Analysis of the Main Building

```
BEGIN
Model Preliminary Configuration
Determine Loads:
    Dead Load, Superimposed Dead Load, Roof Live Load, Wind Loads, Seismic Load
Input Loads
Run Analysis
Cheking Modal Analysis:
    Modal Mass Participation Ratio
        if SUFFICIENT:
            PROCEED
        else:
            Increase Number of Mode Shapes
Structural Capacity:
    if SUFFICIENT:
        PROCEED
    else:
        Remodel Configuration and Dimensions
END
```

Algorithm 2 Analysis of Blast Walls

```
BEGIN
Model Preliminary Configuration
Determine Loads:
    Dead Load, Live Load, Superimposed Dead Load, Seismic Load
Input Loads
Run Analysis
Cheking Modal Analysis:
    Modal Participating Mass Ratio
        if SUFFICIENT:
            PROCEED
        else:
            Increase Number of Mode Shapes
Structural Capacity:
    if SUFFICIENT:
        PROCEED
    else:
        Remodel Configuration and Dimensions
END
```

2.1 Design Criterion

The following criterion shall be met in the design:

1. Loads to be considered in the design include dead loads, superimposed dead loads, roof live loads, wind loads and seismic loads. In addition, a notional load in the form of blast load shall be considered for the design of blast-resisting walls.

2. Analysis method of Allowable Strength Design (ASD) is used for the design of foundations.

3. Analysis method of Load Resistant Factored Design (LRFD) is used for the design of members of concrete and steel structures in accordance with SNI 2847:2019 and SNI 1729:2020 (or alternatively ACI 381-19 and AISC 341-16).

4. The upper structure is modelled in SAP2000 as a 3D frame structure as a Special Moment Frame (SMF) structure.
5. SAP2000 incorporates 2 types of conventions regarding coordinate system, the X-Y-Z system and the 1-2-3 system. For the global coordinate system, the X-Y-Z is used. While 1-2-3 is used for the local coordinate system. For the global coordinate system, the Z-direction refers to the vertical direction while X-Y plane refers to the horizontal plane. The mathematical concept underlying this convention is the concept of 3 dimensional Euclidean vector space $\mathbb{R}^3$.

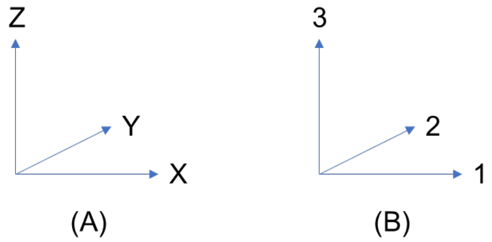


Figure 1: Equivalence of SAP2000 Coordinate Systems

2.2 Overview of the Building

The general plan of Gas Filling Room is configured in accordance an input data from our partner, and is presented in figure 2. While the cross sectional view from the same document is presented in figure 3. The structural models for the main building, the blast-resisting walls and the foundations will developed separately.

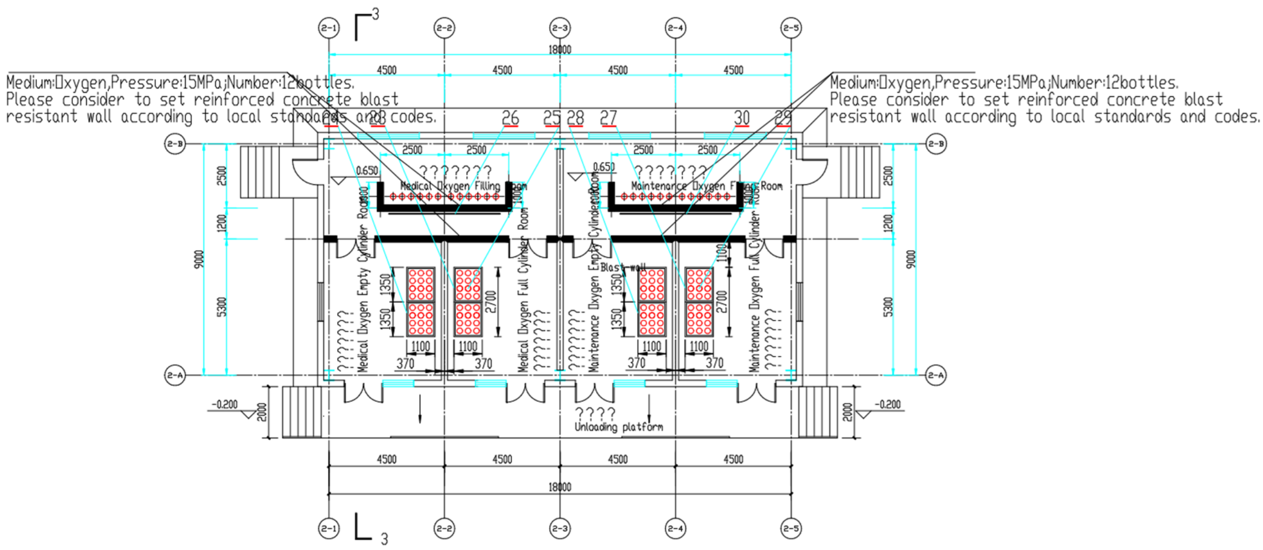


Figure 2: Plan view of gas filling room from the input data

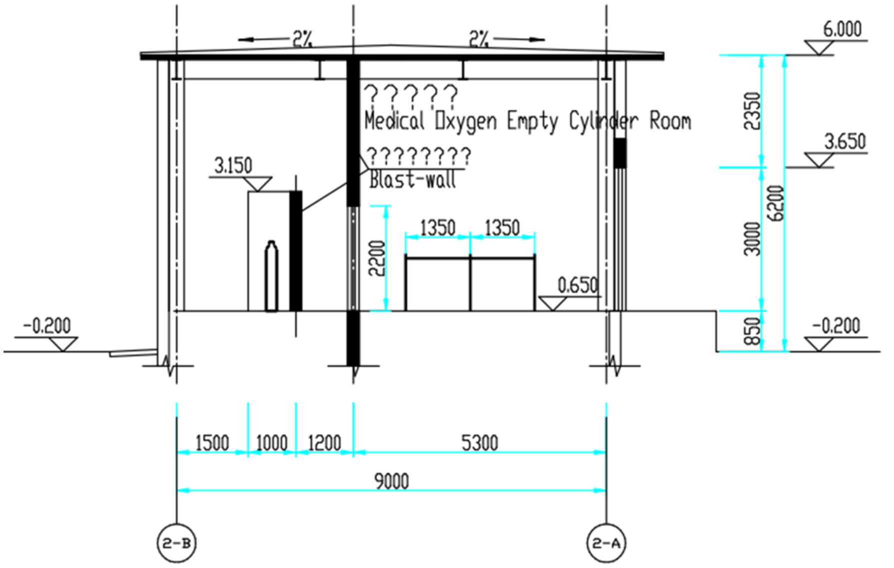


Figure 3: Cross sectional view of gas filling room from the input data

2.3 Fundamental Loads

The fundamental loads which will be incorporated in the analysis is thoroughly described as follows:

1. Dead Load
   Dead loads include the self weight of the structure, permanent loads on the structure and superimposed dead loads which consist of non-structural components such as partitioning walls, windows, doors, flooring surfaces, ceiling, etc. The magnitude of dead load will be denoted by **D**.

2. Live Load
   Gas Filling Room is only one storey building, in which the floor is on the ground. There are no suspended slabs. Thus, no live loads apply to the building.

3. Roof Live Load
   As required in SNI 1727:2020 that a minimum amount of roof live loads shall be applied to the canopy, then some amount of roof live loads will be considered in this design which will be specified in the later section. The magnitude of roof live load will be denoted by **Lr**.

4. Wind Load
   The design for wind loads will follow the guideline in SNI 1727:2020 section 26 and section 27. Several cases for the wind load will be considered in accordance with SNI 1727:2020 figure 27.3-8. Based on these cases, the following permutations of wind load patterns will be generated:

W-X1 (+) : Wind load in X-direction of case 1 in the same direction of the axis.
W-X1 (-) : Wind load in X-direction of case 1 in the same opposite of the axis.
W-X2.1 (+) : Wind load in X-direction of case 2 with an eccentricity vector of (0.15, 0.15) in the same direction of the axis.
W-X2.1 (-) : Wind load in X-direction of case 2 with an eccentricity vector of (0.15, 0.15) in the opposite direction of the axis.
W-X2.2 (+) : Wind load in X-direction of case 2 with an eccentricity vector of (-0.15, -0.15) in the same direction of the axis.
W-X2.2 (-) : Wind load in X-direction of case 2 with an eccentricity vector of (-0.15, -0.15) in the opposite direction of the axis.
W-X3 (+) : Wind load in X-direction of case 3 in the same direction of the axis.
W-X3 (-) : Wind load in X-direction of case 3 in the opposite direction of the axis.
W-X4.1 (+) : Wind load in X-direction of case 4 with an eccentricity vector of (0.15, 0.15) in the same direction of the axis.
W-X4.1 (-) : Wind load in X-direction of case 4 with an eccentricity vector of (0.15, 0.15) in the opposite direction of the axis.
W-X4.2 (+) : Wind load in X-direction of case 4 with an eccentricity vector of (-0.15, -0.15) in the same direction of the axis.
W-X4.2 (-) : Wind load in X-direction of case 4 with an eccentricity vector of (-0.15, -0.15) in the opposite direction of the axis.

W-Y1 (+) : Wind load in Y-direction of case 1 in the same direction of the axis.
W-Y1 (-) : Wind load in Y-direction of case 1 in the same opposite of the axis.
W-Y2.1 (+) : Wind load in Y-direction of case 2 with an eccentricity vector of (0.15, 0.15) in the same direction of the axis.
W-Y2.1 (-) : Wind load in Y-direction of case 2 with an eccentricity vector of (0.15, 0.15) in the opposite direction of the axis.
W-Y2.2 (+) : Wind load in Y-direction of case 2 with an eccentricity vector of (-0.15, -0.15) in the same direction of the axis.
W-Y2.2 (-) : Wind load in Y-direction of case 2 with an eccentricity vector of (-0.15, -0.15) in the opposite direction of the axis.

W-Y3 (+) : Wind load in Y-direction of case 3 in the same direction of the axis.
W-Y3 (-) : Wind load in Y-direction of case 3 in the opposite direction of the axis.
W-Y4.1 (+) : Wind load in Y-direction of case 4 with an eccentricity vector of (0.15, 0.15) in the same direction of the axis.
W-Y4.1 (-) : Wind load in Y-direction of case 4 with an eccentricity vector of (0.15, 0.15) in the opposite direction of the axis.
W-Y4.2 (+) : Wind load in Y-direction of case 4 with an eccentricity vector of (-0.15, -0.15) in the same direction of the axis.
W-Y4.2 (-) : Wind load in Y-direction of case 4 with an eccentricity vector of (-0.15, -0.15) in the opposite direction of the axis.

5. Seismic Load
The loading condition for earthquake uses the response spectrum analysis. The method for generating the spectrum function will be described comprehensively on a later section. The seismic consideration will follow the guidelines in SNI 1726:2019. The directions of interest for the seismic loads are X and Y direction of the global coordinate system. And the seismic loads are denoted by **EQ-X** and **EQ-Y**. However, in the seismic load combinations, the seismic load cases to be included are **Ev** (vertical earthquake) and **Eh** (horizontal earthquake). SNI 1726:2019 section 7.4.2.1 asserts that

$$\textbf{Eh} = \rho Q_E \,,$$

where ρ is the redundancy factor and Q_E is the horizontal seismic load, that is, $Q_E =$ **EQ-X** or $Q_E =$ **EQ-Y**. And SNI 1726:2019 section 7.4.2.2 asserts that

$$\textbf{Ev} = 0.2 S_{DS} \textbf{D} \,,$$

where S_{DS} is the short period spectral acceleration parameter. On the other hand, the seismic load case in terms of overstrength factor **Emh** is given by

$$\textbf{Emh} = \Omega_0 Q_E \,,$$

where Ω_0 is the overstrength factor, determined using SNI 1726:2019 table 12.

6. Modal
   Modal analysis is required to determine the fundamental period of structures as well as their natural behaviour. SAP2000 provides a dedicated computational environment to process modal analysis.

2.4 Load Combinations

The load combinations in use are set in accordance with SNI 1727:2020 which consist of Load Resistant Factor Design (LRFD) and Allowable Strength Design (ASD). For Main Structure, the load combinations are arranged in accordance with SNI 1727:2020 section 2.3.1 for base combinations and section 2.3.6 for the seismic effect including the overstrength factor. The combinations are presented as follows:

1. LRFD

Laod Combination	Remark
1.4**D**	
1.2**D** + 1.6**L** + 0.5(**Lr** or **R**)	**L** does not present in this design, only **D**, **Lr** and **R** do.
1.2**D** + 1.6**Lr** + 0.5**W**	**W** will incorporate all wind load permutations as presented in point 4 section 2.3 earlier. (Not applicable to Blast Resisting Walls)
1.2**D** + 1.0**W** + 0.5**Lr**	**W** will incorporate all wind load permutations as presented in point 4 section 2.3 earlier. (Not applicable to Blast Resisting Walls)
0.9**D** + 1.0**W**	**W** will incorporate all wind load permutations as presented in point 4 section 2.3 earlier. (Not applicable to Blast Resisting Walls)
1.2**D** + **Ev** + **Eh**	Seismic load without overstrength. **Eh** will consider both **EQ-X** and **EQ-Y**.
0.9**D** - **Ev** + **Emh**	Seismic load with overstrength factor. **Emh** will consider both **EQ-X** and **EQ-Y**.

2. ASD

Load Combination	Remark
D	
D + **Lr**	
D + 0.6**W**	**W** will incorporate all wind load permutations as presented in point 4 section 2.3 earlier. (Not applicable to Blast Resisting Walls)
D + 0.75(0.6**W**) + 0.75**Lr**	**W** will incorporate all wind load permutations as presented in point 4 section 2.3 earlier. (Not applicable to Blast Resisting Walls)
0.6**D** + 0.6**W**	**W** will incorporate all wind load permutations as presented in point 4 section 2.3 earlier. (Not applicable to Blast Resisting Walls)
1.0**D** + 0.7**Ev** + 0.7**Eh**	Seismic load without overstrength. **Eh** will consider both **EQ-X** and **EQ-Y**.
1.0**D** + 0.525**Ev** + 0.525**Eh** + 0.75**L**	Seismic load without overstrength. **Eh** will consider both **EQ-X** and **EQ-Y**.
0.6**D** - 0.7**Ev** + 0.7**Eh**	Seismic load without overstrength. **Eh** will consider both **EQ-X** and **EQ-Y**.
1.0**D** + 0.7**Ev** + 0.7**Emh**	Seismic load with overstrength. **Emh** will consider both **EQ-X** and **EQ-Y**.
1.0**D** + 0.525**Ev** + 0.525**Emh** + 0.75**L**	Seismic load overstrength. **Emh** will consider both **EQ-X** and **EQ-Y**.
0.6**D** - 0.7**Ev** + 0.7**Emh**	Seismic load with overstrength. **Emh** will consider both **EQ-X** and **EQ-Y**.

3 MODELING AND DESIGN

This section will cover the modeling of the structures and the method of determining the considered loads as well as their input to the model. The discussion regarding the modeling and design of Gas Filling Room will be divided into 3 parts in accordance with the number of major components of the building namely for Main Structure, for Blast-Resisting Walls and for the Foundations as we have mentioned in the earlier section.

3.1 Main Structure

3.1.1 Model Overview of Main Structure

The model for Main Structure is composed of 3D space frame with fixed supports modelled in SAP2000. The overview of the model is presented in figure 4.

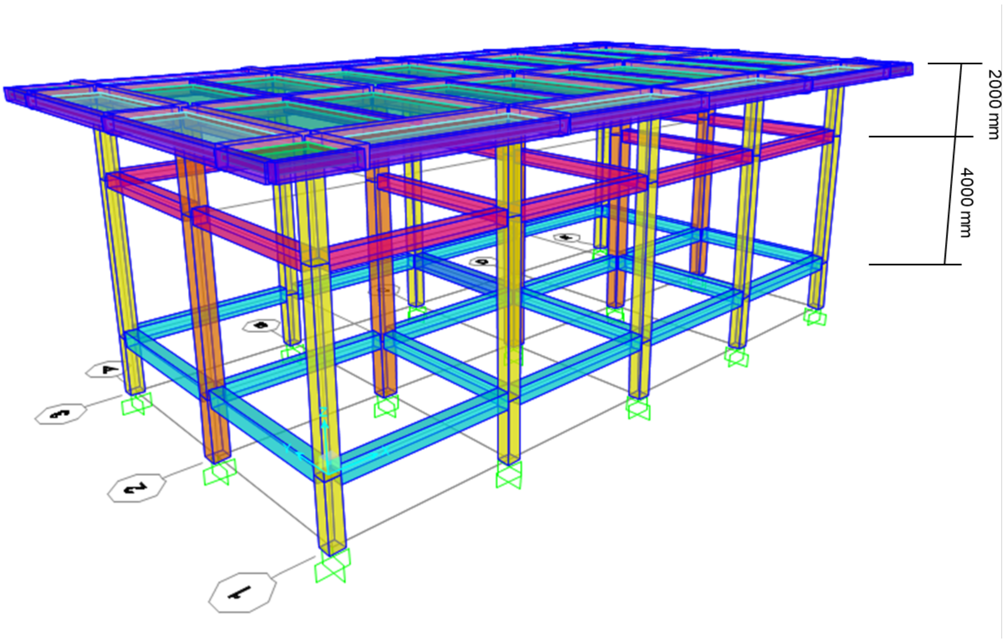


Figure 4: Overview of Main Structure model in SAP2000

And the plan views are presented in figures 5 for Z = 0 mm, Z = 4000 mm and Z = 6000 mm.

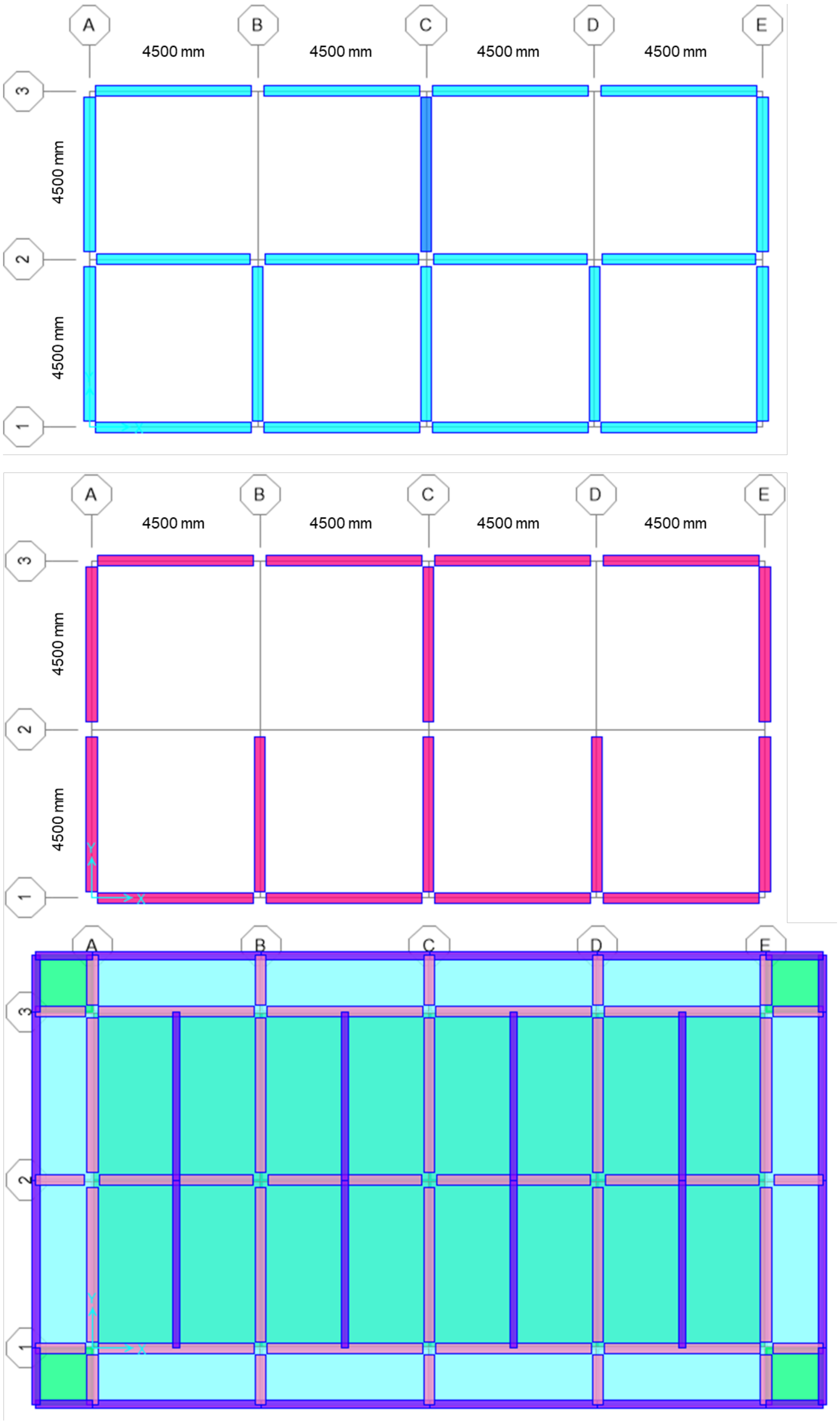


Figure 5: Plan views at Z = 0.000 mm, Z = 4000 mm and Z = 6000 mm

3.1.2 Loads Input and Setting for Main Structure

Dead Load

By default, SAP2000 automatically computes the dead load of structural elements as long as the unit weight of structural materials present. Even though this default setting can be modified, we will not change the setting.

Superimposed Dead Load

The detail of the superimposed dead load is presented in table 3. The input of the superimposed dead load is presented in figures 6.

Table 3: Superimposed Dead Load

No	Material	Load Value	Unit	Structural Element
1	Hebel Unit weight = 800.00 kg/m^3 Thickness = 0.20 m Height = 4.00 m Weight/length = 640.00 kg/m	640.00	kg/m	TIE BEAM
2	Hebel Unit weight = 800.00 kg/m^3 Thickness = 0.20 m Height = 6.00 m Weight/length = 960.00 kg/m	960.00	kg/m	TIE BEAM
3	Wall Finish (Assumed 4 cm of cement plaster) Unit weight = 2000.00 kg/m^3 Thickness = 0.04 m Height = 4.00 Weight/length = 320.00 kg/m	320.00	kg/m	TIE BEAM
4	Ceiling Material (inlcuding hanger) Unit weight = 17.00 kg/m^2	17.00	kg/m2	ROOF SLAB
5	Roofing Finish Unit weight = 0.12 kN/m^2	0.12	kN/m^2	ROOF SLAB

Roof Live Load

The roof live load applied to the concrete slab roof is given in accordance with SNI 1727:2020 table 4.3-1 by 0.96 kN/m^2. Another consideration is the rain load. The detail loading calculation is presented in table 4. And the input of the roof live load as well as the rain load is illustrated in figures 7 and 8.

Table 4: Roof live load and rain load

No	Load Item	Notation	Value	Unit	Structural Element
1	Roof Live Load breakdown: Basic load = 0.24 kN/m^2 Tributari width = 1.00 m Load/length = 0.24 kN/m	Lr	0.24	kN/m	ROOF FRAME
2	Rain on Roof Load breakdown: Basic load = 0.20 kN/m^2 Tributari width = 1.00 m Load/length = 0.20 kN/m	R	0.20	kN/m	ROOF FRAME

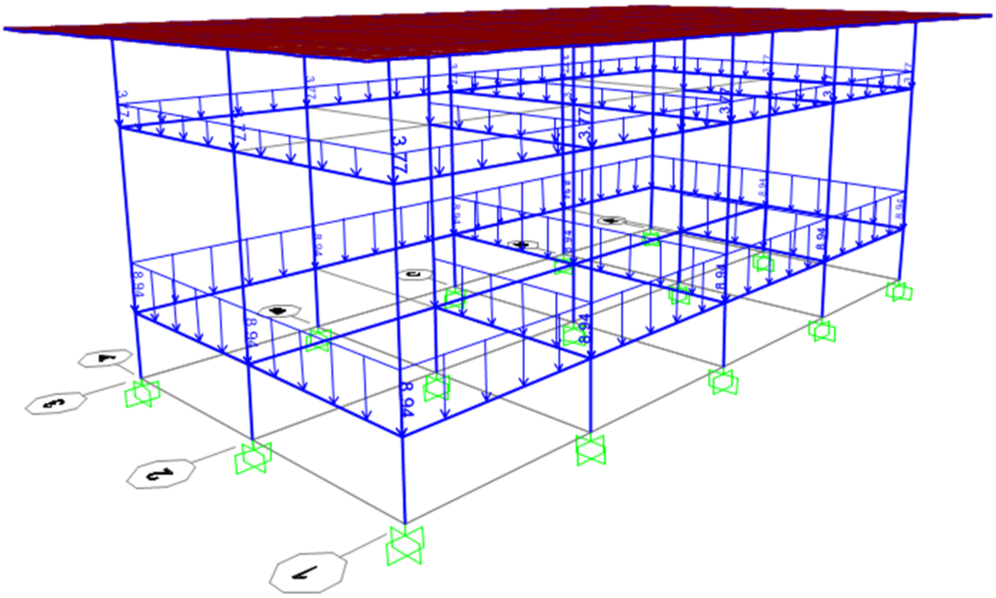


Figure 6: Application of superimposed dead load

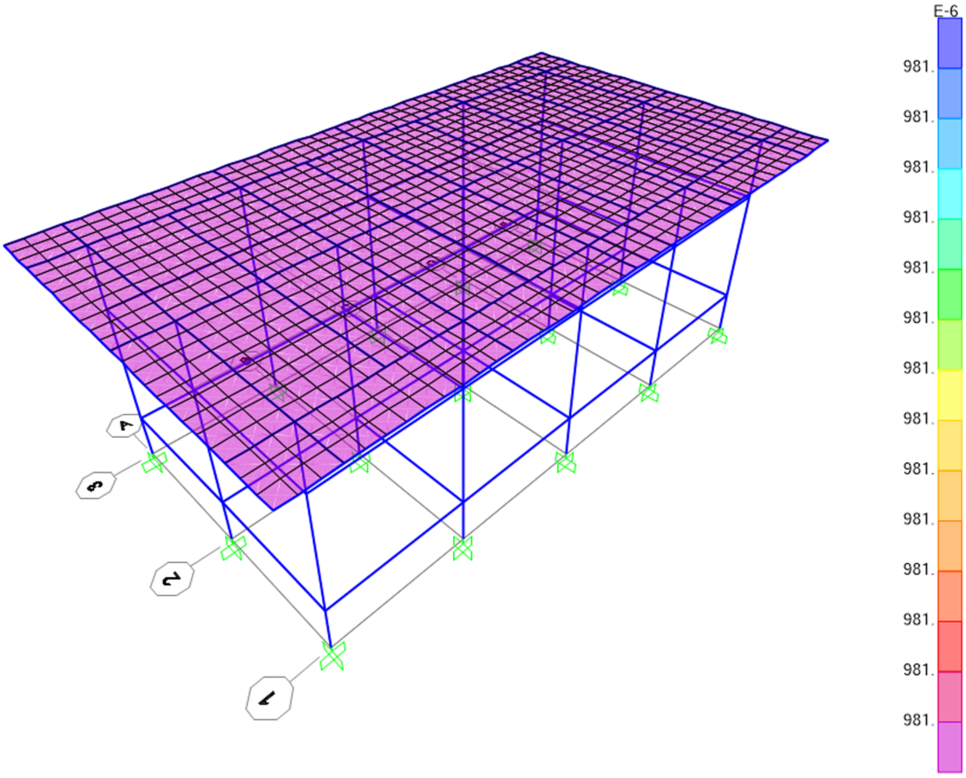


Figure 7: Roof live load (MPa)

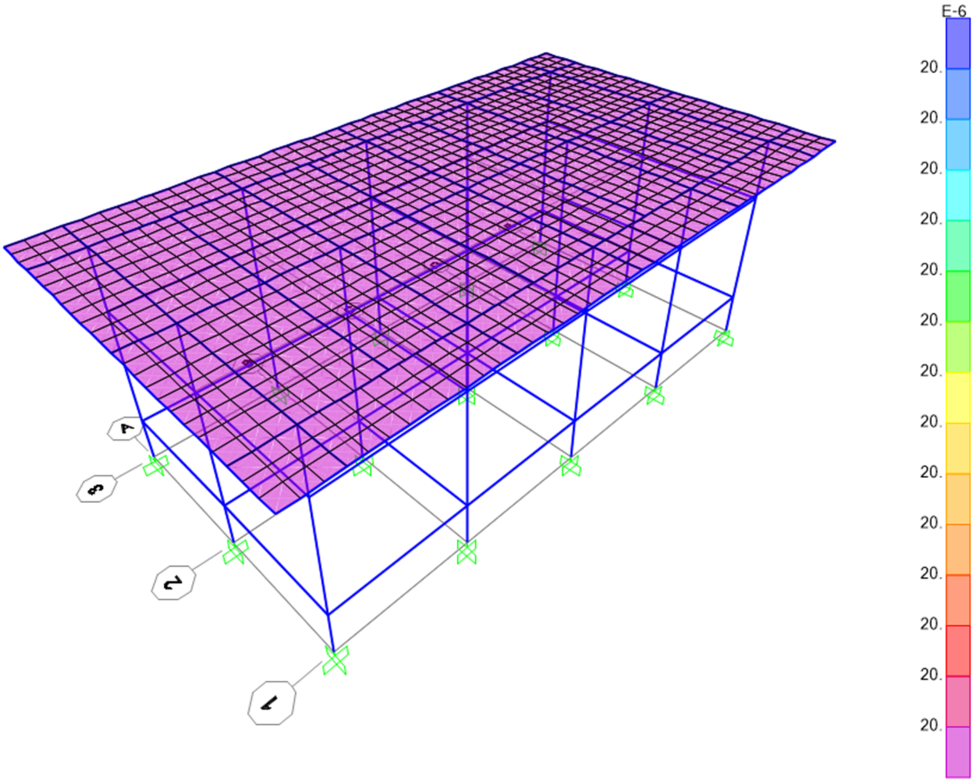


Figure 8: Rain load (MPa)

Wind Load

The wind load will be automatically computed using the dedicated algorithm within SAP2000. SAP2000 provides a computational environment for wind load according to ASCE 7-16, which is equivalent to SNI 1727:2020. Thus, we can use this computational environment. The wind load parameters are given in table 5. The parameters are to be input in the SAP2000 model. Some illustrations of the input is given in figure 9.

Table 5: Wind load parameters

No.	Description	Value	Unit	Remark
	Exposure classification	C		SNI 1727:2020 sect. 26.7.2
V	Basic wind speed	40.00	m/s	
h	Total height of building	7.50	m	
	Type of building	Enclosed		
f_n	Building natural frequency	2.2902	Hertz	Computed in SAP2000
	Building rigidity type	Rigid		SNI 1727:2020 sect. 26.2
G	Gust factor	0.85		For rigid building
C_p	Widward coefficient	0.80		
C_p	Leeward coefficient	0.50		
K_{zt}	Topographical factor	1.00		SNI 1727:2020 sect. 26.8.2
K_d	Wind directionality factor	0.85		SNI 1727:2020 tab. 26.6-1

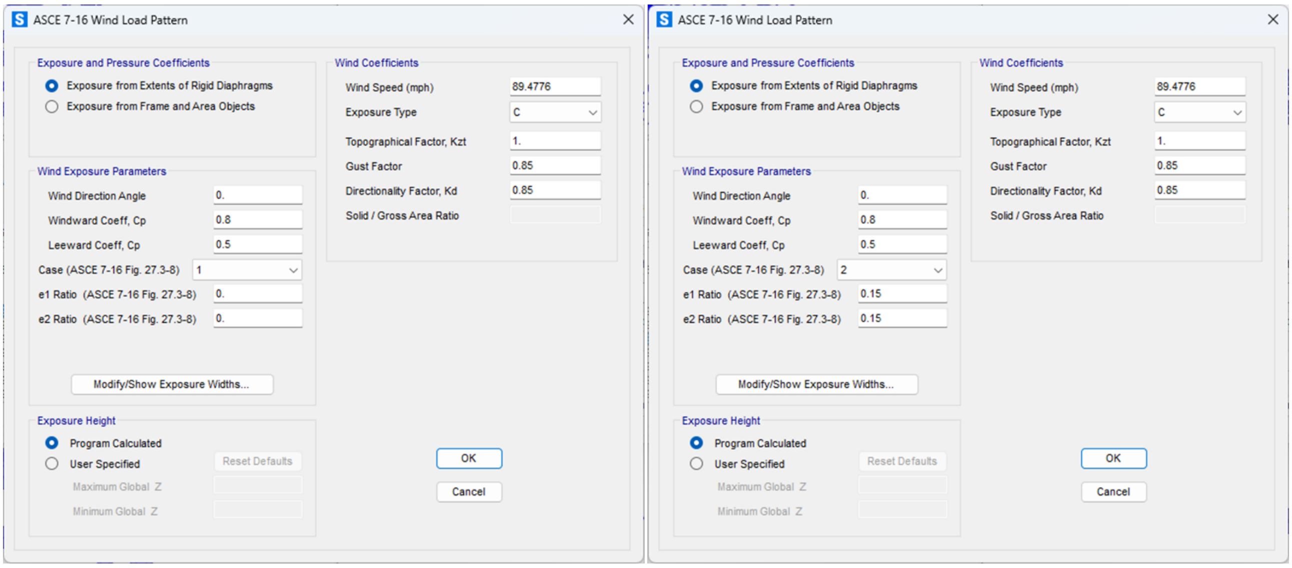


Figure 9: Two examples of wind load parameters tuning (left: for **W-X1 (+)**, right: for **W-X2.1**)

Seismic Load

For the seismic load, we use response spectrum analysis, which is regulated in SNI 1726:2019. SAP2000 provides a dedicated algorithm for computing structural responses from a given design spectra. The preliminary data for the response spectrum is obtained from the soil report as well as the Probabilistic Seismic Hazard Analysis (PSHA) of Kolaka, which is presented in table 6.

Table 6: Seismic paramters for Kolaka

Seismic Parameter	Notation	Value	Unit	Remark
Risk Category		II		SNI 1726:2019 table 3
Importance Factor	I_e	1.000		SNI 1726:2019 table 4
Site class	Class	SE		SNI 1726:2019 table 5
Spectral acceleration at t = 0.2 s	S_s	0.578	g	PSHA Kolaka
Spectral acceleration at t = 1.0 s	S_1	0.179	g	PSHA Kolaka
Site coefficient for short period of 0.2 s	F_a	1.543		PSHA Kolaka
Site coefficient for long period	F_v	4.981		PSHA Kolaka
Spectral acceleration parameter of short period with 5% damping	S_{DS}	0.595	g	SNI 1726:2019 sect. 6.3
Spectral acceleration parameter at 1 s	S_{D1}	0.594	g	SNI 1726:2019 sect. 6.3
Long period transtition	T_L	6.000	s	SNI 1726:2019 figure 20
Response modification coefficient	R	8.000		SNI 1726:2019 table 12
Overstrength factor	Ω_0	3.000		SNI 1726:2019 table 12
Deflection factor	C_d	2.500		SNI 1726:2019 table 12
Cofficient for approximated fundamental period	C_t	0.047		SNI 1726:2019 table 18
Exponent for approximated fundamental period	x	0.900		SNI 1726:2019 table 18
Building height	h	7.500	m	
Approximated fundamental period	T_a	0.286	s	
Approximated natural frequency	ω_n	3.500	Hertz	
Seismic response coefficient	C_s	0.074		SNI 1726:2019 sect. 7.8.1
Redundancy factor	ρ	1.300		SNI 1726:2019 sect. 7.3.4.2
Response scale factor for model in SAP2000	gI_e/R	1226.250		g is expressed in mm/s

The response spectrum function is computed in accordance with SNI 1726:2019 section 6.4. A formal framework as well as the compuatation of response spectrum is demonstrated as follows.

