PROECCO

02

A
CONSTRUCTION
MANUAL ON HOW
TO BUILD
A ROWLOCK BOND
HOUSE

RowLock Bond
STRUCTURAL PRINCIPLES

BLUEPRINT FOR ROW LOCK BOND CONSTRUCTION - Structural principles

INTRO

This **CONSTRUCTION MANUAL** is a comprehensive step-by-step practical guide for construction supervisors, masons, builders, architects and engineers on how to build a multi-story building using the Rowlock Bond (RLB) technology. The manual is presented in three volumes, covering the **01 RLB principles**, the **02 Structural principles** and **03 construction process**. Each volume includes a comprehensive list of annexes covering quality control, specifications and useful tools, to be used to verify the design, structural calculations or construction works against the set standards.

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01

EXECUTIVE SUMMARY

Overview

Buildings featuring the Row Lock Bond technology utilize a combination of load bearing cavity masonry and concrete frame to achieve the desired anti-seismic properties required by the building code.

A. The masonry:

The Row Lock Bond masonry revolves around the RLB fired clay brick. This specific type of brick is produced industrially or semi-industrially and has a minimum strength of 10 MPa, allowing it to be used for load bearing constructions and specifically for cavity masonries like the Row Lock Bond.

B. The anti-seismic reinforced concrete frame:

The masonry cavities allow for a concrete frame to be built within the load bearing walls, perfectly integrating with the building shape. The masonry and the frame are built simultaneously, forming a solid bond and creating a seismic-resistant compound structure.

This structural design guidance analyses the technology and provides structural engineers with methods and tools to verify the compliance of a given project to the Eurocode and Rwanda building code.
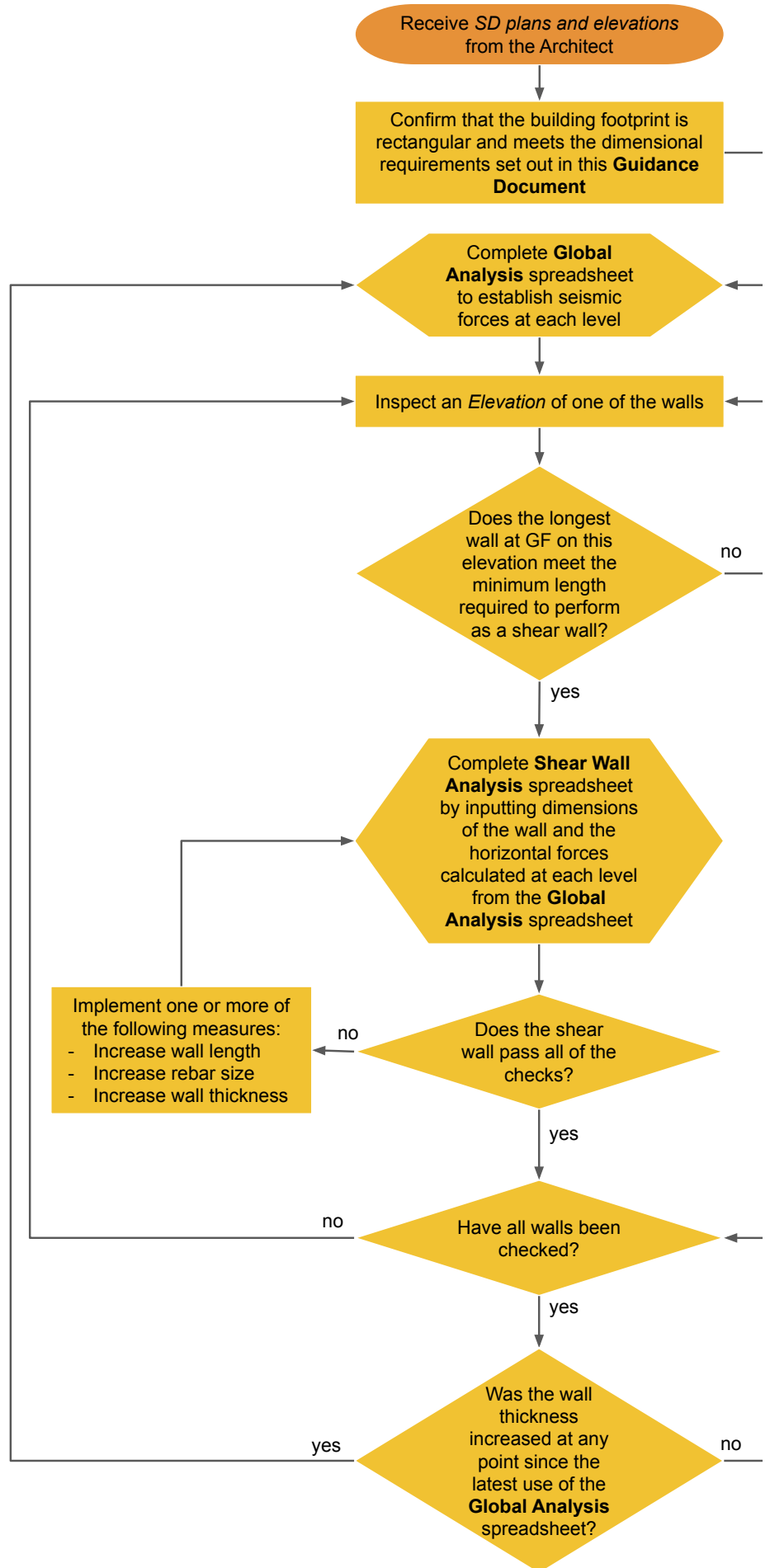
This is done by means of:

– Workflow chart, directing the user through the structural design steps.

– Structural design guidance

– Worked example of the Mpazi Rehousing Project with Rowlock Bond technology

– An Excel spreadsheet to verify the outcomes of the proposed solution.

02

DESIGN FLOW CHART

Sequential RLB
structure design steps

Receive *SD plans and elevations* from the Architect

Confirm that the building footprint is rectangular and meets the dimensional requirements set out in this **Guidance Document**

Complete **Global Analysis** spreadsheet to establish seismic forces at each level

Inspect an *Elevation* of one of the walls

Does the longest wall at GF on this elevation meet the minimum length required to perform as a shear wall?

no

yes

Complete **Shear Wall Analysis** spreadsheet by inputting dimensions of the wall and the horizontal forces calculated at each level from the **Global Analysis** spreadsheet

Implement one or more of the following measures:
- Increase wall length
- Increase rebar size
- Increase wall thickness

no

Does the shear wall pass all of the checks?

yes

no

Have all walls been checked?

yes

Was the wall thickness increased at any point since the latest use of the **Global Analysis** spreadsheet?

yes

no

```
┌─────────────────────────────┐
│  Decide on floor and roof   │
│  structural preference for  │
│  each level (MaxSpan or     │
│  Timber)                    │
└─────────────────────────────┘

┌─────────────────────────────┐
│ Reduce opening widths in    │
│ the wall until the minimum  │
│ length of shear wall has    │
│ been achieved               │
└─────────────────────────────┘

┌─────────────────────────────┐
│ Model a 2D moment frame in  │
│ a suitable structural       │
│ analysis software following │
│ the rules provided in the   │
│ Guidance Document and       │
│ analyse for the forces      │
│ calculated from the Global  │
│ Analysis spreadsheet        │
└─────────────────────────────┘
```

Compare the forces found in each of the members at ground floor level with the **Interaction Diagrams** for the relevant member type

Assume 27.5cm Thick Members

Is it possible to increase from a 21cm wall to a 27.5cm wall — yes / no

Do the forces in each members fall within the envelope of the interaction diagram? — no

Is the design assuming a 21cm wall? — yes / no

yes

Take note of the required rebar size in each element

Structural design of the wall system is complete.

03

STRUCTURAL DESIGN GUIDANCE

A. Key assumptions, limitations and requirements

A.1 Design Codes

Eurocodes have been the primary reference documents when developing the structural guidance and tools for the Rowlock Bond system. However, other design guidance has been used for supplemental information and is referenced accordingly.

A.2 Material Properties

The following material properties have been assumed as a default for the purposes of structural design:

Row Lock Bond masonry

– Normalized mean compressive strength of brick, f_b = 10 Mpa

– Compressive strength of the mortar, f_m = 10 Mpa

– Characteristic compressive strength of masonry, f_k = 5 MPa

– Modulus of Elasticity, E = 1000 f_k = 5000 MPa

– Density, ρ = 20 kN/m^3

– Partial factor for masonry, γ_m = 2.0

– Bricks are considered to be Group 1

  - Volume of all holes are less than 25% of the total volume of the brick

  - Volume of a single hole is less than 12.5% of the total volume

– Bricks are considered to be Category I

  - Units have a declared compressive strength with a probability of failure to reach it not exceeding 5%.

Concrete

– Grade: C20/25

– Characteristic Cube Strength, $f_{cu,k}$ = 25 MPa

– Characteristic Cylinder Strength, f_k = 20 MPa

– Density, ρ = 25kN/m^3

Reinforcement Bar

– Grade: B500

– Characteristic Yield Strength, f_{yk} = 500 MPa

– Design Strength of reinforcement, f_{yd} = 435 MPa

– Partial factor - steel reinforcement, γ_{ms} = 1.15

Row Lock Bond Housing complex, Mpazi, Kimisagara, Kigali
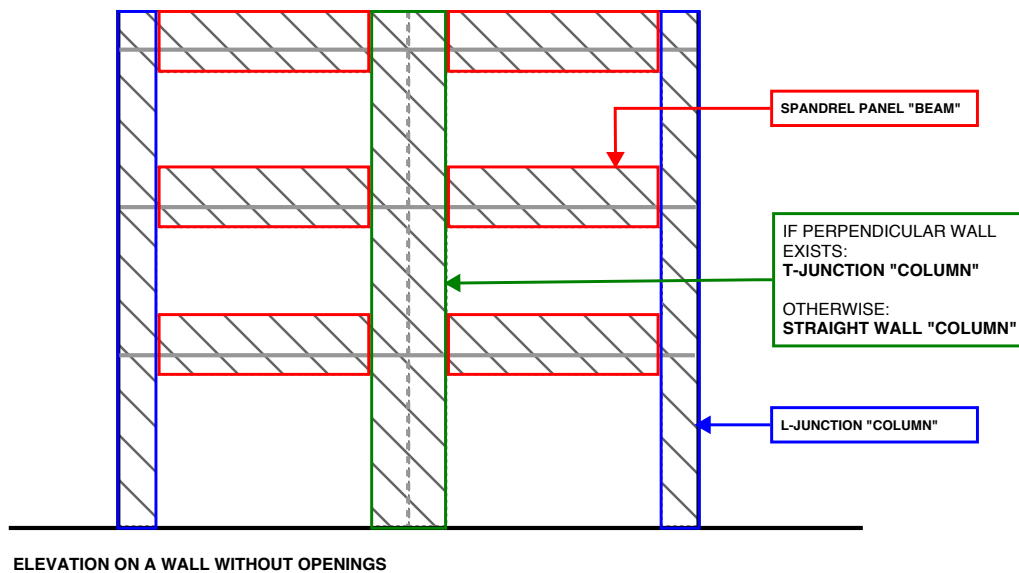
A.3 General Construction Quality and Control

The design methods and guidance included here assume that execution of the construction process is carried out in accordance with EN 1996 Part 2 : Design considerations, selection of materials and execution of masonry including appropriate supervision and inspection. Key requirements are as follows:

– The construction of the masonry walls is being carried out by trained masons who have undergone sufficient training to achieve consistent regularity and results.

– A suitably qualified engineer who is independent from the contractor (i.e. the design engineer) shall be supervising and inspecting the construction on a regular basis, ensuring construction is in line with the design assumptions and details.

– Bricks are saturated with water prior to laying to minimize water absorption from the mortar mix.

– Mortar joints shall be consistently 10mm wide. Mortar shall not be used beyond its workable life.

– Deviation from verticality shall not exceed 20mm in either direction over each storey height, and not more than 50mm over the entire building height.

– Straightness of walls shall not deviate by more than 10mm per meter length, and not more than 50mm over a 10 meter length.

– Cube compression tests are carried out on mortar samples at the beginning of construction to ensure the selected mix is meeting the design requirements. Regular compressive strength testing is carried out on samples from the site mortar to check that the required strengths are being achieved.

– Cube compression tests shall be implemented on concrete samples at the beginning of construction to ensure the selected mix meets the design requirements. Each time concrete is poured on site, at least 6no. cubes must be produced and crushed to confirm consistency throughout the construction.
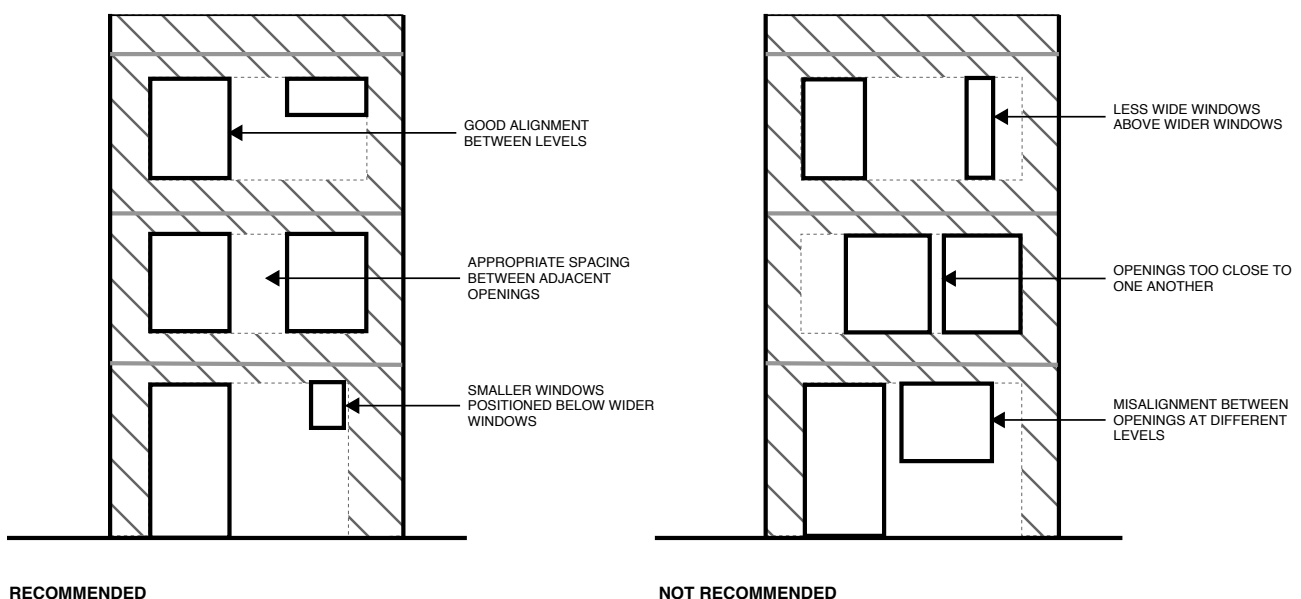
A.4 General Rules around the Positioning of Wall Openings

The Design Tool assumes that the more heavily reinforced wall junctions used in the Rowlock Bond technology i.e. the T-junction, L-junction and Straight Wall "Columns" are continuous over the height of the building and are uncompromised by window, door or service openings. It also assumes that the standard 1.1m deep spandrel panel height is maintained all round the building at each level.

SPANDREL PANEL "BEAM"

IF PERPENDICULAR WALL EXISTS:
T-JUNCTION "COLUMN"

OTHERWISE:
STRAIGHT WALL "COLUMN"

L-JUNCTION "COLUMN"

ELEVATION ON A WALL WITHOUT OPENINGS

Outside of these junction and spandrel elements, the placement of openings is effectively down to the designers discretion. However, there are some good practice rules which are recommended:

– Openings on each level of the building should generally be aligned to minimize the need for vertical load transfer via spandrel panel elements. Vertical load paths should be prioritized wherever possible.

