BURO HAPPOLD

Kazerne Sint-Maarten

Seismic Design Report (Superstructure)

KSM-BH-REP-SS-DD-S-0001

65032

22 July 2026

Revision P03

Copyright © 1976 - 2026 Buro Happold. All rights reserved

Revision	Description	Issued by	Date	Checked
P01				
P02	FINAL DRAFT	AG	5/6/2026	DV
P03	INCORPORATING CLIENT'S COMMENTS	AG	22/7/2026	DV

https://burohappold-my.sharepoint.com/personal/alfredo_gonzalez_burohappold_com/Documents/Projects/StMaartens/DD Final/KSM-BH-REP-SS-DD-S-0001 P03.docx

Report Disclaimer

This Report was prepared by Buro Happold Limited ("BH") in accordance with, and remains subject to, the terms of BH's appointment for the benefit of IMDVB in relation to Purpose of Report. BH responsibility shall be limited to the purpose for which this Report was prepared, with BH assuming no liability for any reliance placed on this Report by any third party.

Notwithstanding the foregoing, the content of this Report shall remain subject to the following:

This Report is based upon the information and knowledge available to BH at the time of writing and, unless otherwise expressly stipulated herein, BH has not sought to verify or validate any data received by BH and/or which may underpin this Report.

BH assumes no ownership nor liability in respect of any third-party design, workmanship and/or deliverables, and shall not be held responsible for the consequences of any negligence, error or omission in respect of the same.

BH does not guarantee any particular outcome, nor does BH provide any warranty in respect of any third-party services, equipment requirements and/or software and is not responsible for the outcome or use of the same; regardless of whether the content of this Report is followed.

Where specifications or materials are mentioned or referred to in this Report, BH does not condone use or supply of materials that would breach any laws, sanctions or other restrictive measures in any jurisdiction including but not limited to use or supply of steel or its components originating from Russia.

This Report shall not be construed as investment or financial advice.

BH reserves the right on notice to withdraw this Report and all associated intellectual property rights, licences or otherwise granted pursuant to BH's appointment should any payments which fall due not be paid in full. Any reliance placed upon the Report in such circumstances shall be deemed null and void.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

author	A Gonzalez
date	22/7/2026
approved	DV
signature	
date	22/7/2026

Copyright © 1976 - 2026 Buro Happold. All rights reserved

Contents

1 Introduction 7

1.1 Project Description 7

1.2 Scope and Objectives 7

2 Structure 9

2.1 Structural Actions 9

2.2 Limit States 9

2.3 Primary and Secondary Elements, and Non-Structural Components 9

2.4 Lateral Force-Resisting System 10

2.5 Base Level 10

2.6 Regularity in Elevation 12

2.7 Regularity in Plan 13

2.8 Full Plastic Mechanism 14

2.9 Capacity Protection 15

2.10 Gravity System and Secondary Elements 15

2.11 Foundations 15

2.12 Movement Joints 16

3 Design Criteria 17

3.1 Model Description 17

3.2 Representation of Seismic Hazard 20

3.2.1 Ultimate Limit State 20

3.2.2 Serviceability Limit State 22

3.2.3 Modal and Directional Combination 22

3.3 Gravity Loads 24

3.4 Global Results: Mode Shapes, Periods and Participation Factors 24

3.5 Inter-Storey Drift 27

3.6 Storey Forces and Base Shear 29

3.6.1.1 Storey Shears 30

Copyright © 1976 - 2026 Buro Happold. All rights reserved

4 Element Design at ULS **32**

4.1 Second-Order Effects 32

4.2 Shear Walls 32

4.3 Concentrically-Braced Steel Frames 39

4.4 Manual Checks 39

4.5 Reactions to Foundations 40

4.6 Indicative Details 40

Appendix A. Model Details – A1. Loading Plans: SDL – A.2 Loading Plans: Imposed Loading

Appendix B. Hand Calculations

Appendix C. ETABS Foundation Forces

Appendix D. ETABS Report

Appendix E. ETABS Element Forces – E.1 Spandrel Forces – E.2 Pier Forces – E.3 Column Forces (Primary) – E.4 Brace Forces – E.5 Beam Forces (Primary)

Table of Tables

Table 1. EC8 criteria for regularity in elevation **12**

Table 2. EC8 criteria for regularity in plan **13**

Table 3. Calculation of eccentricities **14**

Table 4. Gravity loads **24**

Table 5. Mode periods and participation factors **25**

Table 6. Storey shears and moments **30**

Table 7. Pier design results **35**

Table of Figures

Figure 1. Structural system for the building showing (top) the Revit model and (bottom) the ETABS model.8

Figure 2. Seismic base level and scope of the super-structure design (this report). 11

Figure 3. Setbacks in elevation 12

Figure 4. Mass and stiffness distribution along height. 13

Figure 5. Plan aspect ratio. 14

Figure 6. Distribution of plan eccentricity. 14

Figure 7. Analysis model – plan view of level 01. 18

Figure 8. Analysis model – plan view of level 02. 18

Copyright © 1976 - 2026 Buro Happold. All rights reserved

Figure 9. Analysis model – plan view of level 03. 19

Figure 10. Analysis model – plan view of level 04. 19

Figure 11. Seismic Hazard per Global Earthquake Hazard Map, PGA on rock, 475 years return period (10% PE in 50 years). 20

Figure 12. Seismic Hazard per Geotechnical Report. 20

Figure 13. Spectrum settings in ETABS. 21

Figure 14. Mass definition in ETABS. 22

Figure 15. Definition of static seismic load cases in ETABS. 23

Figure 16. Definition of response spectrum analysis load cases in ETABS. 23

Figure 17. First three mode shapes – plan view, in the usual order. 26

Figure 18. Deformation from response spectrum analysis. 27

Figure 19. Inter-storey drift plot. 28

Figure 20. Forces to storeys and base shear. 29

Figure 21. Design storey shear comparison. 31

Figure 22. Response of higher modes. 31

Figure 23. Example buckling mode showing slender columns buckling out of plane prior to sections being enlarged. 32

Figure 24. Shear walls in the model 33

Figure 25. Pier numbering for design 34

Figure 26. Design results for steel elements. The colours represent the utilisation factor as shown in the key at the bottom of the figure. 39

Figure 27. RC Column Indicative details 41

Figure 28. RC wall indicative details 42

Figure 29. Indicative steel connection details 43

Figure 30. Steel splices indicative details 44

Figure 31. Steel brace and embedded beam indicative details 45

Figure 32. Indicative connection details for low roof 45

Figure 33. Indicative connections between low roof and main building 46

Figure 34. Brace connections, low roof 46

Copyright © 1976 - 2026 Buro Happold. All rights reserved

1 Introduction

1.1 Project Description

Kazerne Sint-Maarten is a 4-storey building to house marine equipment, training facilities, and offices. The main building is a reinforced concrete box approximately 16 x 50m in plan. The surrounding structures are a mixture of timber and steel low-rise structures and canopies.

The lateral system consists of braced frames in the upper floors, and reinforced concrete walls in the lower floors.

The foundation system consists of a grade beam supported on piles.

1.2 Scope and Objectives

The document presented herein is intended to describe the level of design provided for the Detailed Design Stage for the lateral force resisting system (LFRS) only. The information contained in this report has been based on the adequate level of calculations expected for the design stage, good practice, experience, the requirements given by the lead designer. The seismic hazard and soil conditions are obtained from the ground engineering report provided by the lead designer.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

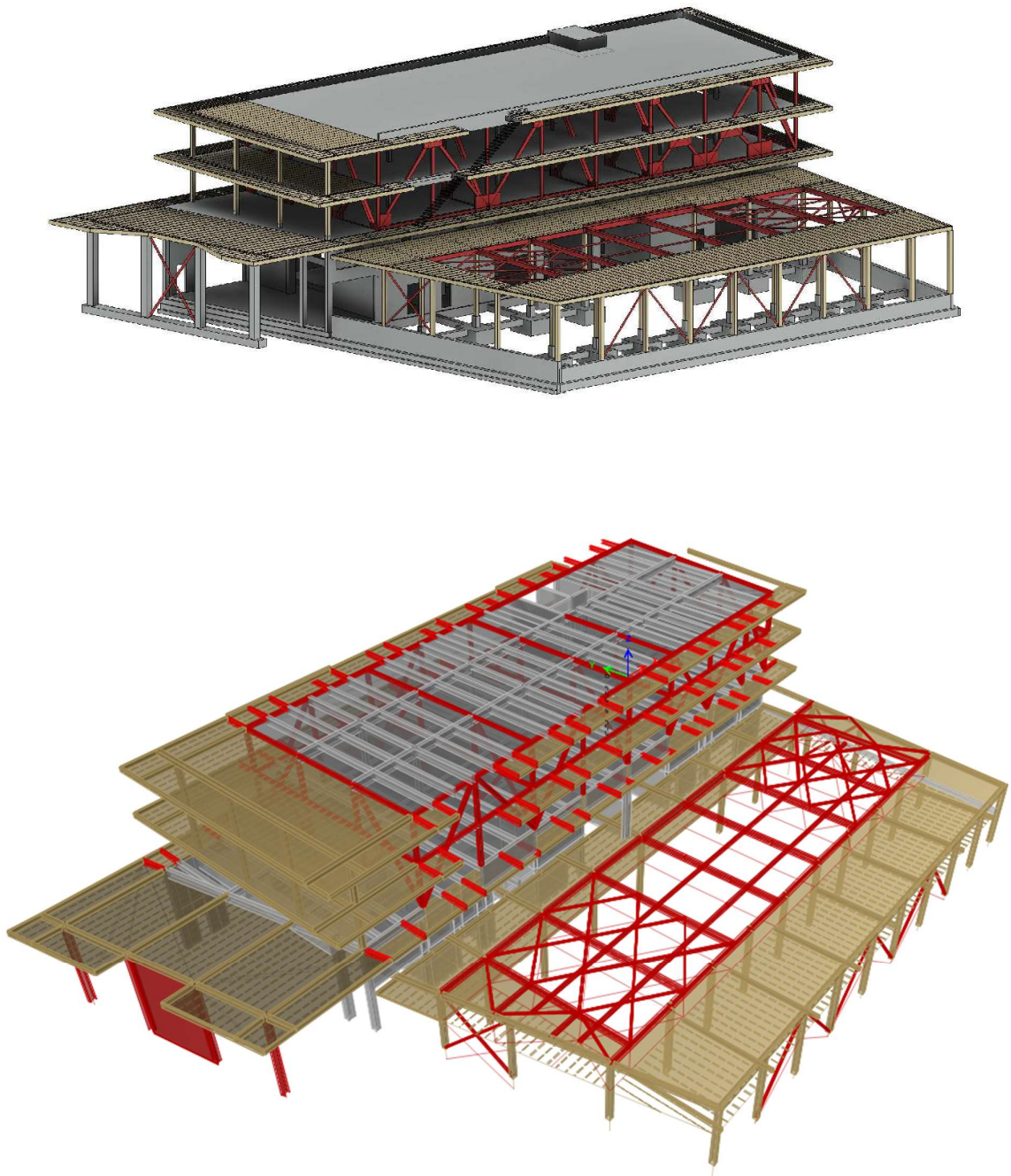


Figure 1. Structural system for the building showing (top) the Revit model and (bottom) the ETABS model.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

2 Structure

2.1 Structural Actions

The following structural actions are considered:

- gravity loads:
  - self-weight of materials;
  - superimposed dead loads, and
  - imposed loads;
- seismic actions

Actions that are not deemed to act in conjunction with seismic actions are not considered in this report. Such actions include wind load, restrained temperature changes, creep, and restrained shrinkage. The effects of these actions, if significant, should be considered separately by the lead designer.

2.2 Limit States

Two design limit states are considered in the design of the building as required by the applicable building codes:

- Serviceability Limit State (SLS), and
- Ultimate Limit State (ULS)

The Serviceability Limit State verifies that the structure does not suffer damage requiring repair under frequently occurring loads, such as everyday occupancy of the building or moderate earthquakes.

The Ultimate Limit State verifies that the structure is strong enough to survive extreme loads, such as rare overloads, infrequent winds or extreme earthquakes.

The actions for which the ULS is verified are severe but can be exceeded. This is especially true of lateral loads. Earthquake loads at ULS, in particular, are defined as having a 10% chance of being exceeded during the life span of the structure (50 years for the No-Collapse requirement), and therefore the design must incorporate safeguards against sudden collapse beyond ULS seismic intensity.

It should also be taken into account by the building owners that damage is only controlled at SLS under applicable building code prescriptions, and not at ULS. This means that a structure that behaves as expected by the code during a ULS-level event could be uneconomical to repair.

2.3 Primary and Secondary Elements, and Non-Structural Components

With the Eurocode 8 terminology and design approach, there is a distinction between primary and secondary seismic structural elements. The primary elements are those that are relied upon to resist lateral forces and provide the required lateral stiffness to control deformations. Both functions (strength and stiffness) are essential to the performance of the structure during an earthquake. The primary elements

Copyright © 1976 - 2026 Buro Happold. All rights reserved

must be designed within strict constraints regarding the amount of gravity load they can take, and are subject to more involved detailing than elements not relied upon for seismic design.

Eurocode 8 allows the designer to consider some elements which do not significantly contribute to lateral strength and stiffness as secondary elements. Secondary elements are ignored in the design for lateral forces, and they are subject to less stringent load limits and detailing requirements, as long as it is demonstrated that they can sustain the deformations of the structure without significant damage and without losing the ability to support gravity loads. In this project, the floor slabs, floor beams and gravity columns are considered secondary. The design of secondary elements is not covered by this report.

Non-structural components support only their weight and inertial forces and must not engage with the lateral deformations of the structure. This is usually achieved either by making sure they either: a) are very flexible and can deform alongside the structure, b) the method of their support to the structure is such that they can move independently of it during an earthquake, or c) sufficient gaps are provided between them and the structure. It is outside the scope of this report to discuss non-structural elements. An estimation of the deformation of the structure at SLS and ULS is included in this report for consideration by the designers of non-structural components.

2.4 Lateral Force-Resisting System

The Lateral Force-Resisting System (LFRS) serves the purpose of withstanding lateral forces induced by wind and earthquakes. Although it also supports gravity loads, the configuration of the LFRS is determined by the lateral forces. The Limit States (LS) described in 2.2 need to be satisfied by either being able to achieve specific levels of strength and/or deformation limits (normally considered to protect secondary structural elements and non-structural components).

A clear and effective lateral stability arrangement is key for the achievement of the structural performance without compromising the architectural aspirations of the building, especially in earthquake-prone countries like St Maarten.

The LFRS for this building consists of a two-storey box-type structure made up of stocky reinforced concrete (RC) shear walls in both directions, supporting a two-storey concentrically-braced steel frame. An adjacent single-storey steel-and-timber roof is connected to the main building for lateral stability. All floors are reinforced concrete slabs on beams, doubling as horizontal diaphragms.

2.5 Base Level

The term “base level” has a specific meaning in the context of Eurocode 8 and seismic analysis. It is the level below which the structure becomes so much stiffer that it can be ignored for most analysis purposes. This happens, typically, at the ground level, below which foundation restraint and/or basement walls drastically increase the lateral stiffness. It is common practice to model the structure above the base level only, at least for response spectrum and equivalent lateral force analyses.

The slab at the base level acts as rigid lateral support and the rigid foundation system is considered to provide fixity against translation and rotation for the walls.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

The location of the base level has been defined at the top of the ground-level slab for this project. The structure that continues below the base level, including grade beam and walls for water tanks, are designed to remain essentially elastic.

The analysis and design of elements below the base level is outside the scope of this report.

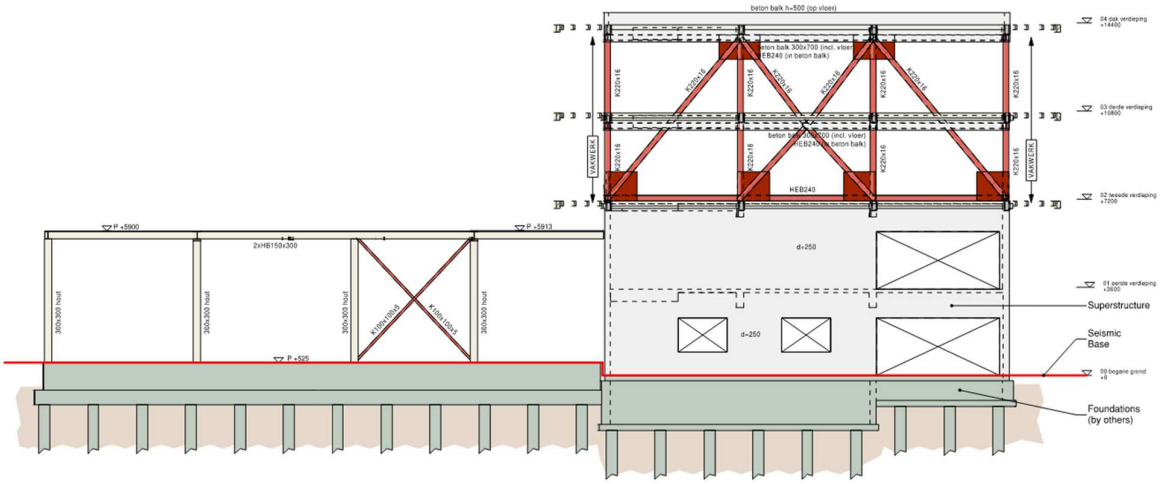


Figure 2. Seismic base level and scope of the super-structure design (this report)**.**

Copyright © 1976 - 2026 Buro Happold. All rights reserved

2.6 Regularity in Elevation

A structure which is irregular in elevation tends to be less capable to dissipate energy than regular structures. This is due to mainly two factors: concentration of ductility demand, and increased effects of higher modes. Therefore it is necessary to determine whether a structure can be classified as regular in elevation before selecting an appropriate ductility reduction factor for design.