Mathematically, a response spectrum function can be modelled as a map $S_a : \mathcal{T} \to \mathbb{R}$, where $\mathcal{T} \subseteq [0, \infty)$ is the time domain of interest with the unit of seconds during earthquake and $\mathbb{R}$ is the set of all real numbers (Rudin, 1976) representing the numeric value of the spectral acceleration in the unit of g (gravitational acceleration). As presented in table 6, several parameters are already known are predetermined from either soil report, PSHA of Kolaka or SNI 1726:2019. Among those predetermined parameters, we have $S_{DS}, S_{D1} \in \mathbb{R}$ and $T_L \in \mathcal{T}$, which would be instrumental in determining the map S_a. And follows from SNI 1726:2019 section 6.4, S_a is defined by

$$\forall T \in \mathcal{T}: \; S_a(T) := \begin{cases} S_{DS}\left(0.4 + 0.6\dfrac{T}{T_0}\right) & : T < T_0 \\ S_{DS} & T_0 \leq T \leq T_s \\ \dfrac{S_{D1}}{t} & : T_s < T \leq T_L \\ \dfrac{S_{D1}T_L}{t^2} & : T > T_L \end{cases}$$

where

$$T_0 = 0.2T_s\,, \quad T_s = \frac{S_{D1}}{S_{DS}}\,.$$

The graph representation of $S_a : \mathcal{T} \to \mathbb{R}$ with $\mathcal{T} = [0, 10]$ is given by figure 10.

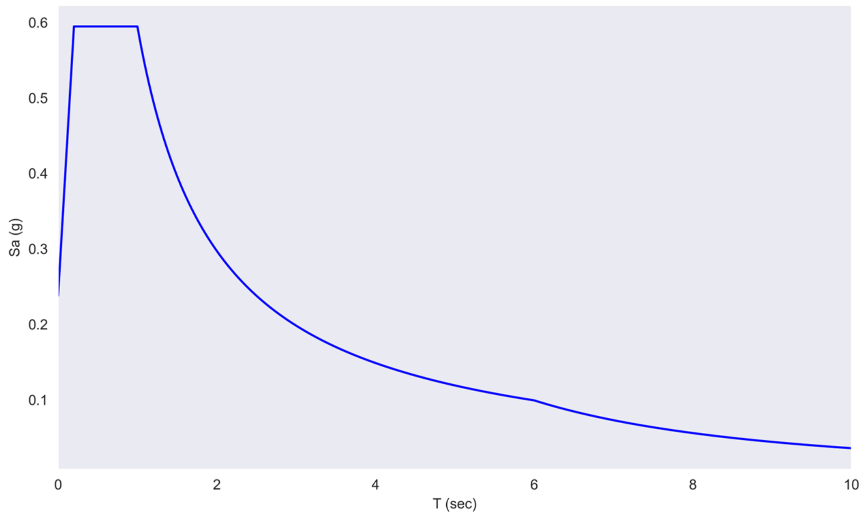


Figure 10: Response spectrum of Kolaka

The response spectrum above is then implemented in the SAP2000 model, as illustrated in figure 11. The load cases for the seismic loads is then set. We take into consideration of two main directional loads, namely along the X and Y directions. For each direction, a 30% seismic load from the other direction is considered. In addition, the response scale factor gI_e/R is also applied in the setting, since our response spectrum is in the unit of g, while our SAP2000 model uses the unit of mm for length and seconds for time. Note that $g =$ 9810 mm/s^2. The setting in the seismic load cases is given in figure 12.

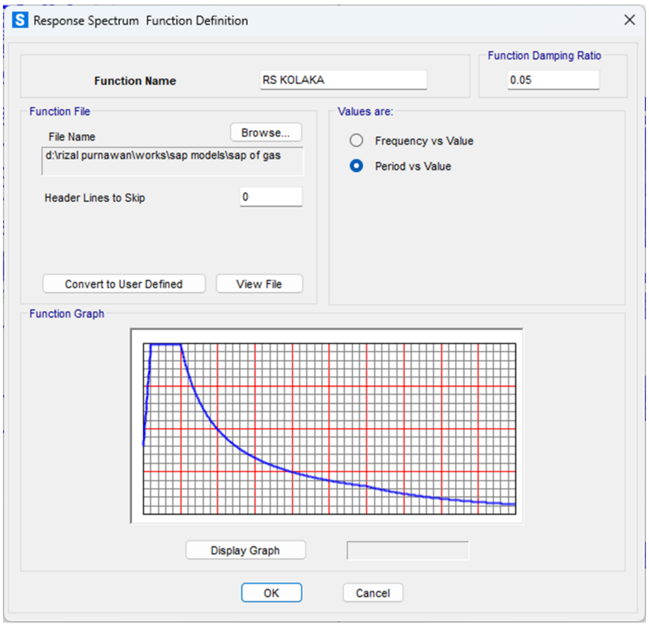


Figure 11: Response spectrum implementation in SAP2000

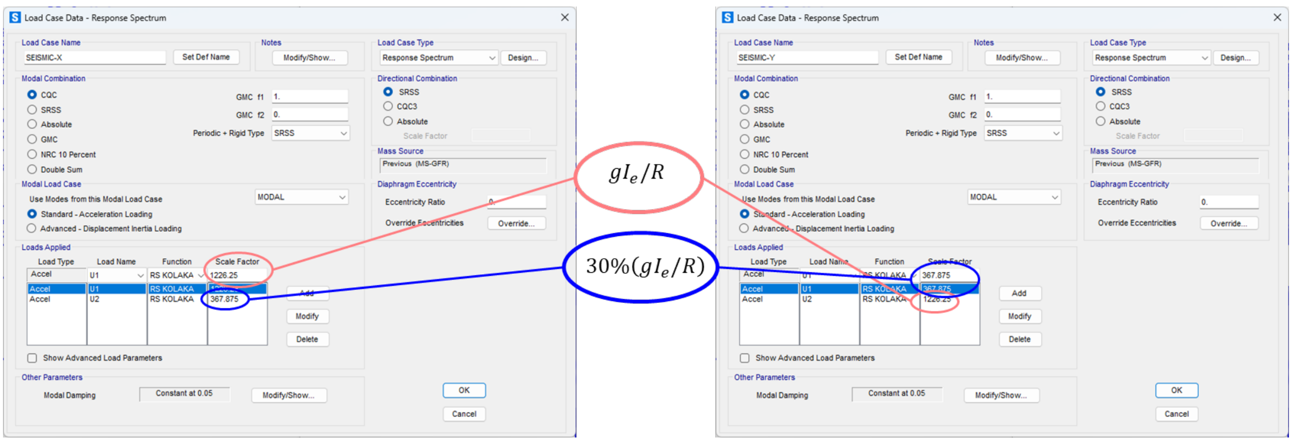


Figure 12: Load case setting for seismic loads (left: for **EQ-X**, right: for **EQ-Y**)

Modal

Modal load case shall be set in order to execute the modal analysis on the structure. A sufficient mode shapes shall be determined such that the participating mass ratio of the modal analysis has a considerable value as close as possible to 1. The number of mode shapes is actually needs to be set as many as the number of degree of freedom in the system. However, for a very complex structure such as ours, it will not be convenient to calculate the number of degree of freedom. In exchange, we guess a considerably large number of mode shapes of about 240. The setting is shown in figure 13. On the later section, we will describe the underlying theory describing modal analysis. In addition, the mass source shall also be set. For a simple explanation, according to Newton's second law, force is given by the multiplication of mass and acceleration. Mass source specifies the mass which will be included into force with the acceleration provided from a response spectrum, time history, etc. For this building, since there are no live loads (except the roof live) on the structure. Then we specify the mass source to be of dead load and superimposed dead load.

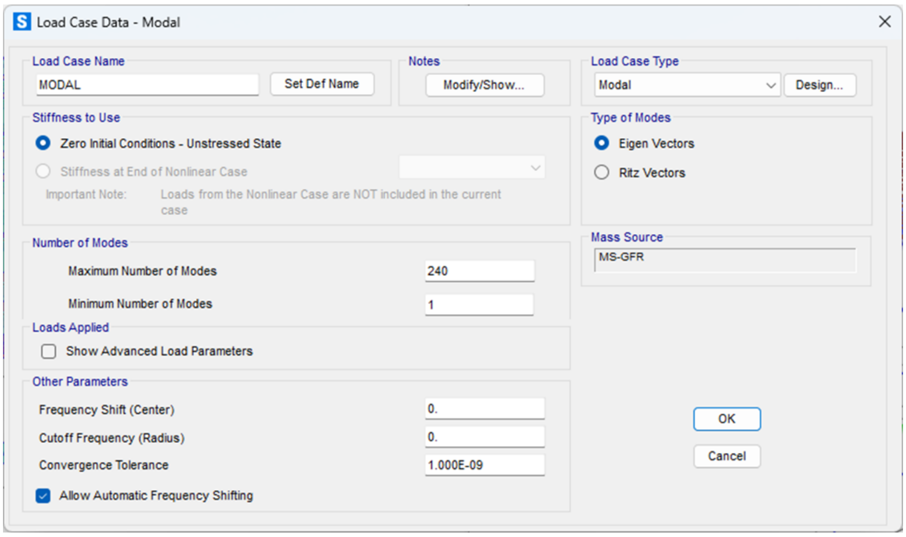


Figure 13: Modal load case

3.2 Blast Resisting Walls

The blast resisting walls in Gas Filling Room are intended for protecting humans from possible explosion of oxygen tanks during gas filling. The location of the walls is inside the building, as indicated in figures 2 and 3. We also use SAP2000 to model the walls in which the walls are modelled as shell elements. The model is separated from the model of the Main Structure. As agreed by all the stakeholders, the explosive loads are not necessarily considered in the analysis.

3.2.1 Models of Blast Resisting Walls

There are 2 types of blast-resisting walls, as indicated in the input data. Let us call the walls BRW-A and BRW-B, which is indicated in figure 14. As indicated in figure 14, there will be 2 BRW-A and 2 BRW-B. The geometrical configuration as well as the position of explosive sources, which are gas tanks, are given in figure 15. And note that the heights of the walls are as indicated in figure 3, namely 3000 mm for BRW-A and 6000 mm for BRW-B.

The developed models in SAP2000 are presented in figure 16. For a preliminary, we assume that the thickness of the wall is 250 mm.

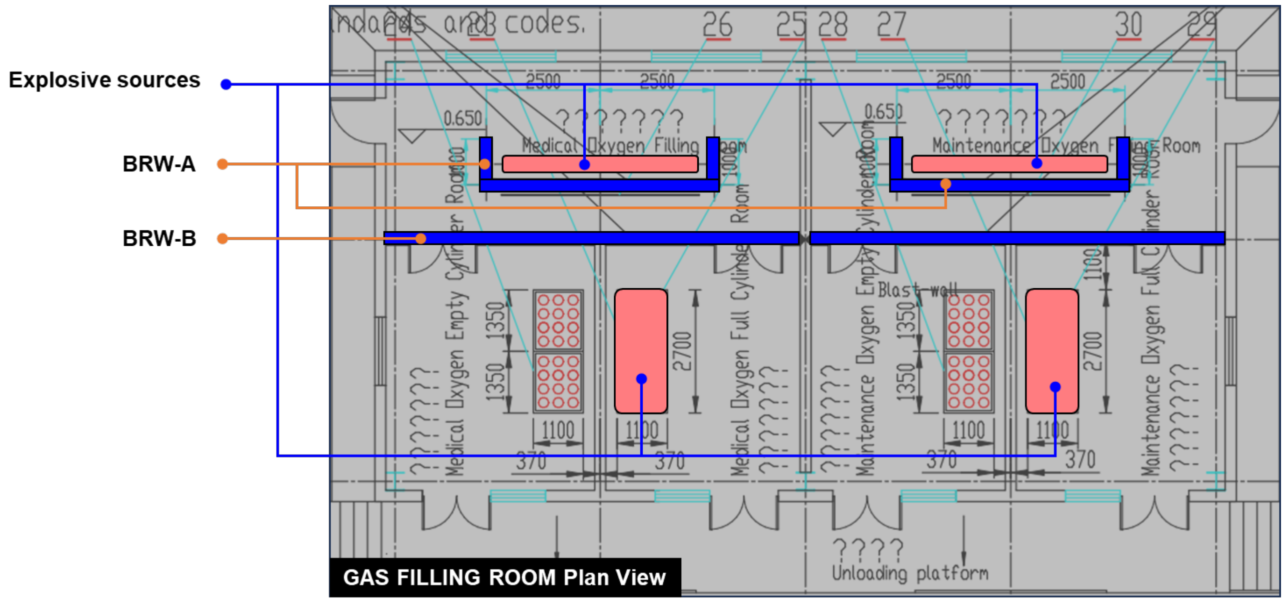


Figure 14: Configuration of blast-resisting wall according to 922A-E421-101-3

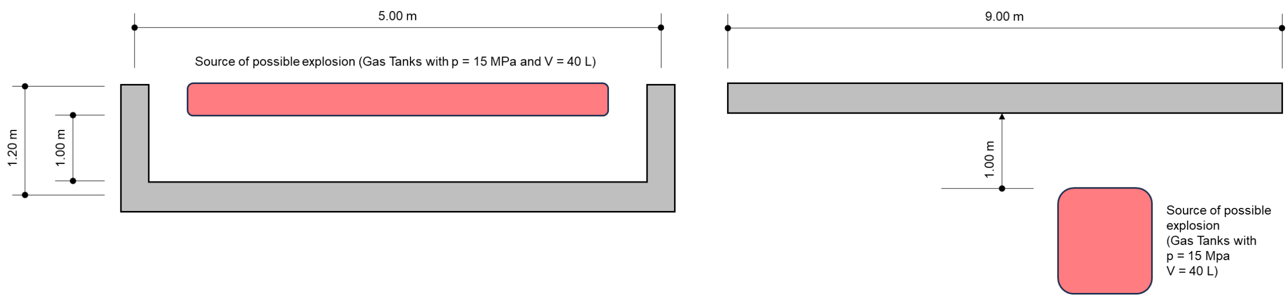


Figure 15: Detailed configuration of blast-resisting walls and the sources of explosion

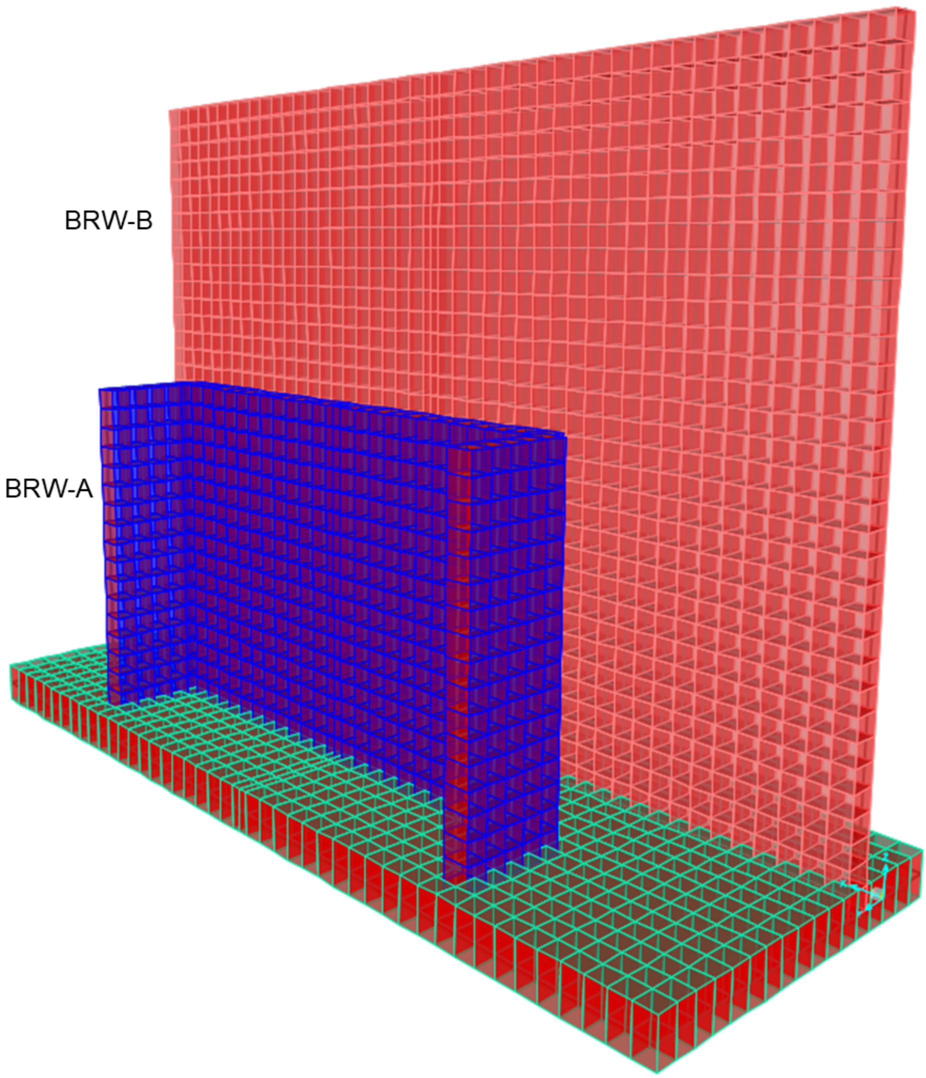


Figure 16: SAP2000 Model of BRW-A and BRW-B as well as their foundation

4 STRUCTURAL ANALYSIS FOR UPPER STRUCTURES

This section covers modal analysis, seismic analysis, steel structural analysis and concrete structural analysis for both Main Structure and the Blast Resisting Walls (BRW-A and BRW-B).

4.1 General Theory in the Analysis

4.1.1 Modal Analysis

Modal analysis is the typical common dynamic simulation for many other dynamic simulations, which is mainly used to determine the vibration characteristic of linear elastic structures such as natural frequency or equivalently fundamental period.

The origin of modal analysis can be described from the equation of motion of a classical mechanical system. Suppose we are given a system with n degree of freedoms, for some $n \in \mathbb{N}$. Then the corresponding equation of motion of elements in the system is given by the matrix differential equation

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{K}\mathbf{u} = \mathbf{f}$$

where $\mathbf{M}, \mathbf{C}, \mathbf{K} \in \mathbb{R}^{n \times n}$ are mass, damping and stiffness matrices, and $\mathbf{u}, \mathbf{f} \in \mathbb{R}^n$ is the displacement and force vectors respectively. Note that $\dot{\mathbf{u}} \in \mathbb{R}^n$ is the first time derivative of $\mathbf{u}$ which is the velocity vector, and $\ddot{\mathbf{u}}$ is the second time derivative of $\mathbf{u}$ which is the acceleration vector.

The point of interest in the modal analysis is at the state that the force is zero and the system is undamped, that is, $\mathbf{f} = \mathbf{0}$ and $\mathbf{C}$ is the zero matrix. And we obtain

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{K}\mathbf{u} = \mathbf{0}\,,$$

which is a second order linear homogeneous matrix differential equation. Now suppose that the system undergoing a harmonic motion. Then the solution of $\mathbf{u}$ takes the form

$$\mathbf{u} = \mathbf{\Phi} \sin(\omega t + \theta)\,,$$

where $\mathbf{\Phi}$ is the amplitude of the harmonic motion, ω is the angular frequency and θ is the phase angle.. And then the acceleration becomes

$$\ddot{\mathbf{u}} = -\omega^2 \mathbf{\Phi} \sin(\omega t + \theta)\,.$$

And by substituting the displacement and acceleration equations above into the equation of motion with undamped system and zero force, we obtain

$$\begin{aligned} \mathbf{K}\mathbf{\Phi}\sin(\omega t + \theta) - \omega^2 \mathbf{M}\mathbf{\Phi}\sin(\omega t + \theta) &= \mathbf{0} \\ \therefore (\mathbf{K} - \omega^2\mathbf{M})\mathbf{\Phi} &= \mathbf{0}\,, \end{aligned}$$

which is an eigenvalue problem (Roman, 2005), with ω^2 being the eigevalue and $\mathbf{\Phi}$ being the eigenvector. A trivial solution to the problem is that $\mathbf{\Phi}$ is a zero vector. However, this will not give us more insights and may not be convenient. Other solutions for nonzero $\mathbf{\Phi}$ can be given if

$$\det(\mathbf{K} - \omega^2\mathbf{M}) = 0\,,$$

where $\det$ denotes the determinant of a square matrix. The number of solutions to the eigenvalue problem is equal to the number of degree of freedoms in the system. Each solution is commonly known as "mode". The eigenvector $\mathbf{\Phi}$ is known as the mode shape. While the square root of the eigenvalue, ω, is the angular natural frequency. The natural frequency in the unit of Hertz (or equally cicle/second) is given by

$$f := \frac{\omega}{2\pi}\,.$$

And the fundamental period is given by

$$T = \frac{1}{f} = \frac{2\pi}{\omega}\,.$$

For a very complex structure, we may have a very large number of degree of freedoms. And computing all the solutions is not convenient. Instead, we need to consider only modes which contribute to the significant deformations of the structure. It can be identified by the modal participating mass ratio. A relatively large number of modal participating mass ratio indicates a large excitation on the structure. The sum of modal participating mass ratio gives an indication that we have included significant modes. As the sum of modal participating mass ratio close to 1, then it means that the significant modes has already been included.

4.1.2 Seismic Analysis

Some verification are needed when using the response spectrum method for computing seismic loads. The verification include base shear control and storey drift limit.

Follows from SNI 1726:2019 section 7.9.1.4.1, the base shear from the model with response spectrum is less than the base shear computed with the method of static equivalent, then the base shear from the model with response spectrum shall be scaled such that it exceeds the value of the static equivalent. Otherwise, the model is already acceptable. Supposing that we perform the analysis iteratively. Suppose V_t is the base shear from our model with response spectrum at the current iteration. And suppose V is the base shear computed with static equivalent method, which is given in accordance with SNI 1726:2019 section 7.8.1 by

$$V = C_s W$$

where C_s is the seismic response coefficient already given in table 6, and W is the total weight of the structure. Then

$$V_t < V \implies V_{t+1} := \frac{V}{V_t} V_t \,.$$

shall be satisfied. Or alternatively, we can upscale the response spectrum with V/V_t.

For the storey drift limit, the drift between each adjacent storeys shall be less than the required limit stated in SNI 1726:2019 section 7.12.1. Suppose Δ_x is the story drift at a level x, then

$$\Delta_x < 0.025 h_x$$

according to 1726:2019 section 7.12.1, where h_x is the building height up to level x. And the lateral displacement at level x shall be given by

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \,,$$

where C_d is the amplification factor which is given in the earlier section, δ_{xe} is the elastic displacement due to seismic loads at level x computed in the model and I_e is the importance factor which is also given in the earlier section. And thus,

$$\Delta_x = \delta_x - \delta_{x-1} \,,$$

where δ_{x-1} is the lateral displacement at one level below level x.

4.1.3 Longitudinal Rebars of Concrete Columns

In the design of longitudinal reinforcement for concrete columns, one needs to consider the axial and moments (in both axes) capacity of the columns. A common method to analyse the structural capacity of columns with longitudinal reinforcements is the so-called PMM interaction. The PMM graph indicates the boundary of the column capacity in terms of axial force and moments. A typical PMM graph is shown in figure 17.

We can confirm that the design is sufficient if the pair $(M_{u,1}, M_{u,2}, P_u)$ of ultimate moment-1, ultimate moment-2 and ultimate axial force is inside the boundary of PMM. SAP2000 can generate the PMM graph of a column configuration. One can even set the number of numeric values of the boundary.

4.2 Analysis of Main Structure

4.2.1 Modal Analysis of Main Structure

For a preliminary, we use 240 mode shapes as indicated in the earlier section. And the modal analysis of the SAP2000 model shows that the number of mode shapes is sufficient since it gives a sufficient sum of modal participating mass ratio. The result of modal participating mass ratio of several modes is presented in table 7. We can see in the table that the sum of modal participating mass ration for displacements X, Y, Z (SumU, SumUY, SumUZ) as well as rotations X, Y, Z (SumRX, SumRY, SumRZ) exceed 0.98, which is acceptable. Thus, we conclude that the significant modes already included in the modal analysis.

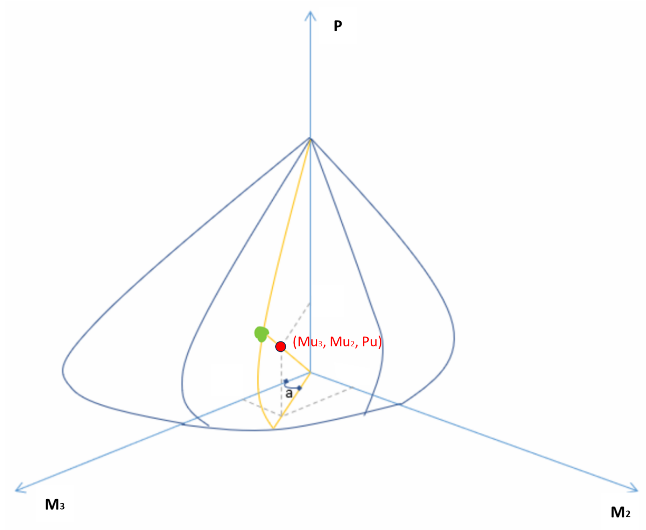


Figure 17: Typical PMM graph with a pair of ultimate forces inside the boundary

Table 7: Modal anaysis result of Main Structure (for mode no. 380 to 400)

Mode	Period sec	Frequency Hertz	SumUX	SumUY	SumUZ	SumRX	SumRY	SumRZ
1	0.4223	2.3678	0.6800	0.0000	0.0000	0.0000	0.0541	0.0020
2	0.4221	2.3691	0.6800	0.7000	0.0000	0.1200	0.0541	0.0020
3	0.3956	2.5278	0.6900	0.7000	0.0000	0.1200	0.0543	0.7000
4	0.0849	11.7809	0.6900	0.7000	0.0000	0.1200	0.0544	0.7000
5	0.0839	11.9181	0.6900	0.7100	0.0000	0.1400	0.0544	0.7000
⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮
236	0.0081	124.1311	1.0000	1.0000	0.9800	1.0000	0.9900	1.0000
237	0.0079	126.0716	1.0000	1.0000	0.9800	1.0000	0.9900	1.0000
238	0.0079	126.5823	1.0000	1.0000	0.9800	1.0000	0.9900	1.0000
239	0.0079	126.7106	1.0000	1.0000	0.9800	1.0000	0.9900	1.0000
240	0.0079	127.0003	1.0000	1.0000	0.9800	1.0000	0.9900	1.0000

4.2.2 Seismic Analysis of Main Structure

First, we will check the base shear from the response spectrum analysis and compare it with the base shear manually computed using the static equivalent method. The seismic base shear can be exported from the SAP2000 model, and is given in table 8. And for computing the base shear using the static equivalent method, we need the weight of the structure as well as additional loads included in the seismic design. We can obtain this from the base reaction of the SAP2000 model accounting for the dead load and the superimposed dead load, which are loads included in our seismic design. The base reaction of these loads is presented in table 9.

Table 8: Seismic base shear with response spectrum from SAP2000 model of Main Structure

Seismic Direction	FX kN	FY kN
EQ-X	234.261	71.229
EQ-Y	70.279	237.430

The static equivalent base shear is given by

$$V = C_s W = 0.074 \cdot 2867.680 = 212.210 \text{ kN}\,.$$

Table 9: Base reaction for dead loads from SAP2000 model of Main Structure

Load Type	FZ kN
DEAD	1877.863
SUPERIMPOSED	989.816
Total weight W	**2867.680**

Then we obtain

$$V_{t_X} = 234.261 \text{ kN} > 212.210 \text{ kN} = V$$
$$V_{t_Y} = 237.430 \text{ kN} > 212.210 \text{ kN} = V$$

which implies that the response spectrum scaling is satisfiable and not required to be upscaled.

Now we check the storey drift of the structure due to seismic loading. The storey drift obtained from the model is presented in figure 18. And we have as follows:

$$\delta_{0e,X} = 0.364 \text{ mm}$$
$$\delta_{1e,X} = 5.792 \text{ mm}$$
$$\delta_{0e,Y} = 0.480 \text{ mm}$$
$$\delta_{1e,Y} = 5.823 \text{ mm}$$

Then we obtain

$$\delta_{0,X} = \frac{C_d \delta_{0e,X}}{I_e} = \frac{2.5 \cdot 0.364}{1.0} = 0.910 \text{ mm},$$
$$\delta_{1,X} = \frac{C_d \delta_{1e,X}}{I_e} = \frac{2.5 \cdot 5.792}{1.0} = 14.480 \text{ mm},$$
$$\delta_{0,Y} = \frac{C_d \delta_{0e,Y}}{I_e} = \frac{2.5 \cdot 0.480}{1.0} = 1.200 \text{ mm},$$
$$\delta_{0,Y} = \frac{C_d \delta_{1e,Y}}{I_e} = \frac{2.5 \cdot 5.823}{1.0} = 14.558 \text{ mm}.$$

And the storey drifts for both X and Y directions are given by

$$\Delta_X = \delta_{1,X} - \delta_{0,X} = 14.480 - 0.910 = 13.570 \text{ mm},$$
$$\Delta_Y = \delta_{1,Y} - \delta_{0,Y} = 14.558 - 1.200 = 13.358 \text{ mm}.$$

Note that the storey height is considered to be $h = 6000$ mm. Then the allowable drift is given by

$$\Delta_a := 0.025h = 0.025 \cdot 6000 = 150.000 \text{ mm}.$$

Then we have

$$\Delta_X = 13.570 < 150.000 = \Delta_a$$
$$\Delta_Y = 13.358 < 150.000 = \Delta_a\,,$$

which implies that the storey drifts are still acceptable.

4.2.3 Concrete Structural Analysis of Main Structure

The concrete structural elements of Main Structure consist of tie-beams, 3 types of beams and 2 types of columns, as shown in 19. The computation of the structural capacity including the reinforcement design is conducted using the dedicated algorithm in SAP2000 which follows ACI 318-19, which is equivalent to SNI 2847:2019. The detail

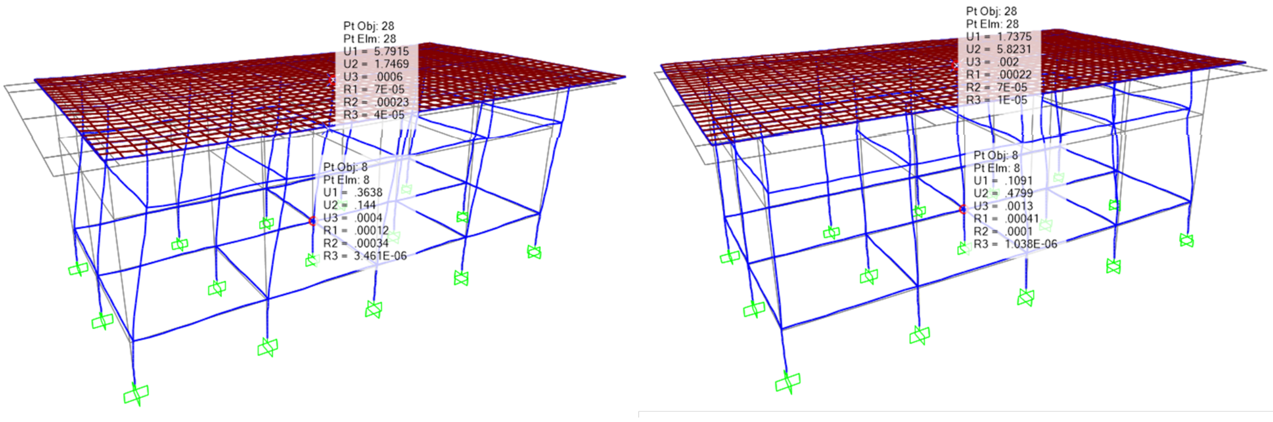


Figure 18: Storey drift from model (left: for X direction, right: for Y direction)

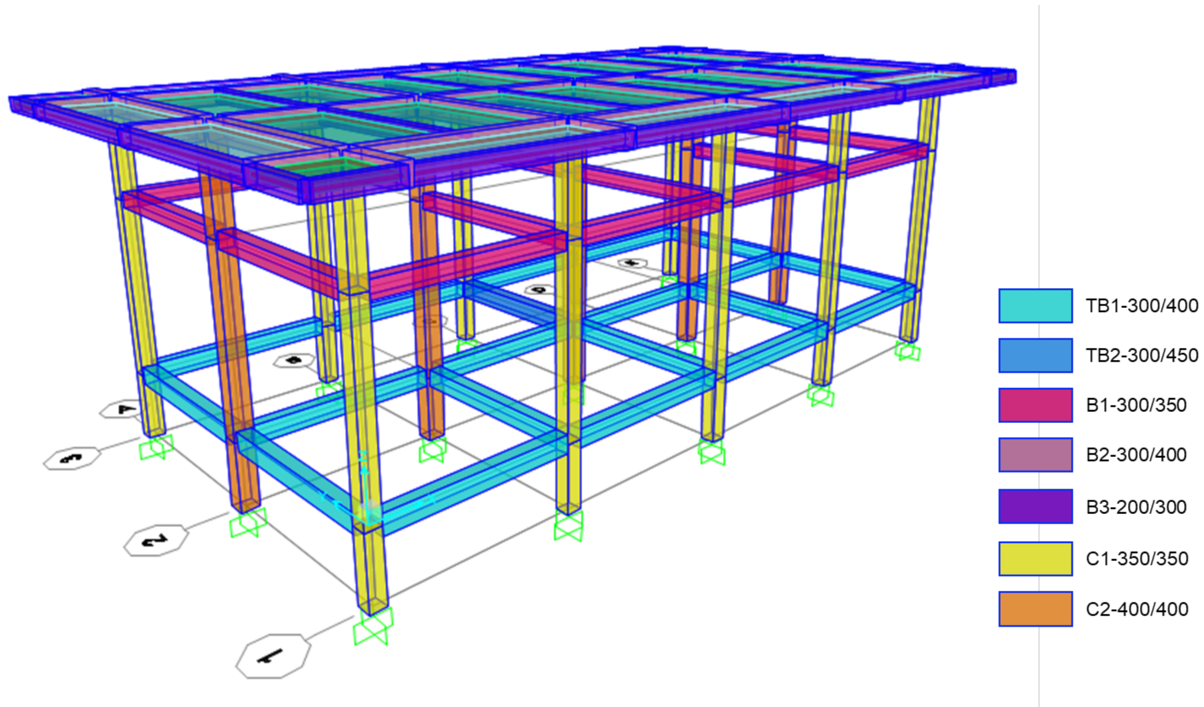


Figure 19: Configuration of concrete structure

calculation of rebars configuration is given the attachment. For the concrete slab of the roof, the calculations of reinforcements are conducted manually and are given in the attachment. The parameter tuning is shown in figure 20. The detail design is presented as follows.