– Sufficient spacing should be left between openings to ensure the wall between does not

GOOD ALIGNMENT BETWEEN LEVELS

APPROPRIATE SPACING BETWEEN ADJACENT OPENINGS

SMALLER WINDOWS POSITIONED BELOW WIDER WINDOWS

LESS WIDE WINDOWS ABOVE WIDER WINDOWS

OPENINGS TOO CLOSE TO ONE ANOTHER

MISALIGNMENT BETWEEN OPENINGS AT DIFFERENT LEVELS
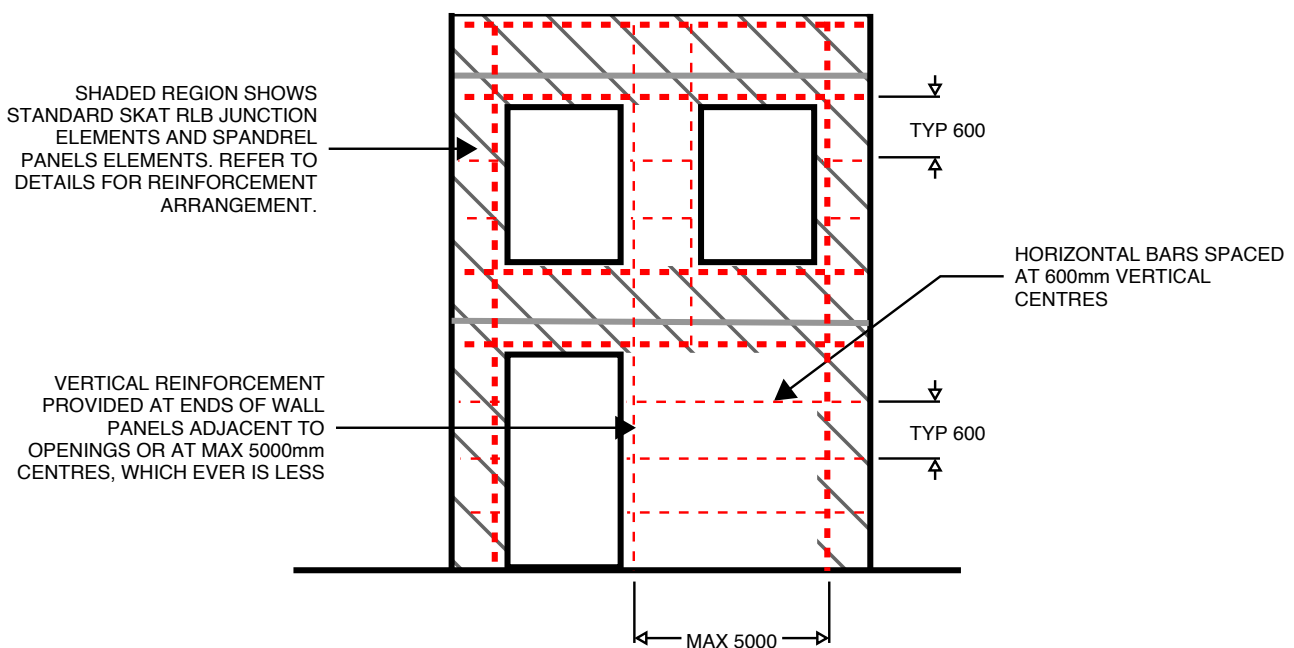
RECOMMENDED

NOT RECOMMENDED

become susceptible to axial buckling when carrying vertical loads from above.

– Ideally, smaller openings should be positioned below wider openings to ensure the best vertical load path.

A.5 Minimum Reinforcement Requirements

– Horizontal reinforcement should be placed in the bed joints or in suitable grooves in the units, with a vertical spacing not exceeding 600 mm.

– The minimum percentage of horizontal reinforcement in the wall, normalized with respect to the gross area of the section, should not be less than 0,05 %

– The vertical reinforcement spread in the wall, as a percentage of the gross area of the horizontal section of the wall, should not be less than 0,08%

– Reinforcing steel bars of not less than 4 mm diameter, bent around the vertical bars at the edges of the wall, should be used

– Vertical reinforcement should be located in pockets, cavities or holes in the units.

– Vertical reinforcements with a cross-sectional area of not less than 200 mm2 should be arranged:

  - at both free edges of every wall element;

  - at every wall intersection;

  - within the wall, in order not to exceed a spacing of 5 m between such reinforcements.

SHADED REGION SHOWS STANDARD SKAT RLB JUNCTION ELEMENTS AND SPANDREL PANELS ELEMENTS. REFER TO DETAILS FOR REINFORCEMENT ARRANGEMENT.

VERTICAL REINFORCEMENT PROVIDED AT ENDS OF WALL PANELS ADJACENT TO OPENINGS OR AT MAX 5000mm CENTRES, WHICH EVER IS LESS

TYP 600

TYP 600

HORIZONTAL BARS SPACED AT 600mm VERTICAL CENTRES

MAX 5000

RECOMMENDED

A.6 Other Limitations of the Design Tool

The Design Tools provided has been developed to facilitate the design of relatively simple buildings with an arrangement based on what may be expected of terraced low-rise affordable housing.

In order to reduce complexity, the Design Tool has been developed with the following limitations:

– Designs structures using the Rowlock Bond system only.

– Designs structures up to three storeys (G+2)

– Designs buildings which are rectangular in plan with no re-entrant corners, refer to Section 3.3.1.

– Based on the design of a "Unit" which has structural walls on four sides with floor/roof structures spanning across the shorter direction.

– An additional Spine Wall can be provided as an optional extra. The Spine Wall is located halfway between the Front and Back Walls.

– A row of up for five identical Units can be considered as a single structure.
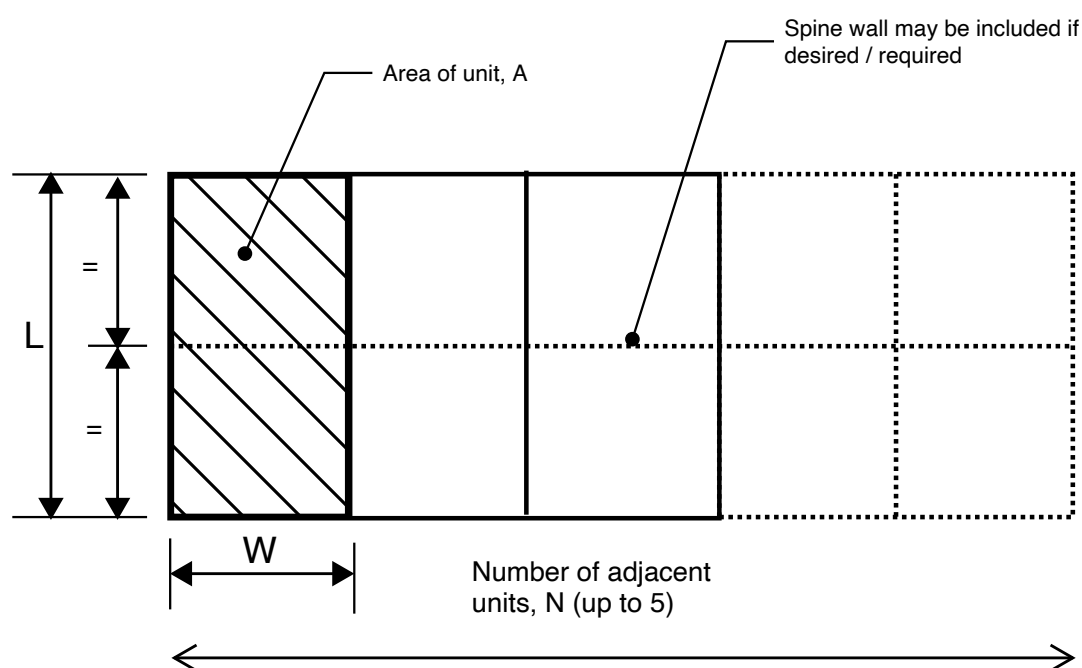
– A wall thickness of either 210mm or 275mm can be selected, aligning with the two Rowlock Bond coursing patterns. Refer to the Construction section of the guidance.

NOTE: that when using 210mm thick walls, while the structure may be justified numerically, the recommended minimum thickness for reinforced masonry walls in EC8 is 240mm and therefore the RLB design does not strictly comply.

– Typical Rowlock Bond reinforcement arrangements are applied around junctions and within spandrel panels, Refer back to Construction section of the guidance

This does not mean that the Rowlock Bond is unsuitable for conditions outside these constraints, only that in the case the building arrangement does not comply then the Design Engineer must revert to their own analysis methods to establish forces acting on the structure. From there, the assumptions, methods and guidance provided in Section 4 and the Worked Example Calculation may be followed to assess the capacity of individual structural elements.
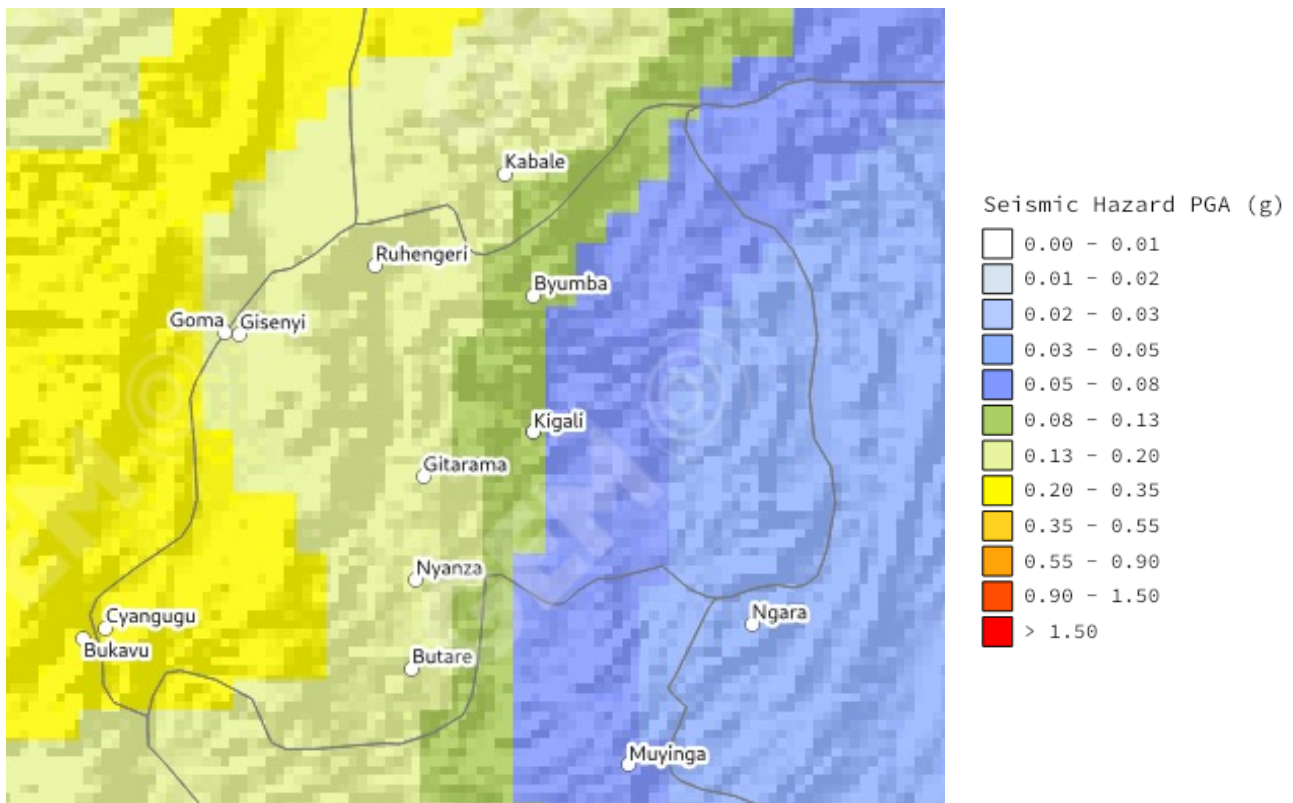
Rwanda seismicity map

B. Seismic design approach for global analysis

B.1 Seismicity of Rwanda

Rwanda, due to its vicinity to the East African Rift System, is widely considered an area of moderate seismic activity. This is demonstrated in various published information and articles including local codes and regulations, such as the Rwanda Building Code (RBC), and reputable international open-source data models such as the Global Earthquake Model (GEM) - see excerpt above of the GEM Global Seismic Hazard map which displays the Peak Ground Accelerations (pga) with a 10% probability of being exceeded in a 50-year period computed for consistent reference ground conditions. These pga values are computed using national and regional probabilistic seismic hazard models developed by various institutions and projects, and by GEM Foundation scientists. These models are developed through statistical analysis of the locations and magnitudes of previous earthquakes in the region, combined with knowledge of how ground motions reduce with distance from the earthquake.

B.2 Establishing the Design Ground Acceleration

According to Eurocode 8 the Design Ground Acceleration, Sd, is based on a number of factors associated with both the site conditions and building typology.

– A response spectrum shape must be selected. There are two standard response spectra profiles provided in Eurocode 8 - Type 1 is assigned to more seismically active regions, specifically when the earthquakes that contribute most to the seismic hazard have a magnitude greater than 5.5 on the Richter Scale. Where this is not the case, Response
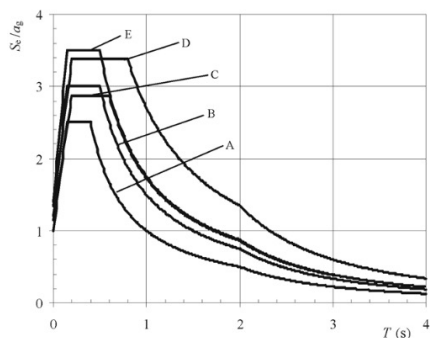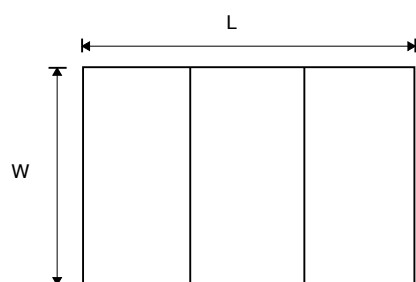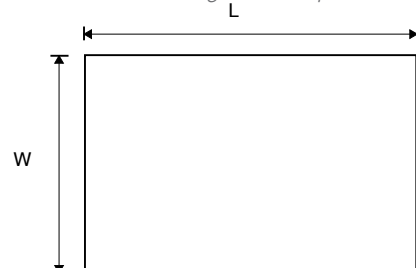
Type 1 elastic response spectra
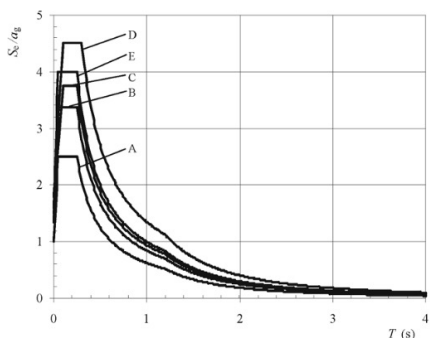
Type 2 elastic response spectra

Spectra Type 2 may be used. Due to the range of seismicity across the country, it is not immediately clear which type of spectra best applies to Rwanda, and therefore it comes down to the Engineer's discretion of which one to use in design. Owing to the low-rise nature of buildings covered by the design tool, it is assumed that the fundamental period of the building will always fall on plateau of the spectra, as such a multiplier of 2.5 to the pga has been adopted as the default value.

– The ground conditions found on the site must be categorized against the Soil Types listed in EC8, Table 3.1. This, combined with the Response Spectra Type selected, corresponds to a Soil Factor which is used as a multiplier of the pga. The categorization of the site specific ground conditions is down to the Engineer's discretion.

– An Importance Factor must be selected. The Eurocode defines Important Class based on the building use, with residential buildings falling into Importance Class 2 according to EC8, Table 4.3. Which returns an Importance Factor of 1.0 which is applied as a multiplier to the pga.

– The ductility of the primary building material of the lateral stability system is taken into account by assigning a Ductility Factor to the structure. For reinforced masonry structures, a ductility factor of 2.0 may be adopted based on EC8, Table 9.1.

B.3 Equivalent Static Force (ESF) Method

The Equivalent Static Force (ESF) method is a simplified analysis technique to substitute the effect of dynamic loading of an expected earthquake by a static force distributed laterally on a structure for design purposes.

The ESF method has been adopted in the design tools provided in order to minimize the necessity for structural analysis softwares, as would be required for more complex design methods such as Response Spectrum Analysis (RSA). However, the forces calculated through the Equivalent Static Force method tend to be conservative and therefore, if a structure does not appear to adequately perform through the use of these design tools, it does not necessarily mean that the structure cannot be demonstrated to perform through other design methods.

B.3.1 Regularity in Plan and Elevation

The ESF method assumed that the building is "regular" in

Building footprint - rectangular shape


$\frac{L}{W} < 4$

Length to width Ratio

both plan and elevation. Building regularity is defined through a number of parameters, but generally aligns with the building being as close to symmetrical as possible in terms of both mass and stiffness distribution in each orthogonal direction.

