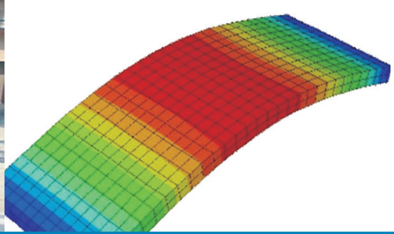
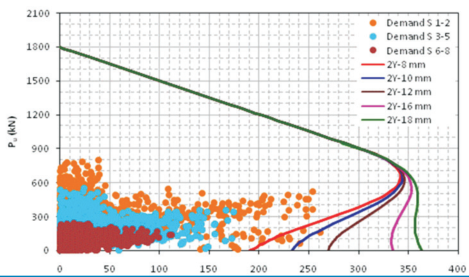




GFRG/RAPIDWALL BUILDING STRUCTURAL DESIGN MANUAL

Structural Engineering Division
Department of Civil Engineering
IIT Madras

Building Materials & Technology Promotion Council
Ministry of Housing & Urban Poverty Alleviation
Government of India

Use of Glass Fibre Reinforced Gypsum (GFRG) Panels in Buildings

STRUCTURAL DESIGN MANUAL

Prepared by

Structural Engineering Division
Department of Civil Engineering
IIT Madras

Published by

Building Materials & Technology Promotion Council
Ministry of Housing & Urban Poverty Alleviation
Government of India

Disclaimer

The information presented in this Manual is supplied in good faith and is based entirely on the test data and design guidelines furnished for GFRG building panel. Indian Institute of Technology Madras is not responsible for incorrectness, if any, in such data furnished for GFRG building panels.

Acknowledgment

The support of Rapid Building Systems Pty Ltd Australia, RBS India Pvt Ltd, Rashtriya Chemicals and Fertilizers (RCF) Mumbai, FACT-RCF Building Products Ltd (FRBL) Cochin – Govt of India Public undertaking, is gratefully acknowledged.

FOREWORD

The Glass Fibre Reinforced Gypsum (GFRG) Panel System, commonly known as Rapidwall, is an alternate technology for construction of buildings originally developed and being in use for last two decades in Australia. The panels are manufactured in semi-automatic machine using phosphogypsum - an industrial waste from fertilizer plants and glass fibre rovings. The technology is evaluated by BMTPC under the Performance Appraisal Certified Scheme (PACS). The evaluation of the system is based on various tests performed on the panels. The panels, due to unique configuration and materials have properties different than normal conventional construction. Besides Australia and China, materials properties, strength and behavior of the panels have been studied in India at IIT Madras.

While evaluating the system, it was realized that the design Manual used in Australia for the system, needed appropriate modification to ensure conformity with the prevailing Indian standards and to incorporate the research findings of the studies undertaken at IIT Madras. Accordingly, this Designs Manual has been prepared by IIT Madras and being published by BMTPC. The Manuals deals with the engineering design aspect of GFRG Panels and endeavours to provide guidelines to the engineers who intend to design building using GFRG Panels.

This Manual is mandatory and required to be followed for design of any buildings using GFRG Panels manufactured by RCF as per the design specifications.

BMTPC places deep appreciation for the valuable technical contributions made by Dr. Devdas Menon and Dr. A. Meher Prasad of IIT Madras in developing painstakingly this Manual. I also acknowledge the team at BMTPC specially Shri J.K.Prasad, Shri A.K.Tiwari and Shri Dalip Kumar for successful operation of the scheme and bringing out the publication in time.

I hope the Manual will serve the purpose for designing the buildings using GFRG Panels and will go a long way in propagating the Rapidwall technology as an alternate technology for building construction.

Dr. Shailesh Kr. Agrawal
Executive Director, BMTPC

19th Day of April, 2012

PREFACE

GFRG is the abbreviation for *glass fibre reinforced gypsum*. It is the name of a new building panel product, made essentially of gypsum plaster, reinforced with glass fibres, and is also known in the industry as Rapidwall®. This product, suitable for rapid mass-scale building construction, was originally developed and used since 1990 in Australia. GFRG is of particular relevance to India, where there is a tremendous need for cost-effective mass-scale affordable housing, and where gypsum is abundantly available as an industrial by-product waste. The product is not only eco-friendly or green, but also resistant to water and fire. GFRG panels are presently manufactured to a thickness of 124 mm, a length of 12m and a height of 3m, under carefully controlled conditions. The panel can be cut to required size. Although its main application is in the construction of walls, it can also be used in floor and roof slabs in combination with reinforced concrete.

The panel contains cavities that may be filled with concrete and reinforced with steel bars to impart additional strength and provide ductility. The panels may be unfilled, partially filled or fully filled with reinforced concrete as per the structural requirement. Experimental studies and research have shown that GFRG panels, suitably filled with reinforced concrete, possess substantial strength to act not only as load-bearing elements, but also as shear walls, capable of resisting lateral loads due to earthquake and wind. It is possible to design such buildings up to ten storeys in low seismic zones (and to lesser height in high seismic zones). However, such construction needs to be properly designed by a qualified structural engineer. Manufacture of GFRG panels with increased thickness (150 mm, 200 mm) with suitable flange thickness can facilitate design and construction of taller buildings.

GFRG panels can also be used advantageously as infills (non-load bearing) in combination with reinforced concrete (RC) framed columns and beams (conventional framed construction of multi-storey buildings) without any restriction on the number of storeys. Also, GFRG panels with embedded micro-beams and RC screed (acting as T-beams) can be used as floor/roof slabs. Design of an eight storeyed building with GFRG panels as load bearing shear walls as well as infill walls in framed construction is included in this manual for reference.

Some of the advantages of construction using GFRG panels are:

(i) Substantial reduction in the structural weight of the building;

(ii) No plastering requirement for walls and ceiling;

(iii) Increased speed of construction with less manpower;

(iv) Saving of cement, steel, river sand, burnt clay bricks / concrete blocks and hence saving of energy and reduced CO_2 emissions, contributing to environmental protection and mitigating climate change;

(v) Use of reprocessed / recycled industrial by product, viz., waste gypsum, to manufacture GFRG panel, helping to abate pollution and protect the environment.

GFRG building systems can be constructed only with technical support or supervision by qualified engineers and constructors, based on structural designs carried out in detail complying to prevailing standards; this is applicable even for low-rise and affordable mass housing, to provide for desirable safety margins against natural disasters (such as earthquakes and cyclones).

GFRG panels can be unfilled when used as partition walls, but when used as external walls, need to be suitably designed (with reinforced concrete filling) in order to resist the design wind pressures. For single-storey construction (suitable for affordable mass housing), unfilled GFRG panels can be used for walls as well as roof (which may be pitched suitably), with local reinforced concrete filling at the joints between walls and between the roof and walls. It is mandatory to provide embedded RC horizontal tie beams over all the walls below the floor slab / roof slab.

Based on the experience in Australia, as well as tests and independent evaluation reported by IIT Madras and SERC Chennai, Building Materials & Technology Promotion Council (BMTPC), Government of India, has accorded approval of GFRG panels for construction in India. However, to facilitate such construction, it was felt necessary to bring out appropriate design and construction manuals that meet the statutory requirements of relevant Indian Standards. The task of preparing the Design Manual has been entrusted to IIT Madras by Rapidwall manufacturers and technology promoters, viz. Rahstriya Chemicals and Fertilizers (RCF) Ltd, Mumbai, a Govt of India fertiliser company. RCF and The Fertilizers and Chemicals Travancore (FACT) Ltd, another fertiliser company owned by the Government of India, are also jointly setting Rapidwall manufacturing plant in Cochin, Kerala.

This Manual deals with the engineering design aspects of GFRG panel construction. Much of the material presented here is adapted from the design manual presently in use in Australia ("Rapidwall® Engineering Design Guidelines", prepared by Dare Sutton Clarke, Australia).

Appropriate changes have been made to ensure conformity with the prevailing Indian Standards, and to incorporate recent research findings at IIT Madras. There is a need for continued structural testing of various aspects of GFRG panels, manufactured in India. GFRG panels, manufactured by RCF, Mumbai have been tested by IIT Madras and found to match the strength and performance of panels manufactured in Australia, as specified by the technology provider. Details regarding the method of construction and its sequence, including erection of GFRG panels, are explained in the GFRG building construction manual.

It is strongly recommended that structural engineers and building designers associated with GFRG panel construction should be thoroughly familiar with the various structural design aspects, as outlined in this Manual. It is also recommended that architects and construction engineers who undertake GFRG / Rapidwall building design and construction gain familiarity with the properties and material characteristics of Rapidwall and its applications and construction systems.

Dr. Devdas Menon (Professor)

Dr. A. Meher Prasad (Professor)

Structural Engineering Division
Department of Civil Engineering
Indian Institute of Technology Madras
Chennai 600 036, India.

December 2011

TABLE OF CONTENTS

1. INTRODUCTION

This Design Manual provides guidelines and recommendations for the design of GFRG / Rapidwall building panels as structural walling and flooring systems in buildings. The Manual is intended to be used by structural design engineers, for the purpose of ensuring that adequate strength and safety requirements are met with in the design of such buildings. Designers are also advised to follow the guidelines given in the Construction Manual for construction using GFRG building panels, joint details, etc.

This Manual, which is a revision of an earlier 2003 version, has been prepared, essentially based on the following references:

1. RAPIDWALL® (2002) Engineering Design Guidelines, compiled by Ms Dare Sutton Clarke Engineers, Adelaide, Australia.
2. RAPIDWALL® (2001) Report on Physical Testing and Development of Design Guidelines for the structural application of Rapidwall® in building construction prepared by Ms Dare Sutton Clarke Engineers, Adelaide, Australia.
3. IS: 456 (2000), Code of practice for plain and reinforced concrete for general building construction, Bureau of Indian Standards, New Delhi.
4. IS: 1905 (1987), Code of practice for structural use of unreinforced masonry, Bureau of Indian Standards, New Delhi.
5. SERC (2002), Evaluation of seismic performance of gypcrete building panels, Structural Engineering Research Centre, Chennai, India; August 2002. Project no. CNP 053241/2.
6. SERC (2002), Investigation on the behaviour of gypcrete panels and blocks under static loading, Structural Engineering Research Centre, Chennai, India; August 2002. Project no. CNP 053241/1.
7. IITM (2002), Material properties and assessment of gypcrete building panels, Indian Institute of Technology, Madras, India; September 2002. Project no. CE/BTCM/2557/2002.
8. Sreenivasa, R. L (2010), Strength and behaviour of glass fibre reinforced gypsum wall panels, Indian Institute of Technology Madras, PhD Thesis.

9. Janardhana, M. (2010), Cyclic behaviour of glass fibre reinforced gypsum wall panels, Indian Institute of Technology Madras, PhD Thesis.

The main content of this Manual deals with the design of GFRG panels subject to (i) axial compression, (ii) compression with out-of-plane bending and (iii) compression with in-plane bending and shear. The design of lintels, slabs and pitched roofs are also included.

GFRG building panels are considered with and without reinforced concrete as infill in the cavities. Other data related to building design, such as fire resistance and sound transmission, are also included in the guidelines. The guidelines are presented in various Sections, each covering a specific property of GFRG building panel.

To facilitate design calculations, example problems are taken up for demonstration, wherever appropriate. In Section s 13-15, detailed examples are provided for the design of GFRG shear walls in multi-storeyed and low-rise buildings. The use of GFRG panels as infill walls in RC framed construction is also demonstrated. All design load calculations in this Manual in general are in accordance with IS:875 1987 – (Parts 1-5) [10,11,12,13,14] and IS:1893 2002 -(Part 1) [13].

The design methodologies presented in this Manual may be treated as a set of guidelines for the safe and efficient structural use of GFRG building panel in general building construction, rather than as a strict code of practice. The guidelines are for use by appropriately experienced and qualified structural engineers who are familiar with concrete and masonry design and construction. The designers are expected to have knowledge about the general behaviour of GFRG building panel.

2. NOTATION

A	=	Gross cross-sectional area of panels with cavities
A_c	=	Net area of cross-section of concrete infill
A_p	=	Net area of cross-section of the panel
A_h	=	Design horizontal seismic coefficient
A_s	=	Total area of steel bars in a panel
A_{st}	=	Area of steel bar in a cell
A_{sv}	=	Area of stirrup steel bars
b	=	Breadth of a section
C_u	=	Total design compressive force (factored)
D	=	Depth or width of cross-section
DL	=	Dead load
d	=	Effective depth of cross-section
d_a	=	Base dimension of the building at the plinth level along the direction of lateral force.
d_p	=	Effective depth
e	=	Eccentricity
EI	=	Flexural rigidity
EL	=	Earthquake load
f_{ck}	=	Characteristic cube strength of concrete
f_{st}	=	Stress in steel bars
f_y	=	Characteristic strength of steel
h	=	Height of the building
h_i	=	Height of i^{th} floor measured from base
I	=	Second moment of area
I_f	=	Importance factor
L	=	Length of panel
LL	=	Live load
M_u	=	Factored bending moment
M_{uc}	=	Out-of-Plane moment capacity, rib parallel to span
$M_{uc\text{-}perp}$	=	Out-of-Plane moment capacity, perpendicular to span

M_{ud}	=	Design bending moment capacity
n	=	Number of floor levels at which masses are distributed
p_t	=	Percentage of tension steel
P_u	=	Factored axial load
P_{uc}	=	Uni-axial Compressive Strength
P_{ud}	=	Design axial load capacity
Q_i	=	Design lateral force at i^{th} floor
R	=	Response reduction factor
s_v	=	Spacing of stirrups (steel bars)
t	=	Thickness of panel
T_a	=	Time period of the building
T_{uc}	=	Uni-axial Tensile Strength
UDL	=	Uniformly distributed load
V_u	=	Factored shear force
V_{uc}	=	Ultimate shear strength
V_{ud}	=	Design shear capacity
V_z	=	Design wind speed at a height z from the ground
W_i	=	Seismic weight of i^{th} floor
WL	=	Wind load
x_i	=	Distance from the centre of the panel to the centre of the rebar in each cavity
x_u	=	Depth of compression zone
y	=	Distance to centroid of cross – section from base
Z	=	Zone factor
γ_f	=	Partial safety factor for load
γ_m	=	Partial safety factor for GFRG building panel
γ_s	=	Partial safety factor for reinforcing steel
η	=	Ratio of infilled cavities to total number of cavities
σ	=	Stress
τ_v	=	Nominal shear stress
τ_c	=	Permissible shear stress

3. PRODUCT DIMENSIONS

These design guidelines are applicable to GFRG building panels, presently manufactured as Rapidwall®, for the typical dimensions and material properties described in this manual. Typical dimensions of a GFRG building panel are 12.0m × 3.0m × 0.124 m, as shown in Fig. 3.1. Each 1.0 m segment of the panel contains four 'cells'. Each cell is 250 mm wide and 124 mm thick, containing a cavity 230 mm × 94 mm, as shown in Fig. 3.2. The various cells are inter-connected by solid 'ribs' (20 mm thick) and 'flanges' (15 mm thick), comprising gypsum plaster, reinforced with 300 - 350 mm glass fibre roving, located randomly but centrally. The skin thickness is 15 mm and rib thickness is 20 mm.

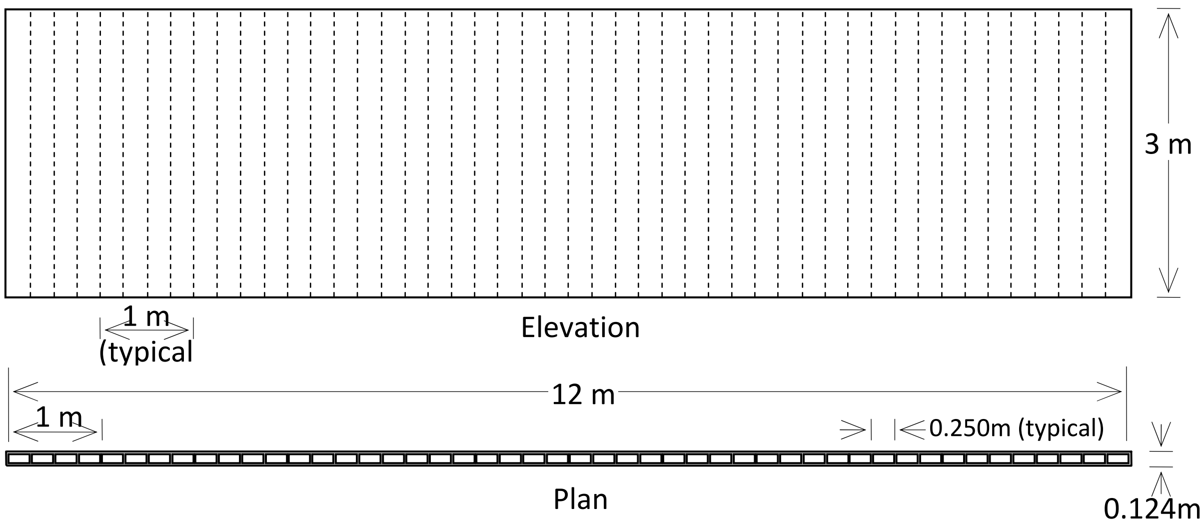


Fig. 3.1: Typical Cross Section of GFRG Panel

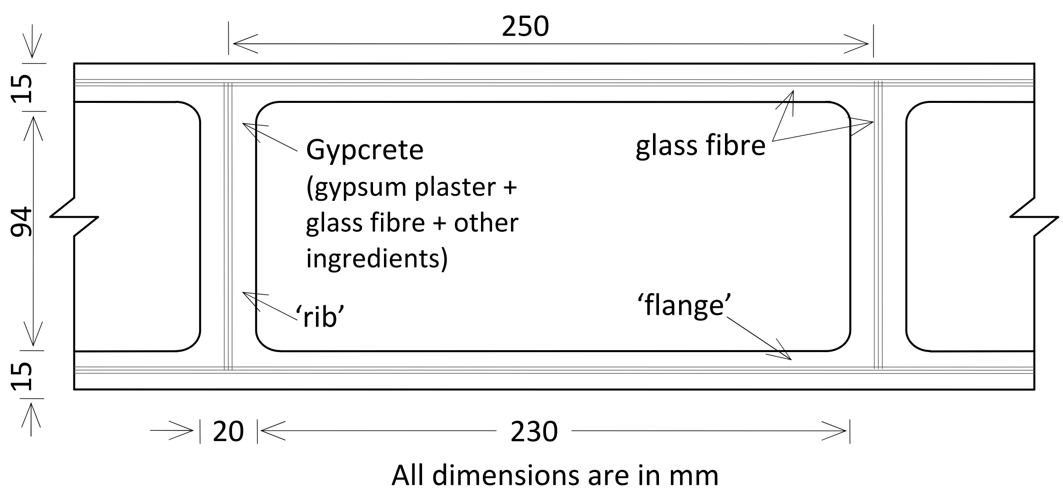


Fig. 3.2: Enlarged View of a Typical Cell

4. APPLICATIONS

GFRG building panels are generally used structurally in the following seven ways:

1. As **load bearing walling** (refer Fig.4.1) in buildings. When the cavities are filled with reinforced concrete, the strength of the panel to resist vertical and lateral loads gets enhanced considerably, rendering such load bearing constructions suitable for multi-storeyed housing. In single or two storeyed constructions, the cavities can remain unfilled[†] or suitably filled with non-structural core-filling such as insulation, sand, quarry dust, polyurethane or lightweight concrete (refer Fig. 4.2).
2. As **partition infill walls** in multi-storey framed buildings. Panels can also be filled suitably. Such walls can also be used as **cladding** for industrial buildings or sport facilities, etc.
3. As **compound walls / security walls**.(refer Fig. 4.3)
4. As horizontal **floor slabs / roof slabs** (refer Fig. 4.4): with reinforced concrete micro beams and screed (T- beam action). This system can also be used in inclined configurations, such as staircase waist slabs and **pitched (sloped) roofing** (refer Fig. 4.5).

4.1 Use as Load Bearing Structural Walling

In typical multi-storeyed constructions involving the use of GFRG as load bearing structural walling, the connections between cross walls and with the foundations and floor/roof are achieved through reinforced concrete filling or R.C beams. All GFRG wall panels at the ground floor are to be erected over a network of RC plinth beams supported on suitable foundation (refer Figs 4.6 and 4.7). 'Starter bars' shall be embedded in the RC plinth beams, at the precious locations where the cavities are to be filled with reinforced concrete, with appropriate lap length. In this manner, the connection at the ground storey between super structure and foundation, spread over the entire wall length over the network of RC plinth beams is ensured.

† Every fourth cavity, however, shall be filled with reinforced concrete of M20 grade and 1-Y 8 mm bar.

Fig. 4.1 Proposed muti-storeyed housing at RCF, Mumbai

Fig. 4.2 Model house constructed at FACT, Cochin

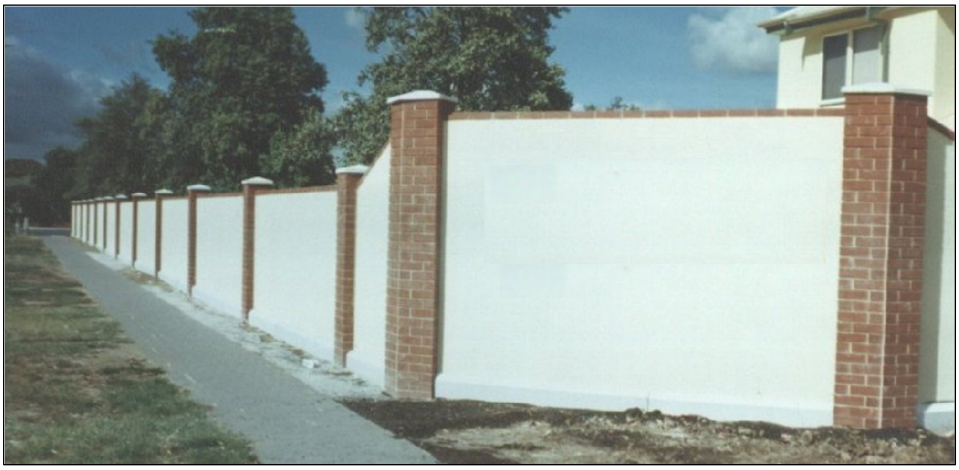
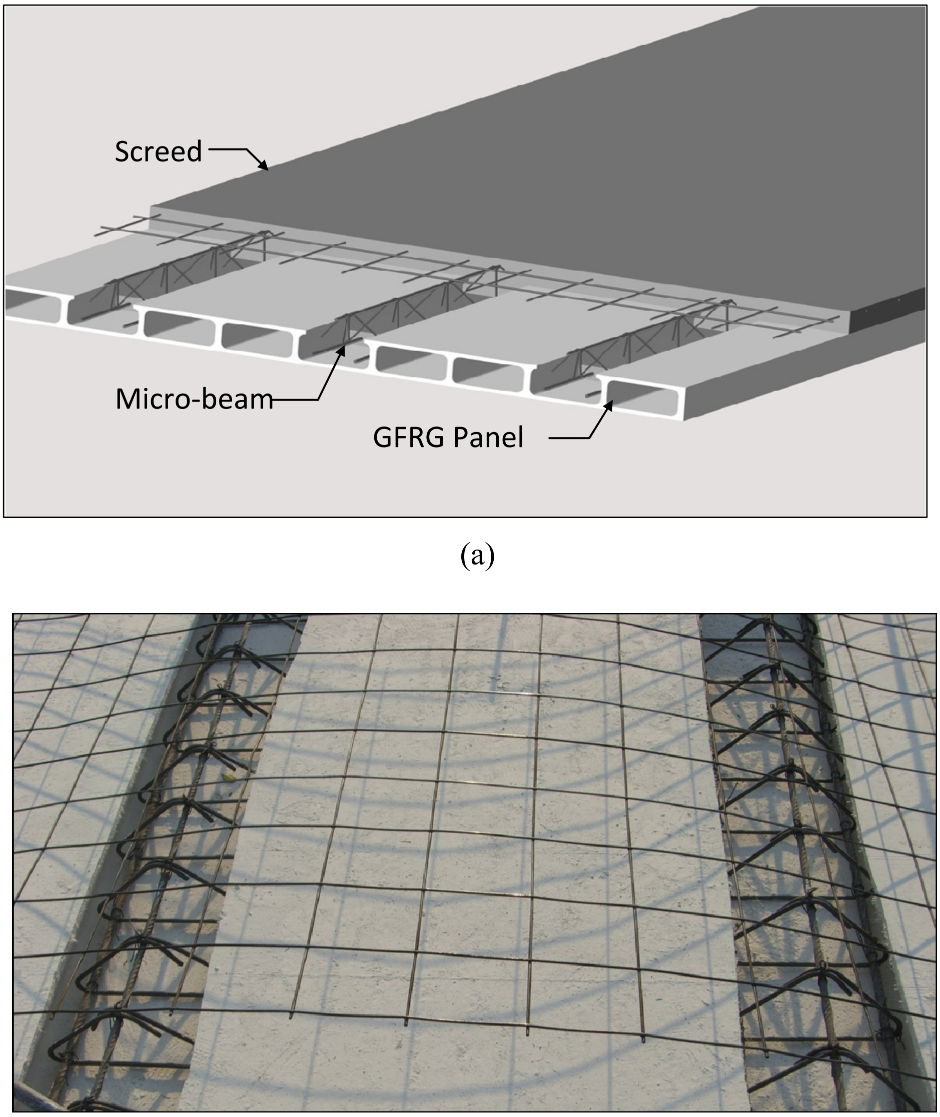


Fig. 4.3 GFRG compound wall



(a)

(b)

Fig. 4.4 GFRG floor slab with micro beam and screed

Fig. 4.5 Small GFRG house with pitched roof

Fig. 4.6 Erection of GFRG panels over plinth beam at site

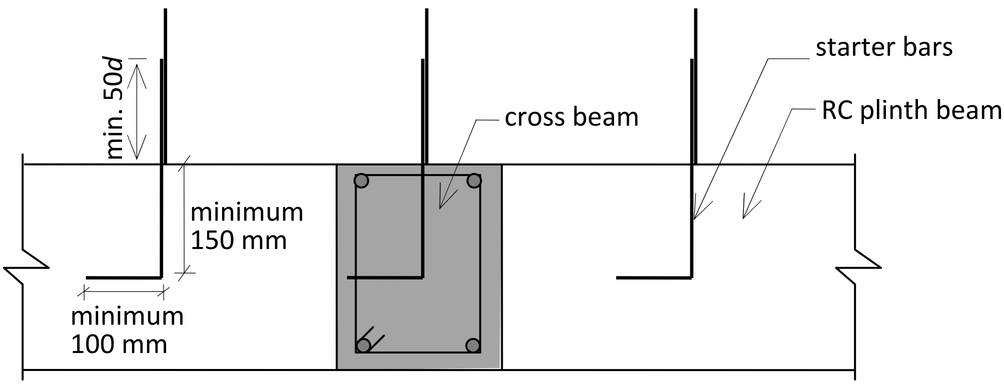


Fig. 4.7 Provision of starter bars in RC plinth beams for the erection of GFRG panels

For constructing an additional GFRG floor above an existing RC building, connectivity between GFRG wall and the existing floor can be achieved by proper detailing (insertion of starter bars with proper anchorage) as shown in Fig. 4.8. If the existing floor slab does not have sufficient depth for anchorage, an additional RC beam may be constructed above the roof before erecting the GFRG walls.

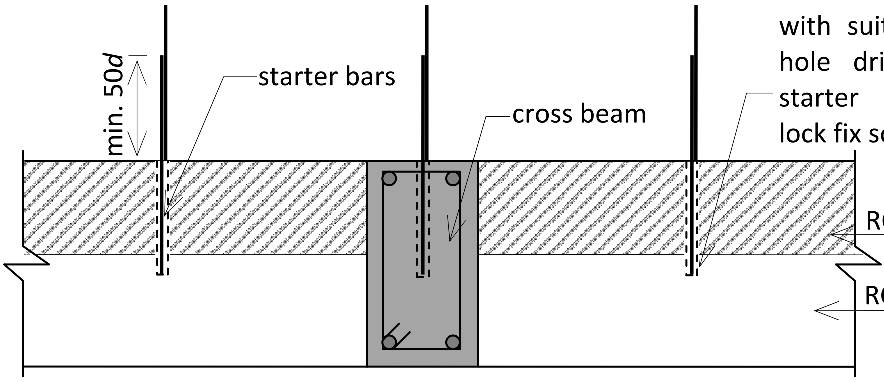


Fig. 4.8 Starter bars in case of an existing RC floor/roof slab

When GFRG panel is used as structural walling, an embedded horizontal RC tie beam has to be provided on top of all the walls. Tie beam size of 200 mm depth and 94 mm width is suggested by cutting and removing the top portion of the web of GFRG panels as shown in Fig. 4.9.

4.2 Use as Floor /Roof Slabs

GFRG panel can also be used for intermediate floor slab/roof slab in combination with RC (Refer Figs 4.4). The strength of GFRG slabs can be significantly enhanced by embedding reinforced concrete micro beams. For providing embedded micro beams, top flange of the respective cavity is cut and removed in such a way that minimum 25 mm flange on both end is protruded as shown in Fig. 4.4. RC concrete screed of minimum 50 mm thickness is provided above the GFRG floor panel, which is reinforced with weld mesh of minimum size of 10 gauge 100 mm × 100 mm. This RC screed and micro beam act together as series of embedded T-beams. The thickness of the RC screed, reinforcement and interval of embedded RC micro beams depends on the span and intensity of imposed load. The connectivity between the horizontal tie beam, embedded RC micro beams, concrete screed and vertical rods in GFRG wall, ensures perfect connection between floor/roof slab and walling system (refer Fig. 4.9).

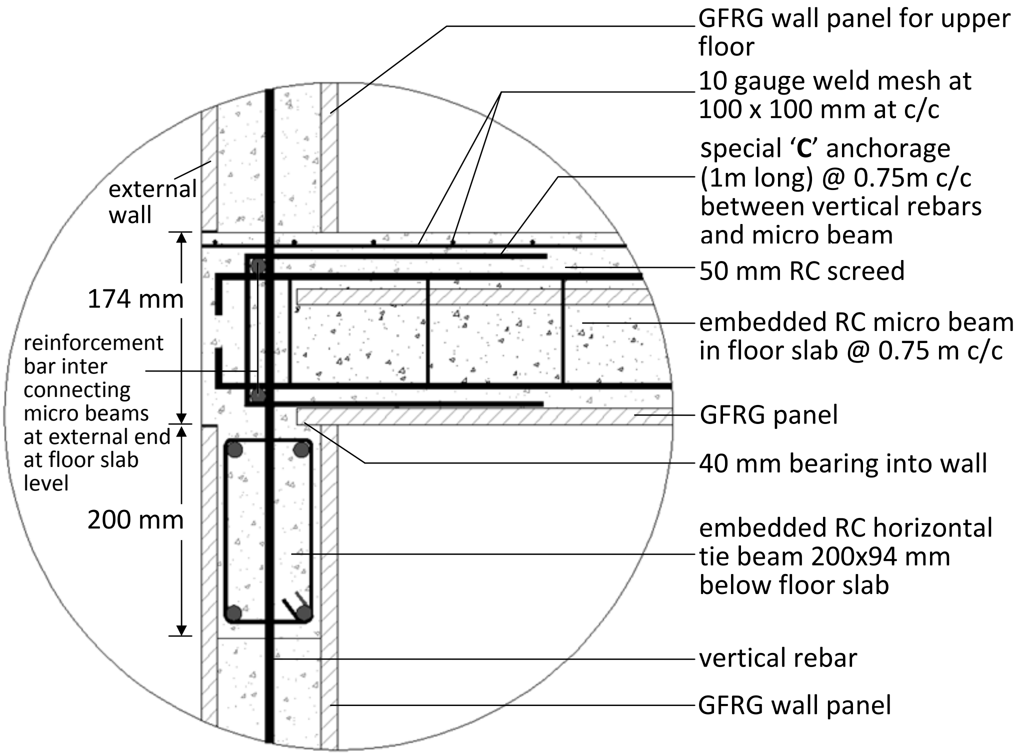


Fig. 4.9 Connectivity between floor slab and wall

5. MECHANICAL PROPERTIES

Table 5.1 provides a summary of typical mechanical properties of the GFRG building panel. These properties have been determined from tests on GFRG building panel[1,5,6,7,8,9]. The compressive strength to be considered when the panel is filled with concrete is given in Table 5.2[1,5,6,7,8].

Table 5.1 Mechanical Properties of GFRG Building Panel (Unfilled)

Mechanical Property	Nominal Value	Remarks
Unit weight	0.433 kN/m^2	
Modulus of elasticity	7500 N/mm^2	
Uni-axial compressive strength, P_{uc}	160 kN/m	Strength obtained from longitudinal compression / tension tests with ribs extending in the longitudinal direction
Uni-axial tensile strength, T_{uc}	34 - 37 kN/m	
Ultimate shear strength, V_{uc}	21.6 kN/m	
Out-of-plane moment capacity, Rib parallel to span, M_{uc}	2.1 kNm /m	
Out-of-plane moment capacity, Rib perpendicular to span, $M_{uc\text{-}perp}$	0.88 kNm /m	
Mohr hardness	1.6	
Out-of-plane flexural rigidity, *EI,* Rib parallel to span	3.5×10^{11} Nmm^2/m	
Out-of-plane flexural rigidity, *EI,* Rib perpendicular to span	1.7×10^{11} Nmm^2/m	
Coefficient of thermal expansion	12×10^{-6} mm/mm /^{o}C	
Water absorption	1.0 % : 1hr 3.85 % : 24hrs	Average water absorption by weight % after certain hours of immersion
Fire resistance: Structural adequacy/Integrity/Insulation	140/140/140 minutes	CSIRO, Australia
Sound transmission class (STC)	40 dB	ISO 140-3-1996[7]

Table 5.2 Properties of compressive strength of GFRG building panel (filled with minimum M20 grade concrete in all the cores)

Property	Nominal Value	Remarks
Uni-axial compressive strength, P_{uc} (Both ends hinged)	1310 kN/m*	Obtained from longitudinal compression tests with ribs in the longitudinal direction
Uni-axial compressive strength, P_{uc} (one end fixed and other end hinged)	1360 kN/m*	— as above —
Ultimate shear strength, V_{uc}	61 kN/m†	Longitudinal cracks (parallel to the ribs)
Fire resistance: Structural adequacy/Integrity/Insulation	241/241/241 minutes	CSIRO, Australia

The nominal value of uniaxial compressive strength and in-plane shear strength of partially infilled GRFG panel with M20 concrete (both ends fixed) may be calculated as follows.

$$P_{uc} = (160 + 1200\eta)\ \text{kN/m} \tag{5.1}$$

$$V_{uc} = (21.6 + 38.4\eta)\ \text{kN/m} \tag{5.2}$$

where η is given by

$$\eta = \frac{\text{Number of infilled cavities in the panel}}{\text{Total number of cavities in the panel}} \tag{5.3}$$

The typical stress-strain curve of GFRG under compression, obtained from tests on small prisms of 250mm high and 520mm wide is shown in the Fig.6.1[(8)].

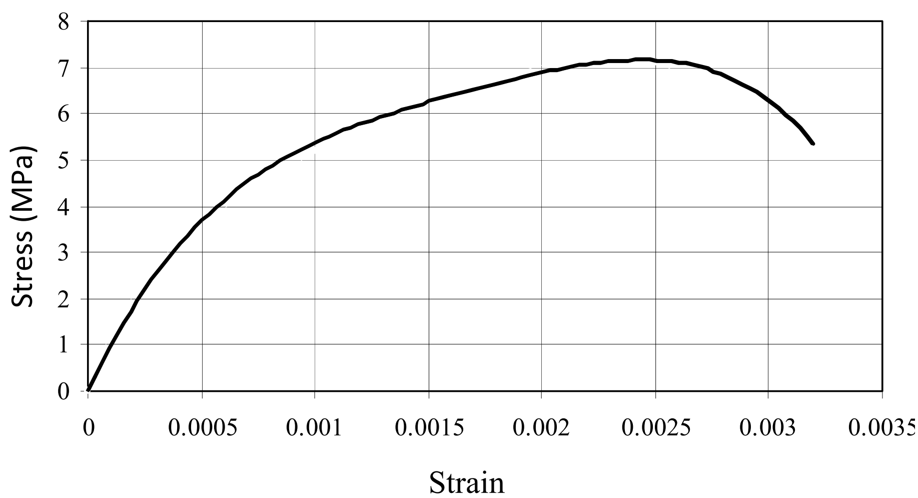


Fig. 5.1 Stress – strain curve of GFRG unfilled prism under compression

* Wu, Y. F. and M. P. Dare (2004), Axial and shear behaviour of glass fibre reinforced gypsum wall panels: tests. Journal of Composites for Construction ASCE, 8(6), 569–78

† Janardhana, M. (2010), Cyclic behaviour of glass fibre reinforced gypsum wall panels, Indian Institute of Technology Madras, PhD Thesis

6. DESIGN PHILOSOPHY

The design capacities given in these guidelines are based on limit states design procedures, considering the ultimate limit state for strength design, treating the 3.0 m high GFRG building panel as the unit material, and considering the strength capacity as obtained from test results. The design should be such that the structure should withstand safely all loads (as per relevant Indian Standards) likely to act on the structure during its lifetime. It shall also satisfy serviceability requirements, such as limitations of deflection and cracking. In general, the structure shall be designed on the basis of the most critical limit state and shall be checked for other limit states.

6.1 Limit States Design

For ensuring the design objectives, the design should be based on the characteristic values of material strengths and applied loads (actions), which take into account the probability of variations in material strength and load. The design values are derived from the characteristic values through the use of partial safety factors, both for material strengths and for loads, for limit states of collapse and serviceability.

6.1.1 *Partial Safety Factors for Load, γ_f*

The design must account for various combinations of loads acting on the structure simultaneously. The various load combinations and corresponding partial safety factor for loads shall be used as given in IS 456: 2000 [3], as summarized in Table 6.1.

Table 6.1 Values of Partial Safety Factor γ_f for Loads

Load Combination	**Limit State of collapse**			**Limit State of Serviceability**		
	DL	*LL*	*WL/EL*	*DL*	*LL*	*WL/EL*
DL+LL	1.5	1.5	-	1.0	1.0	-
DL+WL/EL	1.5 or 0.9(*)	-	1.5	1.0	-	1.0
DL+LL+WL/EL	1.2	1.2	1.2	1.0	0.8	0.8

Note: - For the limit state of serviceability, the values of γ_f given in this table are applicable for short-term effects. While assessing long term effects due to creep, the dead load and that part of live load likely to be permanent should be considered.

(*) this value is to be considered when stability against overturning or stress reversal is critical.

6.1.2 *Partial Safety Factor for Material, γ_m*

The magnitude of partial safety factor for the material must take into account the uncertainty related to the material strength. Although GFRG building panels are manufactured under carefully controlled conditions, it is considered prudent to treat the material like concrete, for which the partial safety factor specified in IS 456: 2000 is 1.50. The partial safety factor for the GFRG building panel (with and without concrete infill) shall be taken as $\boldsymbol{\gamma_m}$ **= 1.50** in general. The above partial safety factor $\gamma_m = 1.50$ is applicable to situations involving out of plane bending where the observed mode of failure is brittle as well as in plane bending of RC filled GFRG panels where the mode of failure is expected to be ductile.

In the case of reinforcing steel, the partial safety factor shall be taken as $\boldsymbol{\gamma_s}$ **= 1.15** in all cases, as recommended in IS 456 : 2000.

While investigating serviceability limit states, the partial safety factor for all materials should be taken as unity**.**

6.2 Response Reduction Factor for Earthquake Resistant Design

Earthquake resistant design shall be carried out in compliance with the requirements of IS 1893 (Part 1). In such design, an important and difficult task is the determination of the response reduction factor (*R*). This is traditionally arrived at, based on the general observed performance of similar buildings during past earthquakes, estimates of general system toughness and the amount of damping present during inelastic response. As GFRG buildings constitute a new type of structure, a reasonable choice of *R* factor can only be made by comparing the GFRG building system with traditional structures, such as reinforced concrete wall building systems for which the response modification factors are already available.

GFRG walls are composite members with partial interaction, and the ductility of a partially interactive member is generally greater than that of a fully interactive reinforced concrete member. In terms of strength reserve, it is recommended that the safety margin adopted for the design of GFRG walls be at least as large as that adopted for concrete structures. Therefore, it is not unreasonable to treat buildings constructed with GFRG walls as reinforced concrete shear wall structures and to adopt the *R* values from the respective code of practice (Wu 2009)[10]. Hence, the response reduction factor (*R*) is taken as 3.0 (IS 1893-2002) for seismic load calculations.

7. AXIAL LOAD CAPACITY

While assessing the axial load capacity of GFRG building panel (under compression), it is important to consider possible eccentricities in loading. A minimum eccentricity (causing out-of-plane bending) must always be accounted for in the design.

7.1 Minimum Eccentricity

According to IS 456: 2000 (cl.32.2.2), the design of a reinforced concrete wall shall take into account the actual eccentricity of the vertical force subjected to a minimum value of 0.05t (6.2 mm for panel thickness t = 124 mm). As per IS 1905: 1987, the design of a masonry wall shall consider appropriate eccentricity, which in no case shall be taken to be less than $t/24$ (5.2 mm for t = 124 mm).

In the case of wall panels supporting floor slabs from one side only, the eccentricity to be considered should be more than the minimum values indicated above. **It is recommended that a value of minimum eccentricity equal to $t/6$ (i.e., 20.7 mm) shall be considered conservatively.** Additional value of eccentricity may be considered when out-of-plane bending is explicitly involved (for example, action of local wind effects on an exposed wall panel).

7.2 Axial Compressive Strength

The characteristic values of axial compressive strength of the GFRG building panel, expressed in kN/m, are obtained from compression test results on GFRG building panel for full height panel, subject to various eccentricities of loading (20 mm, 30 mm and 45 mm) and different boundary conditions[1,8,11]. In general, it is conservative to assume pinned – pinned condition, as shown in Fig. 7.1.

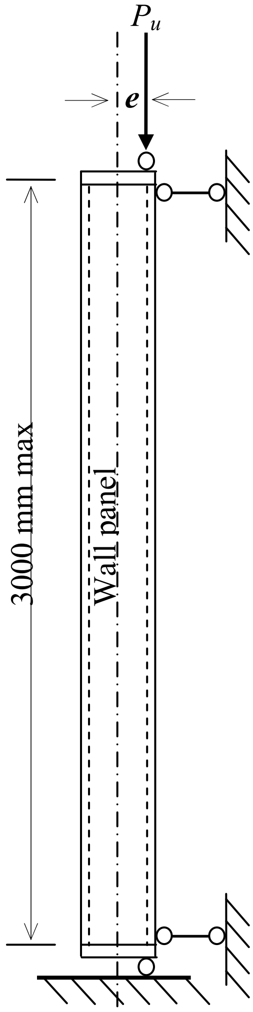


Fig. 7.1 Experimental setup for pinned – pinned panels

It may be noted that, for design purposes, the reported nominal values should be divided by γ_m **= 1.5**. The design values (including partial safety factor) are depicted in Figs 7.2 and 7.3, corresponding to unfilled and filled cases respectively, assuming a linear variation of axial load with eccentricity.

Unfilled Panels

The design axial load capacity (P_{ud}), including partial safety factor, of the unfilled GFRG building panel can be found from the following equation for all heights of the wall less than 3.0m (assuming pinned-pinned end conditions).

$$P_{ud} = (68 - 0.9e) \tag{7.1}$$

where, e is eccentricity in mm and P_{ud} is in kN/m.

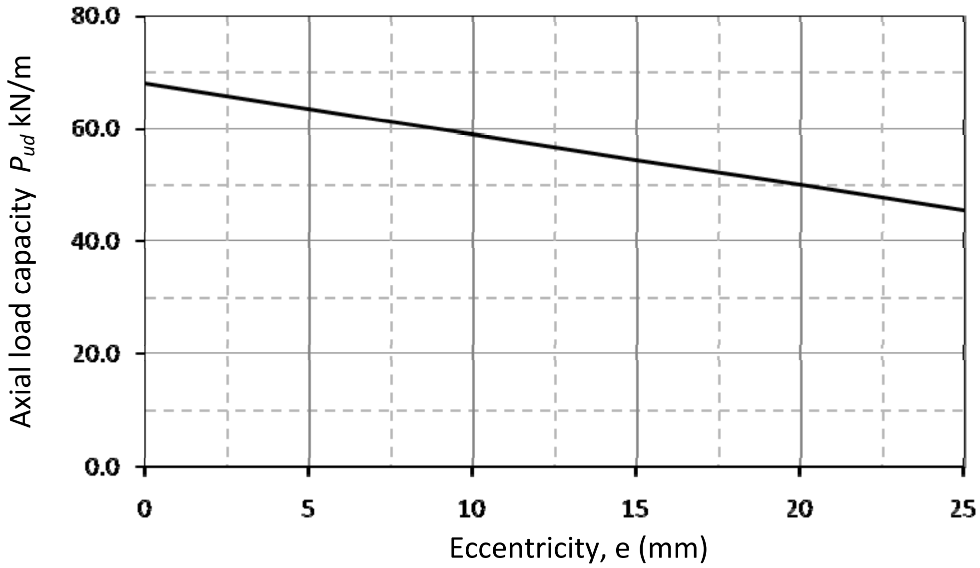


Fig.7.2 Axial Load capacity of unfilled GFRG building panel for all heights of walls < 3 m with both ends pinned

Filled Panels

The design axial load capacity (P_{ud}), including partial safety factor, of the **filled** GFRG building panel (filled with minimum M 20 grade concrete) can be found from the following equation for all heights of the wall less than 3.0m (assuming pinned-pinned end conditions).

$$P_{ud} = (600 - 13.75e) \tag{7.2}$$

where, e is eccentricity in mm and P_{ud} is in kN/m.

The contribution of reinforcing bars in compression shall be ignored. Experimental studies[1,8] have established that there is no significant enhancement in strength under pure compression due to the presence of rebars, as the failure is caused by buckling of the panel.

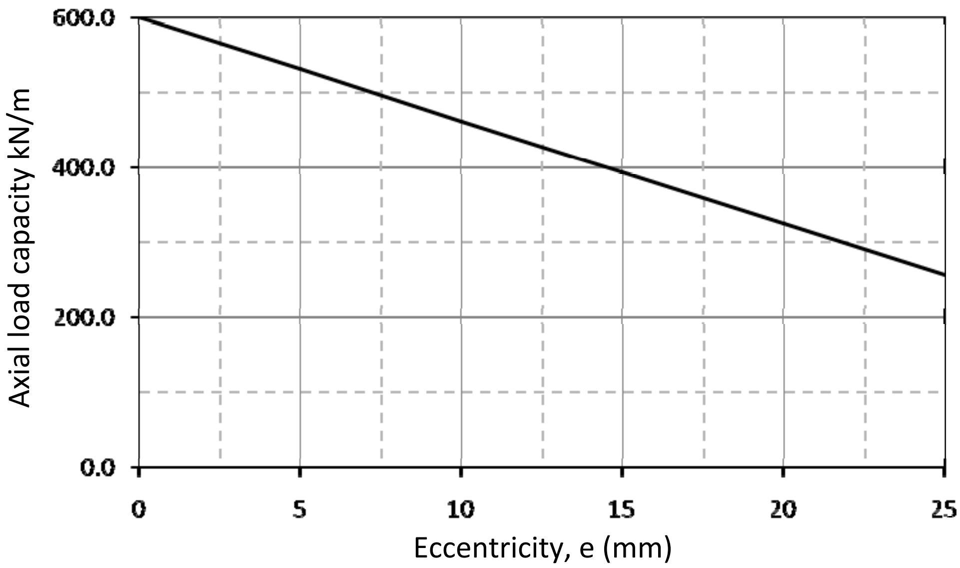


Fig 7.3 Axial load capacity for concrete filled (≥ 20 MPa) GFRG Building panel for all wall heights (≤ 3m) with both ends pinned

7.3 Design Example

An external GFRG building panel of 3.0 m high, filled with 20MPa concrete, is required to resist a vertical load from the roof, comprising a dead load of 75kN/m and a live load of 25kN/m at service stage. Check the adequacy of the panel to resist the vertical load.

The factored axial load on the wall panel is given by 1.5 DL + 1.5 LL as:

$P_u = 1.5 \times (75 + 25)$ = 150 kN/m

Minimum eccentricity = $t/6$ = 20.7 mm

Applying equation (7.2), design capacity in axial compression,

$P_{ud} = (600 - 13.75 \times 20.7)$ = 315.4 kN/m **>** P_u = 150 kN/m; Hence, OK.

8. OUT-OF-PLANE BENDING CAPACITY

Unfilled GFRG Panels

The out-of-plane design flexural strength of the 124 mm thick GFRG building panel without concrete filling is given in Table 8.1. The bending capacity depends on the orientation of the ribs, with respect to the direction of bending. Higher capacity is obtained when the ribs are oriented parallel to the span.

Table 8.1 Out-of-Plane Design Flexural Capacity of Unfilled GFRG Panel

	Ribs parallel to span	Ribs perpendicular to span
Design Moment Capacity, M_{ud}	1.4 kNm/m	0.59 kNm/m

Filled GFRG Panels

When the cavities are filled with concrete, some enhancement in strength can be expected, provided the ribs are aligned parallel to the span. However, full composite action of GFRG and concrete cannot be mobilized on account of bond slip at the inferface. A conservative estimate of the moment capacity can be arrived at by ignoring the contribution of GFRG and considering the action of the concrete beams occupying the cellular cavities (230 mm wide and 94 mm deep), spaced at 250 mm intervals. Accordingly, considering modulus of rupture of concrete as $0.7\sqrt{f_{ck}}$ and applying a partial safety factor of 1.5, the design moment capacity can be derived as follows.

$$M_{ud} = \left[\left(0.7\sqrt{20}\right)\times\left(230\times94^2/6\right)\times10^{-6}\right]\Big/\left(1.5\times0.250\right) \qquad = 2.83 \text{ kNm/m}$$

Further enhancement in capacity is possible by embedding and anchoring reinforcing bars in the middle of the cavities, and designing the reinforced concrete beam elements in accordance with requirements of IS 456-2000. This is demonstrated, by means of an example in section 8.1.

8.1 GFRG Wall Panel Resistance against Wind Loading

The external walls of the building are subjected to wind pressures. It is necessary to check the flexural resistance of the GFRG wall panel against such loading. One way bending action of the panel (full height of 3m) can be assumed conservatively, with simply supported end conditions and a pressure coefficient of unity.

Unfilled GFRG Panels

From Table 8.1, design moment capacity, M_{ud} = 1.4 kNm/m

Design bending moment in the panel due to wind load = $0.6(V_z)^2 \times 3^2 \times (1/8) \times 1.5$

Equating this moment to the design moment capacity of the panel,

$$0.6\,(V_z)^2 \times 3^2 \times (1/8) \times 1.5 = 1.4 \times 1000 \text{ Nm/m}$$

$$\Rightarrow V_z = 37.2 \text{ m/s. (max. safe wind speed for unfilled panel)}$$

Filled GFRG Panels

If the wind speed V_z exceeds the above limit, the panel needs to be filled with concrete and suitably reinforced, as required. In the absence of any reinforcement, considering filling with concrete of minimum M20 grade,

$$0.6\left(V_z\right)^2 \times 3^2 \times \left(1/8\right) \times 1.5 = 2.83 \times 1000 \text{ Nm/m}$$

$$\Rightarrow V_z = 52.9 \text{ m/s.}$$

The capacity can be further enhanced by providing one or two rebars in the middle of each cavity. However, as the bar location is close to the centroidal axis of the section, the enhancement in capacity may not be significant, for out-of-plane bending (unlike in-plane bending).

For example, providing a 10 mm dia bar (Fe 415 grade steel) at the centre of each cavity filled with M20 concrete and designing as a reinforced concrete beam section as per IS 456: 2000.

Effective depth, d = (124 – 2 × 15) / 2 = 47 mm

A_{st} = (78.5 × 1000) / 250 = 314 mm^2/m

Neutral axis depth, x_u, assuming the section to be under reinforced

= (0.87 × 415 × 314) / (0.36 × 20 × 1000) = 15.7 mm

< 0.48 × 47 = 22.6 mm (under reinforced)

M_{ud} = 0.87 × 314 × 415 × (47 – 0.42 × 15.7)

= 4.58 x 1000 Nm/m

Equating the above value to the design bending moment in the panel due to wind load,

$$0.6(V_z)^2 \times 3^2 \times (1/8) \times 1.5 \qquad = 4.58 \times 1000 \text{ Nm/m}$$

$$\Rightarrow V_z \quad = 67.3 \text{ m/s.}$$

In the case of tall buildings, special care shall be taken to ensure that the panels have the desired flexural strength to resist the prescribed design wind speeds (as per IS 875 part 3), including the enhancement in wind speed with height.

9. SHEAR STRENGTH

The unit shear strength capacity of the 124 mm thick, 3.0 m high GFRG panel is given in Table 9.1. The ultimate design shear strength of a GFRG panel is given by the unit shear capacity in Table 9.1 multiplied by the length of the panel[9,11,14].

Table 9.1 Shear Strength of GFRG Panel as Vertical Walls

Application	Design Shear Capacity, V_{ud} (kN/m)
Unfilled GFRG panel	14.4
GFRG panel filled with 20 MPa concrete	40.0
GFRG panel partially filled with 20 MPa concrete	$14.4 + 25.6\eta$ (where, η is defined as in Eq. 5.3)

In a multi-storeyed construction, using GFRG wall panels as load bearing construction, different walls will be subjected to different shear forces, at any storey level under consideration. Larger walls, which are stiffer, will attract more lateral shear. The maximum length of an individual shear wall segment may be limited to 3.5 m in the finite element model used for analysis under factored loads. The average value of factored shear force calculated for all walls in any one direction at any storey level shall not exceed the value indicated in Table 9.1. In few walls, some local increase (up to 20 percent) in shear capacity may be permited, provided the average value for all walls (combined) is within the prescribed limit. Double walls may be provided, if there is a higher demand for shear strength.

In wall construction for multi-storeyed buildings, all cavities should be filled with concrete (of grade not less than M20) and reinforced appropriately. The design of such reinforcement is discussed in the Chapter 10. The rebars shall be provided for the full height in filled GFRG panels. In any case, for both filled and unfilled GFRG wall panels at the interface with foundation plinth beam, starter bars should be provided in each cell embedded in concrete (of strength not less than 20 MPa) for a depth of 450 mm for adequate shear transfer.

10. IN-PLANE BENDING CAPACITY

10.1 Introduction

GFRG panels can be used not only as load bearing walls, but also as walls transferring lateral loads, resisting axial force (*P*), lateral in-plane shear force (*V*) and in-plane bending moment (*M*). Invariably, such wall panels shall be filled with concrete and reinforced with steel.

The in-plane bending capacity of the walls depends on its length, the reinforcement provided, as well as the level of axial load and lateral shear. The design in-plane bending capacity (M_{ud}) and its relationship with the design axial load capacity (P_{ud}) is usually described by means of a P_{ud} - M_{ud} interaction diagram. The values of M_{ud} increase with the length of the wall. However, experimental studies of GFRG panels subjected to lateral loading have shown that failure is initiated by vertical cracking caused by shear failure of the GFRG skin[9]. Following such vertical cracking, the wall segments separated by the vertical cracks tend to behave independently, although their deformations at the top and bottom are governed by the corresponding deformations in the connecting floor diaphragm. Hence, for all practical purposes, the in-plane bending capacity is limited by the corresponding shear capacity. Longer shear walls tend to attract larger lateral loads and will form vertical shear cracks in the middle region, causing a further redistribution of forces, and possible further vertical shear cracking.