The configurations set up in SAP2000 model of TB1, TB2, B1, B2, B3, C1 and C2 are presented in figures 21, 22, 23, 24, 26 and 27 respectively. The structural capacities to be evaluated include the columns PMM capacity, the 6/5 beam-to-column capacity ratios and the joints capacity. Note that, in terms of the beam-to-column capacity ratio, the constant 6/5 refers to the requirement that the columns shall have 1.2 times the capacity of beams. All these parameters are considered sufficient if the ratios are less than 1. The resulting analysis on these parameters are given in figures 28, 29 and 30.

One last step, the column reinforcements shall satisfy a seismic requirement asserted in SNI 2847:2019 section

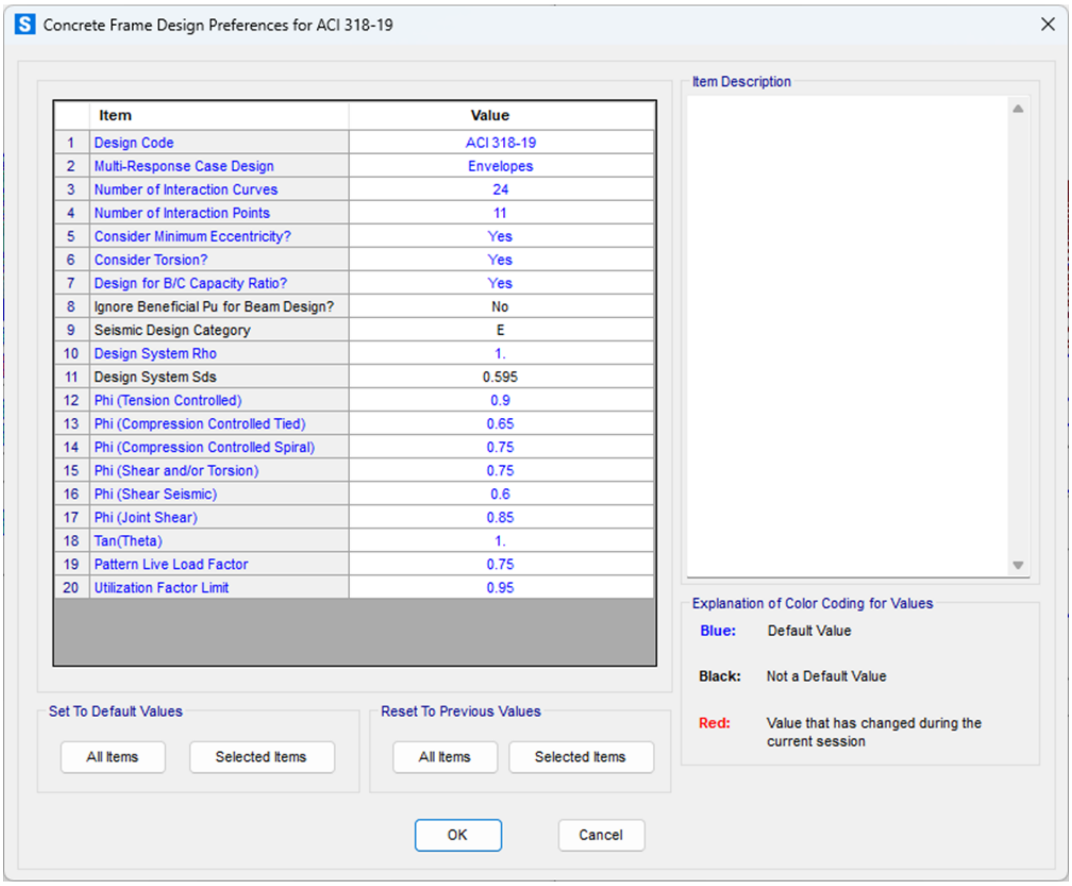


Figure 20: Parameter tuning for concrete structural analysis in SAP2000

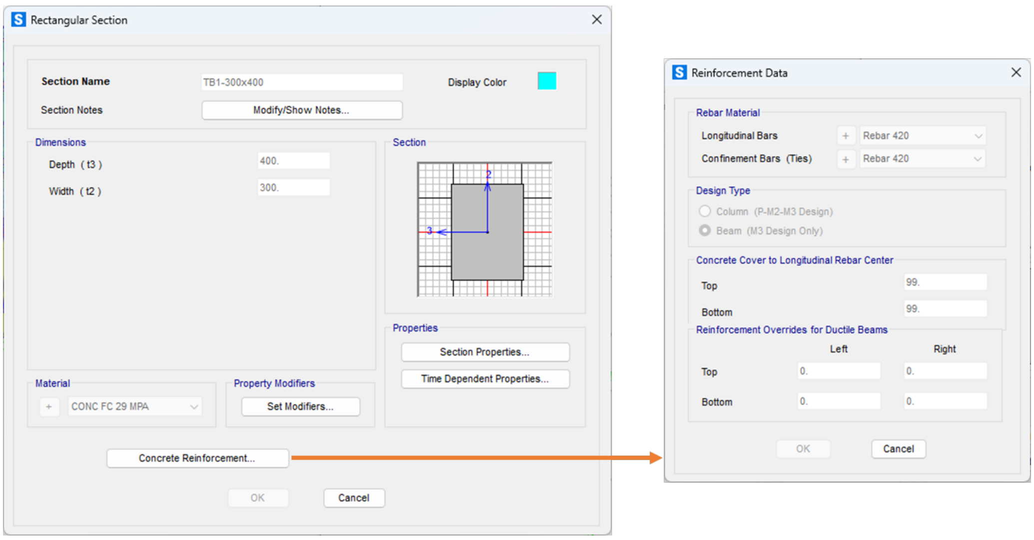


Figure 21: Parameters of TB1 in SAP2000

18.7.4.1, as given by

$$0.01A_g \leq A_{st} \leq 0.06A_g\,, \tag{1}$$

where A_{st} is the cross section of the longitudinal rebars and A_g is the gross cross sectional area of the column, i. e., $A_g = bh$ for a rectangular column with b being the sectional width and h being the sectional height of the column.

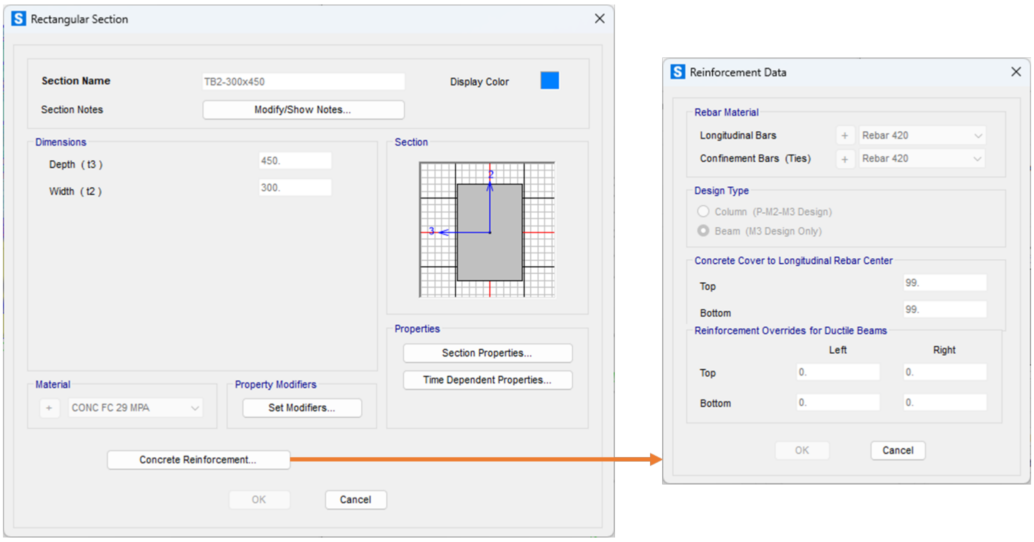


Figure 22: Parameters of TB2 in SAP2000

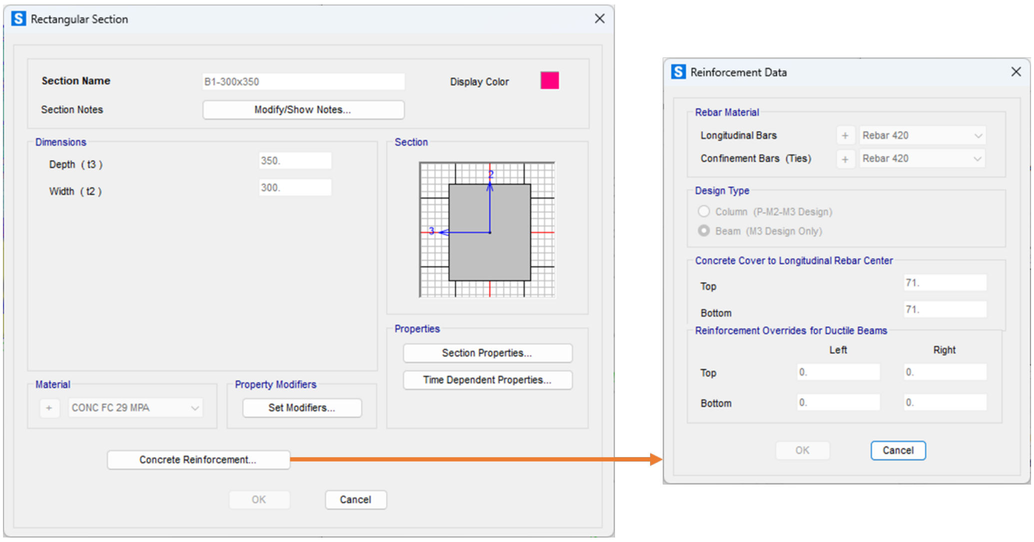


Figure 23: Parameters of B1 in SAP2000

The requirement in expression 1 can equivalently be expressed by

$$0.01 \leq \frac{A_{st}}{A_g} \leq 0.06 \,.$$

The calculation of the analysis regarding this matter is presented in table 10.

Table 10: Reinforcement control in accordance with SNI 2847:2019 section 18.7.4.1

Column	b (mm)	h (mm)	A_g (mm^2)	Reinforcement	n_r	A_{st} (mm^2)	A_{st}/A_g	Satisfiability
C1	350	350	122500	12D19	12	3402.345	0.0278	SATISFIABLE
C2	400	400	160000	8D22	8	3041.062	0.0190	SATISFIABLE

From the calculations given in the attachment, the reinforcement configurations of the elements of concrete structure are presented in table 11.

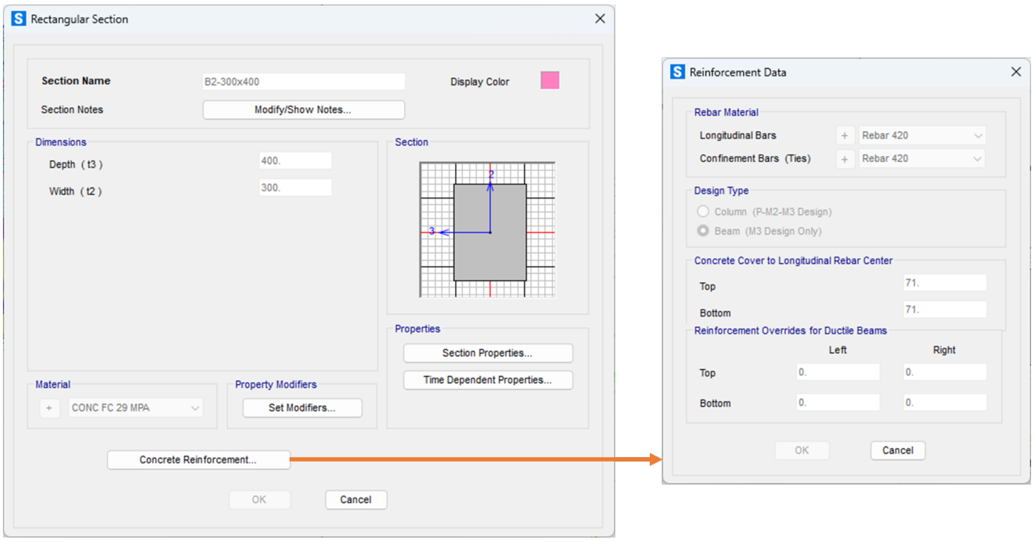


Figure 24: Parameters of B2 in SAP2000

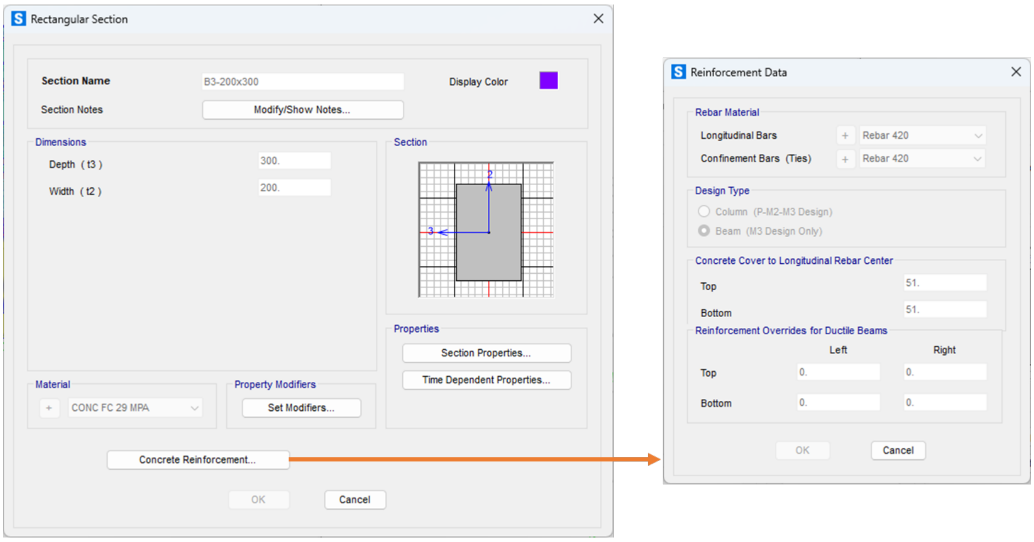


Figure 25: Parameters of B3 in SAP2000

Table 11: Reninforcement of Concrete Structure

ID	Type	b	h	Longitudinal Reinforcement	Transversal Reinforcement		
		mm	mm	End	Mid	End	Mid
S1	Suspended Slab		120	Wire Mesh M8	Wire Mesh M8		
S2	Slab on Ground		200	Wire Mesh M8	Wire Mesh M8		
TB1	Tie-Beam	300	400	9D16	6D16	D13-200	D13-250
TB2	Tie-Beam	300	450	9D16	4D16	D13-200	D13-250
B1	Beam	300	350	9D16	6D16	D13-250	D13-300
B2	Beam	300	400	7D16	6D16	D13-200	D13-200
B3	Beam	200	300	8D16	8D16	D13-200	D13-200
C1	Column	350	350	12D19	12D19	D13-150	D13-150
C2	Column	400	400	8D22	8D22	D13-150	D13-150

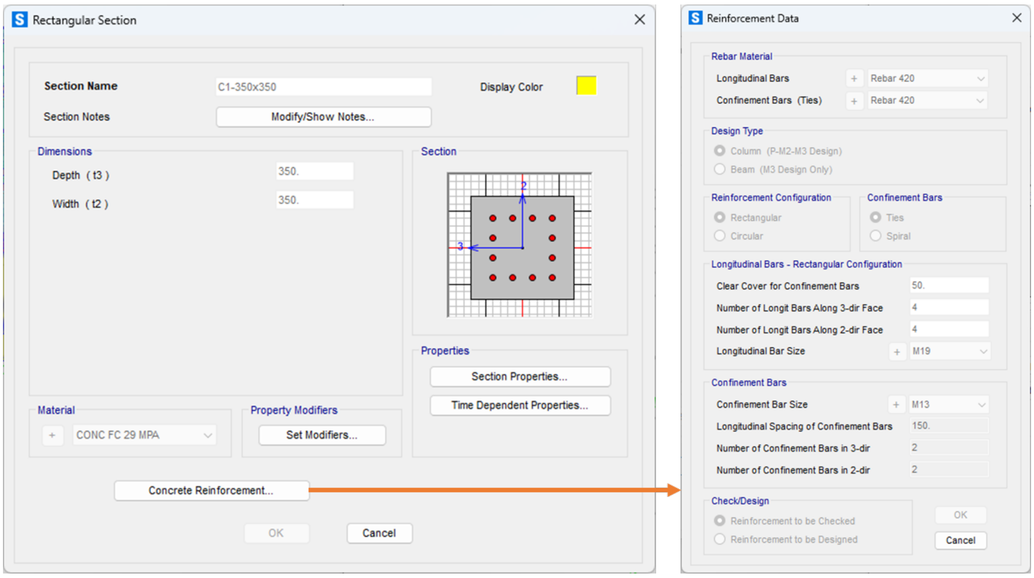


Figure 26: Parameters of C1 in SAP2000

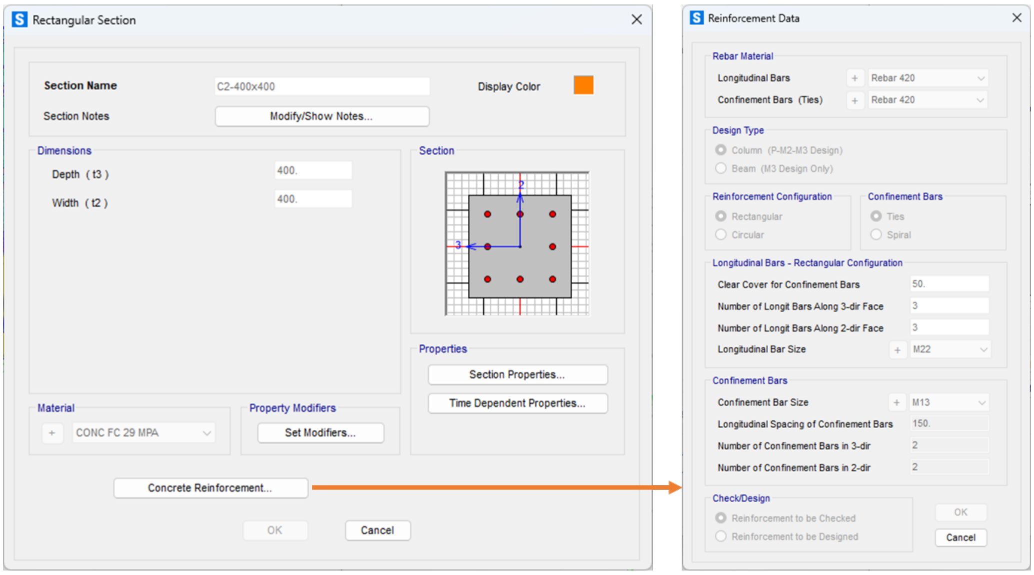


Figure 27: Parameters of C2 in SAP2000

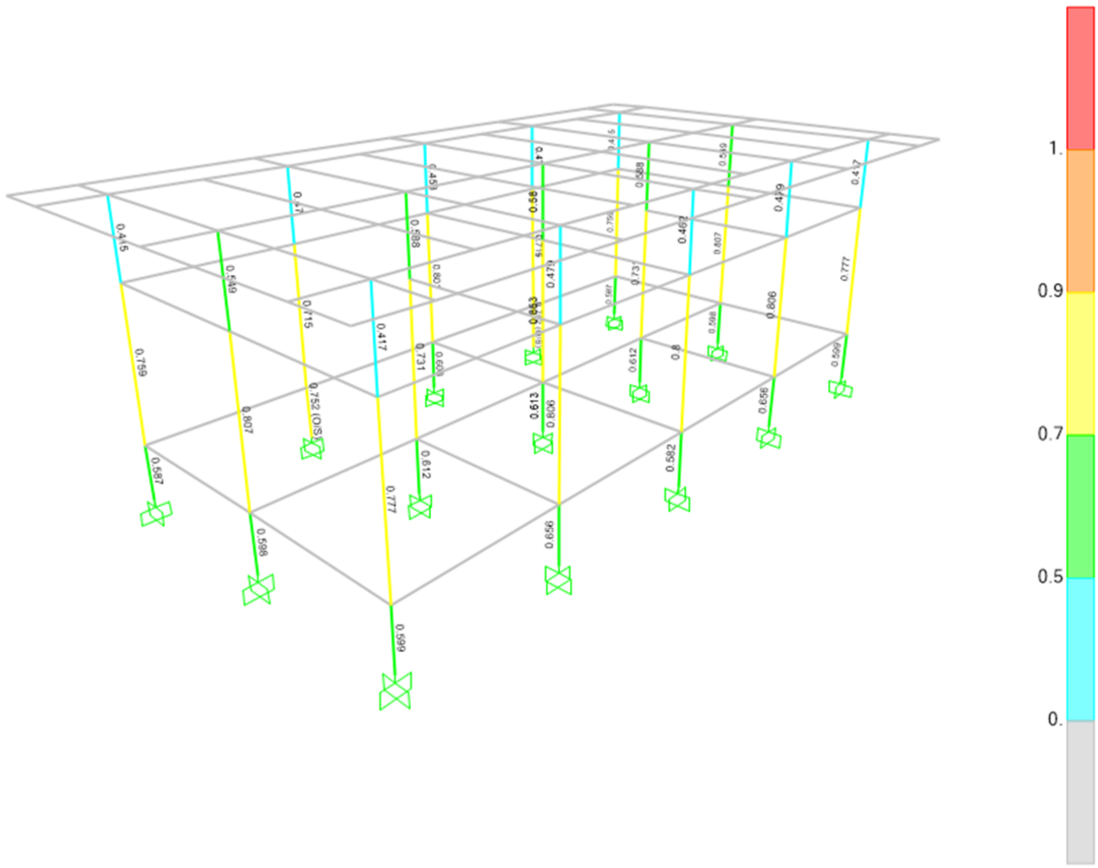


Figure 28: PMM ratio

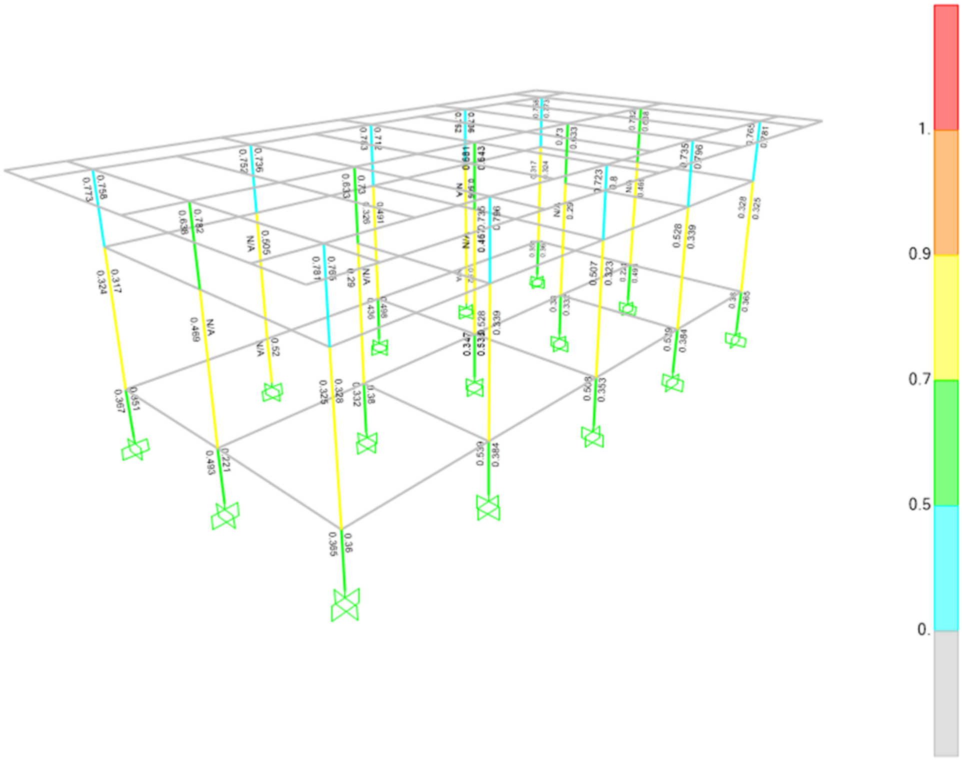


Figure 29: 6/5 beam-to-column ratio

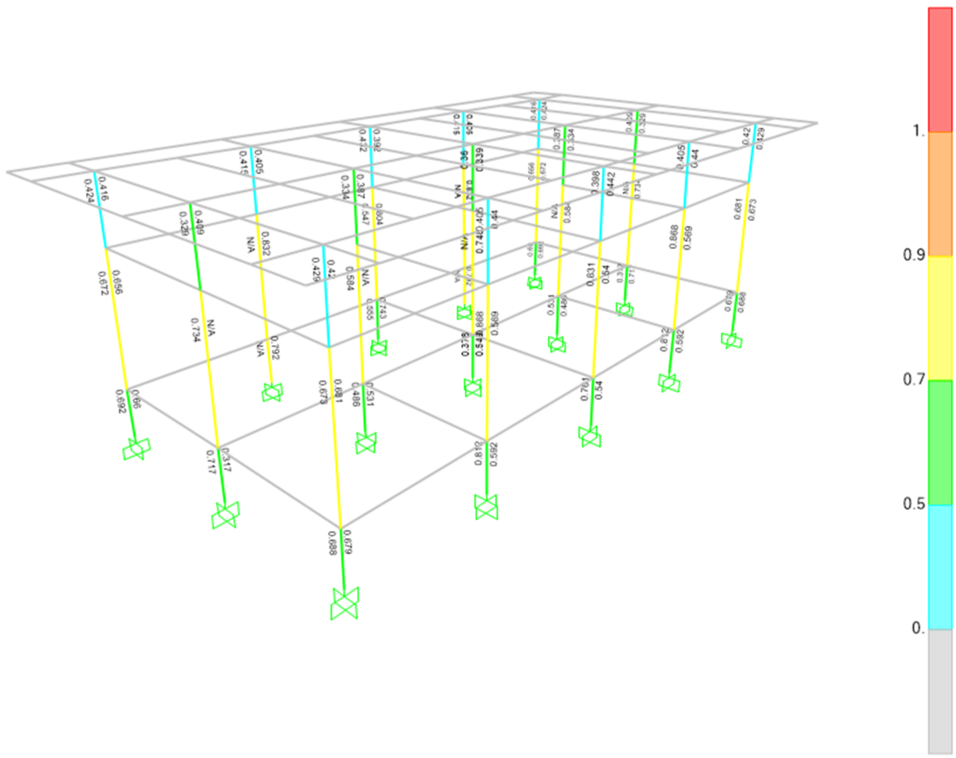


Figure 30: Joint capacity ratio

4.3 Analysis of Blast-Resisting Walls

Note that we model the blast resisting walls (BRWs) together with their footing foundation. The seismic loads for the BRWs are constructed using the response spectrum. Thus, modal analysis is also required. The modal mass participating ration of the BRWs structure is presented in table 12. The result shows that the sum of modal mass participating ratio reaches 0.98 for all the directional components, which is desirable.

Table 12: Modal participating mass ratio for the BRWs

Mode	Period (sec)	Frequency (Hertz)	SumUX	SumUY	SumUZ	SumRX	SumRY	SumRZ
1	0.4182	2.3910	0.0002	0.2900	0.0000	0.5600	0.0002	0.0003
2	0.2329	4.2938	0.7800	0.2900	0.0000	0.5600	0.1900	0.0003
3	0.1927	5.1905	0.7800	0.2900	0.0011	0.5600	0.1900	0.3300
4	0.1800	5.5546	0.7800	0.2900	0.4800	0.5600	0.1900	0.3300
5	0.1516	6.5957	1.0000	0.2900	0.4800	0.5600	0.8500	0.3300
196	0.0019	522.4660	1.0000	1.0000	0.9600	0.9900	0.9800	1.0000
197	0.0019	524.1090	0.0000	0.0000	0.0000	0.9900	0.9800	1.0000
198	0.0019	526.8704	0.0000	0.0000	0.0000	0.9900	0.9800	1.0000
199	0.0019	527.1481	0.0000	0.0000	0.0000	0.9900	0.9800	1.0000
200	0.0019	530.7856	0.0000	0.0000	0.0000	0.9900	0.9800	1.0000

The reinforcement calculations of the BRWs are given in the attachment. The summary of the reinforcements is presented in table 13.

Table 13: Reinforcement summary of Blast Resisting Walls

Element	Reinforcement	
	Horizontal	Vertical
BRW-A	D13-300	D16-200
BRW-B	D13-300	D16-200

5 FOUNDATION DESIGN

The foundations for both Main Structure are computed separately from the upper structures. While for the blast resisting walls, the foundation is computed in a combined model with the walls. We still use SAP2000 for computing the soil compression at the foundations as well as the moments in the foundations structure. The loads to be considered in the foundation model are the output from the support reaction of the upper structures. The components to be evaluated in foundations include the stability of the foundation, the bearing capacity of the foundation and the reinforcement of the foundation. All these evaluations are coducted manually and are given in the attachment.

5.1 Foundation of Main Structure

The foundations of Main Structure are divided into 3 types, namely F1, F2 and F3. The locations of these types are indicated in figure 31. The sketch of the preliminary configuration of F1, F2 and F3 is presented in figure 32.

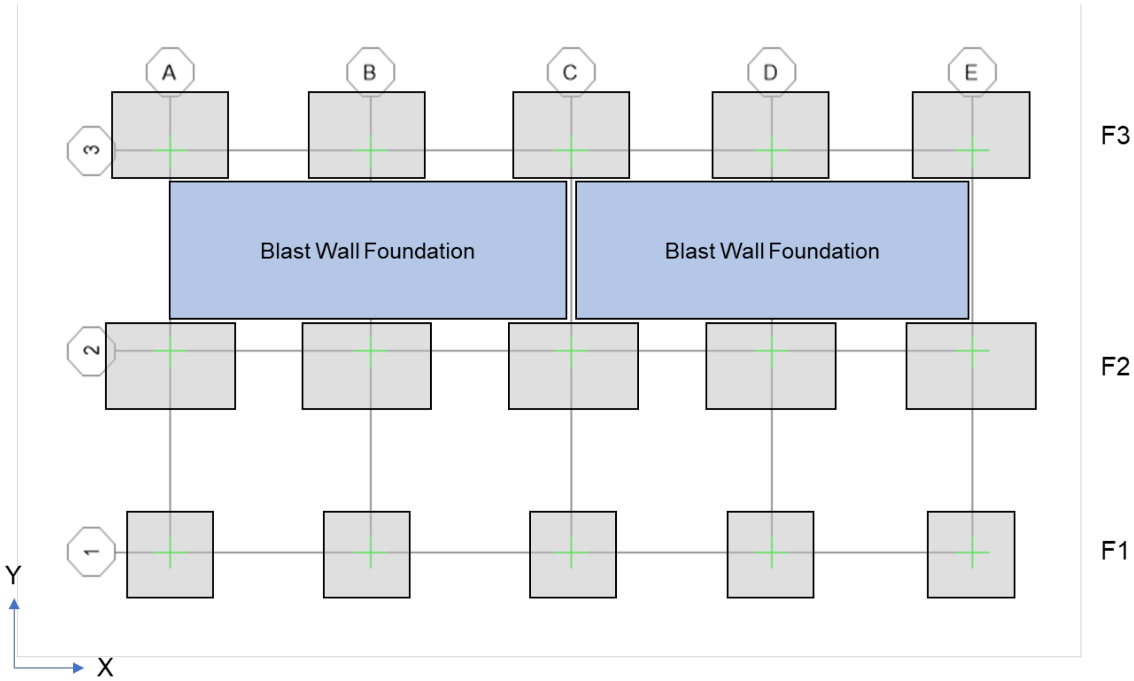


Figure 31: Locations of F1, F2 and F3

We will model F1, F2 and F3 as shells in SAP2000, with area springs assigned to the shells. The spring constant is the modulus of subgrade reaction of the soil, which is computed in accordance with (Das, 2011) by

$$k_s = \frac{E_s}{B(1-\nu_s^2)}\,, \tag{2}$$

where E_s is the Young modulus of soil, ν_s is the Poisson's ratio of soil and B is the width of the foundation shell element mesh. For shallow foundations such as F1 and F2, k_s refers to the gap (compressive-only) spring, instead of the linear spring. And it will be assigned below the shell model of the footing as well as the side surfaces. For the spring below the shell material, we will consider all soil layers in the soil report, which is presented in table 14 together with the computed modulus of subgrade reactions.

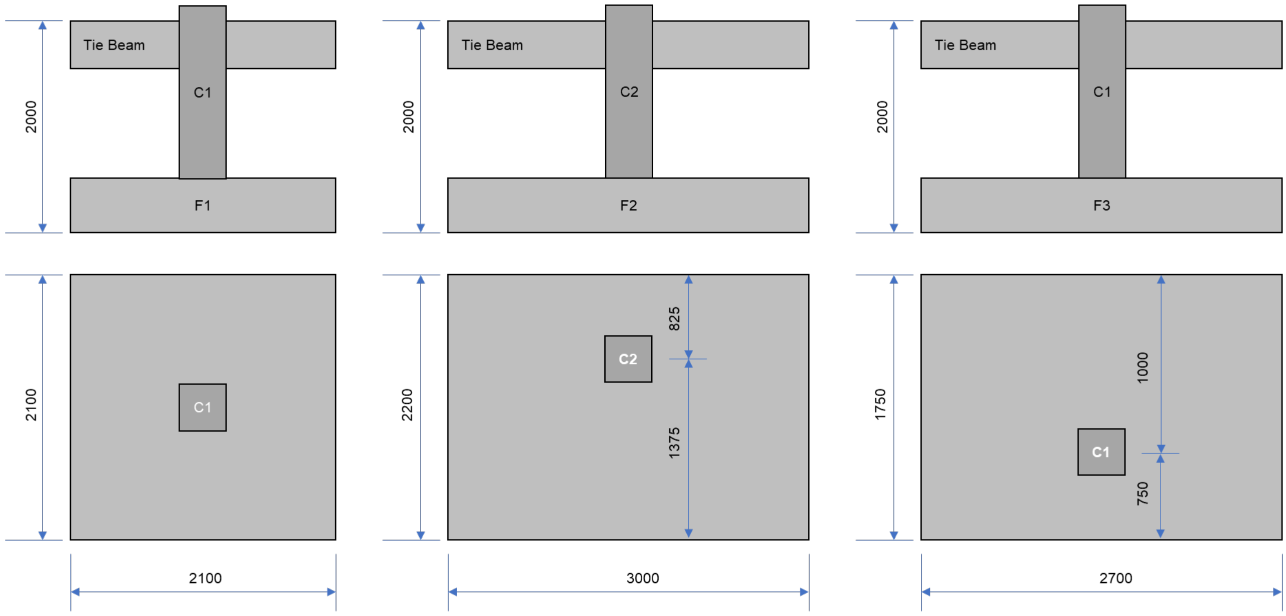


Figure 32: Sketch of the preliminary configuration of F1, F2 and F3

Table 14: Soil properties from soil report

Layer Thickness (m)	$E_{s,k}$ (MPa)	ν_s	k_s (MPa)
19.30	6.40	0.25	0.00683
2.00	8.00	0.25	0.00853
2.00	4.80	0.25	0.00512
1.20	8.00	0.25	0.00853
2.80	60.00	0.25	0.06400

The Poisson's ratio of the soil is not available in the soil report. Alternatively, we can use the data from the correlation between soil consistency and Poisson's ratio (Das and Sobhan, 362). And for the this case, we use $\nu_s = 0.25$. The foundations in SAP2000 are modelled using the feature foundations properties. The parameter tuning for F1, F2 and F3 are shown in figures 33, 34 and 35 respectively. While the models are shown in figures 36, 37 and 38 respectively.