Key indicators include:

- - The building footprint must be rectangular in shape.
- - Plan Length to Width Ratio, should be less than 4.
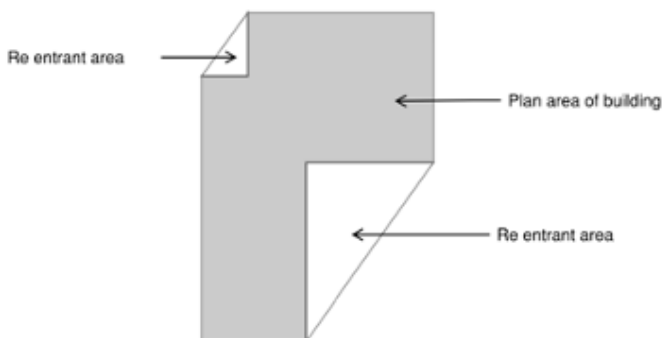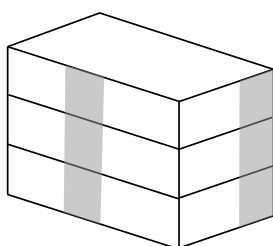- - The building is constructed to an open, flat site with uniform foundation level.
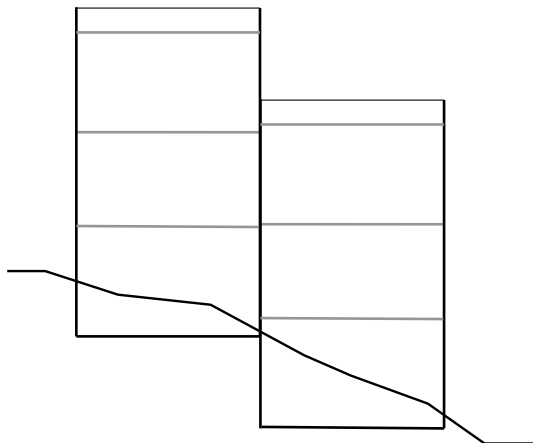
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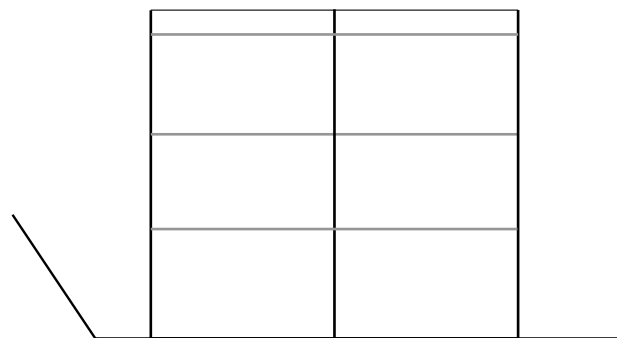
RECOMMENDED

– Re-entrant corners should be limited if not avoided all together

– Lateral stability elements should be continuous through the building
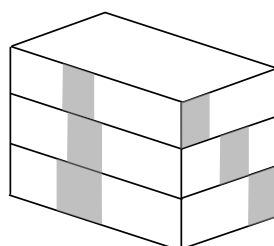
OK   NOT OK

– Lateral stability elements should be distributed in such a way that the center of stiffness roughly aligns with the center of mass.
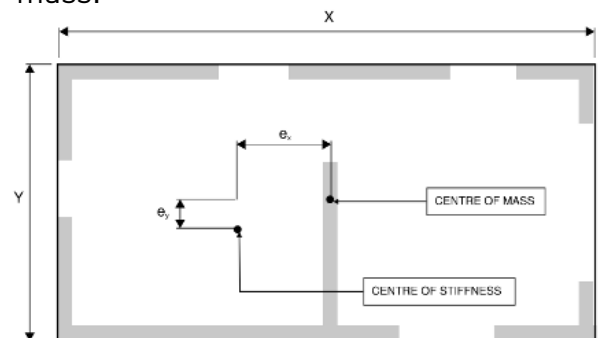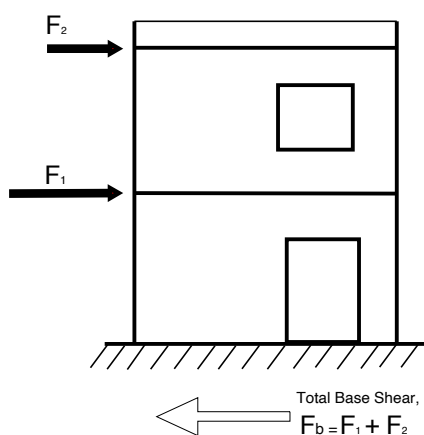

$$\frac{e_x}{X} < 0.1 \qquad \frac{e_y}{Y} < 0.1$$

– In elevation, there should be no significant step backs at each floor level.

$$0 < \frac{L_1 - L_2}{L_1} <= 0.20$$
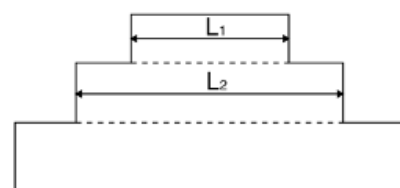
Total Base Shear,
$F_b = F_1 + F_2$

B.3.2 Total Base Shear and Force Distribution

When applying the ESF method, the Total Base Shear is calculated by multiplying the mass of the building by the Design Seismic Acceleration. This force is then distributed over the height of the building based on the assumption that the building responds to the earthquake in its fundamental lateral mode. This assumption is reliant on the building being relatively low-rise and regular in plan and elevation.

The distribution of forces at each level of the building is a function of the total mass acting at that level, the height of that level above the ground, and its proportion of the overall building mass.

The structure is evaluated in two orthogonal horizontal directions parallel to the main axes of the building, and the lateral stability elements designed to resist forces in each of these directions individually but not simultaneously.

SEISMIC FORCE LOAD CASE 1

SEISMIC FORCE LOAD CASE 2


Flexible diaphragm

Rigid diaphragm
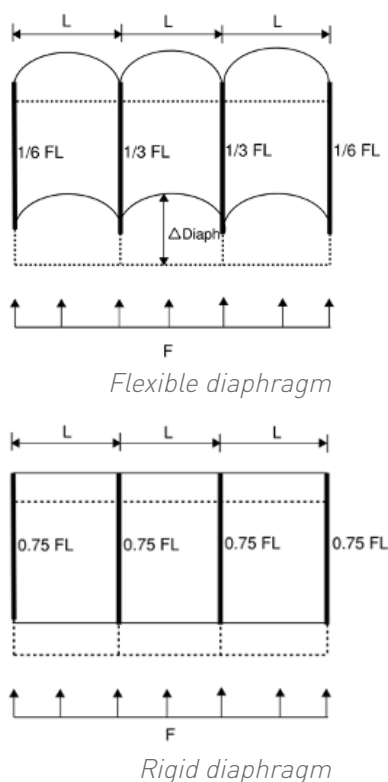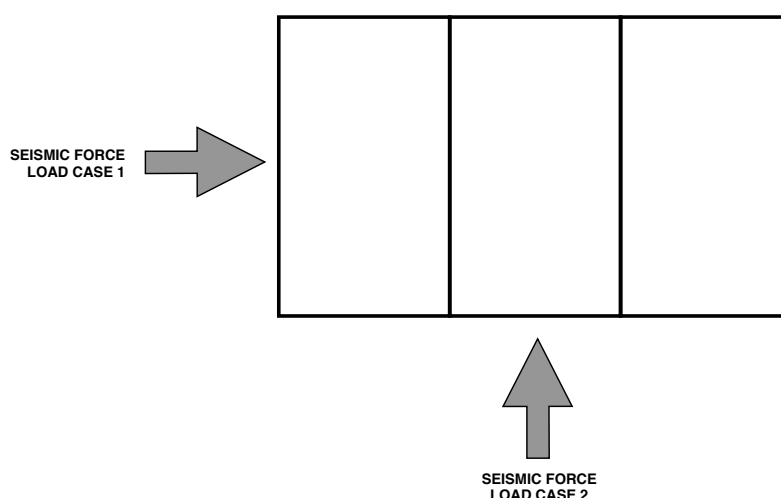
The Design Tool assumes that a flexible diaphragm is provided at each floor level. The typical floor and roof construction systems covered earlier in this design guidance are considered to align with general characteristics of flexible diaphragms.

– For timber floors, it is important that some form of sheathing system is applied to the main spanning elements, either in the form of plywood or other engineered sheet or by the provision of diagonal timber boards.

– For the MaxSpan system, the thickness of concrete above the clay pots is less than 70mm which is stated by EC8 as being the minimum requirement for achieving a rigid diaphragm, therefore the assumption is that this floor system performs as a flexible diaphragm.

Note that the quantification of diaphragm forces or its capacity is not included in this Design Tool and competency should be addressed separately by the Engineer.

A flexible diaphragm can be considered to act as a simply supported beam spanning laterally between two stability elements. Loads are distributed based on the tributary widths, with half of each "span" going to one support and the other half to another. This is different from a rigid diaphragm which distributes forces to stability elements based on their stiffness.

The diagrams below show the assumed tributary areas for seismic force distribution into the walls for the floors and roofs in each of the two seismic directions, based on the assumption that the floors and roofs perform as flexible diaphragms.

**WITHOUT SPINE WALL**

WITH SPINE WALL
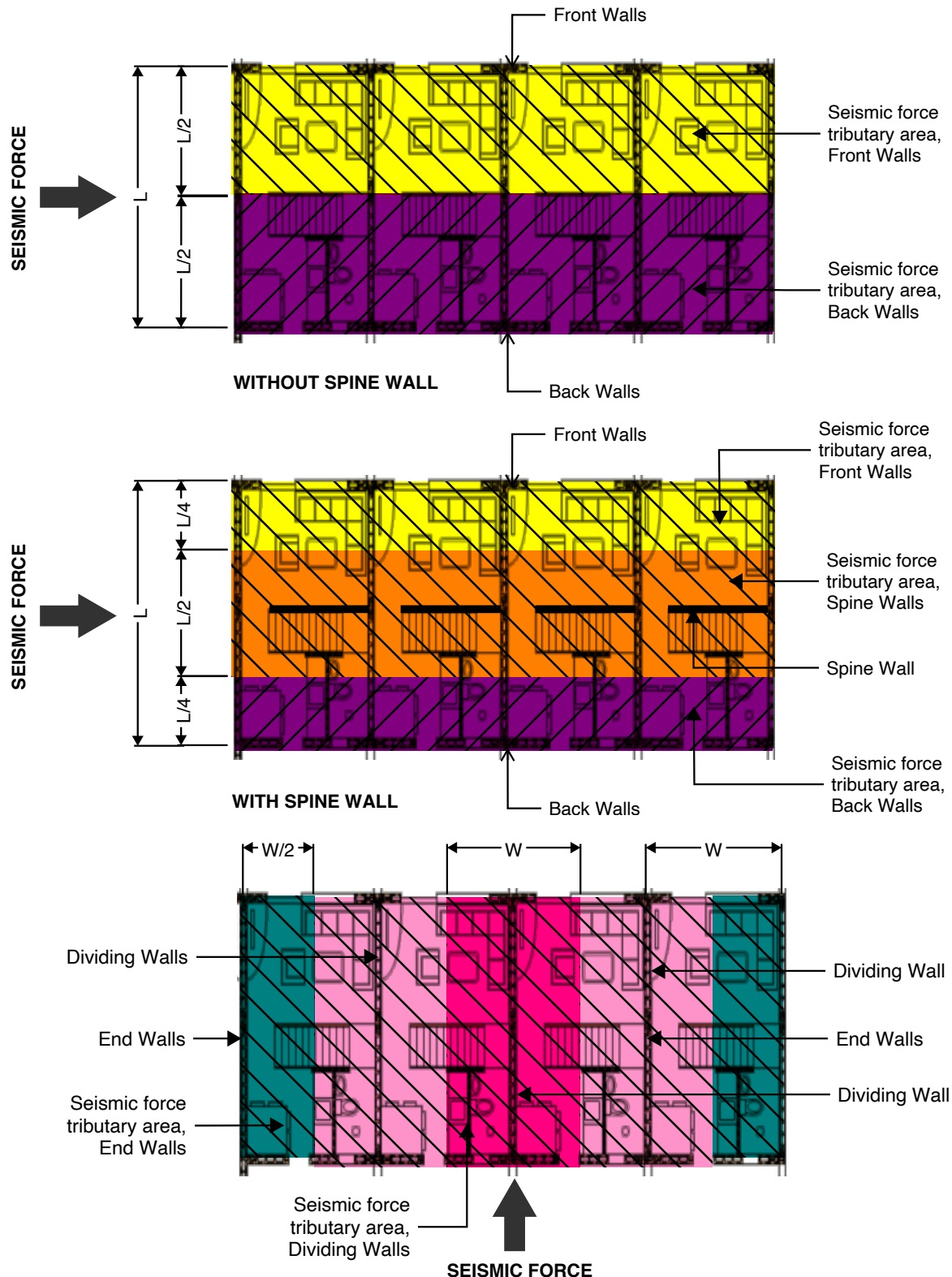
C. VERTICAL LOADS

Vertical loads are generally beneficial to masonry structures resisting lateral forces. Vertical loads include the selfweight of the wall itself, as well as load from any horizontal structures which are supported onto the wall.

Floor and roof structures are assumed to span between longitudinal walls of each unit, therefore when considering seismic forces acting in the longitudinal direction, vertical loads acting on End Walls and Dividing Walls are based on half of the span of the floor and roof loads either side of the wall.
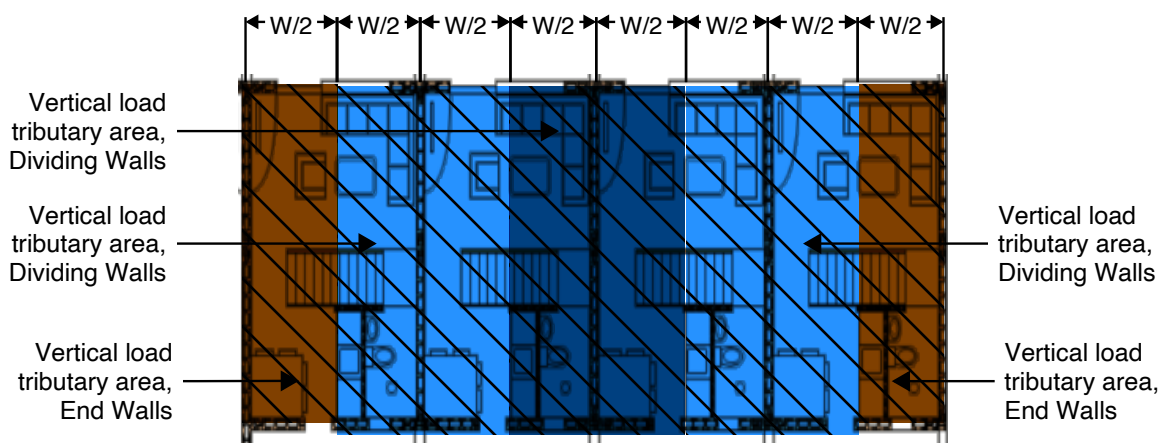
Given that the floors and roofs should also be tied into the transverse walls of the building, it is assumed that a nominal 1m width of floor or roof either side of a transverse wall contributes to the total vertical load in the wall in addition to the wall self weight.
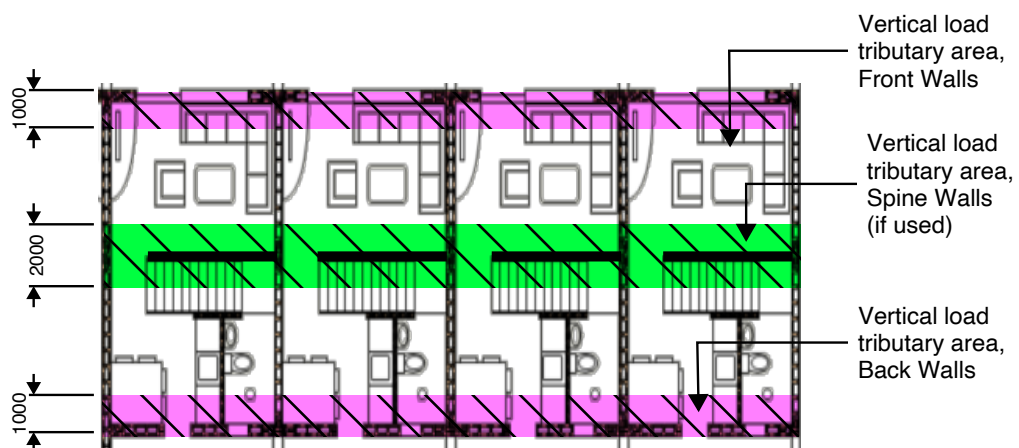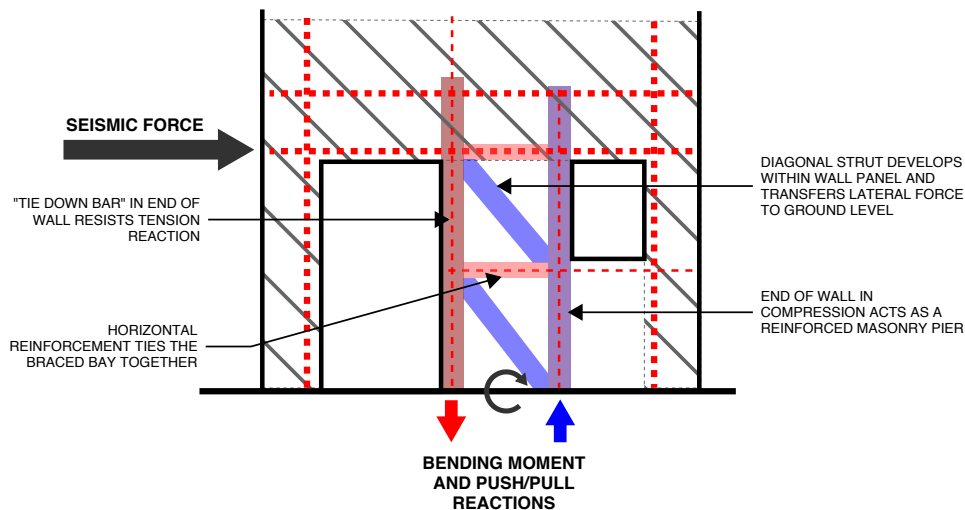
D. ASSUMED STRUCTURAL SYSTEMS

D.1 Shear Walls with Tie Down Bars

D.1.1 Overview

Design Approach - braced bay system

Typical unreinforced masonry shear walls resist horizontal forces and moments through a combination of shear resistance of the masonry and gravity action. However, given the requirements for reinforcement being provided on all sides of a masonry wall panel when using the SKAT RLB system, the panel may be theorized to perform as a "Braced Bay" where a compressions strut forms within the masonry along the diagonal of the reinforcement-surrounded wall panel. Vertical reinforcement forms both a "Tie Down Bar" in the wall end resisting tension, and a reinforced masonry pier to resist compression in the other. Horizontal reinforcement resolves the thrust of the compression strut at each level and ties the bay together.