EC8 section 4.2.3.3 provides criteria for regularity in elevation. An assessment of these criteria for this building is shown in Table 1, below. Two factors undermine regularity: a) there are setbacks in elevation greater than 10% of the plan dimension as shown in Figure 3, and b) the concentrically-braced structure in the upper floors is much less rigid than the box in the lower floors as shown in Figure 4. As a result, the structure is classified as irregular in elevation.

Table 1. EC8 criteria for regularity in elevation

Criteria	Criteria met
Uninterrupted lateral systems from foundations to top	Yes
Constant or gradual reduction in mass	Yes
Constant or gradual reduction in stiffness	No
Constant over-strength	Yes
Asymmetric setbacks smaller than 10%	No

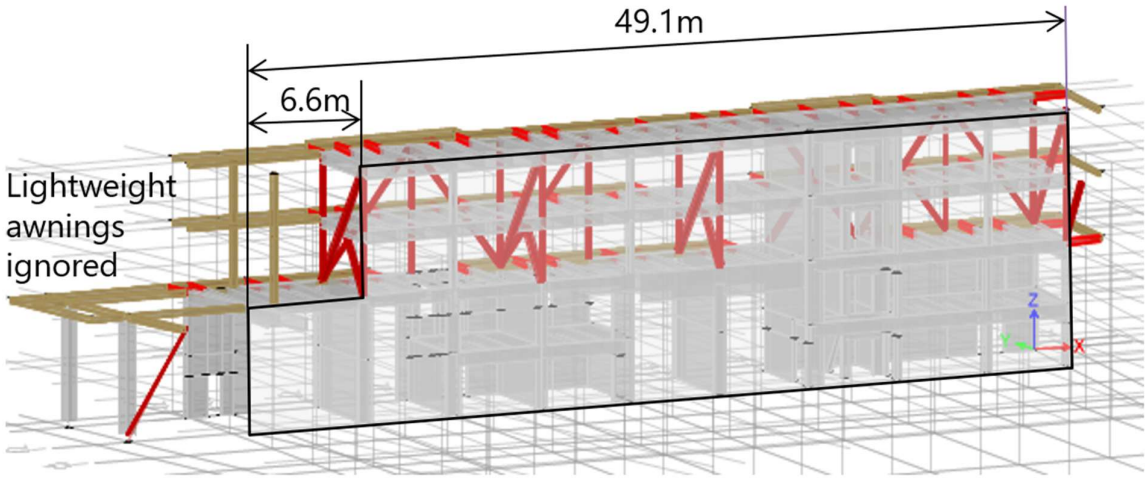


Figure 3. Setbacks in elevation

Copyright © 1976 - 2026 Buro Happold. All rights reserved

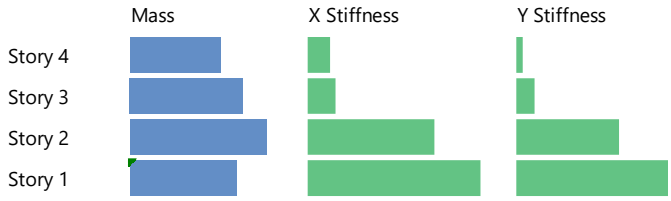


Figure 4. Mass and stiffness distribution along height.

2.7 Regularity in Plan

A structure irregular in plan will have a greater tendency to twist during an earthquake, inducing actions that alternately add and subtract from seismic actions within the LFRS. This leads to an imbalance in ductility demand and therefore a reduction in available ductility. A structure's regularity in plan must therefore be assessed to be able to determine its force reduction factor for design.

An assessment of EC8's section 4.2.3.2 criteria for regularity in plan is shown in Table 2. The eccentricity and diaphragm discontinuity of the first story lead to a classification of irregular in plan. The structure is not 'torsionally flexible' as shown later in this report.

Table 2. EC8 criteria for regularity in plan

Criteria	Criteria met
Approximately symmetrical mass distribution	Yes
Approximately symmetrical stiffness distribution	Yes
Compact plan configuration	No
• Stories 2 & 3	Yes (Figure 5)
• Story 1	No
Large in-plan stiffness	Yes
Plan aspect ratio < 4	Yes
Eccentricity < 8.6% of plan dimension	No (Table 3, Figure 6)
• Stories 2/3	Yes
• Story 1	No

Copyright © 1976 - 2026 Buro Happold. All rights reserved

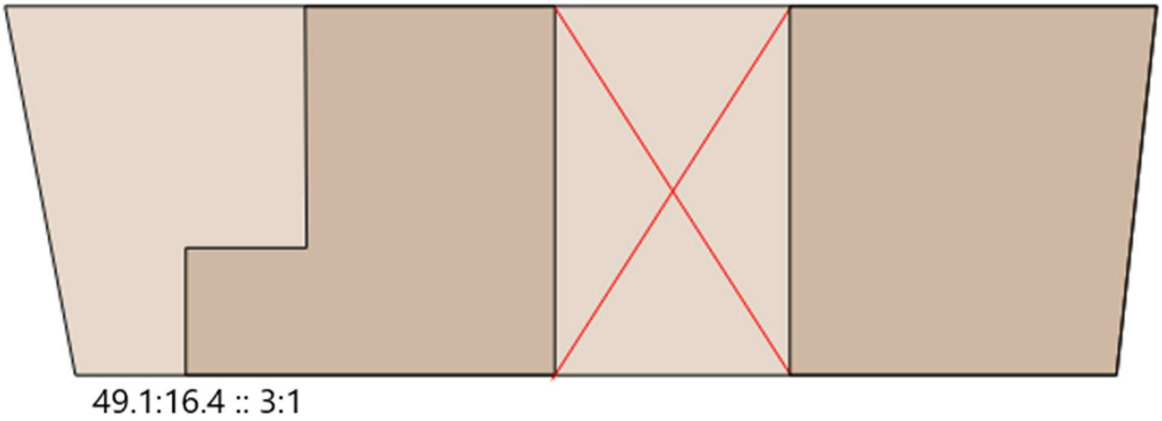


Figure 5. Plan aspect ratio.

Table 3. Calculation of eccentricities

Story	Diaphragm	Mass X	Mass Y	XCM	YCM	Cum Mass X	Cum Mass Y	XCCM	YCCM	XCR	YCR	Lx	Ly	ex	ey
		kg	kg	m	m	kg	kg	m	m	m	m	m	m	m	m
Story4	D1	517392.93	517392.9	-22.6791	-8.0895	517392.9	517392.9	-22.6791	-8.0895	-17.3409	-7.9886	49.10	49.10	5.34	0.10
Story3	D1	777621.71	777621.7	-23.0031	-8.1344	1295015	1295015	-22.8737	-8.1165	-20.8099	-8.0681	49.10	49.10	2.06	0.05
Story2	D1	915337.68	915337.7	-27.3282	-7.9938	2210352	2210352	-24.7184	-8.0657	-24.2906	-7.5186	55.70	55.70	0.43	0.55
Story1	D2	326379.42	326379.4	-33.6492	-9.0403	326379.4	326379.4	-33.6492	-9.0403	-32.2767	-8.1144	16.20	16.20	1.37	0.93
Story1	D3	351043.32	351043.3	-8.067	-8.3142	351043.3	351043.3	-8.067	-8.3142	-12.6953	-6.6952	16.20	16.20	4.63	1.62
Story1	D4	100539.83	100539.8	-24.4831	-27.9188	100539.8	100539.8	-24.4831	-27.9188	-25.4681	-33.1824	55.00	22.00	0.98	5.26

	ex/Lx	ey/Ly
Story4	11.5%	0.1%
Story3	4.1%	0.0%
Story2	0.6%	0.9%
Story 1/F-I	8.3%	5.6%
Story 1/A-D	28.4%	9.4%

Figure 6. Distribution of plan eccentricity.

2.8 Full Plastic Mechanism

A pre-defined so-called plastic mechanism is considered to anticipate the preferred structural behaviour under a seismic scenario. Specifically detailed zones called plastic hinges are located within shear walls and in the steel braces to achieve the aforementioned preferred plastic mechanism. Plastic hinges represent the zones within the structural member where damage under cyclic loading can occur to dissipate the kinematic energy induced by inertia forces yet preventing the building from collapse under the ultimate limit state. Furthermore, plastic hinges are designed based on the pre-defined ductility level, hence their capability to accommodate deformations without excessive strength reduction.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

A clear plastic mechanism is important to ensure a good performance beyond the ULS level and therefore to avoid premature collapse.

The full plastic mechanism consists of flexural plastic hinges forming at the base of the walls and through yielding of the braces in tension.

2.9 Capacity Protection

Capacity design principles are used to protect the structure against non-ductile failure modes, as follows:

- Shear in walls: amplify the shear from analysis by a factor of between 1.5 and q, then size the shear reinforcement following EC2 rules assuming a strut angle of 45 degrees.
- Steel and concrete columns supporting braces: design vertical elements assuming that the braces connected to them have yielded, thus ensuring that the columns will be able to carry gravity loads while the LFRS dissipates energy by deforming into the inelastic range
- Foundations: design foundations to remain essentially elastic while the superstructure develops its plastic mechanism
- Connections: design steel-to-steel and steel-to-concrete connections to develop the ultimate strength of the connecting members where inelastic action is expected (braces)

2.10 Gravity System and Secondary Elements

The gravity system is not part of the scope of this document. Following EC8 principles, the contribution of gravity and secondary elements to lateral resistance is ignored, and these elements are not designed to resist seismic forces. Horizontal elements in this project do not meet the code threshold for considering vertical seismic loads. See also Section 2.3.

2.11 Foundations

The foundation system is a pile-supported grade beam. The piles are assumed unable to carry net tension. The design of the foundation system is outside the scope of this document, but reactions are provided for the designer.

The piles must take compression loads from a combination of gravity loads and overturning from lateral loads, and shear and moment internal actions to equilibrate the seismic base shear. In resisting the seismic base shear, the piles can be assisted by whatever passive thrust can be developed by the ground beams bearing into the soil.

The geotechnical report rules out liquefaction up to the ULS level. However, it is recommended that the piles are detailed with a sufficient level of ductility to develop a plastic mechanism and thus avoid a brittle failure beyond ULS.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

2.12 Movement Joints

The building has been designed as one single structure. Due to its compact size, movement joints are not expected to be required.

Due to the difference in height between the low-level roof and any of the diaphragms, and to avoid lateral forces from the low roof to apply a transverse force on the relatively slender shear walls, connections between the low roof and the main structure are limited to those locations where forces can be transferred directly to perpendicular walls.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

3 Design Criteria

3.1 Model Description

Base Level

The base level of the model, as defined in EC8, is taken as the ground level slab.

Mass

The mass is modelled as 100% of the gravity loads plus a fraction of the live loads. Permanent live loads such as planting and equipment are not reduced (but these are not individually identified in this project); occupancy live loads are reduced to 30% following EC8 rules. Roof live loads are not accounted towards the reactive mass.

Damping

Modal damping is taken as 5% for all modes, following the standard EC8 approach and usual design practice.

Floor Diaphragms

For the estimation of the global structural response, the diaphragms are modelled as rigid.

Walls

Walls are modelled using shell elements with a thin-shell formulation. The in-plane stiffness is reduced to 0.5.

Coupling Beams and Spandrels

Coupling beams and spandrels are modelled using shell elements with their stiffness properties reduced to 0.5.

Gravity System

For the lateral analysis, the contribution to strength and stiffness of the gravity system is ignored. In other words, the components are considered secondary elements as per EC8. Gravity columns and beams are modelled as pinned at both ends; slabs are modelled with their out-of-plane stiffness properties reduced.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

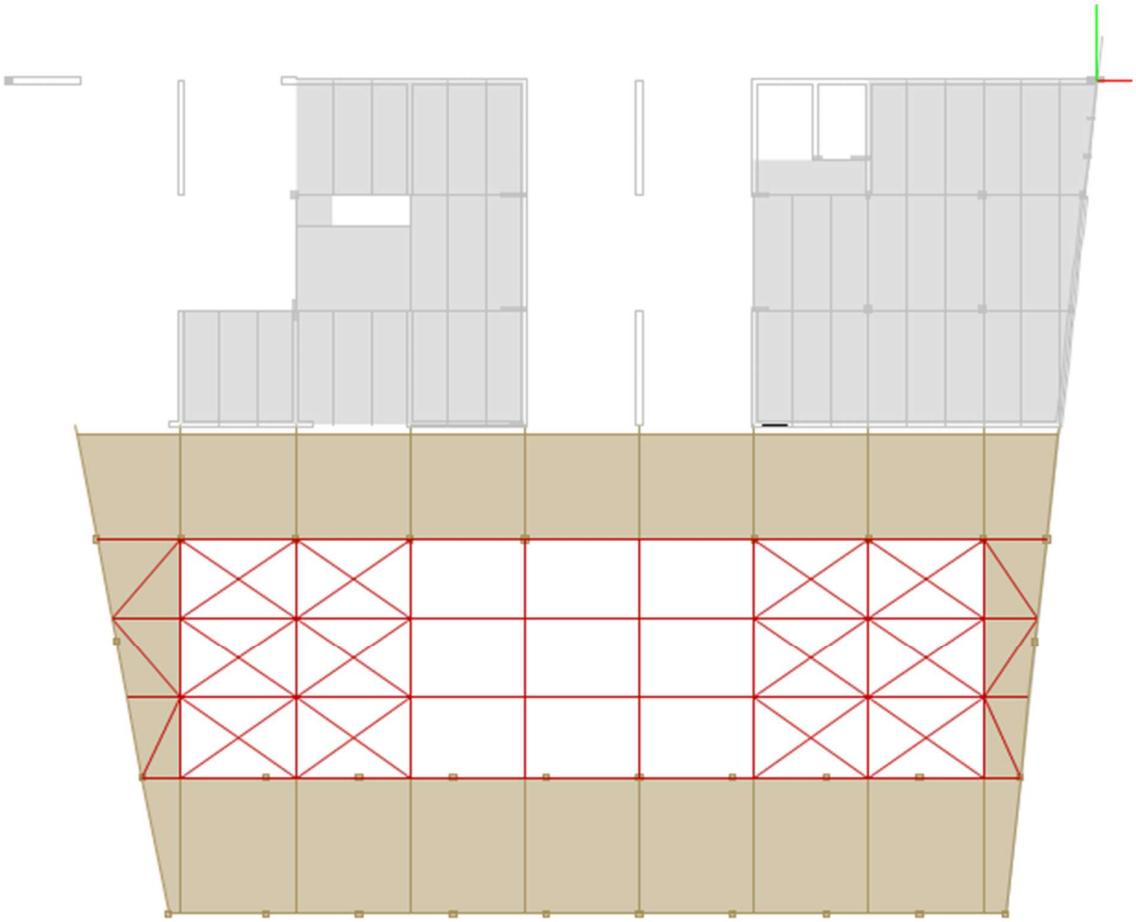


Figure 7. Analysis model – plan view of level 01.

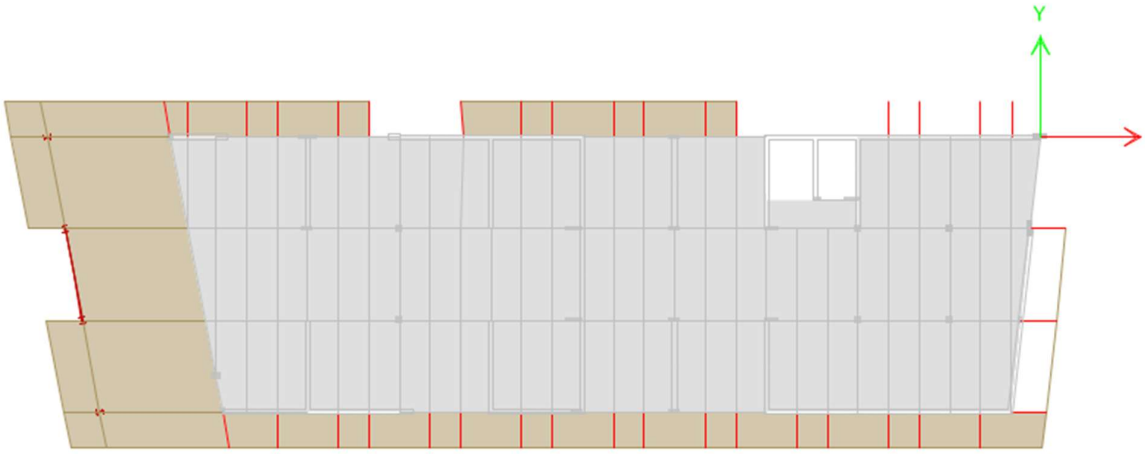


Figure 8. Analysis model – plan view of level 02.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

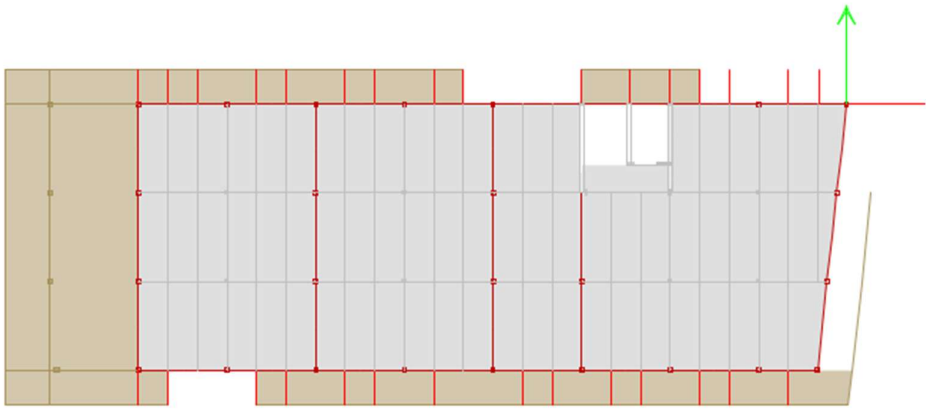


Figure 9. Analysis model – plan view of level 03.