Hence, under factored lateral loads (earthquake or wind), it is recommended that in the finite element model, the long walls are suitably segmented such that no segment exceeds 3.5 m in length. Also, while modelling, care should be taken to consider T, L and I shaped flanged sections as being made up of separate rectangular segments with no shear transfer between them.

Tests have shown that providing two vertical bars in each cavity generates improved performance than a single bar[7]. At the end of this chapter, axial load – moment interaction diagrams (design charts), for various wall lengths, varying from 1.0 m to 3.5m with increments of 0.25 m, for various bar diameters (8 to 18 mm) of Fe 415 and Fe 500 grade steel, M20 and M25 grade concrete, are furnished for convenient use in the design office. The basis for generating these design charts is explained in Section 10.2.

10.2 Basic Design Procedure for P_{ud} - M_{ud} Interaction Diagram Generation

Generation of the interaction diagram of a typical GFRG building panel is based on a simplified procedure, which is a modified version of the 'lower bound solution', originally proposed by Wu (2009)[5]. Certain assumptions are made to develop the approximate interaction curve from the principles of mechanics.

The cross section of a typical GFRG panel infilled with concrete and reinforcement bars in each cell is shown in Fig. 10.1a. The behaviour of the GFRG panel infilled with concrete depends on the bond between the concrete and the GFRG panel. This is reflected in the variation of normal strain (in the vertical direction) along the length of the wall, as shown in Fig. 10.1. If there is no bond, there would not be any interaction between them, resulting in small strain with multiple neutral axes, as shown in Fig. 10.1b. If it is assumed that the concrete cores are fully bonded to the GFRG panel, then the "plane section remain plane" assumption is valid for the entire section and the strain profile will be a straight line with a single neutral axis, as shown in Fig. 10.1c. This behaviour is similar to a reinforced concrete flexural wall. However, the limited bond between the concrete cores and the GFRG panel is difficult to quantify. The probable strain profile is likely to be as shown in Fig. 10.1d. A linear 'lower bound' assumption of strain profile can be assumed with the ultimate compressive strain (ε_{cu}), as shown in Fig. 10.1d. The value of ε_{cu} is limited by the out-of plane buckling strength of the panel and includes enhancement due to strain gradient for short wall lengths.

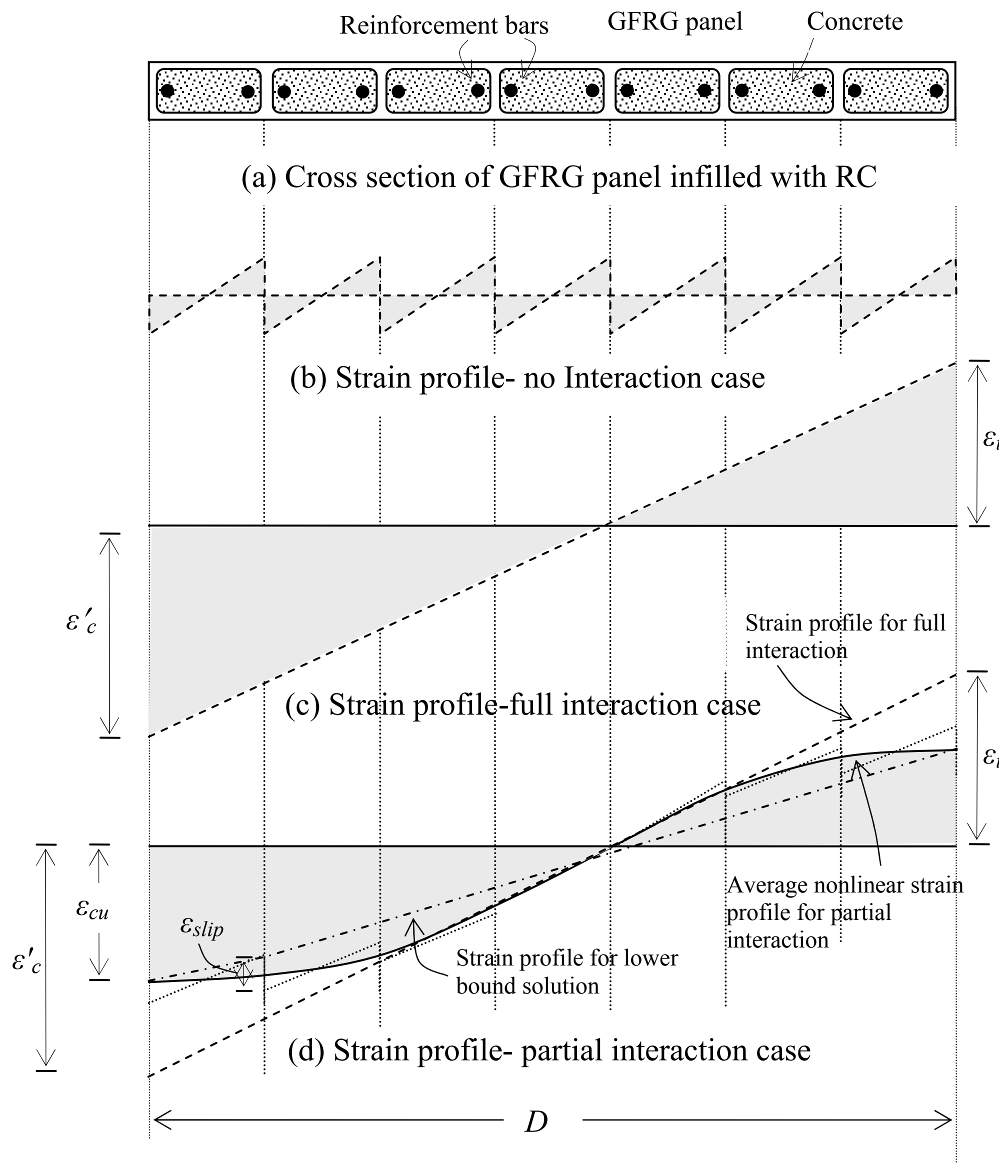


Fig. 10.1 Strain Profiles for Nil, Full and Partial Interaction between GFRG panel and Concrete

10.2 .1 Distribution of Strain at Ultimate limit state

Fig. 10.2 depicts how the value ε_{cu} is to be computed depending on the location of the neutral axis x_u (from the extreme compression location), which in turn depends on the eccentricity of loading, $e = M_{ud} / P_{ud}$.

***Case 1: Pure compression* ($x_u \to \infty$)**

Under pure compression, ($e \to 0,\ x_u \to \infty$), ε_{cu} is limited to:

$$\varepsilon_{c0} = \frac{p}{E} \tag{10.1}$$

where *p* is the compressive stress of the wall at the axial buckling load (to be taken as 1360/1.5 = 907 kN/m) and *E* is the effective Young's modulus of the composite wall given by

$$E = \frac{E_C A_C + E_G A_G}{A_C + A_G} \tag{10.2}$$

where A_C and A_G are the areas of the concrete and the GFRG panel, respectively; and $E_c = 5000\sqrt{f_{ck}}$ (in MPa) and E_G are the values of elastic modulus of concrete and the GFRG panel respectively. Typical values of *E* using M20 and M25 grades of concrete are 17860 MPa and 19500 MPa respectively.

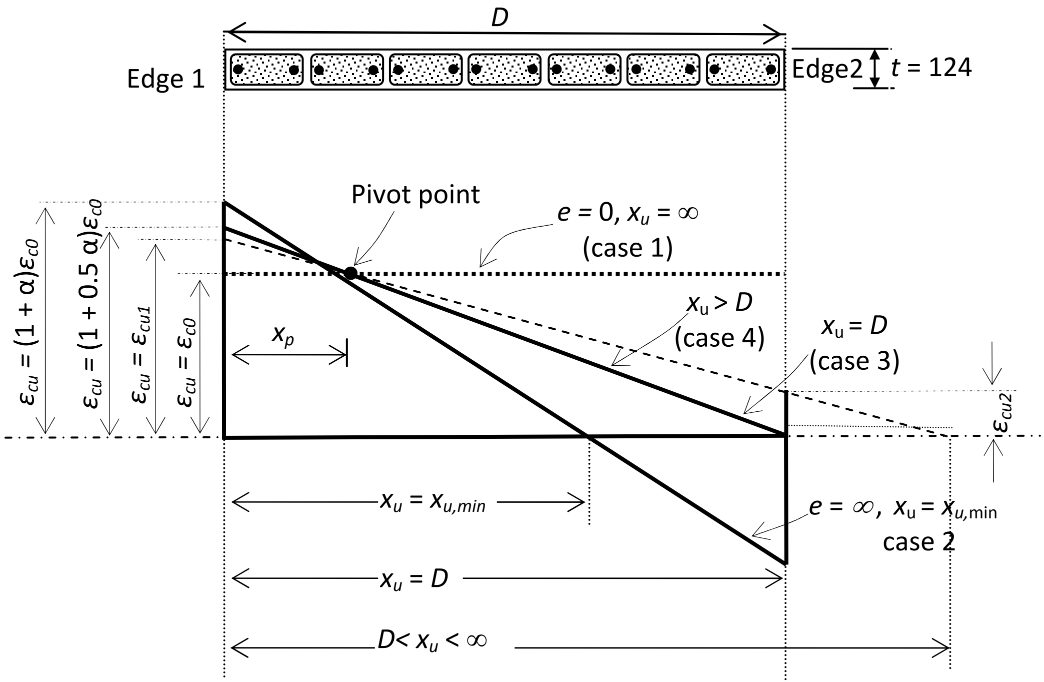


Fig. 10.2 Recommended strain profile for design

***Case 2: Pure bending* ($x_u = x_{u,\min}$)**

For the extreme case of pure in-plane bending ($e \to \infty$, $x_u = x_{u,\min}$), an enhancement over ε_{c0} is proposed as follows:

$$\varepsilon_{cu} = \varepsilon_{c0}(1+\alpha) \tag{10.3}$$

where,
$$\alpha = \begin{cases} 0.8(2-D) & : \text{ for } 1 \le D \le 2.0\,\text{m} \\ 0 & : \text{ for } D > 2.0\text{m} \end{cases} \tag{10.4}$$

***Case 3: Neutral axis at edge of section* ($x_u = D$)**

For the case, $x_u = D$, the value of ε_{cu} shall be taken as:

$$\varepsilon_{cu} = \varepsilon_{c0}\left(1+0.5\alpha\right) \tag{10.5}$$

Case 4: Neutral axis outside section **($x_u > D$)**

When the neutral axis lies outside the section $\left(x_u > D\right)$, the maximum compressive strain $\varepsilon_{cu} = \varepsilon_{cu1}$ can be calculated by linear interpolation. The point of intersection of the two limiting strain profiles, corresponding to $x_u = D$ and $x_u = \infty$, acts like a 'pivot' point through which all strain profiles pass when $x_u > D$, as shown in Figure 10.2. The values of the maximum compressive strain at edge 1, ε_{cu1}, and strain at edge 2, ε_{cu2} are accordingly given by:

$$\varepsilon_{cu1} = \frac{\varepsilon_{c0}}{(1-x_p/x_u)} \qquad \text{for } x_u > D \tag{10.6}$$

$$\varepsilon_{cu2} = \frac{\varepsilon_{cu1}(x_u - D)}{x_u} \qquad \text{for } x_u > D \tag{10.7}$$

where,

$$x_p = D\left(1 - \frac{1}{1+0.5\alpha}\right) \tag{10.8}$$

Case 4: Neutral axis inside section **($x_u < D$)**

When the neutral axis lies inside the section ($x_u < D$), linear interpolation shall be done to obtain the value of ε_{cu}:

$$\varepsilon_{cu} = \varepsilon_{c0}\left[1+0.5\alpha\left(1+\frac{D-x_u}{D-x_{u,\min}}\right)\right] \qquad \text{for } x_{u,\min} < x_u < D \tag{10.9}$$

10.2.2 Generation of Design Interaction Curve

A typical interaction diagram, with critical points marked, is shown in Fig.10.3. The design strength of a wall panel subject to in-plane moment (M_{ud}) and axial load (P_{ud}), with eccentricity, $e = M_{ud}/P_{ud}$, comprises values of P_{ud} and M_{ud} (corresponding to $0 < e < \infty$), all of which can be described by a single curve, termed as design interaction curve. The analysis for design strength basically entails two conditions: strain compatibility and force equilibrium.

Point A: Pure compression: $e = 0$, $x_u \to \infty$

$P_{ud} = p\,D\,t$ kN/m; $\qquad M_{ud} = 0$ kN/m

where D is in m.

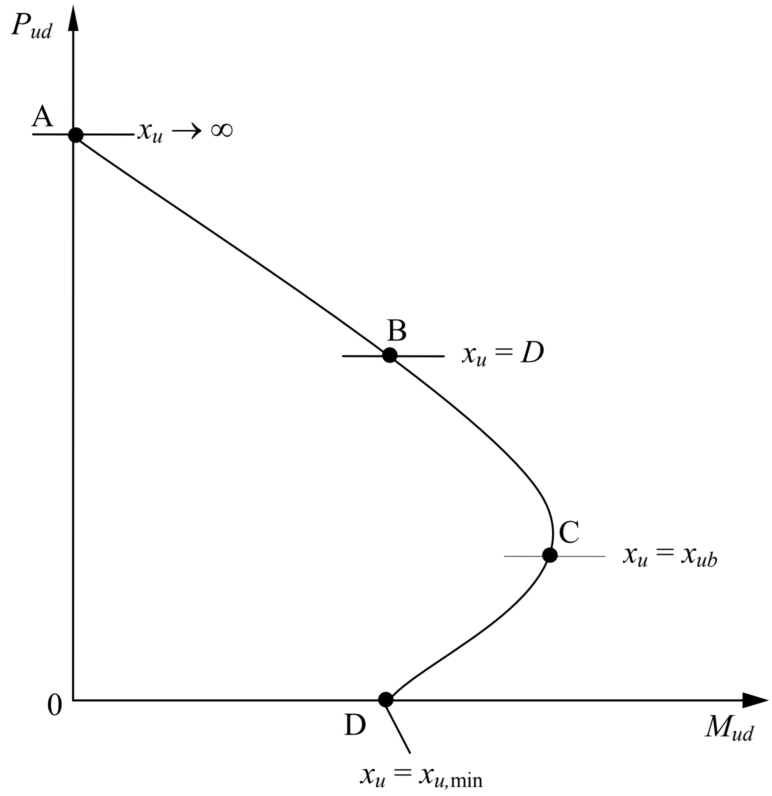


Fig.10.3 Typical P_u – M_u interaction curve

Between points A and B: $x_u > D$

$$P_{ud} = \left(\frac{\varepsilon_{cu1} + \varepsilon_{cu2}}{2} \right) E\, D\, t \tag{10.10}$$

$$M_{ud} = P\left(\frac{D}{2} - \overline{x}_1 \right) \tag{10.11}$$

where $\overline{x}_1$ is the centroid of the compressive force from the edge 1 given by

$$\text{where, } \overline{x}_1 = \frac{D}{3} \frac{(2\varepsilon_{cu2} + \varepsilon_{cu1})}{(\varepsilon_{cu1} + \varepsilon_{cu2})} \tag{10.12}$$

Between points B and C: $x_{ub} \le x_u < D$

where x_{ub} is the neutral axis depth corresponding to balancing point when the strain in the steel (ε_t) reaches its yield strain (ε_y). The corresponding strain and stress variation across the panel width is shown in Fig. 10.4

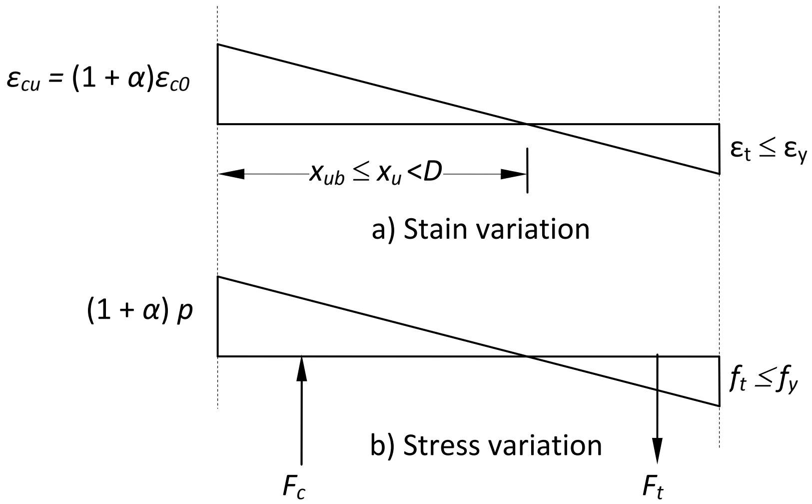


Fig. 10.4 Stress and strain variation across cross section

The resultant compressive force (F_c), the resultant tensile fore (F_t), the values of P_{ud} and M_{ud} can be calculated from the figure 9.3, as follows.

$$F_c = \frac{(1+\alpha)\,p\,t\,x_u}{2} \tag{10.13}$$

$$F_t = \varepsilon_t E_s a_s \frac{(D - x_u)}{2} \tag{10.14}$$

$$P_{ud} = F_c - F_t \tag{10.15}$$

$$M_{ud} = F_c\left(\frac{D}{2} - \frac{x_u}{3}\right) + F_t\left(\frac{D}{2} - \frac{D - x_u}{3}\right) \tag{10.16}$$

where a_s is area of reinforcement per unit lengh.

Between points C and D**:** $x_{u,\min} \le x_u < x_{ub}$

The corresponding strain and stress variation across the panel width is shown in Fig. 10.5.

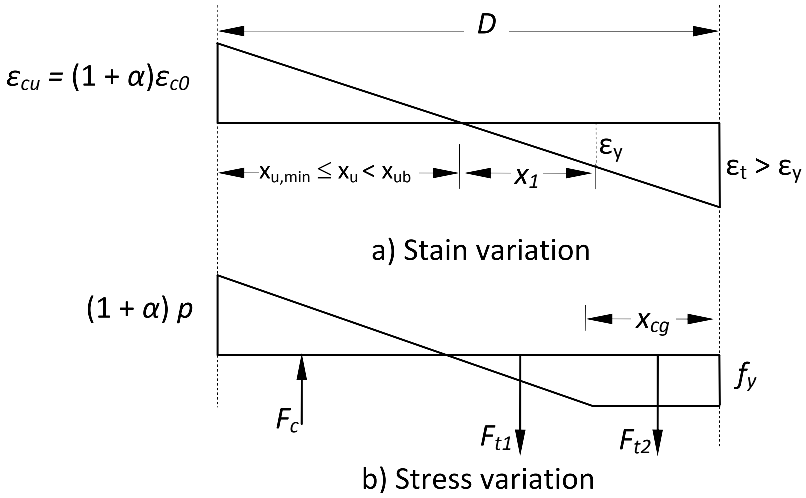


Fig. 10.5 Stress and strain variation across cross section after steel yields

The resultant compressive force (F_c), the resultand tesile force (F_t), and the values of P_{uc} and M_{uc} can be calculated from Fig. 10.5, as follows.

$$F_c = \frac{(1+\alpha)\, p\, t\, x_u}{2} \tag{9.16}$$

$$F_{t1} = 0.5 f_y\, a_s\, x_1 \quad (9.17$$

where,

$$x_1 = \frac{\varepsilon_y x_u}{\varepsilon_{cu}} \tag{9.18}$$

$$F_{t2} = f_y a_s \left(D - x_u - x_1\right) \tag{9.19}$$

$$F_t = F_{t1} + F_{t2} \tag{9.20}$$

$$P_{ud} = F_c - F_t \tag{9.21}$$

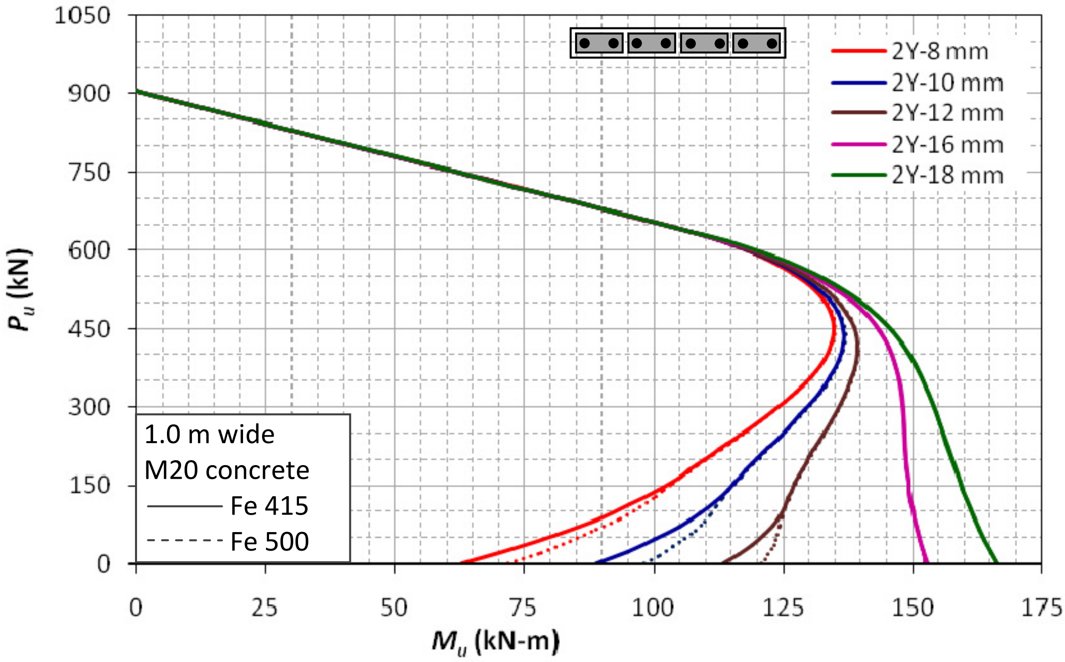
$$M_{ud} = F_c\left(\frac{D}{2} - \frac{x_u}{3}\right) + F_t\left(\frac{D}{2} - x_{cg}\right) \tag{9.22}$$

where x_{cg} is the distance from the right side of the panel, as shown in Fig. 10.5, through which the resultant of forces F_{t1} and F_{t2} acts.

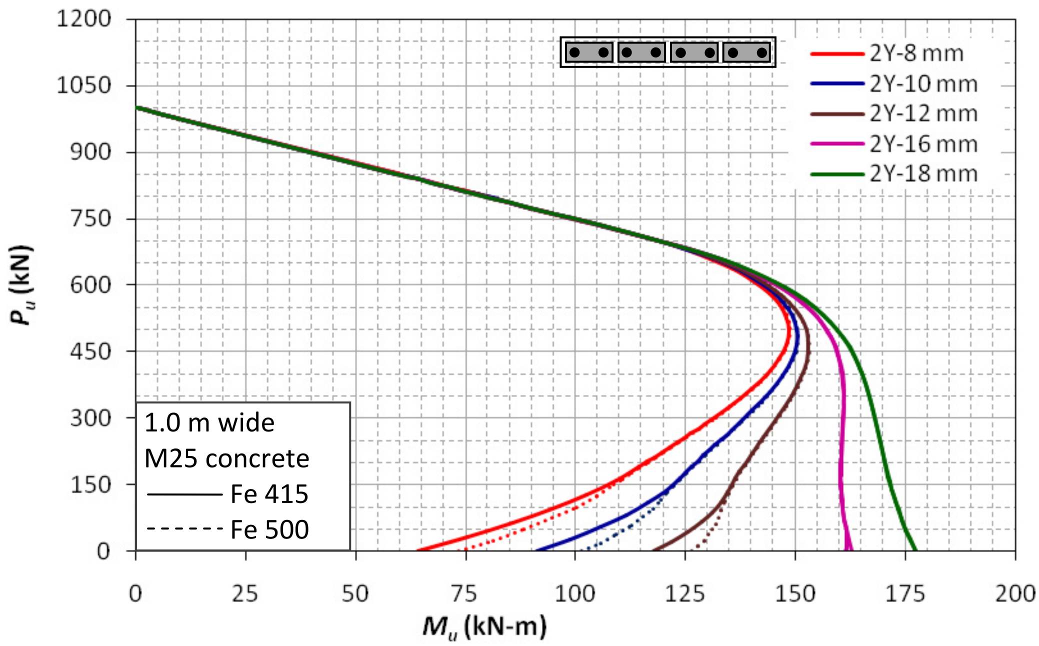
10.2.3 Design Charts

Design interaction curves of GFRG panels fully infilled with reinforced concrete (with two reinforcement bars in each cavity) for 1.0m to 3.5m with intervals of 0.25m, are shown in Figs 10.6 to 10.16.

For low/medium rise buildings, all cavities need not be infilled with reinforced concrete. In general, for low rise (up to 3 storeyed) GFRG houses/buildings, it is recommended that not more than three cavities may be provided in between the infilled ones, and those cavities may be suitably filled with concrete or alternative materials such as mixture of quarry dust and grout, sound insulating materials, etc. Since the lateral forces due to earthquake are lesser in low/medium rise buildings, the infilled cavities may be reinforced with a single reinforcing bar, as per the design requirement. Design interaction curves of GFRG panels partially/fully infilled with concrete and reinforced with a single bar are given in Figs 10.17 to 10.40. The application of these curves are demonstrated in Chapter 14.

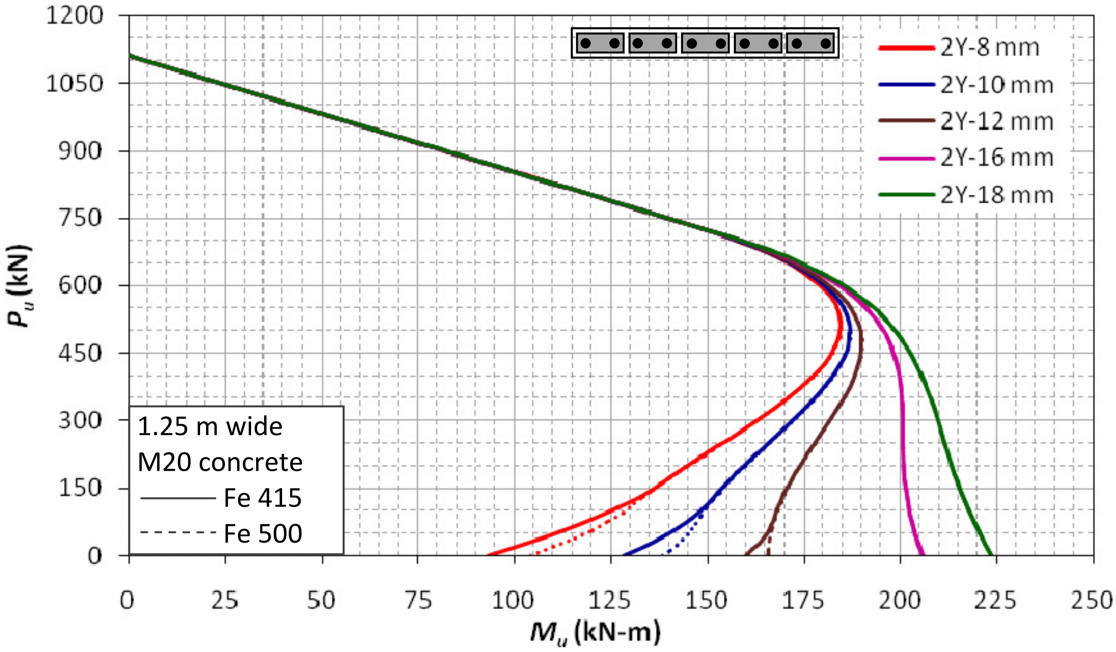


(a) using M20 concrete

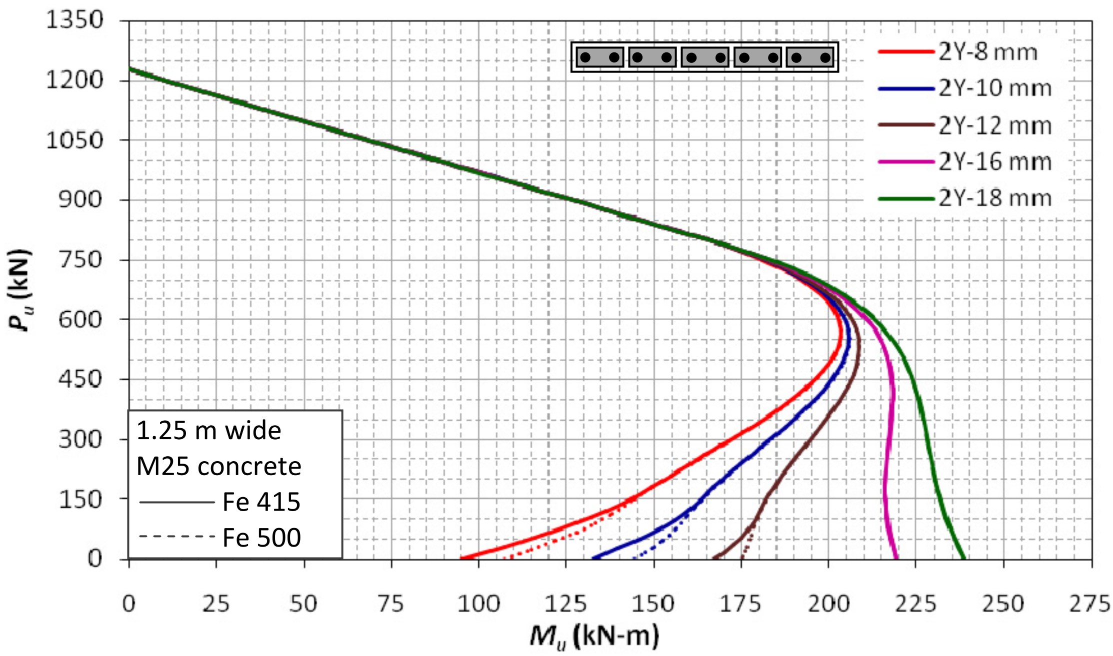


(b) using M25 concrete

Fig. 10.6 Design P_u-M_u plots for **1.0 m wide** GFRG panel with two bars in each cavity

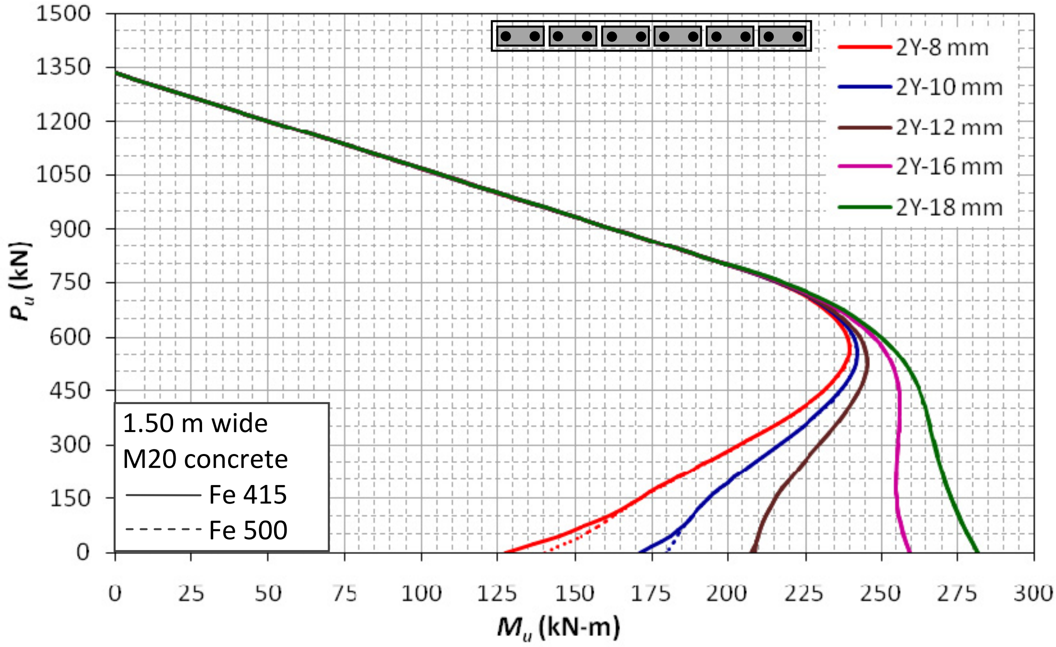


(a) using M20 concrete

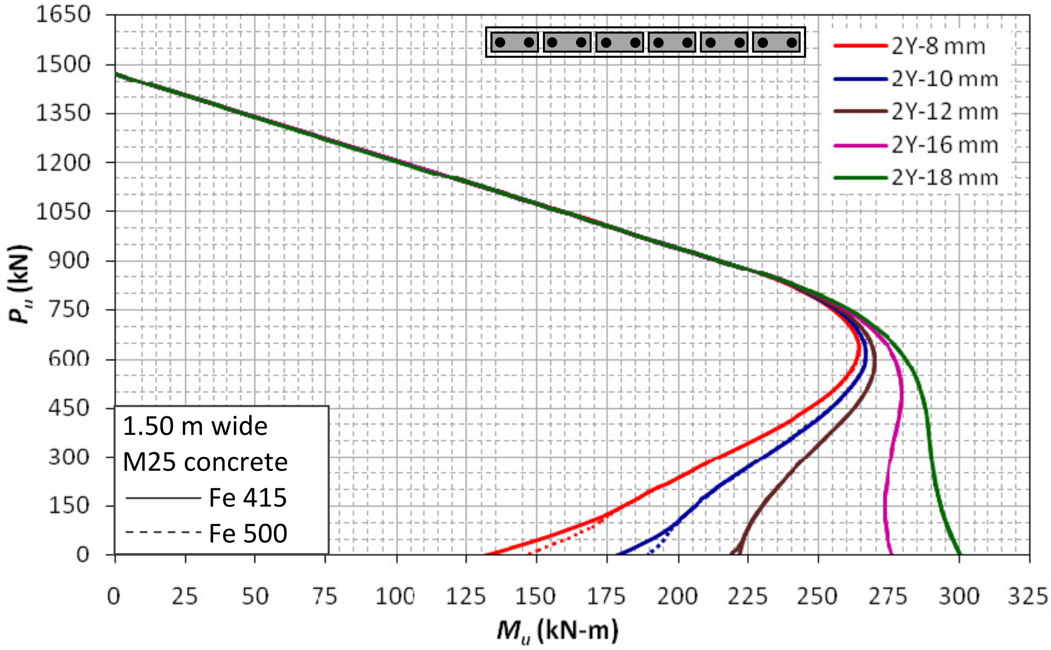


(b) using M25 concrete

Fig. 10.7 Design P_u-M_u plots for **1.25 m wide** GFRG panel with two bars in each cavity

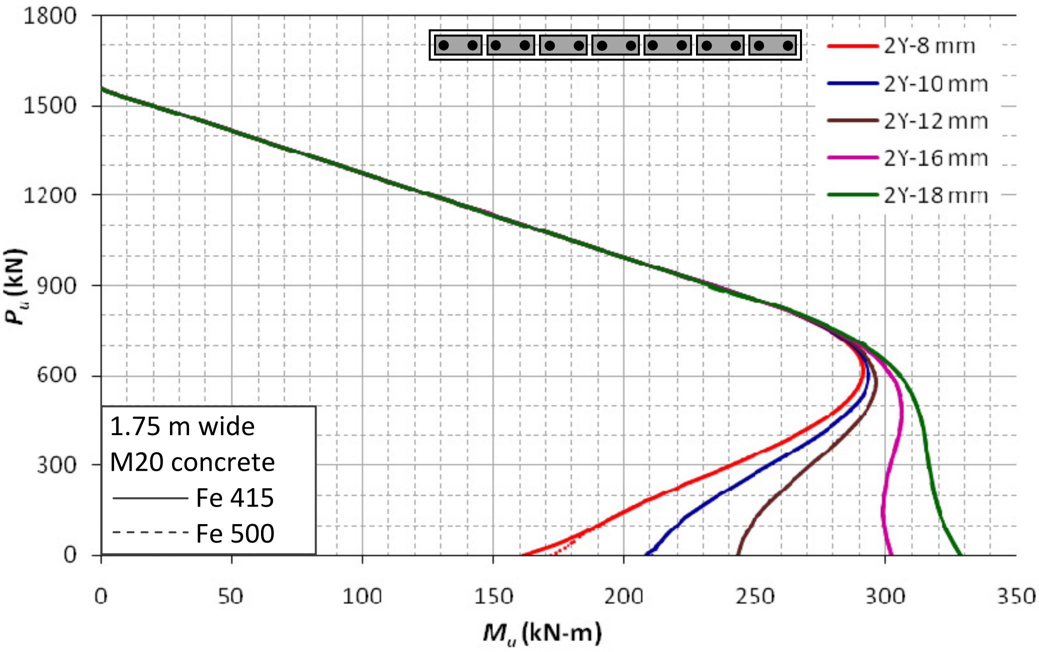


(a) using M20 concrete

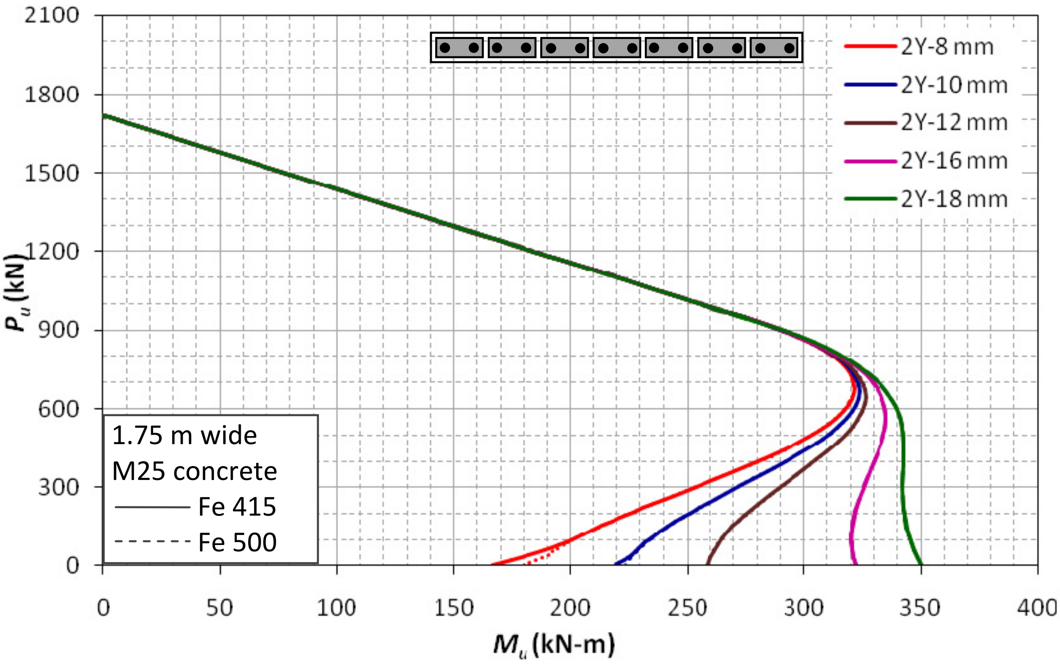


(b) using M25 concrete

Fig. 10.8 Design P_u-M_u plots for **1.50 m wide** GFRG panel with two bars in each cavity

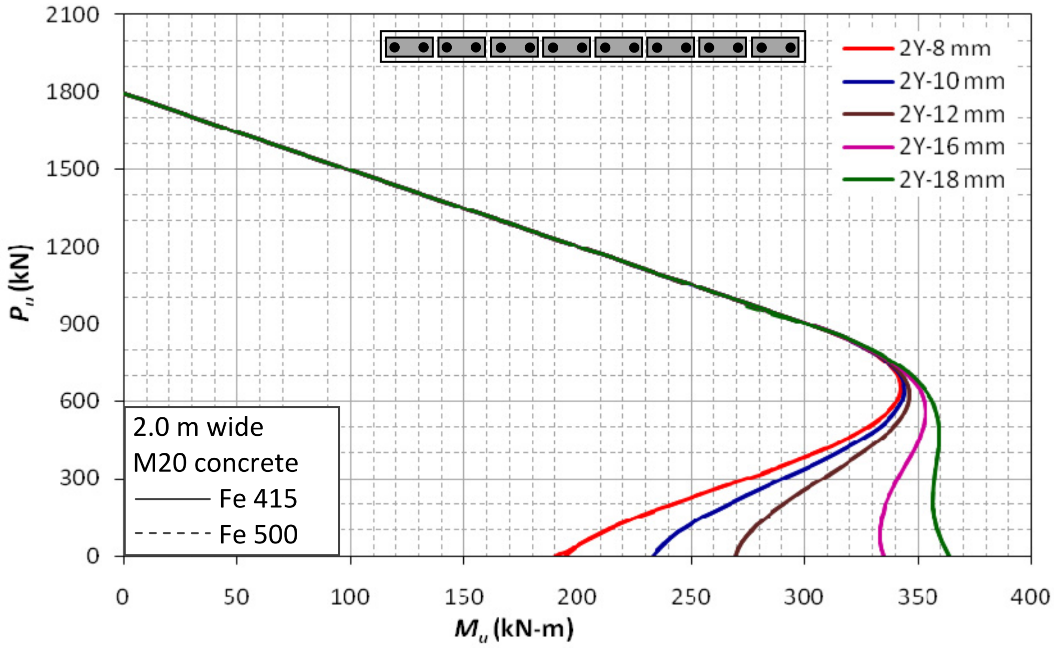


(a) using M20 concrete

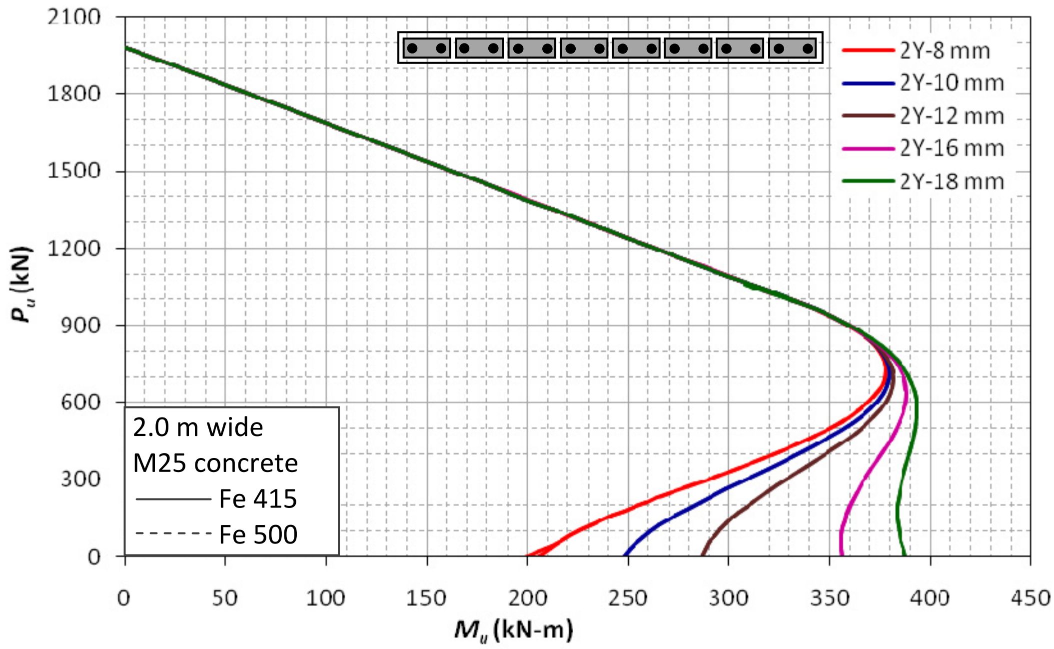


(b) using M25 concrete

Fig. 10.9 Design P_u-M_u plots for **1.75 m wide** GFRG panel with two bars in each cavity

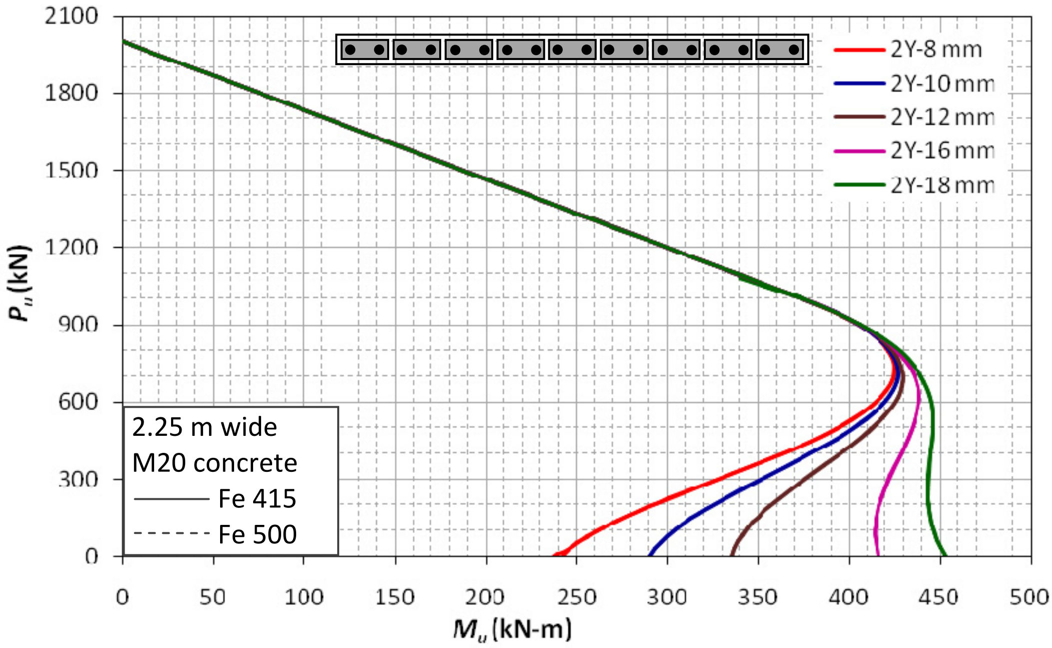


(a) using M20 concrete

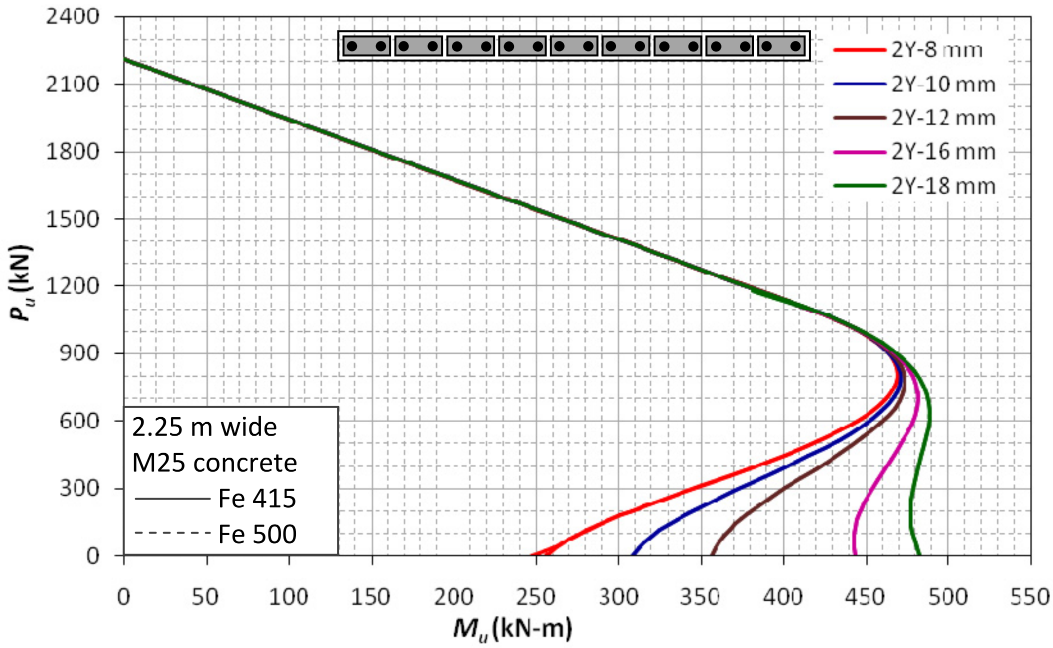


(b) using M25 concrete

Fig. 10.10 Design P_u-M_u plots for **2.0 m wide** GFRG panel with two bars in each cavity

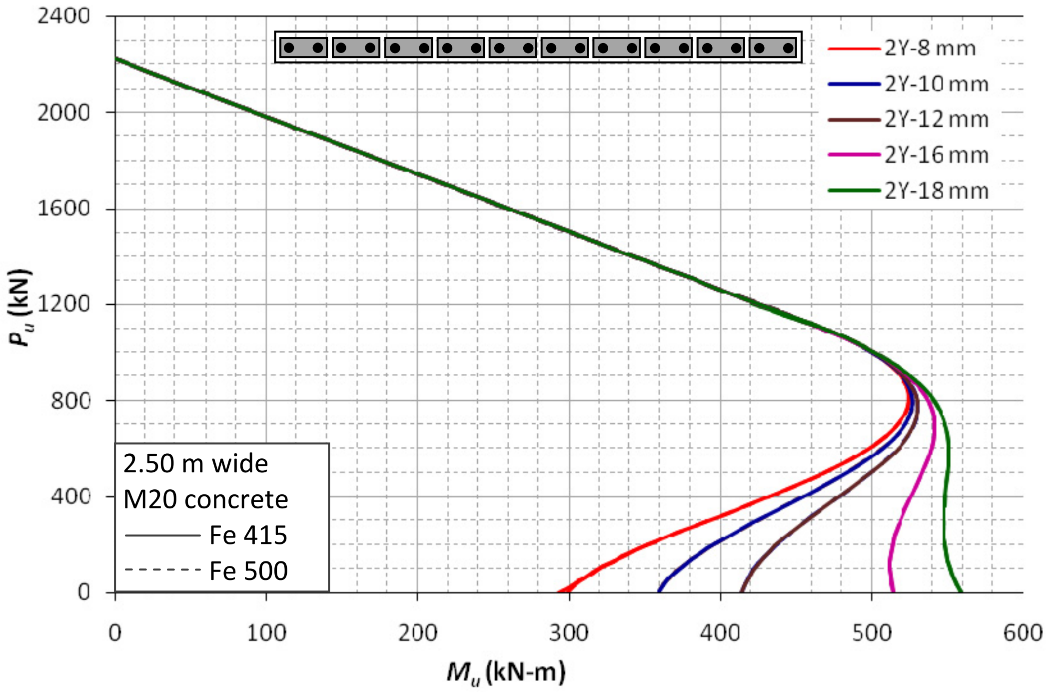


(a) using M20 concrete

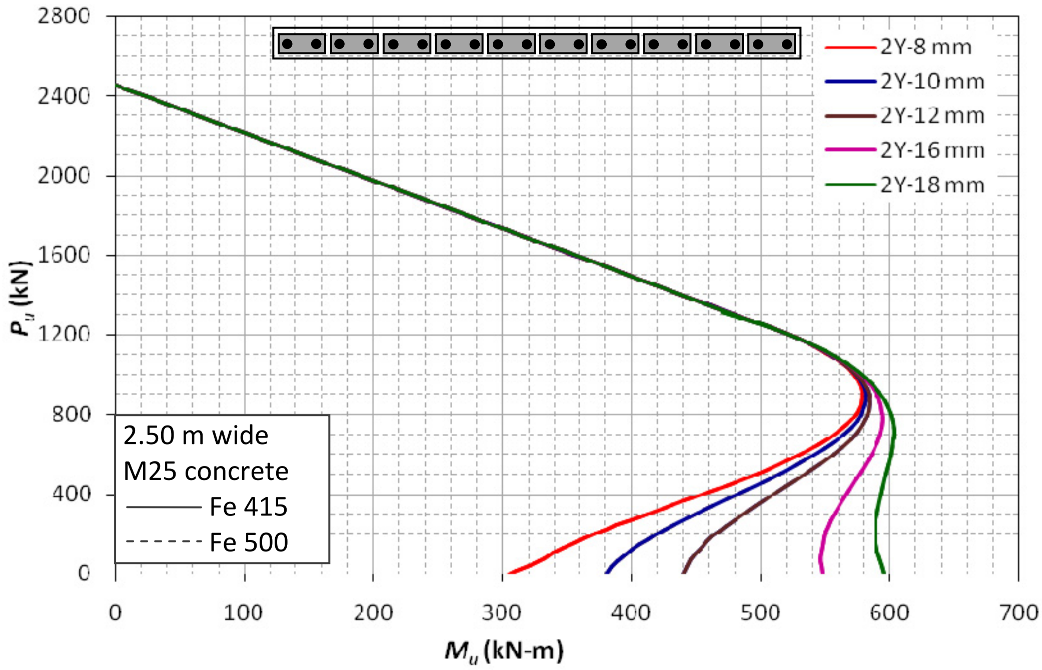


(b) using M25 concrete

Fig. 10.11 Design P_u-M_u plots for **2.25 m wide** GFRG panel with two bars in each cavity