SEISMIC FORCE

"TIE DOWN BAR" IN END OF WALL RESISTS TENSION REACTION

HORIZONTAL REINFORCEMENT TIES THE BRACED BAY TOGETHER

DIAGONAL STRUT DEVELOPS WITHIN WALL PANEL AND TRANSFERS LATERAL FORCE TO GROUND LEVEL

END OF WALL IN COMPRESSION ACTS AS A REINFORCED MASONRY PIER

BENDING MOMENT AND PUSH/PULL REACTIONS

D.1.2 Minimum Requirements

– Minimum Shear Wall Lengths

The effective width of the strut is a function of the diagonal length of the panel. Reducing wall length steepens the strut angles but also decreases the strut length and in turn the effective area. Therefore, reducing the wall length can significantly increase the stress occurring in the strut element.

When a shear wall is located between two openings, the length of the shear wall should be assumed to be the distance between the vertical bars adjacent to each of the openings.
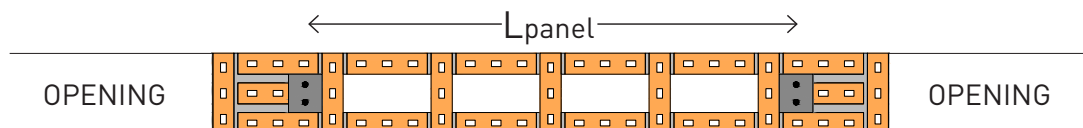

When a shear wall is located adjacent to a "column" or junction element, the length of the shear wall can be assumed to extend to the furthest line of vertical reinforcement within that element.

L_{panel}

OPENING
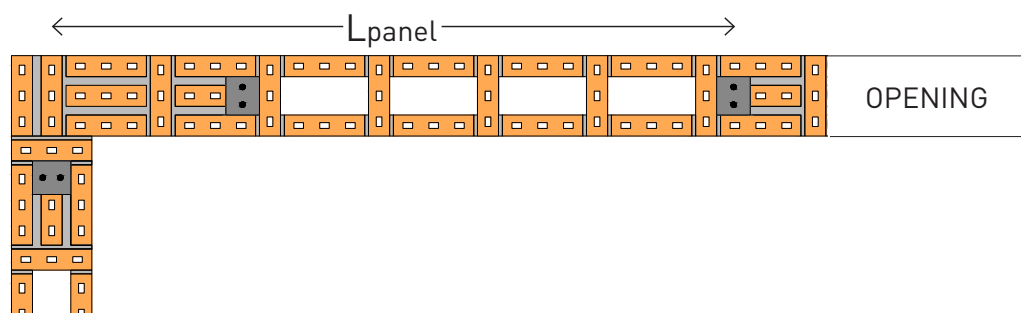
The minimum length of wall required for a strut to perform as a shear wall for different building heights has been established and can be found in the table on the right.

N. of Storeys	$\text{L}_{\text{panel,min}}$(mm)
1(G+0)	1000
2(G+1)	1600
3(G+2)	2200
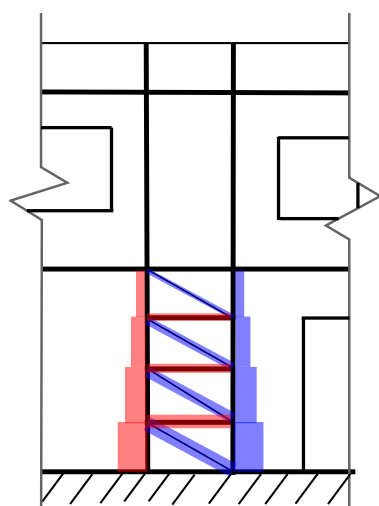
Axial force increase based on spacing of horizontal rebar

– Spacing of Horizontal Rebar

The transfer of horizontal forces between vertical tie downs is achieved through horizontal rebar, reducing vertical spacing of the horizontal rebar affects the strut angle which reduces the stress in the compression strut. The spreadsheet provided conservatively assumes that only one horizontal rebar is provided at mid-height of the shear wall, however minimum reinforcement requirements note that horizontal reinforcement should be spaced at 600mm vertical centres. This may be taken into consideration by the Design Engineer when the wall cannot be demonstrated to perform using the spreadsheet, so that on the lowest portion of the wall experiences the highest forces which may prove easier to accommodate through careful reinforcement detailing.

D.1.3 Suggestions if the Design Cannot be Ratified

If the applied forces are beyond the capacity of the provided structural elements there are a number of ways this can be improved:

– Increase wall panel length

– When designing longitudinal walls (End Walls and Dividing Walls) consider if there is an additional panel of wall on the same wall line which could also perform as a shear wall and therefore share the seismic load. Hand calculations may be required to establish the division of load.

– Increase wall thickness

– Consider the additional horizontal reinforcement at 600mm centers as required by the code to break down the braced bay into more vertical panels than the two assumed. Provide additional vertical reinforcement at the base where the axial force is highest.

D.2 Moment Frame System

D.2.1Overview

Where large openings exist in a wall at the ground floor level, the wall is theorized to perform as a moment frame system where the more heavily reinforced wall junctions perform as columns, and the reinforced spandrel panel acts as the connecting beam. When a number of units exist in a row without seismic joints, all units can be considered to act as a single structure.
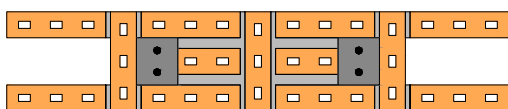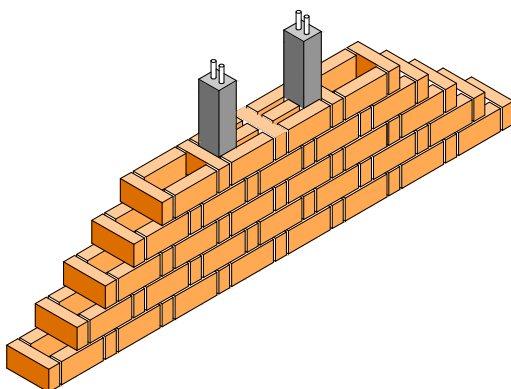
There are three types of column element - 'L'-junction, 'T'-junction and Rectangular - in addition to the spandrel panel which runs horizontally between openings at the level above and below.
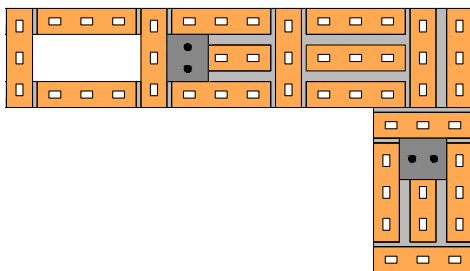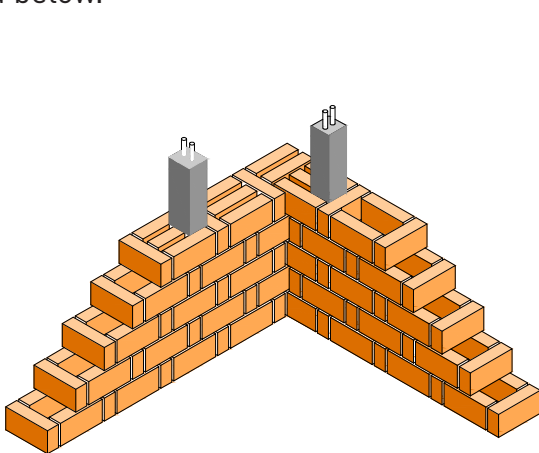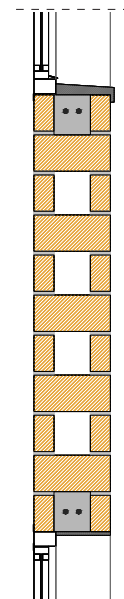

L-junction plan
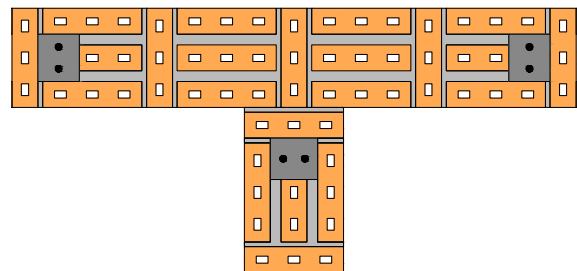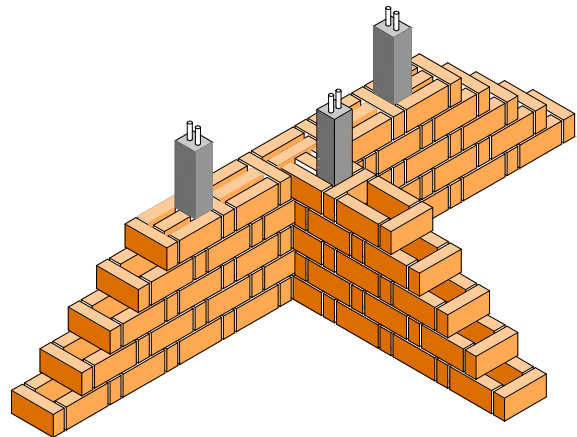
*T-junction plan*

Rectangular column plan

Spandrel panel section

D.2.2 Structural Analysis using Software

In order to establish design forces acting in the individual element, the moment frame must be modeled as a 2D frame in a suitable structural analysis software, such as Autodesk Robot, Tekla TEDDS or Oasys GSA (others also apply).

The geometry of the frame should match that of the building elevation being analysed with the centerlines of elements modeled as indicated below.
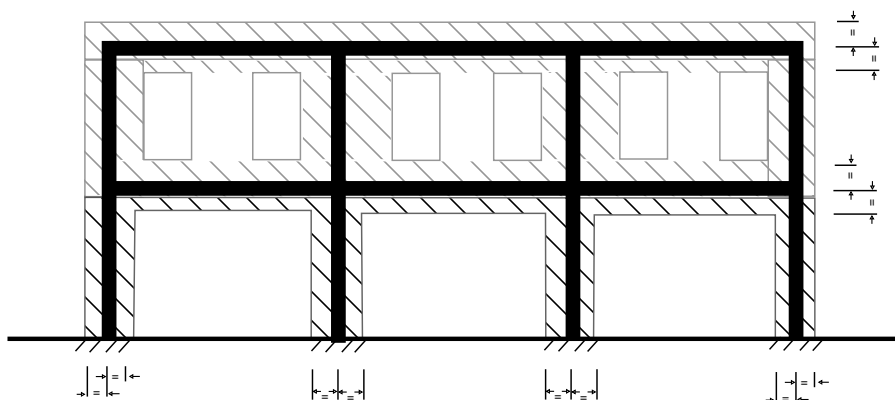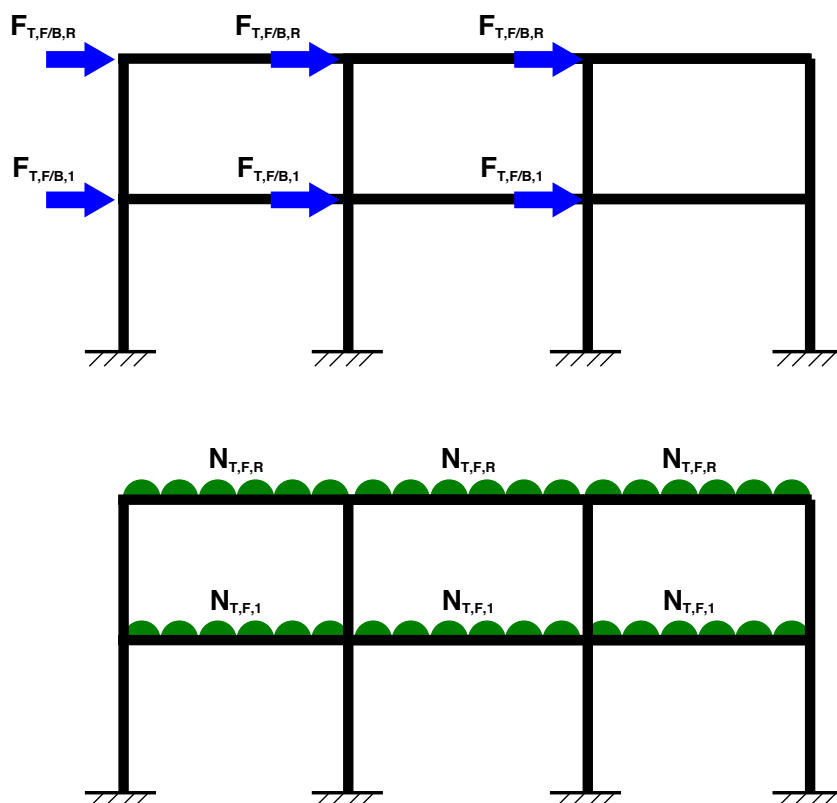

Assumed Centerlines for Model Geometry

The following material properties are recommended for analysis purposes, however the Design Engineer may choose to use other values.

Elastic Modulus of Brickwork $E = 1000 f_k = 5000$ MPa

Poisson's Ratio $\alpha = 0.3$

Shear Modulus $G = 2000$ MPa

Density $\rho = 20$ kN/m^3

Using the outputs from the Global Analysis spreadsheet, horizontal seismic forces should be applied to the model as point loads at each floor or roof level. Forces should be distributed across the units if multiple units are being analyzed together.

Using the outputs from the Global Analysis spreadsheet, vertical loads should be applied to the horizontal beam elements in the model as uniformly distributed loads. Do not apply gravity loads, as the weight of the masonry has already been accounted for in the calculation of the vertical loads.

D.2.3 Interaction Diagrams

A series of M-N charts have been developed for each of the element types. The combined major axis bending moments and axial forces (compression) are plotted on the relevant graphs to ensure they fall within the capacity envelope.

Each M-N chart includes three different curves/envelopes which have been calculated based on different reinforcement bar sizes - B8, B10, B12 - included in the vertical concrete pockets of each element. These different curves are used to select the reinforcement requirements for each member.

210 THICK WALL : 'L'-JUNCTION COLUMN
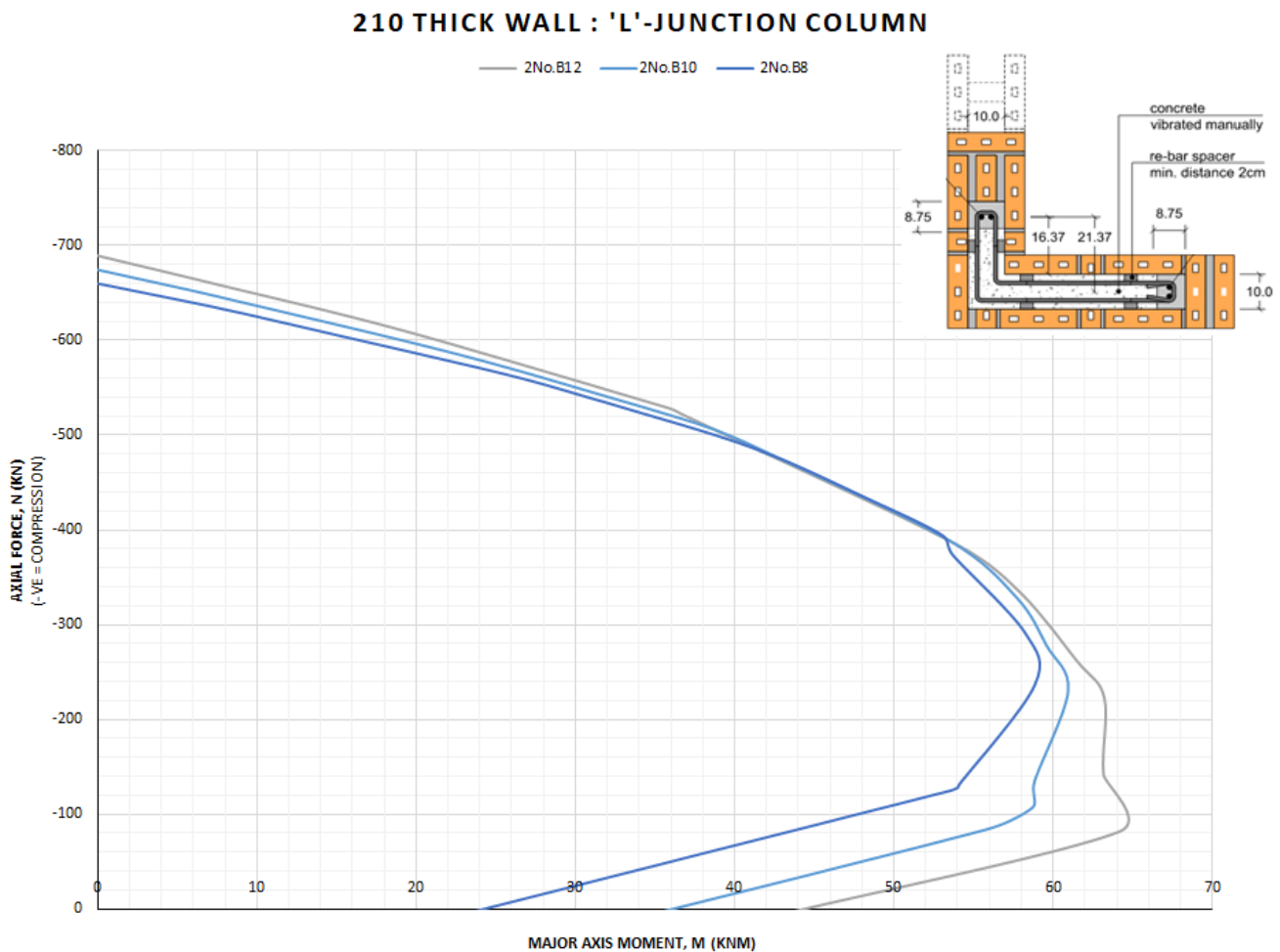
D.2.4 Suggestions if the Design Cannot be Ratified

If the applied forces are beyond the capacity of the provided structural elements there are a couple of ways this can be improved

Increase reinforcement diameters

Increase wall thickness

Decrease the height of the building

Use timber floors in place of max-span slabs

Introduce a wall panel to each unit which is long enough to act as a shear wall

E. MORE COMPLEX STRUCTURES, DETAILED ANALYSIS AND FURTHER WORK

The Design Tools provided with this document have been produced to ease the design process for simple, regular, low rise buildings. However, the design methods included here - Shear Wall and Moment Frame approaches - may be applied to structural elements occurring in more complex structures built using the Rowlock Bond masonry system. In the case of more complex structural arrangements, the Design Engineer should consider the appropriate analysis method, and may need to revert to the use of Response Spectrum Analysis methods using appropriate analysis software.