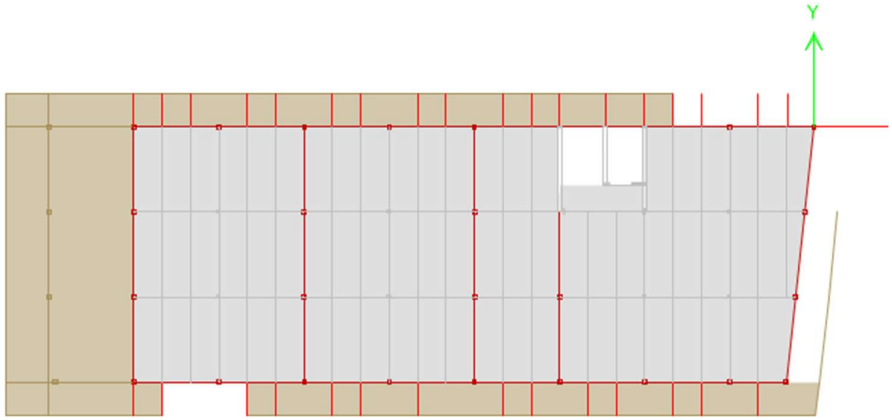


Figure 10. Analysis model – plan view of level 04.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

3.2 Representation of Seismic Hazard

The seismic hazard for the site in terms of 475 year return period events, is identified from two sources: the Global Earthquake Hazard map shows it as 0.26g, as shown in Figure 11, and the geotechnical report identifies it as ranging between 0.3g and 0.35g as shown in Figure 12. Therefore, a value of 0.3g is selected for design.

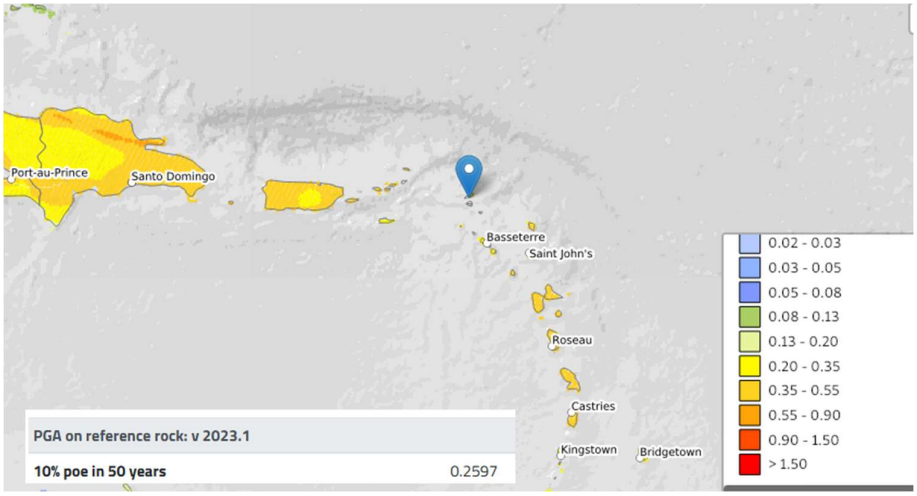


Figure 11. Seismic Hazard per Global Earthquake Hazard Map, PGA on rock, 475 years return period (10% PE in 50 years).

	Island	Sint Maarten	Saba	Sint Eustatius
	Capital	Philipsburg	The Bottom	Oranjestad
	Latitude capital	18.029	17.626	17.482
	Longitude capital	-63.045	-63.249	-62.983
Hazard parameter	Return period (years)			
Peak Ground Acceleration, PGA	95	0.15-0.20	0.15-0.20	0.15-0.20
	475	0.30-0.35	0.25-0.30	0.25-0.30
	975	0.35-0.40	0.35-0.40	0.35-0.40
	2475	0.50-0.60	0.50-0.55	0.50-0.55
Spectral Acceleration, SA (0.2 s)	95	0.3-0.4	0.3-0.4	0.3-0.4
	475	0.7-0.8	0.7-0.8	0.7-0.8
	975	1.0-1.2	0.8-1.0	0.8-1.0
	2475	1.4-1.5	1.3-1.4	1.4-1.5
Spectral Acceleration, SA (1.0 s)	95	0.05-0.10	0.05-0.10	0.05-0.10
	475	0.20-0.25	0.20-0.25	0.20-0.25
	975	0.25-0.30	0.25-0.30	0.25-0.30
	2475	0.40-0.45	0.40-0.45	0.40-0.45

Figure 12. Seismic Hazard per Geotechnical Report.

3.2.1 Ultimate Limit State

- Mapped PGA(10%/50y)=0.26g
- BSA code Z=0.3 → use $\mathbf{a_{gR}}$ **= 0.3g**

Copyright © 1976 - 2026 Buro Happold. All rights reserved

- Ductility class DCM
- NOT torsionally flexible
- Irregular in plan (split diaphragm)
- Irregular in elevation (stiffness change)
- Concentrically braced steel frame above, q_0=4.0
- Large lightly reinforced shear walls below, **q_0=3.0**
- $q = 0.8q_0 = 2.4$
- Soil class B, no liquefaction, S=1.2
- Spectrum type I
- $V_{base} = (2.5 \cdot a_{gR} \cdot S/q) \cdot W = 0.375 \cdot W$

The spectrum for ULS design is based on EC8 Type I as shown in in Figure 13; the mass settings are shown in Figure 14.

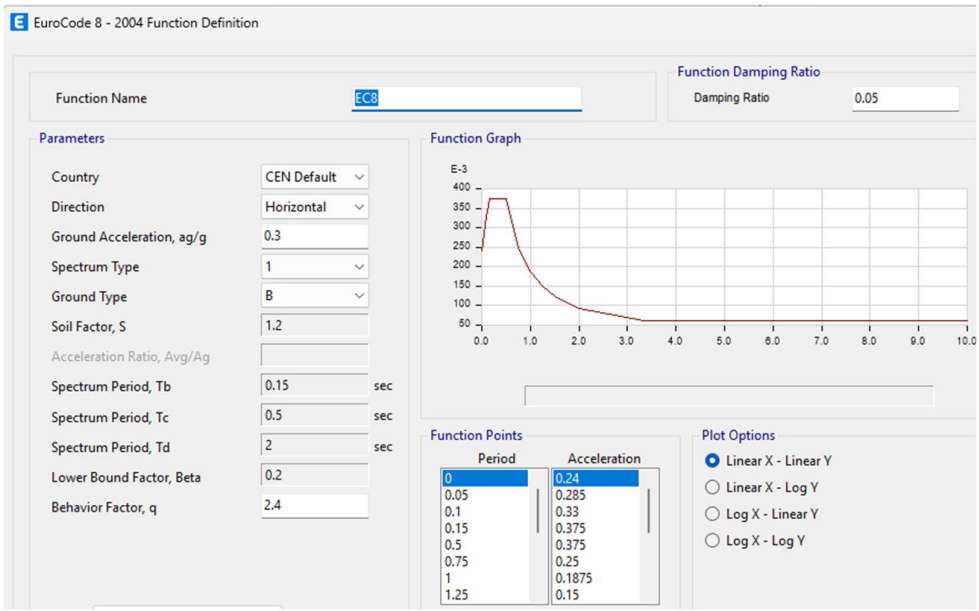


Figure 13. Spectrum settings in ETABS.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

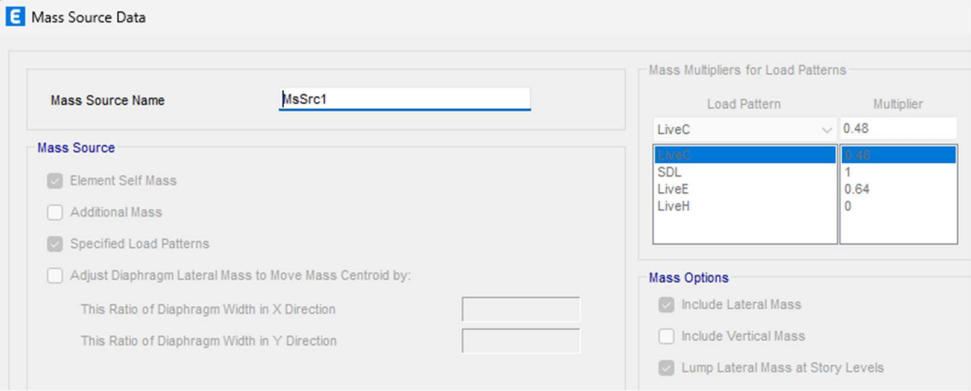


Figure 14. Mass definition in ETABS.

3.2.2 Serviceability Limit State

Same as the above, with ν = 0.5 considered in order to reduce the return period of the seismic demand from 475 years to 95 years. Displacements and drifts are amplified by q to account for the inelastic response.

3.2.3 Modal and Directional Combination

The modal combination method is CQC. The directional combination is SRSS. The load patterns and load case definitions showing the combination types are shown in Figure 15 and Figure 16.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

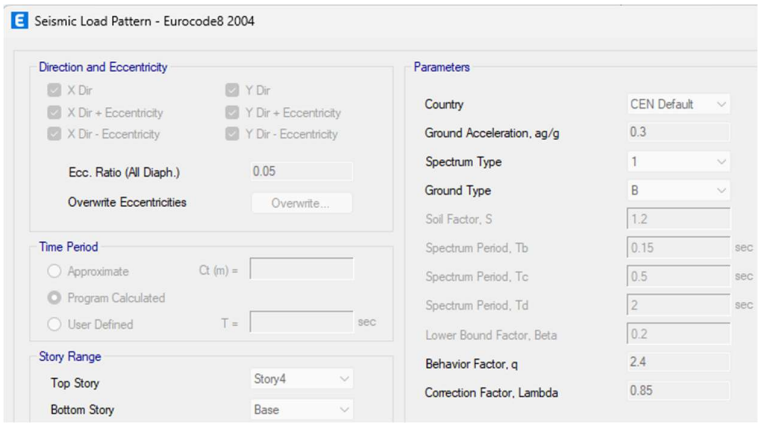


Figure 15. Definition of static seismic load cases in ETABS.

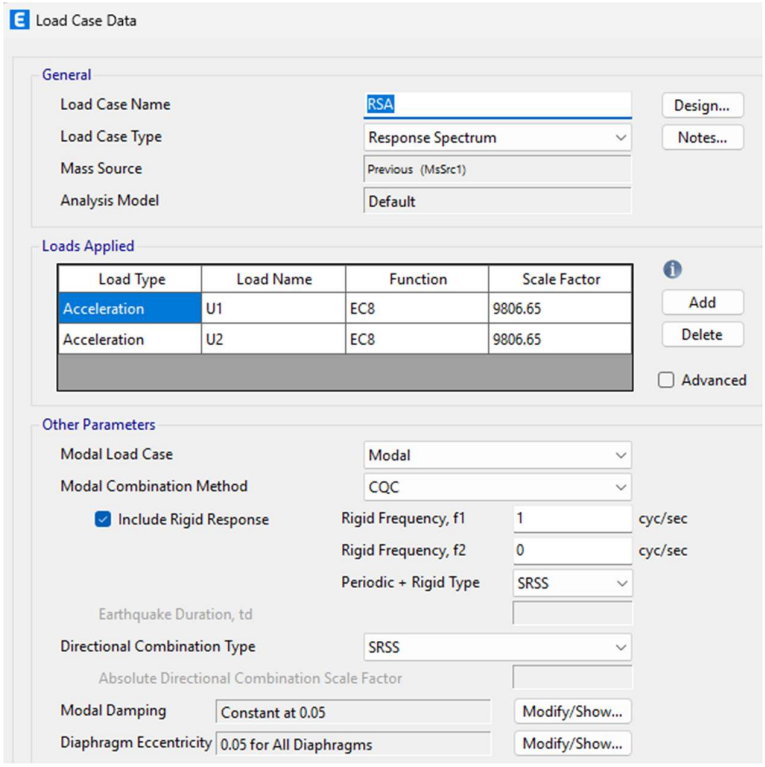


Figure 16. Definition of response spectrum analysis load cases in ETABS.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

3.3 Gravity Loads

The gravity loads are applied as area loads in the model. These are also used to determine the reactive mass for seismic analysis.

Table 4. Gravity loads

Level	Area	Component	Weight	SDL	LL
0	Concrete floor			0	10
1	Concrete floor	Screed	1.6	1.8	8
		Ceiling & MEP	0.2		
	Terrace			0	2.5
	Awning			0	1
2/3	Concrete floor	Screed	1.6	1.8	4
		Ceiling & MEP	0.2		
	Terrace			0	2.5
	Awning			0	1
4	Concrete roof	PV panels	0.2	0.55	2
		Ceiling & MEP	0.2		
		Insulation	0.15		
	Terrace			0	2
	Awning			0	1

3.4 Global Results: Mode Shapes, Periods and Participation Factors

The modal analysis results shows an acceptable dynamic behaviour of the structure. The first two modes are mostly translational, with significant coupling between the two main orthogonal axis due to the presence of diagonal walls and L-shaped walls. This coupling, however, is already taken into account in the design by the use of a response spectrum analysis approach, the directional combination rules, and by considering the structure as irregular in plan.

The period of the structure in the first two modes is between 0.2s and 0.21s, which is fairly rigid but expected for a structure whose bottom half is a rigid concrete box.

The mass participation factors are low, owing to the fact that the upper part of the structure responds quite differently from the bottom leading to increased influence of higher modes.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

The third mode is mostly torsional and has a period of 0.16s, fairly lower than the translational modes. It is therefore considered that the structure is not-torsionally flexible for the purposes of EC8 classification.

In summary:

- 1st Mode (mostly translational in Y)
  - T = 0.21s
  - 32% UX (low), 11% RZ
- 2nd Mode (mostly translational in X)
  - T = 0.20s
  - 51% UX
- 3rd Mode (mostly rotational)
  - T = 0.16s
  - 40% RZ, 19% UY
- Tz < Tx,Ty satisfies EC8 Eq. 4.1b
- Modes 1-10 capture >90% of the mass (16 modes capture 99%)
- Highly Torsional Response, But NOT Torsionally Flexible

The modal periods and mass participation factors are listed in Table 5 below. The first three mode shapes are shown in Figure 17.