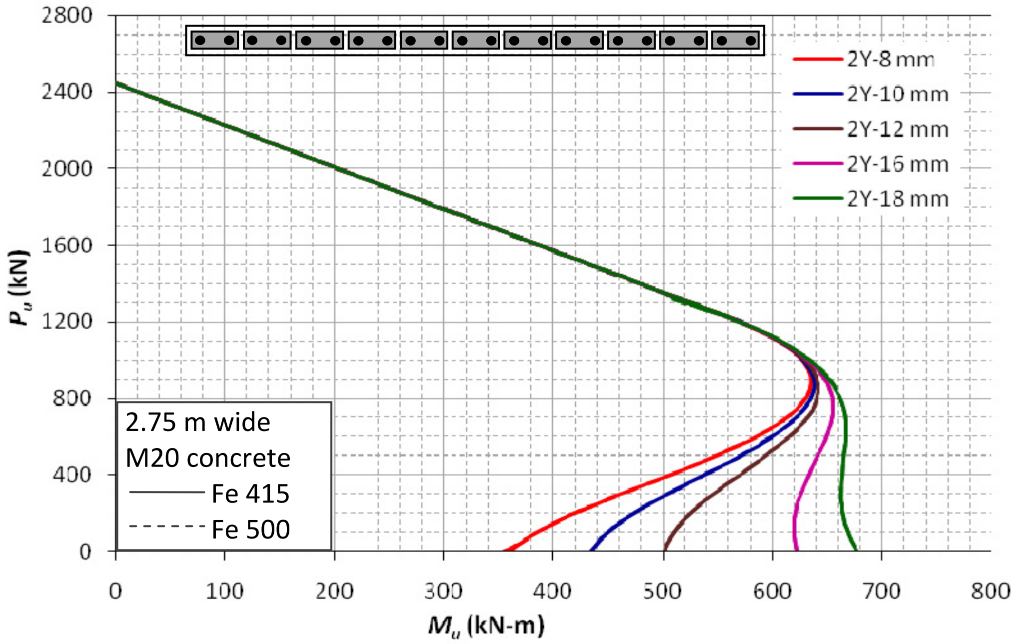


(a) using M20 concrete

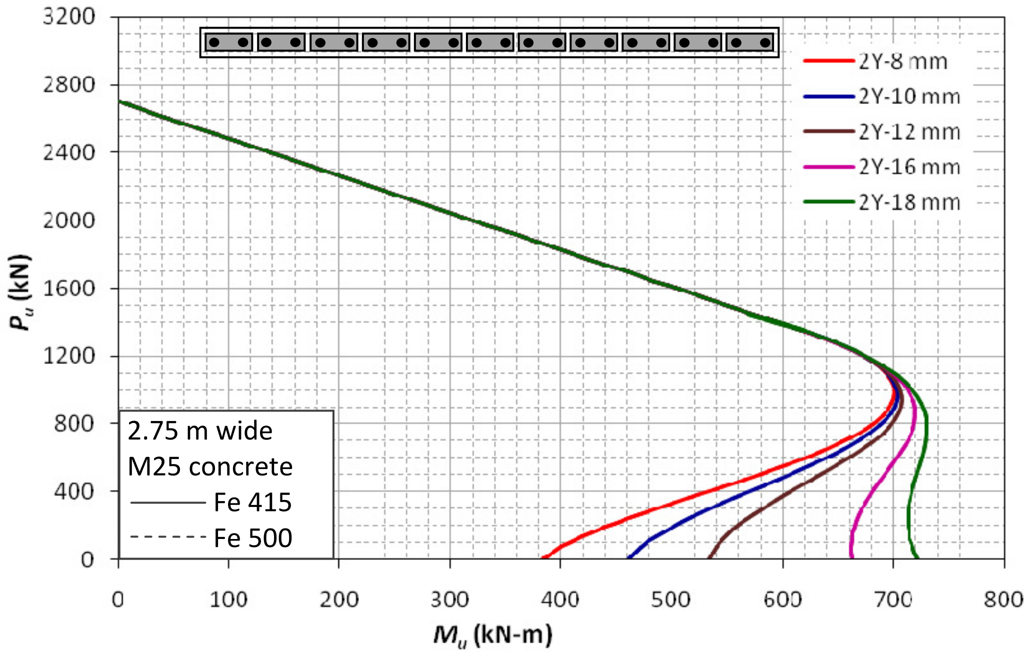


(b) using M25 concrete

Fig. 10.12 Design P_u-M_u plots for **2.50 m wide** GFRG panel with two bars in each cavity

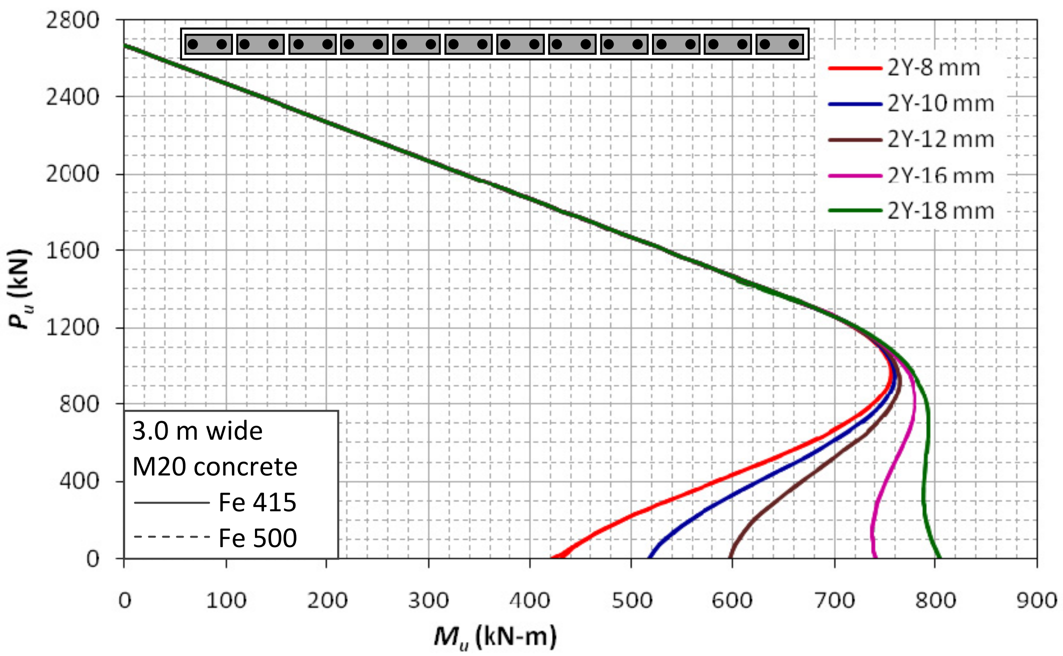


(a) using M20 concrete

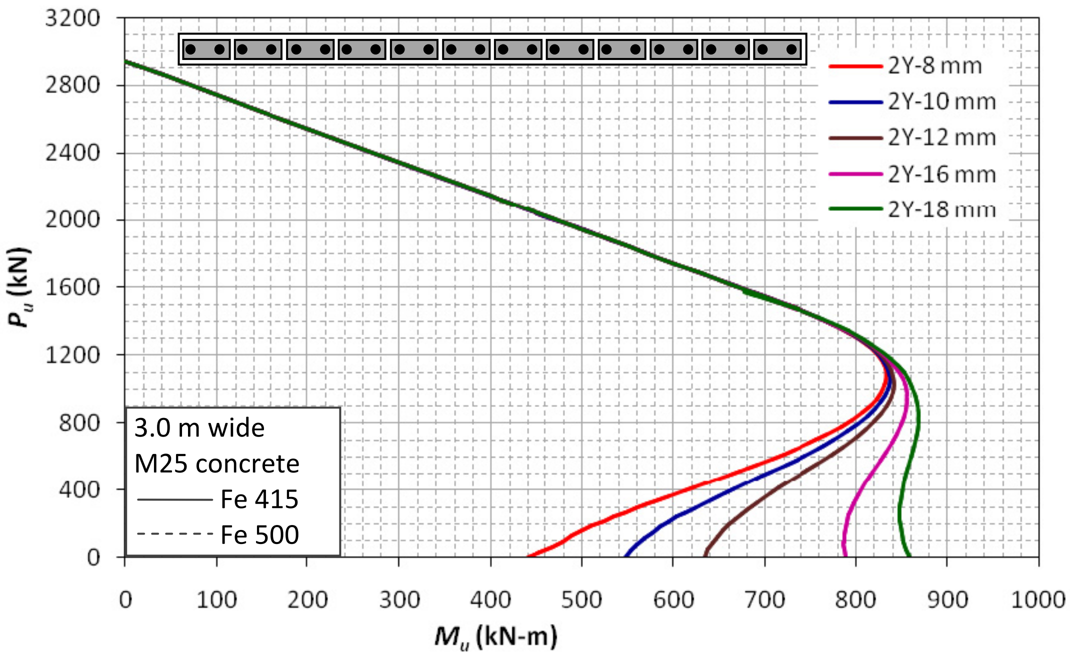


(b) using M25 concrete

Fig. 10.13 Design P_u-M_u plots for **2.75 m wide** GFRG panel with two bars in each cavity

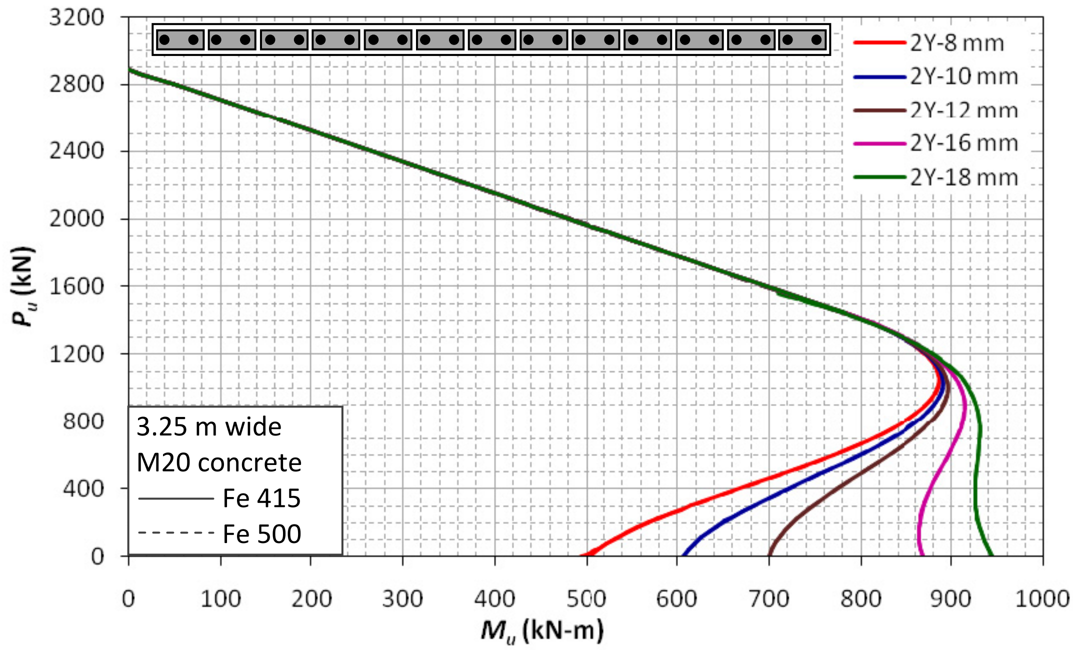


(a) using M20 concrete

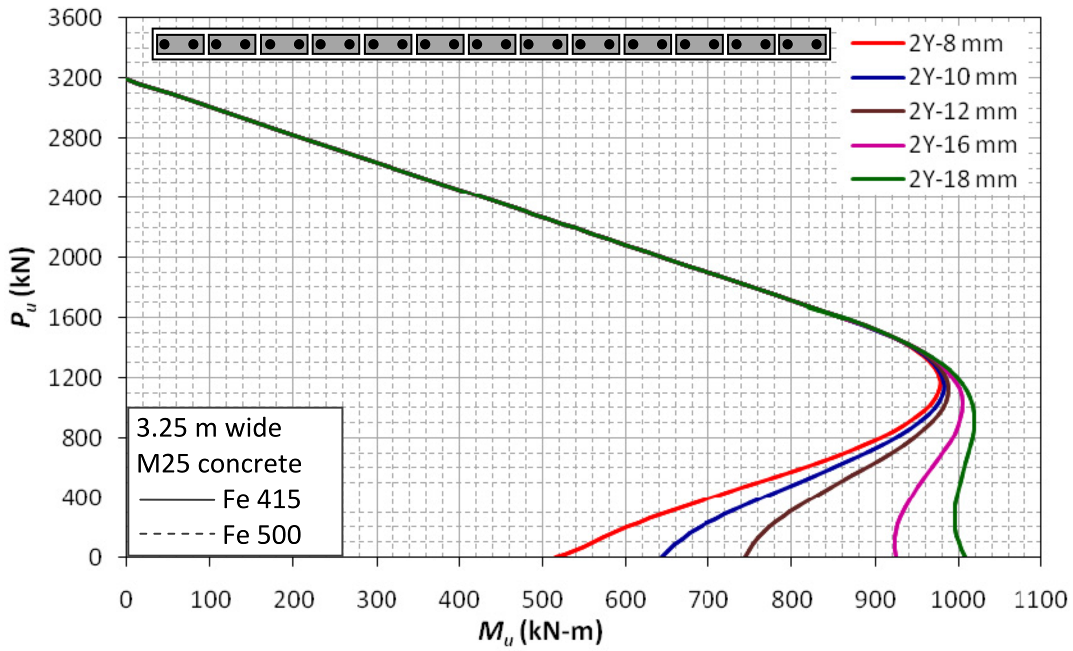


(b) using M25 concrete

Fig. 10.14 Design P_u-M_u plots for **3.0 m wide** GFRG panel with two bars in each cavity

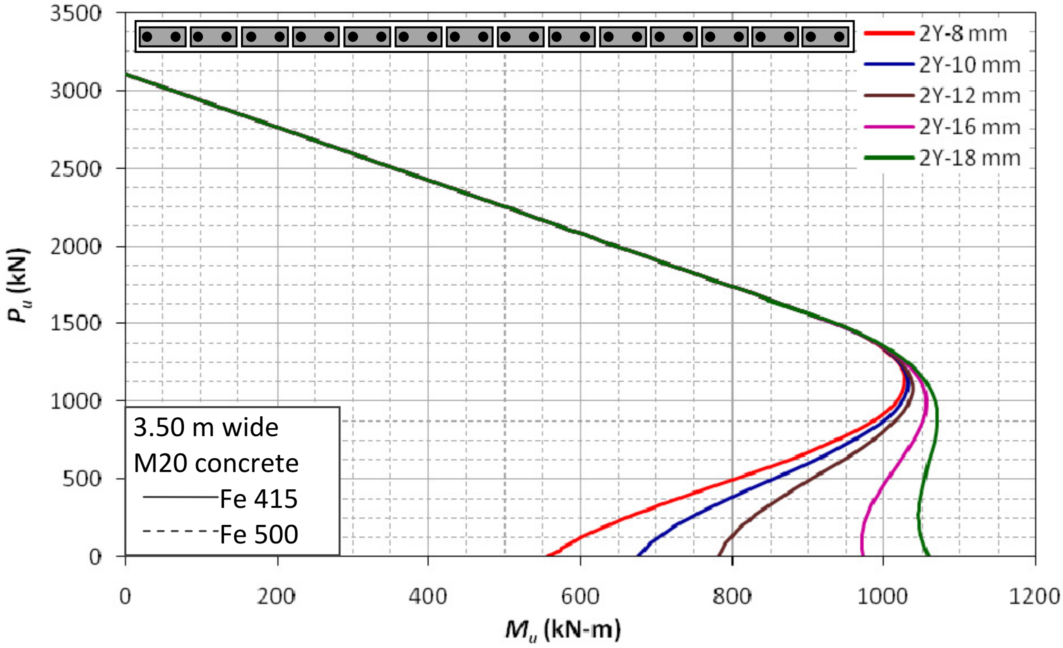


(a) using M20 concrete

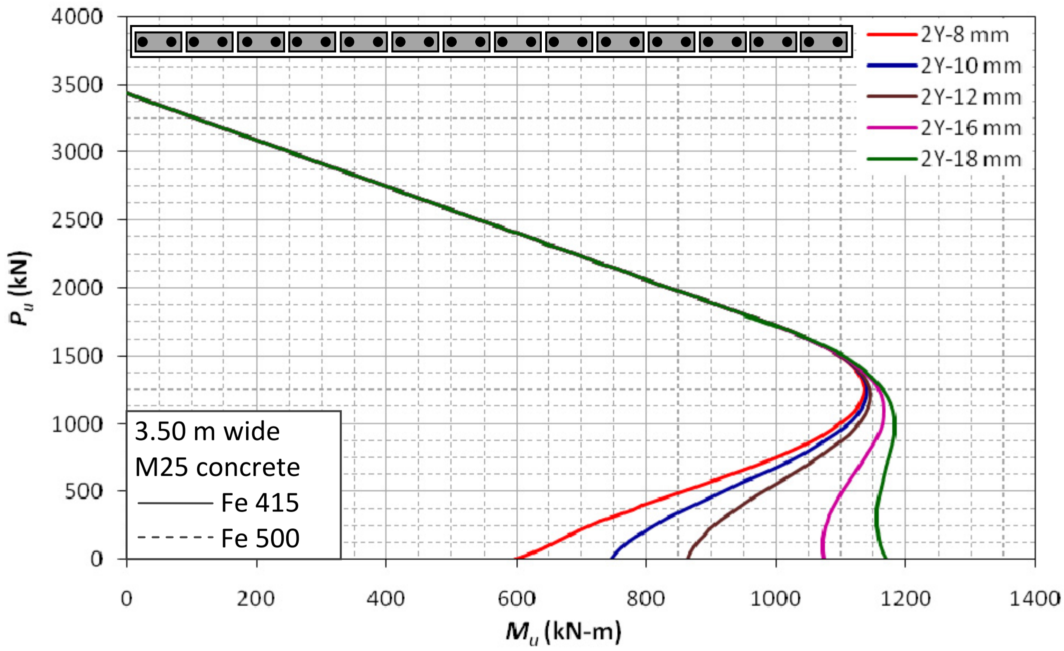


(b) using M25 concrete

Fig. 10.15 Design P_u-M_u plots for **3.25 m wide** GFRG panel with two bars in each cavity



(a) using M20 concrete



(b) using M25 concrete

Fig. 10.16 Design P_u-M_u plots for **3.50 m wide** GFRG panel with two bars in each cavity

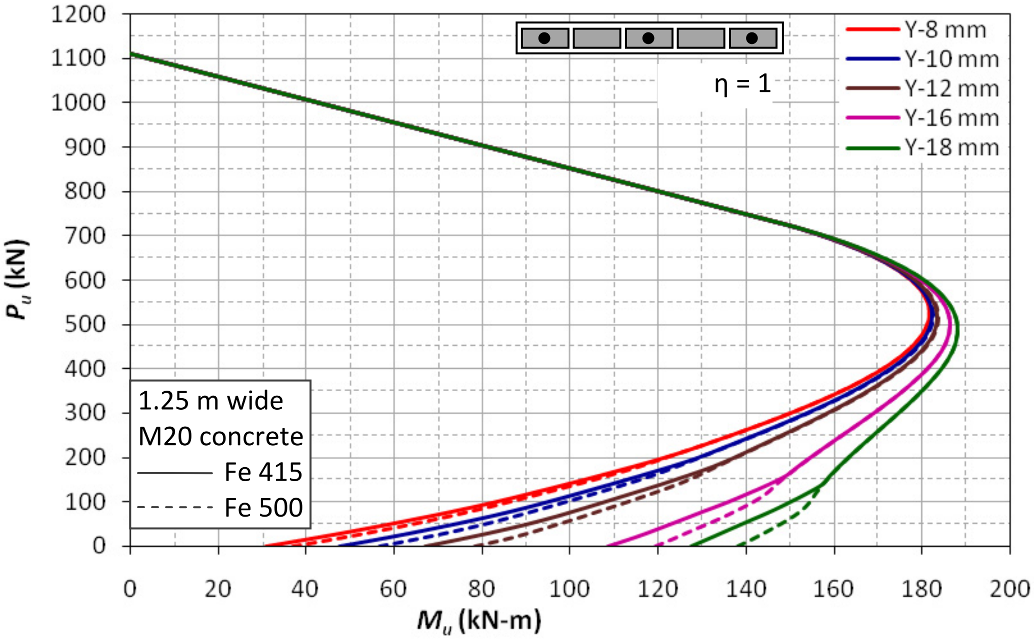


Fig. 10.17 Design P_u-M_u plots for **1.25 m wide** GFRG panel with all cavities infilled with M20 concrete and alternative cavities reinforced with single bar

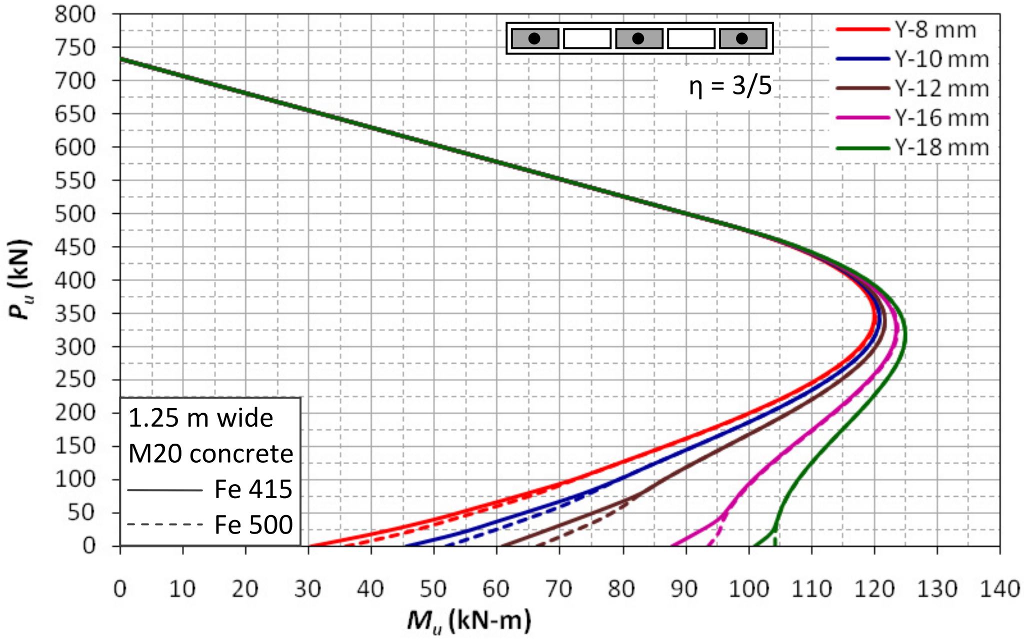


Fig. 10.18 Design P_u-M_u plots for **1.25 m wide** GFRG panel with alternative cavities infilled with M20 concrete and reinforced with single bar

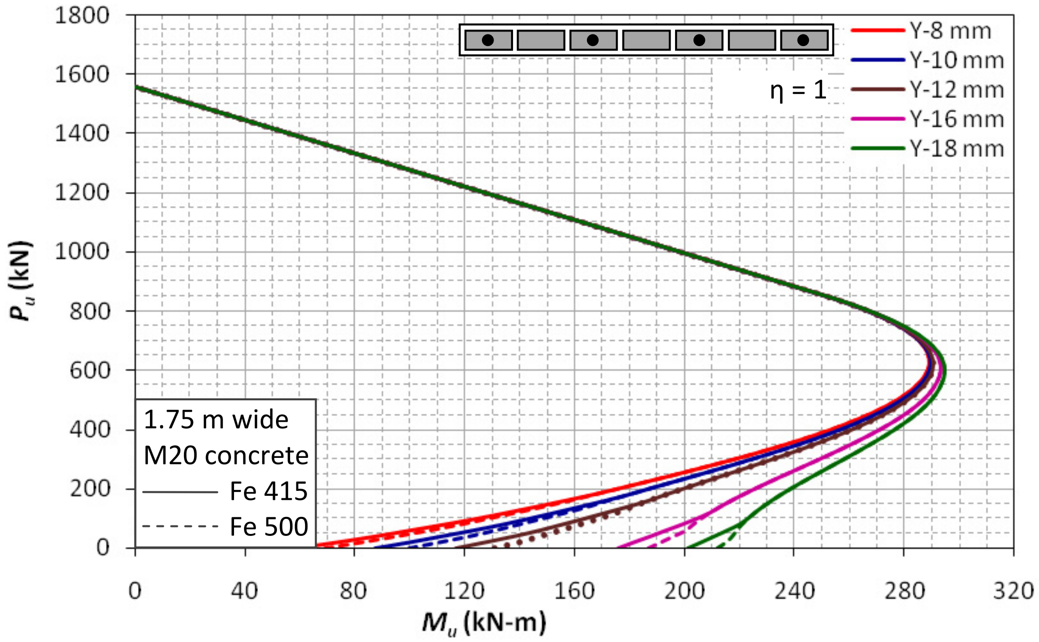


Fig. 10.19 Design P_u-M_u plots for **1.75 m wide** GFRG panel with all cavities infilled with M20 concrete and alternative cavities reinforced with single bar

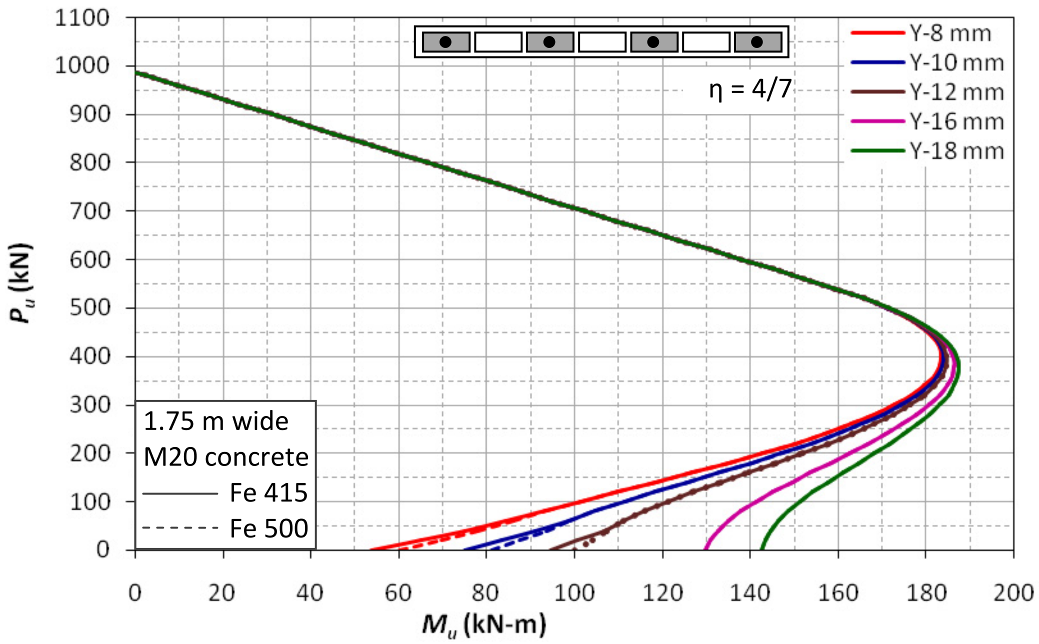


Fig. 10.20 Design P_u-M_u plots for **1.75 m wide** GFRG panel with alternative cavities infilled with M20 concrete and reinforced with single bar

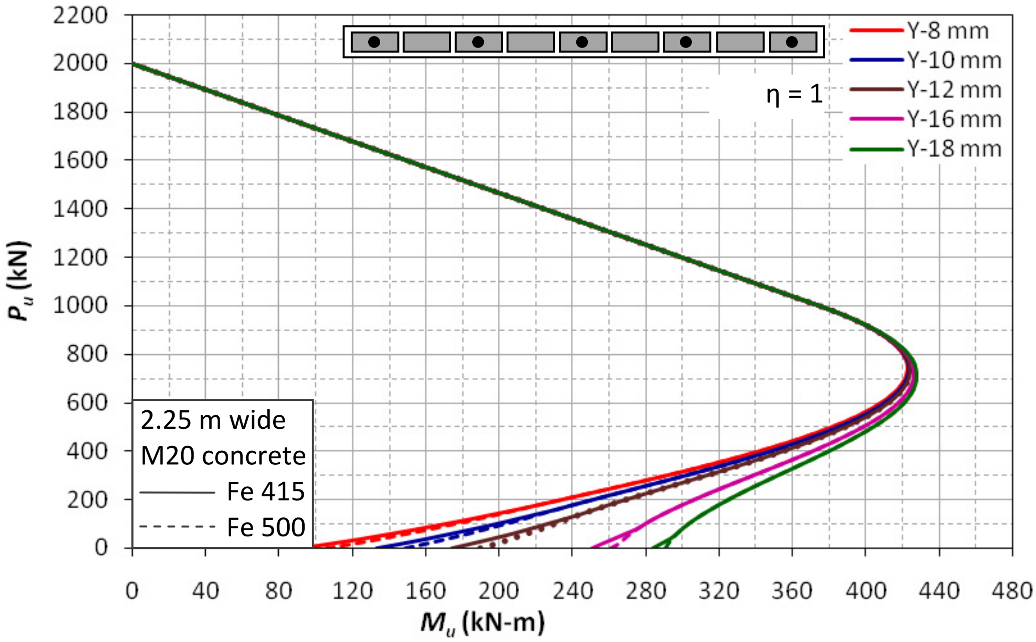


Fig. 10.21 Design P_u-M_u plots for **2.25 m wide** GFRG panel with all cavities infilled with M20 concrete and alternative cavities reinforced with single bar

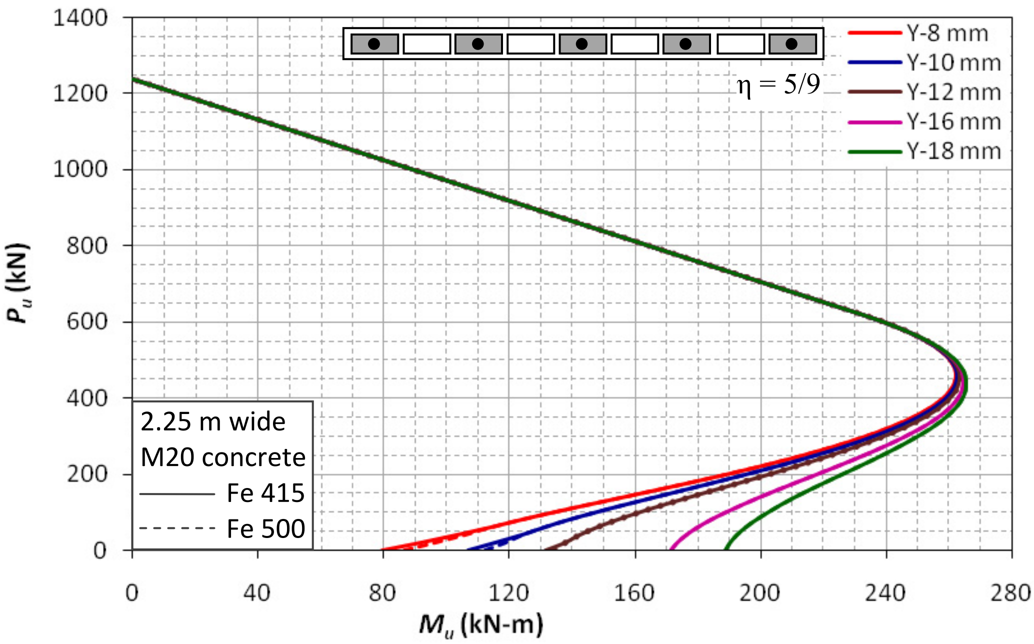


Fig. 10.22 Design P_u-M_u plots for **2.25 m wide** GFRG panel with alternative cavities infilled with M20 concrete and reinforced with single bar

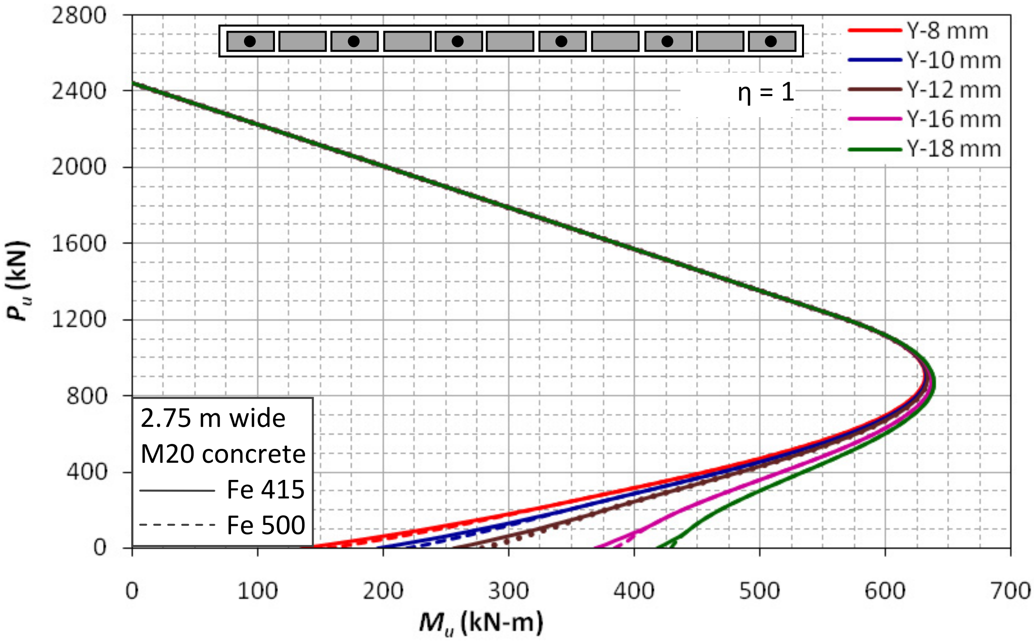


Fig. 10.23 Design P_u-M_u plots for **2.75 m wide** GFRG panel with all cavities infilled with M20 concrete and alternative cavities reinforced with single bar

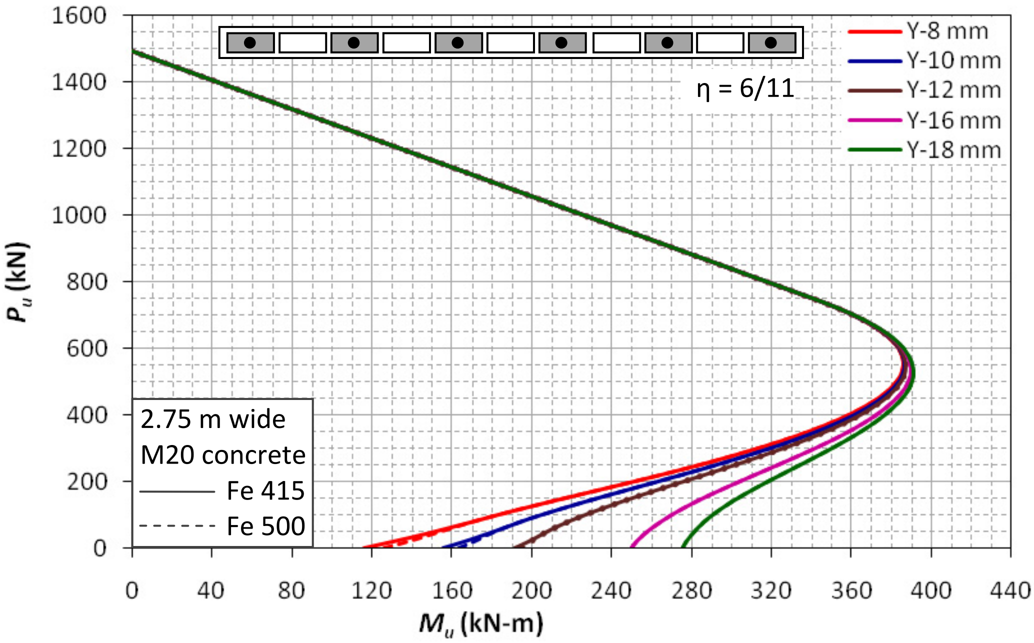


Fig. 10.24 Design P_u-M_u plots for **2.75 m wide** GFRG panel with alternative cavities infilled with M20 concrete and reinforced with single bar

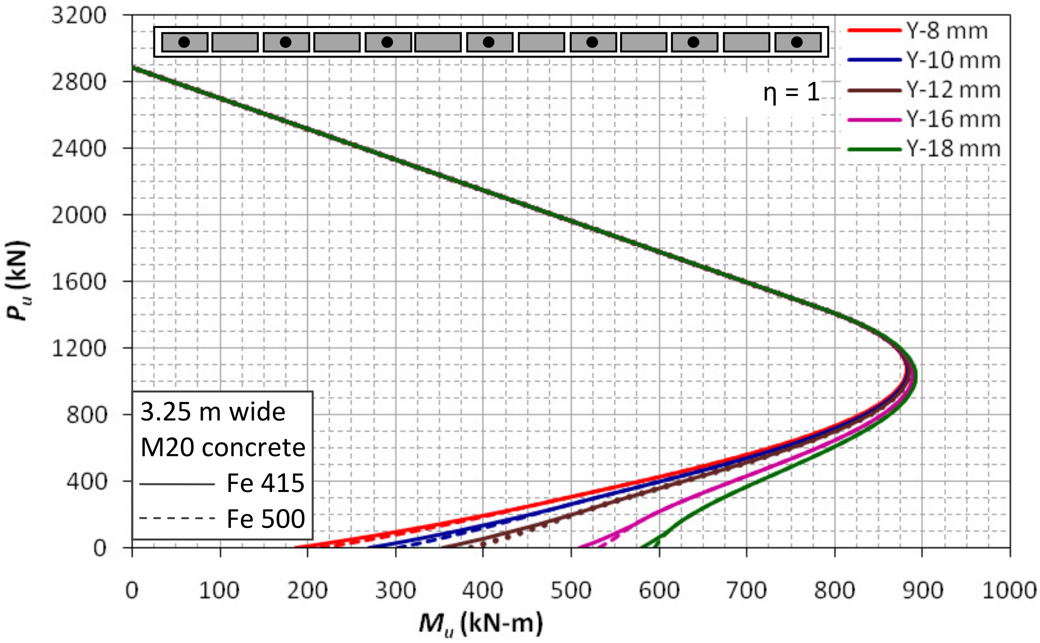


Fig. 10.25 Design P_u-M_u plots for **3.25 m wide** GFRG panel with all cavities infilled with M20 concrete and alternative cavities reinforced with single bar

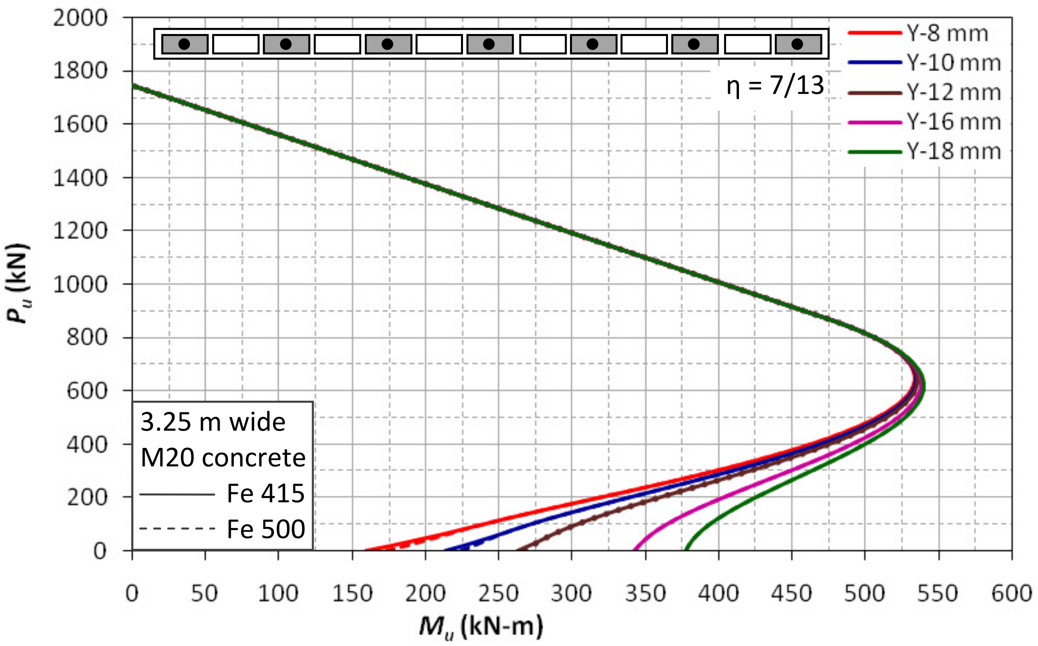


Fig. 10.26 Design P_u-M_u plots for **3.25 m wide** GFRG panel with alternative cavities infilled with M20 concrete and reinforced with single bar

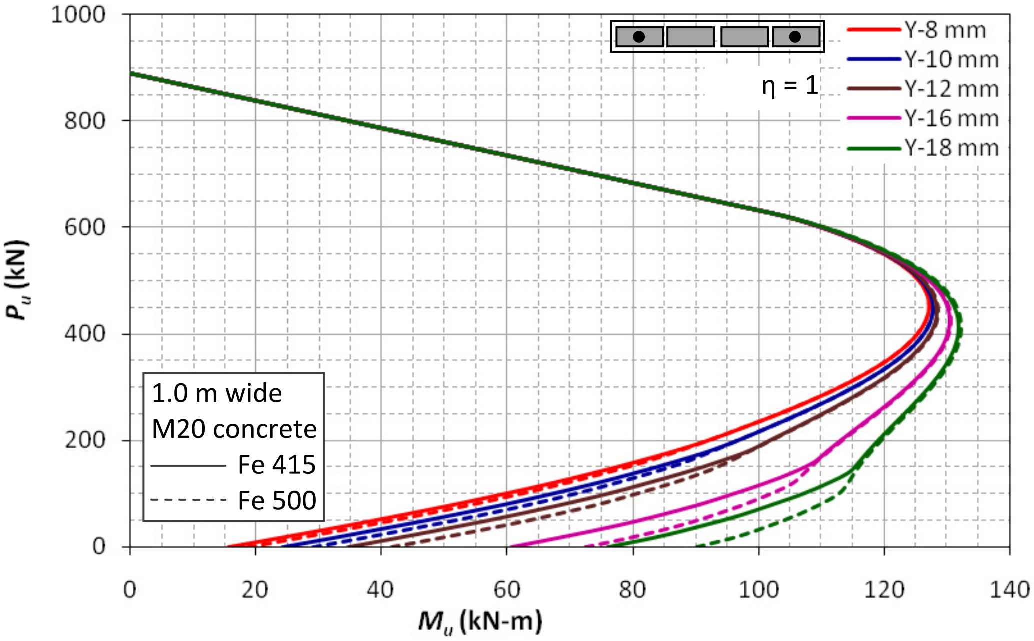


Fig. 10.27 Design P_u-M_u plots for **1.0 m wide** GFRG panel with all cavities infilled with M20 concrete and end cavities reinforced with single bar

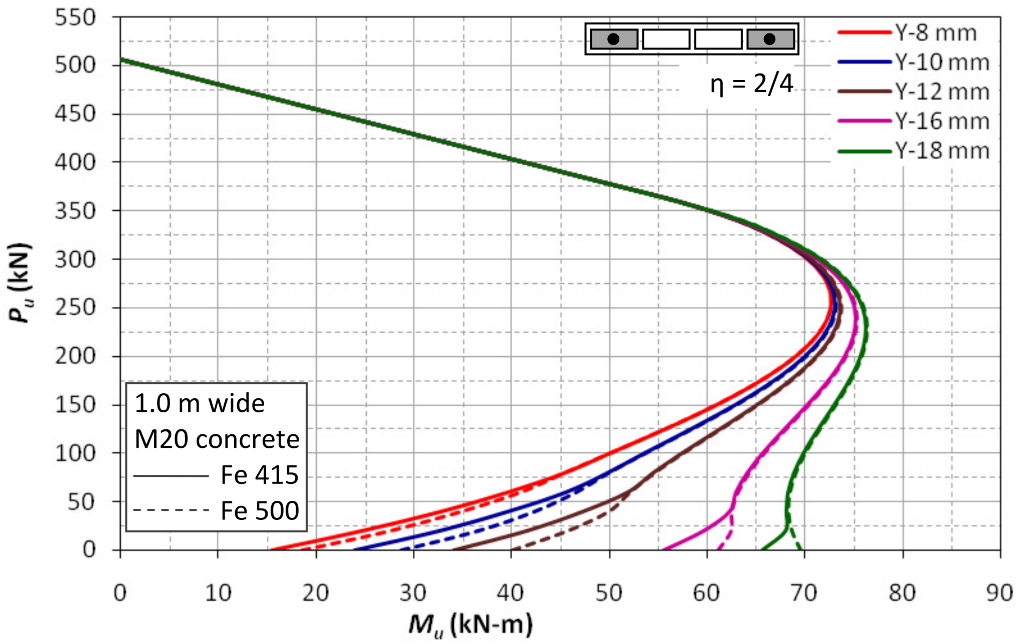


Fig. 10.28 Design P_u-M_u plots for **1.0 m wide** GFRG panel with end cavities only infilled with M20 concrete and reinforced with single bar

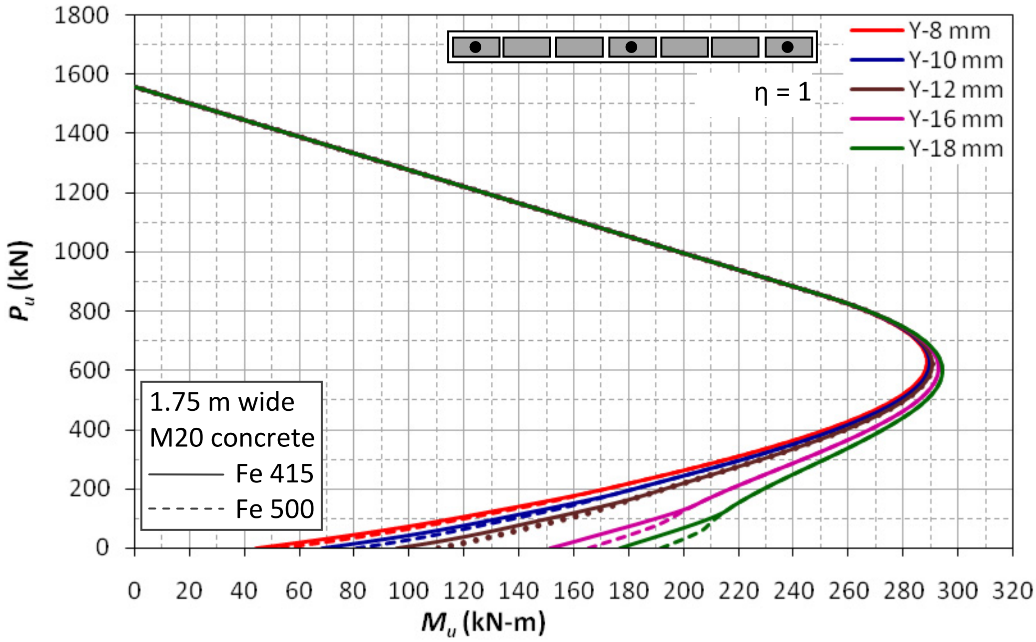


Fig. 10.29 Design P_u-M_u plots for **1.75 m wide** GFRG panel with all cavities infilled with M20 concrete and every third cavity reinforced with single bar

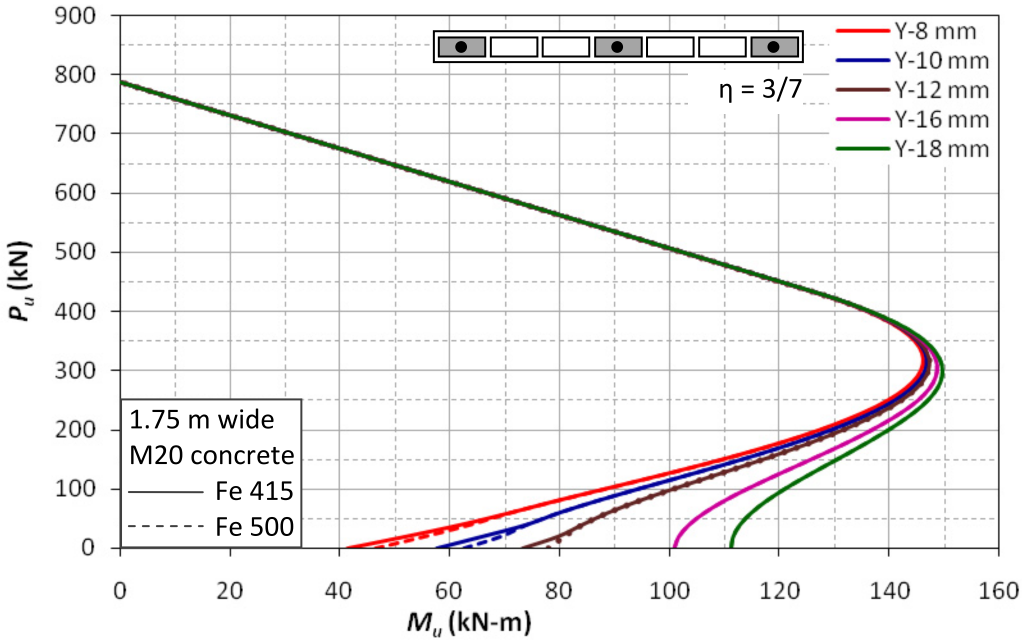


Fig. 10.30 Design P_u-M_u plots for **1.75 m wide** GFRG panel with every third cavity infilled with M20 concrete and reinforced with single bar

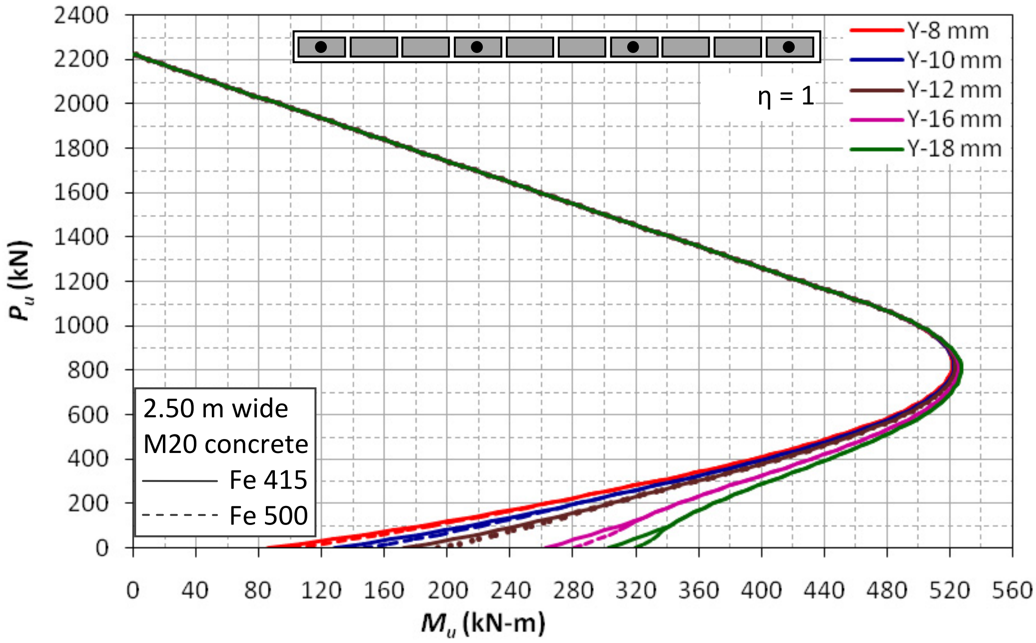


Fig. 10.31 Design P_u-M_u plots for **2.50 m wide** GFRG panel with all cavities infilled with M20 concrete and every third cavity reinforced with single bar

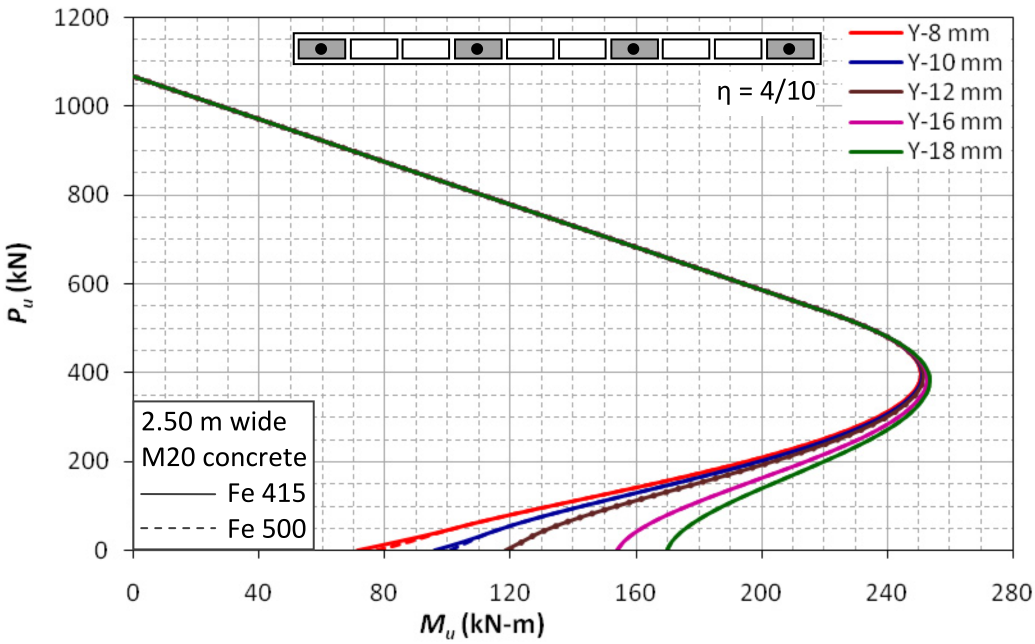


Fig. 10.32 Design P_u-M_u plots for **2.50 m wide** GFRG panel with every third cavity infilled with M20 concrete and reinforced with single bar

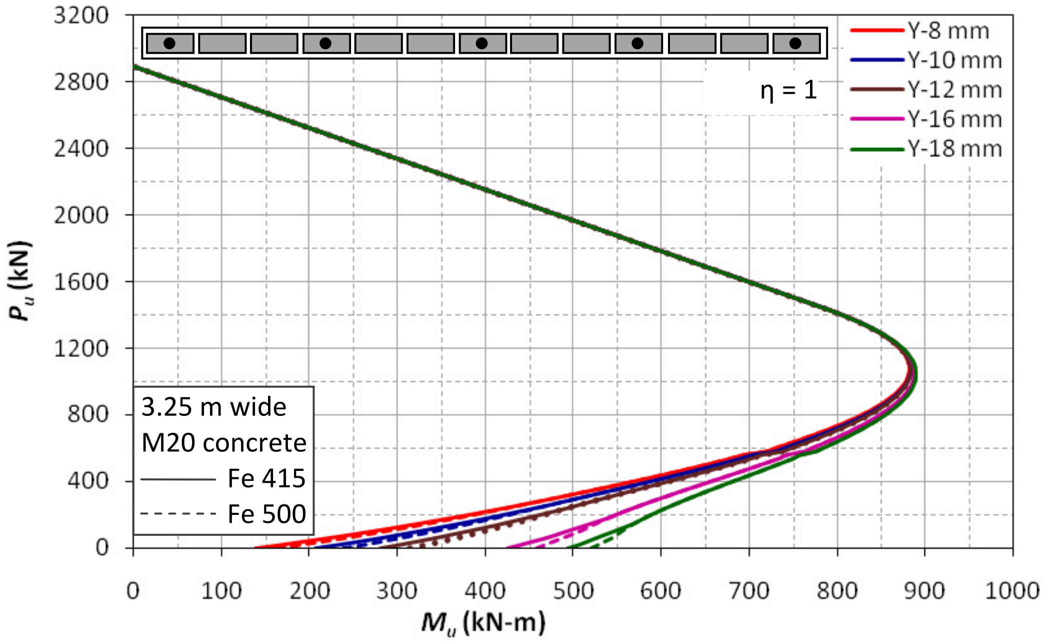


Fig. 10.33 Design P_u-M_u plots for **3.25 m wide** GFRG panel with all cavities infilled with M20 concrete and every third cavity reinforced with single bar