Combining the SKAT RLB system with other structural materials and elements may also be an option - for example, combining the masonry with a reinforced concrete frame to achieve a taller building. Again, the Design Engineer should consider appropriate analysis methods and how the two structural materials interact, but may then apply similar methods as included in the Design Tools to specify the requirements for the masonry portions of the building.

04

WORKED EXAMPLE CALCULATIONS

GLOBAL ANALYSIS - WORKED EXAMPLE

This worked example is based on the design of the Mapzi Rehousing Project with Rowlock Bond technology in Kigali, Rwanda
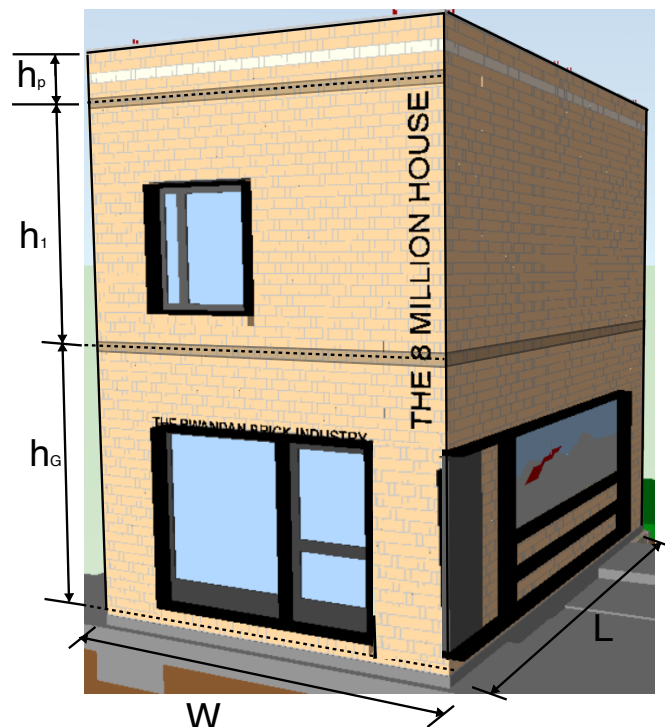
Seismic Design Parameters

Rwanda is considered a region of moderate seismicity given its location adjacent to the East african Rift System. The following seismic parameters are considered appropriate for the site based on the Rwanda Building Code, Global Earthquake Model (GEM) and the requirements and values set out in Eurocode 8 (where Response Spectrum Type 1 is assumed for the purpose of this design).

RBC / GEM	Peak Ground Accelration (PGA)	$a_g = 1.6$ m/s^2
g = 9.81m/s^2	Ratio of PGA to Gravity	$\alpha = a_g / g = 0.16g$
EC8, Table 3.1	Soil Factor - Assume Spectrum Type 1, Soil Type C	$S = 1.15$
EC8, Table 4.3	Importance Factor - Assume Importance Class II	$\gamma_1 = 1.0$
EC8, Table 4.4	Ductility Factor - Reinforced Masonry	$q = 2.0$
EC8, Eqn 3.14	Design Seismic Acceleration	$Sd = 2.5 \, \alpha \, S \, \gamma_1 / q = 0.234g$

Building Geometry

The units are rectangular in plan and the footprint of an individual unit is 4.5m x 6.5m and there are four units constructed in a single terrace. 210mm thick RLB structural walls exist on the 4 sides of each unit, and there is no spine wall. The units are 2storeys tall and each storey has a different height. The masonry walls extend above roof level to form a parapet around each individual roof.

	Unit length	L = 6.5 m	
	Unit width	W = 4.5 m	
	Area of individual unit	A = 29.3 m²	
	Number of storeys	n = 2	
	Ground floor height	h_G = 2.5 m	
	First floor height	h_1 = 2.5 m	
	Parapet height	h_P = 1.4 m	
EC8, Eqn 4.6	Fundamental period	T = 0.3 s	
	Number of adjacent units	N = 4	

Construction

The first floor of the units is from timber joists and the roof structure is from timber trusses. Both span across the width of each unit. The walls are from 210mm thick Rowlock Bond masonry and there is no spine wall. The front and back elevations contain windows of varying sizes at both levels

	Wall thickness	t = 210 mm	
	Slenderness at Ground Floor	$\lambda = h_G / t$ = 11.9	< 15 ∴ OK
Selfweight of structure	Timber trusses/joists dead load	DL = 0.40 kPa	
Finishes, services, ceiling, partitions etc.	Floor Superimposed dead load (incl. partitions)	SDL_{floor} = 0.40 kPa	
Metal sheet, services, ceiling, insulation etc.	Roof Superimposed dead load	SDL_{roof} = 0.20 kPa	Roof LL ignored in seismic scenario
EC1, Table 6.2	Floor Live Load	LL = 1.50 kPa	
Accounts for central cavity and cross bond	Masonry selfweight	SW = 2.63 kPa	
Includes steel frame and 10mm glazing	Glazing (superimposed wall load)	SDL_{wall} = 0.15 kPa	

Global Seismic Forces

This section calculates the vertical loads of each building element before converting to seismic bases shear following the Equivalent Static Force method, by multiplying the total building weight by the design seismic acceleration. Conservatively, the walls are assumed to be entirely solid masonry for this calculation.

	DL Seismic Contribution Factor	$Y_{DL} = 1.0$
EC1,NATable A1.1	LL Seismic Contribution Factor	$Y_{LL} = 0.3$
	Vertical load of masonry per metre height of wall (all units)	$W_M = SW \cdot [\, (2N.W) + L(N+1) \,]$ $W_M = 180$ kN/m
	Vertical load of first floor per unit	$W_1 = A\,(DL + SDL_{floor}) = 37$ kN/unit
	Vertical load of roof per unit	$W_R = A\,(DL + SDL_{floor}) = 18$ kN/unit
EC8, Eqn 4.5	Total Base Shear	$F_b = S_d\,[H.W_M + N(W_1 + W_R)] = 333$ kN

The seismic forces acting at each floor/roof level are then calculated using a simplified method which is appropriate for regular, rectangular low-rise buildings.

Front Elevation
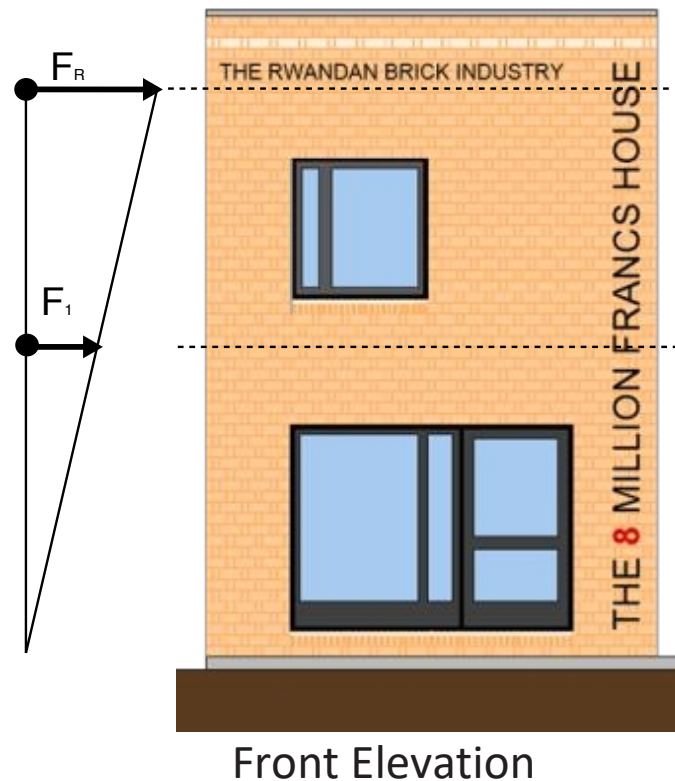
Height above ground (1F)		$z_1 = h_G = 2.5$ m
Height above ground (RF)		$z_R = h_G + h_1 = 5.0$ m
Tributary height of wall (1F)		$h_{trib,1} = 0.5\,(h_G + h_1) = 2.5$ m
Tributary height of wall (RF)		$h_{trib,R} = 0.5\,h_1 + h_P = 2.65$ m
Seismic weight (1F)		$m_1 = h_{trib,1}\,W_M + N\,W_1 = 597$ kN
Seismic weight (RF)		$m_R = h_{trib,R}\,W_M + N\,W_R = 548$ kN
Height x Seismic Weight (1F)		$z_1.m_1 = 1492$ kNm
Height x Seismic Weight (RF)		$z_R.m_R = 1451$ kNm

EC8, Eqn 4.11	First Floor Seismic Force	$F_1 = F_b\,[z_1.m_1 / \Sigma(z_i.m_i)] = 169$ kN
	Roof Seismic Force	$F_R = F_b\,[z_1.m_1 / \Sigma(z_i.m_i)] = 164$ kN

Seismic Forces on Each Wall

The seismic forces acting at each floor level are distributed onto individual walls to allow the analysis to be carried out on each of these walls. The following calculations assume that there is a flexible diaphragm formed at each level, either by the floor/roof structure OR a ring beam (design not included here), and that all of the walls in each of the orthogonal directions have a similar stiffness so that torsional effects do not need to be taken into account.

Longitudinal Walls
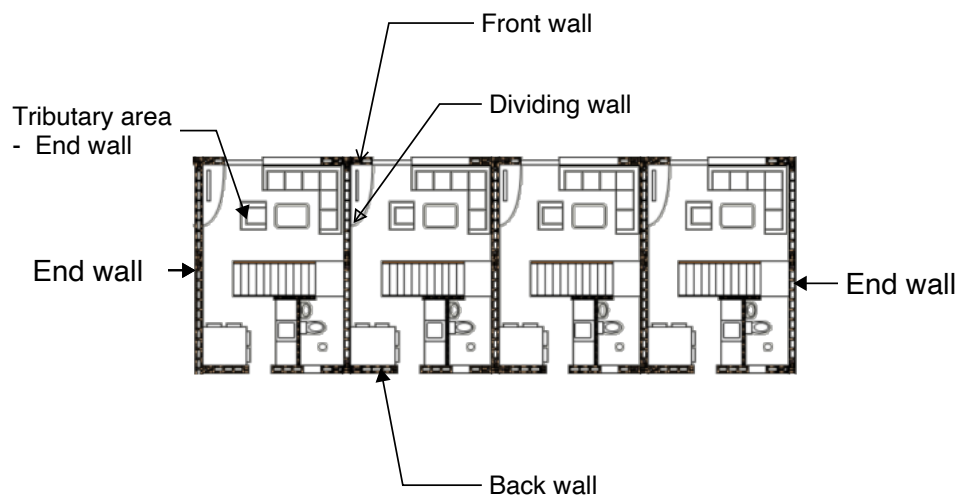
Number of longitudinal walls	$N_L = N+1 = 5$

Force acting on End Walls at First Floor	$F_{L,E,1} = F_1 / 2N = 21.1$ kN
Force acting on End Walls at Roof	$F_{L,E,R} = F_R / 2N = 20.5$ kN

Force acting on Dividing Walls at First Floor	$F_{L,D,1} = F_1 / N = 42.2$ kN
Force acting on Dividing Walls at Roof	$F_{L,D,R} = F_R / N = 41.1$ kN

Transverse Walls

Number of Transverse Walls	$N_T = 2$	No Spine Wall

| Force acting on Front/Back Wall at First Floor	$F_{T,F/B,1} = F_1 / N_T = 21.1$ kN
Force acting on Front/Back Wall at Roof	$F_{T,F/B,R} = F_R / N_T = 20.5$ kN

NOTE: When a spine wall exists, the forces on the Front/Back walls reduce by half

Vertical Loads on Each Wall

Vertical forces are calculated on the assumption that the floor/roof span across the short direction of the building. However, a 1m tributary width of floor/roof is considered to act on the front and back walls as well.

Each wall is given a solidity ratio, and the calculations use this to define the proportion of solid wall weight to glazing weight.

Longitudinal Walls - Supporting Floors and Roof
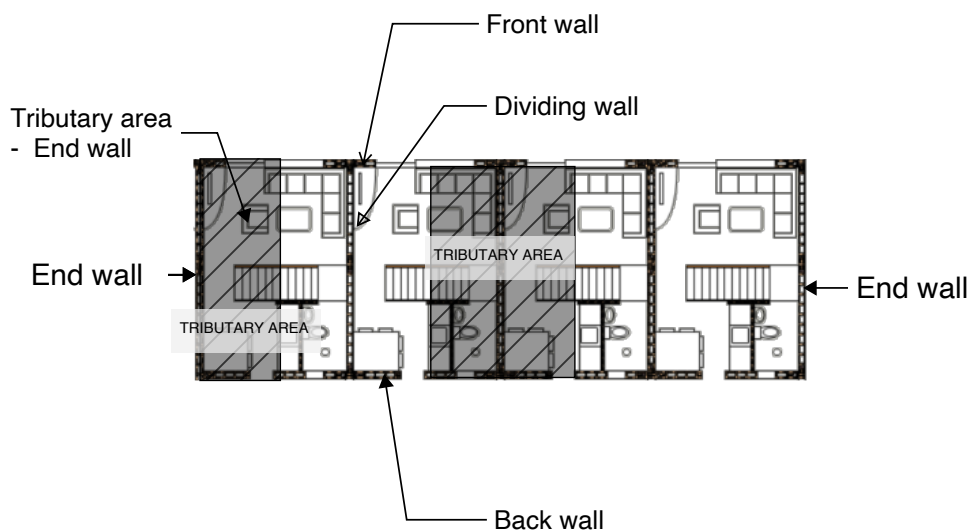
Ratio of Solid Wall to Openings (End Wall@1F)	$\alpha_{L,E,1} = 0.9$
Ratio of Solid Wall to Openings (Dividing Wall@1F)	$\alpha_{L,D,1} = 1.0$

UDL acting on End Wall @1F

$$N_{L,E,1} = h_1[\alpha_{L,E,1}SW + (1-\alpha_{L,E,1})SDL_{wall}] + (W_1/2L)$$
$$N_{L,E,1} = 10.4 \text{ kN/m}$$