Table 5. Mode periods and participation factors

Mode	Period sec	UX	UY	SumUX	SumUY	SumUZ	RZ	SumRZ
1	**0.210**	0.03	**0.32**	0.03	0.32	0.00	0.11	0.11
2	**0.200**	**0.51**	0.01	0.54	0.33	0.00	0.03	0.14
3	**0.160**	0.01	0.19	0.55	0.52	0.00	**0.40**	0.54
4	0.110	0.04	0.00	0.59	0.52	0.00	0.00	0.54
5	0.100	0.00	0.11	0.59	0.63	0.00	0.09	0.62
6	0.080	0.22	0.00	0.81	0.63	0.00	0.06	0.68
7	0.070	0.06	0.05	0.86	0.68	0.00	0.00	0.68
8	0.070	0.00	0.00	0.87	0.69	0.00	0.02	0.70
9	0.070	0.11	0.03	0.97	0.71	0.00	0.08	0.78
10	0.060	0.00	0.00	0.97	0.71	0.00	0.17	0.95
11	0.060	0.00	0.22	0.98	0.93	0.00	0.00	0.95
12	0.050	0.00	0.00	0.98	0.93	0.00	0.00	0.95
13	0.040	0.00	0.03	0.98	0.96	0.00	0.01	0.96

Copyright © 1976 - 2026 Buro Happold. All rights reserved

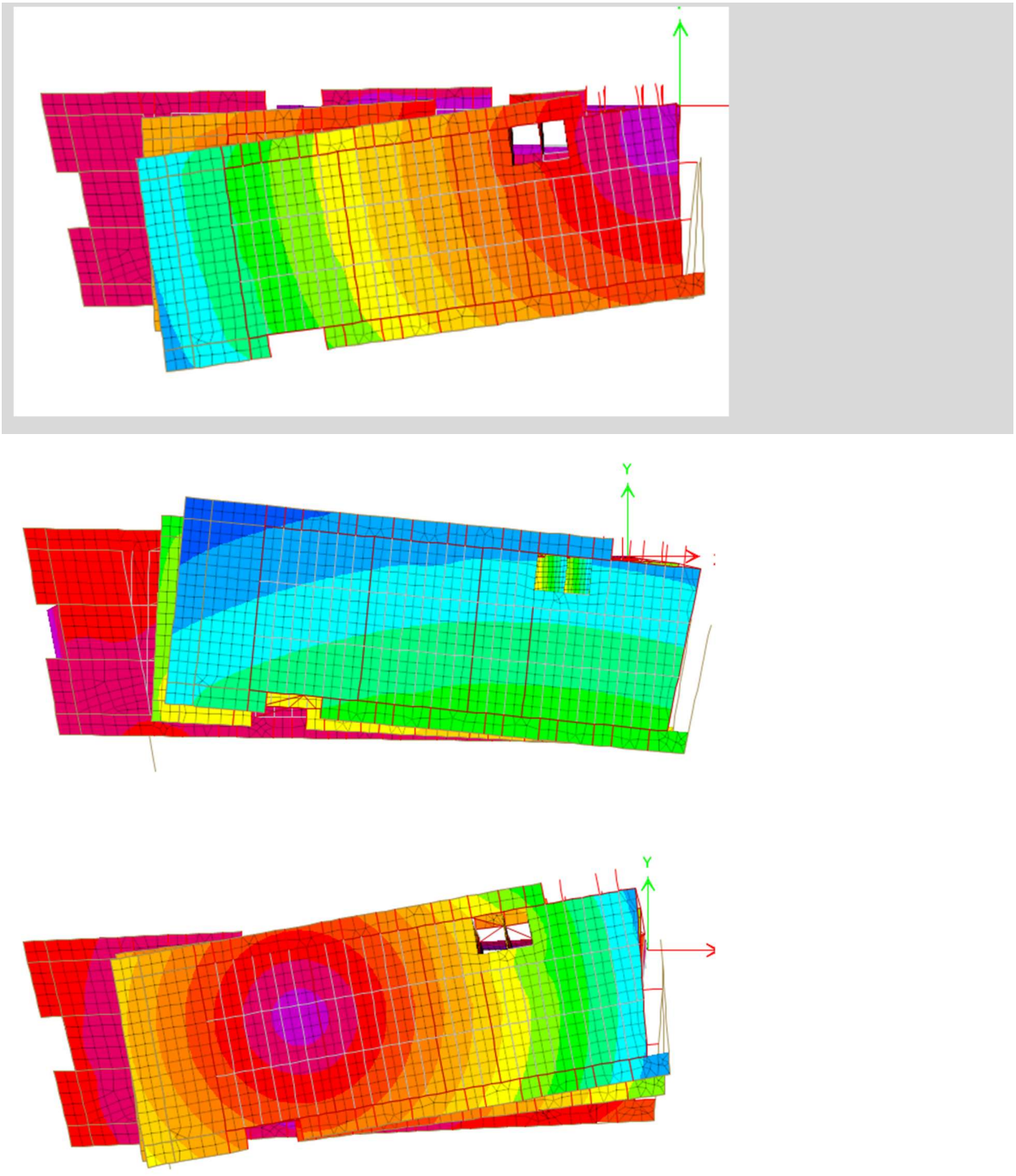


Figure 17. First three mode shapes – plan view, in the usual order.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

3.5 Inter-Storey Drift

Inter-storey drift ratios are well-controlled. Figure 18 shows a slightly torsional

- Maximum ISDR = 0.40% in Y, 0.32% in X
- Target ISDR limit = 0.40%
- Negligible in lower levels
- Further reduction in brace size would require controlling torsional deformation

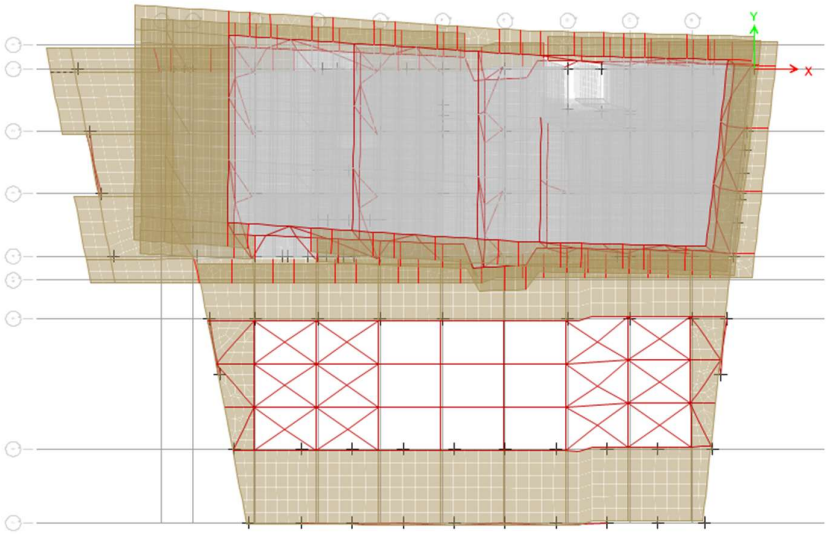


Figure 18. Deformation from response spectrum analysis.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

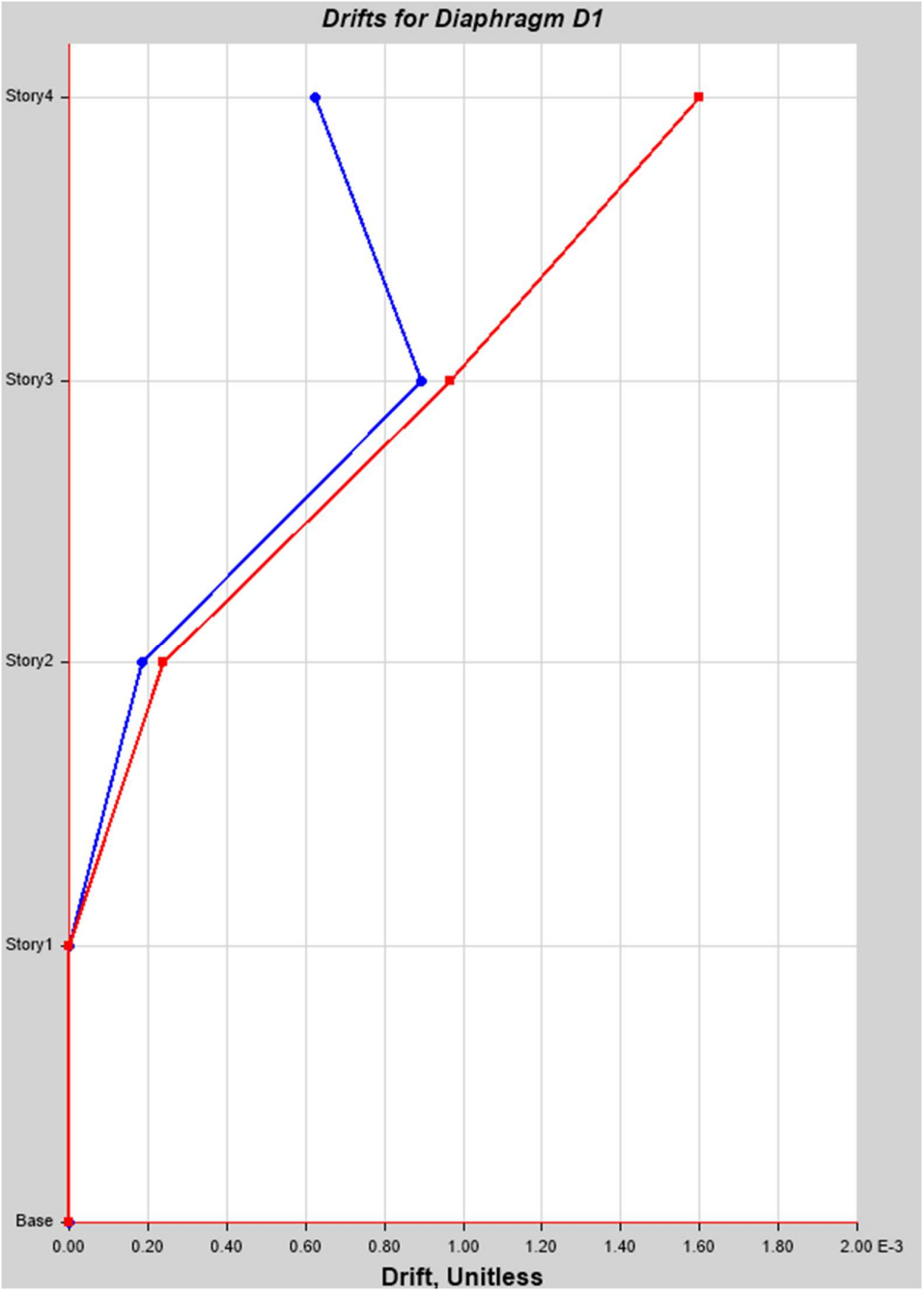


Figure 19. Inter-storey drift plot.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

3.6 Storey Forces and Base Shear

Figure 20 shows the distribution of horizontal forces to the floors using the ELF method. The short period of the first fundamental modes (ca. 0.2s each in X and Y) means the response is on the plateau of the spectrum. Thus, the base shear calculated using the ELF method is , where

$$V_{base} = W * Sd(0.2s) * \lambda,$$

where

W = weight contributing to seismic inertia forces, 32331kN

Sd(0.2s) = 0.3*1.2*2.5/2.4 = 0.375

λ = 0.85, reduction factor to account for the 1st mode modal mass

gives

Vbase = 32331kN * 0.375 * 0.85 = 10306kN

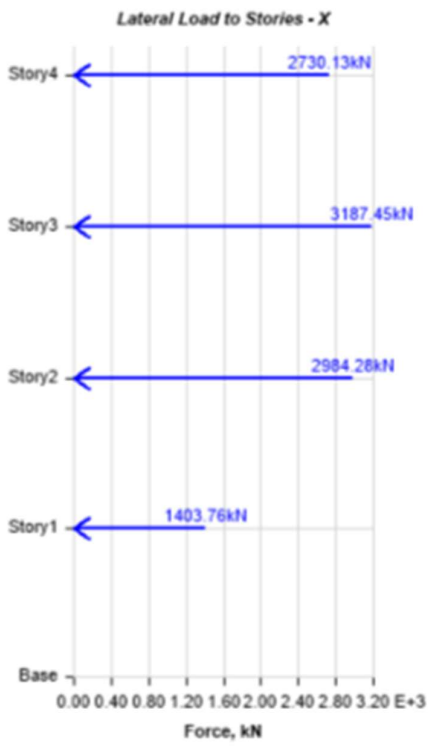


Figure 20. Forces to storeys and base shear.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

3.6.1.1 Storey Shears

Table 6 below shows the storey shears, torsional moments and bending moments calculated using both the ELF and RSA methods. Note that the RSA method reports an envelope of results, which is why the torsional and overturning moments are reported only once.

The torsional moment can be used to estimate the eccentricity in both X and Y directions as the ratio of T to the base shear Vbase. It is clear that the eccentricity in the X direction (affecting loading in the Y direction) is much greater than the eccentricity in the Y direction. Note that this includes the basic eccentricity and the accidental eccentricity. As expected, the eccentricity is greater in the RSA method due to the effects of higher modes.

The ratio between OTM and Vbase can be used to estimate the height of the resultant of seismic forces. It varies between about 8m and 10m for excitation in X and Y directions, respectively, and is again affected by higher mode response (H_eff is greater for RSA than ELF).

The RSA base shear is much less than ELF, as can be seen also in Figure 21 graphically. This is due to the presence of higher modes which are better captured in the RSA method. ELF is therefore a conservative estimate of the response, with RSA being more realistic. The modal mass of higher modes is significant (over 50%), and the higher modes have periods under 0.15s which means they are in the descending part of the spectrum. See this illustrated in Figure 22.

As a result, for design we either scale RSA results up or, more simply, we design for the ELF results. The latter is chosen in this study. Softening and buckling lead to period elongation which is not captured by non-linear modal analysis, making ELF more conservative.

Table 6. Storey shears and moments consistent with design internal forces

Story	Method	Dir	Vbase	T	OTM	Eccentricity	H_eff
Story4	EQ	X	2732	21666	9836	7.9	
Story3	EQ	X	5923	47068	31159	7.9	
Story2	EQ	X	8899	70081	47123	7.9	
Story1	EQ	X	10263	85238	78863	8.3	7.7
Story4	EQ	Y	2732	55415	9836	20.3	
Story3	EQ	Y	5922	120755	31155	20.4	
Story2	EQ	Y	8783	185114	60480	21.1	
Story1	EQ	Y	10127	211658	95253	20.9	9.4
Story4	RSA	X	2540		9143		
Story3	RSA	X	5284		27934		
Story2	RSA	X	6364		37991		
Story1	RSA	X	7339		59143		8.1
Story4	RSA	Y	2202	67966	7929	30.9	
Story3	RSA	Y	3866	117664	21528	30.4	
Story2	RSA	Y	4805	141494	36306	29.4	
Story1	RSA	Y	5479	157371	53969	28.7	9.8

Copyright © 1976 - 2026 Buro Happold. All rights reserved

	ELF		RSA	
	X	Y	X	Y
Story4	2732	2732	2540	2202
Story3	5923	5922	5284	3866
Story2	8899	8783	6364	4805
Story1	10263	10127	7339	5479
			72%	54%

Figure 21. Design storey shear comparison.

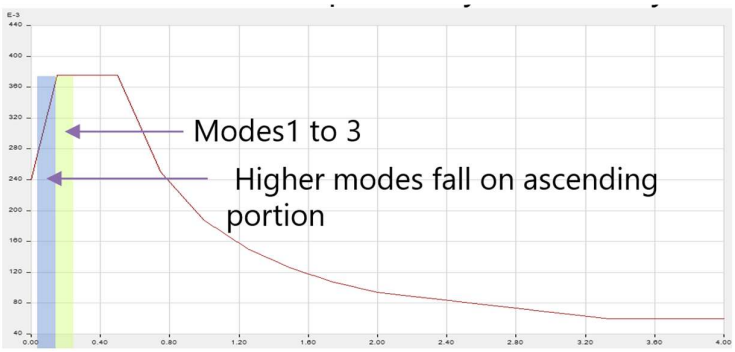


Figure 22. Response of higher modes.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

4 Element Design at ULS

4.1 Second-Order Effects

After the initial modifications to slender elements, buckling analysis (eigenvalue) show all local modes have load factors above 15 with the exception being the column shown in Figure 23, with a load factor of 12. Some X braces have lower load factors, but these are designed to buckle. All the global buckling modes have load factors αcr in excess of 100.

Manual checks confirm the above and the analysis was carried out with P-Delta activated, so no additional factors are applied due to the high load factors.

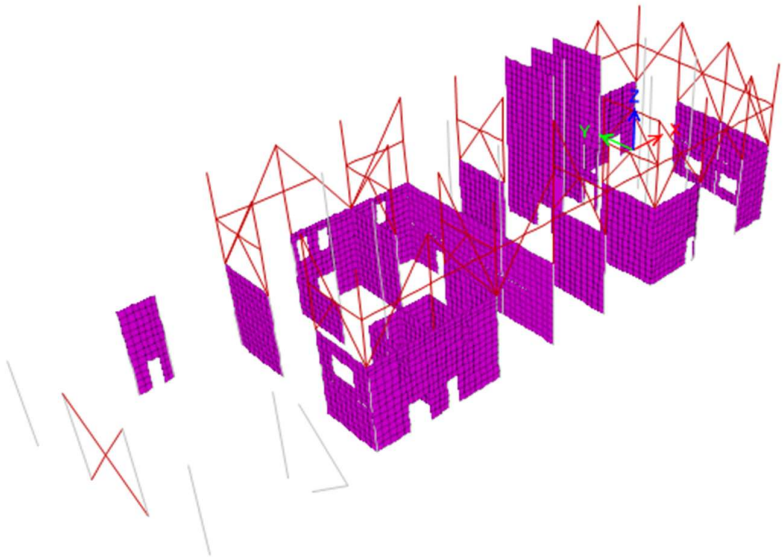


Figure 23. Example buckling mode showing slender columns buckling out of plane prior to sections being enlarged.

4.2 Shear Walls

The vertical reinforcement (resisting in-plane and out-of-plane flexure) requirement for the walls generally around 1.0% but some areas are highly stressed and highly reinforced, especially after new openings were created. The horizontal reinforcement (resisting shear) is generally around 0.5%. The quantification of reinforcement is not part of the scope of this stage, so this is only used to confirm that the walls are capable of being designed and reinforced to comply with EC8 and EC2 rules, and that the shear stresses don't exceed the maximum allowed and therefore the thickness is adequate.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

Edge elements are required for confinement & ductility, as shown in the table.