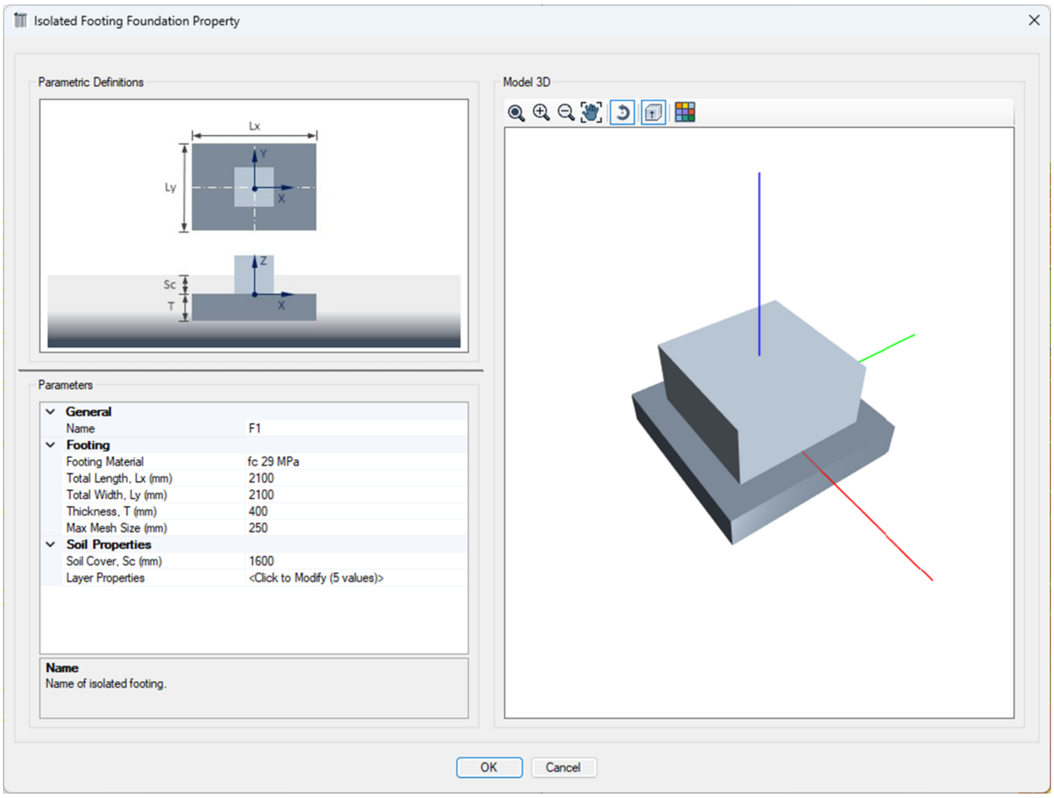


Figure 33: Parameter of F1

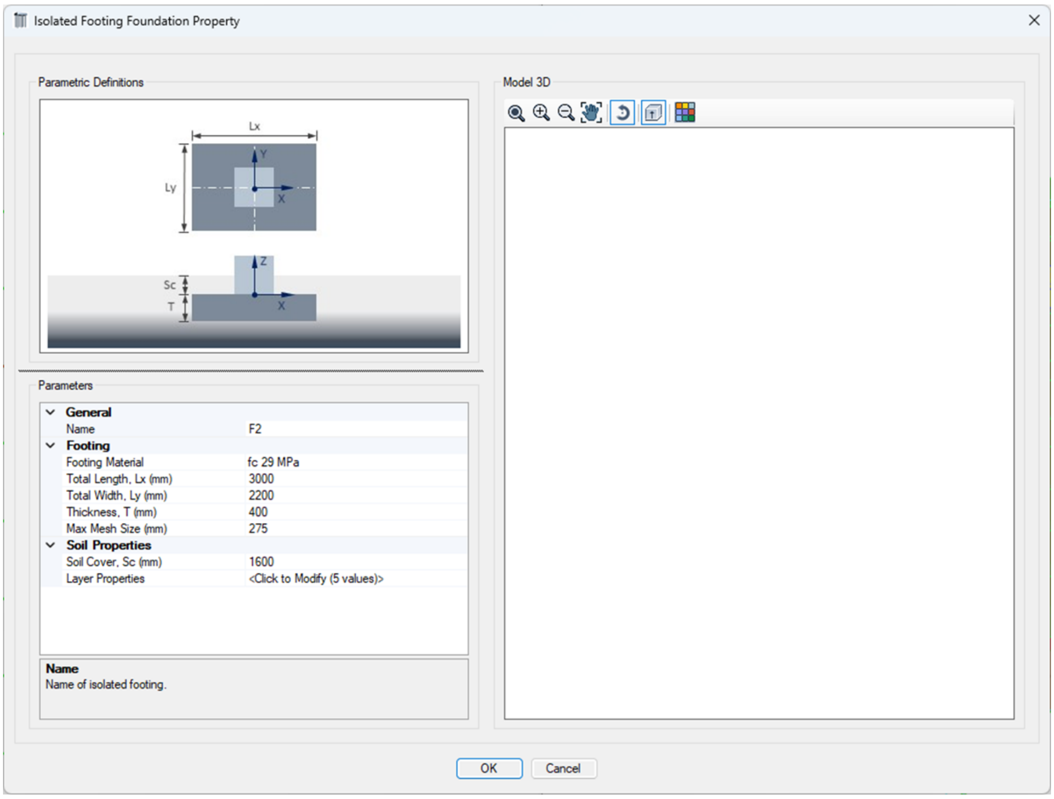


Figure 34: Parameter of F2

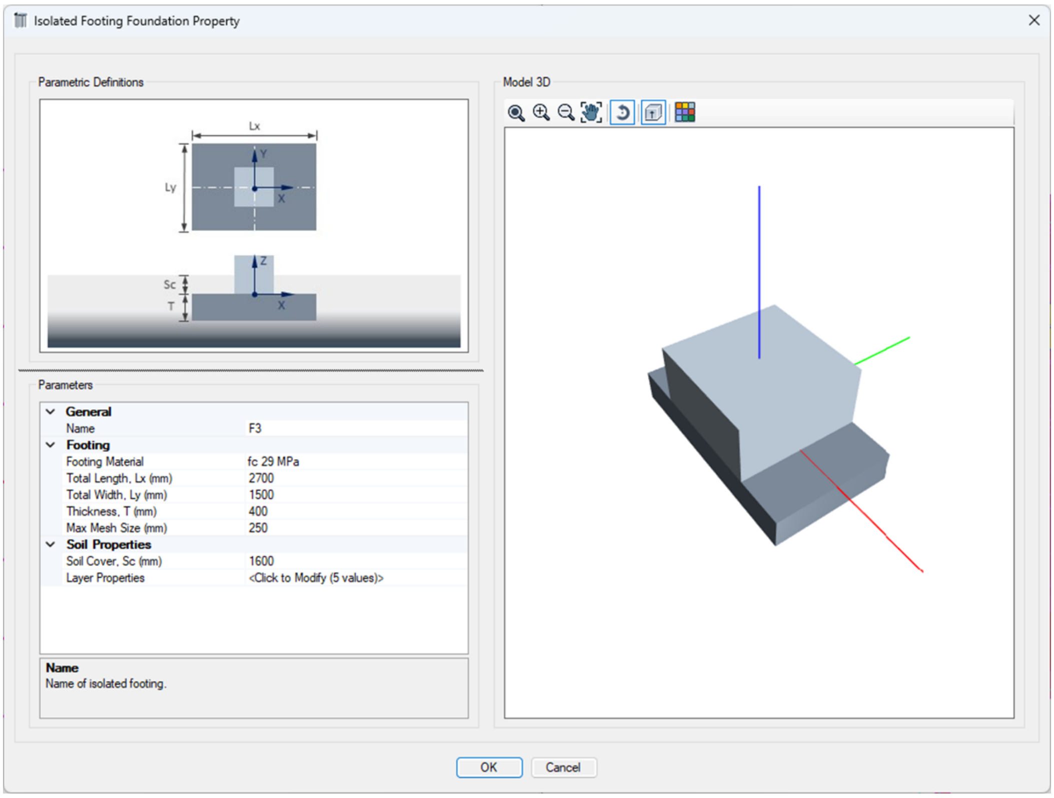


Figure 35: Parameter of F3

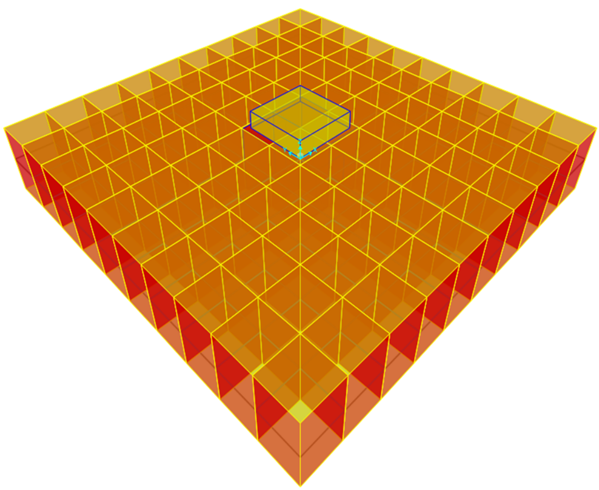


Figure 36: Shell model of F1 in SAP2000

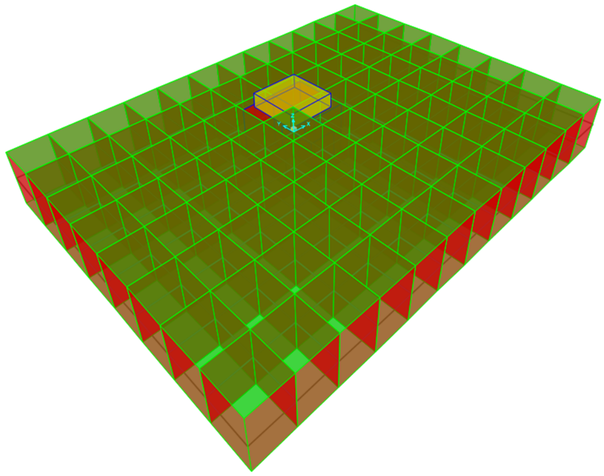


Figure 37: Shell model of F2 in SAP2000

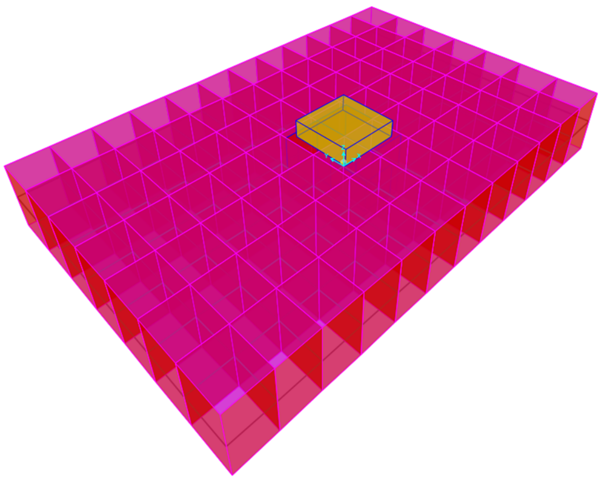


Figure 38: Shell model of F3 in SAP2000

Then the loads from the support reaction of the upper structure are applied to the column location on the foundations. The loads being considered are combinations for foundations including overstrength combinations selected in terms of dominant lateral, vertical and moment in direction-wise.

From the computation in SAP2000, the critical soil stresses of all foundations are illustrated in figures 39, 40, 41. The calculations of the reinforcements of the foundations are given in the attachment. The summary of the reinforcements is presented in table 15.

Table 15: Reinforcement summary of F1, F2 and F3

ID	Reinforcement	
	X-Direction	Y-Direction
F1	D25-100	D22-100
F2	D22-100	D22-100
F3	D22-100	D22-100

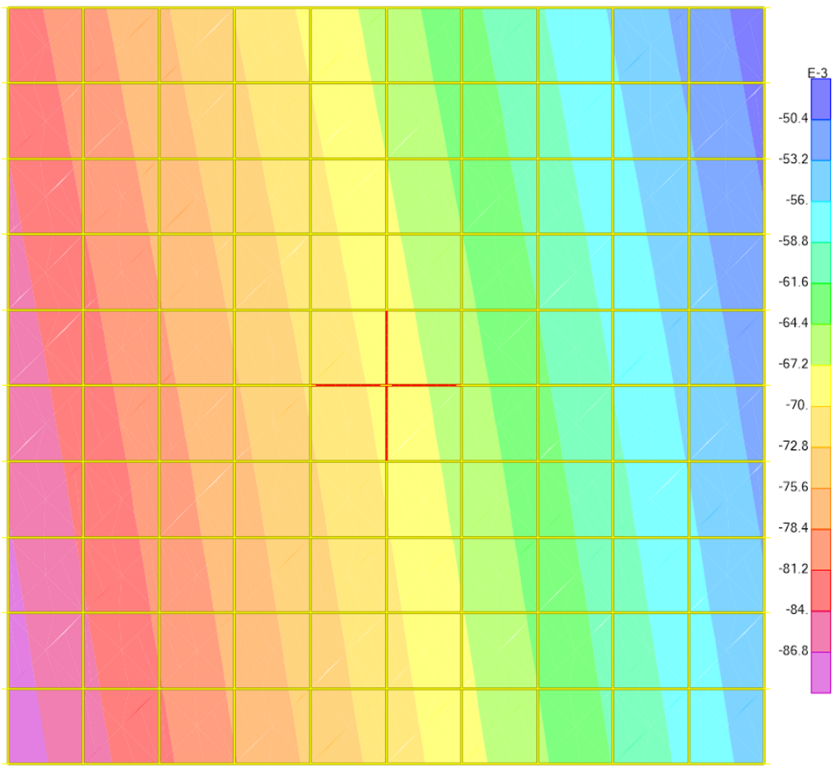


Figure 39: Non-overstrength critical soil pressure of F1 (MPa)

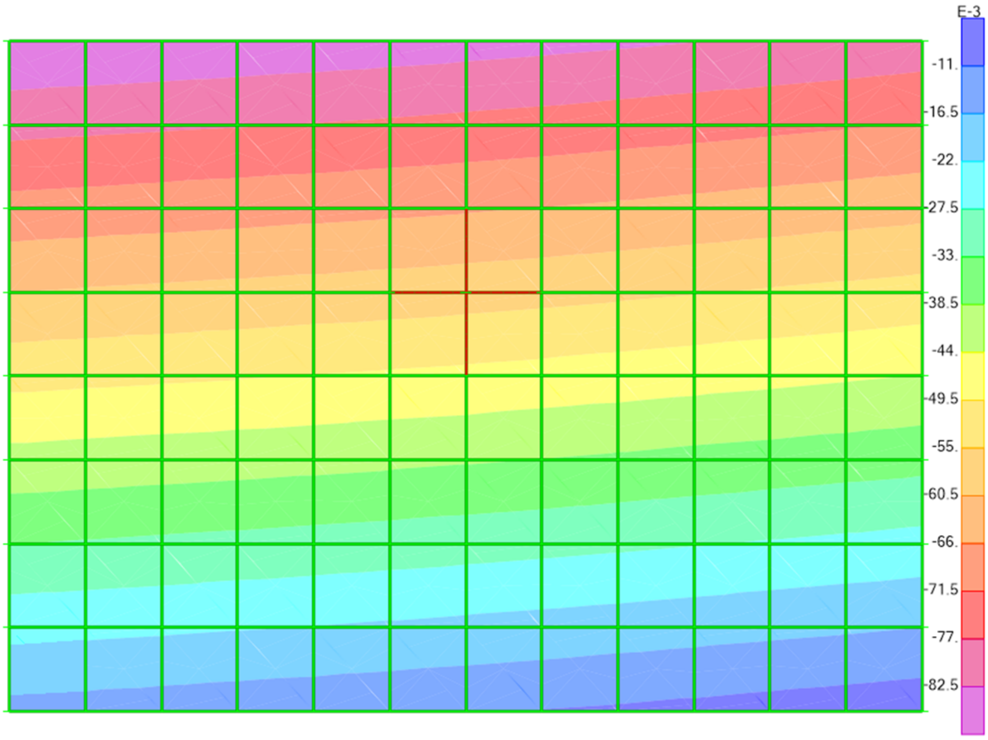


Figure 40: Non-overstrength critical soil pressure of F2 (MPa)

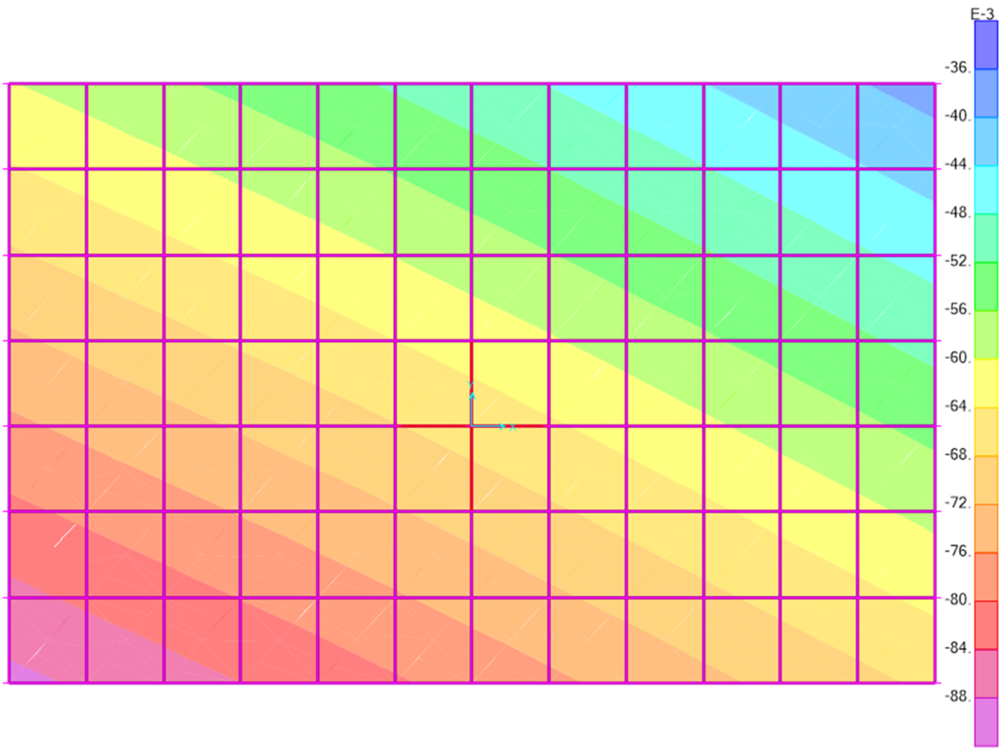


Figure 41: Non-overstrength critical soil pressure of F3 (MPa)

5.2 Foundation of Blast Resisting Walls

Note that we conducted the computation of BRWs foundation together with the upper structure. The detail calculation of the foundation reinforcement is given in the attachment. The most critical soil pressure of the foundation is presented in figures 42 and 43.

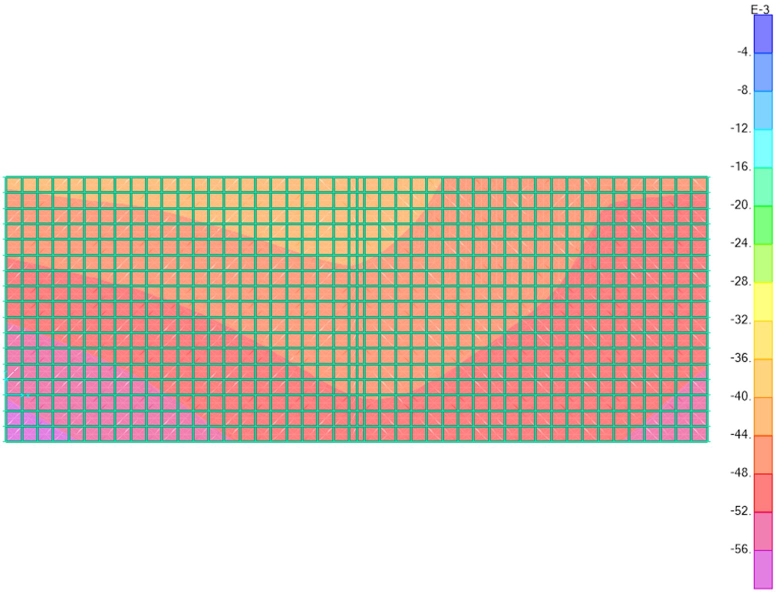


Figure 42: Soil pressure of BRW foundation due to 1.0D + 0.7Ev + 0.7Ehx

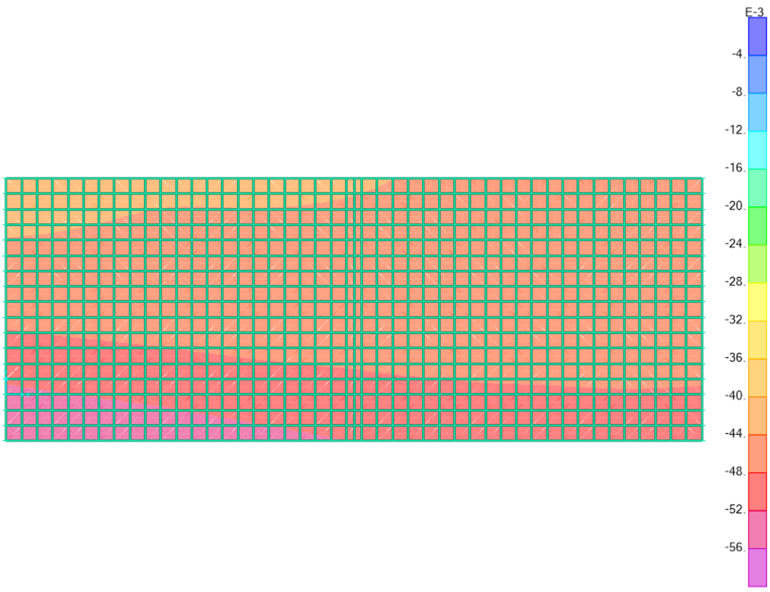


Figure 43: Soil pressure of BRW foundation due to 1.0D + 0.7Ev + 0.7Ehy

The result of the reinforcement calculation is given in table 16.

Table 16: Reinforcement summary of the foundation of blast resisting walls

Direction	Reinforcement
X-Direction	D22-100
Y-Direction	D22-150

References

Das, B. M. (2011). *Principles of Foundation Engineering*. Cengage Learning.

Das, B. M. and Sobhan, K. (362). *Principles of Geotechnical Engineering*. Cengage Learning.

Roman, S. (2005). *Graduate Text in Mathematics: Advanced Linear Algebra*. Springer.

Rudin, W. (1976). *Principles of Mathematical Analysis, 3rd Edition*. McGraw-Hill.

ATTACHMENT 1

Beam Reinforcements

Reinforcement Configuration of Beam TB1

A. INTRODUCTION

This document covers the configuration of beams reinforcement. The analysis of the reinforcement is conducted in SAP2000 with outputs of rebars area for longitudinal rebars and rebars area per length for transfersal rebars.

A.1. Beam Geometric Data

Width of the beam	$b =$	300	mm
Height of the beam	$h =$	400	mm
Concrete clear cover	$cov =$	50	mm
Diameter of longitudinal rebars	$D_L =$	16	mm
Diameter of transfersal rebars	$D_t =$	13	mm

B. LONGITUDINAL REINFORCEMENT

Area of a single rebar	$A_r =$	201.06	mm^2

B.1. At the End Span

B.1.1. Top Fibre

Rebars area at the top fibre (from SAP2000)	$A_{s.top} =$	994.49	mm^2
Number of required rebars	$n =$	5.00	pcs
Number for first layer	$n_1 =$	3.00	pcs
Number for second layer	$n_2 =$	2.00	pcs
Distance between adjacent rebars	$s =$	63.00	mm

CONCLUSION: SATISFIED

B.1.2. Bottom Fibre

Rebars area at the bottom fibre (from SAP2000)	$A_{s.bot} =$	669.52	mm^2
Number of required rebars	$n =$	4.00	pcs
Number for first layer	$n_1 =$	2.00	pcs
Number for second layer	$n_2 =$	2.00	pcs
Distance between adjacent rebars	$s =$	142.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE 9D16

B.2. At the Mid Span

B.2.1. Top Fibre

Rebars area at the top fibre (from SAP2000)	$A_{s.top} =$	517.99	mm^2
Number of required rebars	$n =$	3.00	pcs
Number for first layer	$n_1 =$	3.00	pcs
Number for second layer	$n_2 =$	0.00	pcs
Distance between adjacent rebars	$s =$	63.00	mm

CONCLUSION: SATISFIED

B.2.2. Bottom Fibre

Rebars area at the bottom fibre (from SAP2000)	$A_{s.bot} =$	517.99	mm^2

Number of required rebars	$n =$	3.00	pcs
Number for first layer	$n_1 =$	3.00	pcs
Number for second layer	$n_2 =$	0.00	pcs
Distance between adjacent rebars	$s =$	63.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE 6D16

C. TRANSVERSAL REINFORCEMENT

Number of legs of the rebar	leg =	2.00	
Area of transversal reinforcement	$A_v =$	265.46	mm^2

C.1. At the End Span

Rebars area to space ratio (from SAP2000)	$A_v/s =$	0.93	mm^2/mm
Maximum required spacing	$s_{req} =$	285.45	mm
Spacing in use	$s =$	200.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE D13-200

C.2. At the Mid Span

Rebars area to space ratio (from SAP2000)	$A_v/s =$	0.25	mm^2/mm
Maximum required spacing	$s_{req} =$	1061.86	mm
Spacing in use	$s =$	250.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE D13-250

Reinforcement Configuration of Beam TB2

A. INTRODUCTION

This document covers the configuration of beams reinforcement. The analysis of the reinforcement is conducted in SAP2000 with outputs of rebars area for longitudinal rebars and rebars area per length for transfersal rebars.

A.1. Beam Geometric Data

Width of the beam	$b =$	300	mm
Height of the beam	$h =$	450	mm
Concrete clear cover	$cov =$	50	mm
Diameter of longitudinal rebars	$D_L =$	16	mm
Diameter of transfersal rebars	$D_t =$	13	mm

B. LONGITUDINAL REINFORCEMENT

Area of a single rebar	$A_r =$	201.06	mm^2

B.1. At the End Span

B.1.1. Top Fibre

Rebars area at the top fibre (from SAP2000)	$A_{s.top} =$	974.19	mm^2
Number of required rebars	$n =$	5.00	pcs
Number for first layer	$n_1 =$	3.00	pcs
Number for second layer	$n_2 =$	2.00	pcs
Distance between adjacent rebars	$s =$	63.00	mm

CONCLUSION: SATISFIED

B.1.2. Bottom Fibre

Rebars area at the bottom fibre (from SAP2000)	$A_{s.bot} =$	700.98	mm^2
Number of required rebars	$n =$	4.00	pcs
Number for first layer	$n_1 =$	2.00	pcs
Number for second layer	$n_2 =$	2.00	pcs
Distance between adjacent rebars	$s =$	142.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE 9D16

B.2. At the Mid Span

B.2.1. Top Fibre

Rebars area at the top fibre (from SAP2000)	$A_{s.top} =$	345.72	mm^2
Number of required rebars	$n =$	2.00	pcs
Number for first layer	$n_1 =$	2.00	pcs
Number for second layer	$n_2 =$	0.00	pcs
Distance between adjacent rebars	$s =$	142.00	mm

CONCLUSION: SATISFIED

B.2.2. Bottom Fibre

Rebars area at the bottom fibre (from SAP2000)	$A_{s.bot} =$	345.72	mm^2

Number of required rebars	$n =$	2.00	pcs
Number for first layer	$n_1 =$	2.00	pcs
Number for second layer	$n_2 =$	0.00	pcs
Distance between adjacent rebars	$s =$	142.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE 4D16

C. TRANSVERSAL REINFORCEMENT

Number of legs of the rebar	leg =	2.00	
Area of transversal reinforcement	$A_v =$	265.46	mm^2

C.1. At the End Span

Rebars area to space ratio (from SAP2000)	$A_v/s =$	0.94	mm^2/mm
Maximum required spacing	$s_{req} =$	282.11	mm
Spacing in use	$s =$	200.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE D13-200

C.2. At the Mid Span

Rebars area to space ratio (from SAP2000)	$A_v/s =$	0.25	mm^2/mm
Maximum required spacing	$s_{req} =$	1079.12	mm
Spacing in use	$s =$	250.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE D13-250

Reinforcement Configuration of Beam B1

A. INTRODUCTION

This document covers the configuration of beams reinforcement. The analysis of the reinforcement is conducted in SAP2000 with outputs of rebars area for longitudinal rebars and rebars area per length for transfersal rebars.

A.1. Beam Geometric Data

Width of the beam	$b =$	300	mm
Height of the beam	$h =$	350	mm
Concrete clear cover	$cov =$	50	mm
Diameter of longitudinal rebars	$D_L =$	16	mm
Diameter of transfersal rebars	$D_t =$	13	mm

B. LONGITUDINAL REINFORCEMENT

Area of a single rebar	$A_r =$	201.06	mm^2

B.1. At the End Span

B.1.1. Top Fibre

Rebars area at the top fibre (from SAP2000)	$A_{s.top} =$	941.61	mm^2
Number of required rebars	$n =$	5.00	pcs
Number for first layer	$n_1 =$	3.00	pcs
Number for second layer	$n_2 =$	2.00	pcs
Distance between adjacent rebars	$s =$	63.00	mm

CONCLUSION: SATISFIED

B.1.2. Bottom Fibre

Rebars area at the bottom fibre (from SAP2000)	$A_{s.bot} =$	740.83	mm^2
Number of required rebars	$n =$	4.00	pcs
Number for first layer	$n_1 =$	2.00	pcs
Number for second layer	$n_2 =$	2.00	pcs
Distance between adjacent rebars	$s =$	142.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE 9D16

B.2. At the Mid Span

B.2.1. Top Fibre

Rebars area at the top fibre (from SAP2000)	$A_{s.top} =$	441.65	mm^2
Number of required rebars	$n =$	3.00	pcs
Number for first layer	$n_1 =$	3.00	pcs
Number for second layer	$n_2 =$	0.00	pcs
Distance between adjacent rebars	$s =$	63.00	mm

CONCLUSION: SATISFIED

B.2.2. Bottom Fibre

Rebars area at the bottom fibre (from SAP2000)	$A_{s.bot} =$	441.65	mm^2

Number of required rebars	n =	3.00	pcs
Number for first layer	n_1 =	3.00	pcs
Number for second layer	n_2 =	0.00	pcs
Distance between adjacent rebars	s =	63.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE 6D16

C. TRANSVERSAL REINFORCEMENT

Number of legs of the rebar	leg =	2.00	
Area of transversal reinforcement	A_v =	265.46	mm^2

C.1. At the End Span

Rebars area to space ratio (from SAP2000)	A_v/s =	0.80	mm^2/mm
Maximum required spacing	s_{req} =	332.66	mm
Spacing in use	s =	250.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE D13-250

C.2. At the Mid Span

Rebars area to space ratio (from SAP2000)	A_v/s =	0.25	mm^2/mm
Maximum required spacing	s_{req} =	1079.12	mm
Spacing in use	s =	300.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE D13-300

Reinforcement Configuration of Beam B2

A. INTRODUCTION

This document covers the configuration of beams reinforcement. The analysis of the reinforcement is conducted in SAP2000 with outputs of rebars area for longitudinal rebars and rebars area per length for transfersal rebars.

A.1. Beam Geometric Data

Width of the beam	b =	300	mm
Height of the beam	h =	400	mm
Concrete clear cover	cov =	50	mm
Diameter of longitudinal rebars	D_L =	16	mm
Diameter of transfersal rebars	D_t =	13	mm

B. LONGITUDINAL REINFORCEMENT

Area of a single rebar	A_r =	201.06	mm^2

B.1. At the End Span

B.1.1. Top Fibre

Rebars area at the top fibre (from SAP2000)	$A_{s.top}$ =	661.43	mm^2
Number of required rebars	n =	4.00	pcs
Number for first layer	n_1 =	2.00	pcs
Number for second layer	n_2 =	2.00	pcs
Distance between adjacent rebars	s =	142.00	mm

CONCLUSION: SATISFIED

B.1.2. Bottom Fibre

Rebars area at the bottom fibre (from SAP2000)	$A_{s.bot}$ =	517.99	mm^2
Number of required rebars	n =	3.00	pcs
Number for first layer	n_1 =	3.00	pcs
Number for second layer	n_2 =	0.00	pcs
Distance between adjacent rebars	s =	63.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE 7D16

B.2. At the Mid Span

B.2.1. Top Fibre

Rebars area at the top fibre (from SAP2000)	$A_{s.top}$ =	517.99	mm^2
Number of required rebars	n =	3.00	pcs
Number for first layer	n_1 =	3.00	pcs
Number for second layer	n_2 =	0.00	pcs
Distance between adjacent rebars	s =	63.00	mm

CONCLUSION: SATISFIED

B.2.2. Bottom Fibre

Rebars area at the bottom fibre (from SAP2000)	$A_{s.bot}$ =	517.99	mm^2

Number of required rebars	$n =$	3.00	pcs
Number for first layer	$n_1 =$	3.00	pcs
Number for second layer	$n_2 =$	0.00	pcs
Distance between adjacent rebars	$s =$	63.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE **6D16**

C. TRANSVERSAL REINFORCEMENT

Number of legs of the rebar	$leg =$	2.00	
Area of transversal reinforcement	$A_v =$	265.46	mm^2

C.1. At the End Span

Rebars area to space ratio (from SAP2000)	$A_v/s =$	1.04	mm^2/mm
Maximum required spacing	$s_{req} =$	255.01	mm
Spacing in use	$s =$	200.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE **D13-200**

C.2. At the Mid Span

Rebars area to space ratio (from SAP2000)	$A_v/s =$	1.04	mm^2/mm
Maximum required spacing	$s_{req} =$	255.01	mm
Spacing in use	$s =$	200.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE **D13-200**

Reinforcement Configuration of Beam B3

A. INTRODUCTION

This document covers the configuration of beams reinforcement. The analysis of the reinforcement is conducted in SAP2000 with outputs of rebars area for longitudinal rebars and rebars area per length for transfersal rebars.