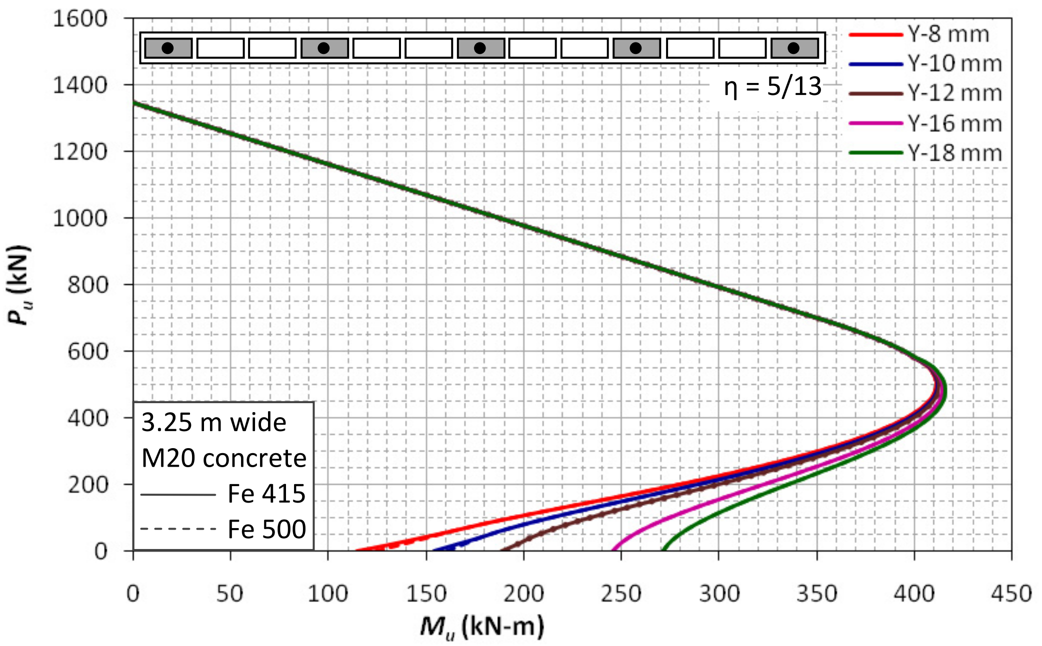


Fig. 10.34 Design P_u-M_u plots for **3.25 m wide** GFRG panel with every third cavities infilled with M20 concrete and reinforced with single bar

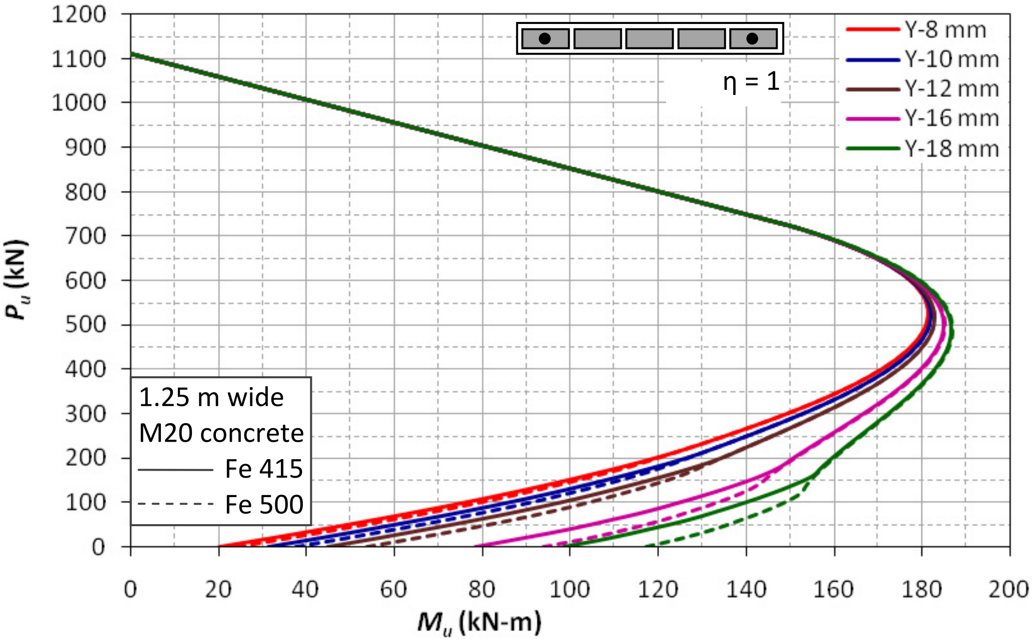


Fig. 10.35 Design P_u-M_u plots for **1.25 m wide** GFRG panel with all cavities infilled with M20 concrete and end cavities reinforced with single bar

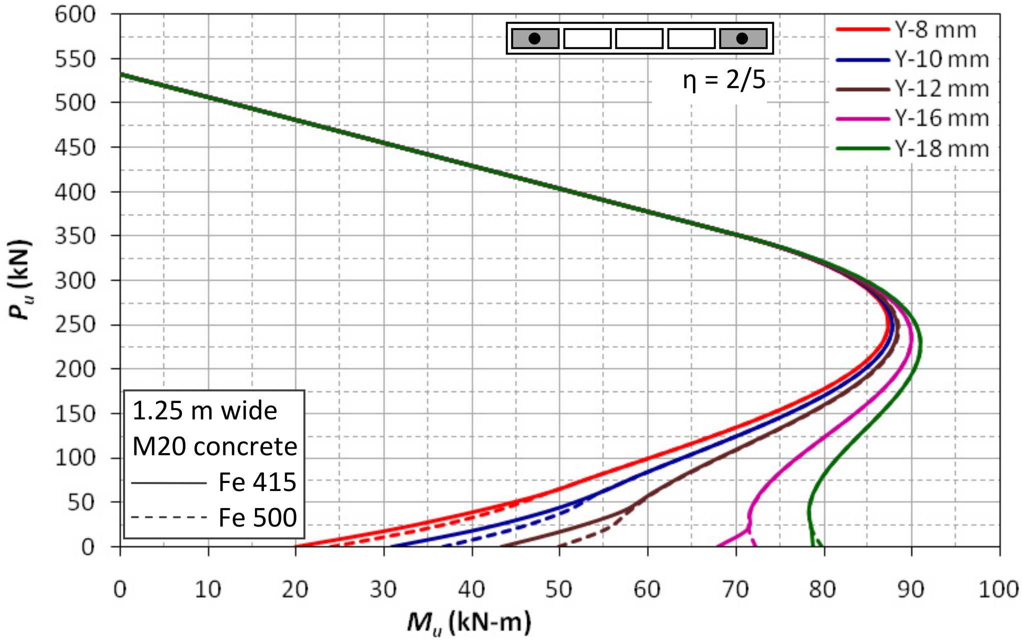


Fig. 10.36 Design P_u-M_u plots for **1.25 m wide** GFRG panel with end cavities infilled with M20 concrete and reinforced with single bar

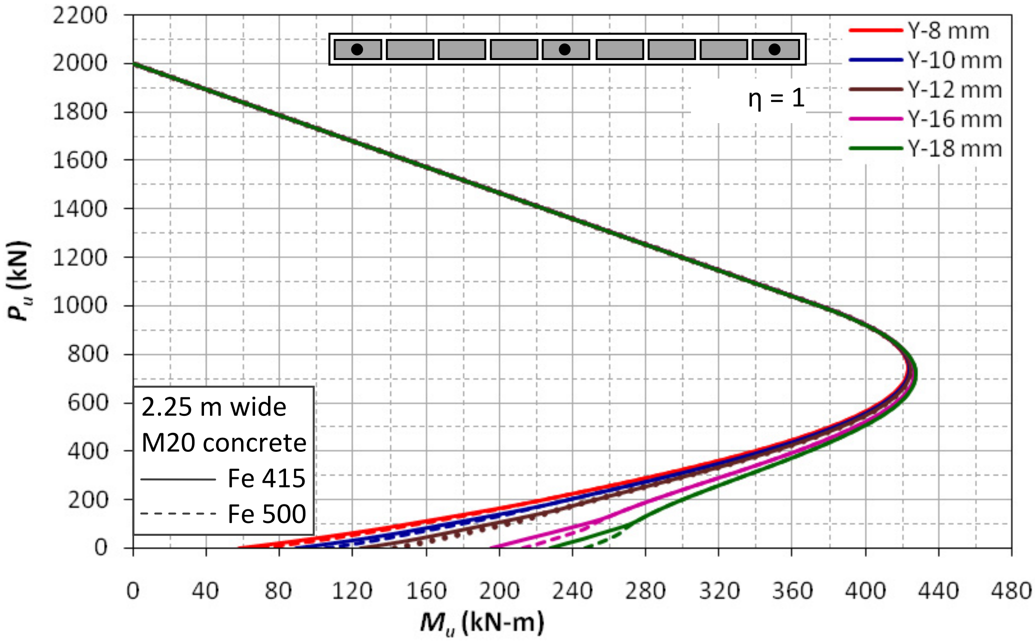


Fig. 10.37 Design P_u-M_u plots for **2.25 m wide** GFRG panel with all cavities infilled with M20 concrete and every fourth cavity reinforced with single bar

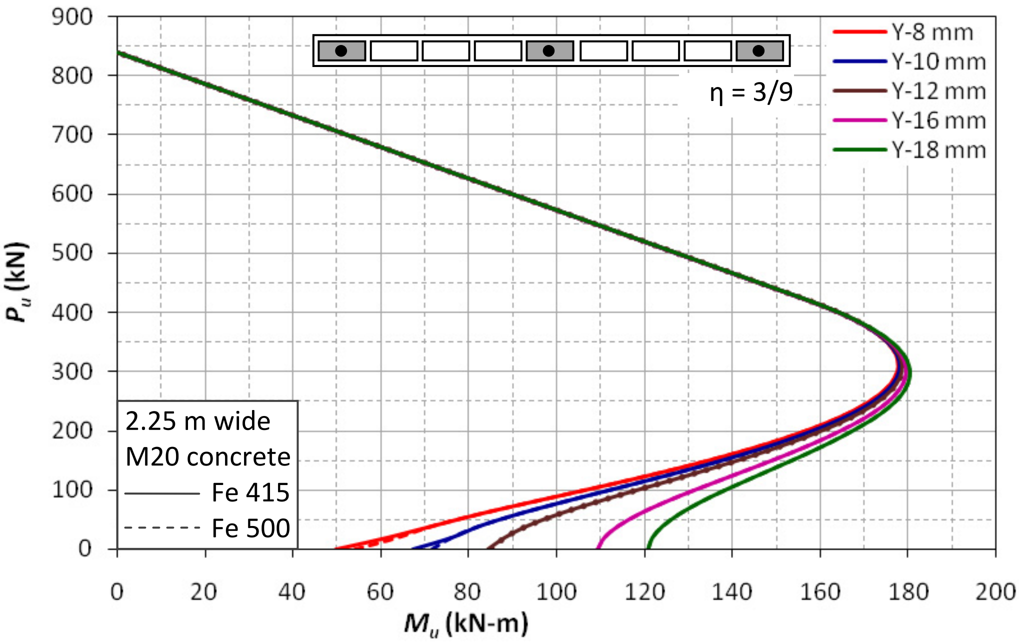


Fig. 10.38 Design P_u-M_u plots for **2.25 m wide** GFRG panel with every fourth cavity infilled with M20 concrete and reinforced with single bar

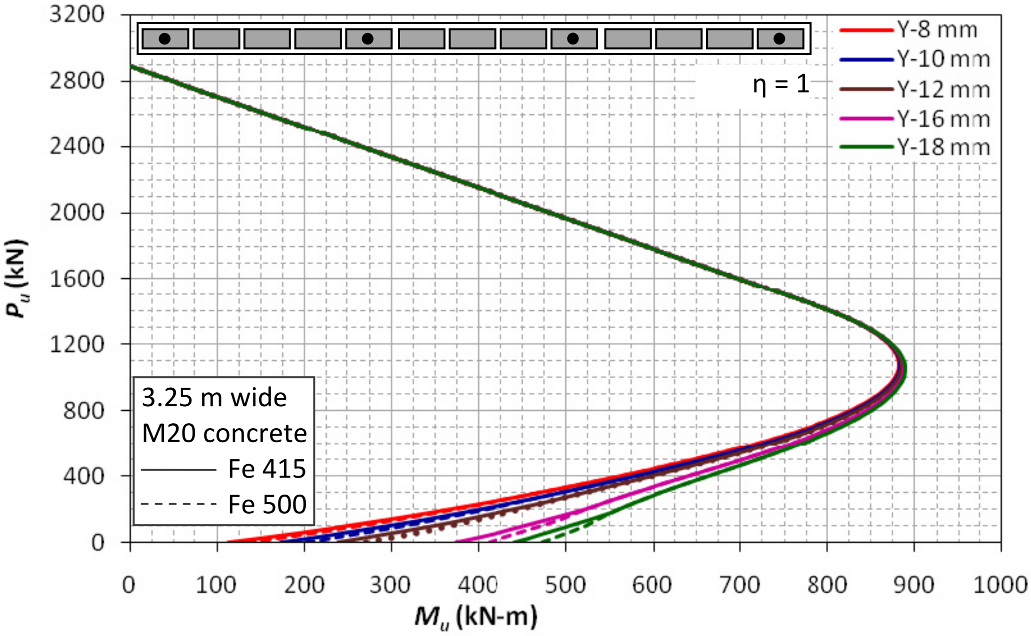


Fig. 10.39 Design P_u-M_u plots for **3.25 m wide** GFRG panel with all cavities infilled with M20 concrete and every fourth cavity reinforced with single bar

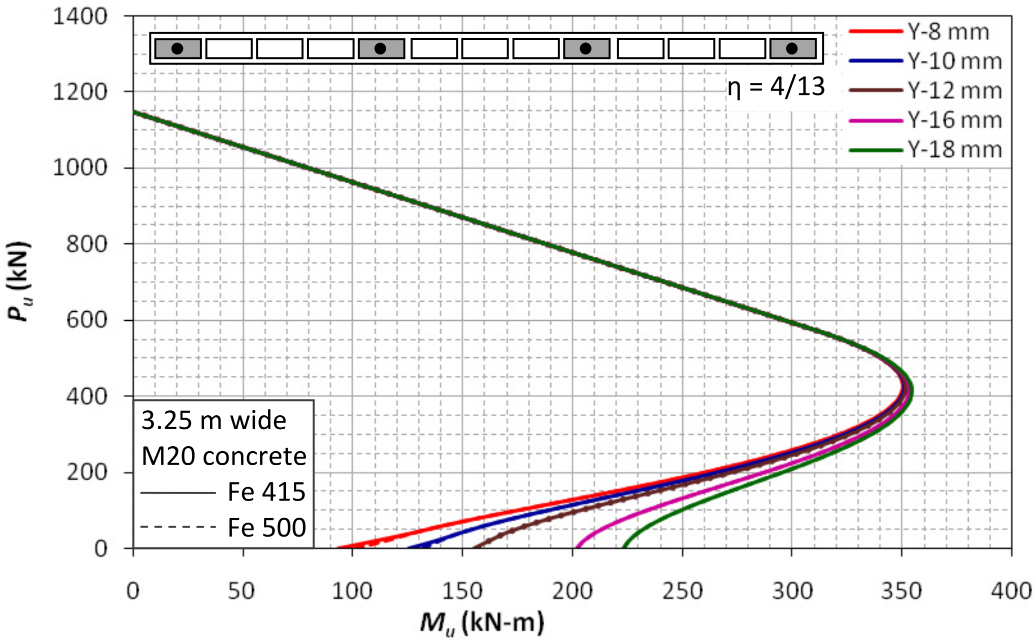
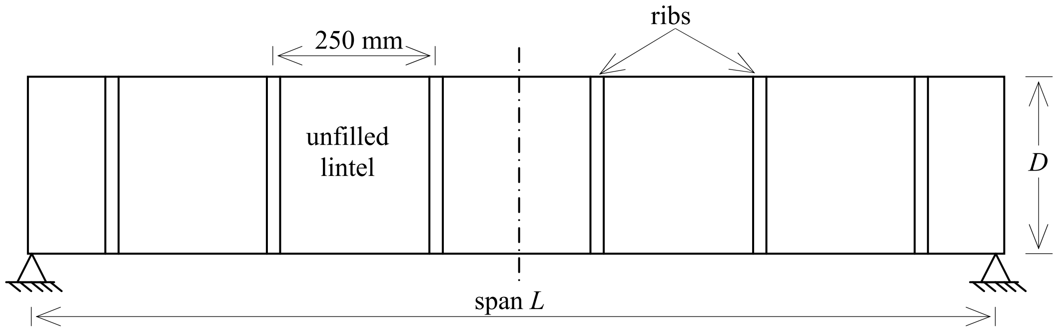


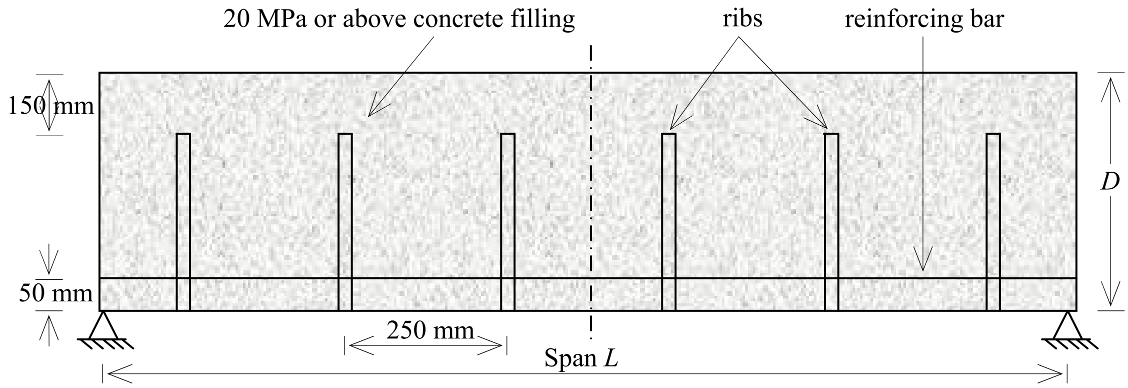
Fig. 10.40 Design P_u-M_u plots for **3.25 m wide** GFRG panel with every fourth cavity infilled with M20 concrete and reinforced with single bar

11. DESIGN OF LINTEL

GFRG panels above window and door cut-outs can be used as lintels to support superimposed loads. These design guidelines are based on simply supported conditions. For lintels that are actually framed above an opening, to limit creep deflection, they may be treated as simply supported.



(a)



(b)

Fig. 11.1 Details of lintels

Note: To maintain a continuous concrete compression zone, the top 150 mm of connecting ribs must be removed.

11.1 Unfilled Lintels

The design moment capacities of unfilled lintels are given in Fig.11.2[1,13]. Shear strength checks are not required for beams with aspect ratios (*D/L*) less than 0.5.

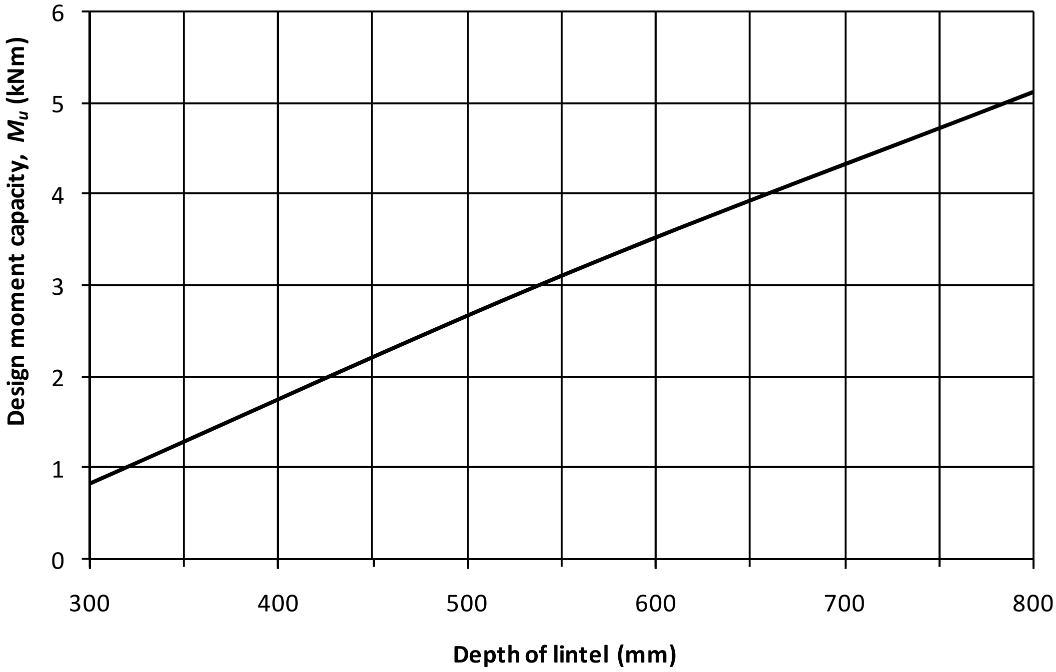


Fig. 11.2 Design moment capacity of unfilled lintel

The displacement of lintels can be calculated with the flexural rigidity *EI* provided in Fig. 11.3.

Note: Due to the variable influence of the connecting ribs, *EI* varies with both depth and span of the lintel.

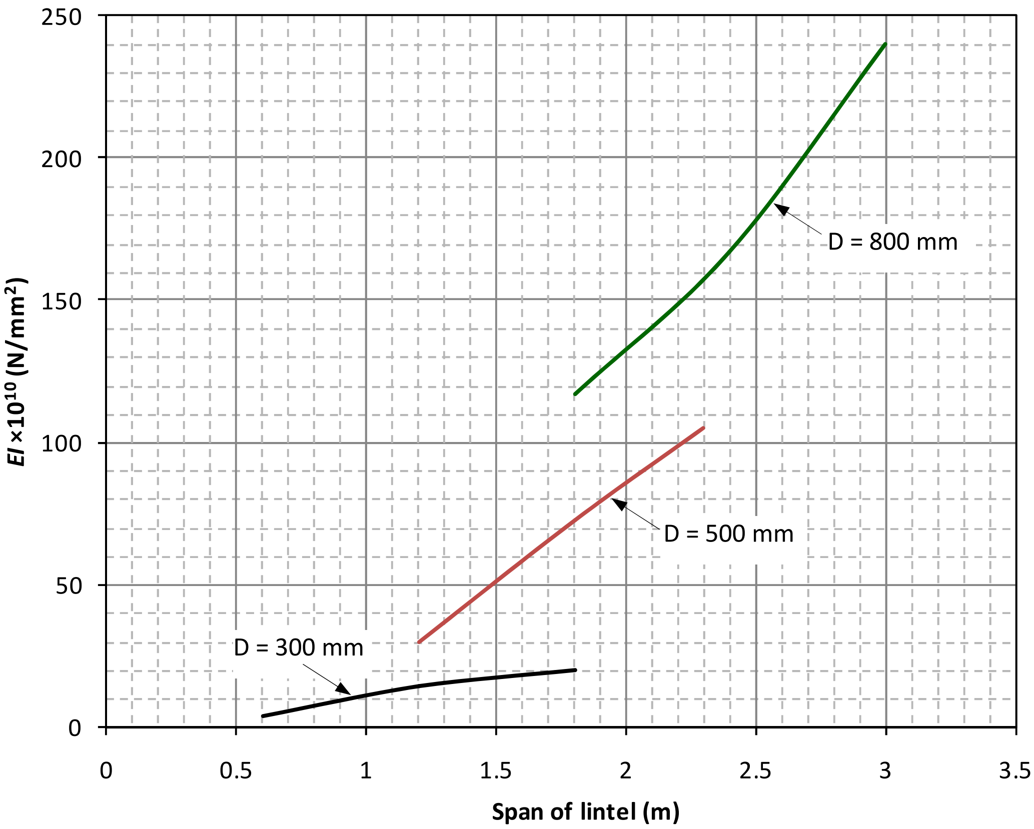


Fig. 11.3 Elastic rigidity of unfilled lintels

11.2 Concrete Filled Lintels

Flexural strength

The flexural strength of lintels shall be calculated in accordance with IS:456-2000 based upon the cross-section shown in Fig. 11.4.

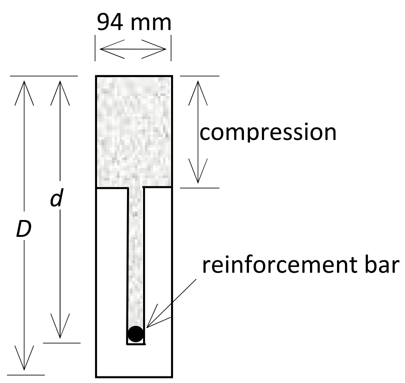


Fig. 11.4 Lintel cross – section for flexural strength calculation

Shear strength

The typical shear failure mode for concrete filled lintels is illustrated Fig.11.5.

The ultimate design shear strength at any shear plane is V_{ud} = 25kN.

This lintel ultimate shear strength is applicable for all values of depth *D*.

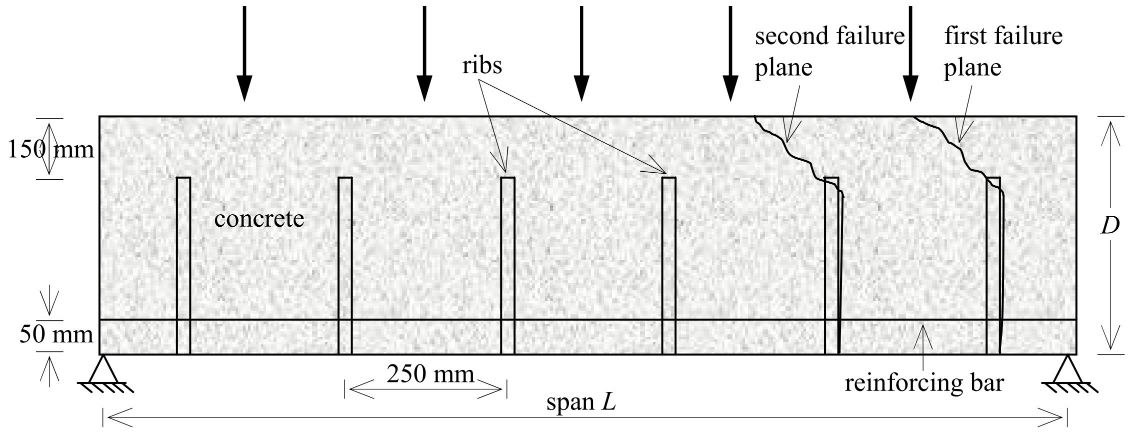


Fig. 11.5 Typical shear failure mode in lintel

11.3 Design Example:

A 3 metre long simply supported lintel filled with 25 MPa concrete carries design *UDL* of 14 kN/m. It is reinforced with Fe415 steel. Design the lintel cross-section. Let the overall depth be 300 m. Assume that the lintel is reinforced with 16mm diameter bar with a clear cover of 30mm.

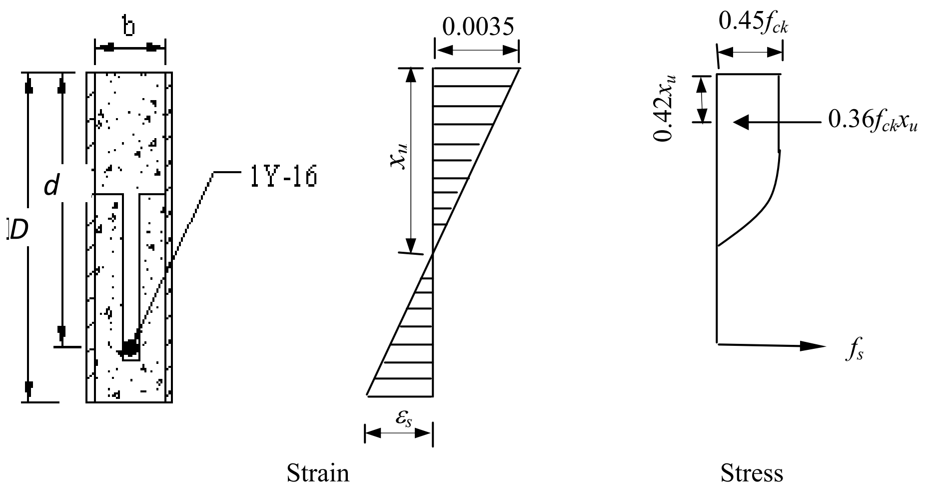


Fig. 13.6 Stress and strain variation in lintel

The effective depth, d = 300 – 30 – 8 = 262 mm.

Design shear force, V_u = $(3/2 - 0.262) \times 14$

= 17.4 kN < 25.0 kN shear strength. O.K.

Depth of neutral axis assuming section as under reinforced

$$x_u = 0.87\, f_y\, A_{st} / (0.36\, f_{ck}\, b)$$

$= \quad (0.87 \times 415 \times 201) / (0.36 \times 25 \times 94)$

$= \quad 85.8$ mm < ($0.48 \times 262 = 126$ mm)

Hence, the section is under reinforced.

Moment of resistance, $M_{ur} \quad = \quad 0.87\, f_y\, A_{st}\, (d - 0.42\, x_u)$

$= \quad 0.87 \times 415 \times 201\, (262 - 0.42 \times 85.8) \times 10^6$

$= \quad 16.4$ kNm

Design bending moment, $M_u \quad = \quad wl^2/8 = 14 \times 3^2/8 = 15.8$ kNm $< M_{ur} = 16.4$ kNm

Hence the section is safe.

12. DESIGN OF FLOOR / ROOF SLAB

As GFRG panels with ribs aligned in direction of bending possess flexural strength (refer Table 8.1), such panels can be used as flexural slab, whose strength can be significantly enhanced by embedding 'micro beams', filled with reinforced concrete. Unfilled GFRG panels can be used as pitched roofs for single storeyed small span buildings (refer Fig. 12.1). Some nominal filling with reinforcement may be done at eaves and ridge locations as shown in Fig. 12.3.

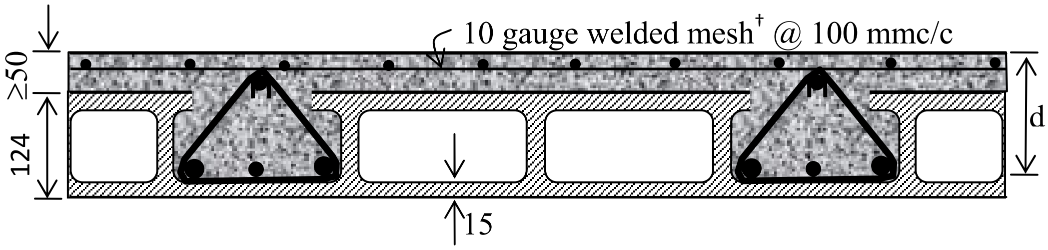


Fig. 12.1 Typical cross-section of panel with micro beams

GFRG-RC composite slab systems can be used efficiently in floor slabs and roof slabs. The ribs are to be oriented along the shorter span, supported on GFRG wall panels. For convenience in design, the contribution of GFRG towards the flexural strength can be ignored and the GFRG treated as lost formwork. Reinforced concrete micro beams are to be provided by filling cavities at regular intervals (typically every third cavity) and provided with reinforcement suitably designed, with a screed concrete of thickness not less than 50 mm as shown in Fig.12.1. One way slab action may be assumed for strength and deflection check, considering T beam action of the embedded micro beams. In the screed concrete, suitable welded wire fabric (of required guage and spacing) shall be provided. The design of reinforcement in the micro beams shall conform to the requirements of IS 456: 2000. Such slabs can be conveniently designed up to spans of 5m.

12.1 Example: Design of a slab (4.0 m span)

Design the floor slab for a room of interior dimensions 4.0m x 5.0m using GFRG building panel supported on GFRG building panels as walls for a residential building. Assume it is part of a residential building and floor finish to be 0.75 kN/m^2. Use M25 concrete and Fe 415 steel.

† In case welded mesh is not available, 6 mm bars at 150 mm spacing, may be used.

Reinforced concrete micro beams are provided at every 750mm in the shorter direction together with 50 mm screed concrete on top of the panel. Ribs are placed along the shorter span and also the floor slab is assumed to be simply supported even though floor / roof slab is integrated with the wall panels by starter bars and infilling. It is assumed that the panel essentially behaves as a one way slab in the shorter direction.

Design is done using codal requirements of IS: 875 (Part-2) 1987 for imposed loads and IS: 456 - 2000 for structural design.

Assumed cross section of the panel is as shown below.

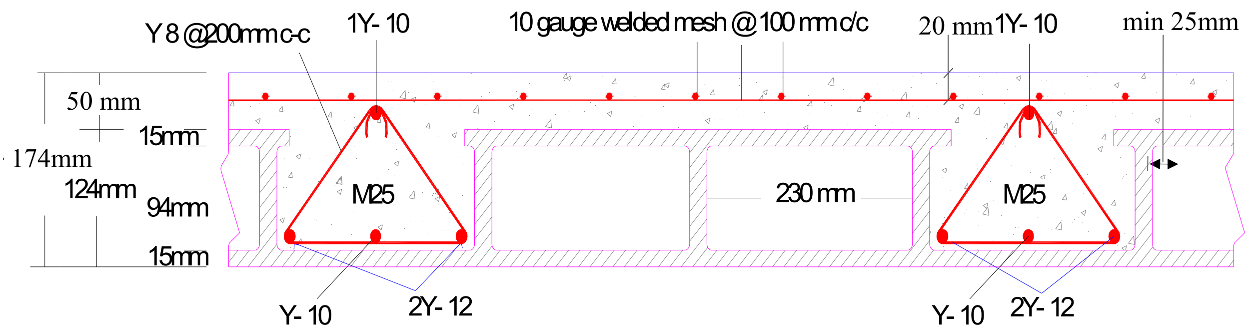


Fig. 12.2 Cross–section of slab panel with series embedded T - beams

Effective Span:

Effective depth assuming 12mm diameter bars,

d = (124 + 50 - 15 – 8 - 12/2) = 145 mm.

Effective span as per clause 22.2 of IS: 456 - 2000:

Effective span (l) is the minimum of

(i) clear span + effective depth = 4.00 + 0.145 = 4.145m

(ii) c/c of supports = 4.00 + 0.124 = 4.124m

Accordingly, l = 4.124 m

Loading

Weight of empty GFRG slab panel = 0.44 kN / m^2

Weight of infilled concrete (every 3rd cavities filled) plus the 50 mm screed concrete = 0.05 × 25 + (0.094 × 0.23 × 25 / 0.75)

= 1.97 kN/m^2

Floor finish = 1.0 kN/m^2

Live load as per IS: 875 (Part-2)1987 = 2.00 kN/m^2

Total service load, w = 5.4 kN/m^2

Design bending moment M_{ud} = (1.5 × 5.4 × 4.124²/8)

= 17.22 kN-m/m.

Design bending moment / rib M_{ud} = 0.75 × 17.22

= 12.92 kN-m

M_{ud}/bd^2 = {12.92 × 10⁶} / {230 × (145)²} = 2.67 N/mm²

From Table 3 of SP-16, Design Aids to IS: 456,

p_t = 0.876

A_{st} = (0.876 / 100)×230×145 = 292 mm².

Provide 2Y12 + 1Y10, giving an area of 305 mm²

Shear force, V_u = 1.5 × 5.4×0.75× (2–0.145) = 11.27 kN

τ_u = (11.27 × 10³) / (230 × 145) = 0.34 N/mm²

From Table 19 of IS: 456,

For p_t = 0.914, τ_c = 0.62 N/mm² > 0.32 N/mm²,

Hence, only nominal stirrup steel is required.

Minimum stirrup steel, A_{sv} = $\left(0.4bs_v\right)/\left(0.87f_y\right)$

Maximum spacing, S_V max = 0.75 × 145 = 108 mm ≅ 100 mm

A_{sv} min = (0.4 × 230 × 100) / (0.87 × 250)

= 42.3 mm²

Provide 6 mmϕ two legged mild steel stirrups @ 100c/c.

Nominal steel for screed concrete = (0.12 / 100) × 50 × 10³ = 61.2 mm² / m

Provide 10 gauge welded mesh @ 100 mm c/c on top, as shown in Fig.12.2.

Note:

- There will be nominal bending moments in the long span (perpendicular) direction, resisted by the composite action of GFRG (without ribs) and screed concrete.
- The limiting tensile strain should not be exceeded at the soffit of the GFRG, to avoid cracking of GFRG in this direction.

- This will also provide a control on deflection. If required (in large spans), the screed thickness can be increased and/or additional reinforcement and concrete may be provided in the long span directions.

12.2 Example: Design of floor slab for a public building (4.4 m span)

Design the floor slab for a room of interior dimensions 4.4m x 5.0m using GFRG building panels as walls for a public building. Consider imposed load intensity of 4 kN/m^2 (as per IS 875 Part-2) and floor finish load is taken as 1.0 kN/m^2. Use M25 concrete and Fe 415 steel.

Reinforced concrete micro beams are provided at every 500 mm spacing in the shorter direction together with 60 mm screed concrete on top of the panel. Ribs are placed along the shorter span and also the floor slab is assumed to be simply supported even though floor and roof slab is integrated with the wall panels by starter bars and infilling. It is assumed that the panel essentially behaves like a one way slab in the shorter direction.
Assumed cross-section of the panel to be as shown below.

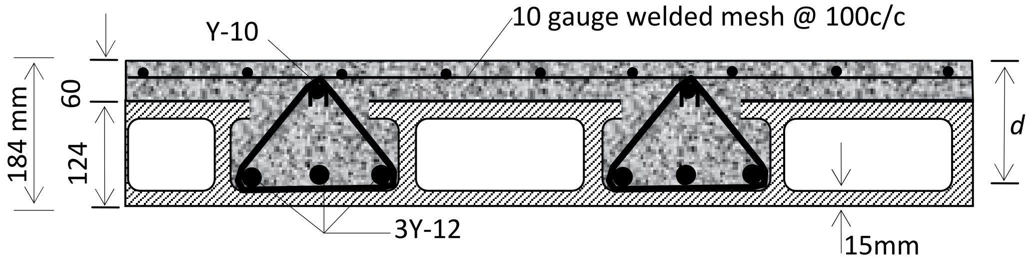


Fig. 12.3 Cross–section of floor slab of industrial building

Effective Span:

Clear span (Shorter direction of room) = 4.4 m

Effective depth assuming 12mm diameter bars,

d = (124 + 60 - 15 – 8 - 16/2) = 153 mm

Effective span as per clause 22.2 of IS: 456 - 2000:

Effective span (l) is the minimum of

(i) clear span + effective depth = 4.40 + 0.153 = 4.553m

(ii) c/c of supports = 4.40 + 0.124 = 4.524m

Accordingly, l = 4.524 m

Loading

Live load as per IS: 875 (Part-2)1987 = 4.00 kN/m^2

Weight of empty GFRG slab panel = 0.44 kN / m^2

Weight of infilled concrete (every 2nd cavities filled) plus the 60 mm screed concrete = 0.06 × 25 + (0.094 × 0.23 × 25 / 0.5)

= 2.581kN/m^2

Floor finish = 1.0 kN/m^2

Total service load, w = 8.02 kN/m^2

Design bending moment M_{ud} = 1.5 $wl^2/8$

= (1.5 × 8.02 × 4.524^2/8)

= 30.8 kN-m/m.

Design bending moment / rib M_{ud} = 0.5 × 30.8 = 15.4 kN-m

M_{ud}/bd^2 = {15.4 × 10^6} / {230 × (153)^2}

= 2.86 N/mm^2

From Table 3 of SP-16, Design Aids to IS: 456

p_t = 0.956

$\therefore\ A_{st}$ = (0.956 /100)×230×153=336.4 mm^2

Provide 3Y12 bars at bottom, giving an area of 339 mm^2 and 1Y10 bar at top

Check for shear reinforcement

Shear force, V_u = 1.5 × 8.02 ×0.5× (4.524/2–0.153) = 12.68 kN

τ_u = (12.68 × 10^3) / (230 × 153) = 0.36 N/mm^2

From Table 61 of SP-16,

for p_t = (3×113×100)/(230×153) = 0.96%

τ_c = 0.6 N/mm^2 > 0.36 N/mm^2,

Hence only nominal stirrup steel is required.

Minimum stirrup steel, A_{sv} = $(0.4bs_v)/(0.87f_y)$

Maximum spacing, S_V max = 0.75 × 145 = 108 mm ≅ 100 mm

A_{sv} min = (0.4 × 230 × 100) / (0.87 × 250)

$= 42.3\ mm^2$

Provide 6 mmϕ two legged mild steel stirrups @ 100c/c.

Nominal steel for screed concrete = $(0.12 / 100) \times 60 \times 10^3 = 72\ mm^2 / m$

Provide 10 gauge welded mesh @ 100 mm c/c on top, as shown in Fig.12.3.

12.3 GFRG Building Panel as Pitched (slope) Roofing Element

GFRG building panels can be used as roofing elements for buildings with pitched (sloped) roof, which is commonly adopted for low income group housing as shown in Fig. 12.4. Design of such roofing for typical low income group housing with a plinth area of 21.06 m^2 is illustrated here.

Live load on the roof is taken as per IS: 875 (Part-2) 1987 as 0.65 kN/m^2 (on projected plan area). It is assumed that water-repellent paint is applied on the top of the roof as in the case of external surface of the building wall panels. The roof panels are assumed to be integrally connected with the wall panel (through provision of reinforced concrete grouts as shown in Fig. 12.4). A finite element analysis has been carried out for a dead load (self weight) of 0.43 kN/m^2 in the unfilled portion and an additional 1.98 kN/m^2 in the filled regions, and for a live load of 0.65 kN/m^2 on the projected plan area of the building. The variation of bending moments in the roof (i) along the slope (ribs parallel to slope), (ii) perpendicular to the rib, and in the wall (both horizontal and vertical bending moments) are shown in Fig. 12.5. From these figures, it can be seen that the maximum design moment parallel to the rib is less than the design moment capacity of 1.4 kNm/m and the maximum design moment perpendicular to the rib is less than the corresponding design moment capacity of 0.59 kNm/m. This means that panels can be used as sloping roof without any reinforced concrete infilling.

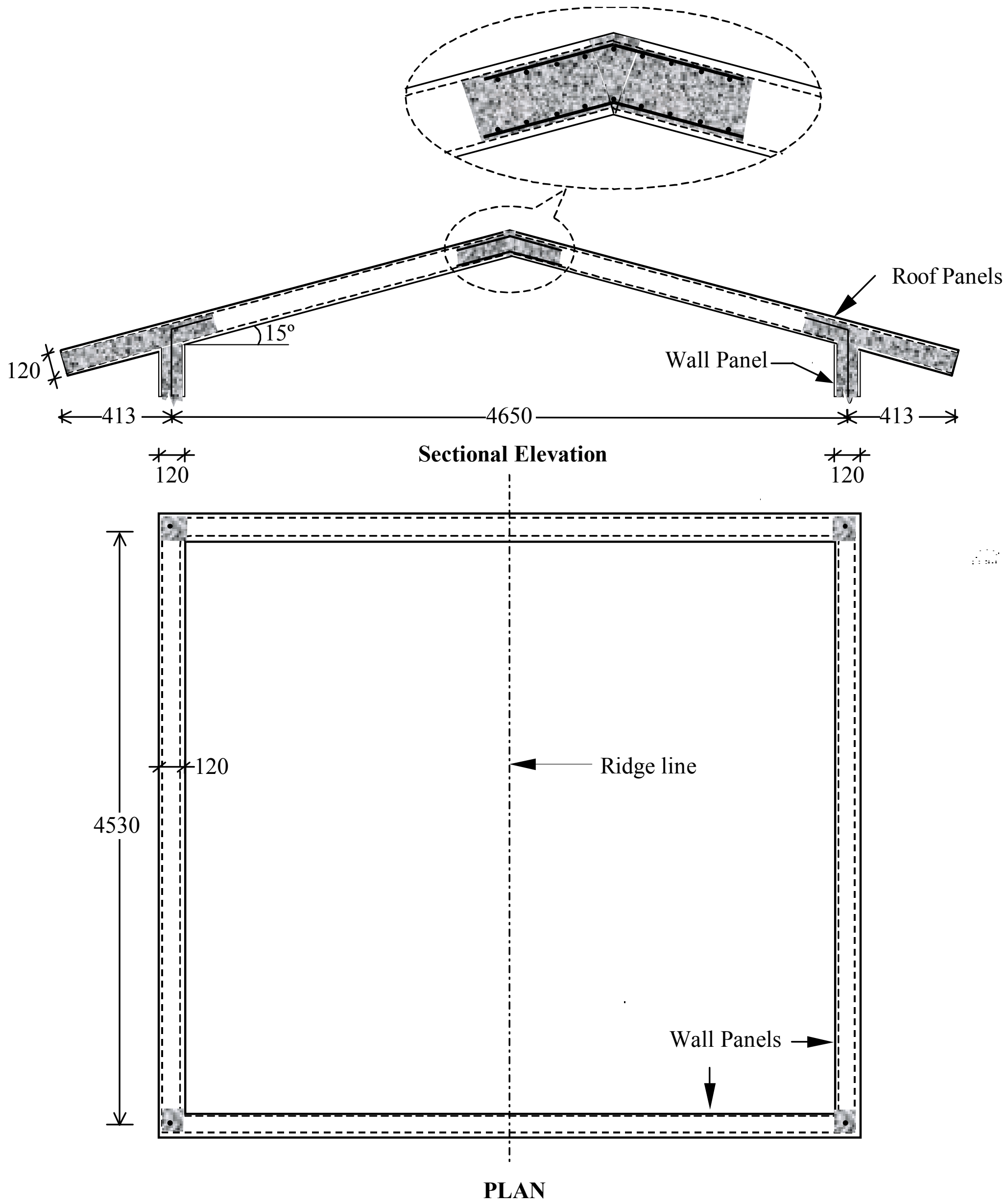
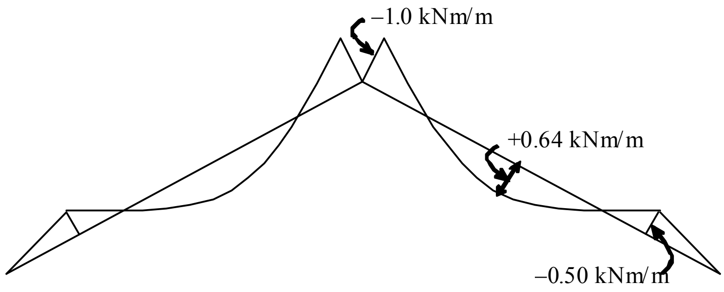
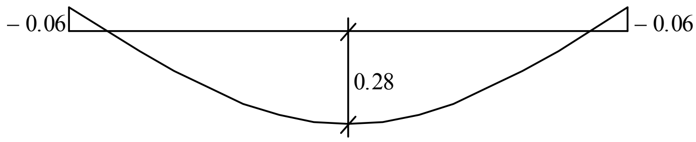


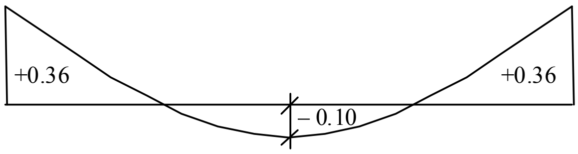
Fig. 12.4 Typical unfilled GFRG pitched roof



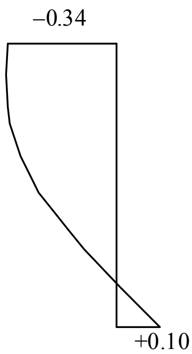
a. Lateral Bending Moments in Roof Panel due to Total Load



b. Longitudinal Bending Moments in kNm/m in Roof Panel due to Total Load



c. Horizontal Bending Moments in kNm/m in Wall due to Total



d. Vertical Bending Moment in kNm/m in Wall due to Total Load

Fig. 12.5 Variation of Bending Moments in the Roof and Wall Panels

13. ANALYSIS AND DESIGN OF A TYPICAL MULTI-STOREYED BUILDING

A typical plan of an 8-storey apartment complex planned to be constructed in Mumbai (Zone III) is considered here for demonstration. It is to be designed for earthquake loads as per IS 1893 (Part1) 2002. Fig. 13.1 shows the plan of a typical block of the apartment building. The building is to be founded on sandy soil with a bearing capacity of 300 kN/m^2 at a depth of 1.5 m below the ground level.

13.1 LOAD CALCULATIONS

Dead and Live Loads

The dead load of the entire walls and slabs are taken from the unit weight of the materials. The live load on the floors and roof are taken as 2 kN/m^2 (IS 875- 2003, part II).

Water Tank, Lift Load and Floor Finish

The capacity of water tanks is 10000 litres. The load from the water tank is calculated as 20 kN/m^2. The load from the lift and accessories is obtained as 10 kN/m^2. The weight of the floor finish is taken as 1 kN/m^2.

Wind load

Basic wind speed (V_b) = 44 m/s

Risk coefficient (k_1) = 1.0

Terrain, height and structure factor (k_2) = 1.07 (category 2 and Class B) as per IS 875.

Topography factor (k_3) = 1.0

Design wind speed (V_z) = V_b. $k_1.k_2.k_3$

= 47.08 m/s

Height of the building = 24 m

Design wind pressure (P_d) = 0.6 V_z^2 = 1.33 kN/m^2.

Seismic Load

Zone : III

Zone factor (Z) = 0.16

Length of the building: 14.31 m

Width of the building: 17.55 m

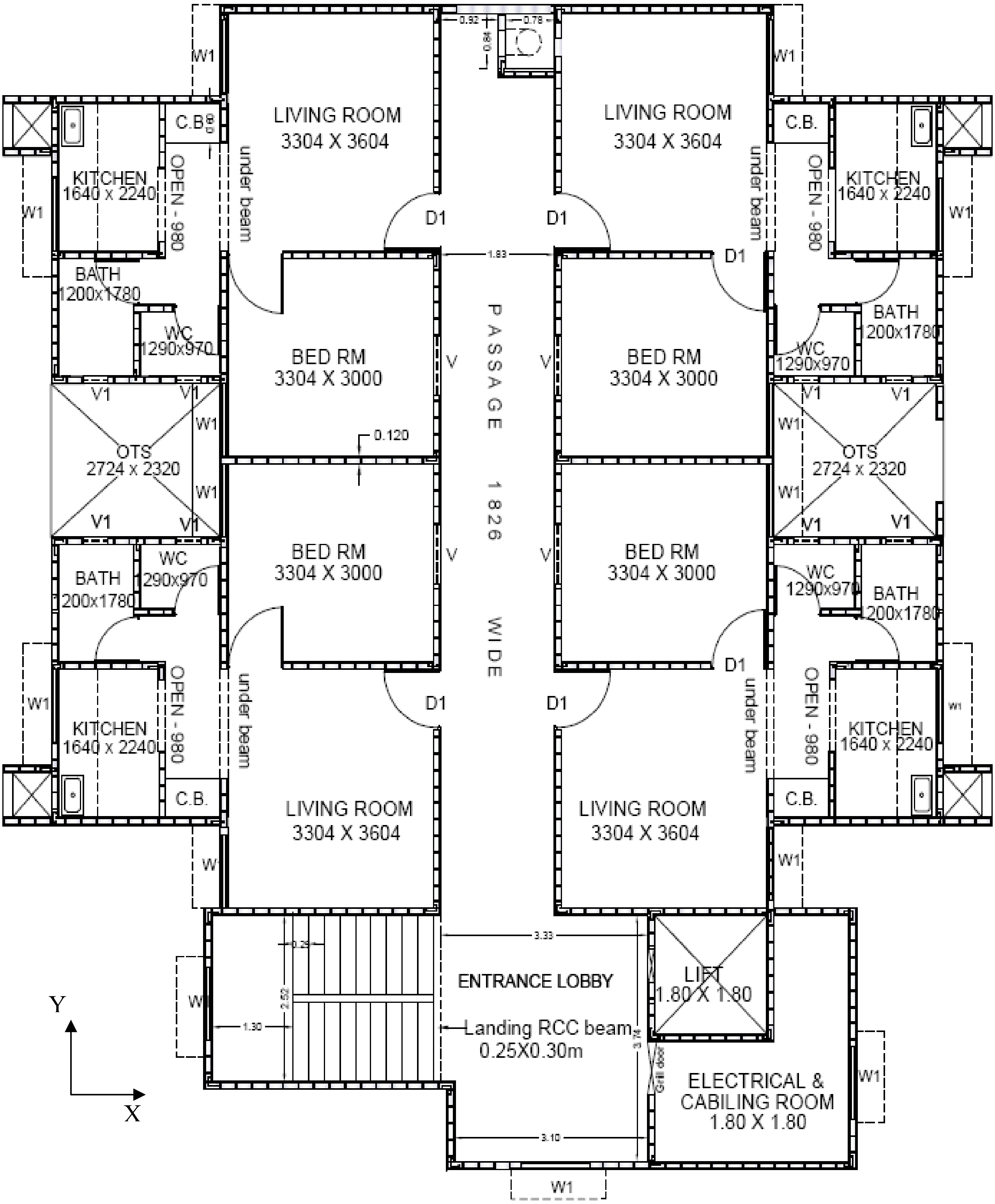


Fig. 13.1 Plan of typical floor of 8 storey building

Height of the building: 24 m

Design horizontal seismic coefficient $A_h = \frac{ZIS_a}{2Rg}$

Importance factor (*I*) = 1.0

Response reduction factor (*R*) = 3.0 (page 23 of IS 1893-2002)

$T_{ax} = \frac{0.09h}{\sqrt{d_a}}$ = $\frac{0.09 \times 24}{\sqrt{14.378}}$ = 0.57 s

$\Rightarrow S_a/g$ = 2.39 (from Fig.2 of IS 1893 Part 1: 2002)

Horizontal seismic coefficient, $A_h = \frac{ZI_f S_a}{2Rg}$ = $\frac{0.16 \times 1 \times 2.39}{2 \times 3}$ = 0.064

Similarly for Y direction, $T_{ay} = \frac{0.09h}{\sqrt{d_a}}$ = $\frac{0.09 \times 24}{\sqrt{17.55}}$ = 0.51 s

$\Rightarrow S_a/g$ = 2.50 (from Fig.2 of IS 1893 Part 1: 2002)

Horizontal seismic coefficient, $A_h = \frac{ZI_f S_a}{2Rg}$ = $\frac{0.16 \times 1 \times 2.50}{2 \times 3}$ = 0.067

Calculation of Base shear

Total seismic weight (DL + 0.25 LL), *W* = 17400 kN

Design seismic base shear along X–direction:

$$V_B = A_h W = 0.064 \times 17400 = 1114 \text{ kN}$$

Design seismic base shear along Y direction:

$$V_B = A_h W = 0.067 \times 17400 = 1166 \text{ kN}$$

The lateral forces on each floor are obtained from the design base shear (V_B) as per clause 7.7.1 of the IS 1893-2003: $Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$

The details of the time periods, base shear and storey lateral forces are tabulated in Table 13.1.

Table 13.1 Summary of lateral force calculation

Direction	X	Y
Time period	0.57s	0.51s
Base shear (kN) V_B	1114	1166
Storey level	Lateral force distribution along X (kN) Q_{ix}	Lateral force distribution along Y (kN) Q_{iy}
1	6	6
2	22	23
3	50	52
4	89	93
5	139	146
6	203	212
7	273	286
8	332	348

13.2 PRELIMINARY DESIGN

13.2.1 Check for adequacy of single wall GFRG panels

Total length of GFRG wall in X-direction L_x = 83 m

Total length of GFRG wall in Y-direction L_y = 69 m

Average design shear force

in X-direction $= V_{Bx} / L_x =$ = 1114/83 =13.4 kN/m

in Y-direction $= V_{By} / L_y$ = 1166/69 =16.9 kN/m

These average values are much less than the design shear capacity of 40.0 kN/m (refer Table 9.1) for infilled GFRG wall panels with M25 concrete, considering composite action of GFRG and RC. Therefore, single wall GFRG panels are sufficient to ensure safety against lateral loads.

13.2.2 Check for shear capacity of RC cores

It is useful to carry out an alternative (conservative) check for overall shear capacity, ignoring the contribution of GFRG and considering only the RC cores (treated as RC columns). All cavities of GFRG panels are assumed to be filled with M25 concrete and 2 - 12 mm dia Fe 415, for initial design, as shown in the Fig. 13.2.