The code imposes slenderness limits for ductility. The boundary elements around openings have been checked for slenderness. Walls supporting large transfer beam remain critical but they now show workable results after architectural modifications that saw their cross-sections enlarged.

The shear walls considered part of the LFRS are shown in Figure 24 and their numbering (index to the results table) in Figure 25. ETABS does not provide a clear graphic output of the design results, so these are provided instead in table format, see Table 7. Pier design results. A complete set of pier and spandrel forces is given in Appendix E.1 and E.2.

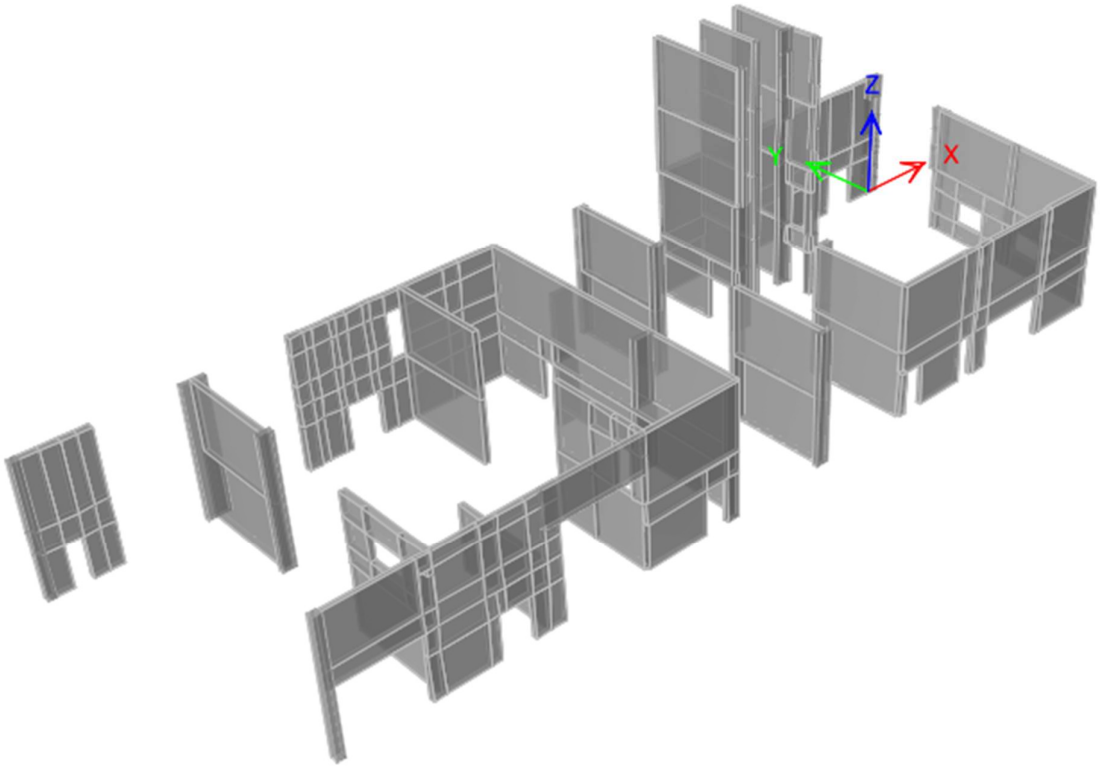


Figure 24. Shear walls in the model

Copyright © 1976 - 2026 Buro Happold. All rights reserved

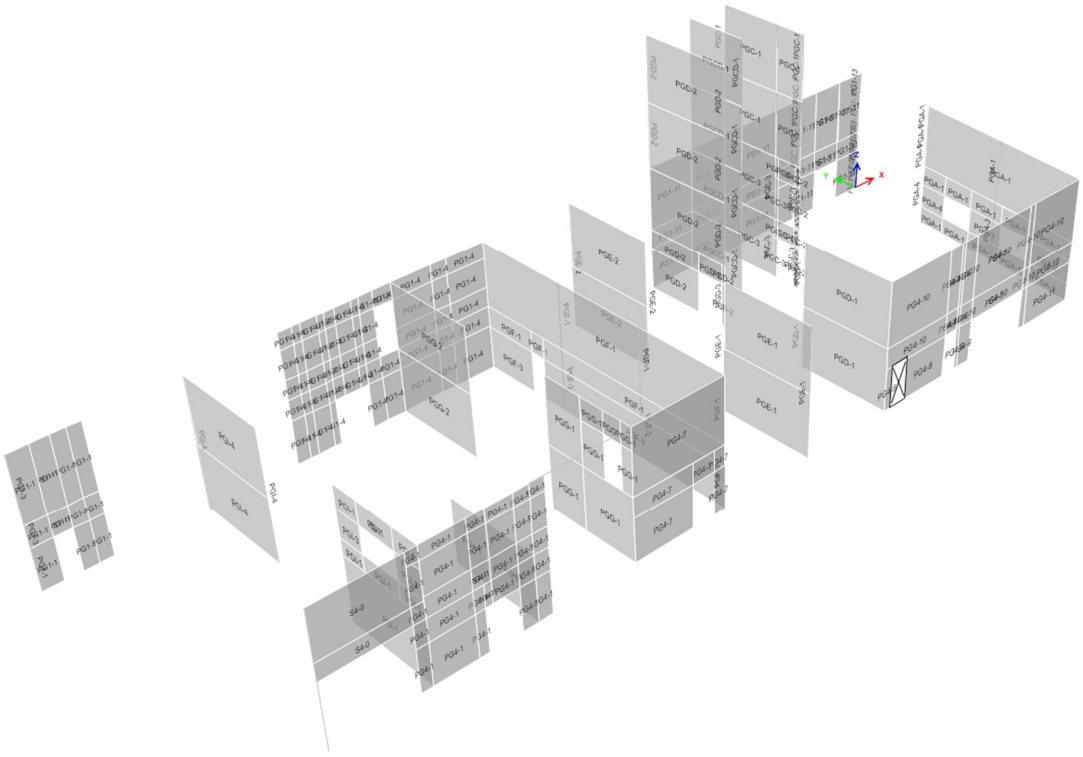


Figure 25. Pier numbering for design

Copyright © 1976 - 2026 Buro Happold. All rights reserved

Table 7. Pier design results

Story	Pier Label	Edge Rebar	End Rebar	Rebar Spacing	Required Reinf. Percentage	Current Reinf. Percentage	D/C Ratio	Pier Leg	Thickness	Shear Rebar	Normalized Compressive Stress	Normalized Compressive Limit	C Depth	Boundary Zone Left	Boundary Zone Right	Pass?	Note
				mm	%	%			mm	mm²/m	MPa	MPa	mm	mm	mm		
Story2	PG1-1	25	25	60			0.897	Top Leg 1	350	4415	0	0				Pass	
Story2	PG1-1	25	25	60			0.781	Bottom Leg 1	350	4415	0.04083	0.2				Pass	
Story1	PG1-1	25	25	150			0.923	Top Leg 1	350	4050	0	0				Pass	
Story1	PG1-1	25	25	150			0.694	Bottom Leg 1	350	4825	0	0			525	Pass	
Story1	PG1-1	25	25	150			0.694	Bottom Leg 2	350	5669	0.1	0.2				Pass	
Story2	PG1-3	32	25	40			1.045	Top Leg 1	400	0	0	0				Fail/check	1
Story2	PG1-3	32	25	40			0.677	Bottom Leg 1	400	0	0.04199	0.2				Pass	
Story1	PG1-3	25	25	150			0.329	Top Leg 1	400	3682	0.03814	0.2				Pass	
Story1	PG1-3	25	25	150			0.351	Bottom Leg 1	400	3682	0	0				Pass	
Story2	PG1-4	20	20	140			0.376	Top Leg 1	256.1	9121	0.03447	0.2				Pass	
Story2	PG1-4	20	20	140			0.567	Bottom Leg 1	250	12762	0	0				Pass	
Story2	PG1-4	20	20	140			0.567	Bottom Leg 2	263.9	12166	0.1	0.2				Pass	
Story1	PG1-4	20	20	140			0.747	Top Leg 1	256.1	11183	0.1	0.2				Pass	
Story1	PG1-4	20	20	140			0.988	Bottom Leg 1	250	15113	0.1	0.2				Pass	
Story1	PG1-4	20	20	140			0.988	Bottom Leg 2	279.2	8807	0.3	0.2	358.1	418.8		Pass	
Story2	PG1-11	20	20	140			0.455	Top Leg 1	250	12302	0.009537	0.2				Pass	
Story2	PG1-11	20	20	140			0.637	Bottom Leg 1	250	11396	0.1	0.2				Pass	
Story1	PG1-11	20	20	140			0.796	Top Leg 1	250	12535	0.1	0.2				Pass	
Story1	PG1-11	20	20	140			0.92	Bottom Leg 1	250	11684	0	0			375	Pass	
Story1	PG1-11	20	20	140			0.92	Bottom Leg 2	250	14829	0.1	0.2				Pass	
Story1	PG1-11	20	20	140			0.92	Bottom Leg 3	250	1232	0	0.2				Pass	
Story1	PG1-12	32	32	50			0.362	Top Leg 1	900	24338	0	0				Pass	
Story1	PG1-12	32	32	50			0.536	Bottom Leg 1	900	24338	0	0				Pass	
Story2	PG1-13	32	32	50			0.447	Top Leg 1	900	24338	0.02088	0.2				Pass	
Story2	PG1-13	32	32	50			0.356	Bottom Leg 1	900	24338	0	0				Pass	
Story1	PG1-13	32	32	50			0.324	Top Leg 1	900	24338	0	0				Pass	
Story1	PG1-13	32	32	50			0.332	Bottom Leg 1	900	24338	0	0				Pass	
Story2	PG4-1	20	20	100			0.463	Top Leg 1	250	10009	0	0				Pass	
Story2	PG4-1	20	20	100			0.298	Bottom Leg 1	250	11447	0.03668	0.2				Pass	
Story1	PG4-1	20	20	100			0.344	Top Leg 1	250	9172	0.02647	0.2				Pass	
Story1	PG4-1	20	20	100			0.78	Bottom Leg 1	250	14331	0.2	0.2	337.3	513.8	513.8	Pass	
Story1	PG4-1	20	20	100			0.78	Bottom Leg 2	250	8797	0.2	0.2	124.7	375	375	Pass	
Story2	PG4-7	20	20	90			0.713	Top Leg 1	250	9759	0	0				Pass	
Story2	PG4-7	20	20	90			0.6	Bottom Leg 1	250	9684	0.007564	0.2				Pass	

Copyright © 1976 - 2026 Buro Happold. All rights reserved

Story1	PG4-7	20	20	90			0.558	Top Leg 1	250	10348	0	0				**Pass**	
Story1	PG4-7	20	20	90			0.927	Bottom Leg 1	250	8992	0.3	0.2	80.6	375	375	**Pass**	
Story1	PG4-7	20	20	90			0.927	Bottom Leg 2	250	14325	0.2	0.2	270.4	517.5		**Pass**	
Story1	PG4-8	25	25	100			0.649	Top Leg 1	250	6511	0	0			375	**Pass**	
Story1	PG4-8	25	25	100			0.649	Top Leg 2	250	15415	0	0				**Pass**	
Story1	PG4-8	25	25	100			0.735	Bottom Leg 1	250	7560	0.2	0.2	57.4	375	375	**Pass**	
Story1	PG4-8	25	25	100			0.735	Bottom Leg 2	250	15415	0.2	0.2	395.1	420	420	**Pass**	
Story1	PG4-9	25	25	100			0.773	Top Leg 1	250	9126	0.2	0.2	146.2	375	375	**Pass**	
Story1	PG4-9	25	25	100			0.935	Bottom Leg 1	250	9126	0.2	0.2	135.2	375	375	**Pass**	
Story2	PG4-10	20	20	100			0.199	Top Leg 1	250	10484	0	0				**Pass**	
Story2	PG4-10	20	20	100			0.351	Bottom Leg 1	250	10380	0.03365	0.2				**Pass**	
Story1	PG4-10	20	20	100			0.316	Top Leg 1	250	9007	0.02699	0.2				**Pass**	
Story1	PG4-10	20	20	100			0.347	Bottom Leg 1	250	8968	0.02907	0.2				**Pass**	
Story1	PG4-11	25	25	100			0.596	Top Leg 1	250	18688	0.1	0.2				**Pass**	
Story1	PG4-11	25	25	100			0.8	Bottom Leg 1	250	19583	0.1	0.2			535.9	**Pass**	
Story2	PGA-1	10	10	250	0.82	0.27		Top Leg 1	250	5506	0	0				**Pass**	
Story2	PGA-1	10	10	250	0.93	0.27		Bottom Leg 1	250	5422	0.02357	0.2				**Pass**	
Story1	PGA-1	10	10	250	1.25	0.26		Top Leg 1	250	6191	0.01659	0.2				**Pass**	
Story1	PGA-1	10	10	250	1.73	0.26		Bottom Leg 1	250	6296	0.03614	0.2				**Pass**	
Story1	PGA-2	10	10	250	2.45	0.28		Top Leg 1	250	12306	0.1	0.2				**Pass**	
Story1	PGA-2	10	10	250	3.39	0.28		Bottom Leg 1	250	12019	0.1	0.2				**Pass**	
Story1	PGA-3	10	10	250	0.7	0.5		Top Leg 1	250	437	0.02855	0.2				**Pass**	
Story1	PGA-3	10	10	250	0.51	0.5		Bottom Leg 1	250	316	0.01226	0.2				**Pass**	
Story1	PGA-4	12	12	250	1.54	0.6		Top Leg 1	400	1540	0.1	0.2				**Pass**	
Story1	PGA-4	12	12	250	3.63	0.6		Bottom Leg 1	400	3631	0	0				**Pass**	
Story4	PGCD-1	20	20	100			0.401	Top Leg 1	250	5596	0.002838	0.2				**Pass**	
Story4	PGCD-1	20	20	100			0.666	Bottom Leg 1	250	5992	0.01416	0.2				**Pass**	
Story3	PGCD-1	25	25	50			0.987	Top Leg 1	250	9407	0	0			551.3	**Pass**	
Story3	PGCD-1	25	25	50			0.937	Bottom Leg 1	250	9203	0.1	0.2			551.3	**Pass**	
Story2	PGCD-1	20	20	50			0.882	Top Leg 1	250	3260	0.1	0.2				**Pass**	
Story2	PGCD-1	20	20	50			0.609	Bottom Leg 1	250	3260	0.1	0.2				**Pass**	
Story1	PGCD-1	20	20	100			0.462	Top Leg 1	250	1650	0.1	0.2				**Pass**	
Story1	PGCD-1	20	20	100			0.467	Bottom Leg 1	250	1650	0.1	0.2				**Pass**	
Story2	PGI-1	10	10	250	0.83	0.22		Top Leg 1	300	4456	0	0				**Pass**	
Story2	PGI-1	10	10	250	1.15	0.22		Bottom Leg 1	300	4373	0.01273	0.2				**Pass**	
Story1	PGI-1	10	10	250	1.19	0.22		Top Leg 1	300	5182	0.02262	0.2				**Pass**	
Story1	PGI-1	10	10	250	1.75	0.22		Bottom Leg 1	300	5325	0.04686	0.2				**Pass**	