A.1. Beam Geometric Data

Width of the beam	$b =$	200	mm
Height of the beam	$h =$	300	mm
Concrete clear cover	$cov =$	25	mm
Diameter of longitudinal rebars	$D_L =$	16	mm
Diameter of transfersal rebars	$D_t =$	13	mm

B. LONGITUDINAL REINFORCEMENT

Area of a single rebar	$A_r =$	201.06	mm^2

B.1. At the End Span

B.1.1. Top Fibre

Rebars area at the top fibre (from SAP2000)	$A_{s.top} =$	661.43	mm^2
Number of required rebars	$n =$	4.00	pcs
Number for first layer	$n_1 =$	2.00	pcs
Number for second layer	$n_2 =$	2.00	pcs
Distance between adjacent rebars	$s =$	92.00	mm

CONCLUSION: SATISFIED

B.1.2. Bottom Fibre

Rebars area at the bottom fibre (from SAP2000)	$A_{s.bot} =$	517.99	mm^2
Number of required rebars	$n =$	3.00	pcs
Number for first layer	$n_1 =$	2.00	pcs
Number for second layer	$n_2 =$	2.00	pcs
Distance between adjacent rebars	$s =$	92.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE 8D16

B.2. At the Mid Span

B.2.1. Top Fibre

Rebars area at the top fibre (from SAP2000)	$A_{s.top} =$	517.99	mm^2
Number of required rebars	$n =$	3.00	pcs
Number for first layer	$n_1 =$	2.00	pcs
Number for second layer	$n_2 =$	2.00	pcs
Distance between adjacent rebars	$s =$	92.00	mm

CONCLUSION: SATISFIED

B.2.2. Bottom Fibre

Rebars area at the bottom fibre (from SAP2000)	$A_{s.bot} =$	517.99	mm^2

Number of required rebars	$n =$	3.00	pcs
Number for first layer	$n_1 =$	2.00	pcs
Number for second layer	$n_2 =$	2.00	pcs
Distance between adjacent rebars	$s =$	92.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE 8D16

C. TRANSVERSAL REINFORCEMENT

Number of legs of the rebar	leg =	2.00	
Area of transversal reinforcement	$A_v =$	265.46	mm^2

C.1. At the End Span

Rebars area to space ratio (from SAP2000)	$A_v/s =$	1.04	mm^2/mm
Maximum required spacing	$s_{req} =$	255.01	mm
Spacing in use	$s =$	200.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE D13-200

C.2. At the Mid Span

Rebars area to space ratio (from SAP2000)	$A_v/s =$	1.04	mm^2/mm
Maximum required spacing	$s_{req} =$	255.01	mm
Spacing in use	$s =$	200.00	mm

CONCLUSION: SATISFIED

REINFORCEMENT CONFIGURATION IN USE D13-200

ATTACHMENT 2

Slab Reinforcements

Design of Concrete Slab S1 (1)

A. INTRODUCTION

This document contains the design and analysis of concrete slab structure. It will covers the determination of the minimum slab thickness, reinforcement design and evaluation against the design forces. The computation of the design forces on the slab is conducted separately using SAP2000.

B. GENERAL GEOMETRIC AND MECHANICAL PROPERTIES

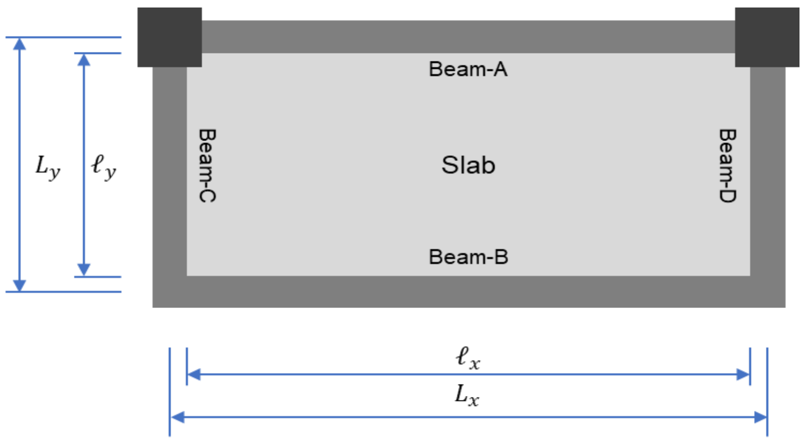
Concrete strength in use	f_c' =	29.00	MPa
Whitney factor corresponding the concrete strength:	β_1 =	0.84	

(SNI 2847:2019 table 22.2.2.4.3)

$$\beta_1 = \begin{cases} 0.85 & : 17\ \mathrm{MPa} \le f_c' \le 28\ \mathrm{MPa} \\ 0.85 - \frac{0.05(f_c'-28)}{7} & : 28\ \mathrm{MPa} < f_c' < 55\ \mathrm{MPa} \\ 0.65 & : f_c' \ge 55\ \mathrm{MPa} \end{cases}$$

Reinforcement code: **WIRE MESH M8**

Reinforcing bars strength in use	f_y =	490.00	MPa
Main rebar diameter	D =	8.00	mm
Secondary (Shrinkage and temperature) rebar diameter	D_t =	8.00	mm
Distance of adjacent main rebars	s_1 =	150.00	mm
Distance of adjacent secondary rebars	s_2 =	150.00	mm



Width of beams surrounding the slab:

Beam-A	b_A =	300.00	mm
Beam-B	b_B =	200.00	mm
Beam-C	b_C =	300.00	mm
Beam-D	b_D =	300.00	mm

Height of beams surrounding the slab:

Beam-A	h_A =	400.00	mm
Beam-B	h_B =	300.00	mm
Beam-C	h_C =	400.00	mm

Beam-D	h_D =	400.00	mm
Slab length axis-to-axis	ℓ_x =	4500.00	mm
Slab width axis-to-axis	ℓ_y =	2250.00	mm
Clear slab length	ℓ_{nx} =	4250.00	mm
Clear slab width	ℓ_{ny} =	1950.00	mm

C. SLAB THICKNESS

The minimum thickness of the slab shall comply with the requirement in SNI 2847:2019 section 8.3.1. And the chosen thickness of the slab shall be sufficient aginst the design loads on th slab as demonstrated later.

Slab length to width ratio: β = 2.18

$$\beta = \frac{\max\{\ell_x, \ell_y\}}{\min\{\ell_x, \ell_y\}}$$

Type of slab: ONE-WAY

For a one-way slab, the minimum required thickness is given in accordane with SNI 2847:2029 table 7.3.1.1 as follows:

Minimum required thickness	$t_{p.min}$ =	80.36	mm
Slab thickness in use	t_p =	120.00	mm

D. DESIGN LOADS ON THE SLAB

The values for the design loads on the slab are determined from the computation using SAP2000. The required additional dead load and the live load are presented as follows:

Additional Dead Loads:

Screeding or moratar overlay (0.5 cm)	SDL =	0.12	kPa
Roof Live Loads:	RL =	0.98	kPa
Rain load	R =	0.02	kPa

The resulting interanl moments from the modelling is presented as follows:

Ultimate moment for reinforcement in x-direction	$M_{u,y}$ =	8297340.00	Nmm/m
Ultimate moment for reinforcement in y-direction	$M_{u,x}$ =	6065470.00	Nmm/m

E. SERVICABILITY OF THE DESIGN

The structure shall be evaluated for the service condition, namely in terms of the deflection, which consists of the short term and the long term. The short term deflection is defined as the deflection which occurs when the live load applied to the structure, while the long term deflection is the deflection which occurs due to all applicable loads designated for the structure which may include the dead loads, additional dead loads, non-structural loads and the live load.

The allowable magnitude of deflections is regulated in SNI 2847:2019. For the roof, the allowable short term deflection is ℓ/180, and for the long term is ℓ/240.

The occuring deflections are computed in SAP2000 from the structural model. And the deflection is considered sufficient if

$$\frac{\delta_s}{\delta_{sa}}, \frac{\delta_l}{\delta_{la}} \leq 1.00$$

is satisfied, where δ_s, δ_l, δs_a, δ_{la} are the short term, long term and the allowable deflections.

Slab span	ℓ =	2250.00	mm
Short term deflection	δ_s =	0.27	mm
Allowable short term deflection	δ_{sa} =	12.50	mm
Short term deflection ratio	$r_{\delta s}$ =	0.02	
Long term deflection	δ_l =	1.30	mm
Allowable long term deflection	δ_{la} =	9.38	mm
Long term deflection ratio	$r_{\delta l}$ =	0.14	

CONCLUSION: SATISFIED

F. REINFORCEMENT DESIGN

The reinforcements to be considered in the desing include the moment-resisting reinforcement and the shrinkage as well as temperature distributing reinforcement. The evaluations on the reinforcement include the ductility requirement and the moment capacity of the reinforcement.

For the ductility requirement, the reinforcement in use shall satisfy

$$A_{s,\min} \le A_s \le A_{s,\max}$$

where $A_{s,min}$ is the minimum required reinforcement and is given in accordance with SNI 2847:2019 sectioin 8.6.1.1 by

$$A_{s,\min} = \begin{cases} 0.0020 & : f_y < 420 \text{ MPa} \\ \max\left\{\frac{0.0018\cdot 420}{f_y}A_g\,,\ 0.0014A_g\right\} & : f_y \ge 420 \text{ MPa} \end{cases},$$

and, on the other hand, $A_{s,max}$ is given by

$$A_{s,\max} = 0.75\rho_b A_g$$

with

$$\rho_b = \frac{0.85 f_c' \beta_1}{f_y}\frac{600}{f_y + 600}.$$

And the nominal moment of the reinforcement is given by

$$M_n = f_y A_s \left(d - \frac{a}{2}\right),$$

where

$$a = \frac{f_y A_s}{0.85 f_c' b}$$

which is obtained from the force couple theorem. Then, the reinforcement is sufficient if

$$r_b := \frac{M_u}{\phi M_n} \le 1.00$$

is satisfied, where ϕ = 0.9 according to SNI 2847:2019.

F.1. Moment-Resisting Reinforcement

Slab thickness	t_p =	120.00	mm
Rebar diameter	D =	8.00	mm
Concrete cover	cov =	20.00	mm
Distance from extreme compresive fibre to tensile rebars	d =	96.00	mm
Distance of adjacent rebars	s =	150.00	mm

Ductility Control:

Cross sectional area of rebars in use	A_s =	335.10	mm^2/m
Gross sectional area of slab	A_g =	120000.00	mm^2/m

Minimum required rebars area	$A_{s,min}$ =	185.14	mm^2/m
Balance rebars ratio	ρ_b =	0.02	
Maximum required rebars	$A_{s,max}$ =	2100.59	mm^2/m

CONCLUSION: SATISFIED

Moment Capacity Control:

Equivalent rectangular height of compresive concrete fibre	a =	6.66	mm
Nominal moment	M_n =	15216362.21	Nmm/m
Strength ratio	r_b =	0.55	

CONCLUSION: SATISFIED

F.2. Shrinkage and Temperature Reinforcement

The requirement of the shrinkage and temperature reinforcement is that it shall satisfied the minimum rebars requirement, which is similar to that of the moment-resisting rebars. In addition, the distance of adjacent rebars cannot exceeds the minimum of $4t_p$ and 450 mm according to SNI 2847:2019 section 24.4.3.3.

We will also check the reinforcement capacity against the ultimate moment in this direction, even though it may not be critical.

Slab thickness	t_p =	120.00	mm
Rebar diameter	D =	8.00	mm
Concrete cover	cov =	20.00	mm
Distance from extreme compresive fibre to tensile rebars	d =	96.00	mm

Shrinkage and Temperature Reinforcement Requirement:

Distance of adjacent rebars	s =	150.00	mm
Cross sectional area of rebars in use	A_s =	335.10	mm^2/m
Gross sectional area of slab	A_g =	120000.00	mm^2/m
Minimum required rebars area	$A_{s,min}$ =	185.14	mm^2/m
Balance rebars ratio	ρ_b =	0.02	
Maximum required rebars	$A_{s,max}$ =	2100.59	mm^2/m

CONCLUSION: SATISFIED

Moment Capacity Control:

Equivalent rectangular height of compresive concrete fibre	a =	6.66	mm
Nominal moment	M_n =	15216362.21	Nmm/m
Strength ratio	r_b =	0.40	

CONCLUSION: SATISFIED

Design of Concrete Slab S1 (2)

A. INTRODUCTION

This document contains the design and analysis of concrete slab structure. It will covers the determination of the minimum slab thickness, reinforcement design and evaluation against the design forces. The computation of the design forces on the slab is conducted separately using SAP2000.

B. GENERAL GEOMETRIC AND MECHANICAL PROPERTIES

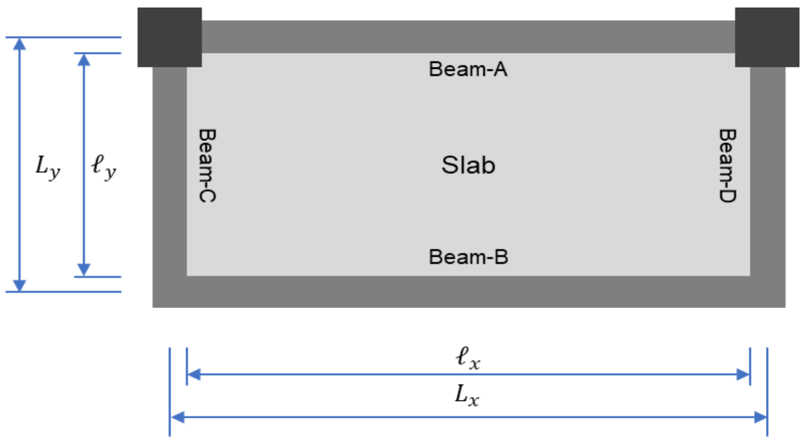
Concrete strength in use	f_c' =	29.00	MPa
Whitney factor corresponding the concrete strength:	β_1 =	0.84	

(SNI 2847:2019 table 22.2.2.4.3)

$$\beta_1 = \begin{cases} 0.85 & : 17\ \mathrm{MPa} \le f_c' \le 28\ \mathrm{MPa} \\ 0.85 - \frac{0.05(f_c'-28)}{7} & : 28\ \mathrm{MPa} < f_c' < 55\ \mathrm{MPa} \\ 0.65 & : f_c' \ge 55\ \mathrm{MPa} \end{cases}$$

Reinforcement code: **WIRE MESH M8**

Reinforcing bars strength in use	f_y =	490.00	MPa
Main rebar diameter	D =	8.00	mm
Secondary (Shrinkage and temperature) rebar diameter	D_t =	8.00	mm
Distance of adjacent main rebars	s_1 =	150.00	mm
Distance of adjacent secondary rebars	s_2 =	150.00	mm



Width of beams surrounding the slab:

Beam-A	b_A =	300.00	mm
Beam-B	b_B =	200.00	mm
Beam-C	b_C =	300.00	mm
Beam-D	b_D =	300.00	mm

Height of beams surrounding the slab:

Beam-A	h_A =	400.00	mm
Beam-B	h_B =	300.00	mm
Beam-C	h_C =	400.00	mm

Beam-D	h_D =	400.00	mm
Slab length axis-to-axis	ℓ_x =	4500.00	mm
Slab width axis-to-axis	ℓ_y =	1500.00	mm
Clear slab length	ℓ_{nx} =	4250.00	mm
Clear slab width	ℓ_{ny} =	1200.00	mm

C. SLAB THICKNESS

The minimum thickness of the slab shall comply with the requirement in SNI 2847:2019 section 8.3.1. And the chosen thickness of the slab shall be sufficient aginst the design loads on th slab as demonstrated later.

Slab length to width ratio: β = 3.54

$$\beta = \frac{\max\{\ell_x, \ell_y\}}{\min\{\ell_x, \ell_y\}}$$

Type of slab: ONE-WAY

For a one-way slab, the minimum required thickness is given in accordane with SNI 2847:2029 table 7.3.1.1 as follows:

Minimum required thickness	$t_{p.min}$ =	53.57	mm
Slab thickness in use	t_p =	120.00	mm

D. DESIGN LOADS ON THE SLAB

The values for the design loads on the slab are determined from the computation using SAP2000. The required additional dead load and the live load are presented as follows:

Additional Dead Loads:			
Screeding or moratar overlay (0.5 cm)	SDL =	0.12	kPa
Roof Live Loads:	RL =	0.98	kPa
Rain load	R =	0.02	kPa

The resulting interanl moments from the modelling is presented as follows:

Ultimate moment for reinforcement in x-direction	$M_{u,y}$ =	5773680.00	Nmm/m
Ultimate moment for reinforcement in y-direction	$M_{u,x}$ =	0.00	Nmm/m

E. SERVICABILITY OF THE DESIGN

The structure shall be evaluated for the service condition, namely in terms of the deflection, which consists of the short term and the long term. The short term deflection is defined as the deflection which occurs when the live load applied to the structure, while the long term deflection is the deflection which occurs due to all applicable loads designated for the structure which may include the dead loads, additional dead loads, non-structural loads and the live load.

The allowable magnitude of deflections is regulated in SNI 2847:2019. For the roof, the allowable short term deflection is ℓ/180, and for the long term is ℓ/240.

The occuring deflections are computed in SAP2000 from the structural model. And the deflection is considered sufficient if

$$\frac{\delta_s}{\delta_{sa}}, \frac{\delta_l}{\delta_{la}} \leq 1.00$$

is satisfied, where δ_s, δ_l, δs_a, δ_{la} are the short term, long term and the allowable deflections.

Slab span	ℓ =	1500.00	mm
Short term deflection	δ_s =	0.12	mm
Allowable short term deflection	δ_{sa} =	8.33	mm
Short term deflection ratio	$r_{\delta s}$ =	0.01	
Long term deflection	δ_l =	1.00	mm
Allowable long term deflection	δ_{la} =	6.25	mm
Long term deflection ratio	$r_{\delta l}$ =	0.16	

CONCLUSION: SATISFIED

F. REINFORCEMENT DESIGN

The reinforcements to be considered in the desing include the moment-resisting reinforcement and the shrinkage as well as temperature distributing reinforcement. The evaluations on the reinforcement include the ductility requirement and the moment capacity of the reinforcement.

For the ductility requirement, the reinforcement in use shall satisfy

$$A_{s,\min} \leq A_s \leq A_{s,\max}$$

where $A_{s,min}$ is the minimum required reinforcement and is given in accordance with SNI 2847:2019 sectioin 8.6.1.1 by

$$A_{s,\min} = \begin{cases} 0.0020 & : f_y < 420 \text{ MPa} \\ \max\left\{\frac{0.0018 \cdot 420}{f_y} A_g \,,\ 0.0014 A_g\right\} & : f_y \geq 420 \text{ MPa} \end{cases},$$

and, on the other hand, $A_{s,max}$ is given by

$$A_{s,\max} = 0.75 \rho_b A_g$$

with

$$\rho_b = \frac{0.85 f'_c \beta_1}{f_y} \frac{600}{f_y + 600}.$$

And the nominal moment of the reinforcement is given by

$$M_n = f_y A_s \left(d - \frac{a}{2}\right),$$

where

$$a = \frac{f_y A_s}{0.85 f'_c b}$$

which is obtained from the force couple theorem. Then, the reinforcement is sufficient if

$$r_b := \frac{M_u}{\phi M_n} \leq 1.00$$

is satisfied, where ϕ = 0.9 according to SNI 2847:2019.

F.1. Moment-Resisting Reinforcement

Slab thickness	t_p =	120.00	mm
Rebar diameter	D =	8.00	mm
Concrete cover	cov =	20.00	mm
Distance from extreme compresive fibre to tensile rebars	d =	96.00	mm
Distance of adjacent rebars	s =	150.00	mm

Ductility Control:

Cross sectional area of rebars in use	A_s =	335.10	mm^2/m
Gross sectional area of slab	A_g =	120000.00	mm^2/m

Minimum required rebars area	$A_{s,min}$ =	185.14	mm^2/m
Balance rebars ratio	ρ_b =	0.02	
Maximum required rebars	$A_{s,max}$ =	2100.59	mm^2/m

CONCLUSION: SATISFIED

Moment Capacity Control:

Equivalent rectangular height of compresive concrete fibre	a =	6.66	mm
Nominal moment	M_n =	15216362.21	Nmm/m
Strength ratio	r_b =	0.38	

CONCLUSION: SATISFIED

F.2. Shrinkage and Temperature Reinforcement

The requirement of the shrinkage and temperature reinforcement is that it shall satisfied the minimum rebars requirement, which is similar to that of the moment-resisting rebars. In addition, the distance of adjacent rebars cannot exceeds the minimum of $4t_p$ and 450 mm according to SNI 2847:2019 section 24.4.3.3.

We will also check the reinforcement capacity against the ultimate moment in this direction, even though it may not be critical.

Slab thickness	t_p =	120.00	mm
Rebar diameter	D =	8.00	mm
Concrete cover	cov =	20.00	mm
Distance from extreme compresive fibre to tensile rebars	d =	96.00	mm

Shrinkage and Temperature Reinforcement Requirement:

Distance of adjacent rebars	s =	150.00	mm
Cross sectional area of rebars in use	A_s =	335.10	mm^2/m
Gross sectional area of slab	A_g =	120000.00	mm^2/m
Minimum required rebars area	$A_{s,min}$ =	185.14	mm^2/m
Balance rebars ratio	ρ_b =	0.02	
Maximum required rebars	$A_{s,max}$ =	2100.59	mm^2/m

CONCLUSION: SATISFIED

Moment Capacity Control:

Equivalent rectangular height of compresive concrete fibre	a =	6.66	mm
Nominal moment	M_n =	15216362.21	Nmm/m
Strength ratio	r_b =	0.00	

CONCLUSION: SATISFIED

Design of Concrete Slab S1 (3)

A. INTRODUCTION

This document contains the design and analysis of concrete slab structure. It will covers the determination of the minimum slab thickness, reinforcement design and evaluation against the design forces. The computation of the design forces on the slab is conducted separately using SAP2000.

B. GENERAL GEOMETRIC AND MECHANICAL PROPERTIES

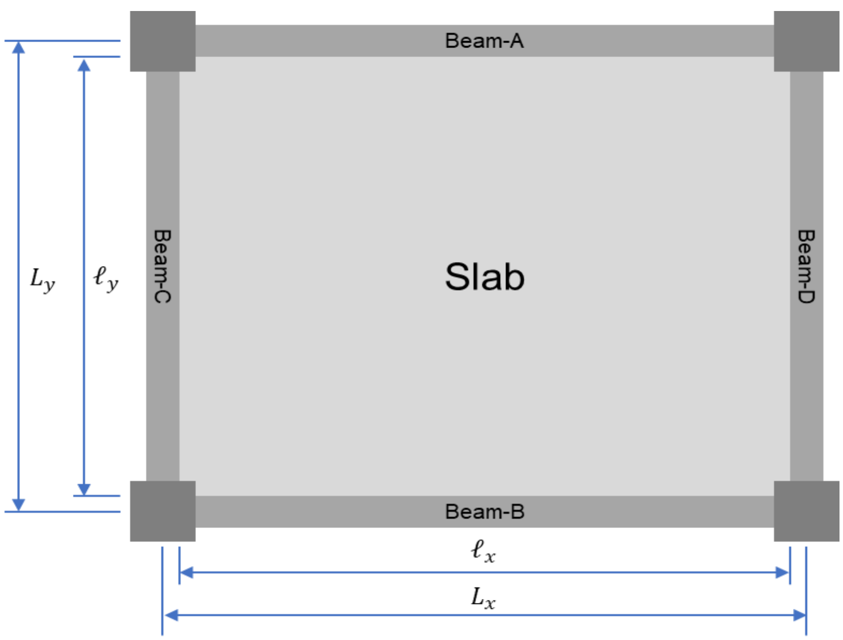
Concrete strength in use	f_c' =	29.00	MPa
Whitney factor corresponding the concrete strength:	β_1 =	0.84	

(SNI 2847:2019 table 22.2.2.4.3)

$$\beta_1 = \begin{cases} 0.85 & : 17\ \mathrm{MPa} \le f_c' \le 28\ \mathrm{MPa} \\ 0.85 - \frac{0.05(f_c'-28)}{7} & : 28\ \mathrm{MPa} < f_c' < 55\ \mathrm{MPa} \\ 0.65 & : f_c' \ge 55\ \mathrm{MPa} \end{cases}$$

Reinforcement code: **WIRE MESH M8**

Reinforcing bars strength in use	f_y =	490.00	MPa
Rebar diameter in x-direction	D_x =	8.00	mm
Secondary (Shrinkage and temperature) rebar diameter	D_y =	8.00	mm
Distance of adjacent main rebars	s_x =	150.00	mm
Distance of adjacent secondary rebars	s_y =	150.00	mm



Width of beams surrounding the slab:

Beam-A	b_A =	200.00	mm
Beam-B	b_B =	300.00	mm
Beam-C	b_C =	200.00	mm

Beam-D	b_D =	300.00	mm
Height of beams surrounding the slab:			
Beam-A	h_A =	300.00	mm
Beam-B	h_B =	400.00	mm
Beam-C	h_C =	300.00	mm
Beam-D	h_D =	400.00	mm
Slab width axis-to-axis	ℓ_x =	1500.00	mm
Slab length axis-to-axis	ℓ_y =	1500.00	mm

C. SLAB THICKNESS

The minimum thickness of the slab shall comply with the requirement in SNI 2847:2019 section 8.3.1. And the chosen thickness of the slab shall be sufficient aginst the design loads on th slab as demonstrated later.

Clear slab width	ℓ_{nx} =	1250.00	mm
Clear slab length	ℓ_{ny} =	1250.00	mm
Slab length to width ratio:	β =	1.00	

$$\beta = \frac{\max\{\ell_x, \ell_y\}}{\min\{\ell_x, \ell_y\}}$$

Preliminary slab thinckness in use	$t_{p.prel}$ =	100.00	mm

Beam-to-slab flexural stiffness ratio:

$$\forall X \in \{A, B, C, D\} : \ \alpha_X := \frac{E_{bX} I_{bX}}{E_{sX} I_{sX}}$$

$$\forall X \in \{A, B, C, D\} : \ E_{bX} = E_{sX} = 4700\sqrt{f'_c}$$

$$\forall X \in \{A, B, C, D\} : \ I_{bX} = \frac{b_X h_X^3}{12}$$

$$\forall X \in \{A, B, C, D\} : \ I_{sX} = \frac{w_X t_p^4}{12}$$

Young's modulus for both slabs and beams	E_c =	25310.27	MPa
Beams moment of inertia:			
	I_{bA} =	450000000.00	mm^4
	I_{bB} =	1600000000.00	mm^4
	I_{bC} =	450000000.00	mm^4
	I_{bD} =	1600000000.00	mm^4
Slab moment of inertia (based on beams)			
	I_{sA} =	125000000.00	mm^4
	I_{sB} =	125000000.00	mm^4
	I_{sC} =	125000000.00	mm^4
	I_{sD} =	125000000.00	mm^4
Beam-to-slab stiffness ratio:			
	α_{fA} =	3.60	
	α_{fB} =	12.80	
	α_{fC} =	3.60	
	α_{fD} =	12.80	
Mean stiffness ratio:	α_{fm} =	8.20	

Minimum required slab thickness according to SNI 2847:2019 table 8.3.1.2.1:

$$t_{p,\min} = \begin{cases} \max\left\{ \frac{\ell_n\left(0.8+\frac{f_y}{1000}\right)}{36+5\beta(\alpha_{fm}-0.2)},\ 125 \right\} & : 0.2 < \alpha_{fm} \leq 2.0 \\ \max\left\{ \frac{\ell_n\left(0.8+\frac{f_y}{1400}\right)}{36+9\beta},\ 90 \right\} & : \alpha_{fm} > 2.0 \end{cases}$$

	$t_{p,min}$ =	90.00	mm
Slab thickness in use	t_p =	120.00	mm

CONCLUSION: SATISFIED

D. DESIGN LOADS ON THE SLAB

The values for the design loads on the slab are determined from the computation using SAP2000. The required additional dead load and the live load are presented as follows:

Additional Dead Loads:

Screeding or moratar overlay (1 cm)	SDL =	0.24	kPa
Roof Live Loads:	RL =	0.98	kPa
Rain load	R =	0.02	kPa

The resulting interanl moments from the modelling is presented as follows:

Ultimate moment for reinforcement in x-direction	$M_{u,y}$ =	3031850.00	Nmm/m
Ultimate moment for reinforcement in y-direction	$M_{u,x}$ =	3051610.00	Nmm/m

E. SERVICABILITY OF THE DESIGN

The structure shall be evaluated for the service condition, namely in terms of the deflection, which consists of the short term and the long term. The short term deflection is defined as the deflection which occurs when the live load applied to the structure, while the long term deflection is the deflection which occurs due to all applicable loads designated for the structure which may include the dead loads, additional dead loads, non-structural loads and the live load.

The allowable magnitude of deflections is regulated in SNI 2847:2019. For the roof, the allowable short term deflection is ℓ/180, and for the long term is ℓ/240.

The occuring deflections are computed in SAP2000 from the structural model. And the deflection is considered sufficient if

$$\frac{\delta_s}{\delta_{sa}},\ \frac{\delta_l}{\delta_{la}} \leq 1.00$$

is satisfied, where δ_s, δ_l, δs_a, δ_{la} are the short term, long term and the allowable deflections.

Slab span	ℓ =	1500.00	mm
Short term deflection	δ_s =	0.03	mm
Allowable short term deflection	δ_{sa} =	8.33	mm
Short term deflection ratio	$r_{\delta s}$ =	0.00	
Long term deflection	δ_l =	0.67	mm
Allowable long term deflection	δ_{la} =	6.25	mm
Long term deflection ratio	$r_{\delta l}$ =	0.11	

CONCLUSION: SATISFIED

F. REINFORCEMENT DESIGN

The reinforcements to be considered in the desing include the moment-resisting reinforcements in both x and y directions. The evaluations on the reinforcement include the ductility requirement and the moment capacity of the reinforcement.

For the ductility requirement, the reinforcement in use shall satisfy

$$A_{s,\min} \leq A_s \leq A_{s,\max}$$

where $A_{s,min}$ is the minimum required reinforcement and is given in accordance with SNI 2847:2019 sectioin 8.6.1.1 by

$$A_{s,\min} = \begin{cases} 0.0020 & : f_y < 420 \text{ MPa} \\ \max\left\{\frac{0.0018 \cdot 420}{f_y} A_g \,,\ 0.0014 A_g\right\} & : f_y \geq 420 \text{ MPa} \end{cases} ,$$

and, on the other hand, $A_{s,max}$ is given by

$$A_{s,\max} = 0.75 \rho_b A_g$$

with

$$\rho_b = \frac{0.85 f'_c \beta_1}{f_y} \frac{600}{f_y + 600} .$$

And the nominal moment of the reinforcement is given by

$$M_n = f_y A_s \left(d - \frac{a}{2}\right) ,$$

where

$$a = \frac{f_y A_s}{0.85 f'_c b}$$

which is obtained from the force couple theorem. Then, the reinforcement is sufficient if

$$r_b := \frac{M_u}{\phi M_n} \leq 1.00$$

is satisfied, where $\phi = 0.9$ according to SNI 2847:2019.

F.1. Reinforcement in X-Direction

Slab thickness	t_p =	120.00	mm
Rebar diameter	D =	8.00	mm
Concrete cover	cov =	20.00	mm
Distance from extreme compresive fibre to tensile rebars	d =	96.00	mm
Distance of adjacent rebars	s =	150.00	mm
Ductility Control:			
Cross sectional area of rebars in use	A_s =	335.10	mm^2/m
Gross sectional area of slab	A_g =	120000.00	mm^2/m
Minimum required rebars area	$A_{s,min}$ =	185.14	mm^2/m
Balance rebars ratio	ρ_b =	0.02	
Maximum required rebars	$A_{s,max}$ =	2100.59	mm^2/m
CONCLUSION: SATISFIED			
Moment Capacity Control:			
Equivalent rectangular height of compresive concrete fibre	a =	6.66	mm
Nominal moment	M_n =	15216362.21	Nmm/m
Strength ratio	r_b =	0.20	
CONCLUSION: SATISFIED			

F.2. Reinforcement in Y-Direction

Slab thickness	t_p =	120.00	mm
Rebar diameter	D =	8.00	mm
Concrete cover	cov =	20.00	mm
Distance from extreme compresive fibre to tensile rebars	d =	96.00	mm
Distance of adjacent rebars	s =	150.00	mm
Ductility Control:			
Cross sectional area of rebars in use	A_s =	335.10	mm^2/m
Gross sectional area of slab	A_g =	120000.00	mm^2/m
Minimum required rebars area	$A_{s,min}$ =	185.14	mm^2/m
Balance rebars ratio	ρ_b =	0.02	
Maximum required rebars	$A_{s,max}$ =	2100.59	mm^2/m
CONCLUSION: SATISFIED			
Moment Capacity Control:			
Equivalent rectangular height of compresive concrete fibre	a =	6.66	mm
Nominal moment	M_n =	15216362.21	Nmm/m
Strength ratio	r_b =	0.20	
CONCLUSION: SATISFIED			

Design of Concrete Slab S2 (Slab on Ground)

A. INTRODUCTION

This document contains the design and analysis of concrete slab structure on ground. It will covers the determination of the minimum slab thickness, reinforcement design and evaluation against the design forces. The computation of the design forces on the slab is conducted separately using SAP2000. The slab will be modelled as a shell in SAP2000 supported by area springs which represent the soil stiffness underneath the slab.

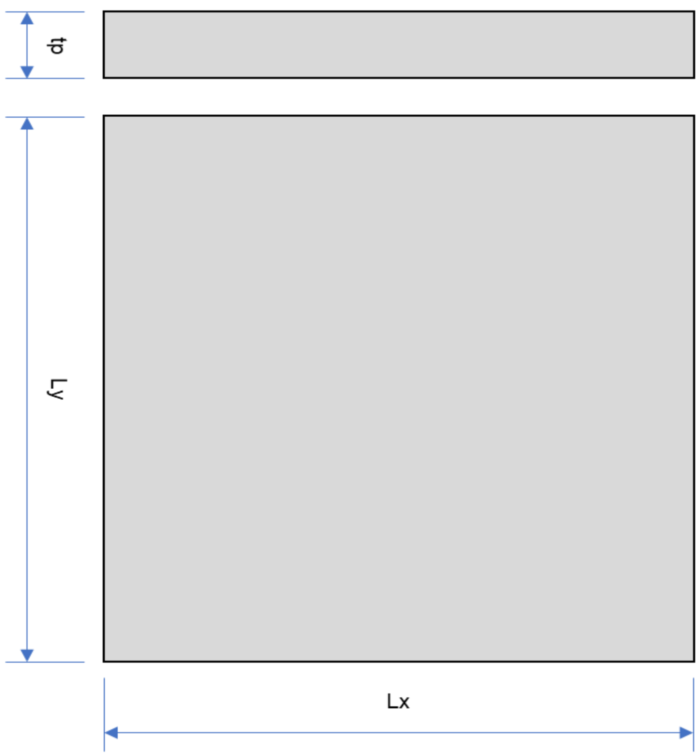
B. PRELIMINARY DESIGN AND INFORMATION

For the preliminary estimation, we use the following specifications:

Concrete strength in use	f_c' =	29.00	MPa
Whitney factor corresponding the concrete strength:	β_1 =	0.84	

(SNI 2847:2019 table 22.2.2.4.3)

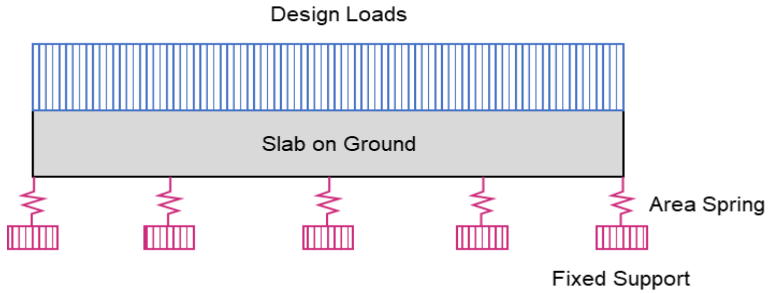
$$\beta_1 = \begin{cases} 0.85 & : 17 \text{ MPa} \leq f_c' \leq 28 \text{ MPa} \\ 0.85 - \frac{0.05(f_c'-28)}{7} & : 28 \text{ MPa} < f_c' < 55 \text{ MPa} \\ 0.65 & : f_c' \geq 55 \text{ MPa} \end{cases}$$



Width of slab	L_x =	4500.00	mm
Length of slab	L_y =	4500.00	mm
Thickness of slab in use	t_p =	200.00	mm

C. MODEL OF SLAB

The slab is modelled in SAP2000 as a shell element supported by area springs. The area springs represent the stiffness of soil underneath the slab. And illustration is presented below.