UDL acting on End Wall @RF
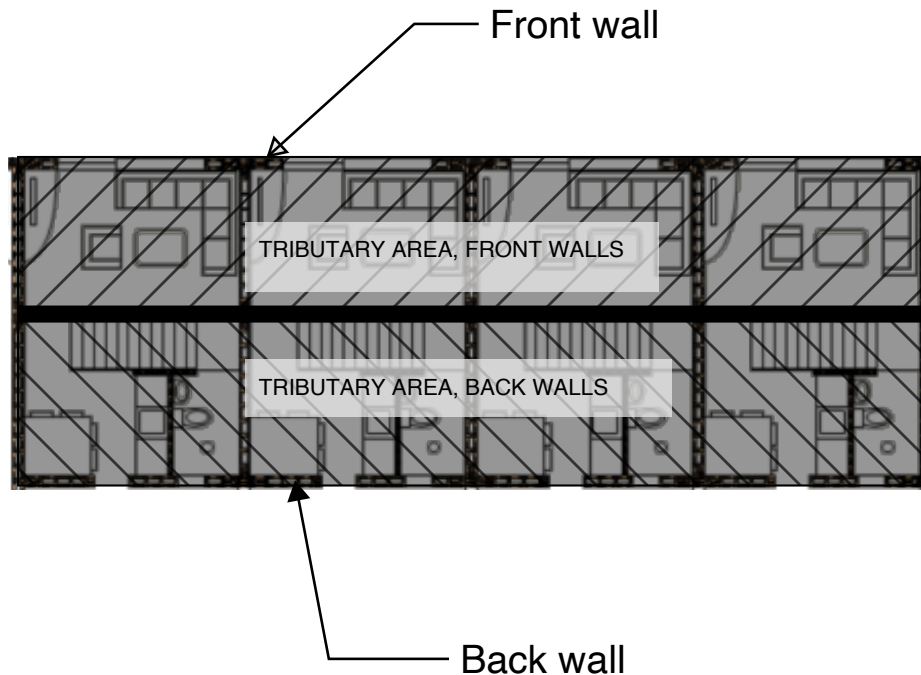
$$N_{L,E,R} = h_P SW + (W_R/2L) = 5 \text{ kN/m}$$

UDL acting on Dividing Wall @1F

$$N_{L,D,1} = h_1[\alpha_{L,D,1}SW + (1-\alpha_{L,D,1})SDL_{wall}] + (W_1/L)$$
$$N_{L,D,1} = 14.2 \text{ kN/m}$$

UDL acting on Dividing Wall @RF

$$N_{L,D,R} = h_P SW + (W_R/L) = 6.4 \text{ kN/m}$$

Transverse Walls - Not supporting floors/roof

Front wall

Back wall

Ratio of Solid Wall to Openings (Front Wall@1F)	$\alpha_{F,1} = 0.7$
Ratio of Solid Wall to Openings (Back Wall@1F)	$\alpha_{B,1} = 0.7$

UDL acting on Front Wall @1F

$$N_{F,1}=h_1[\alpha_{F,1}SW+(1-\alpha_{F,1})SDL_{wall}]+(W_1/A)$$
$$N_{F,1} = 6.4 \text{ kN/m}$$

UDL acting on Front Wall @RF

$$N_{F,R} = h_P SW + (W_R/A) = 4.3 \text{ kN/m}$$

UDL acting on Back Wall @1F

$$N_{B,1}=h_1[\alpha_{B,1}SW+(1-\alpha_{B,1})SDL_{wall}]+(W_1/A)$$
$$N_{B,1} = 6.4 \text{ kN/m}$$

UDL acting on Back Wall @RF

$$N_{B,R} = h_P SW + (W_R/A) = 4.3 \text{ kN/m}$$

SHEAR WALL DESIGN - WORKED EXAMPLE

This worked example is based on the design of the Mapzi Rehousing Project with Rowlock Bond technology in Kigali, Rwanda

Given that there is reinforcement present on all sides of the shear walls, the wall can be theorized to perform as a braced bay, where strut action occurs in the masonry surrounded by reinforcement creating a "compression brace". The vertical tie-down reinforcement at one end of the wall then goes into tension, and the other end into compression.
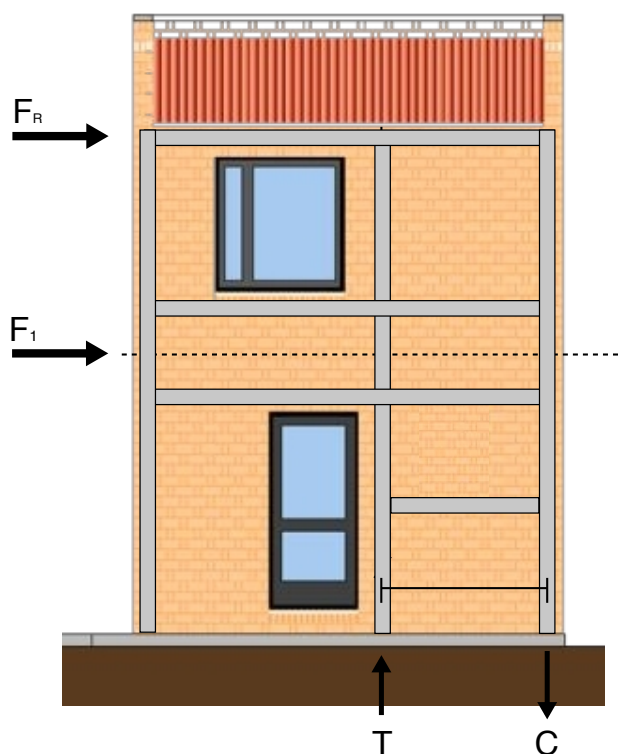
Material Properties

Characteristic compression strength of brick	f_b = 10 MPa
Characteristic compression strength of mortar	$f_m = max(f_b , 10)$ = 10 MPa

EC6, Eqn 3.2

Factors - assuming Group 2 masonry	K = 0.5
	α = 0.7
	β = 0.3

EC6, Eqn 3.1

Char. compression strength of masonry	$f_k = K f_b^{\alpha} f_m^{\beta}$ = 5 MPa

Assuming B500 grade reinforcement

Char. tensile strength reinforcement	f_{yk} = 500 MPa

IStructE Manual for the seismic design of steel and concrete buildings to Eurocode 8

Material Safety Factor - Masonry	γ_m = 2.0
Material Safety Factor - Steel reinforcment	$\gamma_{m,s}$ = 1.15

Eqn 10.1 and 10.2

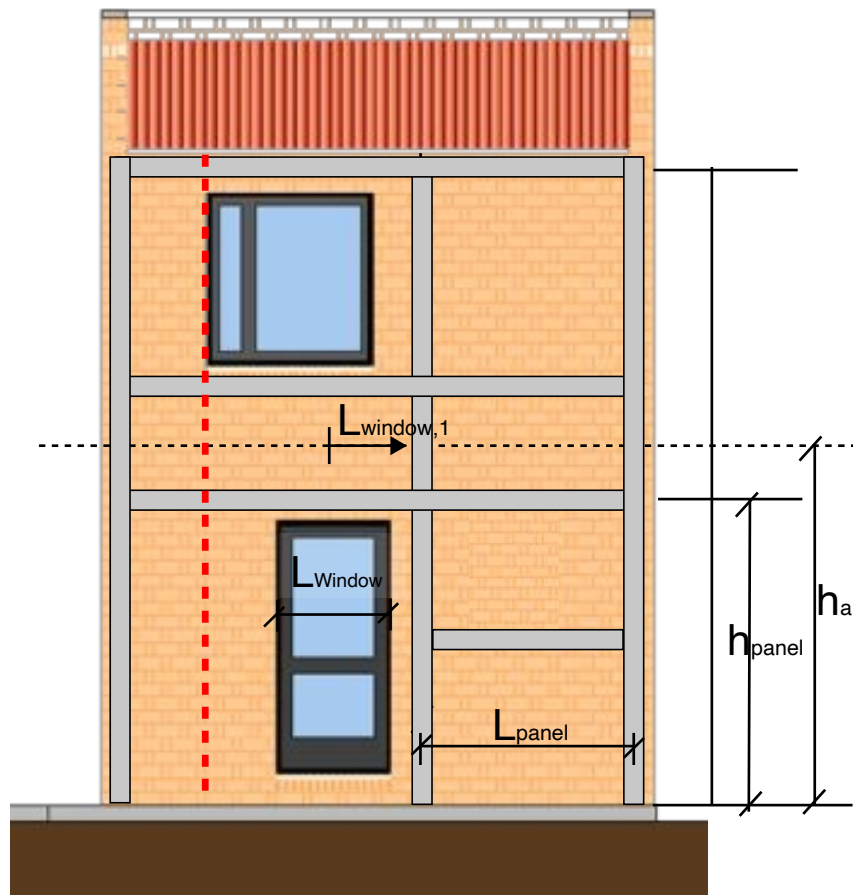|	
Design compression strength of masonry	$f_d = f_k / \gamma_m$ = 435 MPa
Design tensile strength of steel	$f_{yd} = f_{yk} / \gamma_{m,s}$ = 435 MPa

Wall Panel & Strut Geometry at Ground Level - Front Wall

The wall at ground floor level will generally be the critical in any elevation, unless the walls at upper floor are significantly shorter.

Height of wall (from Global Analysis calculation)	$h_G = 2.5$ m
Height of wall panel	$\mathbf{h_{panel}} = h_G - 0.55m = 1.95$ m
Mid Height of wall panel	$h_{mid,panel} = 0.98$ m
Length of wall panel	$l_{panel} = 1.8$ m
Width of adjacent windows	$l_{window,1} = 0.0$ m
	$l_{window,1} = 0.9$ m
Length of strut	$l_{strut} = \sqrt{(h_{panel}^2 + l_{panel}^2)} = 2.0$ m
Cosine(Angle of strut)	$\cos\Theta = l_{panel} / l_{strut} = 0.88$
Width of strut	$w_{strut} = 0.1\, l_{strut} = 204.71$ mm
Thickness of wall leaf	$t_{leaf} = 55$ mm

Typical SKAT RLB brick thickness

Area of strut	$A_{strut} = 2\, t_{leaf}\, w_{strut} = 319\ cm^2$

Effective thickness of wall $\qquad\qquad\qquad\qquad\qquad\qquad\qquad t_{ef} = t = 210\ mm$

Width of wall end $\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad w_{end} = 340 mm$

Area of end of wall $\qquad\qquad\qquad\qquad\qquad\qquad\qquad A_{end} = t\, w_{end} = 71400\ mm2$
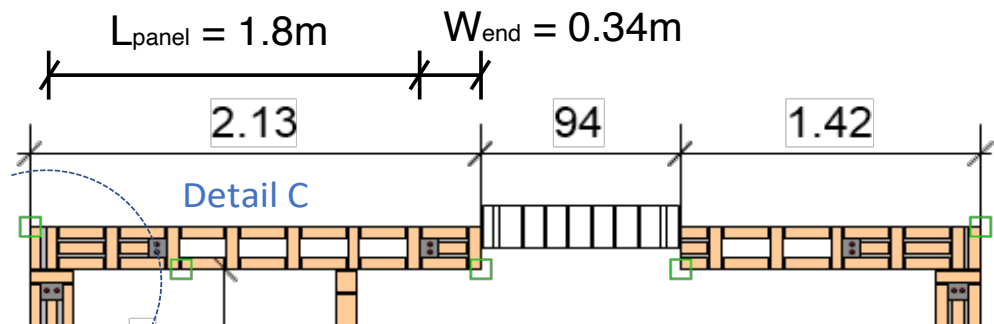
External Design Forces - Front Wall

Forces are taken from the Global Analysis calculation and transformed in forces acting in the theoretical braced bay

	Seismic force acting on Front Wall @1F	$F_{F,1} = 21$ kN
	Seismic force acting on Front Wall @RF	$F_{F,R} = 21$ kN

Heights are taken directly from Global Analysis calculation

Height above ground (1F) $\qquad$ $z_1 = 2.5$ m

Height above ground (RF) $\qquad$ $z_R = 5.0$ m

Values are taken directly from Global Analysis calculation

UDL acting on Front Wall @1F $\qquad$ $N_{F,1} = 6.4$ kN/m

UDL acting on Front Wall @RF $\qquad$ $N_{F,R} = 4.3$ kN/m

Seismic base shear $\qquad$ $F_{b,F} = F_{F,1} + F_{F,R} = 42$ kN

Moment at base of wall $\qquad$ $M_{b,F} = F_{F,1} z_1 + F_{F,R} z_R = 155$ kNm

Total vertical load in wall panel $\qquad$ $N_{panel} = (N_{F,1}+N_{F,2})[l_{panel}+0.5(l_{window,1}+l_{window,2})]$

$N_{panel} = 24$ kN

Compression force in Strut $\qquad$ $N_{c,Ed,strut} = F_{b,F} /cos\Theta = 47$ kN

Compression force in End of Wall $\qquad$ $N_{c,Ed,end} = N_{panel}/2 + M_{b,F}/l_{panel} = 98$ kN

Tension force in End of Wall $\qquad$ $N_{c,Ed,end} = N_{panel}/2 - M_{b,F}/l_{panel} = 74$ kN

Compression Capacity of Strut

The diagonal strut is checked for compressive buckling under seismic loads.

How to design masonry structures using EC 6. 2.Vertical Resistance
Figure 6

Effective length of strut $\qquad$ $l_{ef,strut} = 0.8\, l_{strut} = 2.0$ m

Slenderness of strut $\qquad$ $\lambda_{strut} = l_{ef,strut} / t_{ef} = t = 9.5$

EC6, Eqn 6.7

Eccentricity $\qquad$ $e_{m,k} = 0.05\, t = 10.5$ mm

EC6, Eqn G.2
EC6, Eqn G.4
EC6, Eqn G.3

Factors $\qquad$ $A_1 = 1 - 2(e_{m,k}/t_{ef}) = 0.9$

$\lambda = (l_{strut}/t_{ef})\ \sqrt{(f_k/E)} = 0.308$

$u = \lambda - 0.063 / [0.73-1.17(e_{m,k}/t_{ef})] = -0.1$

EC6, Eqn G.1

Reduction factor $\qquad$ $\Phi_m = A_1 \exp[(-u)^2/2] = 0.90$

Design resistance capacity $\qquad$ $N_{b,Rd,strut} = \Phi_m\, t\, f_k\, /\gamma_m = 249$kN $\quad U = \dfrac{N_{c,Ed,strut}}{N_{b,Rd,strut}} = 19\%$

$\therefore$ OK!

End of Wall Capacities

Buckling of the end of the wall in compression is ignored due to the presence of reinforcement.

Compression Capacity $\qquad$ $N_{c,Rd,end} = f_d\, A_{end} = 179$ kN $\quad U = \dfrac{N_{c,Ed,end}}{N_{c,Rd,end}} = 55\%$

$\therefore$ OK!

Tension in the other end is resisted by the vertical reinforcement.

Steel Reinforcement diameter $\qquad$ $d = 12$ m

Area of steel reinf provided (2No.) $\qquad$ $A_s = 226.2$ mm^2

Tension Capacity $\qquad$ $N_{t,Rd,end} = f_{yd}\, A_s = 98.3$ kN $\quad U = \dfrac{N_{t,Ed,end}}{N_{t,Rd,end}} = 76\%$

$\therefore$ OK!

MOMENT FRAME SYSTEM - WORKED EXAMPLE

This worked example is based on the design of the existing SKAT Rowlock Bond housing located at Mpazi, Nyabagogo, Kigali, Rwanda.

Where large openings exist in a wall at the ground floor level, the wall is theorized to perform as a moment frame system where the more heavily reinforced wall junctions perform as columns, and the reinforced spandrel panel acts as the connecting beam. When a number of units exist in a row without seismic joints, all units can be considered to act as a single structure.

In the case of the Mpazi development, the front wall contains a large opening at ground floor level and so it will be used in this example.
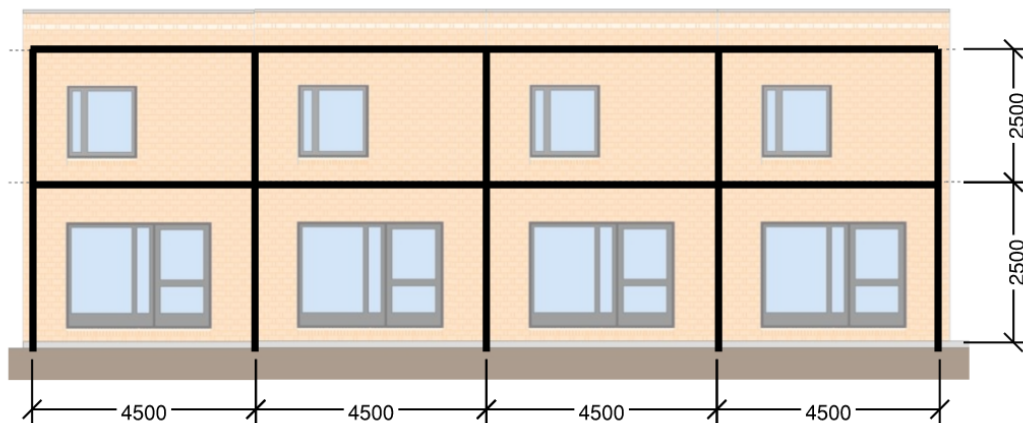
Structural Analysis of Frame - Input

The 2D frame should be modeled in a suitable structural analysis software, such as Autodesk Robot, Tekla TEDDS or Oasys GSA (others also apply). The geometry of the frame should match that of the building elevation being analysed, with the centreline of the horizontal members being set at the relevant floor or roof level. There are three types of column element - 'L'-junction, 'T'-junction and Rectangular - and the appropriate column type should be applied to each vertical element. Each column should be modelled with a fixed base connection.
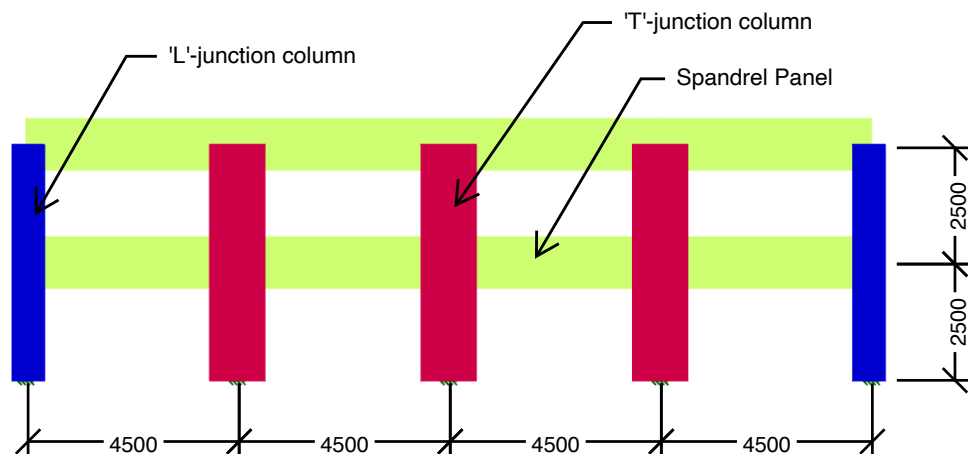
The following material properties have been used for analysis purposes, however this is down to the Design Engineer's discretion.