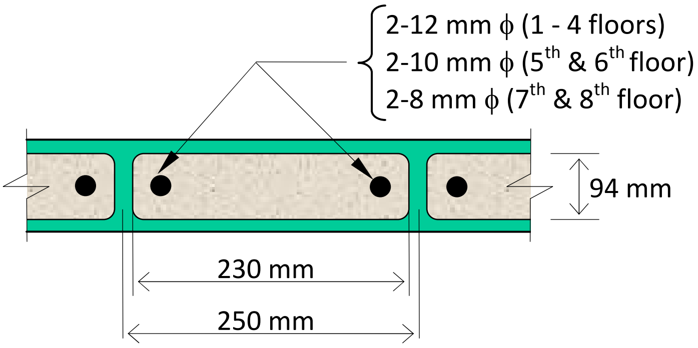


Fig. 13.2 Typical reinforcement detailing of GFRG wall panels in lower floors

Number of infilled M25 concrete cores (230 mm x 94 mm) @ 250 mm spacing

in X direction, n_x = 83/0.25 = 332

in Y direction, n_y = 69/0.25 = 276

Reinforcement ratio, $100A_{st}/bd$ = $100\times\pi\times 12^2/\left(4\times 94\times\left(230-30\right)\right)$ = 0.60

Design shear strength of each infilled concrete column, τ_c = 0.522 MPa
(from IS 456, Table 19)

Shear carrying capacity of each infilled concrete column, V_c = $\tau_c \times A_c$

= 0.522 x 94 x 200/1000 = 9.81 kN

Shear strength of building

in X-direction = n_x x V_c = 332 x 9.81 = 3257 kN > V_{Bx} = 1114 kN

in Y-direction = n_y x V_c = 276 x 9.81 = 2707 kN > V_{By} = 1166 kN

Hence safe.

13.3 DETAILED ANALYSIS AND DESIGN OF WALL SEGMENTS

13.3.1 Modelling and Analysis of the building

Lateral loads can be distributed to individual walls based on their stiffness in a manner similar to that employed in the design of reinforced concrete walls. The stiffness of a GFRG wall can be calculated by treating it as a uniform, monolithic wall 124 mm thick with an equivalent modulus of elasticity given by Eqn. 10.2.

The computational model of the 8 storey building has been developed in the software ETABS as per the architectural plan in Fig. 13.1. The various assumptions considered for the analysis of the given GFRG building is given below.

Assumptions in Modelling

- Walls are modelled as shell elements with the equivalent composite properties as given in Table 13.2.
- Slabs are modelled as membrane elements.
- Rigid diaphragm is assigned to all slab elements for lateral load analysis.
- Every room shall be segmented with joints at the intersections of two walls (which are usually orthogonal). Wall elements shall not be modelled where there are large openings (doors and large windows). If a wall is very long (exceeding 3.5m), it shall be segmented with a joint, preferably in the middle. Two nodes are introduced at the junction of two segments, with a separation of 50 mm, to ensure that the two piers behave independently, although connected on top and bottom by means of continuous horizontal tie beams (having dimensions 94 x 200 mm).
- At the junction of two or more adjoining rooms, offsets (of 50 mm) shall be provided in the two orthogonal directions to ensure independent action of the connecting walls.
- Areas above doors and windows are neglected while modelling.

With the above modelling, if it is observed that the lateral shear resistance in any wall segment, locally exceeds 1.2 times the shear strength (as given in Table 9.1), then this wall shall be segmented by introducing a joint in the middle (to simulate the potential longitudinal shear crack). The analysis shall be repeated with this refined model, and this exercise may be iteratively carried out, with further segmenting in the middle, if required. However, such segmenting shall be terminated if the resulting wall segment is less than 1.0 m. If the final analysis still shows an excessive shear demand in any wall segment, the option of providing double walls at such locations may be explored.

The floor slabs are made of GFRG panels in which every third cavity is provided with embedded RC micro beams (M25) and 50 mm thickness RC screed (M25) as deck slab. The combined modulus of elasticity considered for the modelling of the 8 storey building is given in Table 13.2. The ETABS model of the building is shown in Figure 13.3.

Table 13.2 Material properties

Material	Modulus of Elasticity (MPa)	Area for 250mm length (one cell)mm^2
Concrete (M25)	25000 (E_c)	230 x 94 (= 21620)
GFRG wall	7500 (E_R)	9380
Infilled GFRG with concrete	19500	31000

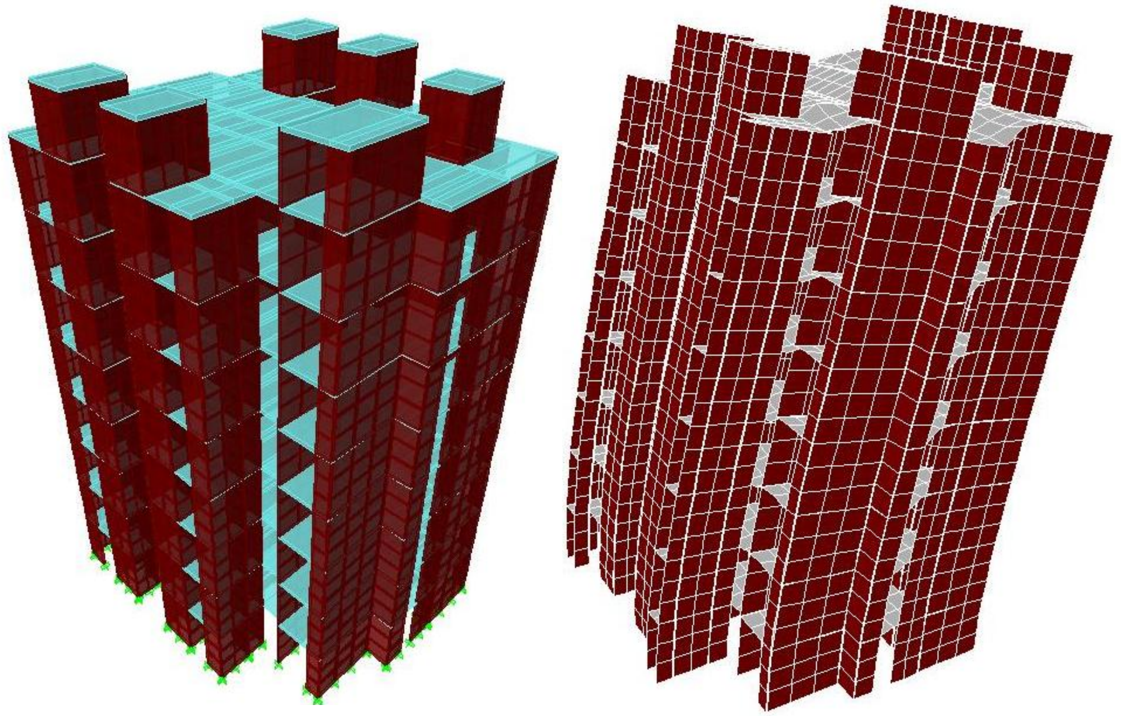

(a) 3D Model
(b) Deformed shape under seismic force

Fig. 13.3 3D Finite element model and deformed shape of 8-storey building

Linear static analysis of the computational model is carried out and the resulting values of axial force, shear force and bending moment in each wall segment (pier) is found out, in both directions, for all load combinations (as per IS 456: 2000). The results are summarized in Annexure 1 for the critical load combination.

13.3.2 Shear check on GFRG wall panels

It can be seen from the analysis results that, in general, the design shear forces in the pier walls are less than the average design strength of 40.0 kN/m. However, in some instances, the shear force is locally high, but does not exceed the limit of 20 percent excess, i.e., 48 kN/m.

13.3.2 Check on in-plane moment capacity of GFRG wall panels

The resultant axial force and in-plane bending moment in each wall, under various load combinations, are checked with the corresponding P-M interaction curves (refer Chapter 10).

The demand of axial force and bending moment of the GFRG wall panel with the capacity curves for various lengths of walls, 1.0m, 1.25,1.5m, 1.75m, 2.0m, 2.25m, 2.5m, 2.75m, 3.25m and 3.5m are shown in Figures 10, 11, 12, 13, 14, 15, 16 and 17 respectively.

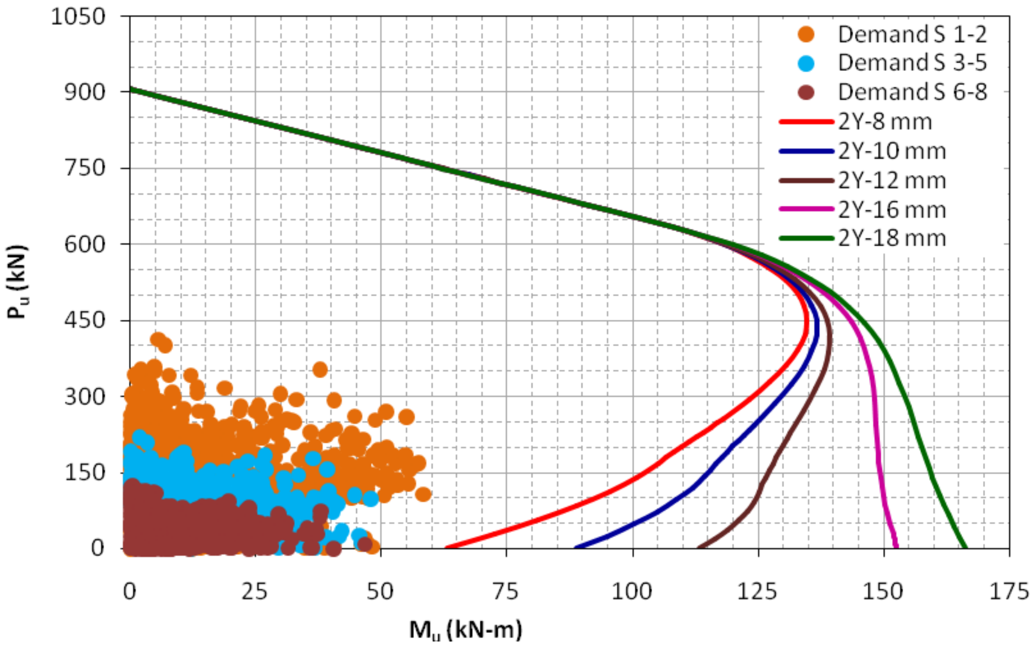


Fig. 13.4 Demand points and the capacity envelope for 1.02 m wide panel with 2 rebars in each cell

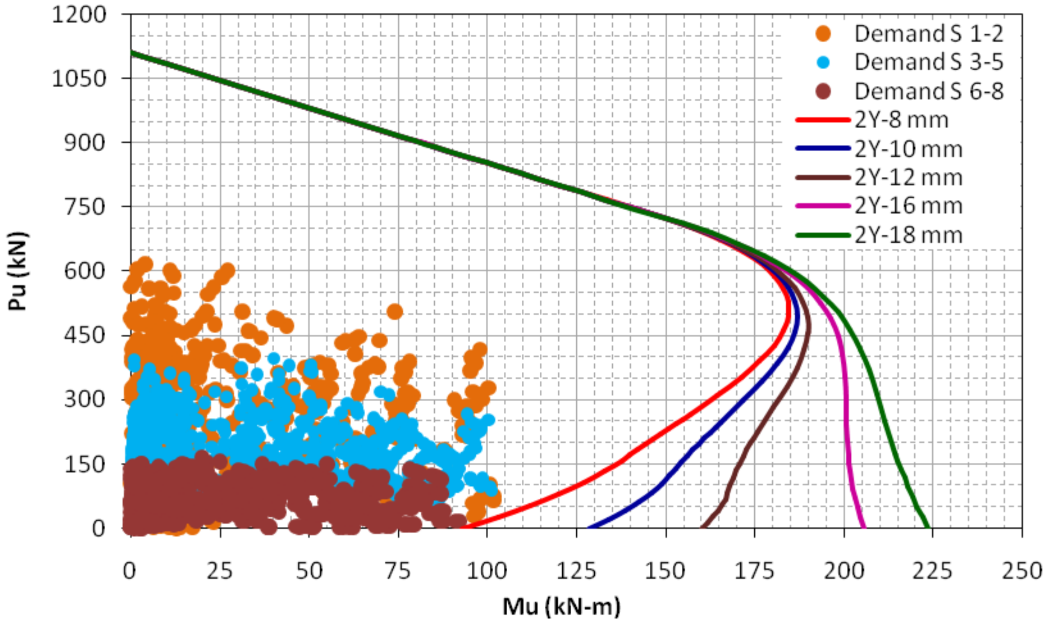


Fig. 13.5 Demand points and the capacity envelope for 1.25 m wide panel with 2 rebars in each cell

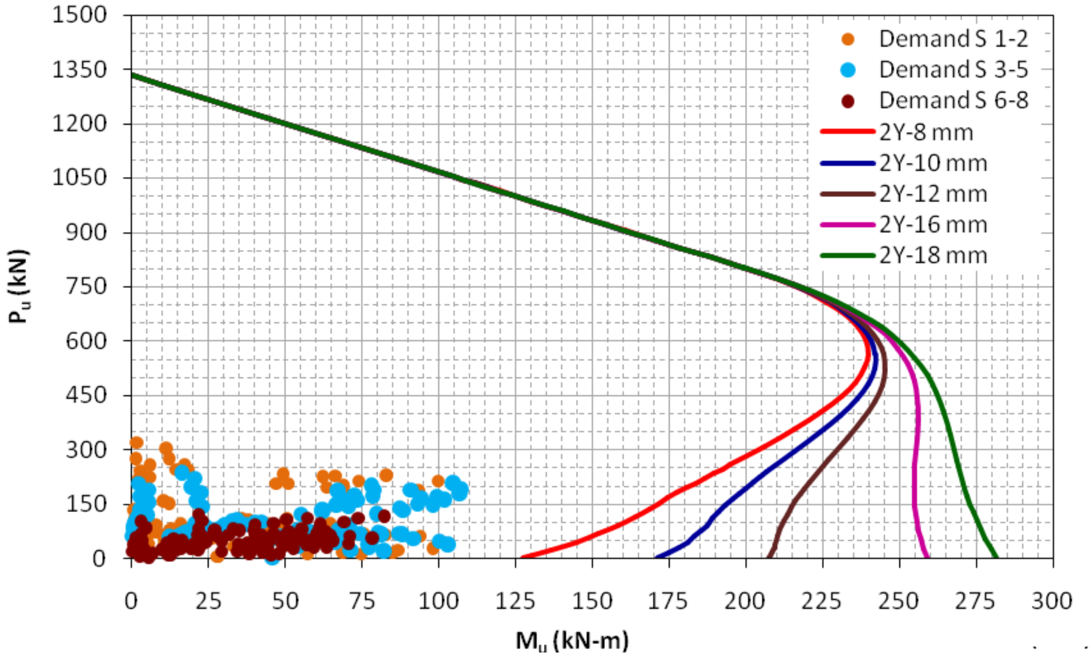


Fig. 13.6 Demand points and the capacity envelope for 1.5 m wide panel with 2 rebars in each cell

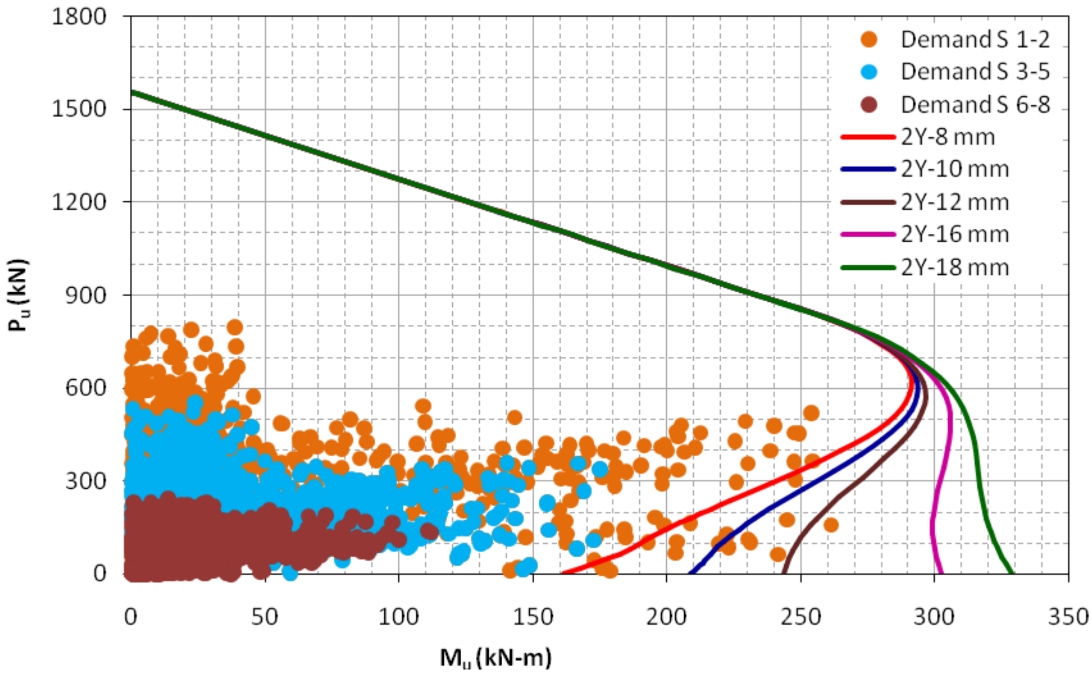


Fig. 13.7 Demand points and the capacity envelope for 1.75 m wide panel with 2 rebars in each cell

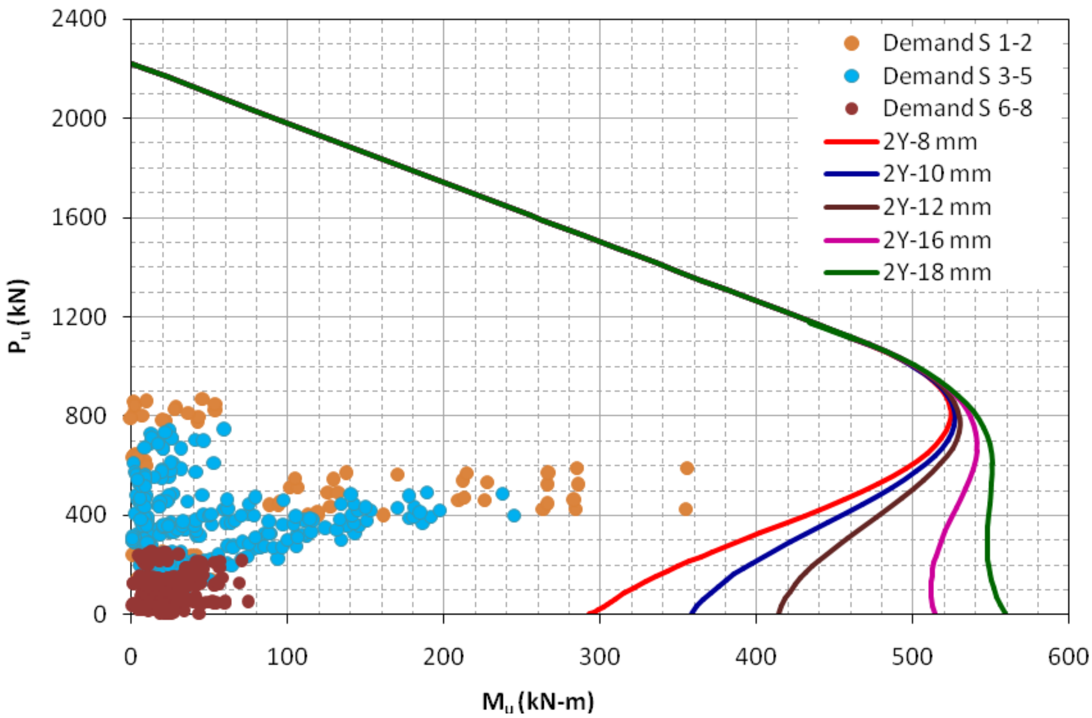


Fig. 13.8 Demand points and the capacity envelope for 2.50 m wide panel with 2 rebars in each cell

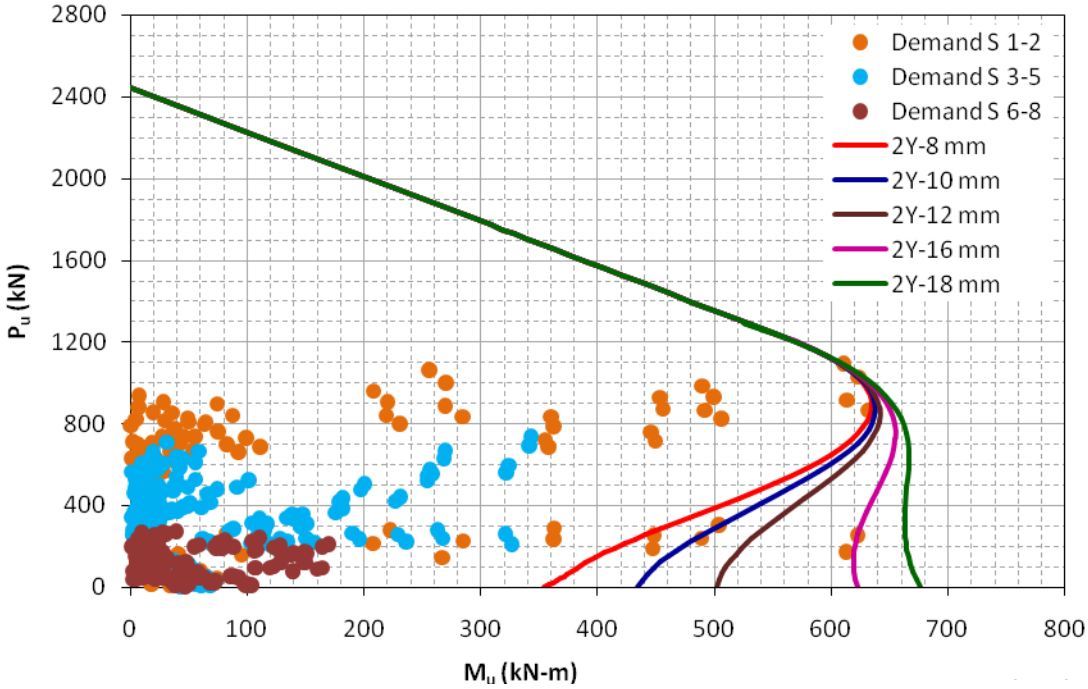


Fig. 13.9 Demand points and the capacity envelope for 2.75 m wide panel with 2 rebars in each cell

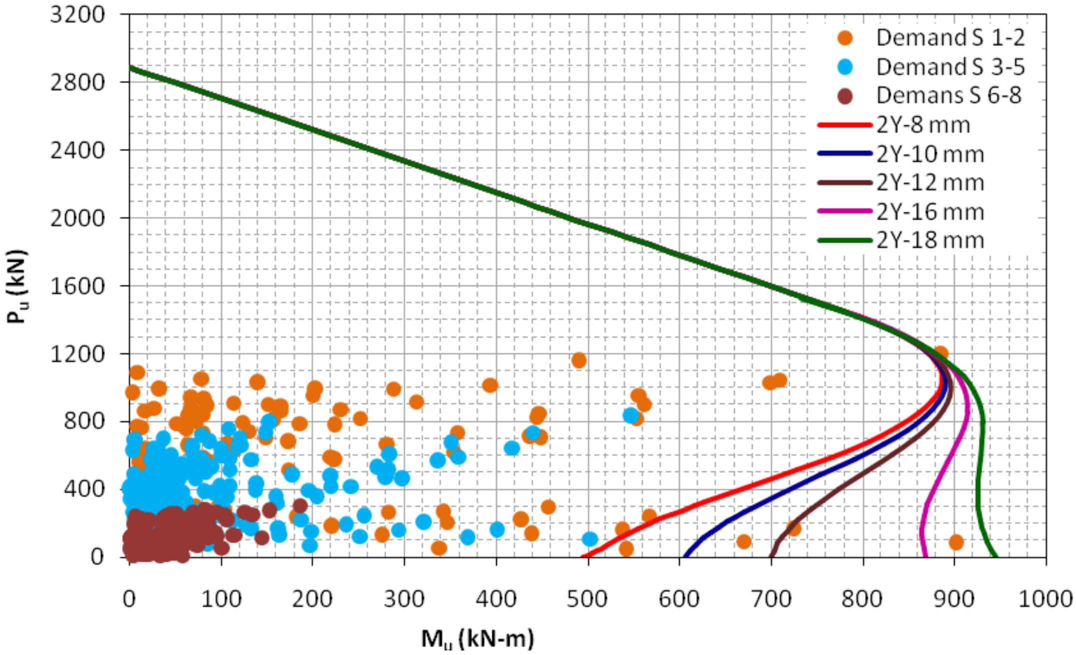


Fig. 13.10 Demand points and the capacity envelope for 3.25 m wide panel with 2 rebars in each cell

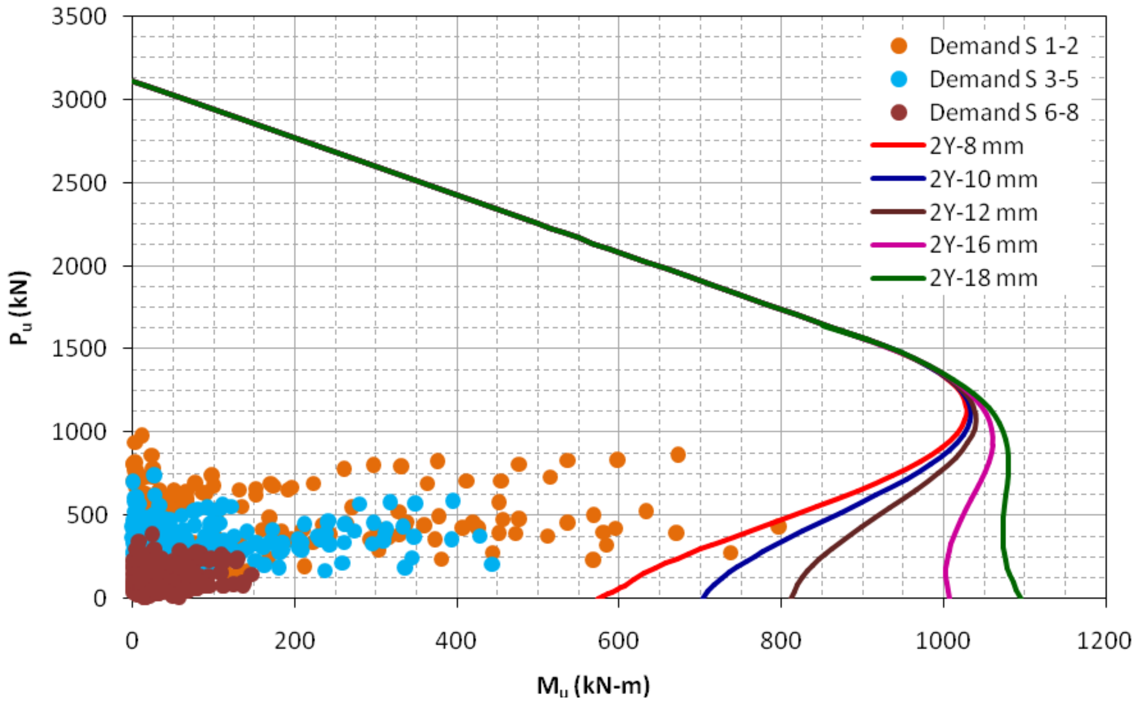


Fig. 13.11 Demand points and the capacity envelope for 3.5 m wide panel with 2 rebars in each cell

In general, all the demand points (M_u, P_u) are found to be enveloped by the interaction curve (failure line), corresponding to 2-12mm dia Fe 415 bars with M25 concrete in each cell of the GFRG panel, except for two particular wall segments of length 2.75m and 3.25m (for which 2-18mm is appropriate).

13.4 DESIGN OF SLABS

Reinforced concrete micro beams at every 750mm are provided in the shorter direction together with 50mm screed concrete on top of the panel. Ribs are placed along the shorter span and also the floor slab is assumed to be simply supported even though floor / roof slab is integrated with the wall panels by starter bars and infilling. It is assumed that the panel behaves essentially like a one way slab in the shorter direction. The design is carried out as demonstrated in Chapter 12.

13.5 DESIGN OF FOUNDATION

The soil report shows that a rock stratum having a safe bearing capacity of 30t/m^2 is available at a depth of 1.5m from the ground level. The maximum vertical load on single walls under service load combinations is 315 kN.

13.5.1 PRELIMINARY DESIGN

Design of stem wall

Assume, width of stem = 200 mm

The vertical load per metre length of the wall = 315 kN

Design vertical load = 315 x 1.5

= 473 kN

Bearing strength f_{br} = 0.45 f_{ck} = 0.45 x 20 = 9 MPa

Limiting bearing resistance = 9 x 1000 x 200 = 1800 kN > 473 kN

Hence, a minimum reinforcement of 0.25 percentage of gross cross sectional area may be provided in each direction (IS 13920, cl.9.1.4).

Minimum vertical reinforcement, $A_{st, min}$ = 0.0025 x 1000 x200 = 500 mm^2

Spacing of 12 mm reinforcement on each face of stem wall, in vertical direction = 1000 x 113/(500/2) > 300mm (Provided)

Spacing of 8 mm reinforcement on each face of stem wall, in horizontal direction = 1000 x 50/(500/2) = 200mm

Hence, provide 12 mm diameter bars at 300 mm spacing along vertical direction, and 8 mm diameter bars at 200 mm spacing along horizontal direction on each side of stem wall, as shown in Fig. 13.15.

Size of footing

The vertical load per metre length of the foundation,

= 315 kN x 1.1

= 347 kN

A strip footing can be selected as the type of foundation.

Area of the footing per metre length = 347 kN/(300kN/m) = 1.16 m

Width of the footing = 1.2 m

Thickness of footing

The uniform pressure at bottom of slab = 315 x 1.5/ 1.4

= 394 kN/m^2

= 0.39 N/mm^2

Shear force at a distance '*d*' from face = 0.390 x 1000 x (500 – d) (13.1)

Permissible shear stress for p_t = 0.25 and M20 concrete, is 0.36 MPa

Shear resistance = 0.36 x 1000 x d (13.2)

Equating equations (13.1) and (13.2), d = 261 mm

Assuming 16mm bars as along the transverse direction,

Overall depth D = 261 + 75 + 16/2 ≈ **350 mm**

Check for gross soil pressure

Gross soil pressure, q_{max} = (315+ 24 x (1.1 x 0.2 + 0.4 x 1.4) + 18 x 1.1 x1.2)/1.4

= 292.7 kN/m^2 < SBC = 300 kN/m^2, Hence, Ok

Flexural Reinforcement in footing slab

Moment at face M_u = 0.394 x 1000 x 500^2 / 2 = 4.92 ×10^7 Nmm

M_u/bd^2 = 68.5×10^6 / (1000 x (400-75-8)^2) = 0.68 N/mm^2

p_t = 0.194 (from Table 2 of SP 16)

A_{st} required = 530 mm^2 > A_{stmin} as per IS 456, cl. 26.5.2.1 = 0.12 percentage of gross cross sectional area = 480 mm^2

Provide **16 mm** diameter bars at 150 mm c/c as shown in Fig. 13.15

13.5.2 DETAILED DESIGN OF FOUNDATION

The finite element model of raft foundation is done in SAFE, which is shown in Fig. 13.12. The loads are imported from the ETAB model. The thickness of raft foundation is assumed as 350 mm throughout, except in some locations where the thickness is reduced to 150 mm for economy, as shown in Fig.13.12

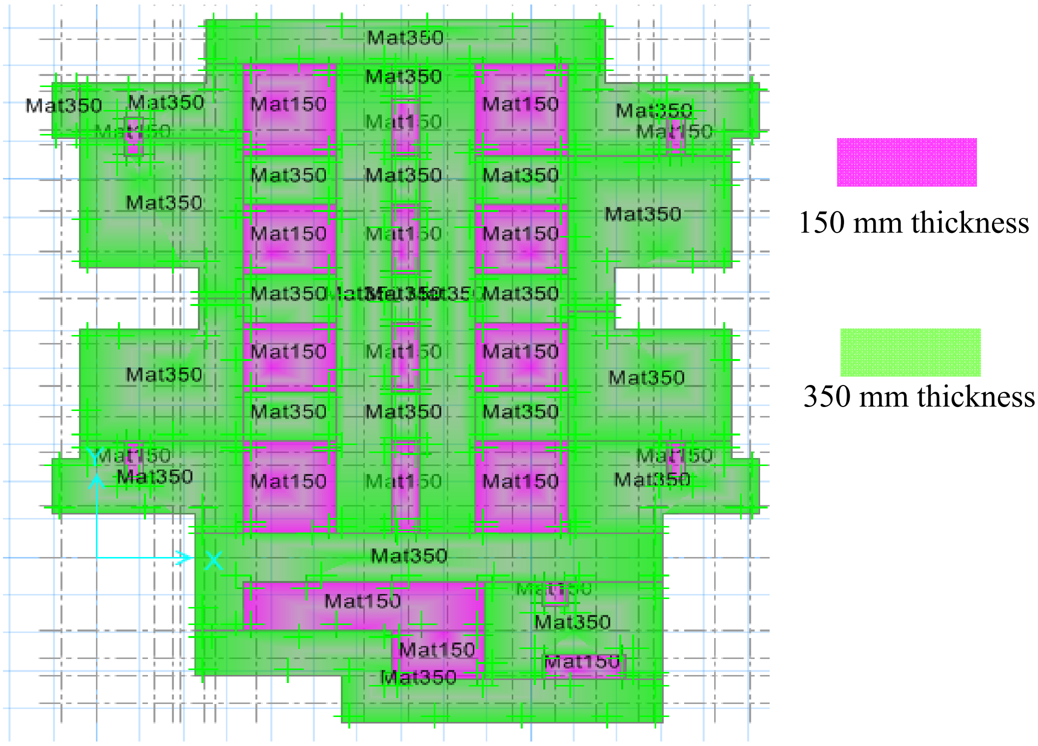


Fig. 13.12 FEM model of raft foundation

The contour of soil pressure is shown in Fig. 13.13. It can be seen from the figure that the maximum soil pressure for service load combination is 260 kN/m^2, which is less than permissible limits. The maximum deflection obtained is given in Fig. 13.14, which is within the permissible limit.

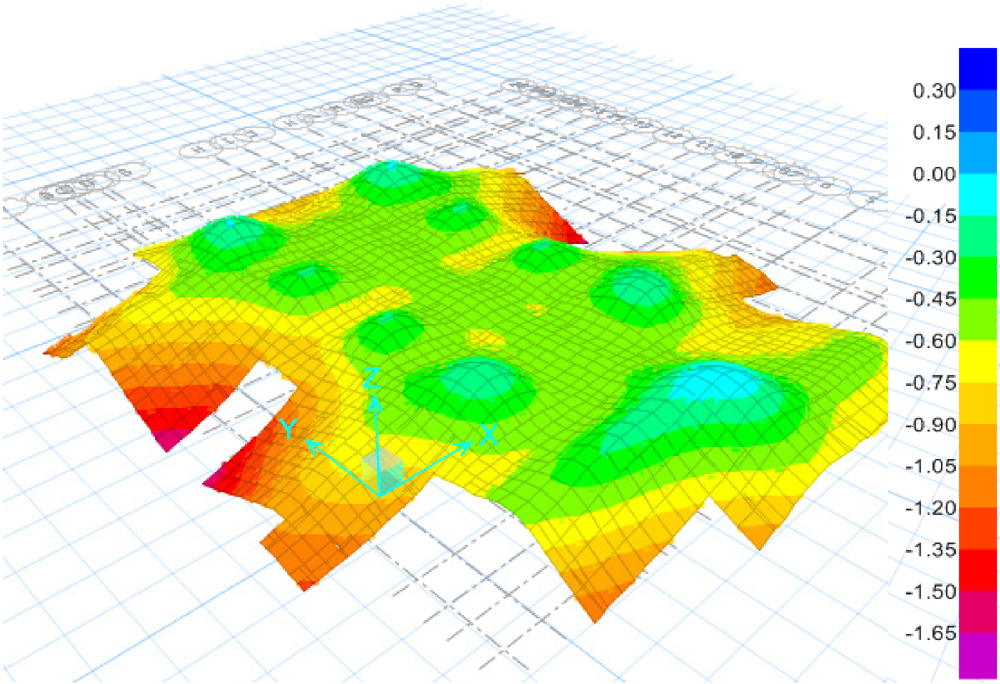


Fig.13.13 Deflection diagram of Raft Foundation (mm)

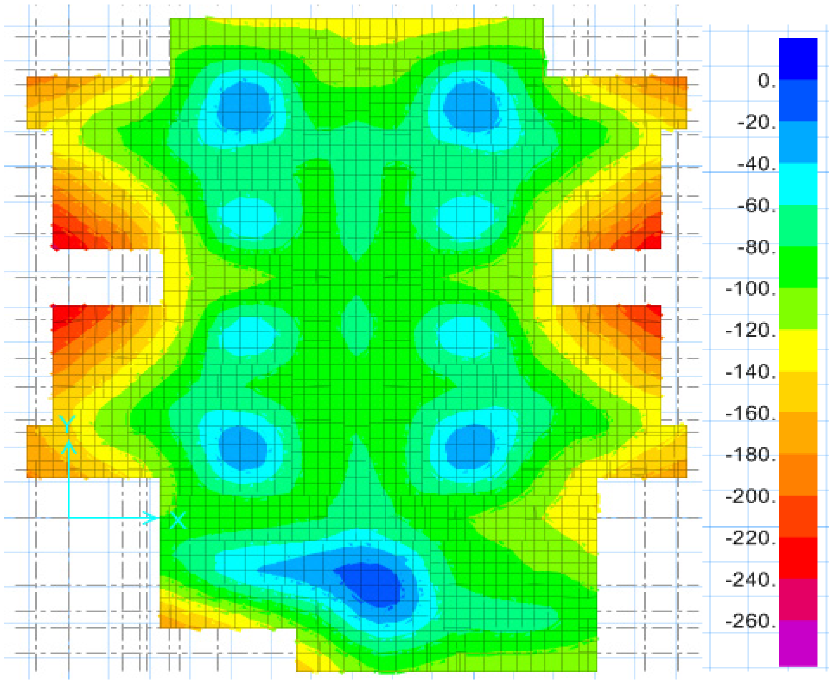


Fig. 13.14 Contour of soil pressure from SAFE model

The reinforcement details of raft foundation obtained from the SAFE model is depicted in figure 13.15.

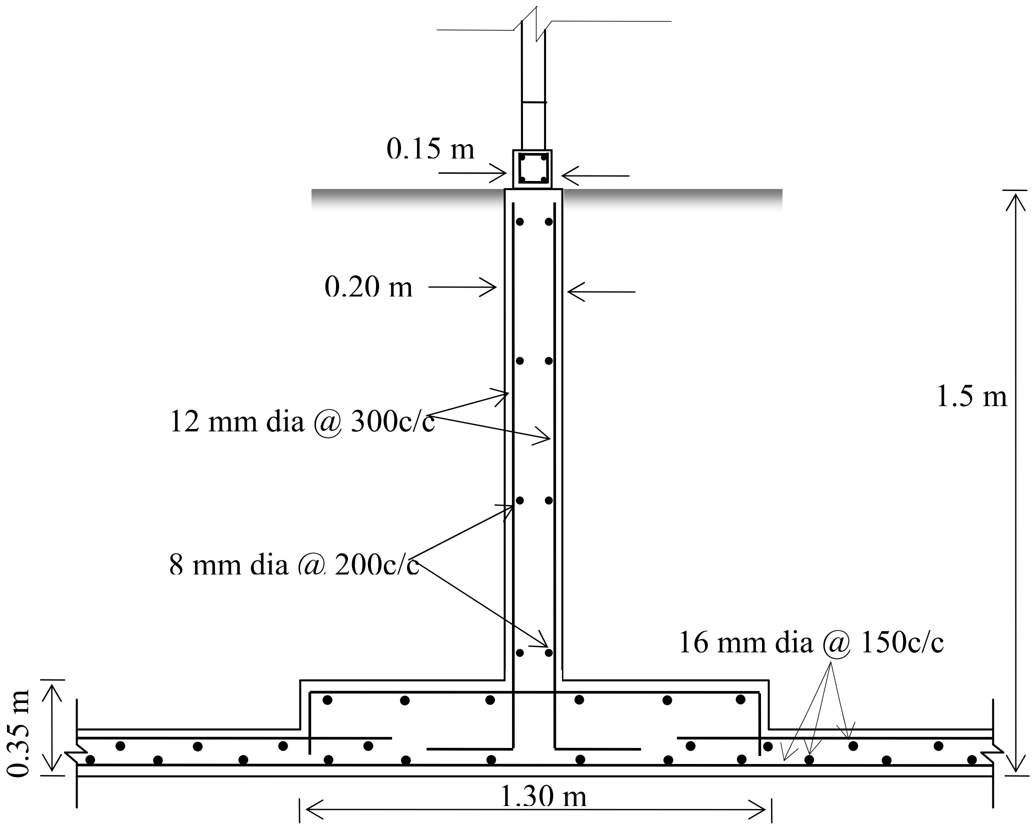
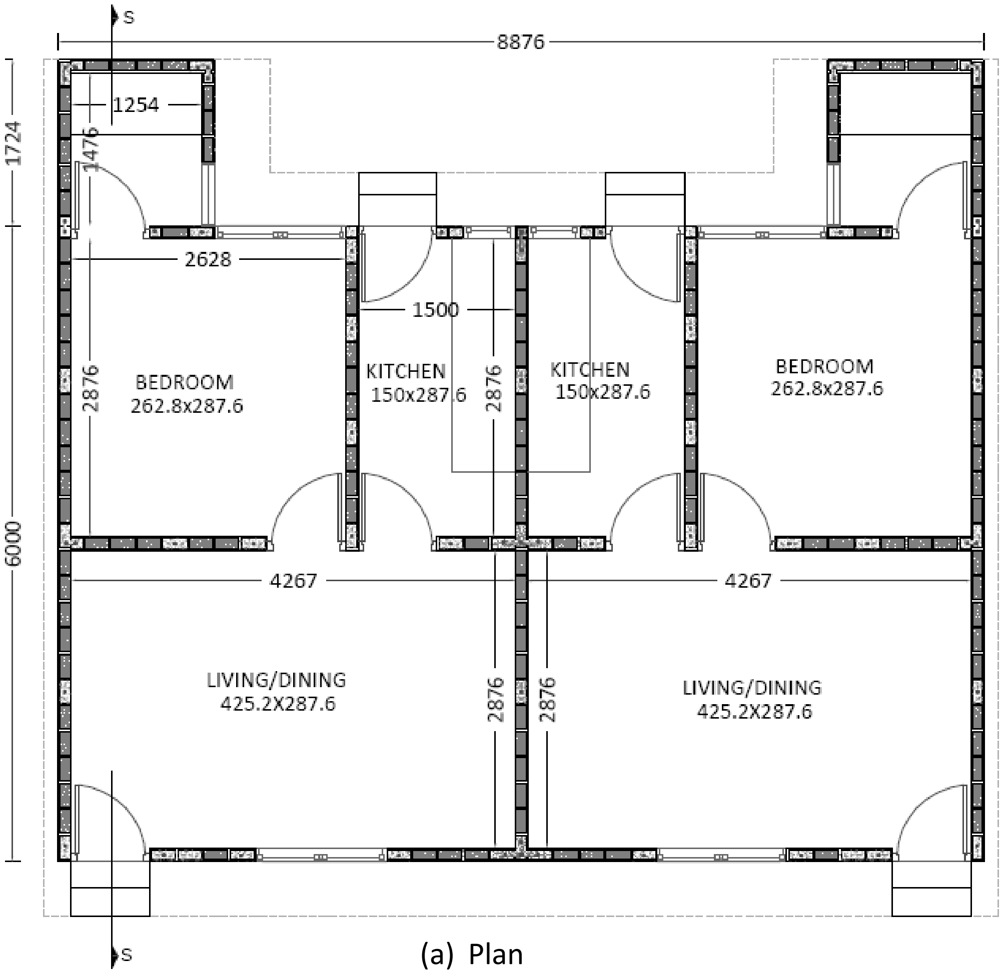


Fig. 13.15 Footing detailing

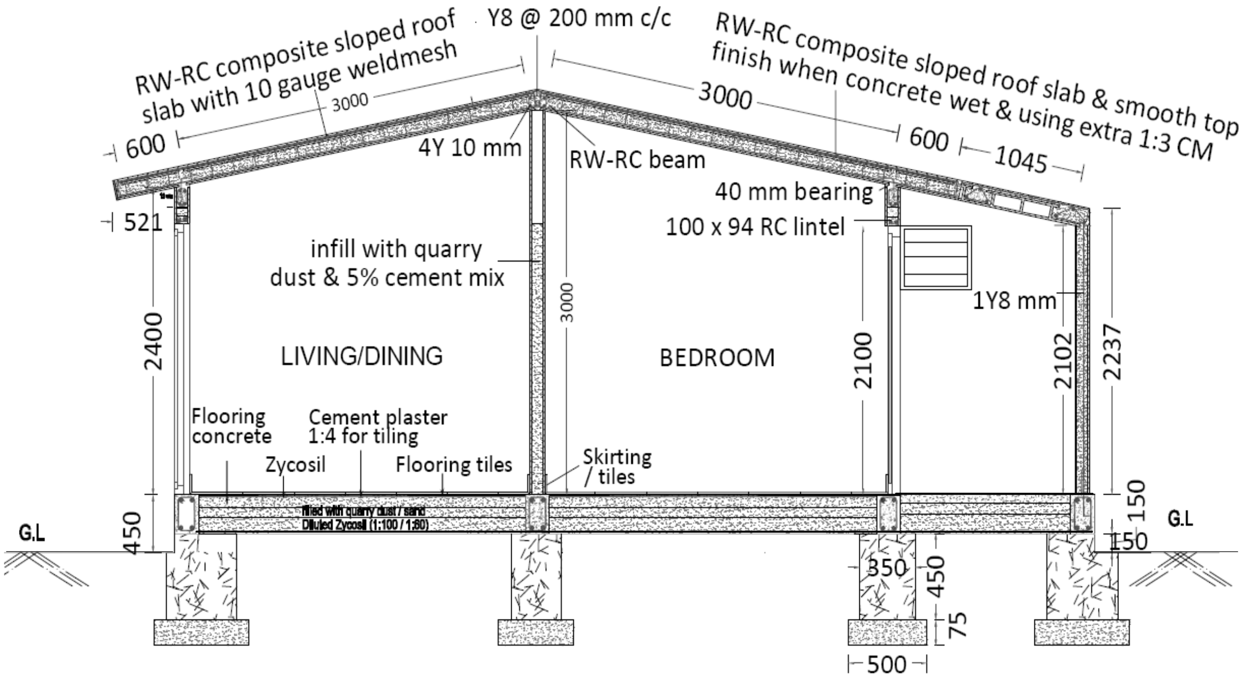
14. LOW RISE AFFORDABLE MASS HOUSING

In Chapter 12, the design of an 8 storeyed building has been presented, in which the shear wall behaviour demands all the cavities to be infilled with concrete and reinforced with two rods each to resist the high lateral force. However, in low rise buildings (up to 4 storeys), the lateral load demand is significantly less, and economy can be achieved by using partially infilled GFRG panels. In this Chapter, the design of single storey, two storeyed, three storeyed and four storeyed houses, using this concept, is demonstrated. A typical ground floor plan of a single storey house is shown in Fig. 14.1a, which is slightly different from two-, three- and four-storeyed houses shown in Fig. 14.2 (on account of space for staircase). The plans of the remaining floors are identical for two, three and four storeyed houses, as shown in Fig. 14.3. The sectional view of single storey and four storeyed houses are shown in Figs 14.1b and 14.4, respectively. The detailed calculation for a typical single storey house, suitable for affordable mass housing is shown separately in Annexure 2.

All houses considered here are assumed to be constructed in seismic zone III, with the soil type II, response reduction factor R = 3, and importance factor I = 3, as per IS 1893 (Part 1) 2002. The seismic design of these houses are done in compliance with IS 1893 (Part 1) 2002. The wind loads are calculated as per IS 875 (Part 3), the basic wind speed of 44 m/s and category 2 are being assumed. The imposed load on the floors and roof are taken as 2.0 kN/m^2 and 0.75 kN/m^2 respectively (as per IS 875- 2003, Part II). The floor finish load is taken as 1 kN/m^2. The design of wall elements alone is demonstrated in this Chapter, since the design of slab under gravity loading is the same for all the houses which, was already described in Chapter 12.



(a) Plan



(b) Sectional elevation

Fig. 14.1 Typical single storey house (Duplex)

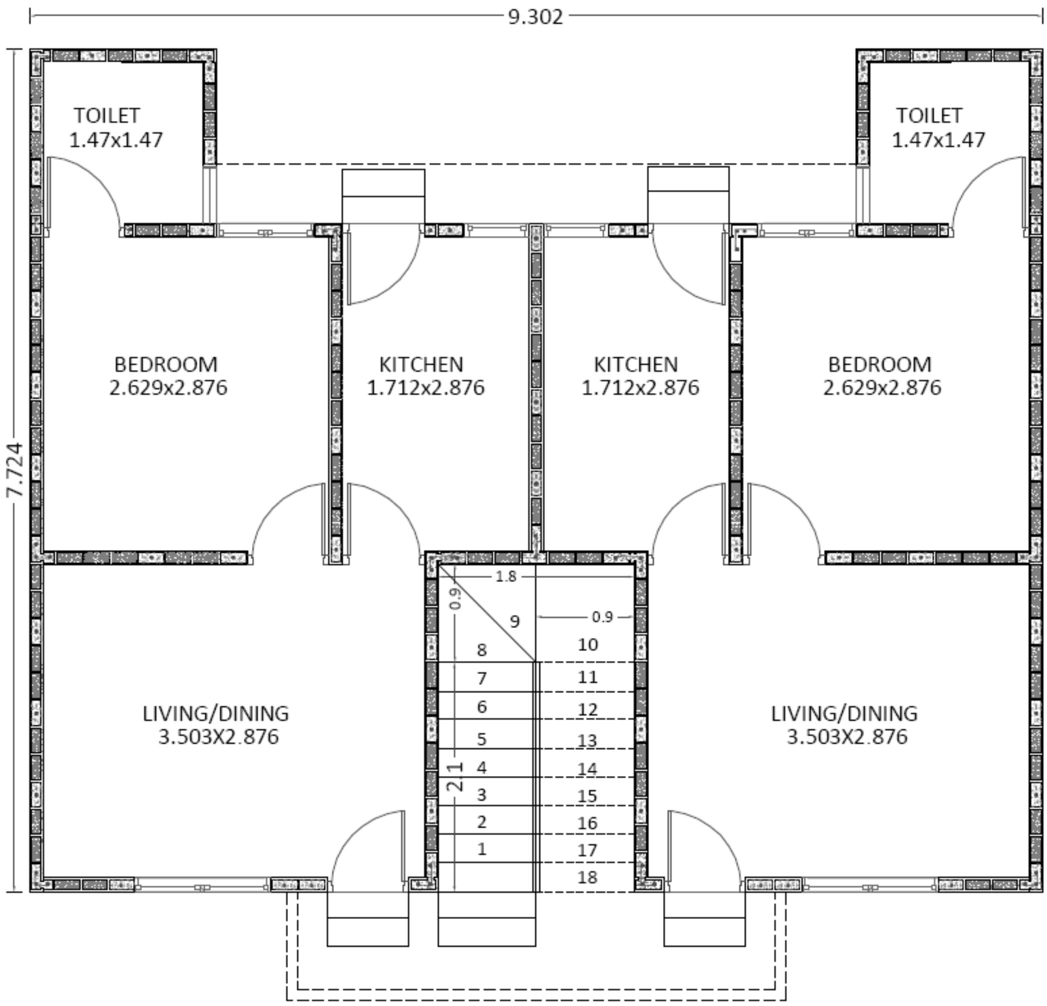


Fig. 14.2 Ground floor plan of two, three, and four storeyed housing

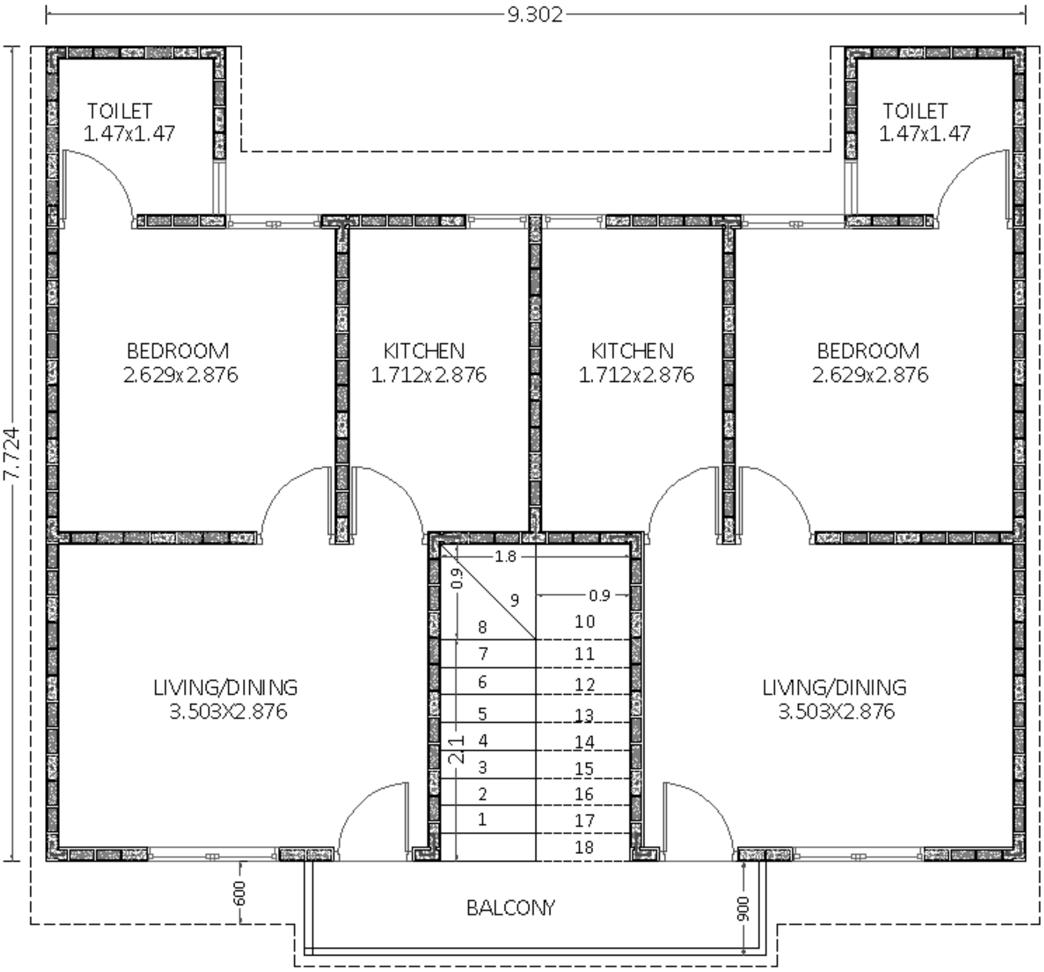


Fig. 14.3 Plan of floors above ground level for two, three and four storeyed housing

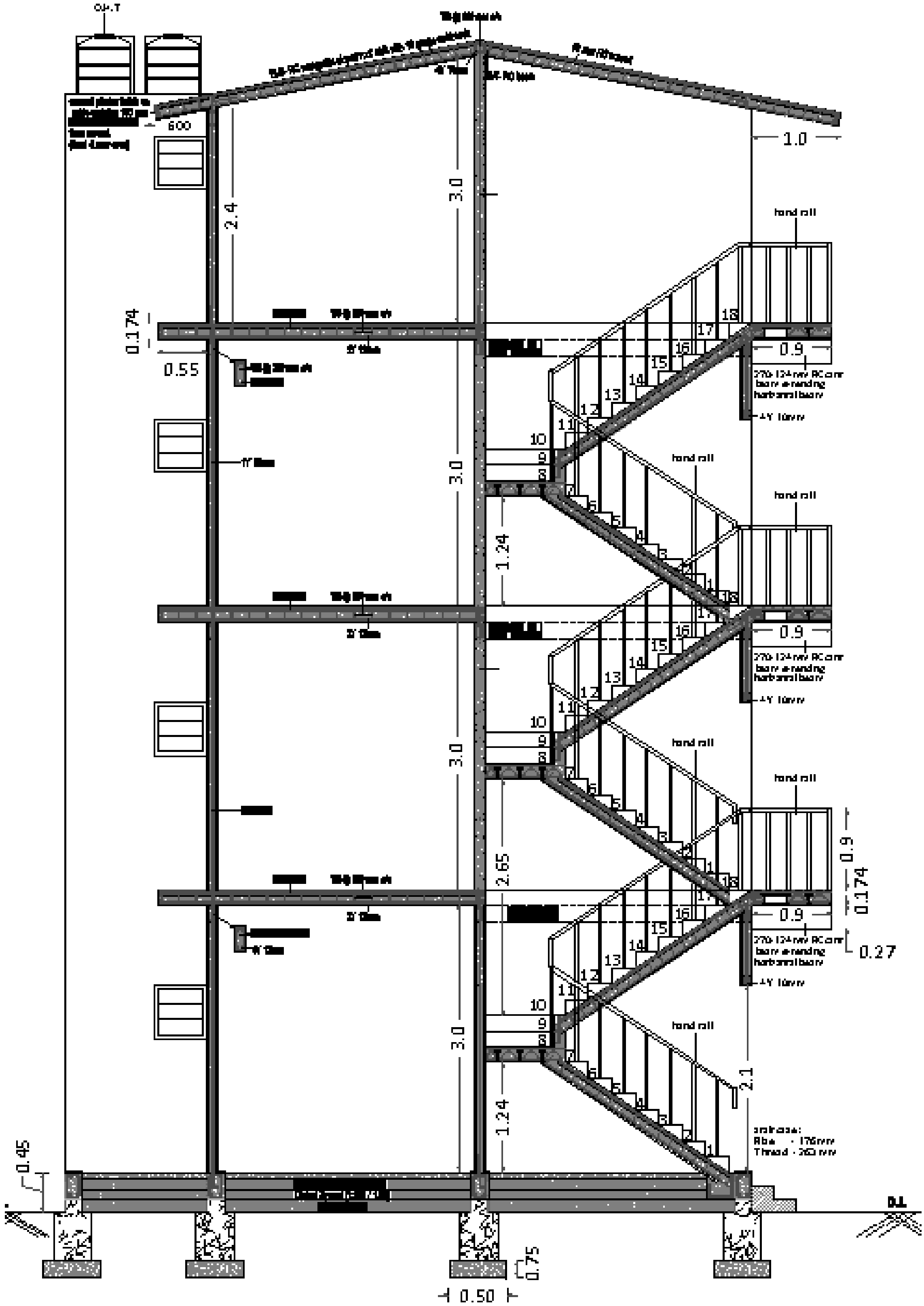


Fig. 14.4 Typical sectional elevation for four storeyed housing

14.1 DESIGN OF LOW/MEDIUM RISE GFRG HOUSING

The design calculations presented here are given in detail for the four storeyed building (refer Figs 14.3 and 14.4), for single storey, two storeyed and three storeyed housing, similar calculations are carried out, and the results are summarised in Table 14.1

14.1.1 Load Calculations

Wind load

Basic wind speed (V_b) = 44 m/s

Risk coefficient (k_1) = 1.0

Terrain, height and structure factor (k_2) = 1.03 (category 2 and Class A)

Topography factor (k_3) = 1.0

Design wind speed (V_z) = V_b. $k_1.k_2.k_3$

= 45.3 m/s

Height of the building = 13.2 m

Design wind pressure (P_d) = 0.6 V_z^2 = 1.23 kN/m^2.