C.1. Area Spring

The area spring is computed in accordance with (Das, 2011) and is given by

$$k_s = \frac{E_s}{B(1 - \nu_s^2)} .$$

The computation of the area spring is presented as follows:

Expected Young's modulus of soil	E_s =	11.97	MPa
Soil Poisson's ratio	ν_s =	0.25	
Foundation width	B =	4500.00	mm
Area spring (modulus of subgrade reaction)	k_s =	0.0028	$N/mm/mm^2$

C.2. Design Loads

The design loads to be used involve dead load, live load, and seismic load, which are presented as follows:

Dead Loads:

	Slab self weight		4.80	kPa
	Floor finish and others		0.79	kPa
	Total dead load	D =	5.59	kPa
Live Load	(100 kg/m2 according DBS-1-000-C-0001 table 35)	L =	4.91	kPa

The seismic load uses response spectrum analysis with the following parameters:

Risk Category		II	
Importance Factor	I_e =	1.000	
Site class		SE	
Spectral acceleration at t = 0.2 s	S_s =	0.578	g
Spectral acceleration at t = 1.0 s	S_1 =	0.179	g
Site coefficient for short period of 0.2 s	F_a =	1.543	
Site coefficient for long period	F_v =	4.981	
Spectral acc. parameter of short period with 5% damping	S_{DS} =	0.595	g
Spectral acceleration parameter at 1 s	S_{D1} =	0.594	g
Long period transtition	T_L =	6.000	
(The following parameters are from SNI 1726:2019 table 29)			
Response coefficient modification	R =	1.500	
Overstrength factor	Ω_0 =	1.500	
Deflection factor	C_d =	1.500	
Seismic response coefficient $C_s = \frac{I_e S_{DS}}{R}$	C_s =	0.396	
Response scale factor for SAP2000 model	$g\ I_e/R$ =	6540.000	mm/s^2

Other seimic load components are provided as follows:

$$E_h = \rho Q_E \,, E_{mh} = \Omega_0 Q_E \,,\ E_v = 0.2\, S_{DS}\, D$$

And the load combinations in use are given as follows:

LRFD Load Combinations:

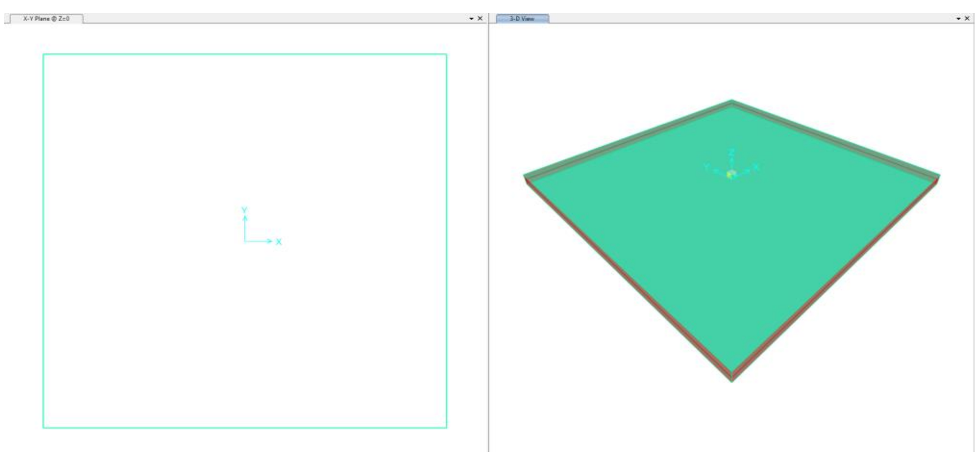
1 1.4D
2 1.2D + 1.6L
3 1.2D + Ev + Ehx + L
4 1.2D + Ev + Ehy + L
5 0.9D -Ev + Ehx
6 0.9D -Ev + Ehy
7 1.2D + Ev + Emhx + L
8 1.2D + Ev + Emhy + L
9 0.9D -Ev + Emhx
10 0.9D -Ev + Emhy

ASD Load Combinations

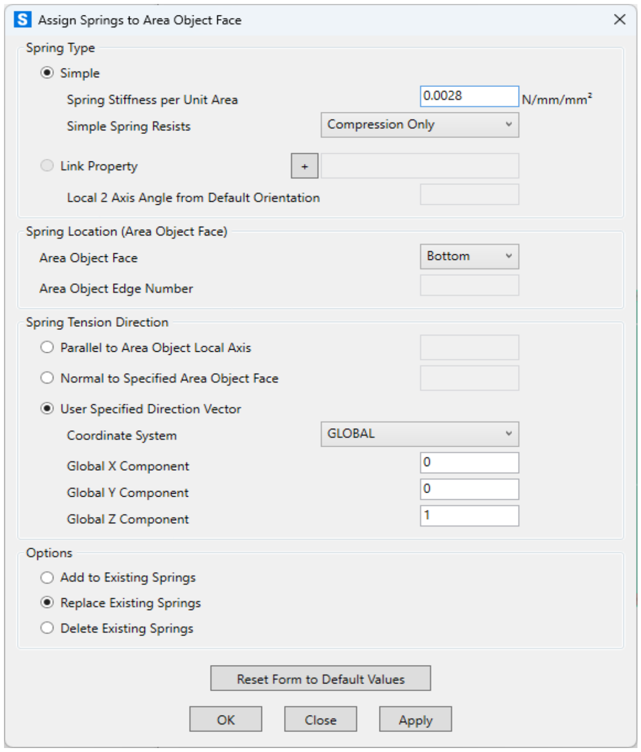
1 D
2 D + L
3 1.0D + 0.7Ev + 0.7Ehx
4 1.0D + 0.7Ev + 0.7Ehy
5 1.0D + 0.525Ev + 0.525Ehx + 0.75L
6 1.0D + 0.525Ev + 0.525Ehy + 0.75L
7 0.6D -0.7Ev + 0.7Ehx
8 0.6D -0.7Ev + 0.7Ehy
9 1.0D + 0.525Ev + 0.525Emhx + 0.75L
10 1.0D + 0.525Ev + 0.525Emhy + 0.75L
11 0.6D -0.7Ev + 0.7Emhx
12 0.6D -0.7Ev + 0.7Emhy

C.3. SAP2000 Model

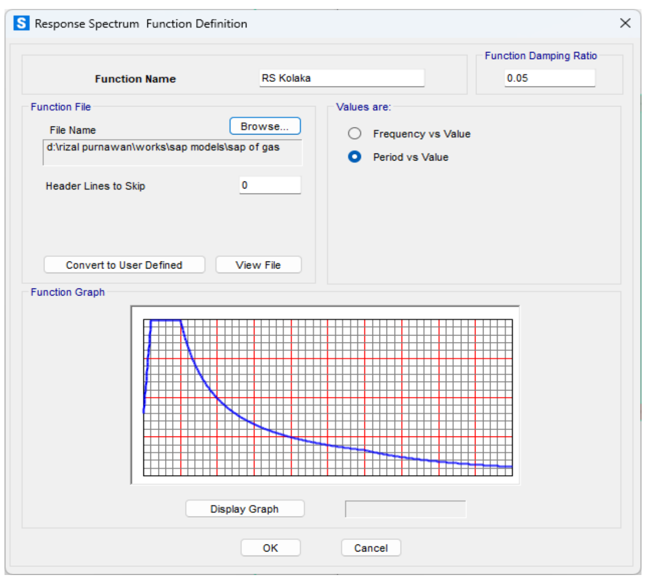
The structural model of the slab is presented as follows:



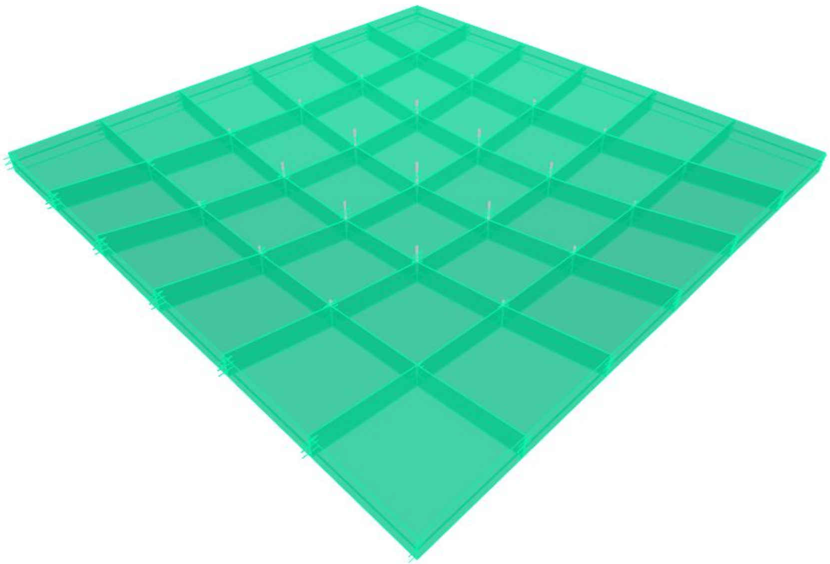
The setting of the area spring is presented as follows:



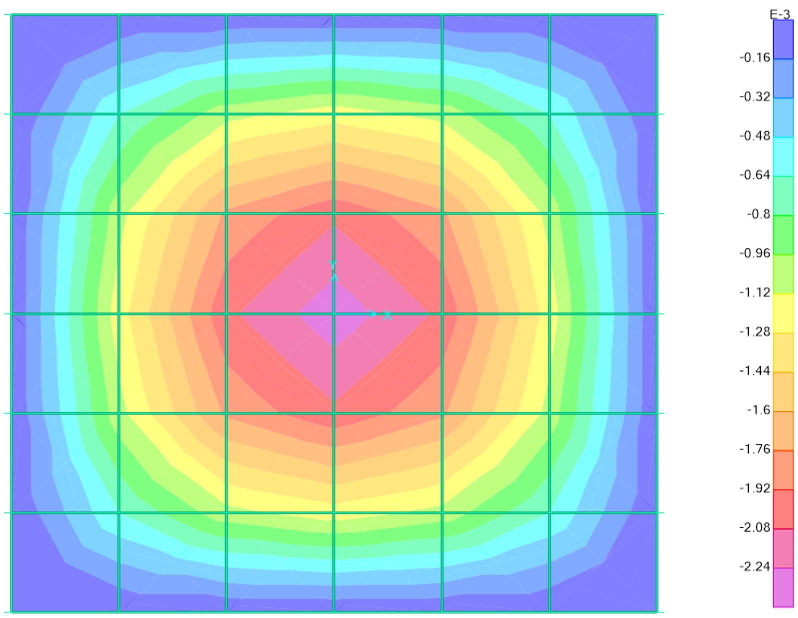
The generated response spectrum is presented as follows:



Deformed shape of the slab due to 1.0D + 1.0L:



Soil pressure due to 1.0D + 1.0L:



D. REINFORCEMENT DESIGN

The reinforcements to be considered in the desing include the moment-resisting reinforcements in both x and y directions. The evaluations on the reinforcement include the ductility requirement and the moment capacity of the reinforcement.

For the ductility requirement, the reinforcement in use shall satisfy

$$A_{s,\min} \leq A_s \leq A_{s,\max}$$

where $A_{s,min}$ is the minimum required reinforcement and is given in accordance with SNI 2847:2019 sectioin 8.6.1.1 by

$$A_{s,\min} = \begin{cases} 0.0020 & : f_y < 420 \text{ MPa} \\ \max\left\{\frac{0.0018\cdot 420}{f_y} A_g \,,\ 0.0014 A_g\right\} & : f_y \geq 420 \text{ MPa} \end{cases},$$

and, on the other hand, $A_{s,max}$ is given by

$$A_{s,\max} = 0.75 \rho_b A_g$$

with

$$\rho_b = \frac{0.85 f'_c \beta_1}{f_y} \frac{600}{f_y + 600}.$$

And the nominal moment of the reinforcement is given by

$$M_n = f_y A_s \left(d - \frac{a}{2}\right),$$

where

$$a = \frac{f_y A_s}{0.85 f'_c b}$$

which is obtained from the force couple theorem. Then, the reinforcement is sufficient if

$$r_b := \frac{M_u}{\phi M_n} \leq 1.00$$

is satisfied, where $\phi = 0.9$ according to SNI 2847:2019.

D.1. Reinforcement in X-Direction

Rebar yield strength	f_y =	490.00	MPa
Slab thickness	t_p =	200.00	mm
Rebar diameter	D =	10.00	mm
Concrete cover	cov =	50.00	mm
Distance from extreme compresive fibre to tensile rebars	d =	145.00	mm
Distance of adjacent rebars	s =	150.00	mm
Ductility Control:			
Cross sectional area of rebars in use	A_s =	523.60	mm^2/m
Gross sectional area of slab	A_g =	200000.00	mm^2/m
Minimum required rebars area	$A_{s,min}$ =	400.00	mm^2/m
Balance rebars ratio	ρ_b =	0.02	
Maximum required rebars	$A_{s,max}$ =	3500.99	mm^2/m
CONCLUSION: **SATISFIED**			
Moment Capacity Control:			
Equivalent rectangular height of compresive concrete fibre	a =	10.41	mm
Nominal moment	M_n =	35866504.81	Nmm/m
Ultimate moment	M_u =	11010010.00	Nmm/m
Strength ratio	r_b =	0.34	
CONCLUSION: **SATISFIED**			

D.2. Reinforcement in Y-Direction

Rebar yield strength	f_y =	490.00	MPa
Slab thickness	t_p =	200.00	mm
Rebar diameter	D =	10.00	mm
Concrete cover	cov =	50.00	mm

Distance from extreme compresive fibre to tensile rebars	d =	145.00	mm
Distance of adjacent rebars	s =	150.00	mm
Ductility Control:			
Cross sectional area of rebars in use	A_s =	523.60	mm^2/m
Gross sectional area of slab	A_g =	200000.00	mm^2/m
Minimum required rebars area	$A_{s,min}$ =	400.00	mm^2/m
Balance rebars ratio	ρ_b =	0.02	
Maximum required rebars	$A_{s,max}$ =	3500.99	mm^2/m
CONCLUSION: SATISFIED			
Moment Capacity Control:			
Equivalent rectangular height of compresive concrete fibre	a =	10.41	mm
Nominal moment	M_n =	35866504.81	Nmm/m
Ultimate moment	M_u =	11010010.00	Nmm/m
Strength ratio	r_b =	0.34	
CONCLUSION: SATISFIED			

E. REFERENCES

Das, Braja M. (2011). *Principles of Foundation Engineering* . Stamford, US: Cengage Learning.

ATTACHMENT 3

Main Structure Foundation Stability and Reinforcement

ANALYSIS OF FOUNDATION F1

A. INTRODUCTION

This document covers the design and analysis of a shallow fondation. The aspects to be analysed in the design of shallow foundations include the bearing capacity, overturning and sliding stability. These tree aspects are the common probable failure modes of shallow foundations. All the theory in geotechnics in use are from (Das, 2011).

Geometry of Foundation:

Foundation width in x-direction	B_x =	2.10	m
Foundation width in y-direction	B_y =	2.10	m
Footing thickness	t_p =	0.40	m
Depth of foundation	D_f =	2.00	m
Column width in x-direction	$B_{c\text{-}x}$ =	0.35	m
Column width in y-direction	$B_{c\text{-}y}$ =	0.35	m

Soil Mechanical Properties:

Borehole reference	BH.2.1.04		
Unit weight	γ =	16.37	kN/m^3
Effective interal friction angle	ϕ' =	1.09	deg
Soil cohession	c' =	40.10	kPa

B. BEARING CAPACITY

For the bearing capacity, we have computed the soil stress due to bearing loads and the moments from upper structure separately using SAP2000. And the soil stress bearing capacity has already been separately analysed in the soil report. The results are presented as follows.

Non-overstrength soil stress	q =	89.00	kPa
Overstrength soil stress	q_{OS} =	115.00	kPa
Soil bearing capacity (soil report)	q_{all} =	97.00	kPa
Increased bearing capacity due to overstrength	1.2 q_{all} =	116.40	kPa

The increase factor 1.2 is accordance with SNI 1726:2019 section 4.2.3.3.

The bearing capacity is satisfied if the following expressions hold:

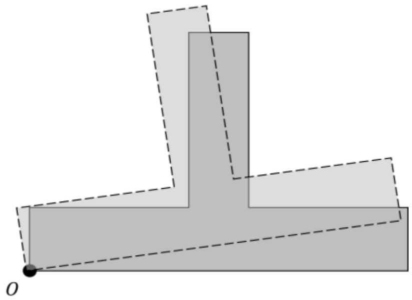
$$r_q := \frac{q}{q_{all}}\ ,\ r_{q.os} := \frac{q_{os}}{1.2q_{all}} \leq 1$$

Stress ratio	r_q =	0.92
Stress ratio with overstrength	$r_{q.os}$ =	0.99

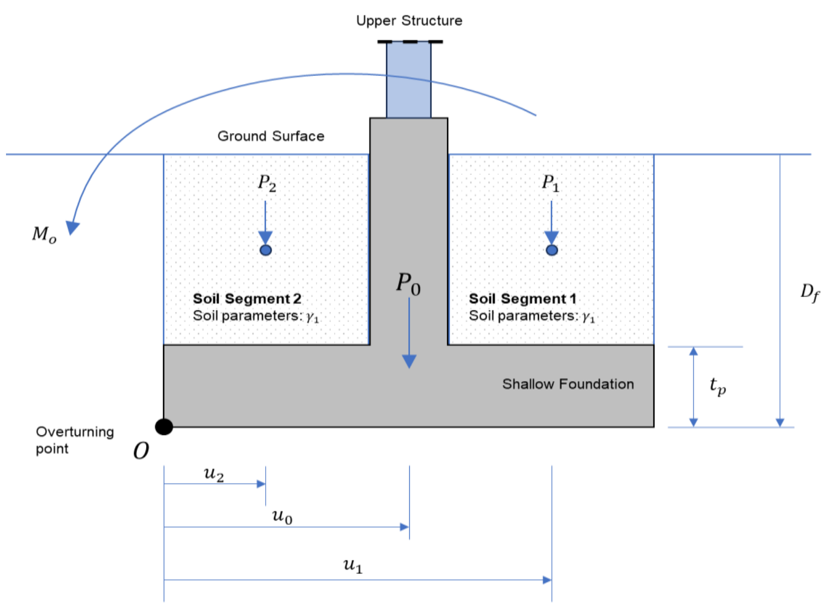
CONCLUSION: SATISFIED

C. OVERTURNING STABILITY

Overturning is another probable phenomenen in a shallow foundation. It is defined as the rotation of the foundation one of its toes (edges). Now let us observe the following illustration.



The measure to be considered in analysing the overturning stability is the ratio of the counter balance to the overturning. Note that overturning is caused by the moment from the upper strucure. Then, there shall be other moments resisting the overturning moment. Then the ratio is the ratio of the magnitudes of the sum of resisting moments to the overturning moment, known as the safety factor of overturning and denoted by FS_o. A more intuitive diagram is presented as follows.



From the diagram above, we can infer that the contributing resisting moments are from the weight of the foundation as well as the force from the upper structure parallel to the gravity, and the weights of soil segments above the footing. And the safety factor of overturning is given in the following expression:

$$\mathrm{FS}_o := \frac{1}{M_o}\sum_{k=0}^{2} u_k P_k \qquad \text{where} \quad \forall k \in \{1,2\}:\; P_k := \gamma_k L(D_f - t_p)$$

And the foundation is deemed stable against overturning if the following expression is satisfied:

$$\mathrm{FS}_o \geq 1.1$$

Note that overturning shall be evaluated in the axis with the worst overtuning.

C.1 Overturning Evaluations (Non-Overstrength)

Soil filling material unit weigth (the value is assumed)	γ_1 =	16.00 kN/m^3
Overturning moment (non-overstrength)	M_o =	26.76 kNm

Components of resisting moment-0:			
Vert. forces from the upper str. as well found. weight	P_0 =	299.05	kN
Overturning distance	u_0 =	1.05	m
Components of resisting moment-1:			
Width of foundation	L =	2.10	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-1	P_1 =	47.04	kN
Overturning distance	u_1 =	1.58	m
Components of resisting moment-2:			
Width of foundation	L =	2.10	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-2	P_2 =	47.04	kN
Overturning distance	u_2 =	0.53	m
Total resisting moment:	$\sum_{k=0}^{2} u_k P_k =$	412.79	kNm
Safety factor of overturning (non-overstrength)	FS_o =	15.43	

CONCLUSION: SATISFIED

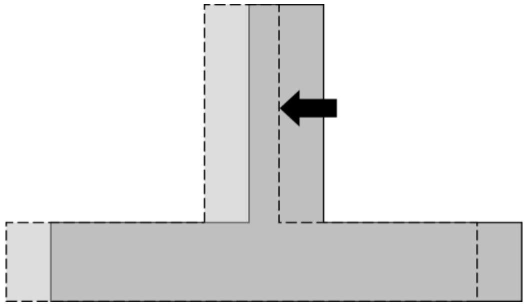
C.2 Overturning Evaluations (Overstrength)

Soil filling material unit weigth (the value is assumed)	γ_1 =	16.00	kN/m^3
Overturning moment (overstrength)	M_o =	59.77	kNm
Components of resisting moment-0:			
Vert. forces from the upper str. as well found. weight	P_0 =	311.00	kN
Overturning distance	u_0 =	1.05	m
Components of resisting moment-1:			
Width of foundation	L =	2.10	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-1	P_1 =	47.04	kN
Overturning distance	u_1 =	1.58	m
Components of resisting moment-2:			
Width of foundation	L =	2.10	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-2	P_2 =	47.04	kN
Overturning distance	u_2 =	0.53	m
Total resisting moment:	$\sum_{k=0}^{2} u_k P_k =$	425.34	kNm
Safety factor of overturning (overstrength)	FS_o =	8.54	

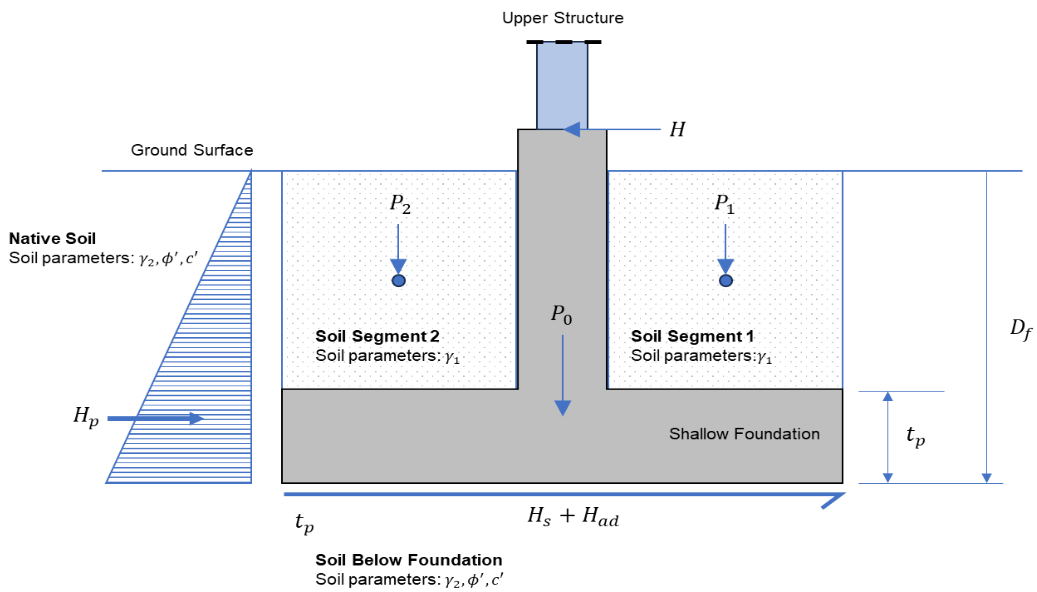
CONCLUSION: SATISFIED

D. SLIDING STABILITY

The phenomena of sliding in shallow foundations is described by the following illustration.



The foundation resistance against sliding takes forms of soil passive pressure, soil lateral resistance below the footing and the adhesive friction between the soil and the footing surface below the footing. A measure to identify the stability of foundations against sliding is the ratio of the sliding resistance to the sliding force, called safety factor of sliding and denoted by $\mathrm{FS_s}$. Now let us observe the following diagram.



The components of sliding stability is presented as follows:

$$H_r = H_p + H_s + H_{ad}$$

where

$$H_p = \left(\frac{\gamma D_f^2 K_p}{2} + 2c' D_f \sqrt{K_p} \right) L \, ,$$

$$H_s = \tan \delta' \sum_{k=0}^{2} P_k \, ,$$

$$H_{ad} = BLc_a$$

and

$$K_p = \tan^2 \left(45 + \frac{\phi'}{2} \right) ,$$

$$\delta' = k_1 \phi' \,,$$
$$c_a = k_2 c' \,.$$

Note that ϕ' is the effective soil friction angle and k_1, k_2 are some factors satisfying $k_1, k_2 \in \left[\frac{1}{2}, \frac{2}{3}\right]$. And the foundation is deemed stable if the following expression is satified:

$$\mathrm{FS}_s := \frac{H_r}{H} = \frac{H_p + H_s + H_{ad}}{H} \geq 1.1$$

D.1 Sliding Evaluation (Non-Overstrength)

Half of internal friction angle + 45 degrees	$0.5\,\phi' + 45 =$	0.79	rad
Rankine coefficient of passive force	$K_p =$	1.04	
Soil lateral factor	$k_1 =$	0.50	
Soil adhesion factor	$k_2 =$	0.50	
Lateral friction angle	$\delta' =$	0.01	rad
Soil-slab adhesion	$c_a =$	20.05	kPa
Passive lateral earth force	$H_p =$	414.73	kN
Soil lateral force below the footing	$H_s =$	3.74	kN
Adhesive force	$H_{ad} =$	88.42	kN
Resisting horizontal force	$H_r =$	506.89	kN
Sliding force (non-overstrength)	$H =$	25.17	kN
Safety factor of sliding (non-overstrength)	$FS_s =$	20.14	
CONCLUSION: SATISFIED			

D.2 Sliding Evaluation (Overstrength)

Half of internal friction angle + 45 degrees	$0.5\,\phi' + 45 =$	0.79	rad
Rankine coefficient of passive force	$K_p =$	1.04	
Soil lateral factor	$k_1 =$	0.50	
Soil adhesion factor	$k_2 =$	0.50	
Lateral friction angle	$\delta' =$	0.01	rad
Soil-slab adhesion	$c_a =$	20.05	kPa
Passive lateral earth force	$H_p =$	414.73	kN
Soil lateral force below the footing	$H_s =$	0.15	kN
Adhesive force	$H_{ad} =$	88.42	kN
Resisting horizontal force	$H_r =$	503.30	kN
Sliding force (overstrength)	$H =$	44.04	kN
Safety factor of sliding (overstrength)	$FS_s =$	13.71	
CONCLUSION: SATISFIED			

E. REFERENCES

Das, Braja M. (2011). *Principles of Foundation Engineering* . Stamford, US: Cengage Learning.

FOOTING REINFORCEMENT OF F1

A. INTRODUCTION

This report covers the analysis of the reinforcement of shallow foundations.

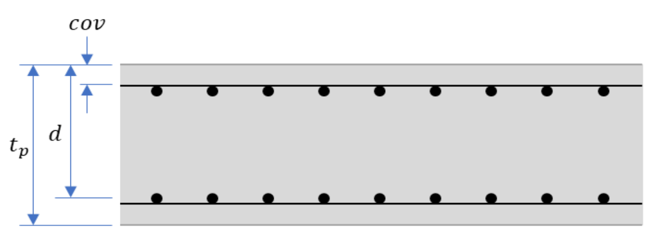
B. MATERIAL PROPERTIES

Concrete compressive strength	f_c' =	29.00	MPa
Rebar yield strength	f_y =	420.00	MPa
Whitney factor	β_1 =	0.84	

Whitney factor describes the rectangular stress block equivalent of concrete structure. The value of the factor is given in accordance with SNI 2847:2019 section 22.2.2.4.3 as follows:

$$\beta_1 = \begin{cases} 0.85 & : 17\ \text{MPa} \leq f_c' \leq 28\ \text{MPa} \\ 0.85 - \frac{0.05(f_c' - 28)}{7} & : 28\ \text{MPa} < f_c' < 55 \\ 0.65 & : f_c' \geq 55\ \text{MPa} \end{cases}$$

C. MECHANICAL AND GEOMETRICAL PROPERTIES



Footing thickness	t_p =	400.00	mm
Rebar diamater (in x-direction)	D_x =	25.00	mm
Rebar diamater (in y-direction)	D_y =	22.00	mm
Number of rebars layers (in x-direction)	n_{lx} =	1.00	
Number of rebars layers (in y-direction)	n_{ly} =	1.00	
Clear distance between adjacent layers	s_l =	40.00	mm
Clear cover	cov =	50.00	mm
Dist. from compressive fibre to centre of tensile rebar (x)	d_x =	315.50	mm
Dist. from compressive fibre to centre of tensile rebar (y)	d_y =	339.00	mm
Distance between adjacent rebars in use:			
For x-direction	s_x =	100.00	mm
For y-direction	s_y =	100.00	mm

D. DUCTILITY REQUIREMENT OF THE REINFORCEMENT

SNI 2847:2019 section 8.6.1.1 asserts that the minimum reinforcement of a concrete slab shall be of the following:

$$A_{s,\min} = \begin{cases} 0.0020 & : f_y < 420\ \text{MPa} \\ \max\left\{\frac{0.0018 \cdot 420}{f_y} A_g\ ,\ 0.0014 A_g\right\} & : f_y \geq 420\ \text{MPa} \end{cases}$$

And the reinforcement also cannot exceed the following requriement:

$$A_{s,\max} = 0.75\rho_b A_g$$
$$\rho_b = \frac{0.85 f'_c \beta_1}{f_y} \frac{600}{f_y + 600}$$

The ductility of the Reinforcement in use is satisfiable if

$$A_{s,\min} \leq A_s \leq A_{s,\max}$$

holds.

D.1. Ductility in X-Direction

Gross sectional area	A_g =	315500.00	mm^2/m
Reinforcement sectional area	A_s =	4908.74	mm^2/m
Required minimum rebars	$A_{s,min}$ =	561.59	mm^2/m
Required maximum rebars	$A_{s,max}$ =	6885.47	mm^2/m

CONCLUSION: SATISFIABLE

D.2. Ductility in Y-Direction

Gross sectional area	A_g =	339000.00	mm^2
Reinforcement sectional area	A_s =	3801.33	mm^2
Required minimum rebars	$A_{s,min}$ =	603.42	mm^2
Required maximum rebars	$A_{s,max}$ =	7398.33	mm^2

CONCLUSION: SATISFIABLE

E. FLEXURAL BENDING CAPACITY OF REINFORCEMENTS

The reinforcement in use shall have a sufficient capacity against the ultimate bending moment in the footing foundation. The nominal bending capacity of the reinforced concrete structure is given by

$$M_n = f_y A_s \left(d - \frac{a}{2}\right)$$

where

$$a = \frac{f_y A_s}{0.85 f'_c b}\,.$$

The bending capacity is satisfiable if the expression

$$r_b = \frac{M_u}{\phi M_n}$$

is satisfied, where Mu is the ultimate bending moment and ϕ = 0.9.

E.1. Bending Capacity in X-Direction

Ultimate bending moment (overstrength considered)	M_u =	444298750.00	Nmm/m
Equivalent thickness of concrete compressive fibre	a =	83.64	mm
Nominal moment	M_n =	564240228.94	Nmm/m
Strength ratio	r_b =	0.87	

CONCLUSION: SATISFIABLE

E.2. Bending Capacity in Y-Direction

Ultimate bending moment (overstrength considered)	M_u =	308580590.00	Nmm/m

Equivalent thickness of concrete compressive fibre	a =	64.77	mm
Nominal moment	M_n =	489529191.60	Nmm/m
Strength ratio	r_b =	0.70	

CONCLUSION: **SATISFIABLE**

F. SUMMARY

The result of the reinforcement design is presented in the following table:

Direction	Reinforcement
X	D25-100
Y	D22-100

ANALYSIS OF FOUNDATION F2

A. INTRODUCTION

This document covers the design and analysis of a shallow fondation. The aspects to be analysed in the design of shallow foundations include the bearing capacity, overturning and sliding stability. These tree aspects are the common probable failure modes of shallow foundations. All the theory in geotechnics in use are from (Das, 2011).

Geometry of Foundation:

Foundation width in x-direction	B_x =	3.00	m
Foundation width in y-direction	B_y =	2.20	m
Footing thickness	t_p =	0.40	m
Depth of foundation	D_f =	2.00	m
Column width in x-direction	$B_{c\text{-}x}$ =	0.40	m
Column width in y-direction	$B_{c\text{-}y}$ =	0.40	m

Soil Mechanical Properties:

Borehole reference		BH.2.1.04	
Unit weight	γ =	16.37	kN/m^3
Effective interal friction angle	ϕ' =	1.09	deg
Soil cohession	c' =	40.10	kPa

B. BEARING CAPACITY

For the bearing capacity, we have computed the soil stress due to bearing loads and the moments from upper structure separately using SAP2000. And the soil stress bearing capacity has already been separately analysed in the soil report. The results are presented as follows.

Non-overstrength soil stress	q =	88.00	kPa
Overstrength soil stress	q_{OS} =	109.00	kPa
Soil bearing capacity (soil report)	q_{all} =	97.00	kPa
Increased bearing capacity due to overstrength	1.2 q_{all} =	116.40	kPa

The increase factor 1.2 is accordance with SNI 1726:2019 section 4.2.3.3.

The bearing capacity is satisfied if the following expressions hold:

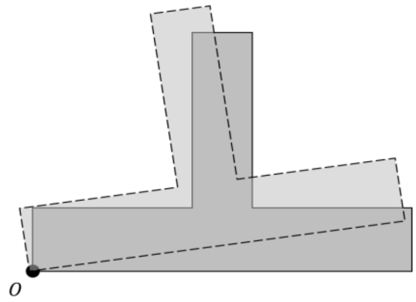
$$r_q := \frac{q}{q_{all}}, \; r_{q.os} := \frac{q_{os}}{1.2 q_{all}} \leq 1$$

Stress ratio	r_q =	0.91
Stress ratio with overstrength	$r_{q.os}$ =	0.94

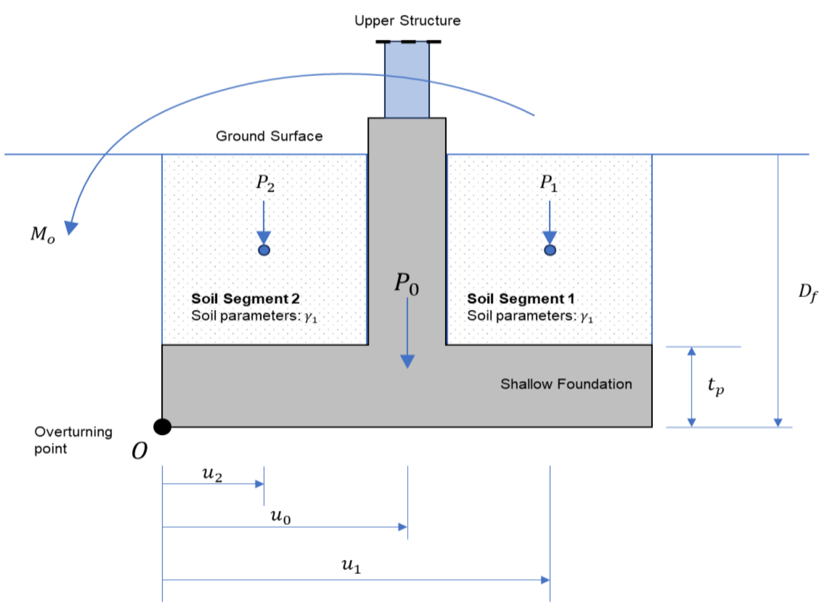
CONCLUSION: SATISFIED

C. OVERTURNING STABILITY

Overturning is another probable phenomenen in a shallow foundation. It is defined as the rotation of the foundation at one of its toes (edges). Now let us observe the following illustration.