Copyright © 1976 - 2026 Buro Happold. All rights reserved

Story2	PGI-2	10	10	250	1.82	0.26		Top Leg 1	300	10405	0	0				**Pass**	
Story2	PGI-2	10	10	250	2.92	0.26		Bottom Leg 1	300	10249	0.02401	0.2				**Pass**	
Story2	PGI-3	10	10	250	1.41	0.24		Top Leg 1	300	4313	0	0				**Pass**	
Story2	PGI-3	10	10	250	1.24	0.24		Bottom Leg 1	300	4313	0.1	0.2				**Pass**	
Story2	PGI-4	12	12	250	1.76	0.33		Top Leg 1	300	4833	0	0				**Pass**	
Story2	PGI-4	12	12	250	3.13	0.44		Bottom Leg 1	94.4	740	0.1	0.2				**Pass**	
Story1	PGI-4	10	10	250	1.7	0.22		Top Leg 1	300	4735	0	0				**Pass**	
Story1	PGI-4	10	10	250	1.71	0.22		Bottom Leg 1	300	4567	0	0				**Pass**	
Story1	PGH-1	12	12	250	0.42	0.34		Top Leg 1	300	5185	0.03096	0.2				**Pass**	
Story1	PGH-1	12	12	250	0.68	0.34		Bottom Leg 1	300	5185	0.1	0.2				**Pass**	
Story1	PGH-2	12	12	250	0.25	0.57		Top Leg 1	400	400	0.01979	0.2				**Pass**	
Story1	PGH-2	12	12	250	0.25	0.57		Bottom Leg 1	400	400	0.01312	0.2				**Pass**	
Story2	PGG-1	10	10	250	1.29	0.22		Top Leg 1	300	4824	0	0				**Pass**	
Story2	PGG-1	10	10	250	1.67	0.24		Bottom Leg 1	300	5764	0.001405	0.2				**Pass**	
Story2	PGG-1	10	10	250	1.67	0.24		Bottom Leg 2	300	6659	0	0				**Pass**	
Story1	PGG-1	10	10	250	1.38	0.22		Top Leg 1	300	5833	0	0				**Pass**	
Story1	PGG-1	10	10	250	2.2	0.22		Bottom Leg 1	300	5843	0.1	0.2				**Pass**	
Story2	PGG-2	10	10	250	1.09	0.22		Top Leg 1	300	5430	0	0				**Pass**	
Story2	PGG-2	10	10	250	1.29	0.22		Bottom Leg 1	300	5395	0.02881	0.2				**Pass**	
Story1	PGG-2	10	10	250	1.27	0.22		Top Leg 1	300	2708	0.0252	0.2				**Pass**	
Story1	PGG-2	10	10	250	1.34	0.22		Bottom Leg 1	300	2207	0.04794	0.2				**Pass**	
Story2	PGF-1	20	20	90			0.097	Top Leg 1	250	5802	0	0				**Pass**	
Story2	PGF-1	20	20	90			0.236	Bottom Leg 1	250	5685	0.03266	0.2				**Pass**	
Story1	PGF-1	10	10	250	0.81	0.26		Top Leg 1	250	3901	0.032	0.2				**Pass**	
Story1	PGF-1	10	10	250	0.95	0.26		Bottom Leg 1	250	3923	0.03463	0.2				**Pass**	
Story1	PGF-2	20	20	90			0.264	Top Leg 1	85.6	611	0.03062	0.2				**Pass**	
Story1	PGF-2	20	20	90			0.443	Bottom Leg 1	250	5013	0.1	0.2				**Pass**	
Story1	PGF-3	10	10	250	2.71	0.28		Top Leg 1	250	6018	0	0				**Pass**	
Story1	PGF-3	10	10	250	3.54	0.28		Bottom Leg 1	250	6100	0.1	0.2				**Pass**	
Story2	PGE-1	20	20	100			0.114	Top Leg 1	300	1667	0.00234	0.2				**Pass**	
Story2	PGE-1	20	20	100			0.373	Bottom Leg 1	71.3	553	0	0				**Pass**	
Story1	PGE-1	10	10	250	0.51	0.22		Top Leg 1	300	1996	0.02659	0.2				**Pass**	
Story1	PGE-1	10	10	250	0.66	0.22		Bottom Leg 1	300	496	0.04531	0.2				**Pass**	
Story2	PGE-2	12	12	250	0.46	0.33		Top Leg 1	300	2016	0.001734	0.2				**Pass**	
Story2	PGE-2	12	12	250	1.25	0.47		Bottom Leg 1	71.3	223	0.1	0.2				**Pass**	
Story1	PGE-2	10	10	250	0.78	0.22		Top Leg 1	300	586	0.02668	0.2				**Pass**	
Story1	PGE-2	10	10	250	1.08	0.22		Bottom Leg 1	300	810	0	0				**Pass**	

Copyright © 1976 - 2026 Buro Happold. All rights reserved

Story2	PGD-1	25	25	120			0.156	Top Leg 1	250	6709	0.1	0.2				**Pass**	
Story2	PGD-1	25	25	120			0.328	Bottom Leg 1	250	6636	0.03844	0.2				**Pass**	
Story1	PGD-1	10	10	250	1.42	0.26		Top Leg 1	250	4997	0.1	0.2				**Pass**	
Story1	PGD-1	10	10	250	1.95	0.26		Bottom Leg 1	250	4960	0.1	0.2				**Pass**	
Story4	PGD-2	20	20	100			0.443	Top Leg 1	250	4014	0.01214	0.2				**Pass**	
Story4	PGD-2	20	20	100			0.595	Bottom Leg 1	250	4153	0.02913	0.2				**Pass**	
Story3	PGD-2	25	25	60			0.619	Top Leg 1	250	7682	0.0377	0.2				**Pass**	
Story3	PGD-2	25	25	60			0.919	Bottom Leg 1	250	7517	0.2	0.2	1065.1	810	810	**Pass**	
Story2	PGD-2	20	20	100			0.99	Top Leg 1	250	7164	0.1	0.2				**Pass**	
Story2	PGD-2	20	20	100			0.986	Bottom Leg 1	250	7161	0.1	0.2				**Pass**	
Story1	PGD-2	25	25	80			0.37	Top Leg 1	250	3182	0.04456	0.2				**Pass**	
Story1	PGD-2	25	25	80			0.661	Bottom Leg 1	250	3383	0	0			525	**Pass**	
Story1	PGD-2	25	25	80			0.661	Bottom Leg 2	250	3383	0.1	0.2				**Pass**	
Story1	PGD-3	25	25	80			0.968	Top Leg 1	300	3617	0	0			450	**Pass**	
Story1	PGD-3	25	25	80			0.884	Bottom Leg 1	300	3617	0	0			450	**Pass**	
Story4	PGC-1	10	10	250	0.75	0.3		Top Leg 1	250	3654	0.02568	0.2				**Pass**	
Story4	PGC-1	10	10	250	1.2	0.3		Bottom Leg 1	250	3675	0.02473	0.2				**Pass**	
Story3	PGC-1	10	10	250	4.06	0.3		Top Leg 1	250	0	0.1	0.2				**Fail/check**	**2**
Story3	PGC-1	10	10	250	5.43	0.3		Bottom Leg 1	250	0	0.1	0.2				**Fail/check**	**2**
Story2	PGC-1	25	25	120			0.902	Top Leg 1	250	6825	0.1	0.2				**Pass**	
Story2	PGC-1	25	25	120			0.465	Bottom Leg 1	250	5458	0	0				**Pass**	
Story2	PGC-2	25	25	80			0.692	Top Leg 1	250	3131	0.1	0.2				**Pass**	
Story2	PGC-2	25	25	80			0.887	Bottom Leg 1	250	3131	0.02944	0.2				**Pass**	
Story1	PGC-2	25	25	80			0.836	Top Leg 1	250	3131	0.1	0.2				**Pass**	
Story1	PGC-2	25	25	80			0.882	Bottom Leg 1	250	3131	0.1	0.2				**Pass**	
Story2	PGC-3	20	20	70			0.938	Top Leg 1	250	2382	0.1	0.2				**Pass**	
Story2	PGC-3	20	20	70			0.627	Bottom Leg 1	250	3475	0.02963	0.2				**Pass**	
Story1	PGC-3	20	20	140			0.948	Top Leg 1	250	1278	0.0198	0.2				**Pass**	
Story1	PGC-3	20	20	140			0.953	Bottom Leg 1	250	2326	0.1	0.2				**Pass**	

Copyright © 1976 - 2026 Buro Happold. All rights reserved

4.3 Concentrically-Braced Steel Frames

The upper storeys of the building are a steel concentrically-braced system. The diagonal braces are designed to yield, and all other elements (beams, columns, connections) are designed to remain essentially elastic while the braces yield. The utilisation factor in Figure 26 represents, for the braces, the ratio between the axial force required from analysis and the available strength; for the beams and columns, the ratio of the force demand at over-strength (i.e., with the braces yielding) and the available strength.

This process of designing all non-yielding elements to be stronger than the yielding elements is termed 'capacity design' or 'capacity protection'. A complete set of internal forces is given in Appendix E.3 to E.5.

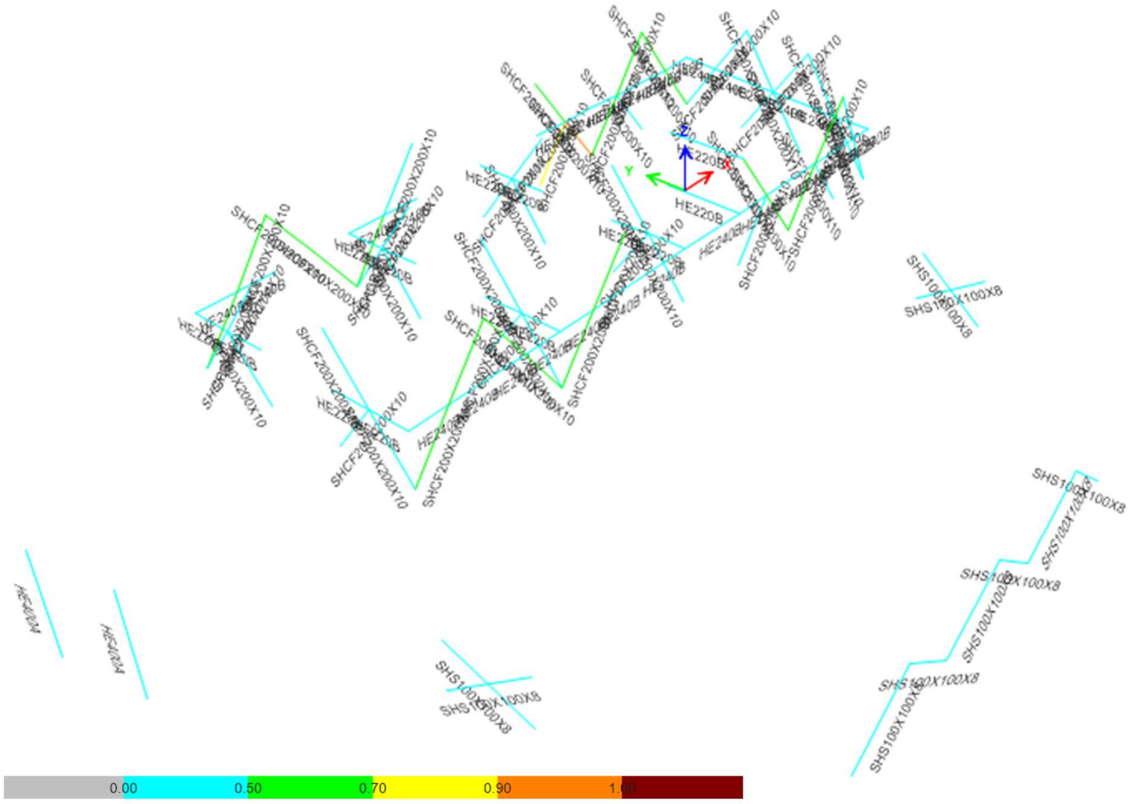


Figure 26. Design results for steel elements. The colours represent the utilisation factor as shown in the key at the bottom of the figure.

4.4 Manual Checks

Appendix B shows manual checks of various elements to complement the ETABS design checks. These include:

- Concrete columns (edge elements) supporting concentrated reactions from braces
- Primary steel columns supporting braces, checked to carry the braces ultimate capacity
- Primary steel beams when subjected to unbalanced loads from intersecting braces

Copyright © 1976 - 2026 Buro Happold. All rights reserved

- Brace anchors into concrete (steel-to-concrete connection)
- Shear studs to transfer horizontal forces between steel frame and concrete walls
- Gusset plate design to resist ultimate strength of braces
- Steel column splices
- Steel beam splices

4.5 Reactions to Foundations

The reactions on the foundations have been provided in the Excel file BASE NODE REACTIONS GEO COMBINATIONS. These are based on a non-linear analysis where the pile supports do not take tension forces. The spreadsheet is interactive so the foundations designer can adjust the contribution of the passive thrust. Pile and ground beam forces are given in Appendix C.

The piles should be designed with spiral reinforcement providing confinement and in a sufficient quantity to ensure the shear strength allows the formation of plastic hinges in the piles. This is to ensure survivability beyond the ULS level in case a) forces are exceeded or b) liquefaction occurs.

The design should consider:

- The variable depth to the bedrock means the piles will have different effective shear spans
- Allowance should be made for different soil conditions/response, so that the depth of the bottom plastic hinge is considered within a range
- The top of the pile should be allowed to develop a plastic hinge, therefore the longitudinal reinforcement should be properly developed into the pile cap/beam
- Provide spiral reinforcement for both confinement and shear strength
- Welding of reinforcement should be avoided as it can affect ductility in unknown ways
- Inclined piles must be avoided

4.6 Indicative Details

The following indicative construction details are intended to inform spatial coordination. The amount of reinforcement and other details must be confirmed in the next design stage.

The steel-to-steel and steel-to-concrete connections should be designed by the contractor following the principles of this report, Eurocode 8, Eurocode 3 and Eurocode 2. The details provided here are for indicative purposes only.

These details have been provided to scale in PDF files separate to this report.

Copyright © 1976 - 2026 Buro Happold. All rights reserved

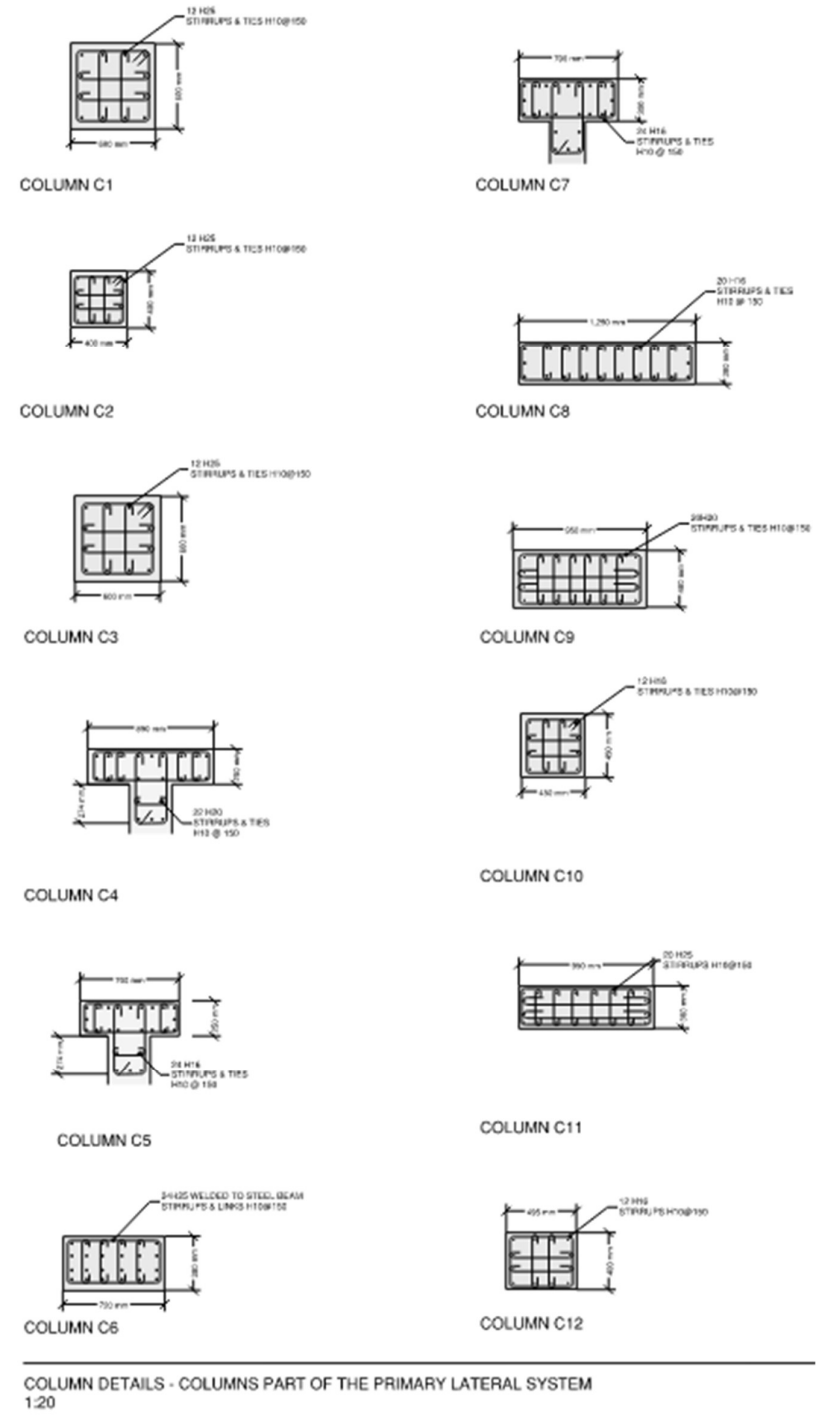
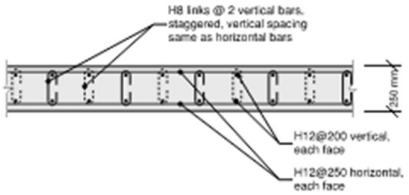
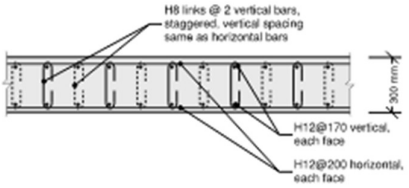


Figure 27. RC Column Indicative details

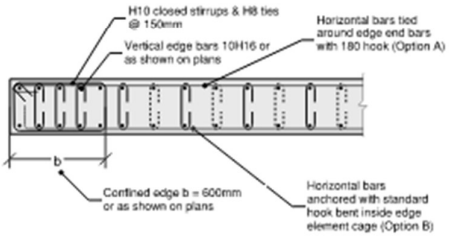
Copyright © 1976 - 2026 Buro Happold. All rights reserved