Seismic Load

Zone : III

Zone factor (Z) = 0.16

Length of the building: 9.3 m

Width of the building: 7.7 m

Height of the building: 13.2 m

Design horizontal seismic coefficient $A_h = \frac{ZI}{2R}\left(\frac{S_a}{g}\right)$

Importance factor (I) = 1.0

Response reduction factor (R) = 3.0 (page 23 of IS 1893-2002)

$T_{ax} = \frac{0.09h}{\sqrt{d}}$ = $\frac{0.09\times13.2}{\sqrt{9.302}}$ = 0.04 s

$\Rightarrow S_a/g$ = 2.50 (from Fig.2 of IS 1893 Part 1: 2002)

Horizontal seismic coefficient, $A_h = \frac{0.16\times1\times2.50}{2\times3}$ = 0.0667

Similarly for Y direction, $T_{ay} = \frac{0.09h}{\sqrt{d}}$ = $\frac{0.09\times13.2}{\sqrt{7.724}}$ = 0.43 s

$\Rightarrow S_a/g$ = 2.5 (from Fig. 2 of IS 1893 Part 1: 2002)

Horizontal seismic coefficient, $A_h = \dfrac{0.16 \times 1 \times 2.50}{2 \times 3}$ = 0.0667

Calculation of Base shear

Total seismic weight (DL + 0.25 LL), *W* = 2649.0 kN

Design seismic base shear along X–direction:

$V_B = A_h W = 0.0667 \times 1949$ = 176.7 kN

Design seismic base shear along Y–direction:

$V_B = A_h W = 0.0667 \times 1949$ = 176.7 kN

14.1.2 Preliminary Design

Check for adequacy of single wall GFRG panels

Total length of GFRG wall in X-direction L_x = 13.0 m

Total length of GFRG wall in Y-direction L_y = 33.0 m

Average design shear force

in X-direction $= V_{Bx} / L_x =$ = 130/13.0 = 13.40 kN/m

in Y-direction $= V_{By} / L_y$ = 130/33.0 = 5.40 kN/m

Table 14.1 Summary of preliminary design of one, two, three and four storeyed GFRG houses

Sl. No.	Parameters	Single storeyed	Two storeyed	Three storeyed	Four storeyed
1	Length (m)	8.88	9.30	9.30	9.30
2	Width (m)	7.72	7.72	7.72	7.72
3	Height (m)	3.00	6.50	9.90	13.20
4	T_a in X dir. (s)	0.09	0.19	0.30	0.40
5	T_a in Y dir. (s)	0.09	0.21	0.32	0.43
6	A_h in X dir.	0.0629	0.0667	0.0667	0.0667
7	A_h in Y dir.	0.0656	0.0667	0.0667	0.0667
8	Total seismic wt. (kN)	586.0	1259.0	1949.0	2649
9	V_B in X dir. (kN)	36.8	84.0	130.0	176.7
10	V_B in Y dir. (kN)	38.5	84.0	130.0	176.7
11	L_X (m)	12.20	13.00	13.00	13.00
12	L_Y (m)	29.00	33.00	33.00	33.00
13	Avg. Design shear force per unit length of GFRG panel in X dir. (kN/m)	3.02	6.50	10.00	13.40
14	Avg. Design shear force per unit length of GFRG panel in Y dir. (kN/m)	1.33	2.50	4.00	5.40
15	Design shear capacity of empty GFRG panels per unit length (kN/m)	14.4	14.4	14.4	14.4

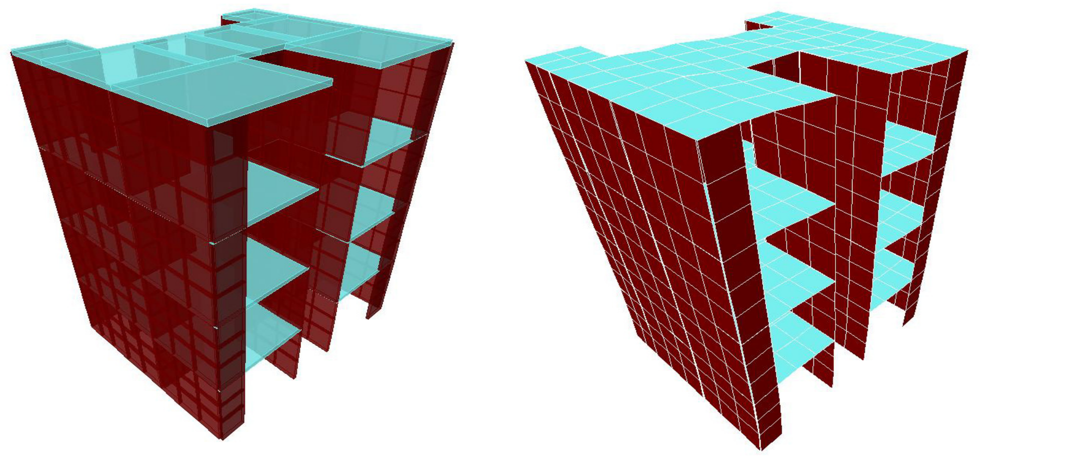
From the above table, it can be seen that the average design in-plane shear force values are less than the design shear capacity of 14.4 kN/m (refer Table 9.1) for unfilled GFRG wall panels. Therefore, single wall empty GFRG panels are sufficient to ensure safety against lateral loads, for all houses.

As shown in Chapter 8, the unfilled GFRG panel has the out of plane bending capacity to resist wind load corresponding to basic wind speed of 37.2 m/s. When all the cavities are infilled with plain concrete of M20 grade, the design wind speed is enhanced to 52 m/s. In this case, the design wind speed is 44 m/s, and it can be shown that partially filled GFRG panels (with M20 concrete fill in every fourth cavity and reinforced with one bar of 10 mm diameter) has the capacity to resist the basic wind speed of 44 m/s. Here, all the cavities left unfilled, are to be filled with a controlled low strength material (mixture of quarry dust and cement grout). This will also contribute to enhancement of out of plane bending moment capacity.

14.1.3 Detailed Analysis and Design of Wall Segments

Modelling and Analysis of the building

The computational model of all houses has been developed in the software ETABS as per the architectural plan (refer Figs 14.1, 14.3 and 14.4). The various assumptions considered for the analysis of the given GFRG building are as given in 13.3.1.



(a) 3D Model (b) Deformed shape under seismic force

Fig. 14.5 3D finite element model and deformed shape of four storeyed houses

The length of each wall element and its label assigned in ETAB model for single storey house is shown in Fig. 14.6. The length of each wall element and its label assigned in ETAB model for ground floor is same for two, three and four storeyed houses as shown in Fig. 14.7, and the remaining floors of two, three and four storeyed houses are shown in Fig.14.8.

The results of the values of maximum shear force, maximum bending moment and corresponding axial force in each wall segment (pier) for all houses including all storeys are summarized in Tables 14.2 to 14.11, for the critical load combinations for wall elements.

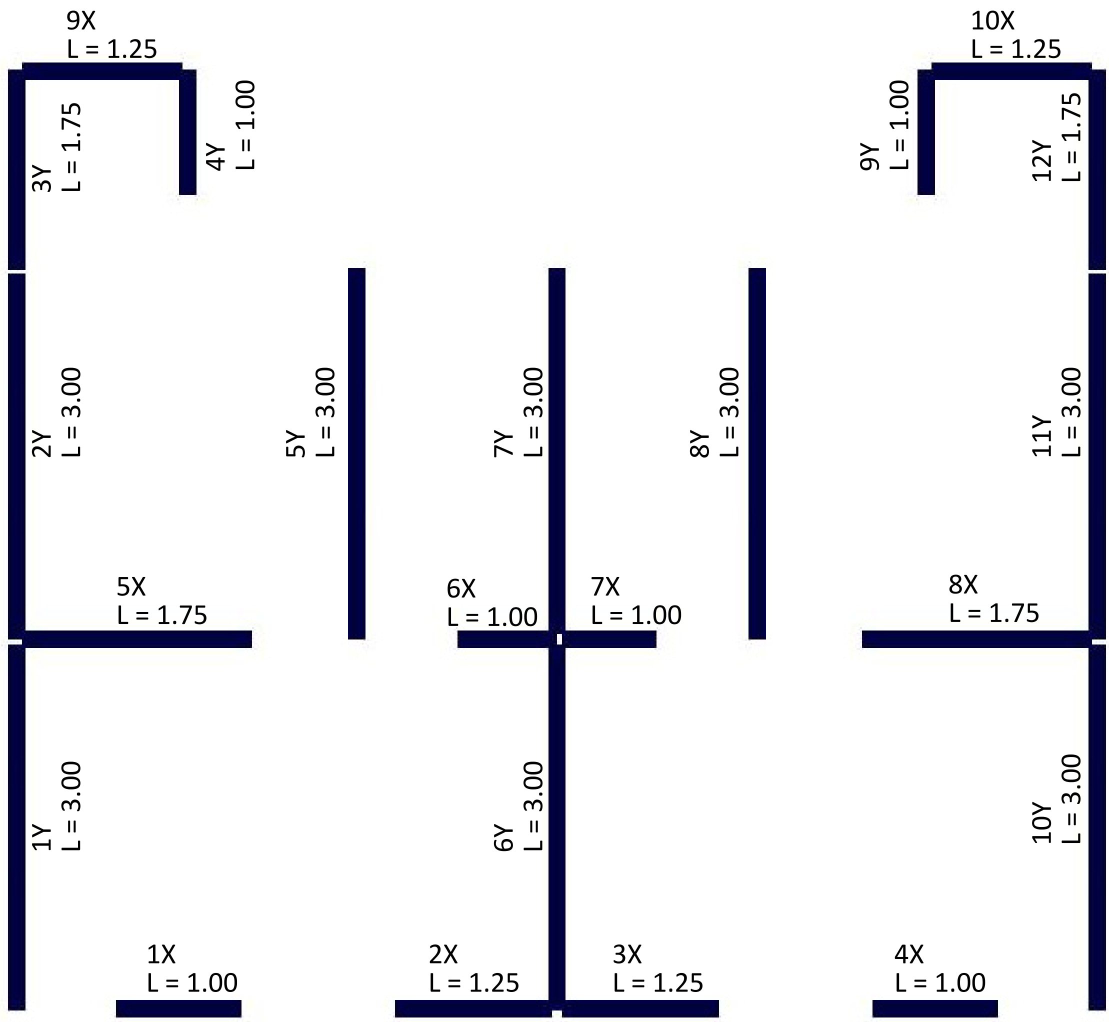


Fig. 14.6 Label and length of wall elements: Single storeyed house

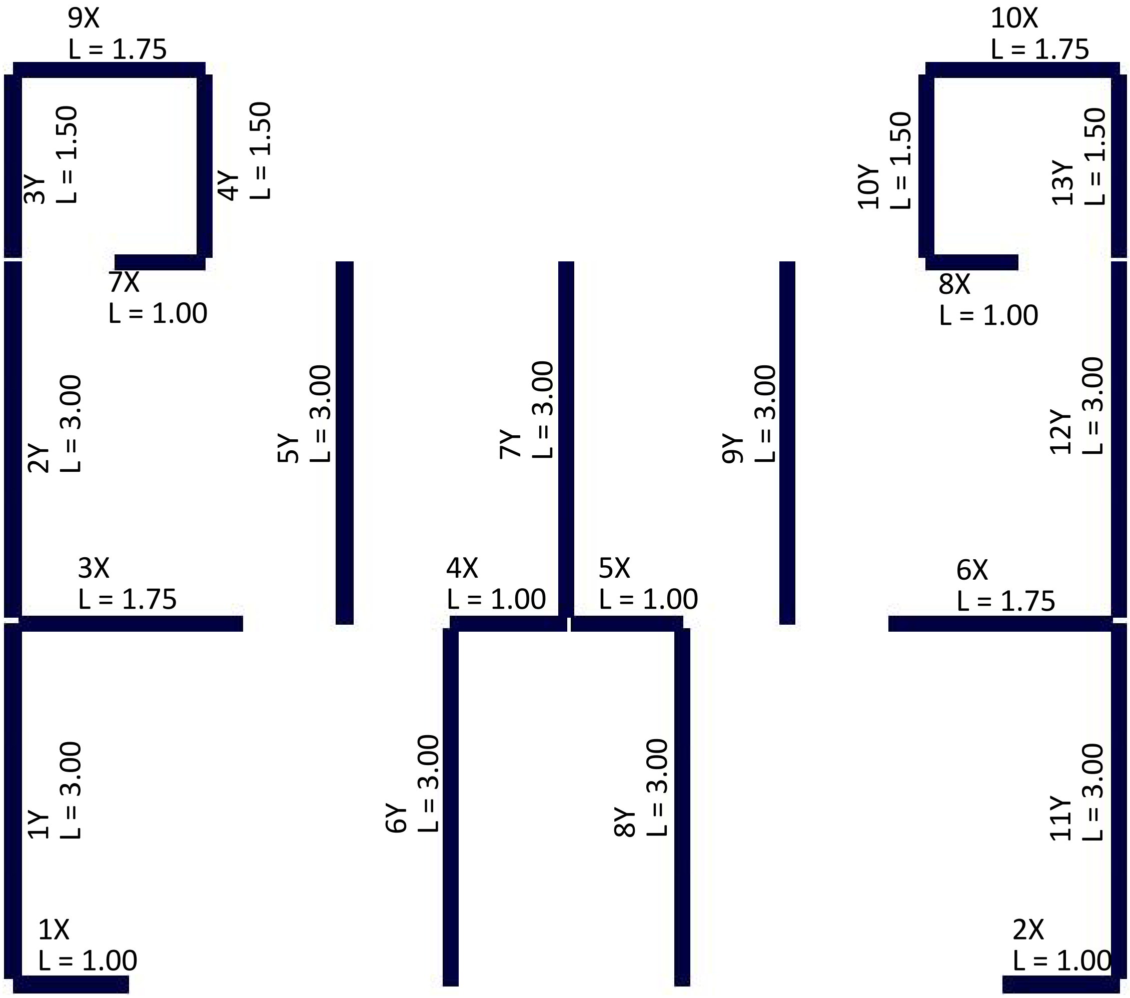


Fig. 14.7 Ground Floor: Label and length of wall elements of two, three and four storeyed house

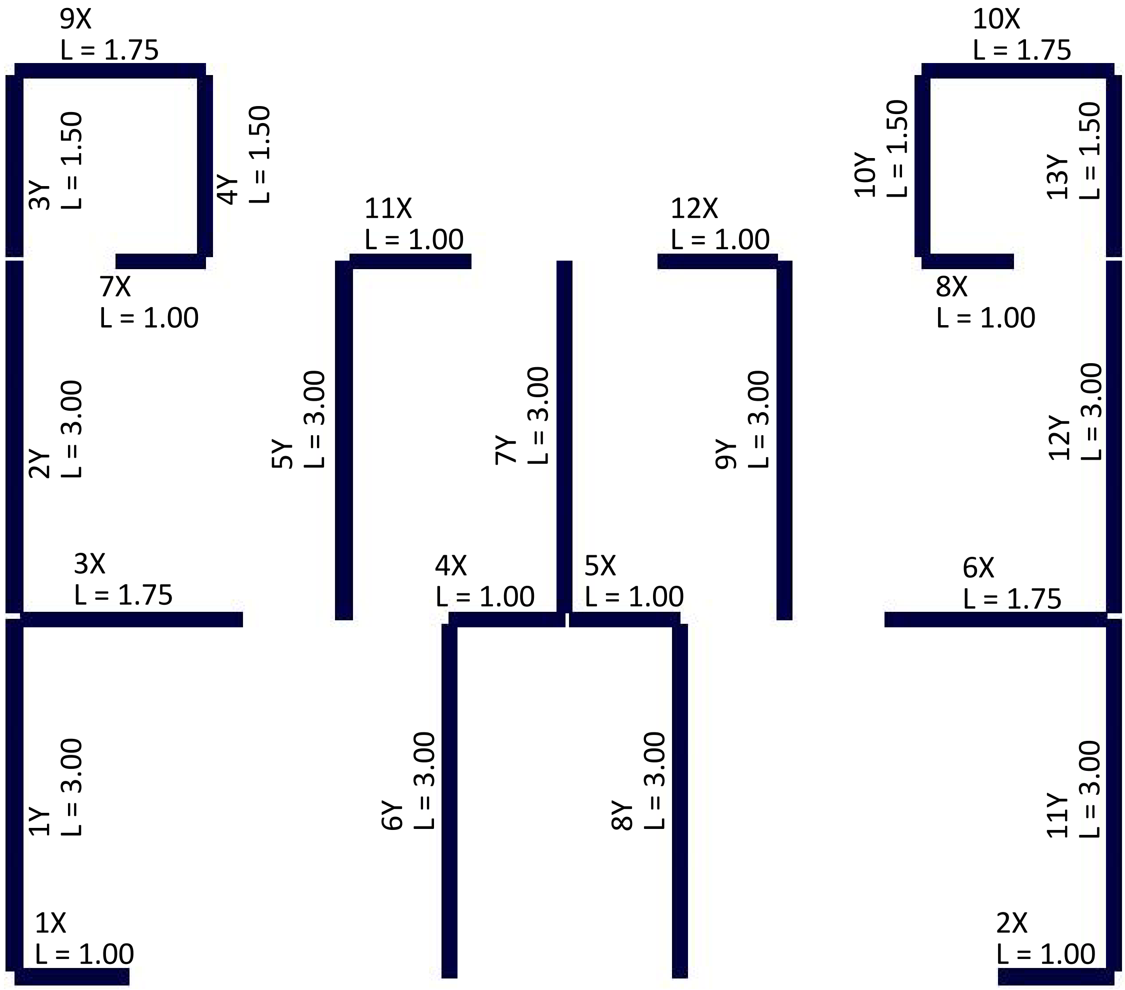


Fig. 14.8 Above Floor: Label and length of wall elements of two, three and four storeyed house

Table 14.2 Single storeyed house: Maximum shear force per unit length (V_u, kN/m) and corresponding bending moments (M_u, kN-m) and axial forces (P_u, kN) of walls in X and Y direction

Name	Length (m)	Number of cavities	Infilling required (addition to end cavities)	η	Shear resistance V_R (kN/m)	Design shear force V_U (kN/m)	Design axial force P_U (kN)	Design In-plane moment M_U (kNm)
1X	1.00	4	0	1/2	27	1	25	3
2X	1.25	5	0	2/5	25	6	10	13
3X	1.25	5	0	2/5	25	3	18	8
4X	1.00	4	0	1/2	27	3	34	5
5X	1.75	7	1	3/7	25	3	46	18
6X	1.00	4	0	1/2	27	3	4	4
7X	1.00	4	0	1/2	27	1	9	2
8X	1.75	7	1	3/7	25	6	39	19
9X	1.25	5	0	2/5	25	5	16	10
10X	1.25	5	0	2/5	25	4	13	9
1Y	3.00	12	2	1/3	23	2	45	9
2Y	3.00	12	2	1/3	23	1	13	13
3Y	1.75	7	1	3/7	25	2	4	8
4Y	1.00	4	0	1/2	27	8	30	18
5Y	3.00	12	2	1/3	23	1	79	15
6Y	3.00	12	2	1/3	23	1	50	8
7Y	3.00	12	2	1/3	23	2	34	22
8Y	3.00	12	2	1/3	23	2	44	15
9Y	1.00	4	0	1/2	27	8	23	19
10Y	3.00	12	2	1/3	23	2	42	11
11Y	3.00	12	2	1/3	23	2	51	11
12Y	1.75	7	1	3/7	25	2	9	3

Table 14.3 Two storeyed house: Maximum shear force per unit length (V_u, kN/m) and corresponding bending moments (M_u, kN-m) and axial forces (P_u, kN) of walls of storey 1

Name	Length (m)	Number of cavities	Infilling required (addition to end cavities)	η	Shear resistance V_R (kN/m)	Design shear force V_U (kN/m)	Design axial force P_U (kN)	Design In-plane moment M_U (kNm)
1X	1.00	4	0	1/2	27	3	67	11
2X	1.00	4	0	1/2	27	8	33	16
3X	1.75	7	1	3/7	25	13	116	64
4X	1.00	4	0	1/2	27	8	38	15
5X	1.00	4	0	1/2	27	8	58	14
6X	1.75	7	1	3/7	25	15	40	65
7X	1.00	4	0	1/2	27	3	34	7
8X	1.00	4	0	1/2	27	3	35	6
9X	1.75	7	1	3/7	25	11	55	44
10X	1.75	7	1	3/7	25	11	55	45
1Y	3.00	12	2	1/3	23	5	102	30
2Y	3.00	12	2	1/3	23	5	114	29
3Y	1.50	6	0	1/3	23	4	7	9
4Y	1.50	6	0	1/3	23	3	59	8
5Y	3.00	12	2	1/3	23	3	147	35
6Y	3.00	12	2	1/3	23	4	131	29
7Y	3.00	12	2	1/3	23	4	149	29
8Y	3.00	12	2	1/3	23	4	131	29
9Y	3.00	12	2	1/3	23	3	148	35
10Y	1.50	6	0	1/3	23	3	10	9
11Y	3.00	12	2	1/3	23	5	102	30
12Y	3.00	12	2	1/3	23	5	114	29
13Y	1.50	6	0	1/3	23	4	61	9

Table 14.4 Two storeyed house: Maximum shear force per unit length (V_u, kN/m) and corresponding bending moments (M_u, kN-m) and axial forces (P_u, kN) of walls of storey 2

Name	Length (m)	Number of cavities	Infilling required (addition to end cavities)	η	Shear resistance V_R (kN/m)	Design shear force V_U (kN/m)	Design axial force P_U (kN)	Design In-plane moment M_U (kNm)
1X	1.00	4	0	1/2	27	8	15	14
2X	1.00	4	0	1/2	27	9	10	14
3X	1.75	7	1	3/7	25	5	50	16
4X	1.00	4	0	1/2	27	8	19	12
5X	1.00	4	0	1/2	27	8	22	12
6X	1.75	7	1	3/7	25	11	30	29
7X	1.00	4	0	1/2	27	3	19	4
8X	1.00	4	0	1/2	27	1	10	2
9X	1.75	7	1	3/7	25	6	27	18
10X	1.75	7	1	3/7	25	7	27	19
11X	1.00	4	0	1/2	27	4	11	8
12X	1.00	4	0	1/2	27	7	11	12
1Y	3.00	12	2	1/3	23	4	52	19
2Y	3.00	12	2	1/3	23	4	43	22
3Y	1.50	6	0	1/3	23	4	1	10
4Y	1.50	6	0	1/3	23	3	13	7
5Y	3.00	12	2	1/3	23	1	74	8
6Y	3.00	12	2	1/3	23	3	20	17
7Y	3.00	12	2	1/3	23	3	68	11
8Y	3.00	12	2	1/3	23	3	60	13
9Y	3.00	12	2	1/3	23	2	66	9
10Y	1.50	6	0	1/3	23	6	4	14
11Y	3.00	12	2	1/3	23	4	52	19
12Y	3.00	12	2	1/3	23	4	56	20
13Y	1.50	6	0	1/3	23	4	34	9

Table 14.5 Three storeyed house: Maximum shear force per unit length (V_u, kN/m) and corresponding bending moments (M_u, kN-m) and axial forces (P_u, kN) of walls of storey 1

Name	Length (m)	Number of cavities	Infilling required (addition to end cavities)	η	Shear resistance V_R (kN/m)	Design shear force V_U (kN/m)	Design axial force P_U (kN)	Design In-plane moment M_U (kNm)
1X	1.00	4	0	1/2	27	6	111	21
2X	1.00	4	0	1/2	27	11	32	25
3X	1.75	7	1	3/7	25	21	202	115
4X	1.00	4	0	1/2	27	14	48	27
5X	1.00	4	0	1/2	27	13	96	24
6X	1.75	7	1	3/7	25	23	29	116
7X	1.00	4	0	1/2	27	5	52	12
8X	1.00	4	0	1/2	27	4	46	11
9X	1.75	7	1	3/7	25	17	82	78
10X	1.75	7	1	3/7	25	17	86	78
1Y	3.00	12	2	1/3	23	8	142	55
2Y	3.00	12	2	1/3	23	8	177	53
3Y	1.50	6	1	1/2	27	6	6	17
4Y	1.50	6	1	1/2	27	4	93	14
5Y	3.00	12	2	1/3	23	4	229	64
6Y	3.00	12	2	1/3	23	6	194	54
7Y	3.00	12	2	1/3	23	6	244	54
8Y	3.00	12	2	1/3	23	6	195	54
9Y	3.00	12	2	1/3	23	4	229	64
10Y	1.50	6	1	1/2	27	5	0	15
11Y	3.00	12	2	1/3	23	8	142	55
12Y	3.00	12	2	1/3	23	8	177	53
13Y	1.50	6	1	1/2	27	6	130	15

Table 14.6 Three storeyed house: Maximum shear force per unit length (V_u, kN/m) and corresponding bending moments (M_u, kN-m) and axial forces (P_u, kN) of walls of storey 2

Name	Length (m)	Number of cavities	Infilling required (addition to end cavities)	η	Shear resistance V_R (kN/m)	Design shear force V_U (kN/m)	Design axial force P_U (kN)	Design In-plane moment M_U (kNm)
1X	1.00	4	0	1/2	27	7	31	11
2X	1.00	4	0	1/2	27	10	31	17
3X	1.75	7	1	3/7	25	15	114	54
4X	1.00	4	0	1/2	27	16	37	27
5X	1.00	4	0	1/2	27	16	53	26
6X	1.75	7	1	3/7	25	20	39	66
7X	1.00	4	0	1/2	27	4	40	7
8X	1.00	4	0	1/2	27	3	23	4
9X	1.75	7	1	3/7	25	13	55	44
10X	1.75	7	1	3/7	25	15	58	46
11X	1.00	4	0	1/2	27	5	22	11
12X	1.00	4	0	1/2	27	11	19	21
1Y	3.00	12	2	1/3	23	8	101	40
2Y	3.00	12	2	1/3	23	9	62	47
3Y	1.50	6	1	1/2	27	6	21	16
4Y	1.50	6	1	1/2	27	5	86	13
5Y	3.00	12	2	1/3	23	2	143	19
6Y	3.00	12	2	1/3	23	6	61	35
7Y	3.00	12	2	1/3	23	5	117	24
8Y	3.00	12	2	1/3	23	5	129	27
9Y	3.00	12	2	1/3	23	3	156	18
10Y	1.50	6	1	1/2	27	8	10	19
11Y	3.00	12	2	1/3	23	8	101	40
12Y	3.00	12	2	1/3	23	9	118	46
13Y	1.50	6	1	1/2	27	7	85	17

Table 14.7 Three storeyed house: Maximum shear force per unit length (V_u, kN/m) and corresponding bending moments (M_u, kN-m) and axial forces (P_u, kN) of walls of storey 3

Name	Length (m)	Number of cavities	Infilling required (addition to end cavities)	η	Shear resistance V_R (kN/m)	Design shear force V_U (kN/m)	Design axial force P_U (kN)	Design In-plane moment M_U (kNm)
1X	1.00	4	0	1/2	27	8	15	14
2X	1.00	4	0	1/2	27	9	12	15
3X	1.75	7	1	3/7	25	5	20	17
4X	1.00	4	0	1/2	27	13	20	19
5X	1.00	4	0	1/2	27	12	20	19
6X	1.75	7	1	3/7	25	12	12	35
7X	1.00	4	0	1/2	27	3	13	4
8X	1.00	4	0	1/2	27	1	9	2
9X	1.75	7	1	3/7	25	6	8	16
10X	1.75	7	1	3/7	25	8	9	20
11X	1.00	4	0	1/2	27	4	1	7
12X	1.00	4	0	1/2	27	6	6	12
1Y	3.00	12	2	1/3	23	5	51	23
2Y	3.00	12	2	1/3	23	6	41	32
3Y	1.50	6	1	1/2	27	5	19	12
4Y	1.50	6	1	1/2	27	4	33	10
5Y	3.00	12	2	1/3	23	1	68	10
6Y	3.00	12	2	1/3	23	6	51	29
7Y	3.00	12	2	1/3	23	3	33	15
8Y	3.00	12	2	1/3	23	5	78	25
9Y	3.00	12	2	1/3	23	2	57	18
10Y	1.50	6	1	1/2	27	8	7	21
11Y	3.00	12	2	1/3	23	5	51	23
12Y	3.00	12	2	1/3	23	6	57	27
13Y	1.50	6	1	1/2	27	5	34	14

Table 14.8 Four storeyed house: Maximum shear force per unit length (V_u, kN/m) and corresponding bending moments (M_u, kN-m) and axial forces (P_u, kN) of walls of storey 1

Name	Length (m)	Number of cavities	Infilling required (addition to end cavities)	η	Shear resistance V_R (kN/m)	Design shear force V_U (kN/m)	Design axial force P_U (kN)	Design In-plane moment M_U (kNm)
1X	1.00	4	0	1/2	27	9	159	31
2X	1.00	4	0	1/2	27	14	27	36
3X	1.75	7	3	**5/7**	33	29	298	170
4X	1.00	4	0	1/2	27	20	53	39
5X	1.00	4	0	1/2	27	18	139	35
6X	1.75	7	3	**5/7**	33	31	10	171
7X	1.00	4	0	1/2	27	7	72	16
8X	1.00	4	0	1/2	27	6	54	16
9X	1.75	7	1	3/7	25	23	107	114
10X	1.75	7	1	3/7	25	23	117	115
1Y	3.00	12	2	1/3	23	11	168	84
2Y	3.00	12	2	1/3	23	12	243	81
3Y	1.50	6	1	1/2	27	8	30	25
4Y	1.50	6	1	1/2	27	5	132	20
5Y	3.00	12	2	1/3	23	5	314	98
6Y	3.00	12	2	1/3	23	8	248	84
7Y	3.00	12	2	1/3	23	8	348	83
8Y	3.00	12	2	1/3	23	8	250	84
9Y	3.00	12	2	1/3	23	5	314	98
10Y	1.50	6	1	1/2	27	6	133	20
11Y	3.00	12	2	1/3	23	11	168	84
12Y	3.00	12	2	1/3	23	12	243	81
13Y	1.50	6	1	1/2	27	8	208	23

Table 14.9 Four storeyed house: Maximum shear force per unit length (V_u, kN/m) and corresponding bending moments (M_u, kN-m) and axial forces (P_u, kN) of walls of storey 2

Name	Length (m)	Number of cavities	Infilling required (addition to end cavities)	η	Shear resistance V_R (kN/m)	Design shear force V_U (kN/m)	Design axial force P_U (kN)	Design In-plane moment M_U (kNm)
1X	1.00	4	0	1/2	27	8	38	12
2X	1.00	4	0	1/2	27	12	36	24
3X	1.75	7	1	3/7	25	22	189	95
4X	1.00	4	0	1/2	27	25	49	43
5X	1.00	4	0	1/2	27	23	90	40
6X	1.75	7	3	**5/7**	33	27	39	107
7X	1.00	4	0	1/2	27	5	61	10
8X	1.00	4	0	1/2	27	4	30	8
9X	1.75	7	1	3/7	25	19	79	72
10X	1.75	7	1	3/7	25	21	89	75
11X	1.00	4	0	1/2	27	8	37	16
12X	1.00	4	0	1/2	27	15	25	30
1Y	3.00	12	2	1/3	23	12	136	67
2Y	3.00	12	2	1/3	23	14	182	76
3Y	1.50	6	1	1/2	27	10	7	25
4Y	1.50	6	1	1/2	27	8	143	21
5Y	3.00	12	2	1/3	23	3	215	34
6Y	3.00	12	2	1/3	23	9	77	56
7Y	3.00	12	2	1/3	23	7	243	41
8Y	3.00	12	2	1/3	23	7	190	46
9Y	3.00	12	2	1/3	23	4	235	28
10Y	1.50	6	1	1/2	27	11	4	28
11Y	3.00	12	2	1/3	23	12	136	67
12Y	3.00	12	2	1/3	23	14	181	76
13Y	1.50	6	1	1/2	27	10	131	25

Table 14.10 Four storeyed house: Maximum shear force per unit length (V_u, kN/m) and corresponding bending moments (M_u, kN-m) and axial forces (P_u, kN) of walls of storey 3

Name	Length (m)	Number of cavities	Infilling required (addition to end cavities)	η	Shear resistance V_R (kN/m)	Design shear force V_U (kN/m)	Design axial force P_U (kN)	Design In-plane moment M_U (kNm)
1X	1.00	4	0	1/2	27	7	29	11
2X	1.00	4	0	1/2	27	10	37	16
3X	1.75	7	1	3/7	25	16	97	48
4X	1.00	4	0	1/2	27	23	39	37
5X	1.00	4	0	1/2	27	22	50	35
6X	1.75	7	1	3/7	25	22	55	62
7X	1.00	4	0	1/2	27	4	43	7
8X	1.00	4	0	1/2	27	3	18	5
9X	1.75	7	1	3/7	25	14	52	41
10X	1.75	7	1	3/7	25	16	60	46
11X	1.00	4	0	1/2	27	9	18	15
12X	1.00	4	0	1/2	27	12	18	19
1Y	3.00	12	2	1/3	23	10	98	46
2Y	3.00	12	2	1/3	23	12	62	62
3Y	1.50	6	1	1/2	27	8	26	19
4Y	1.50	6	1	1/2	27	7	77	19
5Y	3.00	12	2	1/3	23	2	97	19
6Y	3.00	12	2	1/3	23	9	53	41
7Y	3.00	12	2	1/3	23	7	115	33
8Y	3.00	12	2	1/3	23	7	203	42
9Y	3.00	12	2	1/3	23	4	115	27
10Y	1.50	6	1	1/2	27	11	37	27
11Y	3.00	12	2	1/3	23	10	98	46
12Y	3.00	12	2	1/3	23	12	119	57
13Y	1.50	6	1	1/2	27	8	80	23

Table 14.11 Four storeyed house: Maximum shear force per unit length (V_u, kN/m) and corresponding bending moments (M_u, kN-m) and axial forces (P_u, kN) of walls of storey 4

Name	Length (m)	Number of cavities	Infilling required (addition to end cavities)	η	Shear resistance V_R (kN/m)	Design shear force V_U (kN/m)	Design axial force P_U (kN)	Design In-plane moment M_U (kNm)
1X	1.00	4	0	1/2	27	8	15	15
2X	1.00	4	0	1/2	27	8	15	15
3X	1.75	7	1	3/7	25	6	20	18
4X	1.00	4	0	1/2	27	16	10	25
5X	1.00	4	0	1/2	27	16	7	25
6X	1.75	7	1	3/7	25	12	20	40
7X	1.00	4	0	1/2	27	3	12	6
8X	1.00	4	0	1/2	27	2	3	3
9X	1.75	7	1	3/7	25	4	7	12
10X	1.75	7	1	3/7	25	7	9	19
11X	1.00	4	0	1/2	27	7	0	14
12X	1.00	4	0	1/2	27	10	11	18
1Y	3.00	12	2	1/3	23	5	50	24
2Y	3.00	12	2	1/3	23	7	42	36
3Y	1.50	6	1	1/2	27	6	30	12
4Y	1.50	6	1	1/2	27	5	27	12
5Y	3.00	12	2	1/3	23	2	29	11
6Y	3.00	12	2	1/3	23	7	50	39
7Y	3.00	12	2	1/3	23	4	31	19
8Y	3.00	12	2	1/3	23	7	79	37
9Y	3.00	12	2	1/3	23	2	51	20
10Y	1.50	6	1	1/2	27	10	12	24
11Y	3.00	12	2	1/3	23	5	50	24
12Y	3.00	12	2	1/3	23	7	57	32
13Y	1.50	6	1	1/2	27	6	31	16

Shear Check on GFRG Wall Panels

In four storeyed house, every third cavity is infilled with M20 concrete and reinforced with single Fe 415 dia bar, except in walls named 3X and 6X of storeys 1 and 2 where 5 out of 7 cavities are infilled . The design shear capacity of each GFRG panel partially infilled with concrete for all houses including all storeys is shown in

Tables 14.8 to 14.11. It can be seen from the analysis results that the design shear forces in all pier walls are less than the design strength. Hence, the design is safe against lateral shear force

Check on In-Plane Moment Capacity of GFRG Wall Panels

The resultant axial force and in-plane bending moment in each wall of all houses, under various load combinations, are checked with the corresponding P-M interaction curves.

The demand of axial force and bending moment of each GFRG wall panel of single storey house (Demand 1 S), two storeyed house (Demand 2 S), three storeyed house (Demand 3 S) and four storeyed house (Demand 4 S), corresponding to the capacity curves for various lengths of walls, 1.0m, 1.5m, 1.75m and 3.50m are shown in Figs. 14.09 to 14.13.

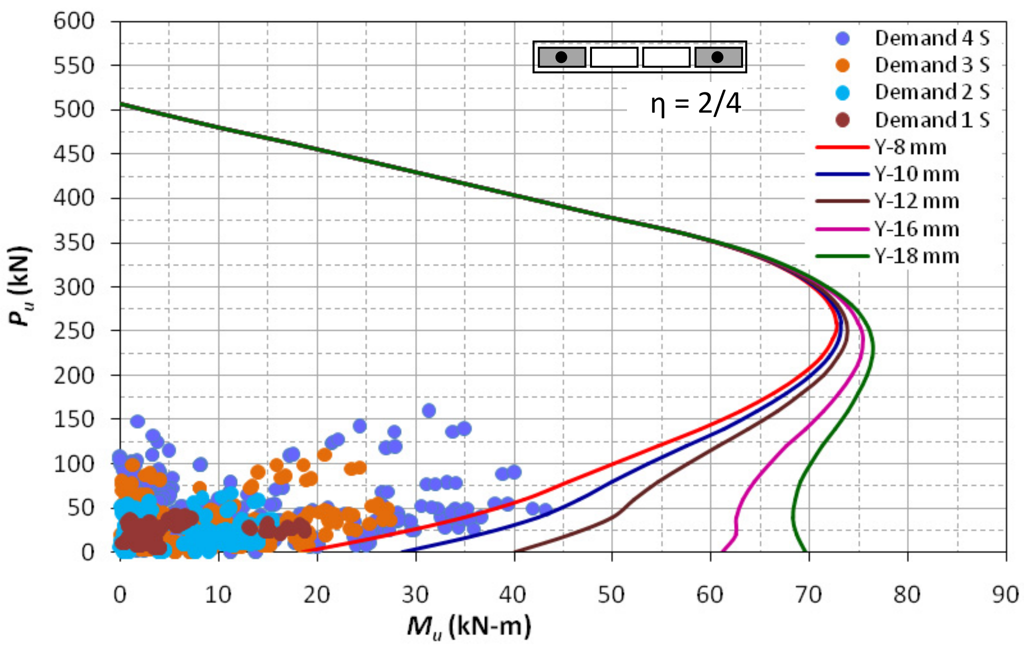
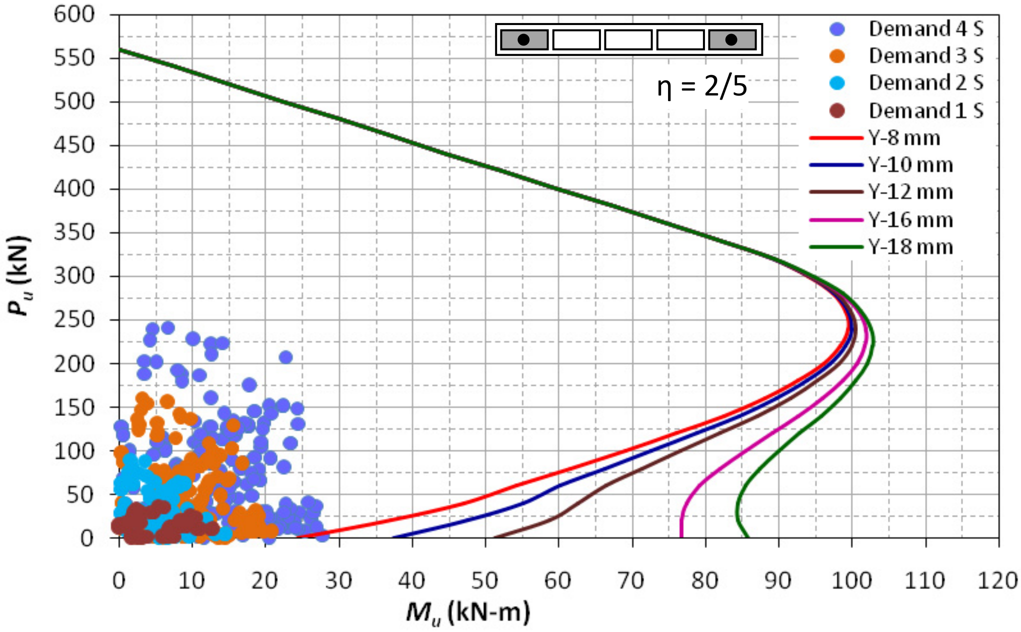


Fig.14.9 Demand points and capacity envelope for **1.02 m wide** panel



`

Fig.14.10 Demand points and capacity envelope for **1.50 m wide** panel

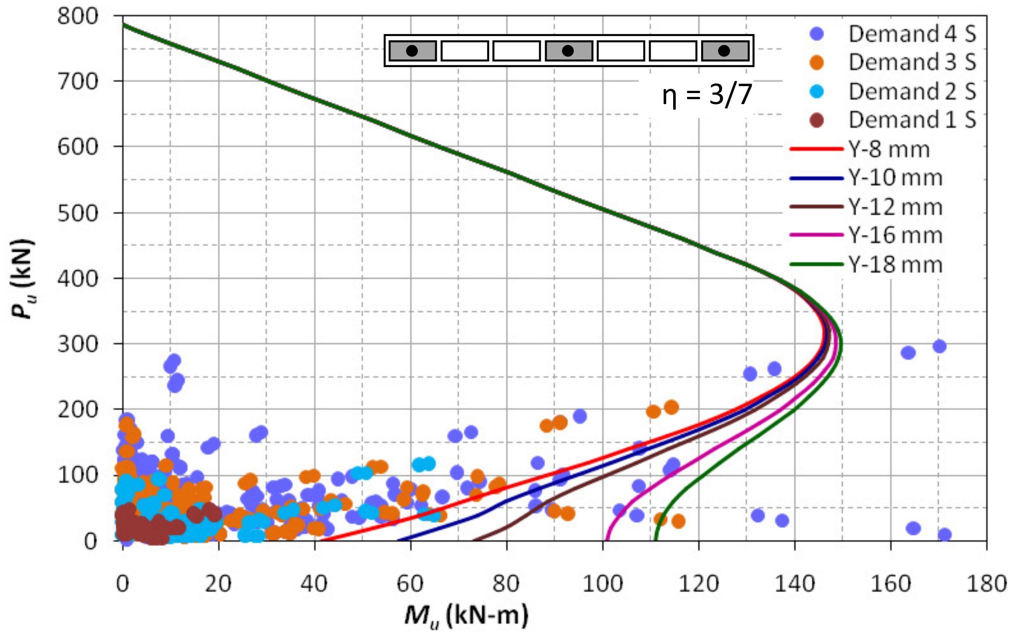


Fig.14.11 Demand points and capacity envelope for **1.75 m wide** panel with 2 number of rebar in each cell

The points which lie outside the capacity curve of 1.75m wide panels with Y-8 mm, are the demand points for the wall elements named 3X and 6X (refer Fig. 14.11) of storey 1 of the three and four storeyed house, and storey 2 of the four storeyed house. Hence all cavities of these walls are to be infilled with concrete and 2Y-8 mm bars to meet the demand which is shown in Fig.14.12.

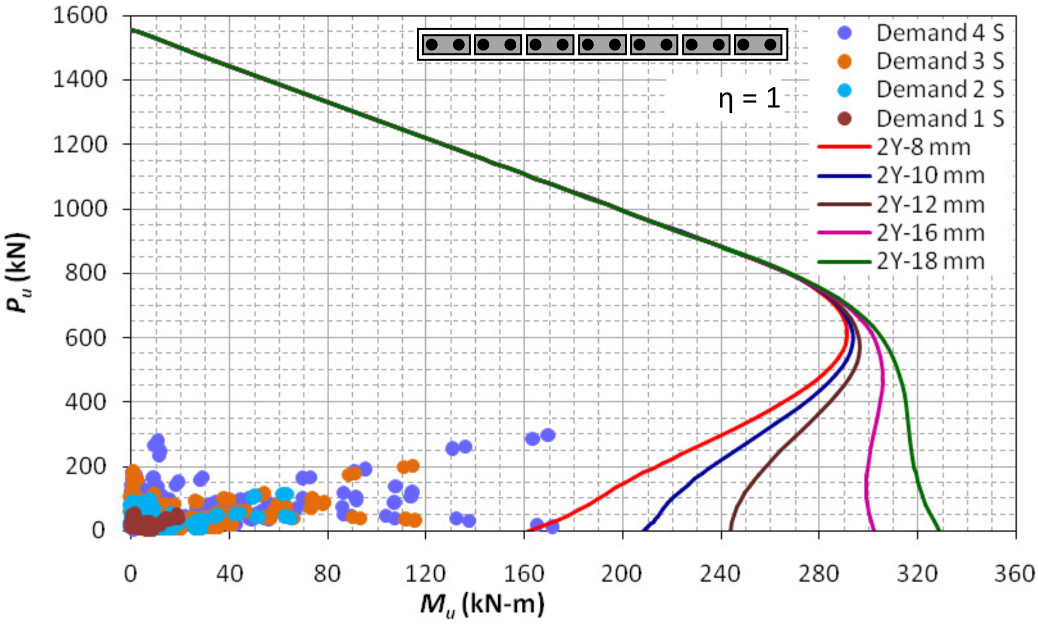


Fig.14.12 Demand points and capacity envelope for **1.75 m wide** panel with 2 number of rebar in each cell

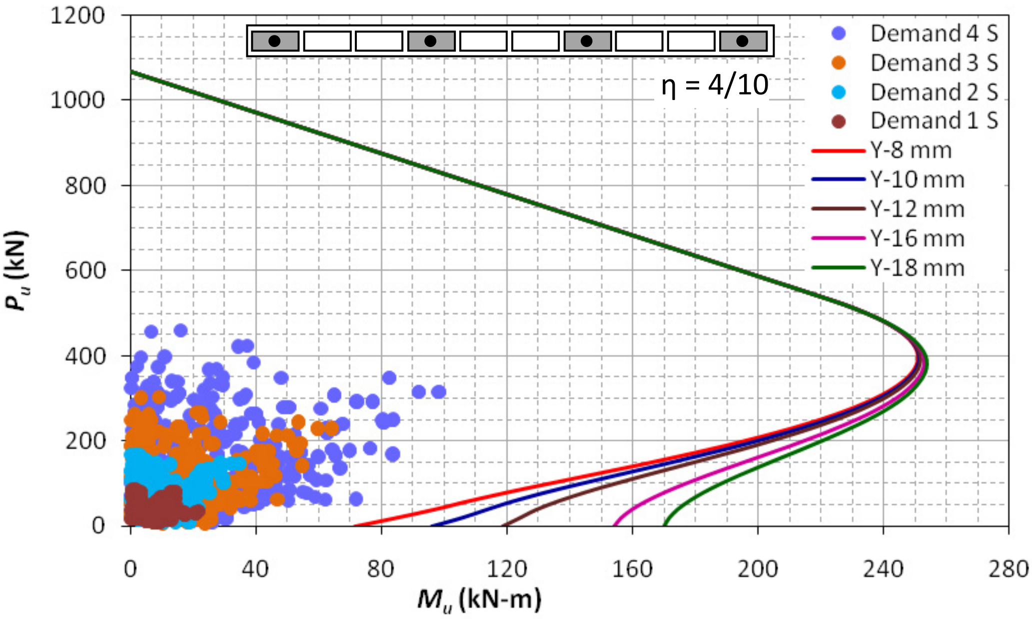


Fig.14.13 Demand points and capacity envelope for **3.0 m wide** panel

In general, all the demand points (Mu, Pu) are found to be enveloped by the interaction curve (failure line), corresponding to every fourth cavity of GFRG panels infilled with M20 concrete and reinforced with 1-10 mm diameter Fe 415 bar.

14.2 SUMMARY

In this chapter, the design of a single storey (duplex) house, a two storeyed house, a three storeyed house and a four storeyed house, have been demonstrated. It is seen from the design examples that, for all houses (up to 4 storeys), a maximum of 3 numbers of cavities can remain hollow (unfilled) in between the infilled ones. The empty cavities may be suitably filled with controlled low strength materials such as concrete, mixture of quarry dust and grout, sound insulating materials, etc. The comparison of reinforcement requirement for various types of houses is shown in Table 14.12. The number of cavities which can be left unfilled with concrete depends on many factors such as, room size (length of walls), seismic parameters, wind pressure, etc.

Table 14.12 Comparision of reinforcement requirement for various types of houses

Sl. No.	House Type	Infilling & Reinforcement Required for walls	Additional infilling and reinforcement required, if any
1	Single storey House (Duplex)	Every 4th cavity to be infilled with M20 concrete and 1Y-10mm Fe 415 bar	Other cavities may be suitably filled with concrete or alternative materials
2	Two storeyed House	Every 4th cavity to be infilled with M20 concrete and 1Y-10mm Fe 415 bar	— as above —
3	Three storeyed House	Every 4th cavity to be infilled with M20 concrete and 1Y-10mm Fe 415 bar	All cavities of walls 3X & 6X in storey 1 (refer Fig. 14.3) to be infilled with M20 concrete and 2Y-8mm Fe 415
4	Four storeyed House	Every 4th cavity to be infilled with M20 concrete and 1Y-10mm Fe 415 bar	All cavities of walls 3X & 6X in storeys 1 and 2 (refer Figs 14.3 & 14.4) to be infilled with M20 concrete and 2Y-8mm Fe 415

15. GFRG PANEL AS INFILL WALL IN RC FRAMED CONSTRUCTION

GFRG panels can be used as infill walls (and slabs) in conventional framed construction. This system is well suited for low rise to high rise buildings, without limitation in the number of storeys. The use of GFRG panels as infills has many advantages over conventional masonry infill. The dead load (and hence, seismic forces) is considerably reduced, thereby reducing the structural cost of the frame elements and foundations. Furthermore, plastering of the GFRG panel is not required. The GFRG wall and composite floor panels can be integrated with the construction of the columns and beams (refer ANNEXURE 3), as shown in Fig. 15.1. Although such integral composite action provides for more efficient structural behaviour, it is conservative to ignore the contribution of the GFRG walls in the composite action (hybrid behaviour), and to analyse and design the columns and beams as per the conventional practice using a bare frame analysis†.

More details on the construction aspects are provided in the Construction Manual. A comparison of material quantities for a typical eight storey building construction, designed by three different methods, is shown in ANNEXURE 4.