The measure to be considered in analysing the overturning stability is the ratio of the counter balance to the overturning. Note that overturning is caused by the moment from the upper strucure. Then, there shall be other moments resisting the overturning moment. Then the ratio is the ratio of the magnitudes of the sum of resisting moments to the overturning moment, known as the safety factor of overturning and denoted by FS_o. A more intuitive diagram is presented as follows.



From the diagram above, we can infer that the contributing resisting moments are from the weight of the foundation as well as the force from the upper structure parallel to the gravity, and the weights of soil segments above the footing. And the safety factor of overturning is given in the following expression:

$$\mathrm{FS}_o := \frac{1}{M_o}\sum_{k=0}^{2} u_k P_k \qquad \text{where} \quad \forall k \in \{1,2\}: \; P_k := \gamma_k L(D_f - t_p)$$

And the foundation is deemed stable against overturning if the following expression is satisfied:

$$\mathrm{FS}_o \geq 1.1$$

Note that overturning shall be evaluated in the axis with the worst overtuning.

C.1 Overturning Evaluations in X-Direction (Non-Overstrength)

Soil filling material unit weigth (the value is assumed)	γ_1 =	16.00	kN/m^3
Overturning moment (non-overstrength)	M_o =	26.02	kNm

Components of resisting moment-0:		
Vert. forces from the upper str. as well found. weight	$P_0 =$	307.65 kN
Overturning distance	$u_0 =$	1.50 m
Components of resisting moment-1:		
Width of foundation	$L =$	2.20 m
Depth of foundation	$D_f =$	2.00 m
Footing thickness	$t_p =$	0.40 m
Weight of soil segment-1	$P_1 =$	73.22 kN
Overturning distance	$u_1 =$	2.25 m
Components of resisting moment-2:		
Width of foundation	$L =$	2.20 m
Depth of foundation	$D_f =$	2.00 m
Footing thickness	$t_p =$	0.40 m
Weight of soil segment-2	$P_2 =$	50.69 kN
Overturning distance	$u_2 =$	0.75 m
Total resisting moment:	$\sum_{k=0}^{2} u_k P_k =$	664.23 kNm
Safety factor of overturning (non-overstrength)	$FS_o =$	25.53

CONCLUSION: SATISFIED

C.2 Overturning Evaluations in X-Direction (Overstrength)

Soil filling material unit weigth (the value is assumed)	$\gamma_1 =$	16.00 kN/m^3
Overturning moment (overstrength)	$M_o =$	60.05 kNm
Components of resisting moment-0:		
Vert. forces from the upper str. as well found. weight	$P_0 =$	308.79 kN
Overturning distance	$u_0 =$	1.50 m
Components of resisting moment-1:		
Width of foundation	$L =$	2.20 m
Depth of foundation	$D_f =$	2.00 m
Footing thickness	$t_p =$	0.40 m
Weight of soil segment-1	$P_1 =$	73.22 kN
Overturning distance	$u_1 =$	2.25 m
Components of resisting moment-2:		
Width of foundation	$L =$	2.20 m
Depth of foundation	$D_f =$	2.00 m
Footing thickness	$t_p =$	0.40 m
Weight of soil segment-2	$P_2 =$	50.69 kN
Overturning distance	$u_2 =$	0.75 m
Total resisting moment:	$\sum_{k=0}^{2} u_k P_k =$	665.94 kNm
Safety factor of overturning (overstrength)	$FS_o =$	13.31

CONCLUSION: SATISFIED

C.3 Overturning Evaluations in Y-Direction (Non-Overstrength)

Soil filling material unit weigth (the value is assumed)	γ_1 =	16.00	kN/m^3
Overturning moment (non-overstrength)	M_o =	27.62	kNm
Components of resisting moment-0:			
Vert. forces from the upper str. as well found. weight	P_0 =	182.37	kN
Overturning distance	u_0 =	0.83	m
Components of resisting moment-1:			
Width of foundation	L =	3.00	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-1	P_1 =	122.88	kN
Overturning distance	u_1 =	1.51	m
Components of resisting moment-2:			
Width of foundation	L =	3.00	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-2	P_2 =	15.36	kN
Overturning distance	u_2 =	0.20	m
Total resisting moment:	$\sum_{k=0}^{2} u_k P_k =$	339.38	kNm
Safety factor of overturning (non-overstrength)	FS_o =	12.29	

CONCLUSION: SATISFIED

C.4 Overturning Evaluations in Y-Direction (Overstrength)

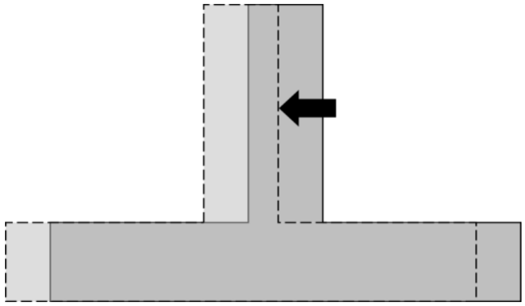
Soil filling material unit weigth (the value is assumed)	γ_1 =	16.00	kN/m^3
Overturning moment (overstrength)	M_o =	63.80	kNm
Components of resisting moment-0:			
Vert. forces from the upper str. as well found. weight	P_0 =	122.81	kN
Overturning distance	u_0 =	0.83	m
Components of resisting moment-1:			
Width of foundation	L =	2.20	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-1	P_1 =	90.11	kN
Overturning distance	u_1 =	1.51	m
Components of resisting moment-2:			
Width of foundation	L =	3.00	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-2	P_2 =	15.36	kN
Overturning distance	u_2 =	0.20	m

2

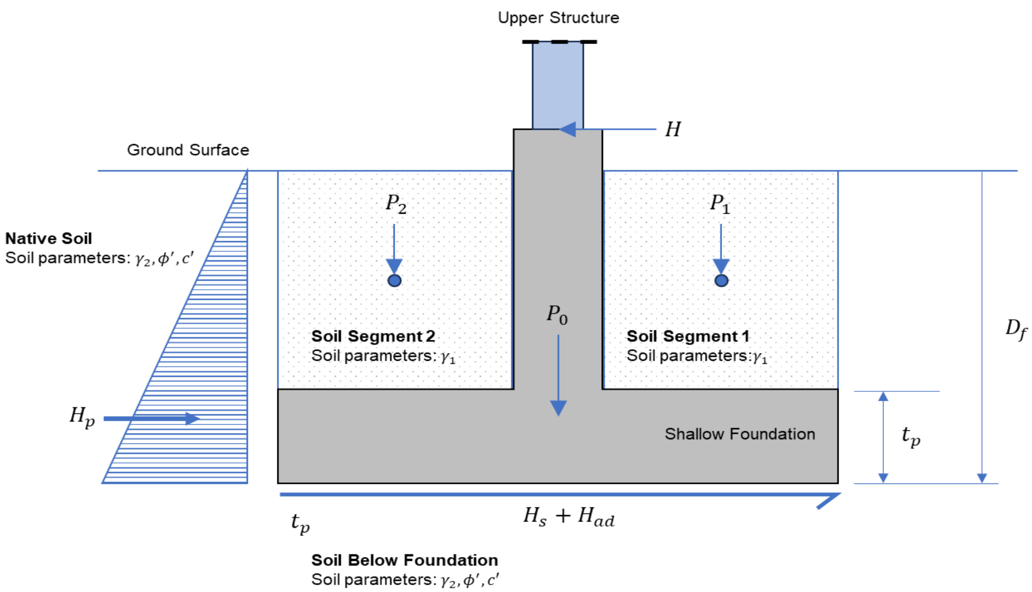
Total resisting moment:	$\sum_{k=0} u_k P_k =$	240.69 kNm
Safety factor of overturning (overstrength)	FS_o =	4.53
CONCLUSION: **SATISFIED**		

D. SLIDING STABILITY

The phenomena of sliding in shallow foundations is described by the following illustration.



The foundation resistance against sliding takes forms of soil passive pressure, soil lateral resistance below the footing and the adhesive friction between the soil and the footing surface below the footing. A measure to identify the stability of foundations against sliding is the ratio of the sliding resistance to the sliding force, called safety factor of sliding and denoted by FS_s. Now let us observe the following diagram.



The components of sliding stability is presented as follows:

$$H_r = H_p + H_s + H_{ad}$$

where

$$H_p = \left(\frac{\gamma D_f^2 K_p}{2} + 2c' D_f \sqrt{K_p} \right) L \,,$$

$$H_s = \tan \delta' \sum_{k=0}^{2} P_k \,,$$

$$H_{ad} = BLc_a$$

and

$$K_p = \tan^2\left(45 + \frac{\phi'}{2}\right),$$
$$\delta' = k_1\phi',$$
$$c_a = k_2c'.$$

Note that ϕ' is the effective soil friction angle and k_1, k_2 are some factors satisfying $k_1, k_2 \in \left[\frac{1}{2}, \frac{2}{3}\right]$. And the foundation is deemed stable if the following expression is satified:

$$\mathrm{FS}_s := \frac{H_r}{H} = \frac{H_p + H_s + H_{ad}}{H} \geq 1.1$$

D.1 Sliding Evaluation in X-Direction (Non-Overstrength)

Half of internal friction angle + 45 degrees	0.5 ϕ' + 45 =	0.79	rad
Rankine coefficient of passive force	K_p =	1.04	
Soil lateral factor	k_1 =	0.50	
Soil adhesion factor	k_2 =	0.50	
Lateral friction angle	δ' =	0.01	rad
Soil-slab adhesion	c_a =	20.05	kPa
Passive lateral earth force	H_p =	434.48	kN
Soil lateral force below the footing	H_s =	4.11	kN
Adhesive force	H_{ad} =	132.33	kN
Resisting horizontal force	H_r =	570.91	kN
Sliding force (non-overstrength)	H =	14.86	kN
Safety factor of sliding (non-overstrength)	FS_s =	38.43	
CONCLUSION: **SATISFIED**			

D.2 Sliding Evaluation in X-Direction (Overstrength)

Half of internal friction angle + 45 degrees	0.5 ϕ' + 45 =	0.79	rad
Rankine coefficient of passive force	K_p =	1.04	
Soil lateral factor	k_1 =	0.50	
Soil adhesion factor	k_2 =	0.50	
Lateral friction angle	δ' =	0.01	rad
Soil-slab adhesion	c_a =	20.05	kPa
Passive lateral earth force	H_p =	434.48	kN
Soil lateral force below the footing	H_s =	4.12	kN
Adhesive force	H_{ad} =	132.33	kN
Resisting horizontal force	H_r =	570.93	kN
Sliding force (overstrength)	H =	34.28	kN
Safety factor of sliding (overstrength)	FS_s =	19.98	
CONCLUSION: **SATISFIED**			

D.3 Sliding Evaluation in Y-Direction (Non-Overstrength)

Half of internal friction angle + 45 degrees	0.5 ϕ' + 45 =	0.79	rad

Rankine coefficient of passive force	K_p =	1.04	
Soil lateral factor	k_1 =	0.50	
Soil adhesion factor	k_2 =	0.50	
Lateral friction angle	δ' =	0.01	rad
Soil-slab adhesion	c_a =	20.05	kPa
Passive lateral earth force	H_p =	592.47	kN
Soil lateral force below the footing	H_s =	3.05	kN
Adhesive force	H_{ad} =	132.33	kN
Resisting horizontal force	H_r =	727.85	kN
Sliding force (non-overstrength)	H =	24.11	kN
Safety factor of sliding (non-overstrength)	FS_s =	30.19	

CONCLUSION: SATISFIED

D.4 Sliding Evaluation in Y-Direction (Overstrength)

Half of internal friction angle + 45 degrees	0.5 ϕ' + 45 =	0.79	rad
Rankine coefficient of passive force	K_p =	1.04	
Soil lateral factor	k_1 =	0.50	
Soil adhesion factor	k_2 =	0.50	
Lateral friction angle	δ' =	0.01	rad
Soil-slab adhesion	c_a =	20.05	kPa
Passive lateral earth force	H_p =	434.48	kN
Soil lateral force below the footing	H_s =	2.17	kN
Adhesive force	H_{ad} =	132.33	kN
Resisting horizontal force	H_r =	568.98	kN
Sliding force (overstrength)	H =	55.49	kN
Safety factor of sliding (overstrength)	FS_s =	12.30	

CONCLUSION: SATISFIED

E. REFERENCES

Das, Braja M. (2011). *Principles of Foundation Engineering* . Stamford, US: Cengage Learning.

FOOTING REINFORCEMENT OF F2

A. INTRODUCTION

This report covers the analysis of the reinforcement of shallow foundations.

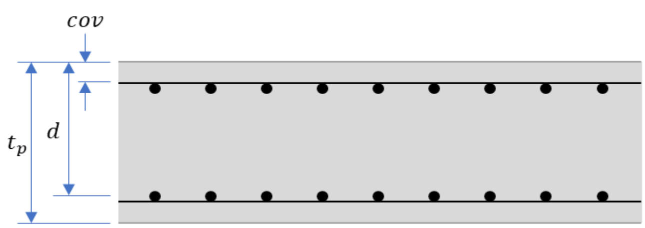
B. MATERIAL PROPERTIES

Concrete compressive strength	f_c' =	29.00	MPa
Rebar yield strength	f_y =	420.00	MPa
Whitney factor	β_1 =	0.84	

Whitney factor describes the rectangular stress block equivalent of concrete structure. The value of the factor is given in accordance with SNI 2847:2019 section 22.2.2.4.3 as follows:

$$\beta_1 = \begin{cases} 0.85 & : 17\ \mathrm{MPa} \leq f_c' \leq 28\ \mathrm{MPa} \\ 0.85 - \frac{0.05(f_c'-28)}{7} & : 28\ \mathrm{MPa} < f_c' < 55 \\ 0.65 & : f_c' \geq 55\ \mathrm{MPa} \end{cases}$$

C. MECHANICAL AND GEOMETRICAL PROPERTIES



Footing thickness	t_p =	400.00	mm
Rebar diamater (in x-direction)	D_x =	22.00	mm
Rebar diamater (in y-direction)	D_y =	22.00	mm
Clear cover	cov =	50.00	mm
Dist. from compressive fibre to centre of tensile rebar (x)	d_x =	361.00	mm
Dist. from compressive fibre to centre of tensile rebar (y)	d_y =	317.00	mm
Distance between adjacent rebars in use:			
For x-direction	s_x =	100.00	mm
For y-direction	s_y =	100.00	mm

D. DUCTILITY REQUIREMENT OF THE REINFORCEMENT

SNI 2847:2019 section 8.6.1.1 asserts that the minimum reinforcement of a concrete slab shall be of the following:

$$A_{s,\min} = \begin{cases} 0.0020 & : f_y < 420\ \mathrm{MPa} \\ \max\left\{\frac{0.0018 \cdot 420}{f_y} A_g\ ,\ 0.0014 A_g\right\} & : f_y \geq 420\ \mathrm{MPa} \end{cases}$$

And the reinforcement also cannot exceed the following requriement:

$$A_{s,\max} = 0.75 \rho_b A_g$$

$$\rho_b = \frac{0.85 f_c' \beta_1}{f_y} \frac{600}{f_y + 600}$$

The ductility of the Reinforcement in use is satisfiable if

$$A_{s,\min} \leq A_s \leq A_{s,\max}$$

holds.

D.1. Ductility in X-Direction

Gross sectional area	A_g =	361000.00	mm^2/m
Reinforcement sectional area	A_s =	3801.33	mm^2/m
Required minimum rebars	$A_{s,min}$ =	642.58	mm^2/m
Required maximum rebars	$A_{s,max}$ =	7878.46	mm^2/m

CONCLUSION: SATISFIABLE

D.2. Ductility in Y-Direction

Gross sectional area	A_g =	317000.00	mm^2
Reinforcement sectional area	A_s =	3801.33	mm^2
Required minimum rebars	$A_{s,min}$ =	564.26	mm^2
Required maximum rebars	$A_{s,max}$ =	6918.20	mm^2

CONCLUSION: SATISFIABLE

E. FLEXURAL BENDING CAPACITY OF REINFORCEMENTS

The reinforcement in use shall have a sufficient capacity against the ultimate bending moment in the footing foundation. The nominal bending capacity of the reinforced concrete structure is given by

$$M_n = f_y A_s \left(d - \frac{a}{2}\right)$$

where

$$a = \frac{f_y A_s}{0.85 f'_c b}.$$

The bending capacity is satisfiable if the expression

$$r_b = \frac{M_u}{\phi M_n}$$

is satisfied, where Mu is the ultimate bending moment and ϕ = 0.9.

E.1. Bending Capacity in X-Direction

Ultimate bending moment (overstrength considered)	M_u =	383451710.00	Nmm/m
Equivalent thickness of concrete compressive fibre	a =	64.77	mm
Nominal moment	M_n =	524653454.10	Nmm/m
Strength ratio	r_b =	0.81	

CONCLUSION: SATISFIABLE

E.2. Bending Capacity in Y-Direction

Ultimate bending moment (overstrength considered)	M_u =	354139910.00	Nmm/m
Equivalent thickness of concrete compressive fibre	a =	64.77	mm
Nominal moment	M_n =	454404929.09	Nmm/m
Strength ratio	r_b =	0.87	

CONCLUSION: SATISFIABLE

F. SUMMARY

The result of the reinforcement design is presented in the following table:

Direction	Reinforcement
X	D22-100
Y	D22-100

ANALYSIS OF FOUNDATION F3

A. INTRODUCTION

This document covers the design and analysis of a shallow fondation. The aspects to be analysed in the design of shallow foundations include the bearing capacity, overturning and sliding stability. These tree aspects are the common probable failure modes of shallow foundations. All the theory in geotechnics in use are from (Das, 2011).

Geometry of Foundation:

Foundation width in x-direction	B_x =	2.70	m
Foundation width in y-direction	B_y =	1.75	m
Footing thickness	t_p =	0.40	m
Depth of foundation	D_f =	2.00	m
Column width in x-direction	$B_{c\text{-}x}$ =	0.40	m
Column width in y-direction	$B_{c\text{-}y}$ =	0.40	m

Soil Mechanical Properties:

Borehole reference	BH.2.1.04		
Unit weight	γ =	16.37	kN/m^3
Effective interal friction angle	ϕ' =	1.09	deg
Soil cohession	c' =	40.10	kPa

B. BEARING CAPACITY

For the bearing capacity, we have computed the soil stress due to bearing loads and the moments from upper structure separately using SAP2000. And the soil stress bearing capacity has already been separately analysed in the soil report. The results are presented as follows.

Non-overstrength soil stress	q =	89.00	kPa
Overstrength soil stress	q_{OS} =	103.00	kPa
Soil bearing capacity (soil report)	q_{all} =	93.00	kPa
Increased bearing capacity due to overstrength	1.2 q_{all} =	111.60	kPa

The increase factor 1.2 is accordance with SNI 1726:2019 section 4.2.3.3.

The bearing capacity is satisfied if the following expressions hold:

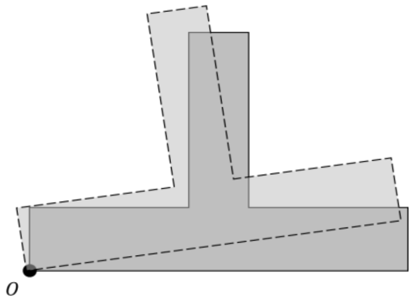
$$r_q := \frac{q}{q_{all}}\ ,\ r_{q.os} := \frac{q_{os}}{1.2 q_{all}} \le 1$$

Stress ratio	r_q =	0.96
Stress ratio with overstrength	$r_{q.os}$ =	0.92

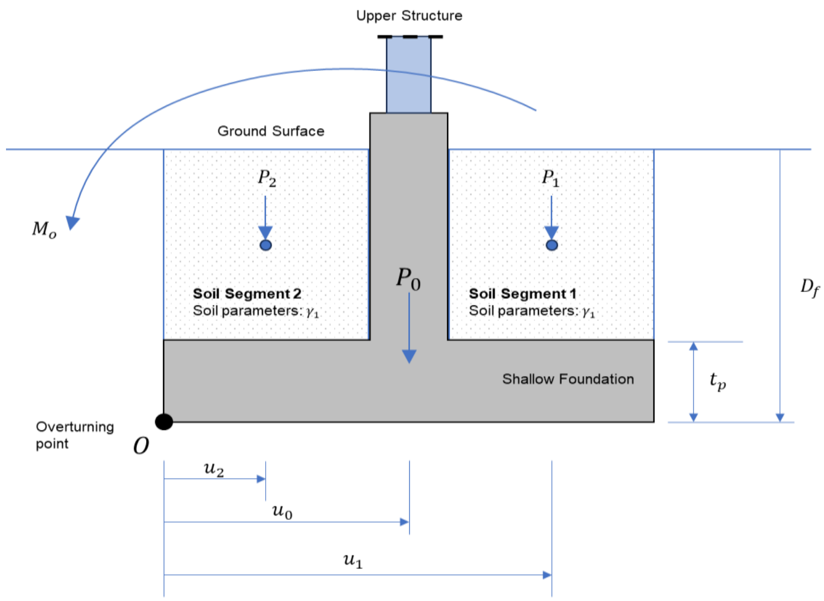
CONCLUSION: SATISFIED

C. OVERTURNING STABILITY

Overturning is another probable phenomenen in a shallow foundation. It is defined as the rotation of the foundation at one of its toes (edges). Now let us observe the following illustration.



The measure to be considered in analysing the overturning stability is the ratio of the counter balance to the overturning. Note that overturning is caused by the moment from the upper strucure. Then, there shall be other moments resisting the overturning moment. Then the ratio is the ratio of the magnitudes of the sum of resisting moments to the overturning moment, known as the safety factor of overturning and denoted by $\mathrm{FS_o}$. A more intuitive diagram is presented as follows.



From the diagram above, we can infer that the contributing resisting moments are from the weight of the foundation as well as the force from the upper structure parallel to the gravity, and the weights of soil segments above the footing. And the safety factor of overturning is given in the following expression:

$$\mathrm{FS}_o := \frac{1}{M_o}\sum_{k=0}^{2} u_k P_k \qquad \text{where} \quad \forall k \in \{1,2\}: \; P_k := \gamma_k L(D_f - t_p)$$

And the foundation is deemed stable against overturning if the following expression is satisfied:

$$\mathrm{FS}_o \geq 1.1$$

Note that overturning shall be evaluated in the axis with the worst overtuning.

C.1 Overturning Evaluations in X-Direction (Non-Overstrength)

Soil filling material unit weigth (the value is assumed)	γ_1 =	16.00	kN/m^3
Overturning moment (non-overstrength)	M_o =	25.00	kNm

Components of resisting moment-0:			
Vert. forces from the upper str. as well found. weight	P_0 =	289.56	kN
Overturning distance	u_0 =	1.35	m
Components of resisting moment-1:			
Width of foundation	L =	1.75	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-1	P_1 =	51.52	kN
Overturning distance	u_1 =	2.03	m
Components of resisting moment-2:			
Width of foundation	L =	1.75	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-2	P_2 =	30.24	kN
Overturning distance	u_2 =	0.68	m
Total resisting moment:	$\sum_{k=0}^{2} u_k P_k =$	515.64	kNm
Safety factor of overturning (non-overstrength)	FS_o =	20.62	
CONCLUSION:	**SATISFIED**		

C.2 Overturning Evaluations in X-Direction (Overstrength)

Soil filling material unit weigth (the value is assumed)	γ_1 =	16.00	kN/m^3
Overturning moment (overstrength)	M_o =	57.70	kNm
Components of resisting moment-0:			
Vert. forces from the upper str. as well found. weight	P_0 =	301.48	kN
Overturning distance	u_0 =	1.35	m
Components of resisting moment-1:			
Width of foundation	L =	1.75	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-1	P_1 =	51.52	kN
Overturning distance	u_1 =	2.03	m
Components of resisting moment-2:			
Width of foundation	L =	1.75	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-2	P_2 =	30.24	kN
Overturning distance	u_2 =	0.68	m
Total resisting moment:	$\sum_{k=0}^{2} u_k P_k =$	531.74	kNm
Safety factor of overturning (overstrength)	FS_o =	11.06	
CONCLUSION:	**SATISFIED**		

C.3 Overturning Evaluations in Y-Direction (Non-Overstrength)

Soil filling material unit weigth (the value is assumed)	γ_1 =	16.00	kN/m^3
Overturning moment (non-overstrength)	M_o =	22.31	kNm
Components of resisting moment-0:			
Vert. forces from the upper str. as well found. weight	P_0 =	310.84	kN
Overturning distance	u_0 =	0.75	m
Components of resisting moment-1:			
Width of foundation	L =	2.70	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-1	P_1 =	55.30	kN
Overturning distance	u_1 =	1.25	m
Components of resisting moment-2:			
Width of foundation	L =	2.70	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-2	P_2 =	38.02	kN
Overturning distance	u_2 =	0.38	m
Total resisting moment:	$\sum_{k=0}^{2} u_k P_k =$	316.50	kNm
Safety factor of overturning (non-overstrength)	FS_o =	14.18	

CONCLUSION: SATISFIED

C.4 Overturning Evaluations in Y-Direction (Overstrength)

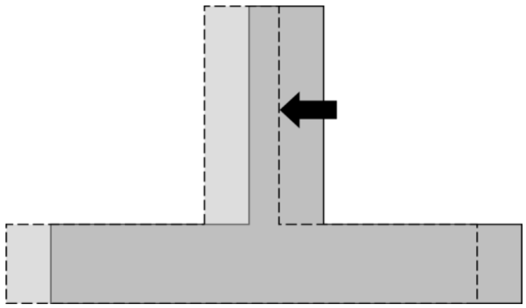
Soil filling material unit weigth (the value is assumed)	γ_1 =	16.00	kN/m^3
Overturning moment (overstrength)	M_o =	44.88	kNm
Components of resisting moment-0:			
Vert. forces from the upper str. as well found. weight	P_0 =	305.23	kN
Overturning distance	u_0 =	0.75	m
Components of resisting moment-1:			
Width of foundation	L =	1.75	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-1	P_1 =	35.84	kN
Overturning distance	u_1 =	1.25	m
Components of resisting moment-2:			
Width of foundation	L =	2.70	m
Depth of foundation	D_f =	2.00	m
Footing thickness	t_p =	0.40	m
Weight of soil segment-2	P_2 =	38.02	kN
Overturning distance	u_2 =	0.38	m

Total resisting moment:	$\sum_{k=0}^{2} u_k P_k =$	287.98 kNm
Safety factor of overturning (overstrength)	FS_o =	7.70

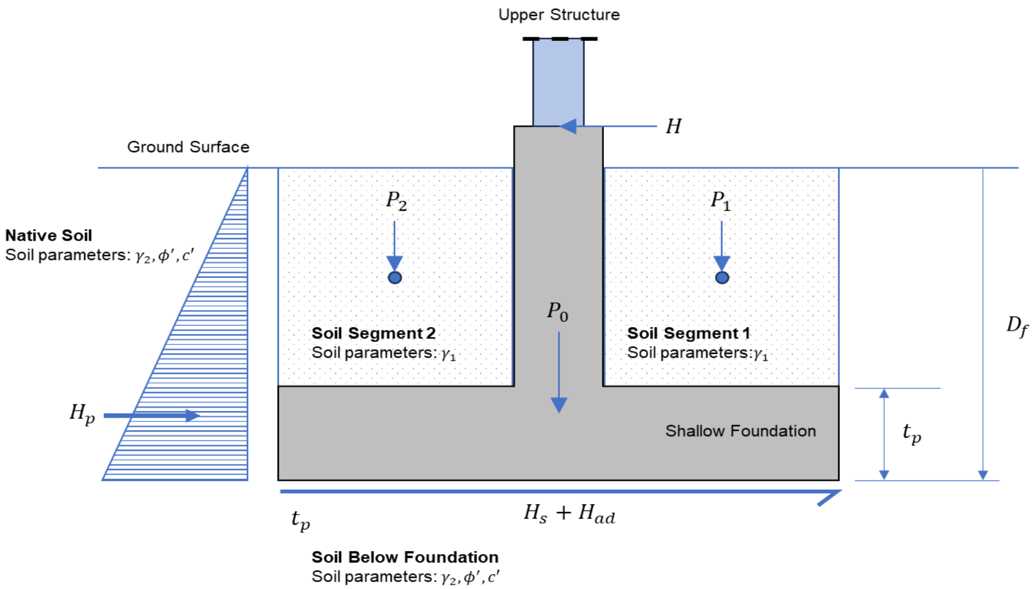
CONCLUSION: **SATISFIED**

D. SLIDING STABILITY

The phenomena of sliding in shallow foundations is described by the following illustration.



The foundation resistance against sliding takes forms of soil passive pressure, soil lateral resistance below the footing and the adhesive friction between the soil and the footing surface below the footing. A measure to identify the stability of foundations against sliding is the ratio of the sliding resistance to the sliding force, called safety factor of sliding and denoted by $\mathrm{FS_s}$. Now let us observe the following diagram.



The components of sliding stability is presented as follows:

$$H_r = H_p + H_s + H_{ad}$$

where

$$H_p = \left(\frac{\gamma D_f^2 K_p}{2} + 2c' D_f \sqrt{K_p} \right) L \,,$$

$$H_s = \tan \delta' \sum_{k=0}^{2} P_k \,,$$

$$H_{ad} = BLc_a$$

and

$$K_p = \tan^2\left(45 + \frac{\phi'}{2}\right),$$
$$\delta' = k_1\phi',$$
$$c_a = k_2c'.$$

Note that ϕ' is the effective soil friction angle and k_1, k_2 are some factors satisfying $k_1, k_2 \in \left[\frac{1}{2}, \frac{2}{3}\right]$. And the foundation is deemed stable if the following expression is satified:

$$\mathrm{FS}_s := \frac{H_r}{H} = \frac{H_p + H_s + H_{ad}}{H} \geq 1.1$$

D.1 Sliding Evaluation in X-Direction (Non-Overstrength)

Half of internal friction angle + 45 degrees	0.5 ϕ' + 45 =	0.79	rad
Rankine coefficient of passive force	K_p =	1.04	
Soil lateral factor	k_1 =	0.50	
Soil adhesion factor	k_2 =	0.50	
Lateral friction angle	δ' =	0.01	rad
Soil-slab adhesion	c_a =	20.05	kPa
Passive lateral earth force	H_p =	345.61	kN
Soil lateral force below the footing	H_s =	3.53	kN
Adhesive force	H_{ad} =	94.74	kN
Resisting horizontal force	H_r =	443.88	kN
Sliding force (non-overstrength)	H =	17.72	kN
Safety factor of sliding (non-overstrength)	FS_s =	25.04	

CONCLUSION: SATISFIED

D.2 Sliding Evaluation in X-Direction (Overstrength)

Half of internal friction angle + 45 degrees	0.5 ϕ' + 45 =	0.79	rad
Rankine coefficient of passive force	K_p =	1.04	
Soil lateral factor	k_1 =	0.50	
Soil adhesion factor	k_2 =	0.50	
Lateral friction angle	δ' =	0.01	rad
Soil-slab adhesion	c_a =	20.05	kPa
Passive lateral earth force	H_p =	345.61	kN
Soil lateral force below the footing	H_s =	3.65	kN
Adhesive force	H_{ad} =	94.74	kN
Resisting horizontal force	H_r =	443.99	kN
Sliding force (overstrength)	H =	40.90	kN
Safety factor of sliding (overstrength)	FS_s =	13.03	

CONCLUSION: SATISFIED

D.3 Sliding Evaluation in Y-Direction (Non-Overstrength)

Half of internal friction angle + 45 degrees	$0.5\ \phi' + 45 =$	0.79	rad
Rankine coefficient of passive force	$K_p =$	1.04	
Soil lateral factor	$k_1 =$	0.50	
Soil adhesion factor	$k_2 =$	0.50	
Lateral friction angle	$\delta' =$	0.01	rad
Soil-slab adhesion	$c_a =$	20.05	kPa
Passive lateral earth force	$H_p =$	533.22	kN
Soil lateral force below the footing	$H_s =$	3.84	kN
Adhesive force	$H_{ad} =$	94.74	kN
Resisting horizontal force	$H_r =$	631.81	kN
Sliding force (non-overstrength)	$H =$	12.41	kN
Safety factor of sliding (non-overstrength)	$FS_s =$	50.90	

CONCLUSION: SATISFIED

D.4 Sliding Evaluation in Y-Direction (Overstrength)

Half of internal friction angle + 45 degrees	$0.5\ \phi' + 45 =$	0.79	rad
Rankine coefficient of passive force	$K_p =$	1.04	
Soil lateral factor	$k_1 =$	0.50	
Soil adhesion factor	$k_2 =$	0.50	
Lateral friction angle	$\delta' =$	0.01	rad
Soil-slab adhesion	$c_a =$	20.05	kPa
Passive lateral earth force	$H_p =$	345.61	kN
Soil lateral force below the footing	$H_s =$	3.61	kN
Adhesive force	$H_{ad} =$	94.74	kN
Resisting horizontal force	$H_r =$	443.95	kN
Sliding force (overstrength)	$H =$	28.56	kN
Safety factor of sliding (overstrength)	$FS_s =$	18.65	

CONCLUSION: SATISFIED

E. REFERENCES

Das, Braja M. (2011). *Principles of Foundation Engineering* . Stamford, US: Cengage Learning.

FOOTING REINFORCEMENT OF F3

A. INTRODUCTION

This report covers the analysis of the reinforcement of shallow foundations.