Elastic Modulus of Brickwork   E = 5000 MPa
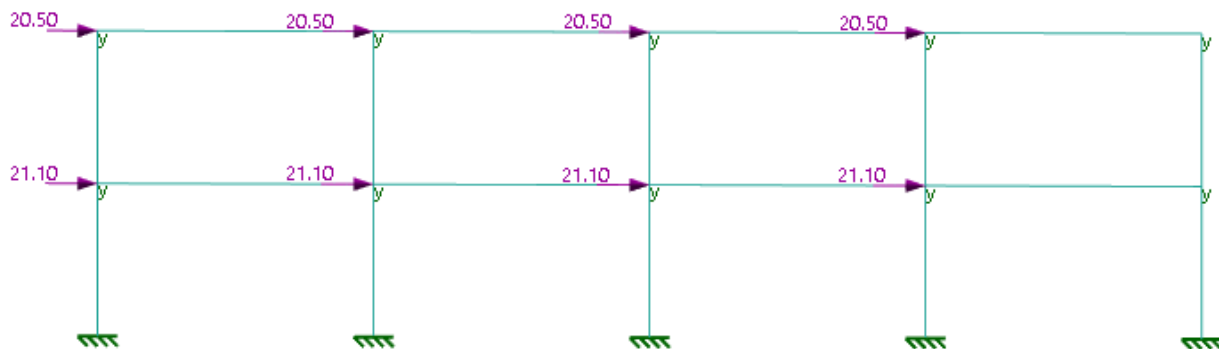
Poissons Ratio α = 0.25

Shear Modulus   G = 2000 MPa

Density ρ = 1800 kg/m3

Using the outputs from the Global Analysis spreadsheet, horizontal seismic forces should be applied to the model as point loads at each floor or roof level. Forces should be distributed across the units if multiple units are being analysed together.


Using the outputs from the Global Analysis spreadsheet, vertical loads should be applied to the horizontal beam elements in the model as uniformly distributed loads. Do not apply gravity loads, as the weight of the masonry has already been accounted for in the calculation of the vertical loads
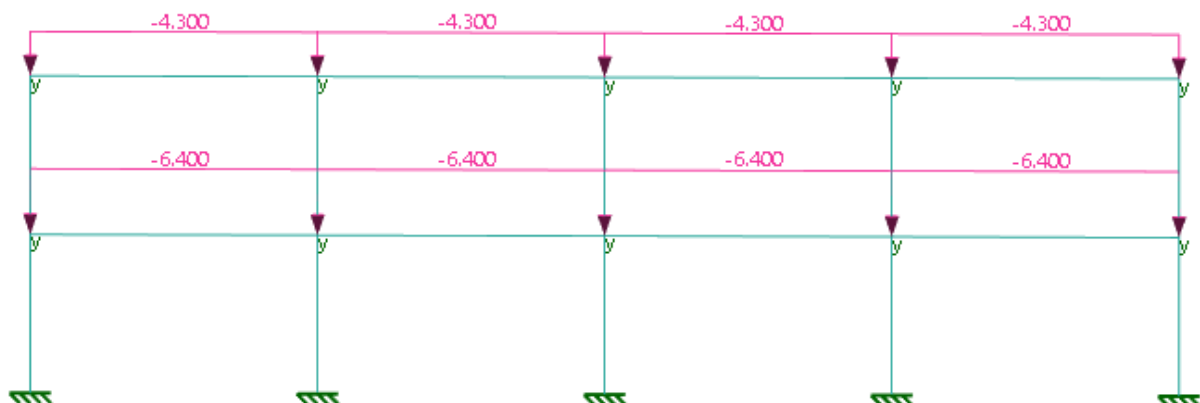
Structural Analysis of Frame - Output

Once the correct geometry and loads have been applied to the model, it should be analysed with the unfactored forces and loads combined in a single load case, and the resulting internal forces calculated.
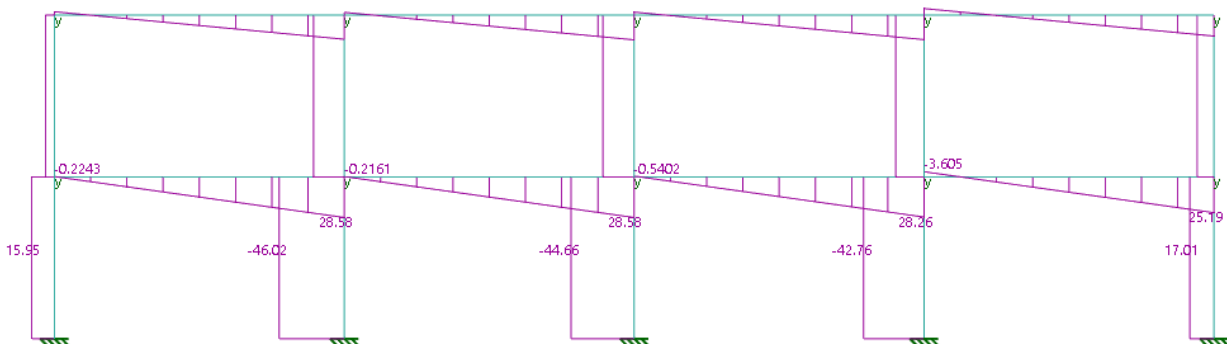
The images below show the internal forces for the ground floor frame elements at the Mpazi development.
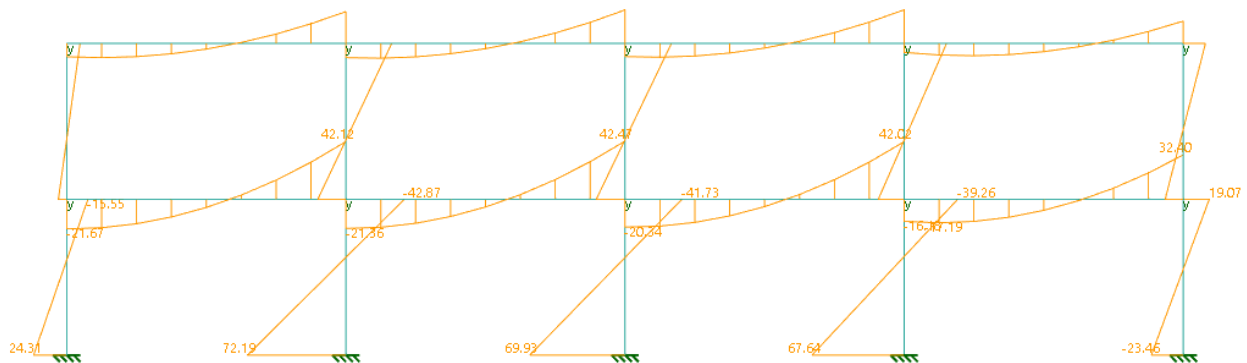
MAJOR AXIS BENDING MOMENTS
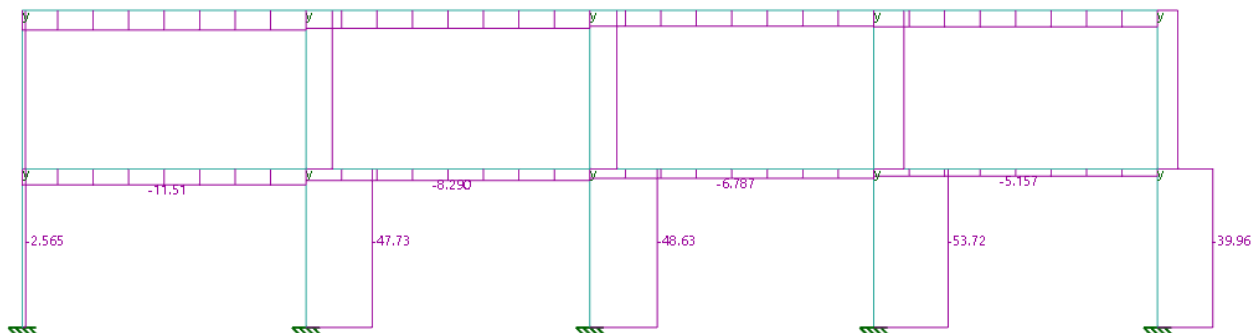
AXIAL FORCES

SHEAR FORCES
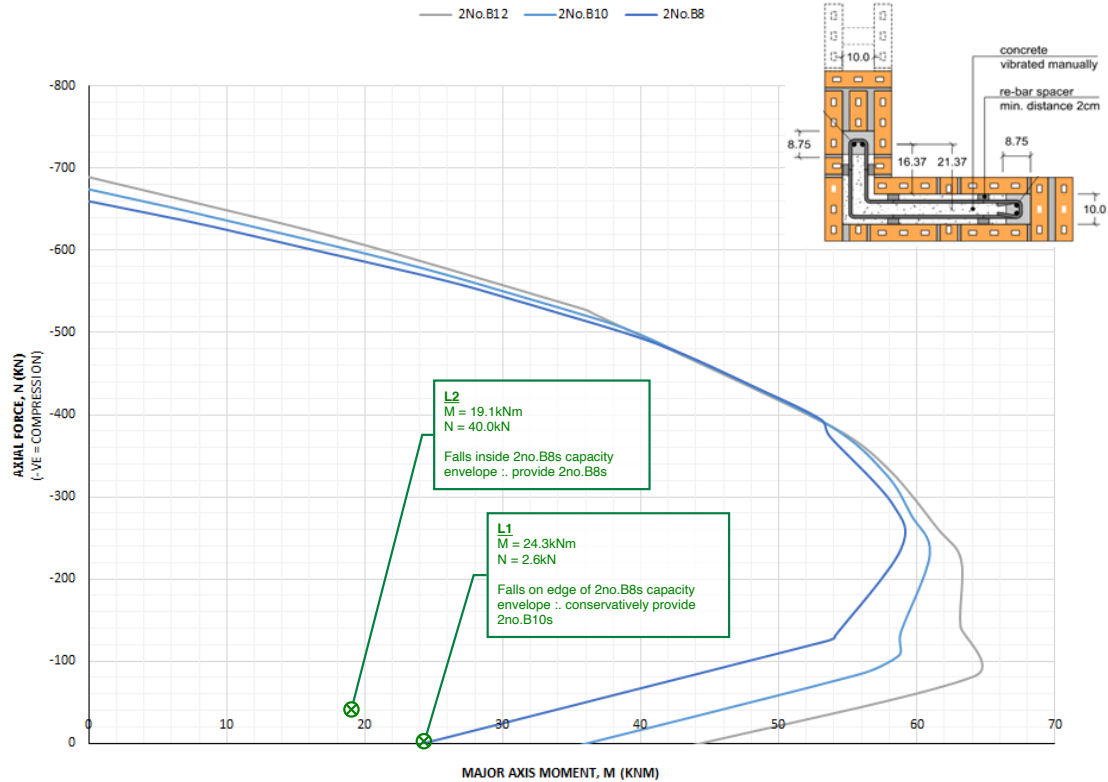
ASSESSMENT AGAINST M-N CURVES

A series of M-N charts have been developed for each of the element types. The combined major axis bending moments and axial forces (compression) are plotted on the relevant graphs to ensure they fall within the capacity envelope.