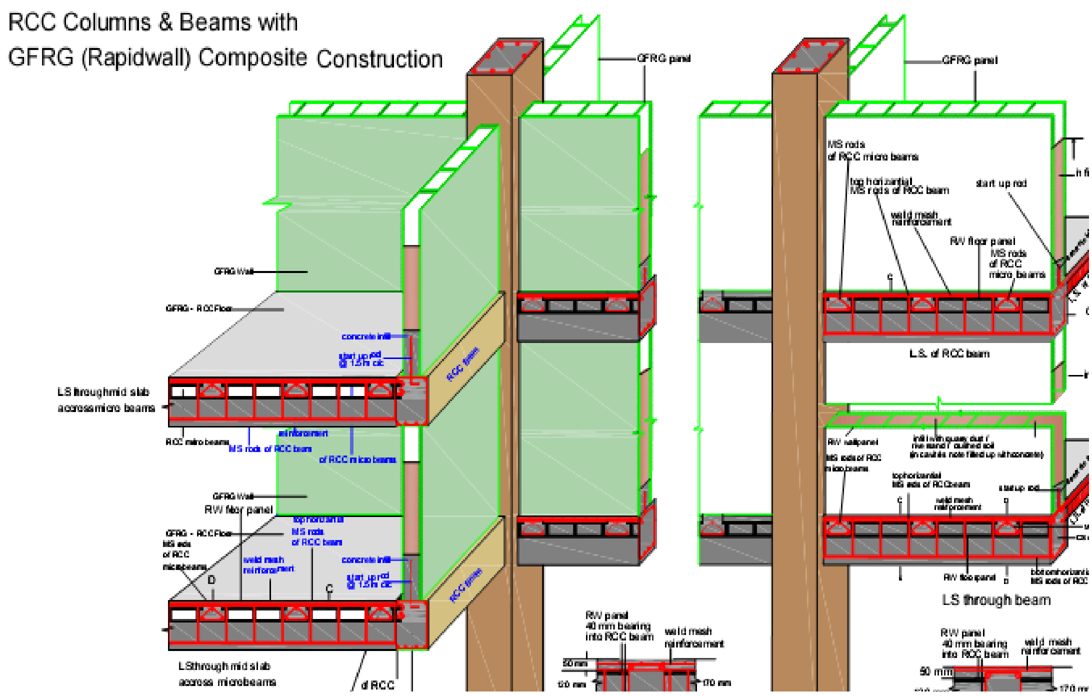


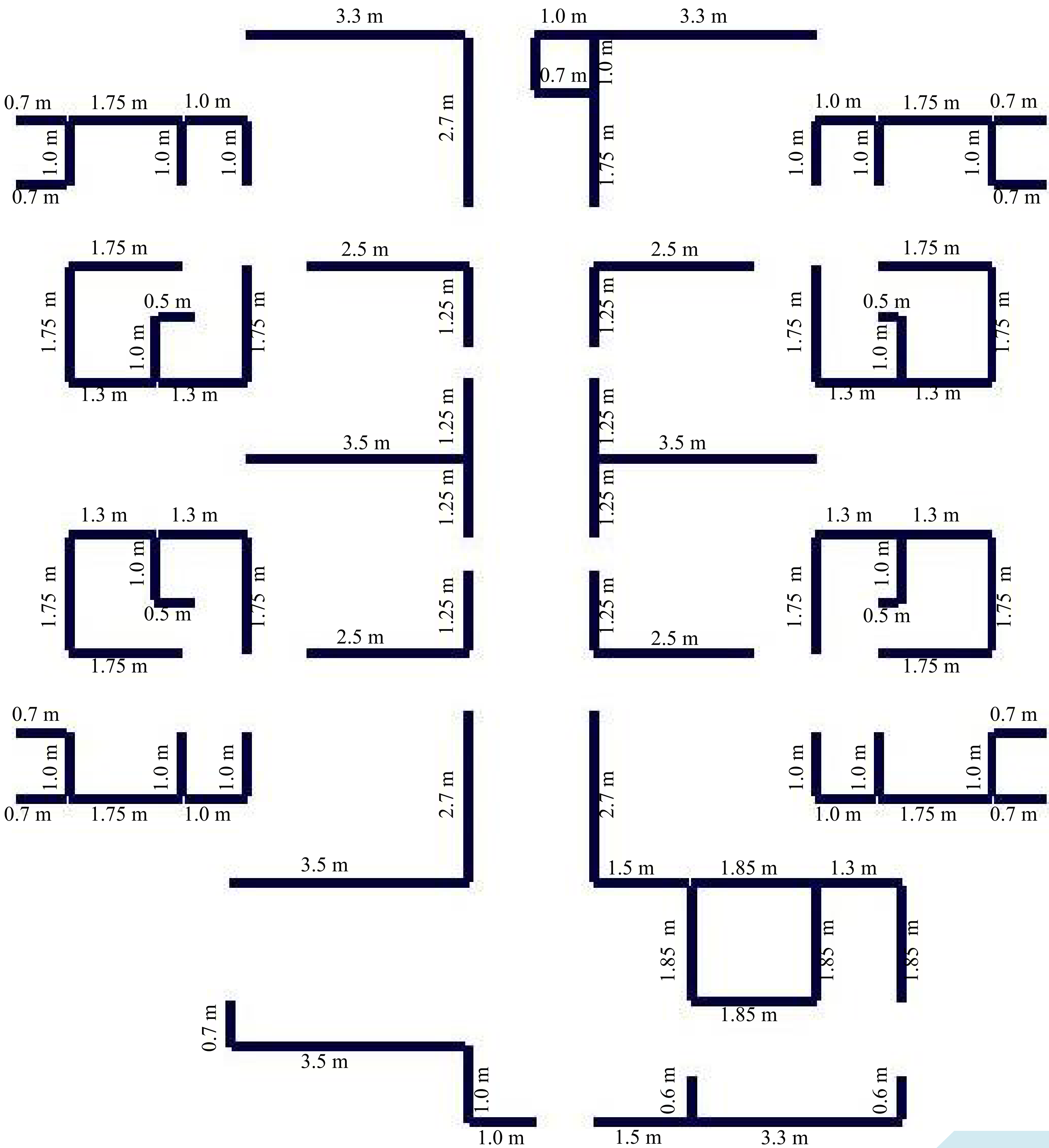
Fig. 15.1 RC columns and beams with GFRG panels as infill walls

† 'Hybrid' construction involves GFRG panels put in place prior to concreting of columns and beams. This can structurally enhance the performance, particularly against lateral loads. The design implication of such 'hybrid' behaviour is a topic presently under research (not included in this manual).

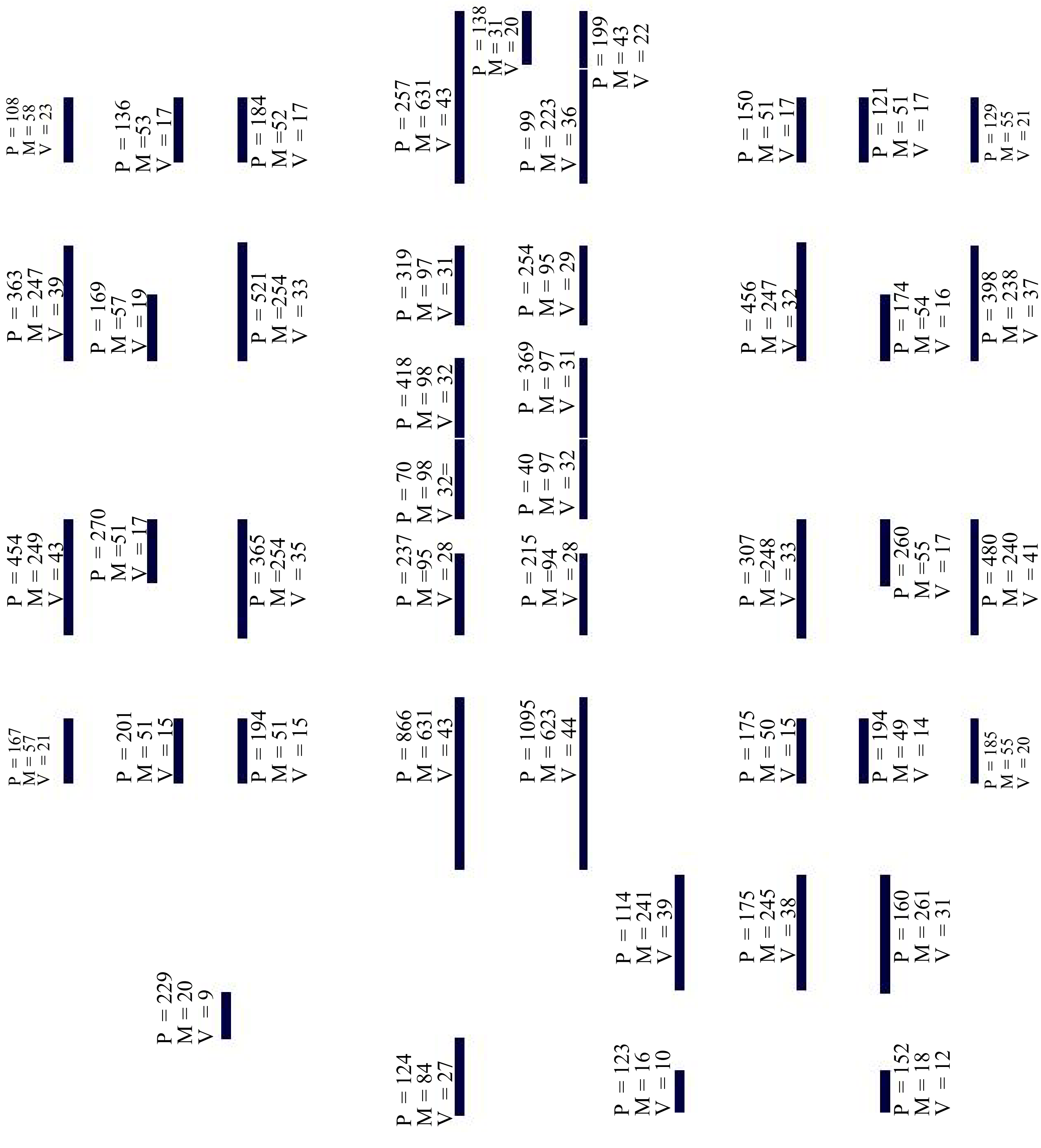
Annexure 1

Finite Element Analysis Results of 8-Storeyed GFRG Building (refer Chapter 13)

The arrangement of various pier walls (refer Fig. 13.1) is show below. Subsequent figures show the values of V_u, P_u, M_u for each pier for the critical load combination (maximum lateral shear, V_u, under earthquake loading) for all storeys and in both X and Y directions.

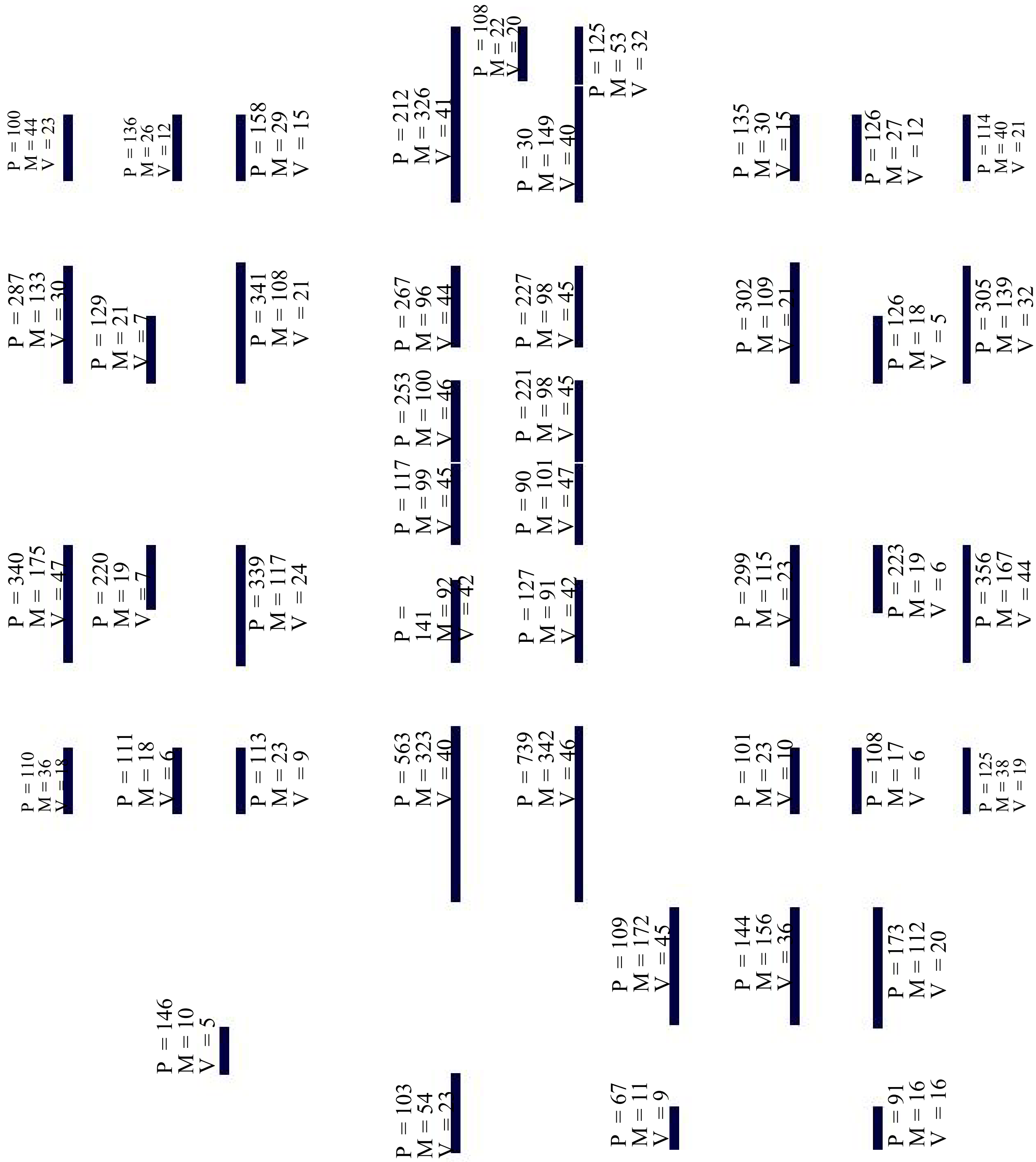


STOREY 1: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in Y-direction



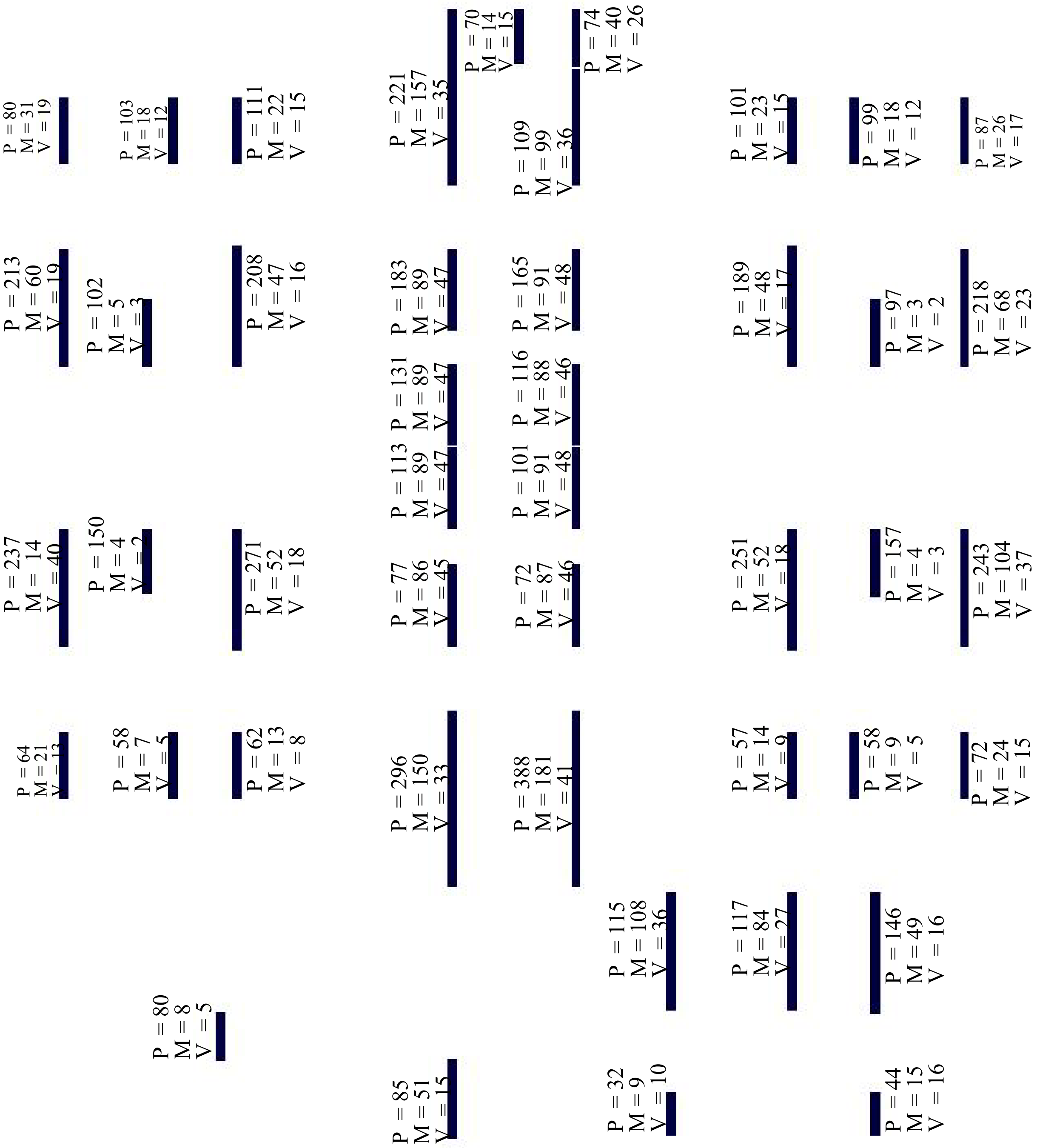
STOREY 2: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in Y-direction

STOREY 3: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in Y-direction

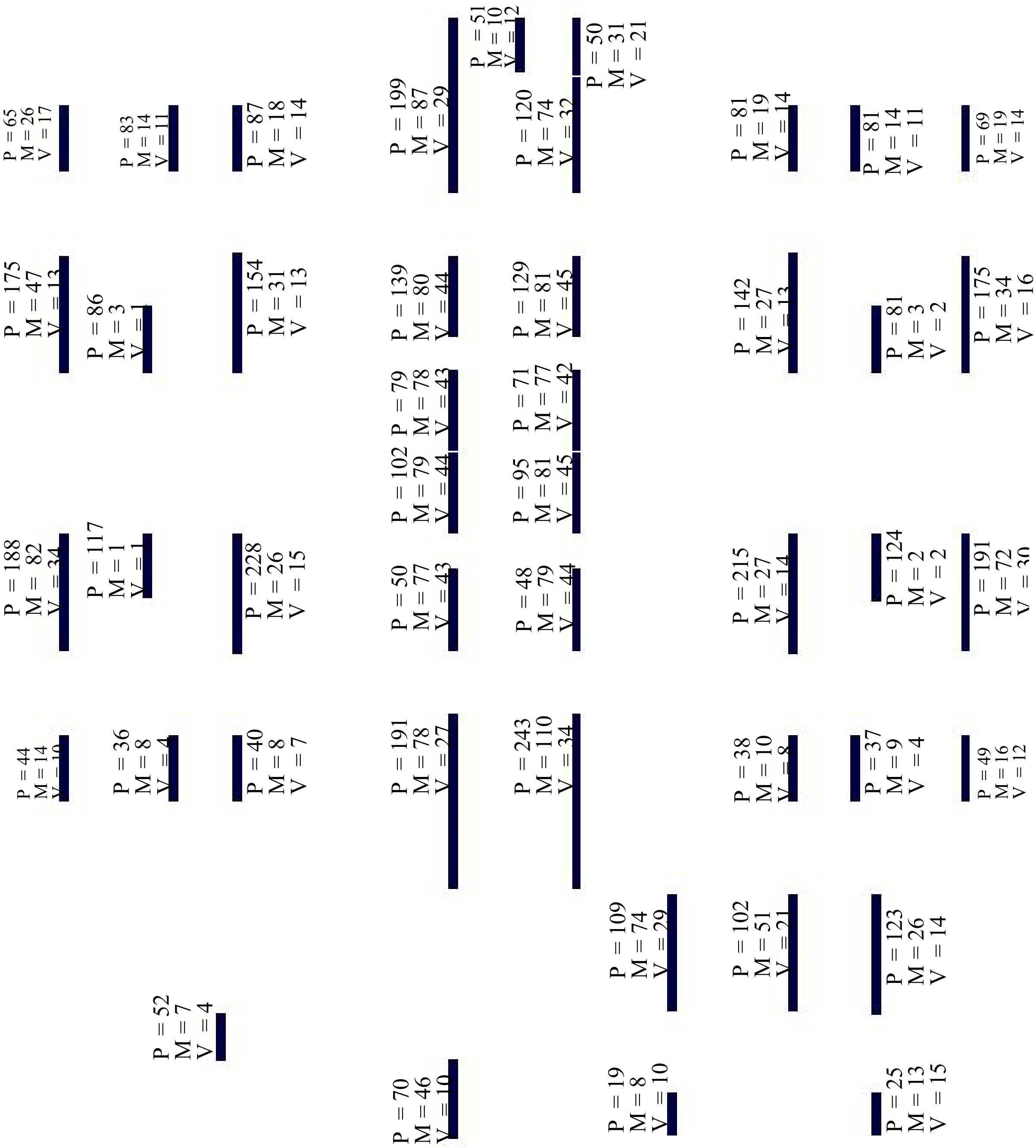


STOREY 4: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in Y-direction

STOREY 5: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in Y-direction

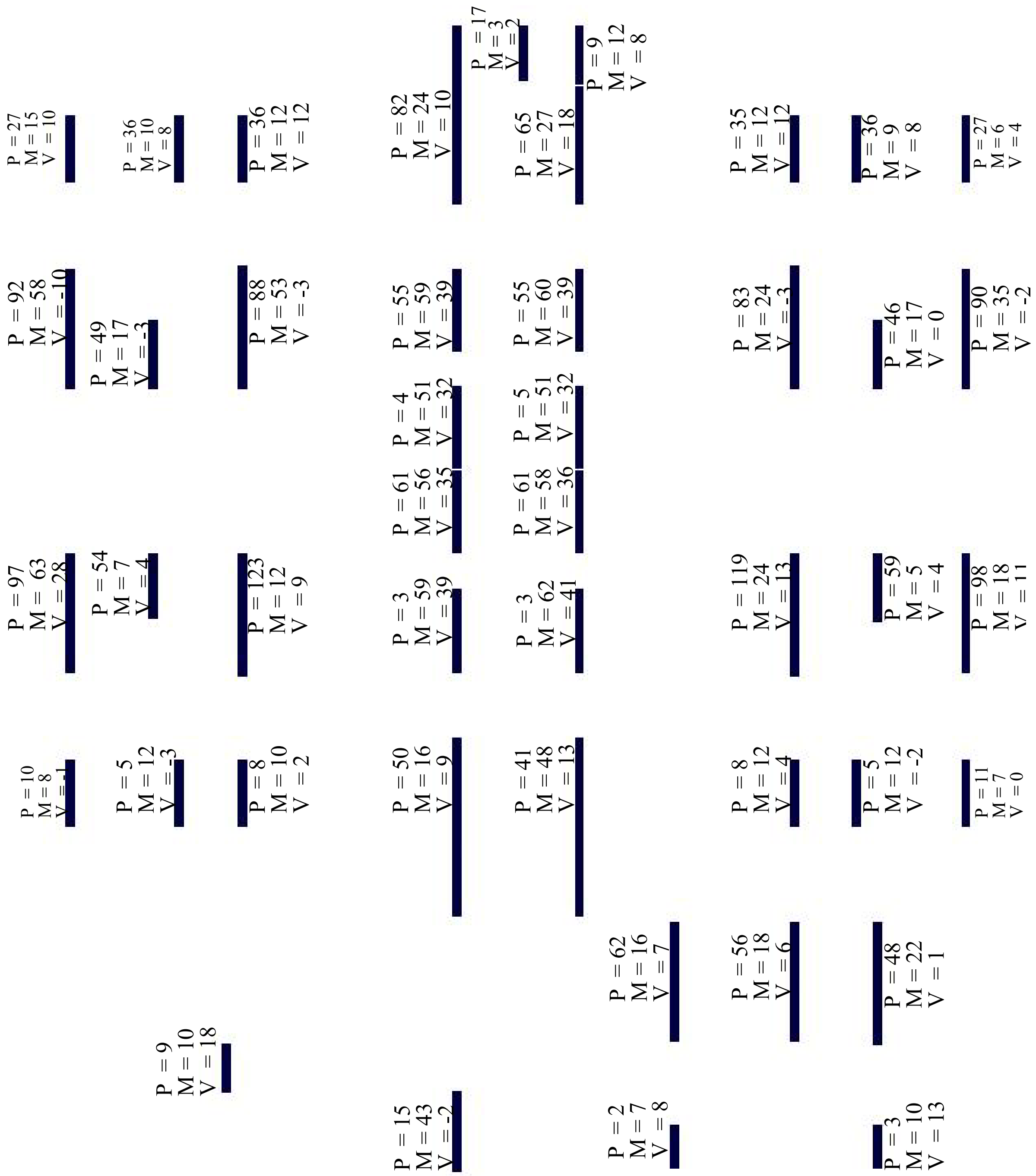


STOREY 6: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in Y-direction

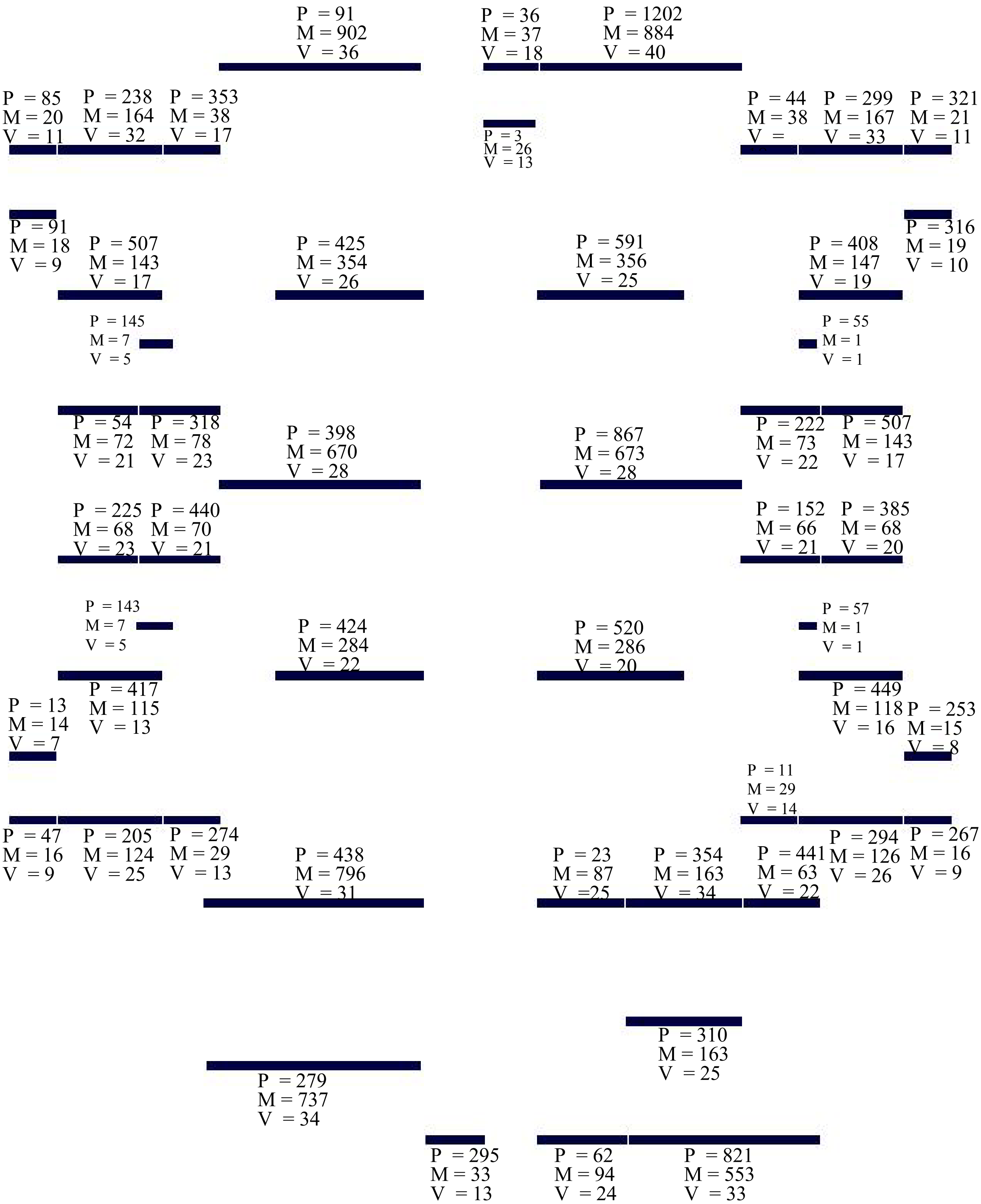


STOREY 7: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in Y-direction

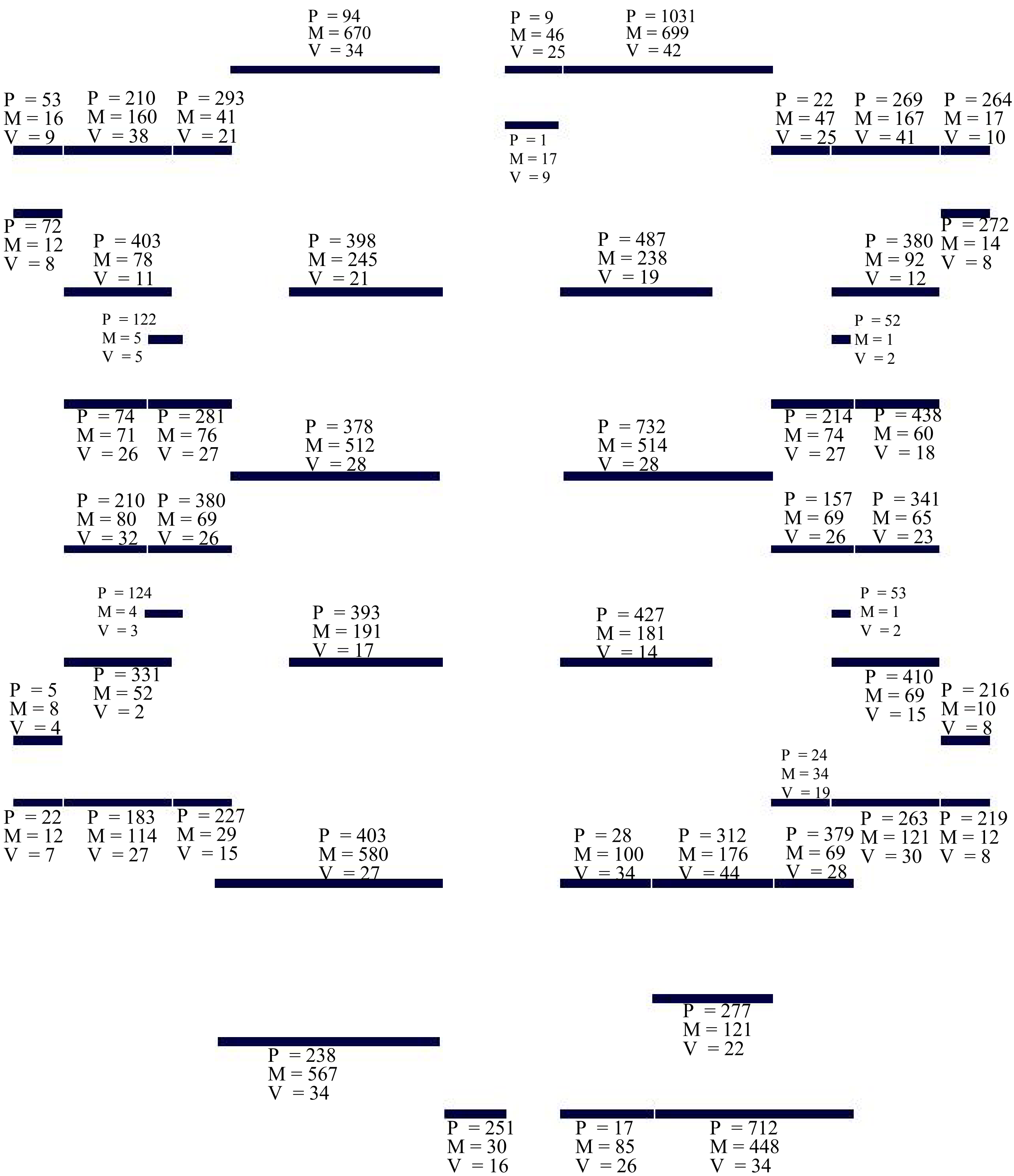
STOREY 8: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in Y-direction



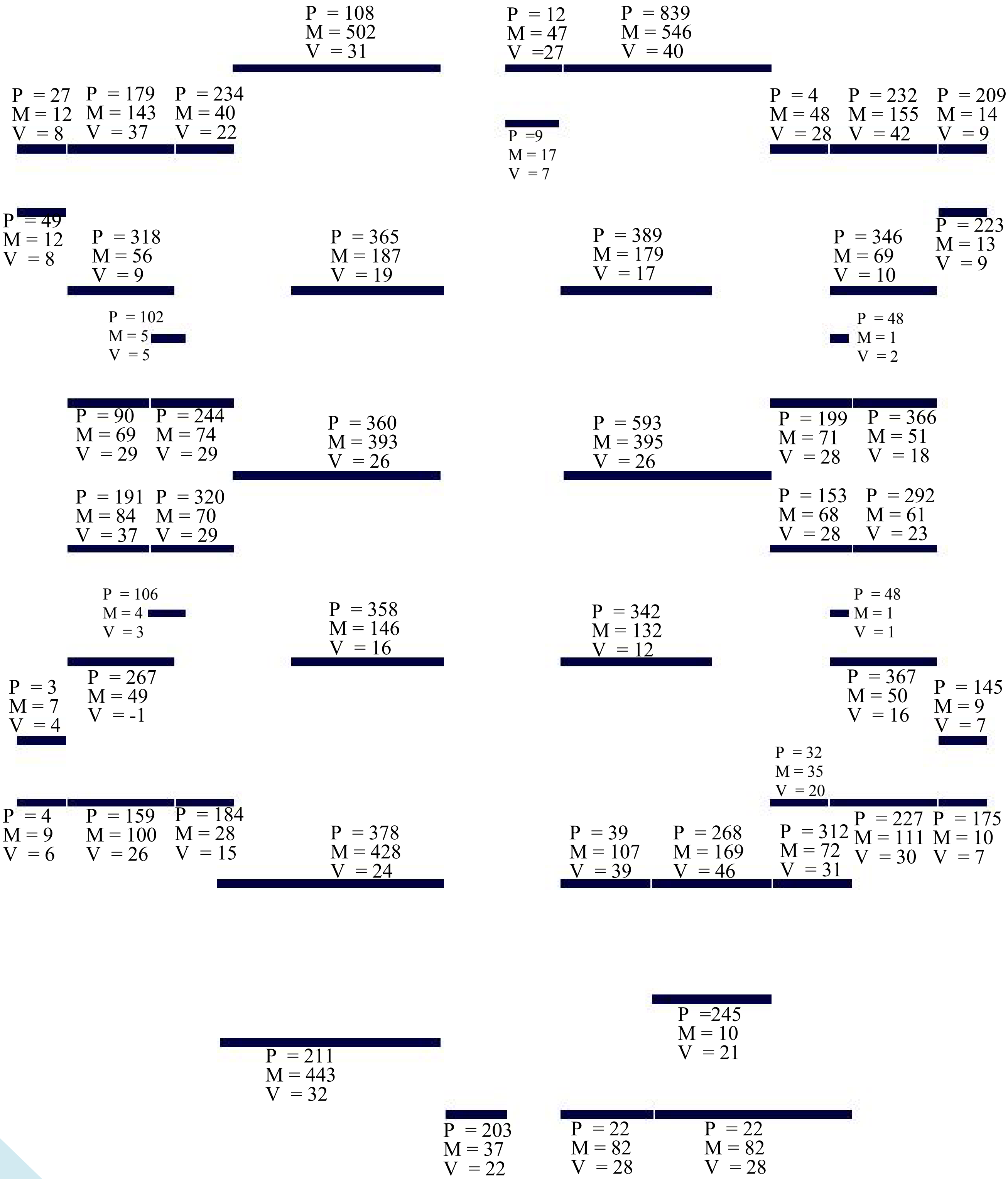
STOREY 1: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in X-direction



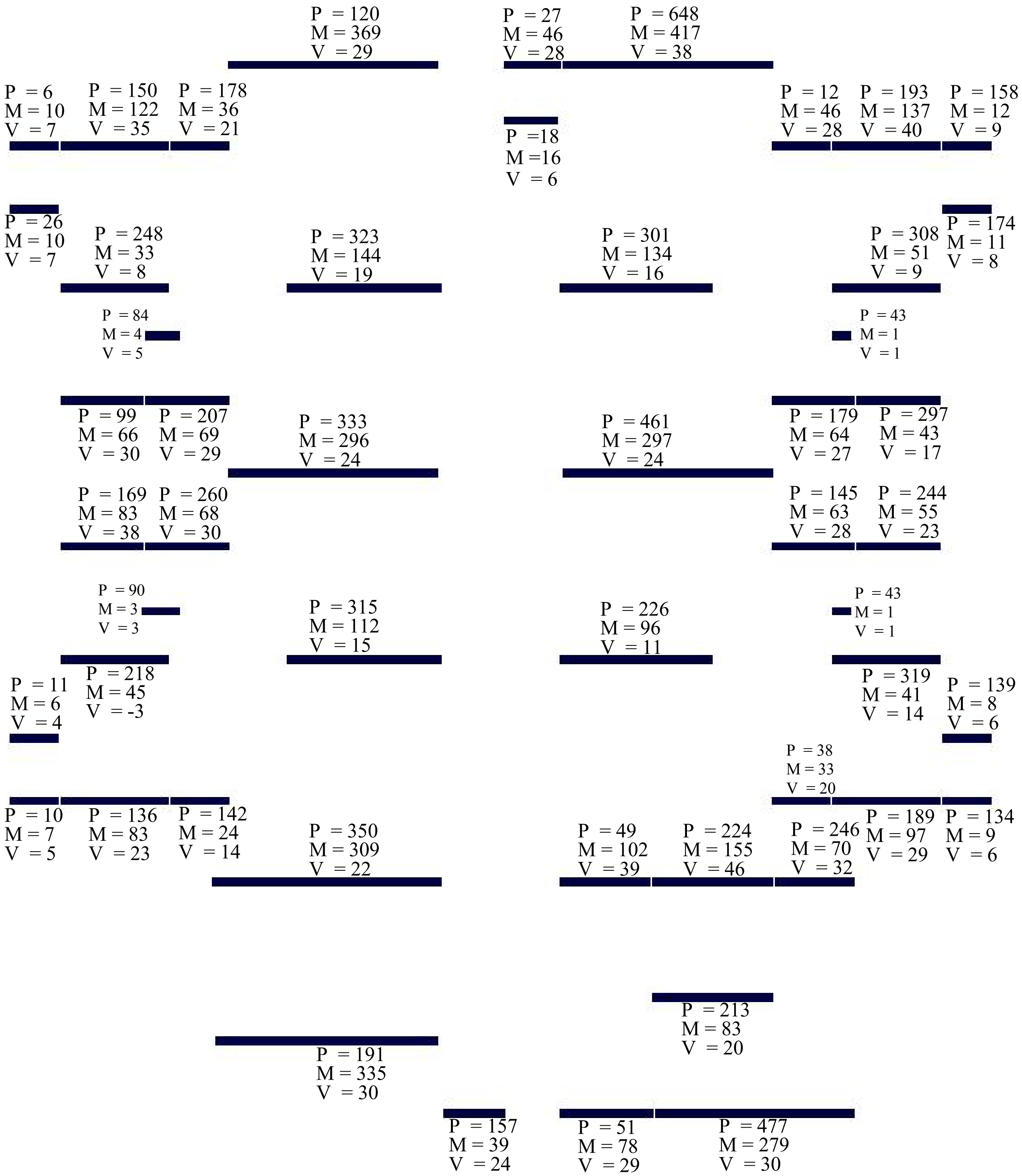
STOREY 2: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in X-direction



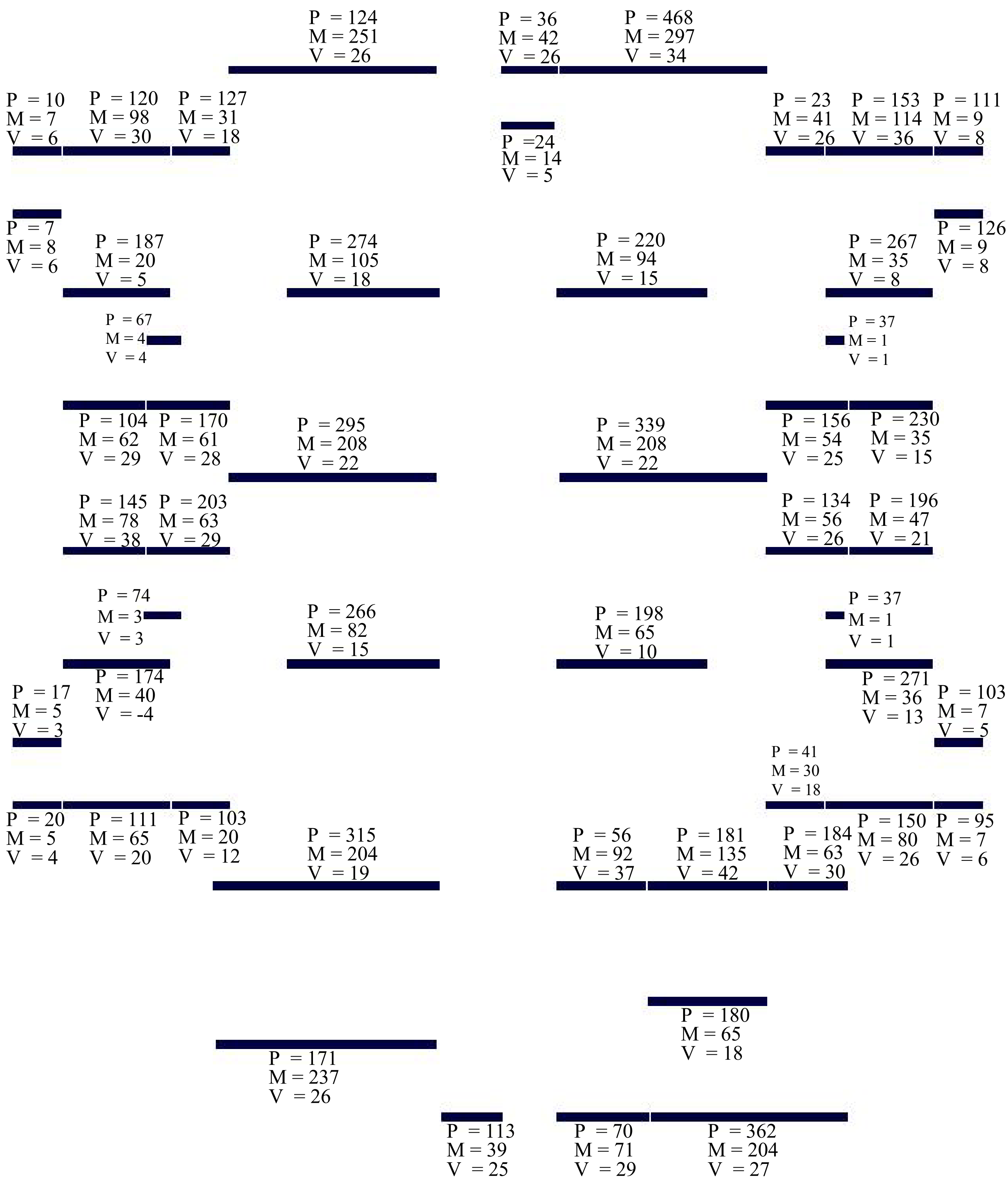
STOREY 3: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in X-direction



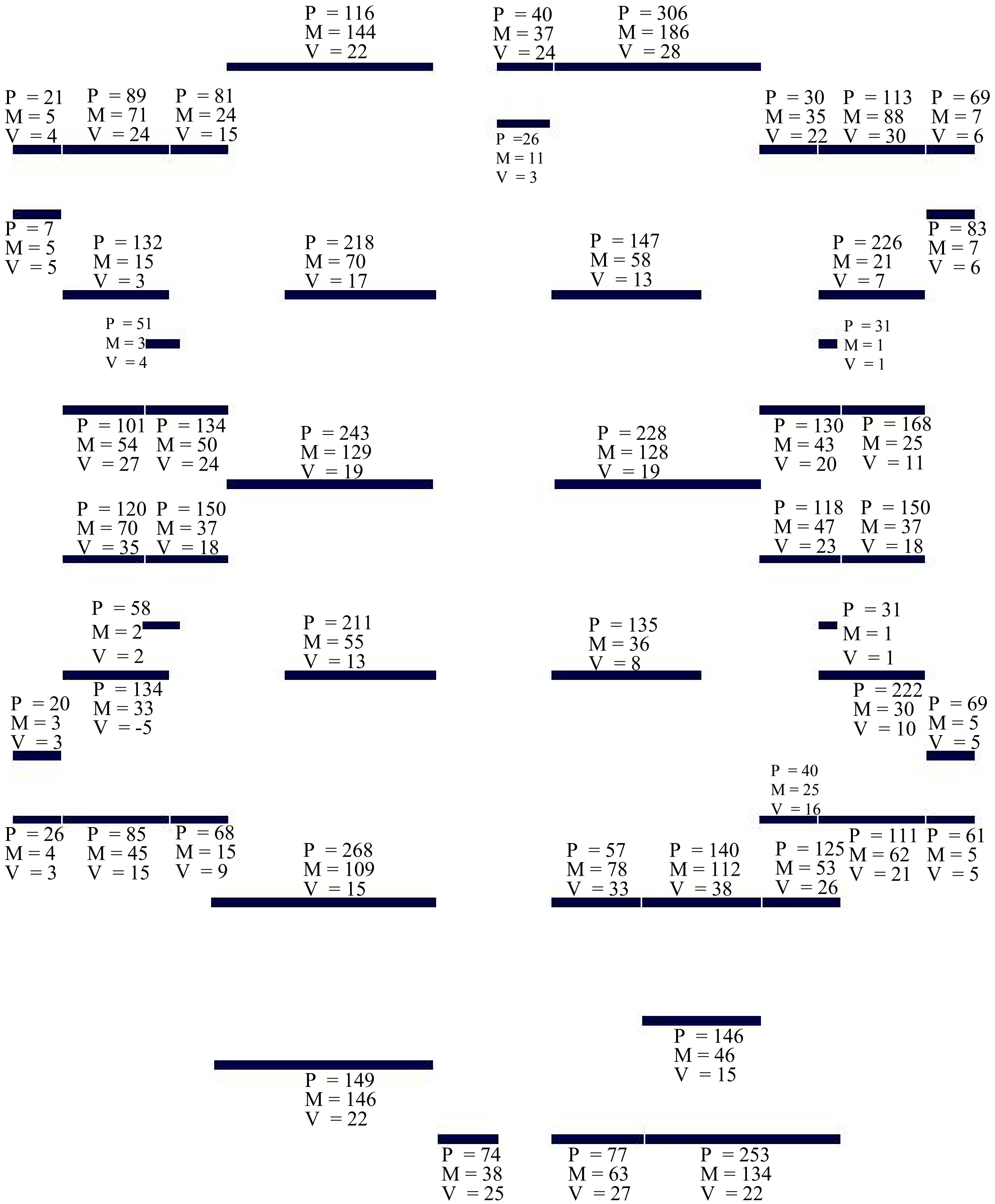
STOREY 4: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in X-direction



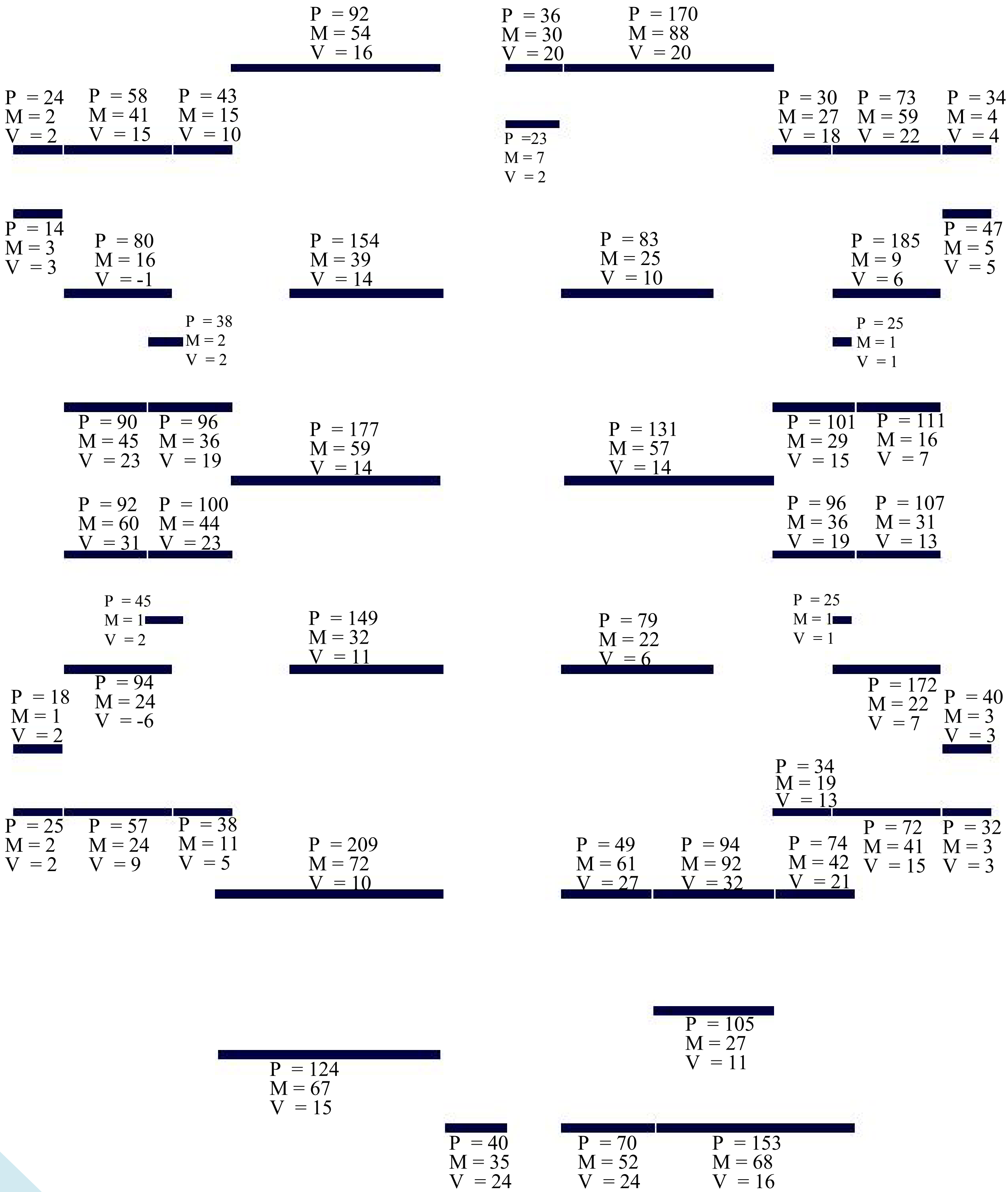
STOREY 5: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in X-direction



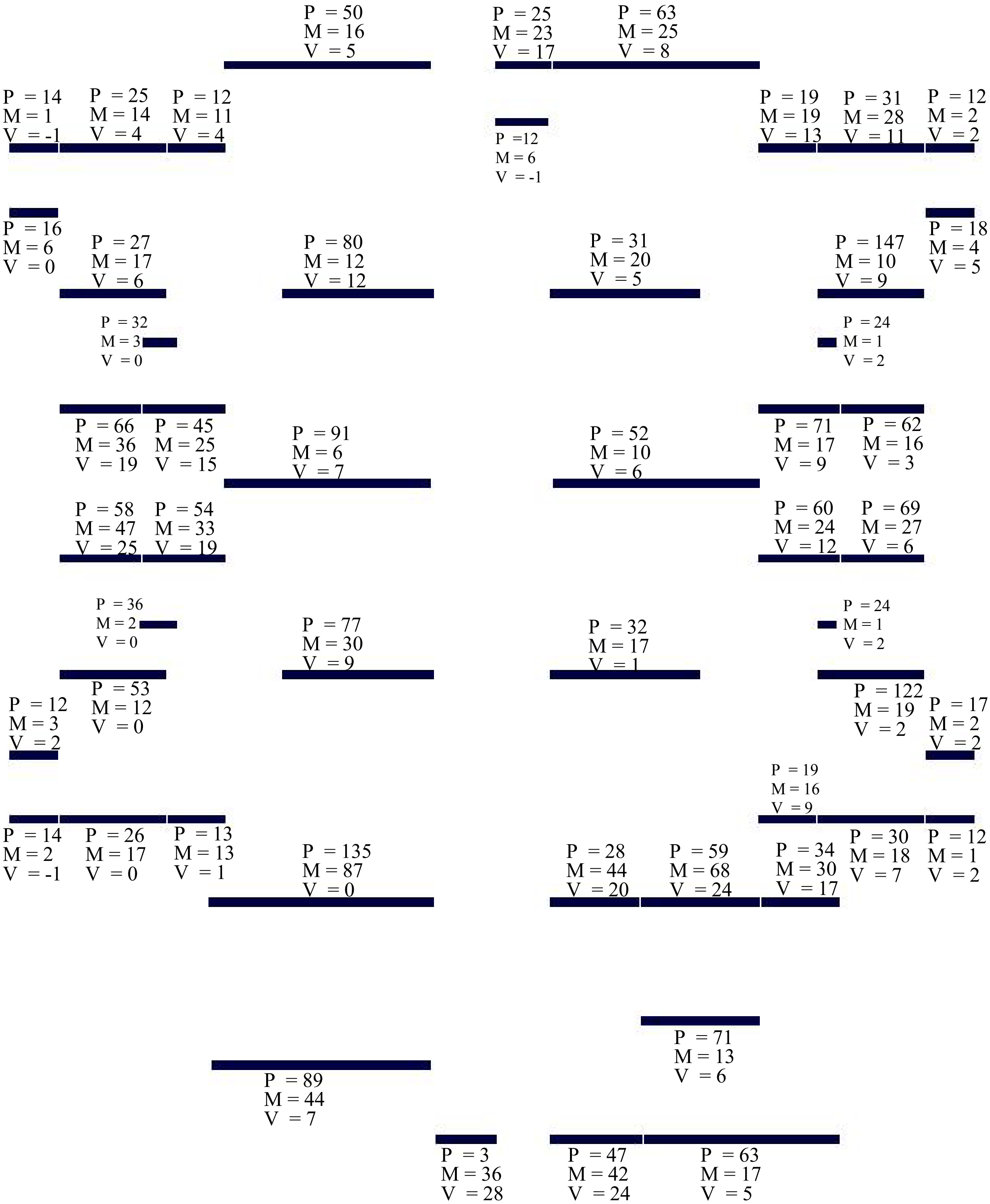
STOREY 6: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in X-direction



STOREY 7: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in X-direction



STOREY 8: The maximum shear force per unit length (V, kN/m) and corresponding bending moments (M, kN-m) and axial forces (P, kN) of walls in X-direction



Annexure 2

DESIGN OF INDIVIDUAL SMALL HOUSE FOR AFFORDABLE MASS HOUSING (Refer Chapter 14)

The design of a typical individual small house for affordable mass housing, of 26 sq. ft. (2.416 m^2) carpet area, is given here. The plan and sectional elevation of the house are shown in Fig. A2.1 and Fig. 14.2b respectively. The design load parameters are assumed to be the same as that of the design examples given in Chapter 14.