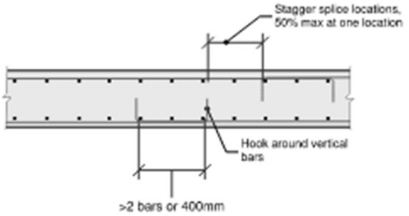
TYPICAL WALL SECTION, 250MM



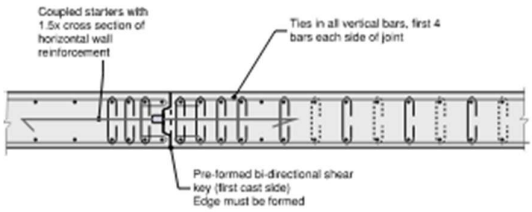
TYPICAL WALL SECTION, 300MM



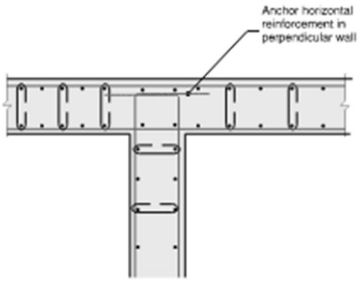
TYPICAL WALL END AND EDGE AROUND OPENINGS



TYPICAL WALL HORIZONTAL BAR SPLICES



TYPICAL VERTICAL CONSTRUCTION JOINT IN WALLS



TYPICAL WALL INTERSECTION DETAIL

Figure 28. RC wall indicative details

Copyright © 1976 - 2026 Buro Happold. All rights reserved

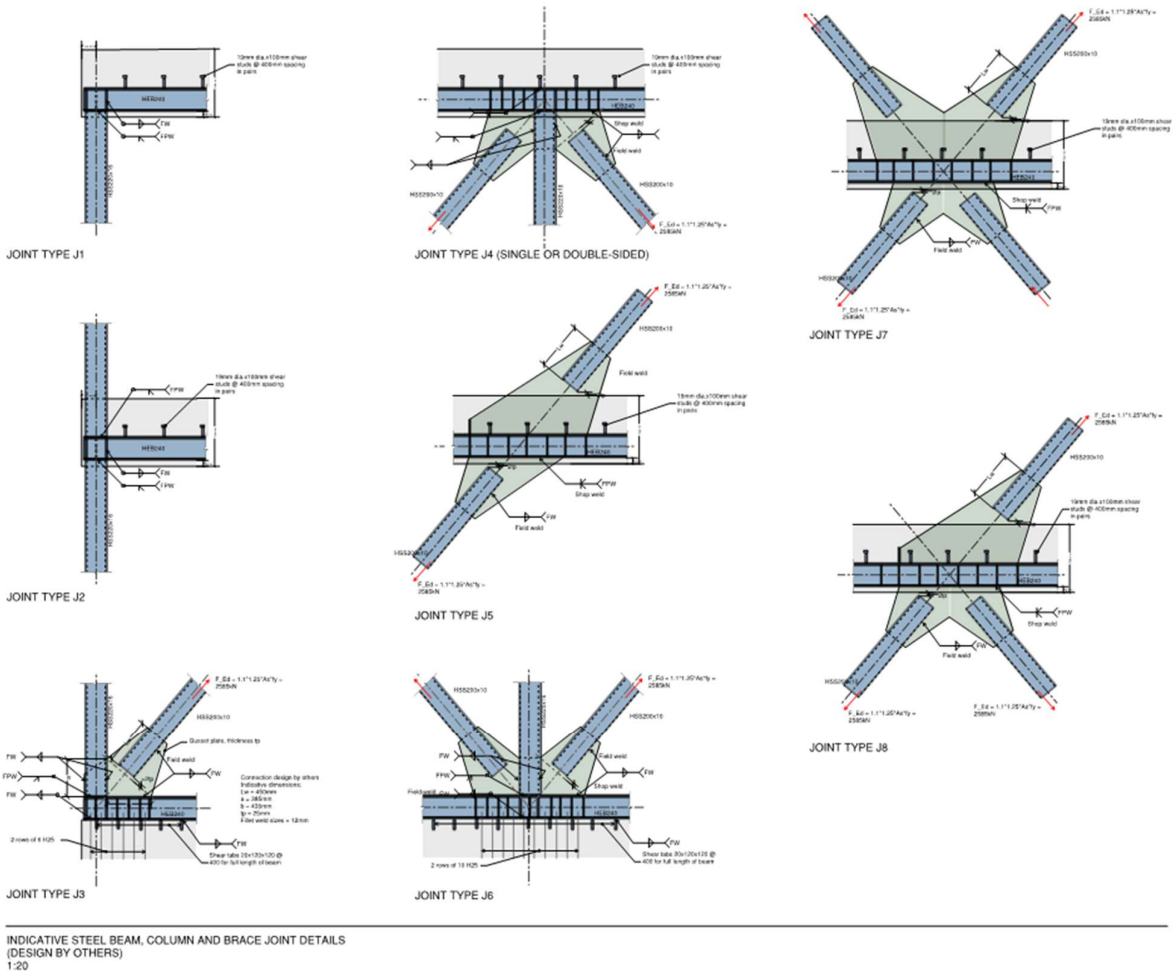


Figure 29. Indicative steel connection details

Copyright © 1976 - 2026 Buro Happold. All rights reserved

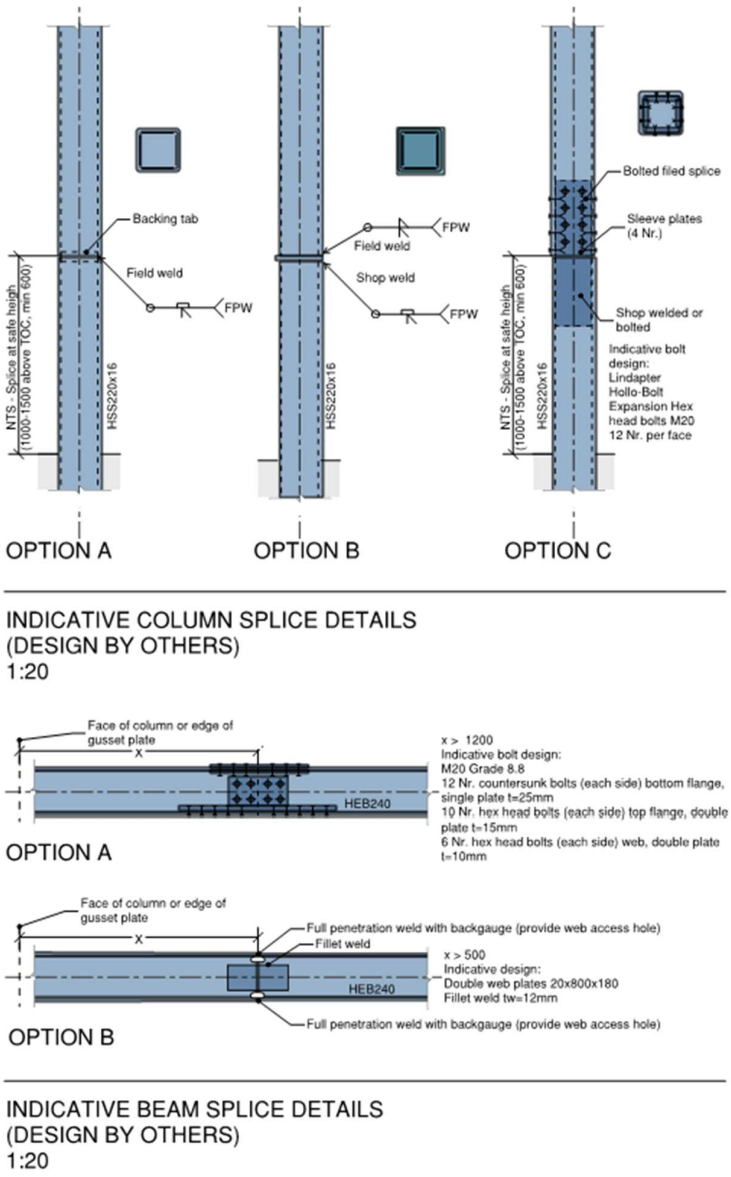
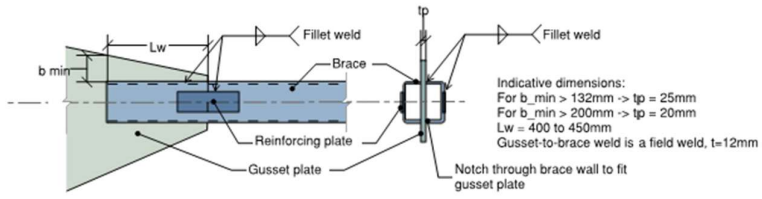
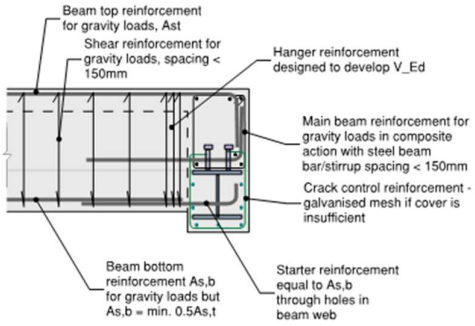


Figure 30. Steel splices indicative details

Copyright © 1976 - 2026 Buro Happold. All rights reserved

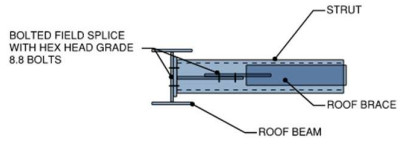


INDICATIVE GUSSET PLATE TO BRACE CONNECTION DETAIL
(DESIGN BY OTHERS)
1:20

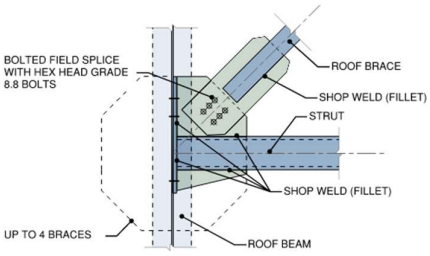


INDICATIVE CONCRETE FLOOR STRUCTURE (GRAVITY ONLY)
TO STEEL BEAMS
(DESIGN BY OTHERS)
1:20

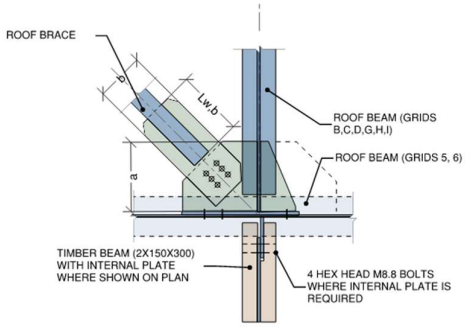
Figure 31. Steel brace and embedded beam indicative details



JOINT TYPE J9 - SECTION



JOINT TYPE J9 - PLAN
NDICATIVE DESIGN:
BRACE TO PLATE WELD: 6mm, Lw,b=180mm
BRACE CONNECTION PLATE: b=150x t=12, 6x D16 bolts
GUSSET PLATE a=400, t=12, welds = 8mm (2 sides)



JOINT TYPE J10 - PLAN
INDICATIVE DESIGN:
- BRACE TO PLATE WELD: 6mm, Lw,b=180mm
- BRACE CONNECTION PLATE: b=150x t=12, 6x D16 bolts
- GUSSET PLATE a=400, t=12, welds = 8mm (2 sides)

Figure 32. Indicative connection details for low roof

Copyright © 1976 - 2026 Buro Happold. All rights reserved

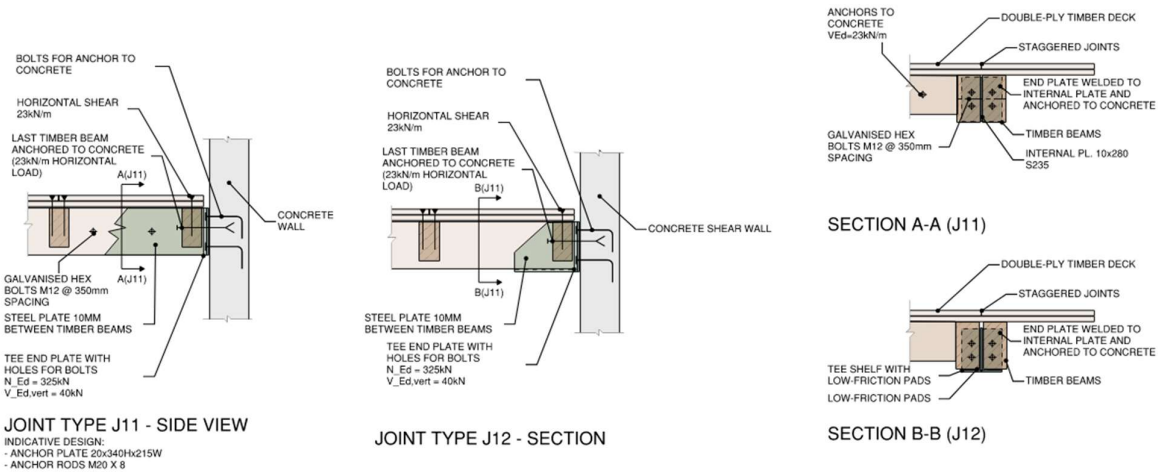


Figure 33. Indicative connections between low roof and main building

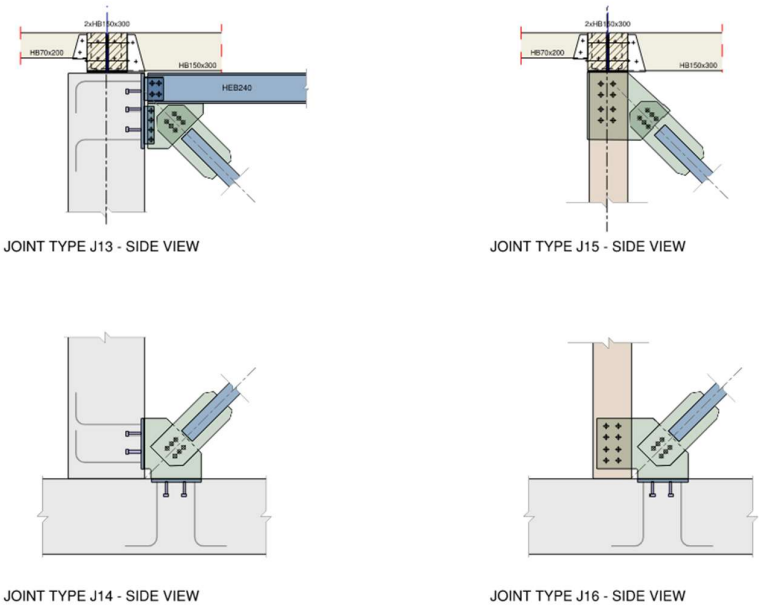
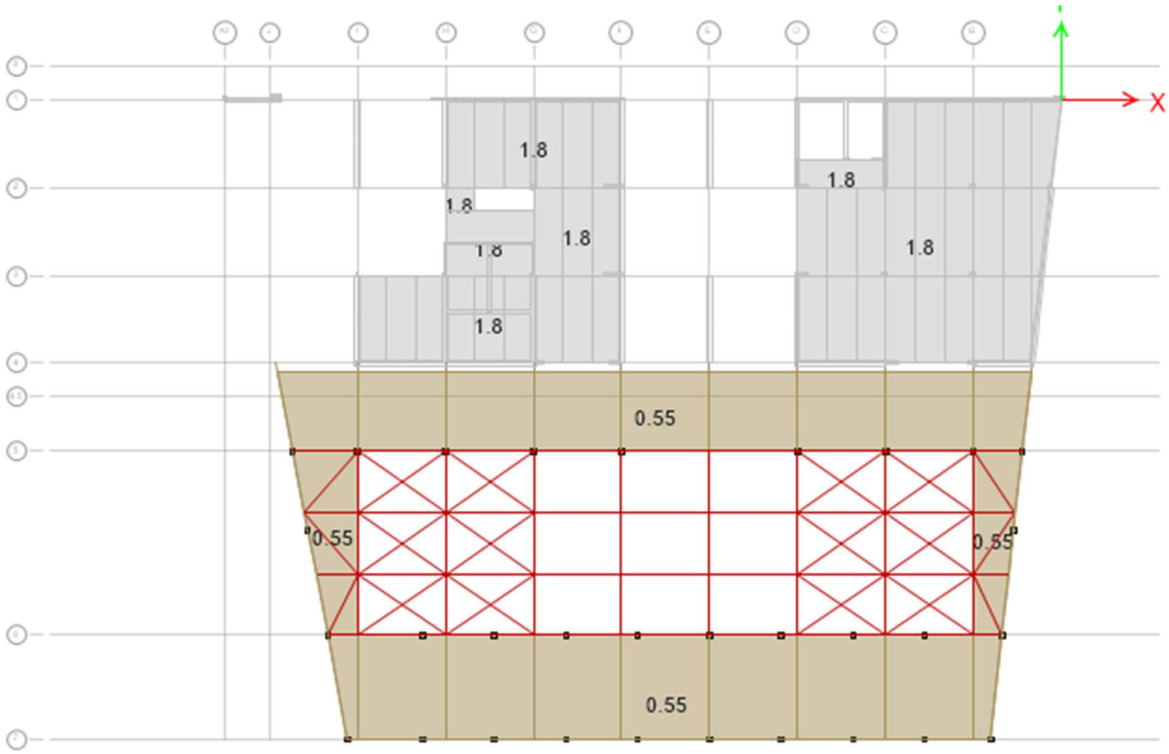