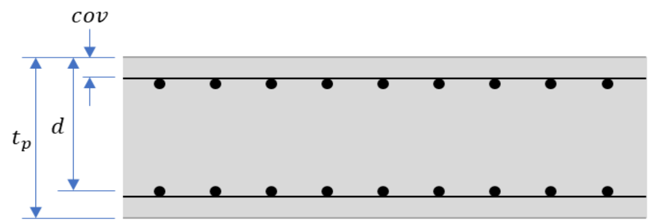
B. MATERIAL PROPERTIES

Concrete compressive strength	f_c' =	29.00	MPa
Rebar yield strength	f_y =	420.00	MPa
Whitney factor	β_1 =	0.84	

Whitney factor describes the rectangular stress block equivalent of concrete structure. The value of the factor is given in accordance with SNI 2847:2019 section 22.2.2.4.3 as follows:

$$\beta_1 = \begin{cases} 0.85 & : 17 \text{ MPa} \leq f_c' \leq 28 \text{ MPa} \\ 0.85 - \frac{0.05(f_c'-28)}{7} & : 28 \text{ MPa} < f_c' < 55 \\ 0.65 & : f_c' \geq 55 \text{ MPa} \end{cases}$$

C. MECHANICAL AND GEOMETRICAL PROPERTIES



Footing thickness	t_p =	400.00	mm
Rebar diamater (in x-direction)	D_x =	22.00	mm
Rebar diamater (in y-direction)	D_y =	22.00	mm
Clear cover	cov =	50.00	mm
Dist. from compressive fibre to centre of tensile rebar (x)	d_x =	361.00	mm
Dist. from compressive fibre to centre of tensile rebar (y)	d_y =	317.00	mm
Distance between adjacent rebars in use:			
For x-direction	s_x =	100.00	mm
For y-direction	s_y =	100.00	mm

D. DUCTILITY REQUIREMENT OF THE REINFORCEMENT

SNI 2847:2019 section 8.6.1.1 asserts that the minimum reinforcement of a concrete slab shall be of the following:

$$A_{s,\min} = \begin{cases} 0.0020 & : f_y < 420 \text{ MPa} \\ \max\left\{\frac{0.0018 \cdot 420}{f_y} A_g \text{ , } 0.0014 A_g\right\} & : f_y \geq 420 \text{ MPa} \end{cases}$$

And the reinforcement also cannot exceed the following requriement:

$$A_{s,\max} = 0.75 \rho_b A_g$$

$$\rho_b = \frac{0.85 f_c' \beta_1}{f_y} \frac{600}{f_y + 600}$$

The ductility of the Reinforcement in use is satisfiable if

$$A_{s,\min} \leq A_s \leq A_{s,\max}$$

holds.

D.1. Ductility in X-Direction

Gross sectional area	A_g =	361000.00	mm^2/m
Reinforcement sectional area	A_s =	3801.33	mm^2/m
Required minimum rebars	$A_{s,min}$ =	642.58	mm^2/m
Required maximum rebars	$A_{s,max}$ =	7878.46	mm^2/m

CONCLUSION: SATISFIABLE

D.2. Ductility in Y-Direction

Gross sectional area	A_g =	317000.00	mm^2
Reinforcement sectional area	A_s =	3801.33	mm^2
Required minimum rebars	$A_{s,min}$ =	564.26	mm^2
Required maximum rebars	$A_{s,max}$ =	6918.20	mm^2

CONCLUSION: SATISFIABLE

E. FLEXURAL BENDING CAPACITY OF REINFORCEMENTS

The reinforcement in use shall have a sufficient capacity against the ultimate bending moment in the footing foundation. The nominal bending capacity of the reinforced concrete structure is given by

$$M_n = f_y A_s \left(d - \frac{a}{2}\right)$$

where

$$a = \frac{f_y A_s}{0.85 f_c' b}.$$

The bending capacity is satisfiable if the expression

$$r_b = \frac{M_u}{\phi M_n}$$

is satisfied, where Mu is the ultimate bending moment and ϕ = 0.9.

E.1. Bending Capacity in X-Direction

Ultimate bending moment (overstrength considered)	M_u =	397002020.00	Nmm/m
Equivalent thickness of concrete compressive fibre	a =	64.77	mm
Nominal moment	M_n =	524653454.10	Nmm/m
Strength ratio	r_b =	0.84	

CONCLUSION: SATISFIABLE

E.2. Bending Capacity in Y-Direction

Ultimate bending moment (overstrength considered)	M_u =	332357640.00	Nmm/m
Equivalent thickness of concrete compressive fibre	a =	64.77	mm
Nominal moment	M_n =	454404929.09	Nmm/m
Strength ratio	r_b =	0.81	

CONCLUSION: SATISFIABLE

F. SUMMARY

The result of the reinforcement design is presented in the following table:

Direction	Reinforcement
X	D22-100
Y	D22-100

ATTACHMENT 4

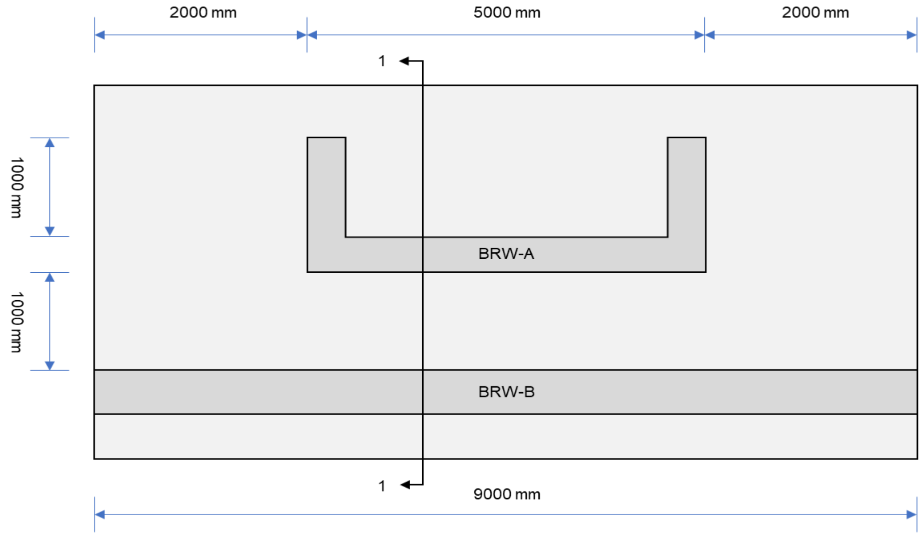
Blast Resisting Walls Analysis and Reinforcement

Stability of Blast Resisting Wall Foundation

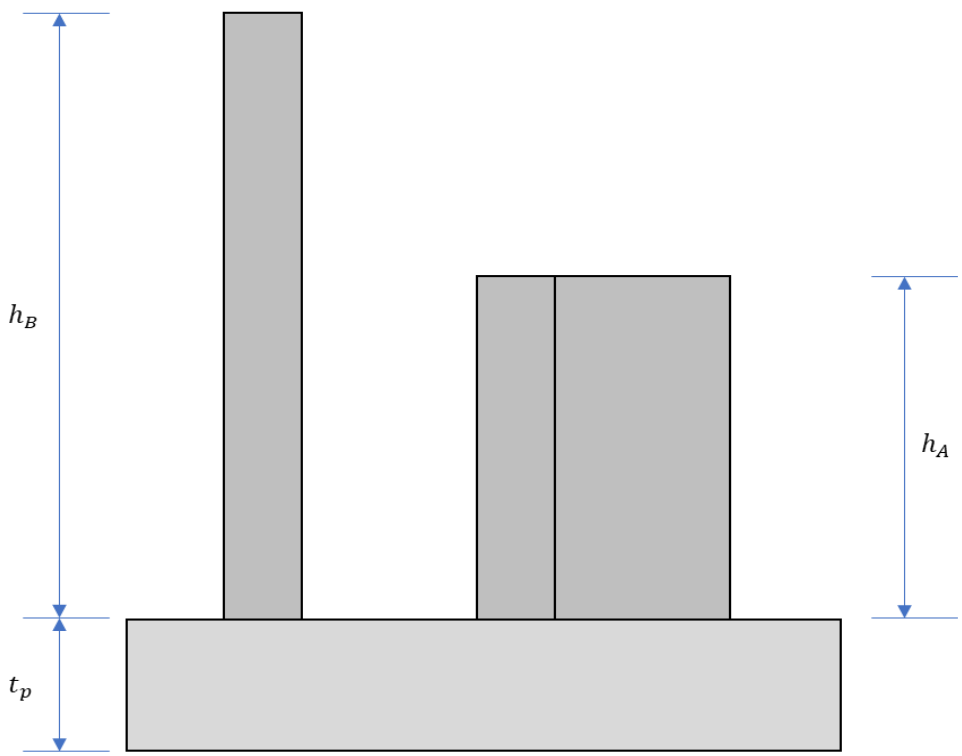
A. INTRODUCTION

This document covers the analysis of the substructure stability of the blast resisting walls. There are 2 blast resisting walls which are called BRW-A and BRW-B which are constructed from concrete structures. The foundation of the two walls is combined in a single mat foundation. The basic arrangement of the foundation is presented as follows.

Plan View



Cross Section



The loads considered for the analysis include the dead load and the seismic load. The load combinations in use are the ASD combinations in accordance with SNI 1727:2020. For the bearing capacity of the foundation, we use SAP2000 by modelling the structure as well as the foundation and the applied loads. For the overturning and sliding stability, we analyze them manually.

B. BEARING CAPACITY

The blast resisting walls as well as the foundation are modelled in SAP2000. The dead load is the self weight of the structure as well as the foundation. The seismic load is computed using the response spectrum method in SAP2000 by providing the seismic parameters. In this document, we cover the comparison of the soil stress from the SAP2000 with the bearing capacity is given as follows:

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

where

$$
\begin{aligned}
c' &= \text{ cohesion} \\
\phi' &= \text{ internal friction angle} \\
\gamma &= \text{ soil unit weight} \\
q &= \gamma D_f \\
D_f &= \text{ depth of foundation} \\
B &= \text{ width of foundation} \\
F_{cs} &= 1 + \frac{B N_q}{L N_c} \\
F_{qs} &= 1 + \frac{B}{L}\tan\phi' \\
F_{\gamma s} &= 1 - 0.4\frac{B}{L} \\
F_{qd} &= \begin{cases} 1 & : \phi = 0 \\ 1 + 2\tan\phi'(1-\sin\phi')^2 \frac{D_f}{B} & : \phi' > 0 \ \wedge \ \frac{D_f}{B} \le 1 \\ 1 + 2\tan\phi'(1-\sin\phi')^2 \tan^{-1}\left(\frac{D_f}{B}\right) & : \phi' > 0 \ \wedge \ \frac{D_f}{B} > 0 \end{cases} \\
F_{cd} &= \begin{cases} 1 + 0.4\frac{D_f}{B} & : \phi = 0 \ \wedge \ \frac{D_f}{B} \le 1 \\ F_{qd} - \frac{1-F_{qd}}{N_c \tan\phi'} & : \phi' > 0 \\ 1 + 0.4\tan^{-1}\left(\frac{D_f}{B}\right) & : \phi = 0 \ \wedge \ \frac{D_f}{B} > 1 \end{cases} \\
F_{\gamma d} &= 1 \\
F_{ci} = F_{qi} &= \left(1 - \frac{2\beta}{\pi}\right)^2 \\
F_{\gamma i} &= \left(1 - \frac{\beta}{\phi'}\right) \\
\beta &= \text{ inclination angle of force from vertical direction} \\
N_q &= \tan^2\left(\frac{\pi}{4} + \frac{\phi'}{2}\right) e^{\pi \tan\phi'} \\
N_c &= \frac{N_q - 1}{\tan\phi'} \\
N_\gamma &= 2(N_q + 1)\tan\phi'
\end{aligned}
$$

The capacity will also consider both non-overstrength and overstrength analaysis. The soil stress of non-overstrength will be denoted by q, while q_{os} will denote the overstrength. The allowable bearing capacity is given by q_a. The bearing capacity is satisfiable if the following expressions are satisfied:

$$r_b := \frac{q}{q_a} \leq 1\,, \quad r_{b.os} := \frac{q_{os}}{1.2q_a}$$

Soil cohesion	c' =	40.10	kPa
Soil unit weight	γ =	16.37	kN/m^3
Width of foundation	B =	3.60	m
Length of foundation	L =	9.00	m
Depth of foundation	D_f =	1.00	m
Effective stress	q =	16.37	kPa
Internel friction angle	ϕ' =	0.02	rad
Force inclination angle from vertical	β =	0.00	rad
Bearing capacity factors:			
	N_q =	1.10	
	N_c =	5.40	
	N_γ =	0.08	
Shape factors:			
	F_{cs} =	1.08	
	F_{qs} =	1.01	
	$F_{\gamma s}$ =	0.84	
Depth factors:			
Depth to width factor	Df/B =	0.28	
	F_{qd} =	1.01	
	F_{cd} =	1.11	
	$F_{\gamma d}$ =	1.00	
Force inclination factors:			
	F_{ci} =	1.00	
	F_{qi} =	1.00	
	$F_{\gamma i}$ =	1.00	
Ultimate bearing capacity	q_u =	280.21	kPa
Factor of safety in use	FS =	2.50	
Allowable bearing capacity	q_a =	112.08	kPa
Non-overstrength soil stress	q =	59.00	kPa
Overstrength soil stress	q_{os} =	69.00	kPa
Strength ratio non-overstrength	r_b =	0.53	
Strength ratio overstrength	$r_{b.os}$ =	0.51	

CONCLUSION: SATISFIABLE

C. OVERTURNING STABILITY

Overturning stability of the foundation of blast resisting walls shall be controlled. Since the analysis involves seismic forces, we will use the static equivalent method to estimate the seismic forces since we analyse this manually.

According to SNI 1726:2019 section 7.8, the seismic lateral force is given by a map

$$Q_E : [0,h] \to \mathbb{R}$$

where h is the total height of the wall, and is defined in accordance with the same section by

$$\forall u \in [0,h] : \; Q_E(u) := C_s W(u) ,$$

where C_s is the seismic response coefficient, given in accordance with SNI 1726:2019 section 7.8.1, and

$$W : [0,h] \to \mathbb{R}$$

is the weight map of the wall, defined by

$$\forall u \in [0,h] : \; W(u) := \int_u^h A_w \gamma_c \, \mathrm{d}x$$

with A_w being the cross section of the wall and γ_c being the wall density. In our case, A_w and γ_c are constants, thus,

$$\forall u \in [0,h] : \; W(u) := A_w \gamma_c \int_u^h \mathrm{d}x = A_w \gamma_c (h-u) .$$

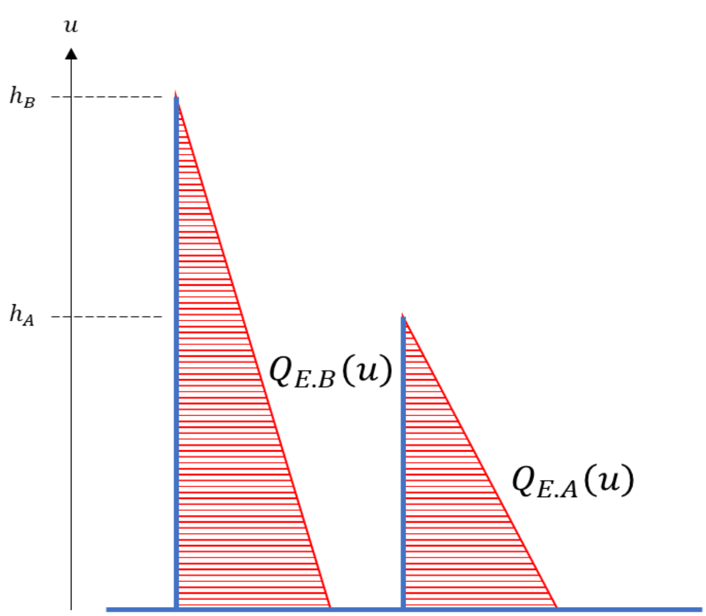
On the other hand, C_s is given by

$$C_s = \frac{I_e S_{DS}}{R}$$

where I_e is the importance factor, S_{D1} is the short period spectral parameter and R is the response modification coefficient.

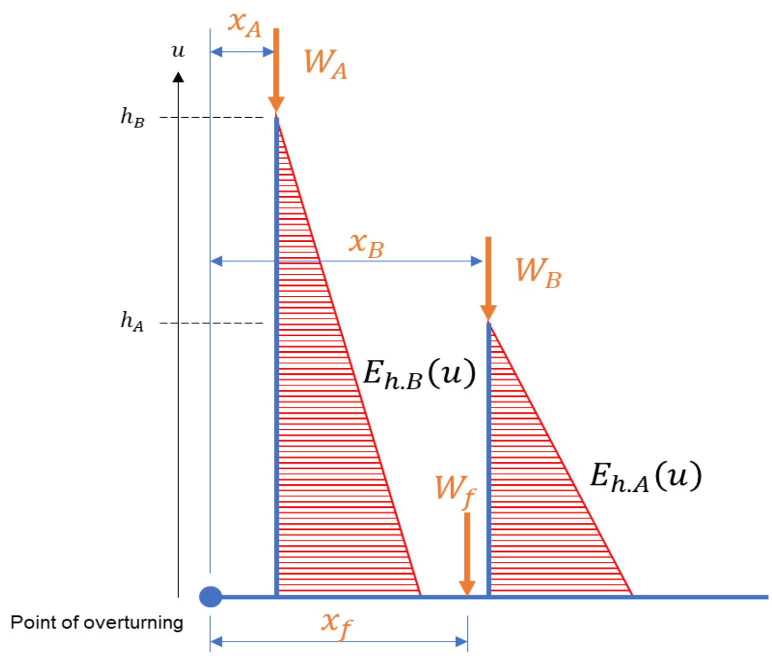
Based on our designation, the distribution of the seismic lateral force is illustrated in the following figure:

Blast resisting walls and the foundation are modelled as frames.



Actual Lateral Earthquake Force Distribution

The overturning scheme is presented in accordance with the following diagram:



In the diagram above, W_A, W_B, W_T are the gravitational forces consisting the combination of the dead loads and the vertical earthquake E_v. While $E_{h.A}$ and $E_{h.B}$ are the earthquake horizontal loads of BRW-A and BRW-B respectively, determined in accordance with SNI 1726:2019 section 7.4. Then the overturning shall be evaluated following the seismic load combinations.

In case of non-overstrength seismic, Eh.A and Eh.B is given in accordance with SNI 1726:2019 section 7.4.2.1 by

$$\forall \alpha \in \{A, B\} \forall u \in [0, h] : \; E_{h.\alpha}(u) = \rho Q_{E.\alpha}(u) \,.$$

In analysing overturning, we need to calculate the moments causing overturning. And these moments are from Eh.A and Eh.B which are given by

$$\begin{aligned}
M_o &:= C\left(\frac{1}{h_A}\int_0^{h_A} uE_{h.A}(u)\,\mathrm{d}u + \frac{1}{h_B}\int_0^{h_B} uE_{h.B}(u)\,\mathrm{d}u\right) \\
&= C\left(\frac{1}{h_A}\int_0^{h_A} u\rho Q_{E.A}(u)\,\mathrm{d}u + \frac{1}{h_B}\int_0^{h_B} u\rho Q_{E.B}(u)\,\mathrm{d}u\right) \\
&= C\cdot\rho\cdot C_s\left(\frac{1}{h_A}\int_0^{h_A} uA_{w.A}\gamma_c(h_A - u)\,\mathrm{d}u + \frac{1}{h_B}\int_0^{h_B} uA_{w.B}\gamma_c(h_B - u)\,\mathrm{d}u\right) \\
&= C\cdot\rho\cdot C_s\cdot\gamma_c\left(\frac{A_{w.A}}{h_A}\int_0^{h_A} u(h_A - u)\,\mathrm{d}u + \frac{A_{w.B}}{h_B}\int_0^{h_B} u(h_B - u)\,\mathrm{d}u\right) \\
&= C\cdot\rho\cdot C_s\cdot\gamma_c\left(\frac{A_{w.A}}{h_A}\left[\frac{h_A u^2}{2} - \frac{u^3}{3}\right]_0^{h_A} + \frac{A_{w.B}}{h_B}\left[\frac{h_B u^2}{2} - \frac{u^3}{3}\right]_0^{h_B}\right) \\
&= C\cdot\rho\cdot C_s\cdot\gamma_c\left(\frac{A_{w.A}}{h_A}\frac{h_A^3}{6} + \frac{A_{w.B}}{h_B}\frac{h_B^3}{6}\right) \\
&= C\cdot\frac{\rho\cdot C_s\cdot\gamma_c}{6}(A_{w.A}h_A^2 + A_{w.B}h_B^2)
\end{aligned}$$

where C is the factor for the corresponding load combination.

While the resisting moment is given by

$$M_r := \sum_{\alpha\in\{A,B,f\}} x_\alpha W_\alpha = \sum_{\alpha\in\{A,B,f\}} x_\alpha (C_1 D_\alpha + C_2 E_{v_\alpha}) \,.$$

Where C1 is the factor for the dead load and C2 is the factor for Ev with respect to the corresponding load combination.

The foundation is considered stable against overturning if

$$FS_o := \frac{M_r}{M_o} \geq 1.1 \,.$$

The calculation is presented as follows:

Walls and Foundation Specification

Cross section of BRW-A	$A_{w.A}$ =	1.63	m^2
Cross section of BRW-B	$A_{w.B}$ =	2.25	m^2
Total height of BRW-A	h_A =	3.00	m
Total height of BRW-B	h_B =	6.00	m
Foundation thickness	t_f =	0.40	m
Foundation area	A_f =	36.00	m^2
Concrete density	γ_c =	24.00	kN/m^3
Weight of BRW-A	D_A =	117.00	kN
Weight of BRW-B	D_B =	324.00	kN
Weight of foundation	D_f =	345.60	kN
Distance of overturning point to locations of loads:			
	x_A =	1.13	m
	x_B =	2.38	m
	x_f =	2.00	m

General Seismic Parameters

Shor period response acceleration	S_{DS} =	0.60	g
Importance factor	I_e =	1.00	
Response modification parameter	R =	5.00	
Redundancy factor	ρ =	1.00	
Overstrength factor	Ω_0 =	2.50	
Seismic response coefficient	C_s =	0.12	
Vertical earthquake:			
For BRW-A	E_{vA} =	13.92	kN
For BRW-B	E_{vB} =	38.56	kN
For foundation	E_{vf} =	41.13	kN

Load Combination: 1.0D + 0.7Ev + 0.7Eh

Load factor for dead load	C_1 =	1.00
Load factor for vertical earthquake	C2 =	0.70
Load factor for horizontal earthquake	C =	0.70

Resisting moment	M_r =	1724.97	kNm
Overturning moment	M_o =	31.86	kNm
Overturning factor of safety	**Fs_o =**	**54.14**	

Load Combination: 0.6D - 0.7Ev + 0.7Eh

Load factor for dead load	C_1 =	0.60	
Load factor for vertical earthquake	C2 =	-0.70	
Load factor for horizontal earthquake	C =	0.70	
Resisting moment	M_r =	822.75	kNm
Overturning moment	M_o =	31.86	kNm
Overturning factor of safety	**Fs_o =**	**25.82**	

CONCLUSION: SATISFIABLE

Footing Reinforcement of Blast Resisting Walls

A. INTRODUCTION

This report covers the analysis of the reinforcement of shallow foundations.

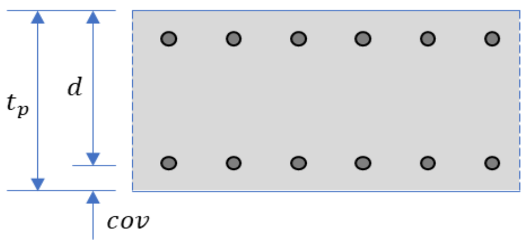
B. MATERIAL PROPERTIES

Concrete compressive strength	f_c' =	29.00	MPa
Rebar yield strength	f_y =	420.00	MPa
Whitney factor	β_1 =	0.84	

Whitney factor describes the rectangular stress block equivalent of concrete structure. The value of the factor is given in accordance with SNI 2847:2019 section 22.2.2.4.3 as follows:

$$\beta_1 = \begin{cases} 0.85 & : 17\text{ MPa} \leq f_c' \leq 28\text{ MPa} \\ 0.85 - \frac{0.05(f_c'-28)}{7} & : 28\text{ MPa} < f_c' < 55 \\ 0.65 & : f_c' \geq 55\text{ MPa} \end{cases}$$

C. MECHANICAL AND GEOMETRICAL PROPERTIES



Footing thickness	t_p =	400.00	mm
Rebar diamater (in x-direction)	D_x =	22.00	mm
Rebar diamater (in y-direction)	D_y =	22.00	mm
Clear cover	cov =	50.00	mm
Dist. from compressive fibre to centre of tensile rebar (x)	d_x =	317.00	mm
Dist. from compressive fibre to centre of tensile rebar (y)	d_y =	339.00	mm
Distance between adjacent rebars in use:			
For x-direction	s_x =	100.00	mm
For y-direction	s_y =	150.00	mm

D. DUCTILITY REQUIREMENT OF THE REINFORCEMENT

SNI 2847:2019 section 8.6.1.1 asserts that the minimum reinforcement of a concrete slab shall be of the following:

$$A_{s,\min} = \begin{cases} 0.0020A_g & : f_y < \ 420\text{ MPa} \\ \max\left\{\frac{0.00178\cdot 420}{f_y}A_g\,,\ 0.0014A_g\right\} & : f_y \geq \ 420\text{ MPa} \end{cases}$$

And the reinforcement also cannot exceed the following requriement:

$$A_{s,\max} = 0.75\rho_b A_g$$

$$\rho_b = \frac{0.85 f'_c \beta_1}{f_y} \frac{600}{f_y + 600}$$

The ductility of the Reinforcement in use is satisfiable if

$$A_{s,\min} \leq A_s \leq A_{s,\max}$$

holds.

D.1. Ductility in X-Direction

Gross sectional area	A_g =	317000.00	mm^2/m
Reinforcement sectional area	A_s =	3801.33	mm^2/m
Required minimum rebars	$A_{s,min}$ =	564.26	mm^2/m
Required maximum rebars	$A_{s,max}$ =	6918.20	mm^2/m

CONCLUSION: SATISFIABLE

D.2. Ductility in Y-Direction

Gross sectional area	A_g =	339000.00	mm^2
Reinforcement sectional area	A_s =	2534.22	mm^2
Required minimum rebars	$A_{s,min}$ =	603.42	mm^2
Required maximum rebars	$A_{s,max}$ =	7398.33	mm^2

CONCLUSION: SATISFIABLE

E. FLEXURAL BENDING CAPACITY OF REINFORCEMENTS

The reinforcement in use shall have a sufficient capacity against the ultimate bending moment in the footing foundation. The nominal bending capacity of the reinforced concrete structure is given by

$$M_n = f_y A_s \left(d - \frac{a}{2}\right)$$

where

$$a = \frac{f_y A_s}{0.85 f'_c b}\,.$$

The bending capacity is satisfiable if the expression

$$r_b = \frac{M_u}{\phi M_n}$$

is satisfied, where Mu is the ultimate bending moment and ϕ = 0.9.

E.1. Bending Capacity in X-Direction

Ultimate bending moment (overstrength considered)	M_u =	297385220.00	Nmm/m
Equivalent thickness of concrete compressive fibre	a =	64.77	mm
Nominal moment	M_n =	454404929.09	Nmm/m
Strength ratio	r_b =	0.73	

CONCLUSION: SATISFIABLE

Rebars in use: D22-100

E.2. Bending Capacity in Y-Direction

Ultimate bending moment (overstrength considered)	M_u =	257331270.00	Nmm/m
Equivalent thickness of concrete compressive fibre	a =	43.18	mm
Nominal moment	M_n =	337842519.39	Nmm/m
Strength ratio	r_b =	0.85	

CONCLUSION: SATISFIABLE

Rebars in use: D22-150

Reinforcement of BRW-A

A. INTRODUCTION

This report covers the analysis of the reinforcement of shallow foundations.

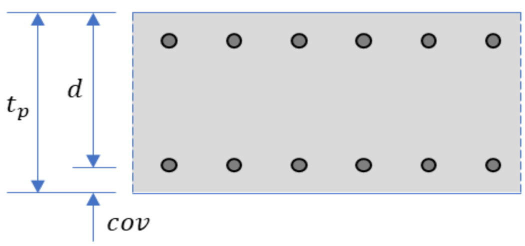
B. MATERIAL PROPERTIES

Concrete compressive strength	f_c' =	29.00	MPa
Rebar yield strength	f_y =	420.00	MPa
Whitney factor	β_1 =	0.84	

Whitney factor describes the rectangular stress block equivalent of concrete structure. The value of the factor is given in accordance with SNI 2847:2019 section 22.2.2.4.3 as follows:

$$\beta_1 = \begin{cases} 0.85 & : 17 \text{ MPa} \le f_c' \le 28 \text{ MPa} \\ 0.85 - \frac{0.05(f_c'-28)}{7} & : 28 \text{ MPa} < f_c' < 55 \\ 0.65 & : f_c' \ge 55 \text{ MPa} \end{cases}$$

C. MECHANICAL AND GEOMETRICAL PROPERTIES



Footing thickness	t_p =	250.00	mm
Rebar diamater (in x-direction)	D_x =	13.00	mm
Rebar diamater (in y-direction)	D_y =	16.00	mm
Clear cover	cov =	50.00	mm
Dist. from compressive fibre to centre of tensile rebar (x)	d_x =	177.50	mm
Dist. from compressive fibre to centre of tensile rebar (y)	d_y =	192.00	mm
Distance between adjacent rebars in use:			
For x-direction	s_x =	300.00	mm
For y-direction	s_y =	200.00	mm

D. DUCTILITY REQUIREMENT OF THE REINFORCEMENT

SNI 2847:2019 section 8.6.1.1 asserts that the minimum reinforcement of a concrete slab shall be of the following:

$$A_{s,\min} = \begin{cases} 0.0020 A_g & : f_y < \ 420 \text{ MPa} \\ \max\left\{\frac{0.00178 \cdot 420}{f_y} A_g \ , \ 0.0014 A_g\right\} & : f_y \ge \ 420 \text{ MPa} \end{cases}$$

And the reinforcement also cannot exceed the following requriement:

$$A_{s,\max} = 0.75 \rho_b A_g$$

$$\rho_b = \frac{0.85 f'_c \beta_1}{f_y} \frac{600}{f_y + 600}$$

The ductility of the Reinforcement in use is satisfiable if

$$A_{s,\min} \leq A_s \leq A_{s,\max}$$

holds.

D.1. Ductility in X-Direction

Gross sectional area	A_g =	177500.00	mm^2
Reinforcement sectional area	A_s =	442.44	mm^2
Required minimum rebars	$A_{s,min}$ =	315.95	mm^2
Required maximum rebars	$A_{s,max}$ =	3873.76	mm^2

CONCLUSION: SATISFIABLE

D.2. Ductility in Y-Direction

Gross sectional area	A_g =	192000.00	mm^2
Reinforcement sectional area	A_s =	1005.31	mm^2
Required minimum rebars	$A_{s,min}$ =	341.76	mm^2
Required maximum rebars	$A_{s,max}$ =	4190.20	mm^2

CONCLUSION: SATISFIABLE

E. FLEXURAL BENDING CAPACITY OF REINFORCEMENTS

The reinforcement in use shall have a sufficient capacity against the ultimate bending moment in the footing foundation. The nominal bending capacity of the reinforced concrete structure is given by

$$M_n = f_y A_s \left(d - \frac{a}{2}\right)$$

where

$$a = \frac{f_y A_s}{0.85 f'_c b} .$$

The bending capacity is satisfiable if the expression

$$r_b = \frac{M_u}{\phi M_n}$$

is satisfied, where Mu is the ultimate bending moment and ϕ = 0.9.

E.1. Bending Capacity of Horizontal Rebars

Ultimate bending moment (overstrength considered)	M_u =	13094520.00	Nmm/m
Equivalent thickness of concrete compressive fibre	a =	7.54	mm
Nominal moment	M_n =	32283547.86	Nmm/m
Strength ratio	r_b =	0.45	

CONCLUSION: SATISFIABLE

Rebars in use: D13-300

E.2. Bending Capacity of Vertical Rebars

Ultimate bending moment (overstrength considered)	M_u =	30881920.00	Nmm/m
Equivalent thickness of concrete compressive fibre	a =	17.13	mm
Nominal moment	M_n =	77451979.09	Nmm/m
Strength ratio	r_b =	0.44	

CONCLUSION: **SATISFIABLE**

Rebars in use: **D16-200**

Reinforcement of BRW-B

A. INTRODUCTION

This report covers the analysis of the reinforcement of shallow foundations.

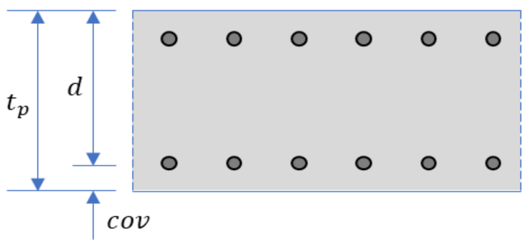
B. MATERIAL PROPERTIES

Concrete compressive strength	f_c' =	29.00	MPa
Rebar yield strength	f_y =	420.00	MPa
Whitney factor	β_1 =	0.84	

Whitney factor describes the rectangular stress block equivalent of concrete structure. The value of the factor is given in accordance with SNI 2847:2019 section 22.2.2.4.3 as follows:

$$\beta_1 = \begin{cases} 0.85 & : 17 \text{ MPa} \leq f_c' \leq 28 \text{ MPa} \\ 0.85 - \frac{0.05(f_c' - 28)}{7} & : 28 \text{ MPa} < f_c' < 55 \\ 0.65 & : f_c' \geq 55 \text{ MPa} \end{cases}$$

C. MECHANICAL AND GEOMETRICAL PROPERTIES



Footing thickness	t_p =	250.00	mm
Rebar diamater (in x-direction)	D_x =	13.00	mm
Rebar diamater (in y-direction)	D_y =	16.00	mm
Clear cover	cov =	50.00	mm
Dist. from compressive fibre to centre of tensile rebar (x)	d_x =	177.50	mm
Dist. from compressive fibre to centre of tensile rebar (y)	d_y =	192.00	mm
Distance between adjacent rebars in use:			
For x-direction	s_x =	300.00	mm
For y-direction	s_y =	200.00	mm

D. DUCTILITY REQUIREMENT OF THE REINFORCEMENT

SNI 2847:2019 section 8.6.1.1 asserts that the minimum reinforcement of a concrete slab shall be of the following:

$$A_{s,\min} = \begin{cases} 0.0020 A_g & : f_y < \ 420 \text{ MPa} \\ \max\left\{\frac{0.00178 \cdot 420}{f_y} A_g \ , \ 0.0014 A_g\right\} & : f_y \geq \ 420 \text{ MPa} \end{cases}$$

And the reinforcement also cannot exceed the following requriement:

$$A_{s,\max} = 0.75 \rho_b A_g$$

$$\rho_b = \frac{0.85 f'_c \beta_1}{f_y} \frac{600}{f_y + 600}$$

The ductility of the Reinforcement in use is satisfiable if

$$A_{s,\min} \leq A_s \leq A_{s,\max}$$

holds.

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The reinforcement in use shall have a sufficient capacity against the ultimate bending moment in the footing foundation. The nominal bending capacity of the reinforced concrete structure is given by

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The bending capacity is satisfiable if the expression

$$r_b = \frac{M_u}{\phi M_n}$$

is satisfied, where Mu is the ultimate bending moment and ϕ = 0.9.

E.1. Bending Capacity of Horizontal Rebars

Ultimate bending moment (overstrength considered)	M_u =	15316440.00	Nmm/m
Equivalent thickness of concrete compressive fibre	a =	7.54	mm
Nominal moment	M_n =	32283547.86	Nmm/m
Strength ratio	r_b =	0.53	

CONCLUSION: SATISFIABLE

Rebars in use: D13-300

E.2. Bending Capacity of Vertical Rebars

Ultimate bending moment (overstrength considered)	M_u =	58006500.00	Nmm/m
Equivalent thickness of concrete compressive fibre	a =	17.13	mm
Nominal moment	M_n =	77451979.09	Nmm/m
Strength ratio	r_b =	0.83	

CONCLUSION: **SATISFIABLE**

Rebars in use: **D16-200**