Each M-N chart includes three different curves/envelopes which have been calculated based on different reinforcement bar sizes - B8, B10, B12 - included in the vertical concrete pockets of each element. These different curves are used to select the reinforcement requirements for each member.
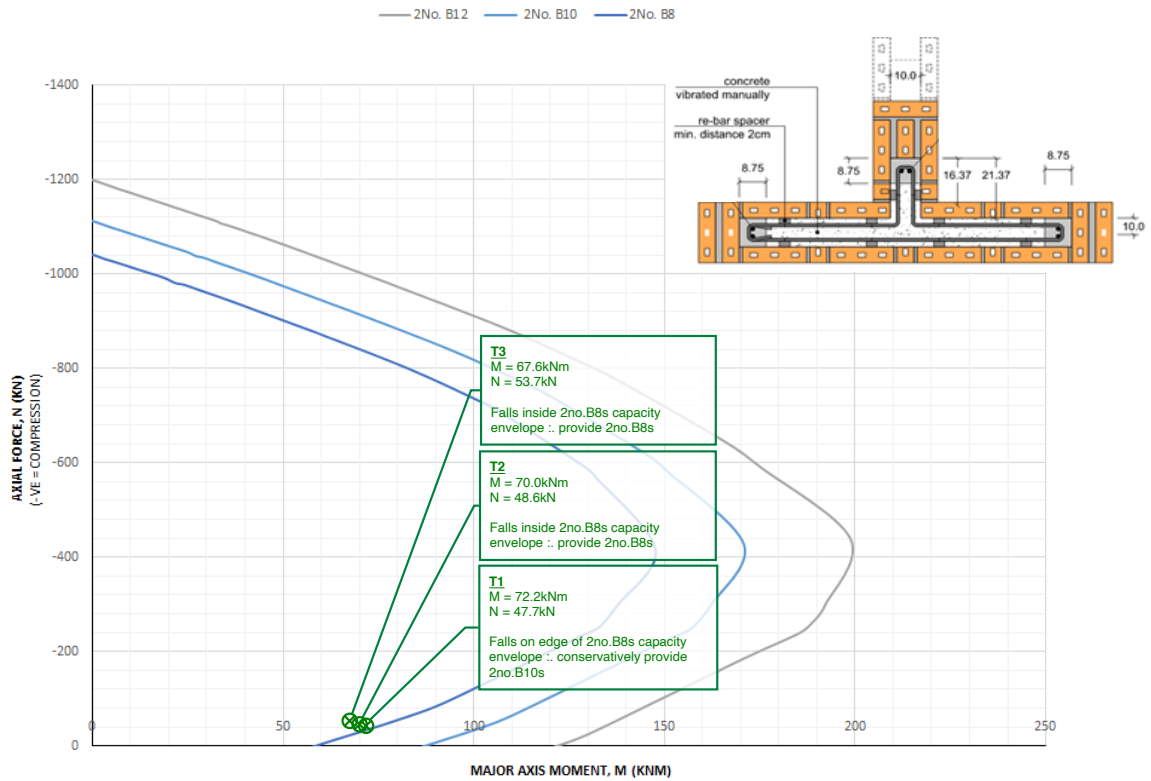
The following pages demonstrate how the forces in each element should be plotted on the relevant M-N chart. The element numbers are shown below for clarity.
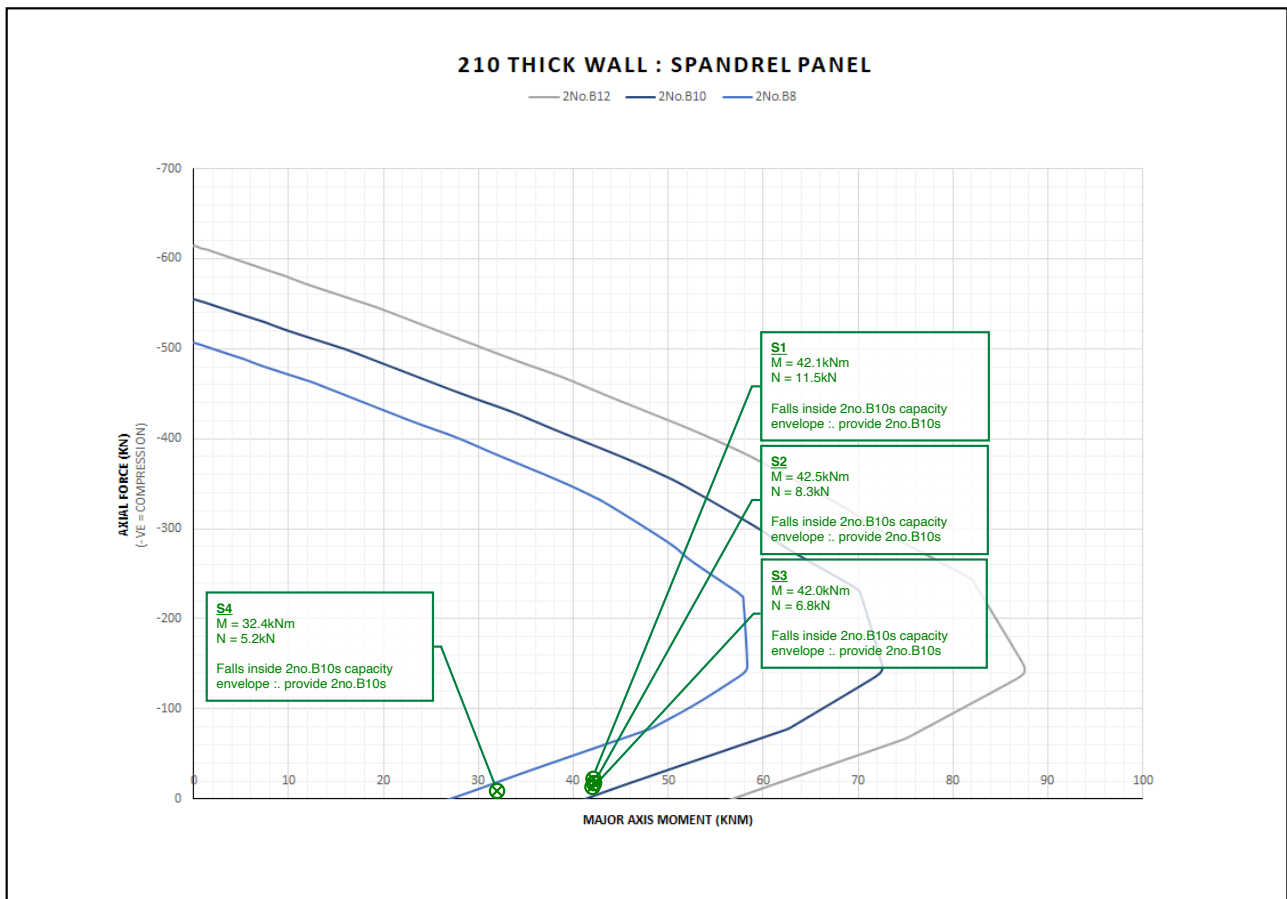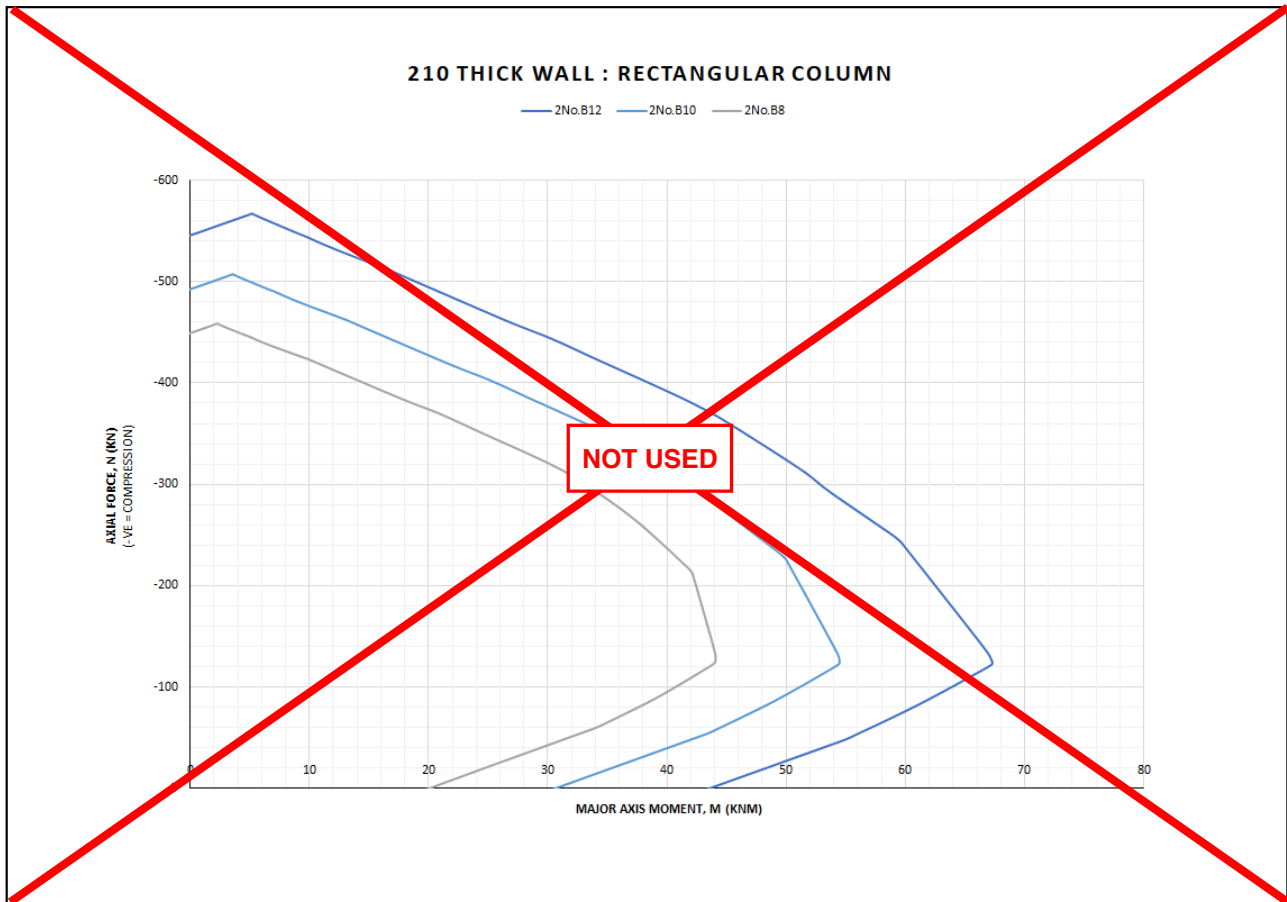
	S1		S2		S3		S4	
L1		T1		T2		T3		L2

210 THICK WALL : 'L'-JUNCTION COLUMN

— 2No.B12   — 2No.B10   — 2No.B8

L2
M = 19.1kNm
N = 40.0kN

Falls inside 2no.B8s capacity
envelope ∴ provide 2no.B8s

L1
M = 24.3kNm
N = 2.6kN

Falls on edge of 2no.B8s capacity
envelope ∴ conservatively provide
2no.B10s

AXIAL FORCE, N (KN)
(-VE = COMPRESSION)

MAJOR AXIS MOMENT, M (KNM)

concrete
vibrated manually

re-bar spacer
min. distance 2cm

210 THICK WALL : 'T'-JUNCTION COLUMN

— 2No. B12   — 2No. B10   — 2No. B8

T3
M = 67.6kNm
N = 53.7kN

Falls inside 2no.B8s capacity
envelope ∴ provide 2no.B8s

T2
M = 70.0kNm
N = 48.6kN

Falls inside 2no.B8s capacity
envelope ∴ provide 2no.B8s

T1
M = 72.2kNm
N = 47.7kN

Falls on edge of 2no.B8s capacity
envelope ∴ conservatively provide
2no.B10s

AXIAL FORCE, N (KN)
(-VE = COMPRESSION)

MAJOR AXIS MOMENT, M (KNM)

concrete
vibrated manually

re-bar spacer
min. distance 2cm

210 THICK WALL : RECTANGULAR COLUMN

— 2No.B12 — 2No.B10 — 2No.B8

NOT USED

AXIAL FORCE, N (KN)
(-VE = COMPRESSION)

-600
-500
-400
-300
-200
-100

MAJOR AXIS MOMENT, M (KNM)

10 20 30 40 50 60 70 80

210 THICK WALL : SPANDREL PANEL

— 2No.B12 — 2No.B10 — 2No.B8

AXIAL FORCE (KN)
(-VE = COMPRESSION)

-700
-600
-500
-400
-300
-200
-100

MAJOR AXIS MOMENT (KNM)

0 10 20 30 40 50 60 70 80 90 100

S1
M = 42.1kNm
N = 11.5kN

Falls inside 2no.B10s capacity
envelope ∴ provide 2no.B10s

S2
M = 42.5kNm
N = 8.3kN

Falls inside 2no.B10s capacity
envelope ∴ provide 2no.B10s

S3
M = 42.0kNm
N = 6.8kN

Falls inside 2no.B10s capacity
envelope ∴ provide 2no.B10s

S4
M = 32.4kNm
N = 5.2kN

Falls inside 2no.B10s capacity
envelope ∴ provide 2no.B10s

05

EXCEL SPREADSHEET

RLB calculation tool for Structural and Civil Engineers

Refer to the **-Global analysis and Shear wall Design.xlsx-** Excel spreadsheet for a comprehensive Row Lock Bond structural calculation tool for buildings up to 3 storeys

COMPANY NAME	client :-		sheet	
	project :-		by	
contact information:	title :-		date	
			chkd	

Ref.	Calculations	Output

Disclaimer
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Spreadsheet key

- Input cells to be completed by user
- Instruction, must be followed
- Output Cell

Introduction
This calculation spreadsheet calculates the seismic force acting along each structural wall line in a single or group of simple housing units built with Skat Rowlock Bond Maosonry Technology.

Limitations of the spreadsheet include - Refer to Construction Manual for more information:
- The housing unit must be rectangular in plan, with no re-entrant corners.
- SKAT RLB junction elements to be continuous over height of the building.
- The unit may have a single additional transverse wall (Spine Wall) centred on the plan
- The housing unit can be single storey, two storey or three storey.
- Each storey of the unit must have the same building footprint i.e. no step backs.
- A structural diaphragm is achieved at each floor level and the roof level.
- All diaphragms are considered to be flexible.
- Up to 5no. identical units can be considered to act as a single building.
- The overall building footprint should not exceed a Length to Width ratio of 4.0

Inputs
Seismic Design Paramaters

Ref.	Symbol	Description	Value	Unit
RBC / GEM	a_g	Peak ground acceleration	1,6	m/s²
g=9.81 m/s²	α	Ratio of pga, gravity	0,16	g
EC8, Fig 3.2 & 3.3		Response Spectrum Type	1	
EC8, Table 3.1		Soil Type	D	
Table 3.2 & 3.3	S	Soil factor	1,35	
EC8, Table 4.3	γ_a	Importance Facor	1	
EC8, Table 4.4	q	Ductility Factor	2	
EC8, Eqn 3.14	S_d	Design seismic acceleration on plateau	0,275	g

Geometry

Ref.	Symbol	Description	Value	Unit
SKAT RLB Construction Manual	L	Unit Length	9	m
	W	Unit Width	4	m
	A	Area of Unit Footprint	36,0	m²

Ref.	Symbol	Description	Value	Unit
	n	Number of storeys	2	
	h_G	Ground Floor Height	2,5	m
	h_1	First Floor Height	2,5	m
	h_2	Not Used, Set as 0.0m	2,5	m
	h_P	Parapet Height, maximum 1m height	1,0	m
	H	Building Height	6,00	m
EC8, Eqn 4.6	T_1	Fundamental Period	0,29	s

COMPANY NAME	client :-		sheet	
	project :-		by	
contact information:	title :-		date	
			chkd	

Ref.	Calculations		Output

	Y/N	Spine Wall Included?	Yes	
	N	Number of Adjacent Units	2	

Construction

	1F	Select Floor Construction	MaxSpan Slab	
	2F	Set as N/A	N/A	
	RF	Select Roof Construction	Timber Trusses/Joists	

	DL	Timber Trusses/Joists	0,40 KN/m²	
	DL	Maxspan Floor	2,60 KN/m²	
	SDL	Floor Superimposed Dead Load (incl. Partitions)	0,40 KN/m²	
	SDL	Roof Superimposed Dead Load	0,20 KN/m²	
EC1, Table 6.2	LL	Live loads	1,50 KN/m²	
	SW	Masonry Walls Selfweight	2,63 KN/m²	
	SDL	Glazing	0,15 KN/m²	

Global Seismic Forces

Ref.				
EC0 6.4.3.4, Table A1.1	Y_w	DL Seismic Contribution Factor	1,0	
	Y_p	LL Seismic Contribution Factor	0,3	
	W_M	Vertical load of Masonry per Metre Height of Wall (all Units)	134 kN/m	
	W_1	Vertical load of First Floor per Unit	124 kN/unit	
	W_2	N/A	N/A kN/unit	
	W_R	Vertical Load of Roof per Unit	22 kN/unit	
EC8, Eqn 4.5	F_b	Total Base Shear	233 kN	

	z_1	Height Above Ground (1F)	2,50 m	
	z_2	Height Above Ground (2F)	N/A m	
	z_R	Height Above Ground (RF)	5,00 m	
	$h_{trib,1}$	Tributary Height of Wall (1F)	2,50 m	
	$h_{trib,2}$	Tributary Height of Wall (2F)	N/A m	
	$h_{trib,R}$	Tributary Height of Wall (RF)	2,25 m	
	m_1	Seismic Weight (1F)	584 kN	
	m_2	Seismic Weight (2F)	N/A kN	
	m_R	Seismic Weight (RF)	345 kN	
	$z_1.m_1$	Height x Seismic Weight (1F)	1459 kNm	
	$z_2.m_2$	Height x Seismic Weight (2F)	N/A kNm	
	$z_R.m_R$	Height x Seismic Weight (RF)	776 kNm	
EC8, Eqn 4.11	F_1	First Floor Seismic Force	152 kN	
	F_2	Second Floor Seismic Force	N/A kN	
	F_R	Roof Sesimic Force	81 kN	

Seismic Forces on Each Wall

Longitudinal Walls

	N_L	Number of Longitudinal Walls	3	
	$F_{L,E,1}$	Force acting on End Walls at First Floor	38,1 kN	
	$F_{L,E,2}$	Force acting on End Walls at Second Floor	N/A kN	
	$F_{L,E,R}$	Force acting on End Walls at Roof	20,3 kN	
	$F_{L,D,1}$	Force acting on Dividing Walls at First Floor	76,2 kN	
	$F_{L,D,2}$	Force acting on Dividing Walls at Second Floor	N/A kN	
	$F_{L,D,R}$	Force acting on Dividing Walls at Roof	40,5 kN	

Transverse Walls

	N_T	Number of Transverse Walls	3	
	$F_{T,F,B,1}$	Force acting on Front/Back Elevation Walls at 1st Floor per unit	19,0 kN/unit	
	$F_{T,F,B,2}$	Force acting on Front/Back Elevation Walls at 2nd Floor per unit	N/A kN/unit	

F. APPENDIX A : NOMENCLATURE

A	area of individual unit
A_{strut}	Area of strut
A_g	peak ground acceleration
A_s	cross cross-sectional area of steel reinforcement
b	width of a section
D_L	dead load
E	modulus of elasticity of masonry
F_b	total base shear
f_b	normalised mean compressive strength of a masonry unit
f_{ck}	characteristic compressive strength of concrete infill
f_k	characteristic compressive strength of masonry;
f_m	compressive strength of masonry mortar
f_{yd}	design tensile strength of reinforcing steel
f_{yk}	characteristic strength of reinforcing steel
$F,_L$	force acting on longitudinal walls
$F,_T$	force acting on transverse walls
h_a	height of wall
h_{panel}	height of wall panel
h_g	ground floor height
h_1	first floor height
$h_{trib,1}$	Tributary height of wall at level 1
$h_{trib,R}$	Tributary height of wall
h_p	parapet height
h_{ef}	effective height of a wall;
L_c	length of the compressed part of a wall
L_{cl}	clear length of an opening
L_{ef}	effective span of a masonry beam;
l_{panel}	length of wall panel
L	Length of unit
L_L	Live load
m_1	Seismic weight at level 1
m_R	Seismic weight at roof level
M_d	design bending moment at the bottom of a core;
M_{md}	design value of the greatest moment at the middle of the height of the wall;
M_{Rd}	design value of the moment of resistance;
M_{Ed}	design value of the moment applied;
n	number of storeys;
N	number of adjacent units

N_{md}	design value of the vertical load at the middle of the height of a wall or column;
N_{Rd}	design value of the vertical resistance of a masonry wall or column;
N_{Rdc}	design value of the vertical concentrated load resistance of a wall;
N_{Ed}	design value of the vertical load;
q	ductility factor
S	Soil factor
S_w	self weight
S_d	Design seismic acceleration
S_{DL}	superimposed dead load
t	thickness of wall
T	fundamental period
t_{min}	minimum thickness of a wall
t_{ef}	effective thickness of a wall;
V_{Ed}	design value of a shear load;
V_{Rd}	design value of the shear resistance
W	Width of individual unit
W_{ed}	design lateral load per unit area;
W_M	Vertical load of masonry wall per meter height of wall
W_1	Vertical load of first floor per unit
W_R	Vertical load of roof per unit
Y_{dl}	dead load seismic contribution factor
Y_{ll}	live load seismic contribution factor
z	lever
Z_1	height above ground level at level 1
Z_R	height above ground at roof level

Greek letters

ϕ	effective diameter of the reinforcing steel;
Φ	reduction factor;
γ_m	partial factor for masonry
$\gamma_{m,s}$	partial factor for steel reinforcement
γ_1	Importance factor
α	ratio of PGA to gravity
$\alpha_{,L}$	ratio of solid longitudinal walls to openings
$\alpha_{,T}$	ratio of solid transverse walls to openings