Figure 34. Brace connections, low roof

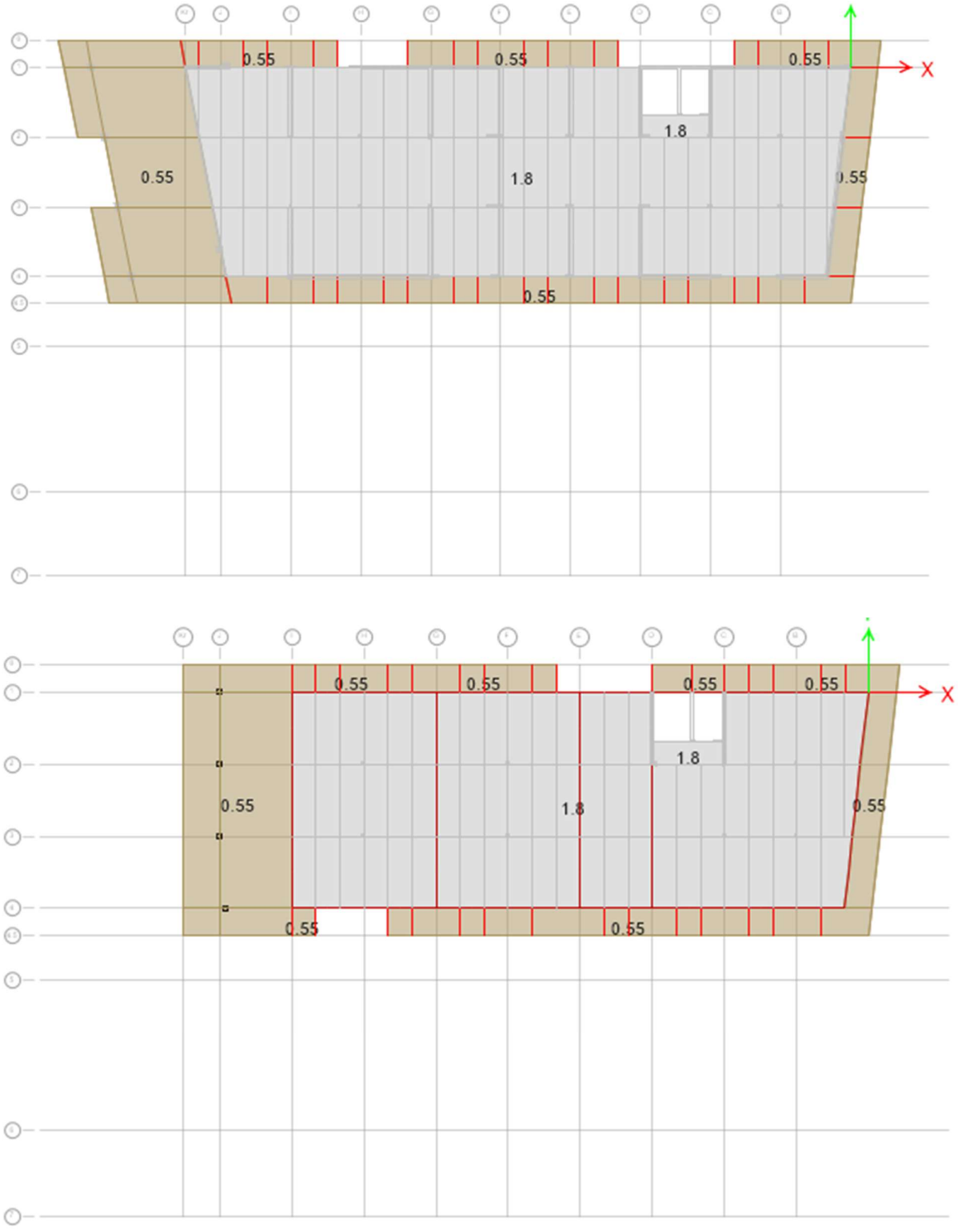
Copyright © 1976 - 2026 Buro Happold. All rights reserved

Appendix A Model Details

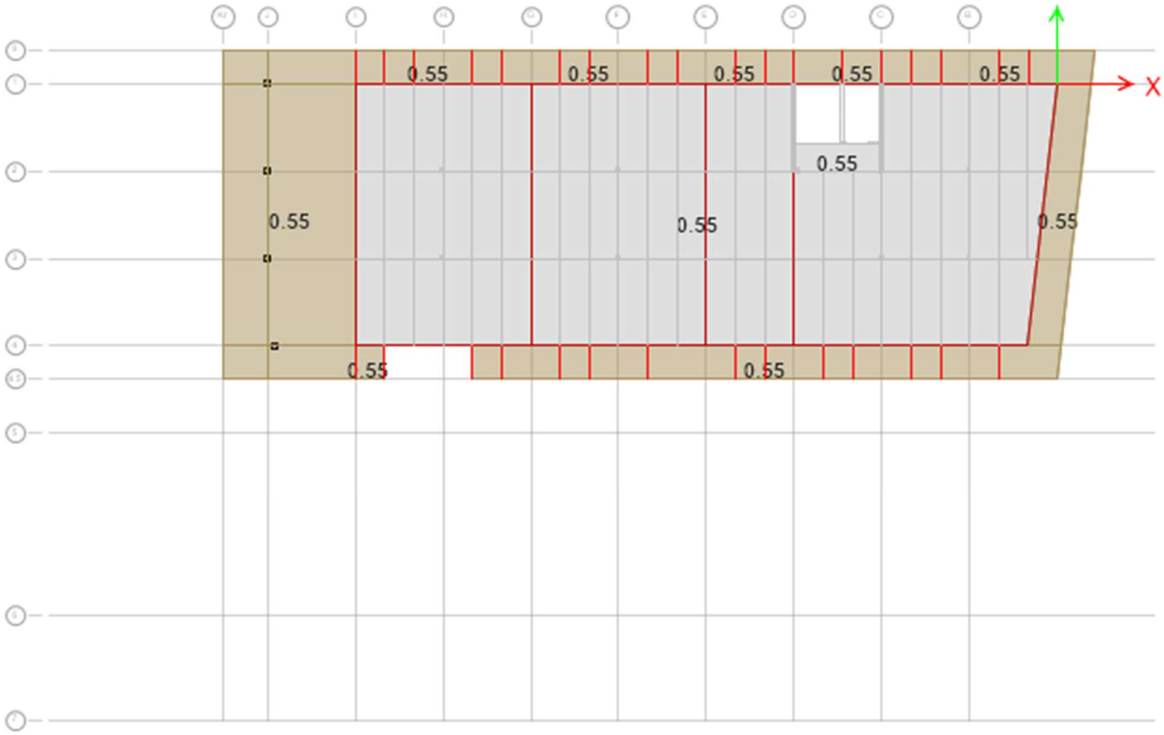
A.1 Loading Plans: SDL



Copyright © 1976 - 2026 Buro Happold. All rights reserved

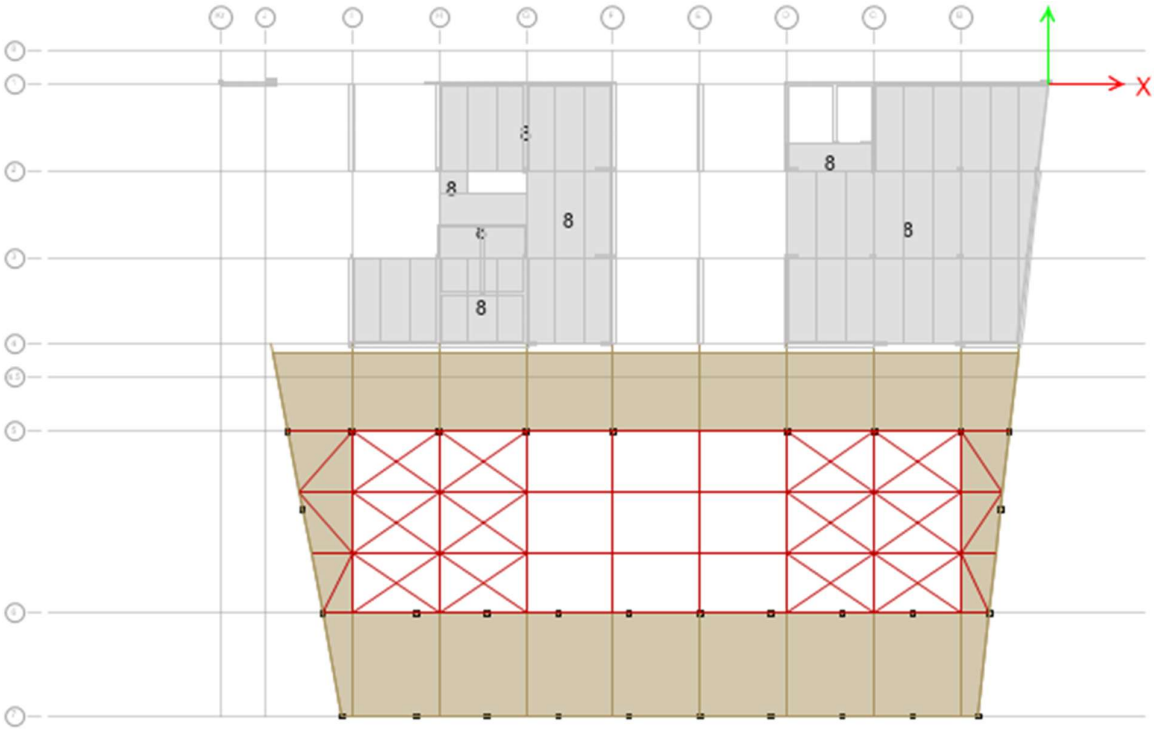


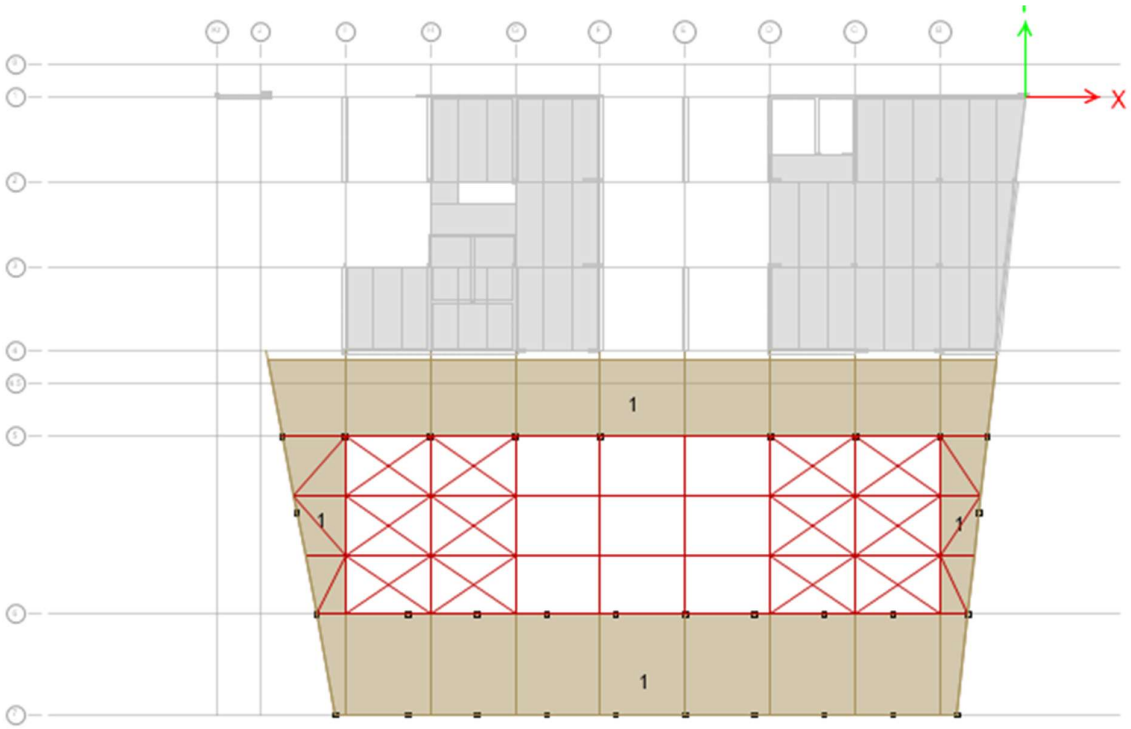
Copyright © 1976 - 2026 Buro Happold. All rights reserved



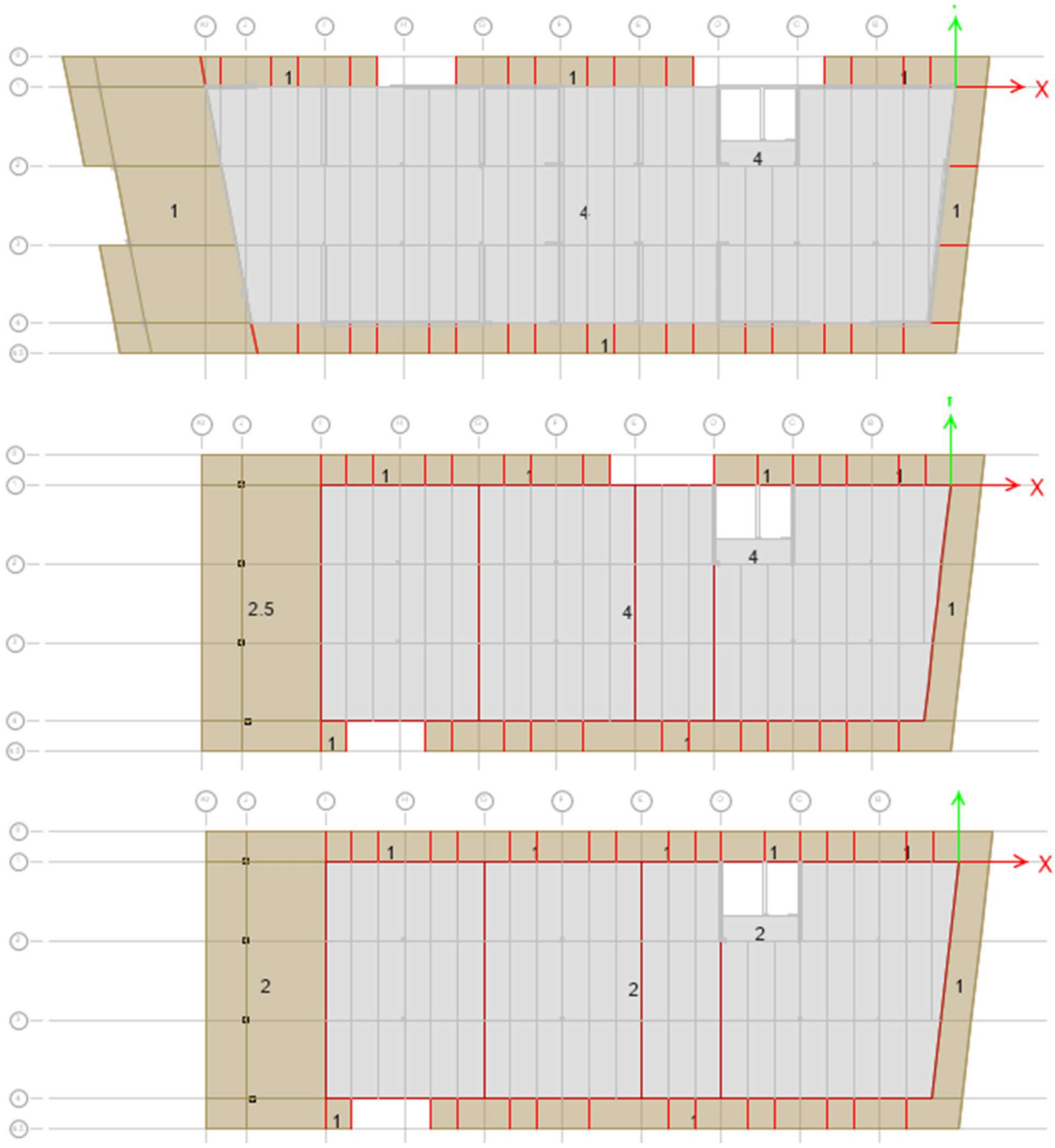
Copyright © 1976 - 2026 Buro Happold. All rights reserved

A.2 Loading Plans: Imposed Load





Copyright © 1976 - 2026 Buro Happold. All rights reserved



Copyright © 1976 - 2026 Buro Happold. All rights reserved

Appendix B Manual Checks

Copyright © 1976 - 2026 Buro Happold. All rights reserved

Appendix C ETABS Foundation Forces

Copyright © 1976 - 2026 Buro Happold. All rights reserved

Appendix D ETABS Report

Copyright © 1976 - 2026 Buro Happold. All rights reserved

Appendix E ETABS Element Forces

E.1 Spandrel Forces

E.2 Pier Forces

E.3 Column Forces (Primary)

E.4 Brace Forces

E.5 Beam Forces (Primary)

Copyright © 1976 - 2026 Buro Happold. All rights reserved

A Gonzalez
Buro Happold Limited
Level 5-8, The Featherstone Building
66 City Road
London, EC1Y 2AL
UK

T: +44 (0)207 927 9700
F: +44 (0)870 787 4145
Email: alfredo.gonzalez@burohappold.com