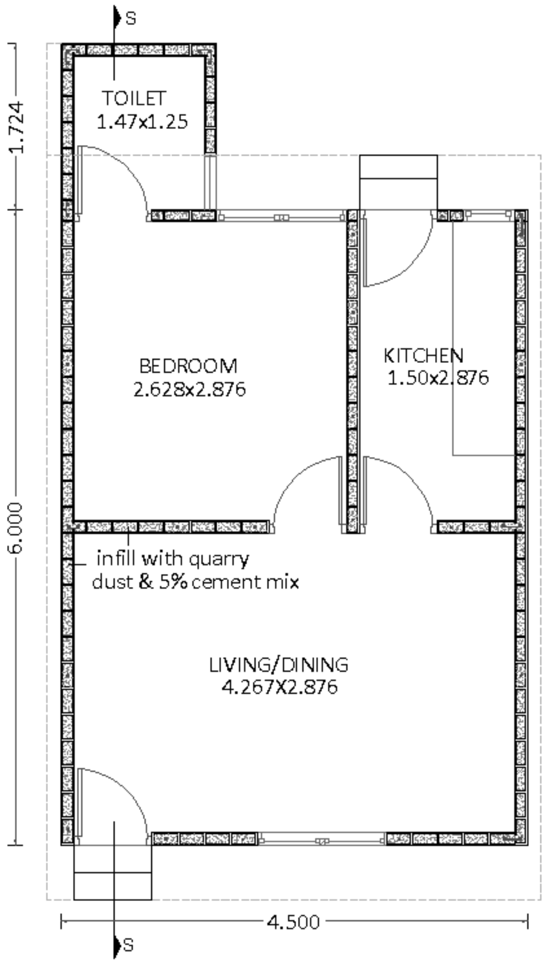


Fig. A2.1 Plan of single storey house

A2.1 DESIGN CALCULATIONS

A2.1.1 Load Calculations

Wind load

Basic wind speed (V_b) = 44 m/s

Risk coefficient (k_1) = 1.0

Terrain, height and structure factor (k_2) = 1.0 (category 2 and Class A)

Topography factor (k_3) = 1.0

Design wind speed (V_z) = V_b. $k_1.k_2.k_3$

= 44.0 m/s

Height of the building = 3.0 m

Design wind pressure (P_d) = 0.6 V_z^2 = 1.16 kN/m^2

Seismic Load

Zone: III

Zone factor (Z) = 0.16

Length of the building: 4.50 m

Width of the building: 7.72 m

Height of the building: 3.0 m

Design horizontal seismic coefficient $A_h = \frac{ZI}{2R}\left(\frac{S_a}{g}\right)$

Importance factor (I) = 1.0

Response reduction factor (R) = 3.0 (Table 7 of IS 1893 Part 1: 2002)

$T_{ax} = \frac{0.09h}{\sqrt{d}} = \frac{0.09\times3.0}{\sqrt{4.500}}$ = 0.13 s

$\Rightarrow S_a/g$ = 2.50 (from Fig.2 of IS 1893 Part 1: 2002)

Horizontal seismic coefficient, $A_h = \frac{0.16\times1\times2.50}{2\times3}$ = 0.0667

Similarly for Y direction, $T_{ay} = \frac{0.09h}{\sqrt{d}} = \frac{0.09\times3.0}{\sqrt{7.724}}$ = 0.97 s

$\Rightarrow S_a/g$ = 2.46 (from Fig. 2 of IS 1893 Part 1: 2002)

Horizontal seismic coefficient, $A_h = \frac{0.16\times1\times2.46}{2\times3}$ = 0.0656

Calculation of Base shear

Total seismic weight (DL + 0.25 LL), W = 294 kN

Design seismic base shear along X–direction:

$V_B = A_hW = 0.0667\times294$ = 19.61 kN

Design seismic base shear along Y direction:

$V_B = A_hW = 0.0656\times294$ = 19.29 kN

A2.1.2 Preliminary Design

Check for adequacy of single wall GFRG panels

Total length of GFRG wall in X-direction L_x = 6.25 m

Total length of GFRG wall in Y-direction L_y = 17.75 m

Average design shear force

in X-direction $= V_{Bx} / L_x =$ = 19.61/6.25 =3.14 kN/m

in Y-direction $= V_{By} / L_y$ = 19.29/17.75 =1.08 kN/m

These average values are much less than the design shear capacity of 14.4 kN/m (refer Table 9.1) for unfilled GFRG wall panels. Therefore, single wall GFRG panels are sufficient to ensure safety against lateral loads.

A2.2 DETAILED ANALYSIS AND DESIGN OF WALL SEGMENTS

Here, all the wall elements are analysed separately. For this purpose, walls are segmented at all joints and labelled as shown in Fig. A3.2. The axial force, in-plane shear force and in-plane bending moment due to all the loads and load combinations are calculated for each wall.

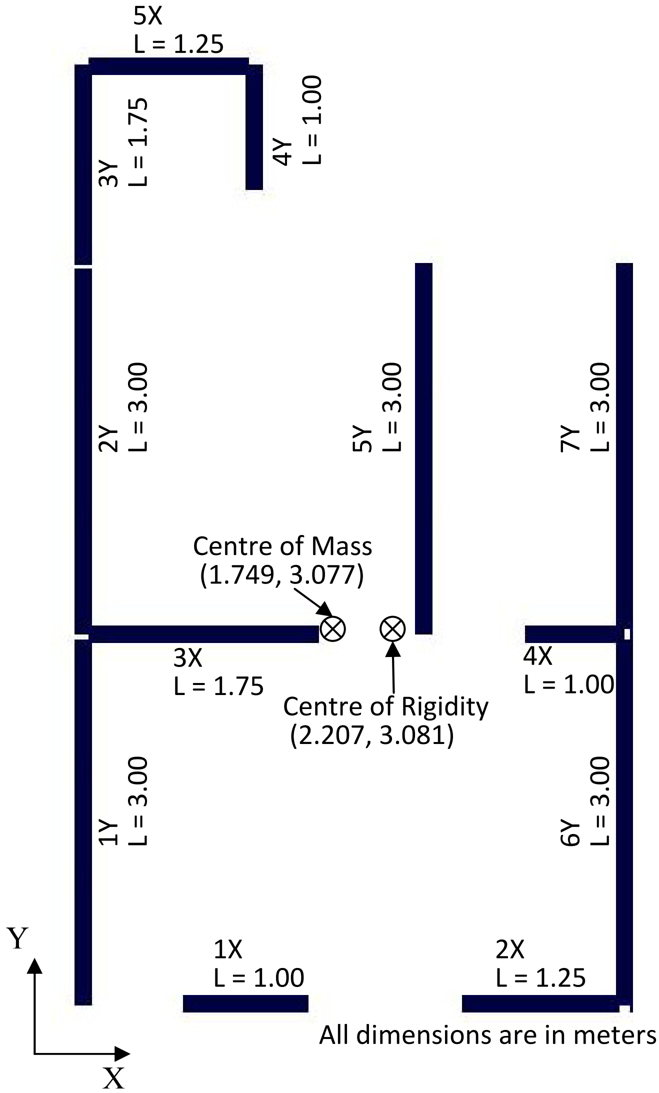


Fig. A2.2 Label and length of wall elements

The following load cases are considered:

1) Dead load (DL)
2) Imposed load (IL)
3) Earthquake load (EQ), considering the effect of torsion also.

The load combinations considered are as follows:

1) 1.5 (DL + IL)
2) 1.2 (DL + IL ± EL)
3) 1.5 (DL ± EL)
4) 0.9 DL ± 1.5 EL

A2.2.1 Forces in Walls due to Dead and Imposed Loads

The axial loads on the walls from dead and imposed loads are calculated based on tributary area distribution. The results are summarised in the Table A2.1.

Table A2.1 Axial force in walls due to gravity loads

Label of wall	1X	2X	3X	4X	5X	1Y	2Y	3Y	4Y	5Y	6Y	7Y
Axial force (kN)	22.2	7.9	31.0	3.6	10.6	6.7	8.3	2.6	19.6	30.9	28.9	34.7

A2.2.1 Forces in Walls due to Earthquake Load

Design seismic base shear along X–direction = 19.61 kN

Design seismic base shear along Y direction = 19.29 kN

The design base shear is distributed to each wall as in-plane shear, based on its relative stiffness. The stiffness of each wall (k_i) is calculated by,

$$k_i = 12EI/h^3 \qquad \text{(A2.1)}$$

where, E is the modulus of elasticity of GFRG wall, I is the moment of inertia of the wall and h is the height of the GFRG wall. It is assumed that the base shear in X-direction is resisted only by walls aligned in X-direction, and similarly in Y-direction. The distribution of forces among the walls in X and Y directions are shown in Tables A2.1 and A2.2 respectively.

Table A2.2 In-plane shear force in walls aligned in X-direction due to storey shear

Elements	L (m)	t (m)	h (m)	I (m^4)	k_i (N/mm)	$k_i/\Sigma k_i$	Shear force V_X (kN)	Shear force per meter V_U (kN/m)
1X	1.00	0.124	3	0.0103	46156	0.089	1.7	1.7
2X	1.25	0.124	3	0.0202	90148	0.173	3.4	2.7
3X	1.75	0.124	3	0.0554	247365	0.476	9.3	5.3
4X	1.00	0.124	3	0.0103	46156	0.089	1.7	1.7
5X	1.25	0.124	3	0.0202	90148	0.173	3.4	2.7

Table A2.3 In-plane shear force in walls aligned in Y-direction due to storey shear

Elements	L (m)	t (m)	h (m)	I (m^4)	k_i (N/mm)	$k_i/\Sigma k_i$	Shear force V_Y (kN)	Shear force per meter V_U (kN/m)
1Y	3.00	0.124	3	0.2790	1246200	0.191	3.7	1.2
2Y	3.00	0.124	3	0.2790	1246200	0.191	3.7	1.2
3Y	1.75	0.124	3	0.0554	247365	0.038	0.7	0.4
4Y	1.00	0.124	3	0.0103	46156	0.007	0.1	0.1
5Y	3.00	0.124	3	0.2790	1246200	0.191	3.7	1.2
6Y	3.00	0.124	3	0.2790	1246200	0.191	3.7	1.2
7Y	3.00	0.124	3	0.2790	1246200	0.191	3.7	1.2

The centre of mass (CM) and centre of rigidity (CR) are obtained as (1.749, 3.077) and (2.207, 3.081) respectively and are shown in Fig. A2.2. It is seen that there is only eccentricity in X-direction as the eccentricity in Y-direction is negligible.

The design eccentricity (e_{di}), as per IS 1893 Part 1 2002, is given below:

$$e_{di} = \begin{cases} 1.5e_{si} + 0.05bi \\ e_{si} - 0.05bi \end{cases} \tag{A2.1}$$

where, e_{si} is the static eccentricity and b_i is the floor plan dimension of the floor perpendicular to the direction considered.

Static eccentricity in X-direction, e_{si} = (2.207-1.749) = 0.46 m

Static eccentricity in Y-direction, e_{si} = 0 m

Floor plan dimension of the building perpendicular to the X-direction, b_i = 7.72 m

Floor plan dimension of the building perpendicular to the Y-direction, b_i = 4.5 m

The design eccentricity in X-direction = $\begin{cases} 1.5\times 0.46 + 0.05\times 4.5 = 0.91\text{m} \\ 0.46 - 0.05\times 4.5 = 0.43\text{m} \end{cases}$

The design eccentricity in Y-direction = $\begin{cases} 1.5\times 0 + 0.05\times 7.72 = 0.39\text{m} \\ 0 - 0.05\times 7.72 = -0.39\text{m} \end{cases}$

The torsional moment (M_T) due to shear in X-direction ,

= $V_B \times e_{si} = (19.61 \times 0.39)$ = ±7.57 kNm

The torsional moment (M_T) due to shear in X-direction (considering severe case),

$= V_B \times e_{si} = (19.29 \times 0.91) = 17.55$ kNm

This torsional moment is distributed among the walls using the equation,

$$F_{iR} = \frac{k_i r_i}{\sum k_i r_i^2} M_T \qquad \text{(A2.1)}$$

where, F_{iR} is the in-plane shear force in the wall due to torsional moment and r_i is the distance to the centre of corresponding wall from the centre of rigidity.

Thus, the total in-plane shear force is calculated by adding the components of shear due to both lateral force and torsional moment, and the corresponding in-plane bending moment is also calculated by multiplying the in-plane shear force with 0.5 times the corresponding height of wall. The results are tabulated in Table A2.4.

Table A2.4 In-plane shear force and bending moment due to EQ

Elements	r_i (m)	Shear due to torsion F_{iR}(kN)	Shear due to torsion (kN)	Total shear force (kN)	Total shear force per meter V_U (kN/m)	In-plane moment (kNm)
1X	-3.08	0.3	1.7	2.1	2.1	3.1
2X	-3.08	0.7	3.4	4.1	3.3	6.1
3X	-0.1	0.1	9.3	9.4	5.4	14.1
4X	-0.1	0.0	1.7	1.8	1.8	2.6
5X	4.52	1.0	3.4	4.4	3.5	6.6
1Y	2.207	1.9	3.7	5.6	1.9	8.4
2Y	2.207	1.9	3.7	5.6	1.9	8.4
3Y	2.207	0.4	0.7	1.1	0.6	1.7
4Y	0.817	0.0	0.1	0.2	0.2	0.2
5Y	-0.543	-0.5	3.7	3.2	1.1	4.8
6Y	-2.173	-1.9	3.7	1.8	0.6	2.7
7Y	-2.173	-1.9	3.7	1.8	0.6	2.7

The structure is analysed for all load combinations given above, and the resultant shear force, axial force and bending moment for critical load combinations are tabulated in Table A2.5.

Table A2.5 The maximum shear force per unit length (V_u, kN/m) and corresponding bending moments (M_u, kN-m) and maximum axial forces (P_u, kN) of walls in X and Y directions

Name	Length (m)	Number of cavities	Infilling required (addition to end cavities)	η	Shear resistance V_R (kN/m)	Design shear force V_U (kN/m)	Design axial force P_U (kN)	Design In-plane moment M_U (kNm)
1X	1.00	4	0	1/2	27	3.1	33	4.7
2X	1.25	5	0	2/5	25	4.9	12	9.2
3X	1.75	7	1	3/7	25	8.0	47	21.1
4X	1.00	4	0	1/2	27	2.6	5	3.9
5X	1.25	5	0	2/5	25	5.3	16	9.9
1Y	3.00	12	2	1/3	23	2.8	10	12.6
2Y	3.00	12	2	1/3	23	2.8	12	12.6
3Y	1.75	7	1	3/7	25	0.9	4	2.5
4Y	1.00	4	0	1/2	27	0.2	29	0.4
5Y	3.00	12	2	1/3	23	1.6	46	7.2
6Y	3.00	12	2	1/3	23	0.9	43	4.1
7Y	3.00	12	2	1/3	23	0.9	52	4.1

Shear Check on GFRG Wall Panels

The shear force demand and capacity of each wall elements are tabulated in Table A2.5. It can be seen that it is required to infill every fourth cavity with M20 concrete to satisfy the in-plane shear demand.

Check on In-Plane Moment Capacity of GFRG Wall Panels

The resultant axial force and in-plane bending moment in each wall, under critical load combinations, are checked with the corresponding P-M interaction curves.

The demand of axial force and bending moment of the GFRG wall panel with the capacity curves for various lengths of walls, 1.0m, 1.25m, 1.75m, and 3.0m are shown in Figs A2.3.to A2.6.

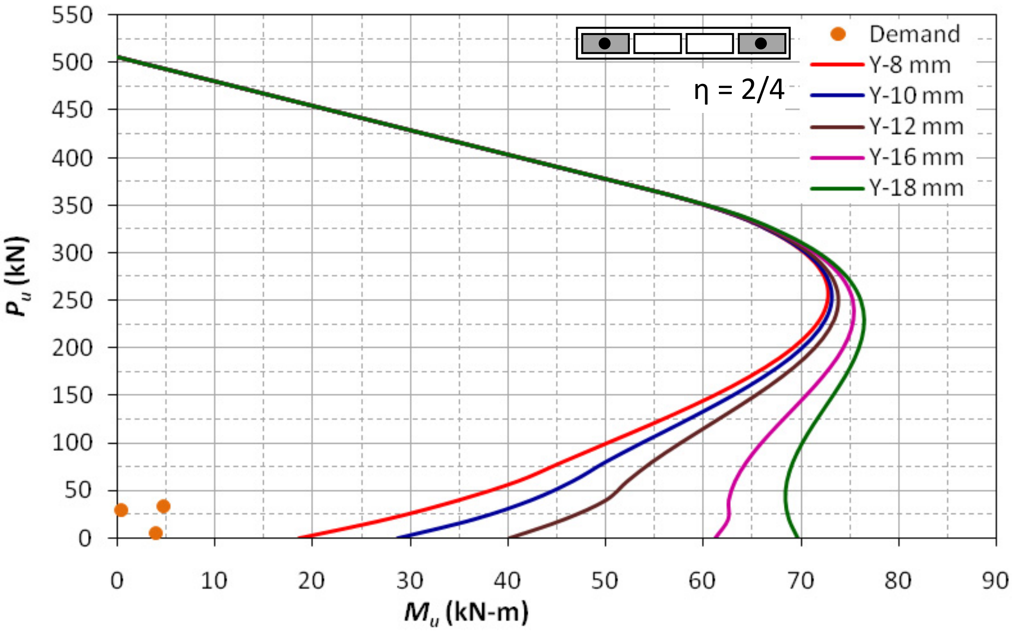
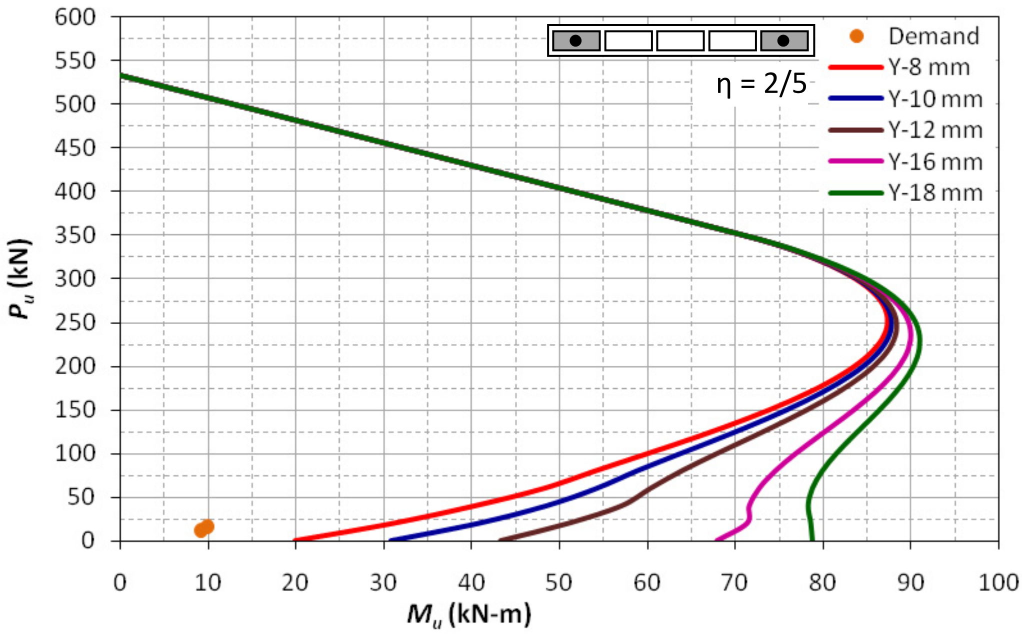


Fig. A2.3 Demand points and the capacity envelope for **1.02 m wide** panel



`

Fig. A2.4 Demand points and the capacity envelope for **1.25 m wide** panel

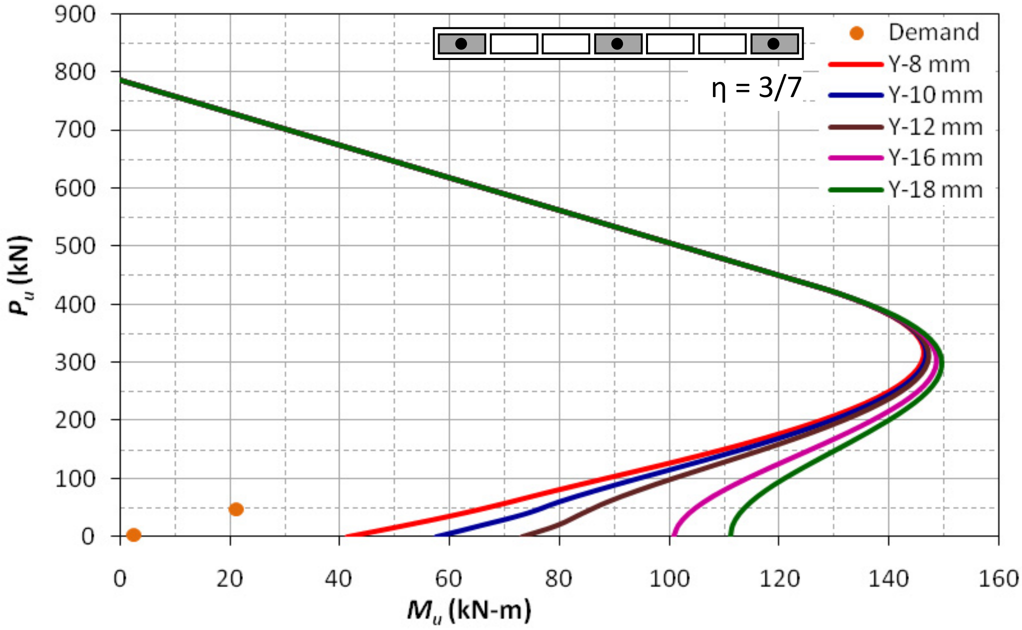


Fig. A2.5 Demand points and the capacity envelope for 1.75 m wide panel with 2 number of rebars in each cell

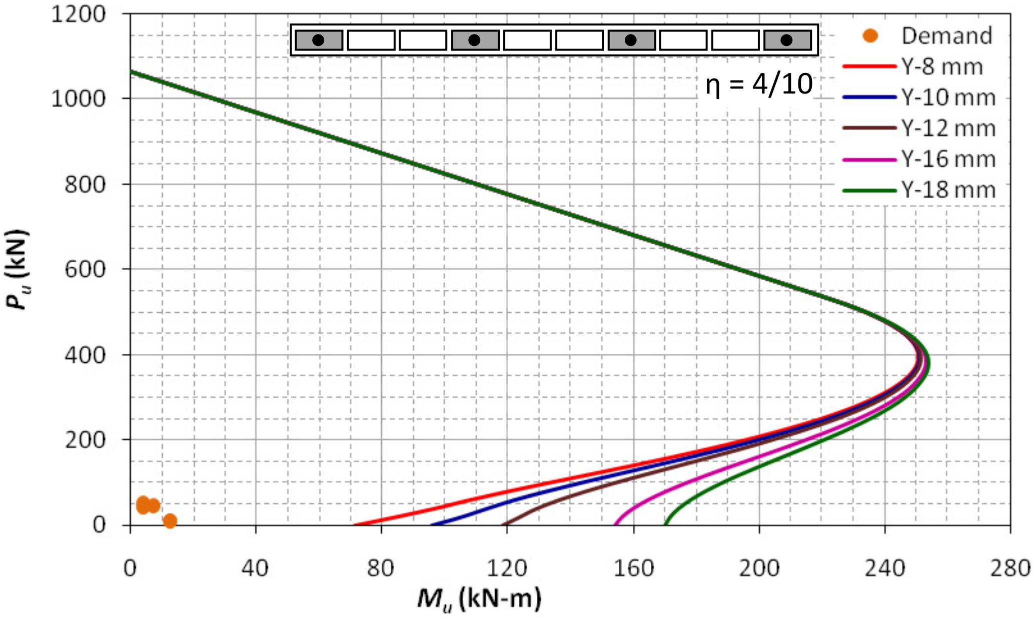


Fig. A2.6 Demand points and the capacity envelope for **3.0 m wide** panel

It can be seen from the above *P – M* interaction curves that all the demand points (Mu, Pu) are found to be enveloped by the interaction curve (failure line), corresponding to every fourth cavity of GFRG panels infilled with M20 concrete and reinforced with 1-10 mm diameter Fe 415 bar.

Annexure 3

(refer Chapter 15)

Details of the sectional details of typical beams, columns, GFRG wall, GFRG slab (with micro-beams) and sequence of construction are given in the figures to follow.

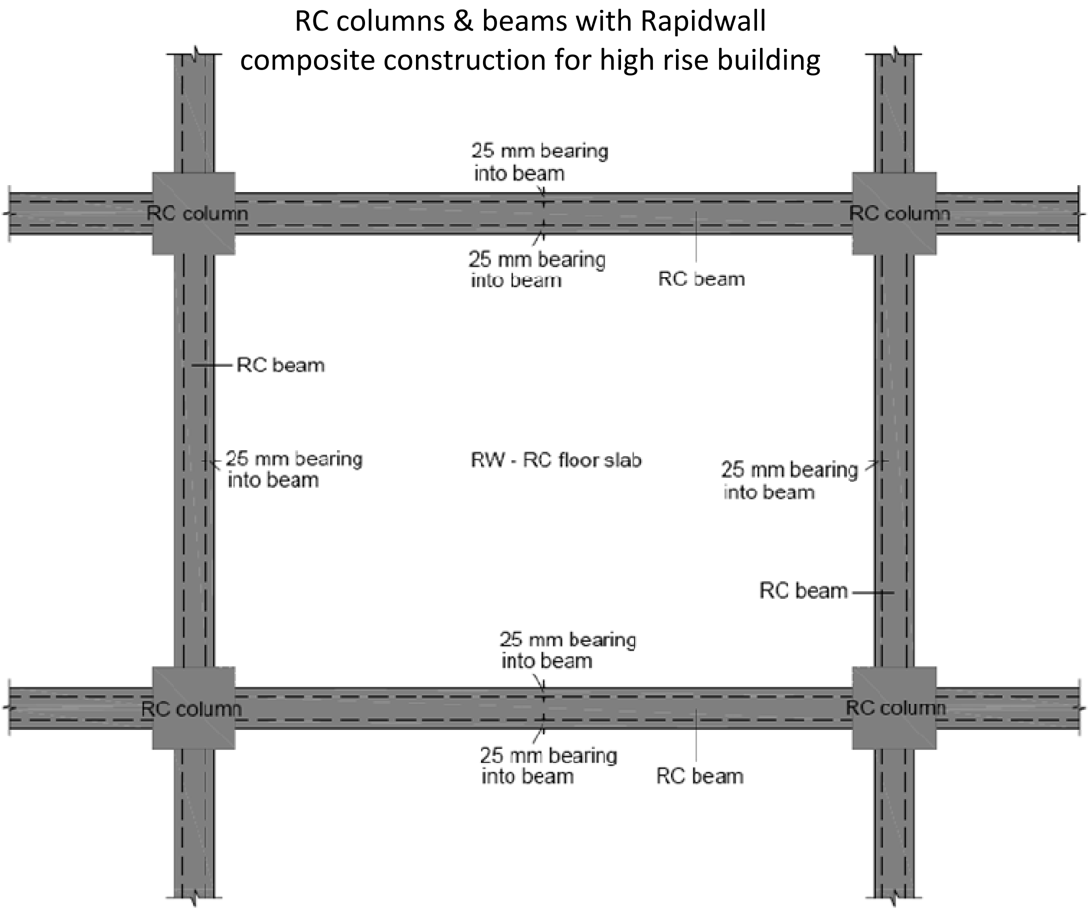


Fig. A3.1 Typical plan of RC columns & beams

However, if there is vertical discontinuity of infill wall present, it is mandatory to check provisions of soft storey as given in IS 1893 and suitable additional reinforcement provided in the columns.

RC columns & beams with Rapidwall composite construction for high rise building

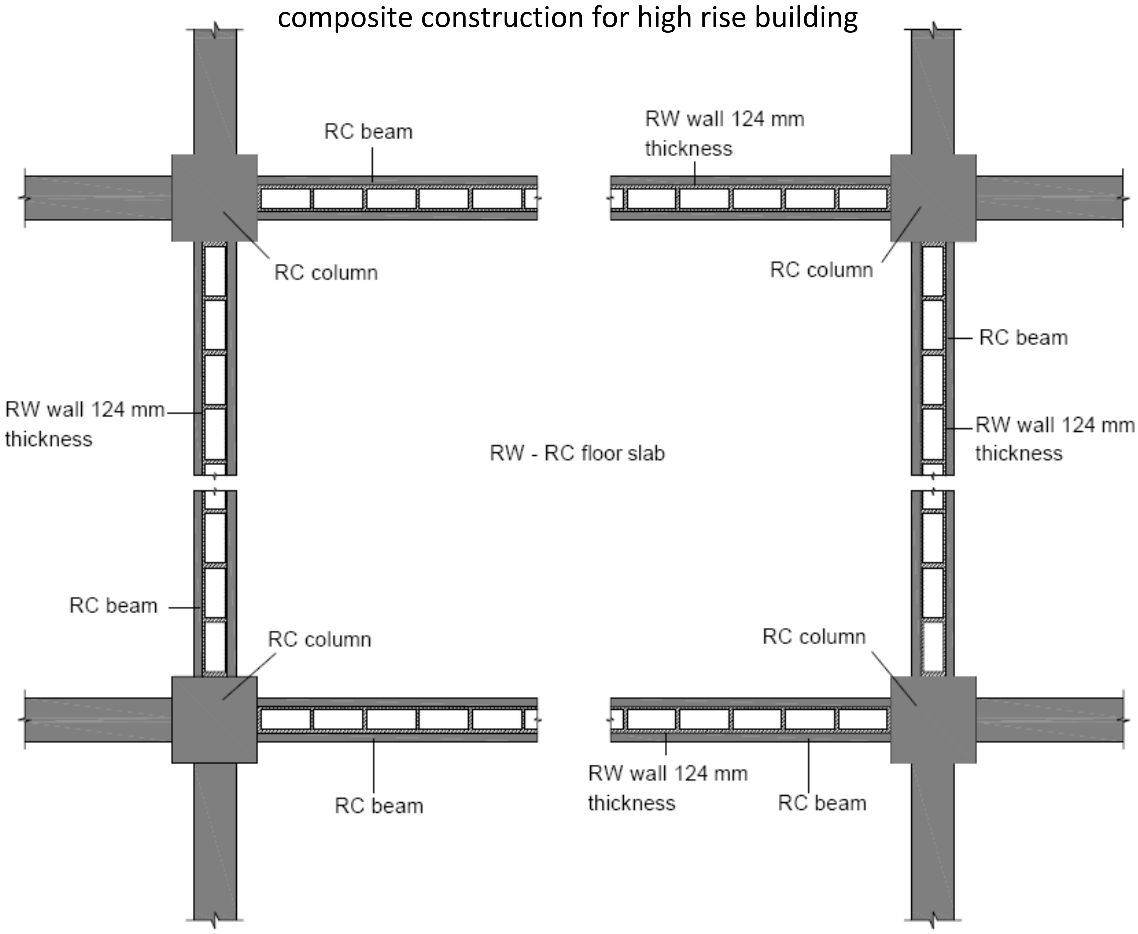


Fig. A3.2 Plan of RC columns & beams

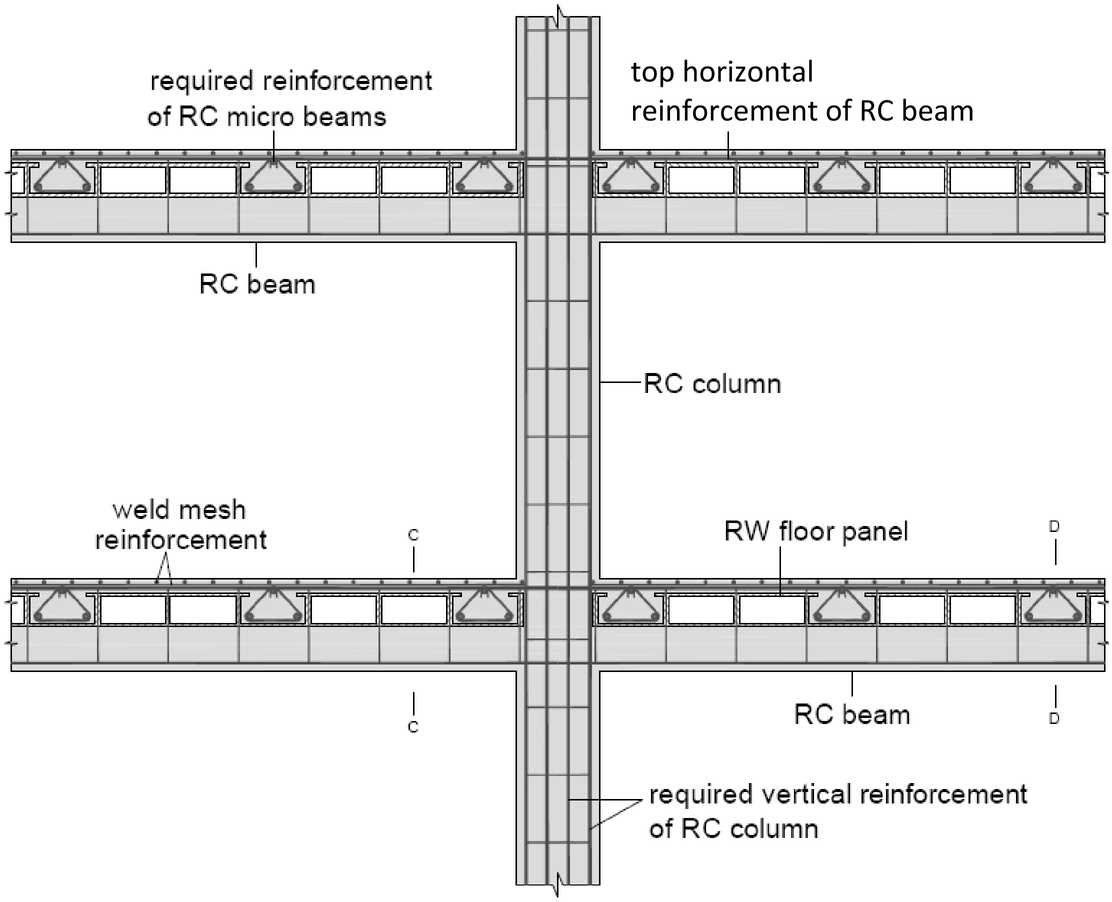
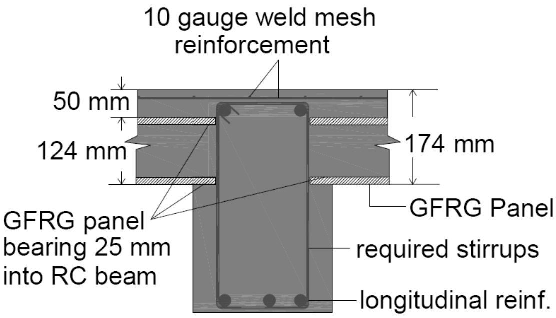
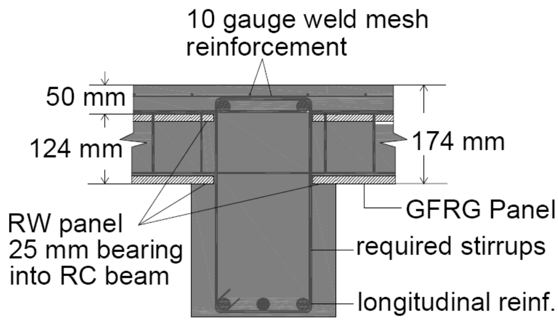


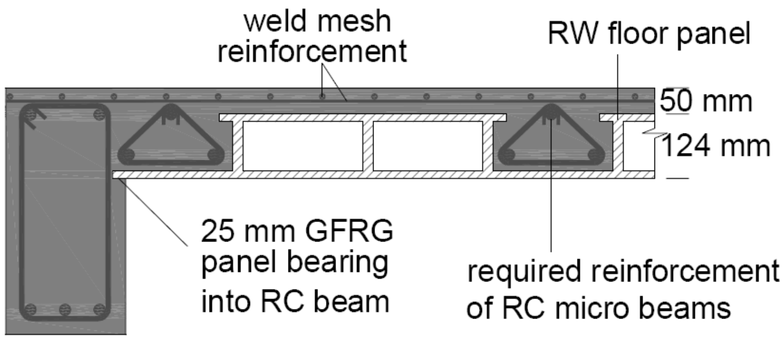
Fig. A3.3 RC columns and beams with GFRG composite construction for high rise building



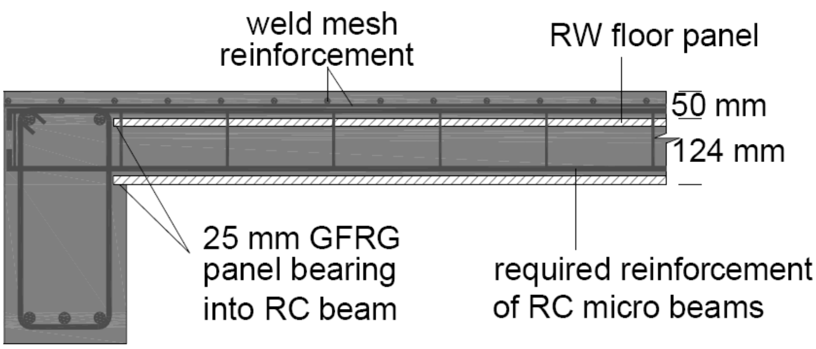
a) Section C-C



b) Section D-D

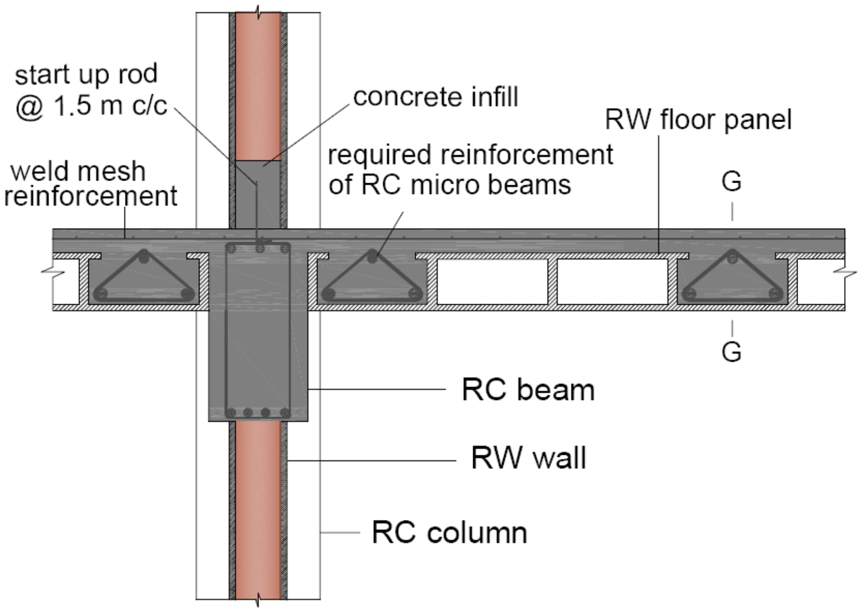


c) Cross section of external beam with floor slab across RC micro beams

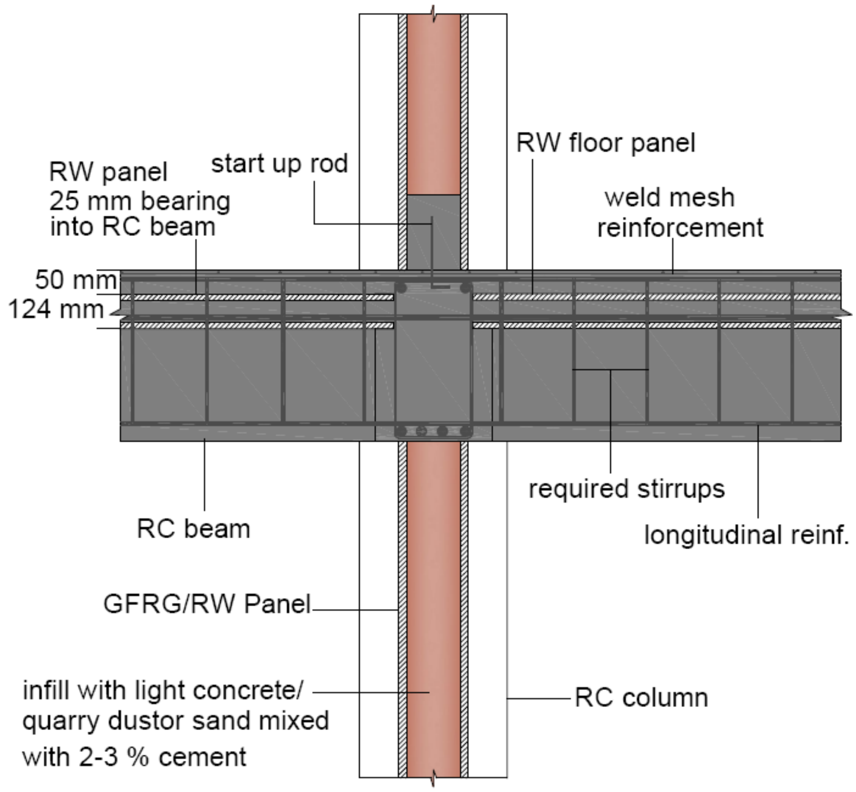


d) Cross section of external beam with floor slab and longitudinal section of micro beams

Fig. A3.4 Cross section of RC beam with RW-RC floor slab

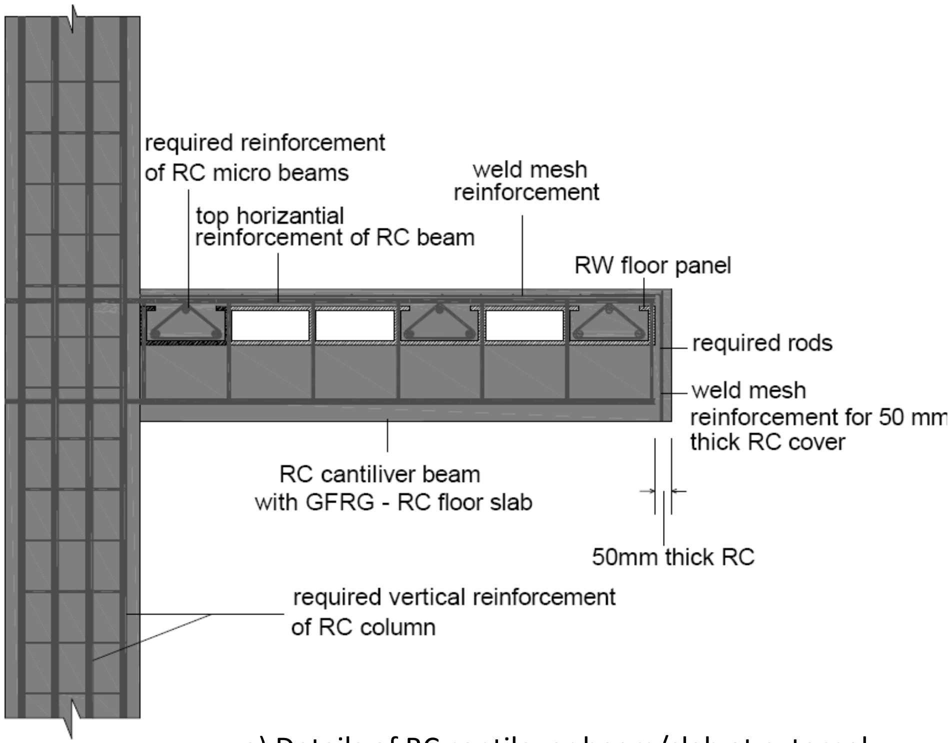


a) cross section of embedded micro beams

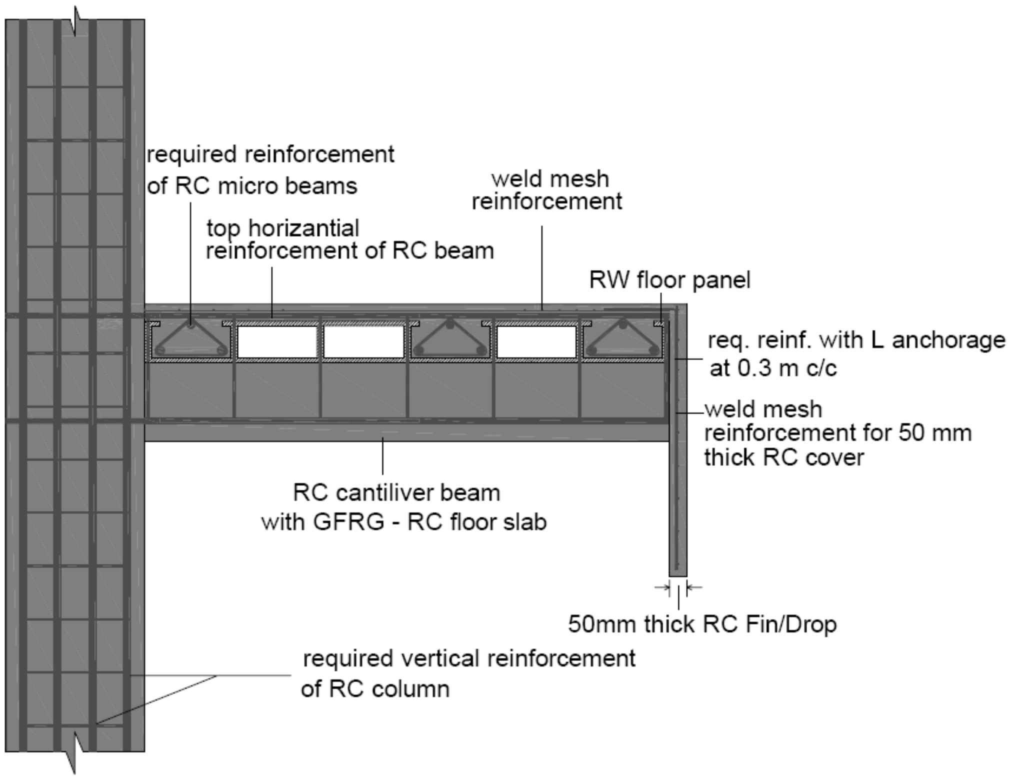


b) section through micro beam

Fig. A3.5 Cross section of RC beam and columns with GFRG slab



a) Details of RC cantilever beam/slab at external face to use as balcony etc



b) Details of RC cantilever beam/slab at external face with RC fin to use as balcony etc

Fig. A3.6 Cross-section of RC column and GFRG slab

Annexure 4

Comparison of Construction of conventional reinforced concrete (RC) building (with masonry infill) with GFRG load-bearing building (option 1) and GFRG -RC hybrid building (option 2)

The design of eight storey GFRG load-bearing building demonstrated in Chapter 13 is carried out separately for a conventional reinforced concrete building (with brick masonry infill) and GFRG-RC hybrid building (refer Chapter 15). The comparison between these three types of construction is summarized in Table A4.1.

Table A4.1 Comparison of Conventional RC building with GFRG building and GFRG-RC Hybrid Building, based on structural design for seismic zone III

Description / items	**Types of building**			**Saving/ benefits compared to conventional RCC building**	
	Conventional construction with RCC framed structure & brickwall as alternative	**GFRG Building (Option 1)**	**Hybrid Building Construction (Option 2)**	**Option 1 (GFRG Building)**	**Option 2 (GFRG- RC Hybrid}**
Dwelling units	32 nos	32 nos	32 nos		
Carpet area	32.68 sqm	32.70 sqm	32.70 sqm		
Built up area/ unit	41.19 sqm	36.64 sqm	36.64 sqm	12.42 %	12.42 %
Super built up area / unit	56.85 sqm	52.37 sqm	52.37sqm	8.55 %	8.55 %
Total built up area	1835 sqm (19747 sft)	1676 sqm (18209 sft)	1676 sqm (18029sft)	9.48 %	9.48%a
Dead load of building	3149 MT	1278 MT	1680 MT	59.41%	46.65%
Live load of building	367 MT	335 MT	335 MT		
Dead load + live load	3516 MT	1613 MT	2015MT		
Design load (for gravity load design}	5274MT	2419.5 MT	3022.5 MT	54.21 %	42.68%
Seismic Loads:					
Design base-shear (Lateral Load} in seismic zone 1	129.6MT	54.47MT	71 MT	58.02%	45.22%
Do in seismic zone 111	207.4 MT	87.2MT	113.6MT	57.95%	45.23%
Do in seismic Zone 1V	311.1 MT	130.8MT	170.4MT	57.95%	45.18%
Do in seismic Zone V	466.7MT	196.2 MT	255. MT	57.96%	
Foundation:	216 cum	139 cum	161 cum	35.65 %	25.46 %

RCC raft- qty of RCC	(700 mm thickness)	(350mm thickness)	(500 mm thickness)		
Raft foundation- steel qty	17 MT	9.5MT	12MT	44.12 %	29.41 %
Super structure- RCC qty	496 MT	376 cum	390 cum	24.19 %	21.37%
Superstructure – steel qty	85 MT	31.5MT	70 MT	62.94%	17.64%
CC block/ brick work	896 cum	nil	nil	No brick /cc blocks	No need for brick/ cc blocks
Rapidwall	nil	6250 sqm	6100 sqm	-	-
Cement plastering	9460.97 sqm	10.76 sqm	1394sqm	99.88%	85.26 %

Above comparative analysis is based on three separate estimates of the BOQs referring to separate 3 structural designs made by IIT Madras (based on three methods of construction).

The above quantity comparison brings out the relative economic advantage in the proposed GFRG construction using RC frames with GFRG infills and RC-GFRG composite floor slabs .

Annexure 5

STRUCTURAL DETAILS OF GFRG BUILDINGS

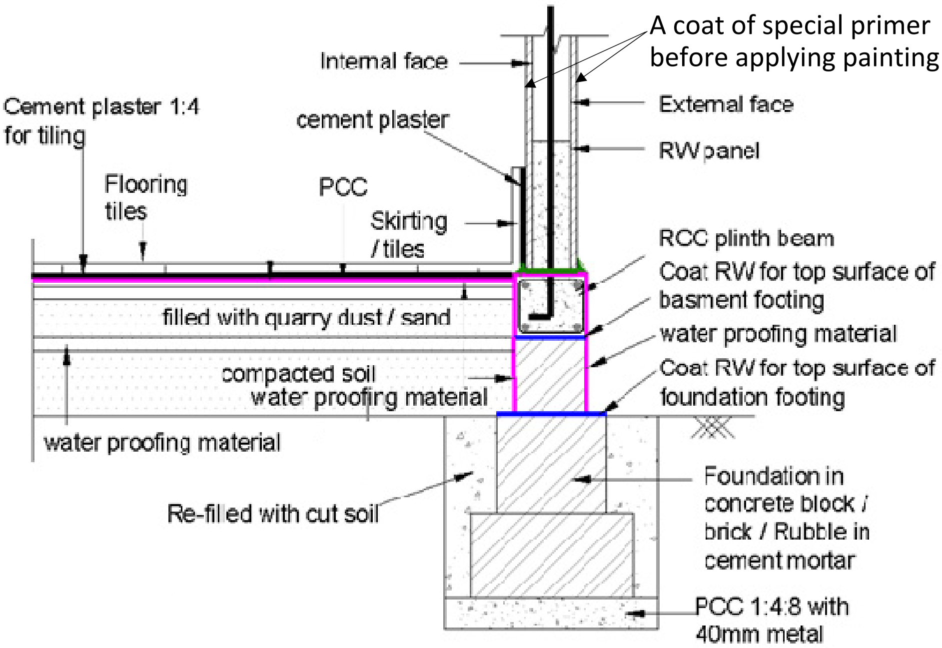


Fig. A5.1 Foundation, basement, plinth beam and start-up bar for erection of GFRG panels

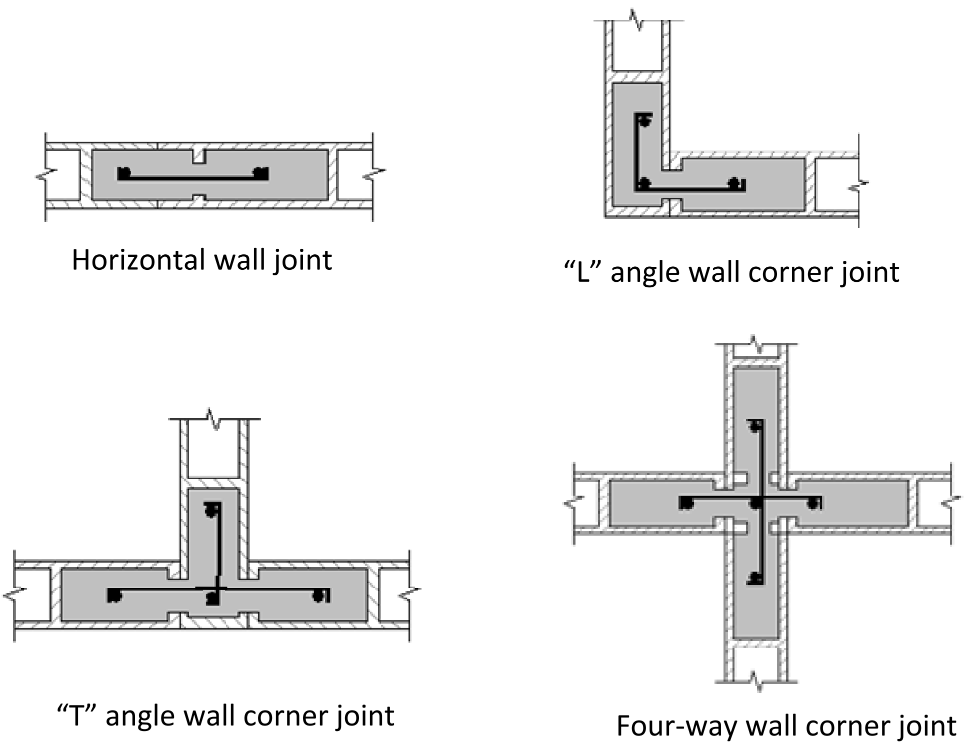


Fig. A5.2 Typical wall corner joints

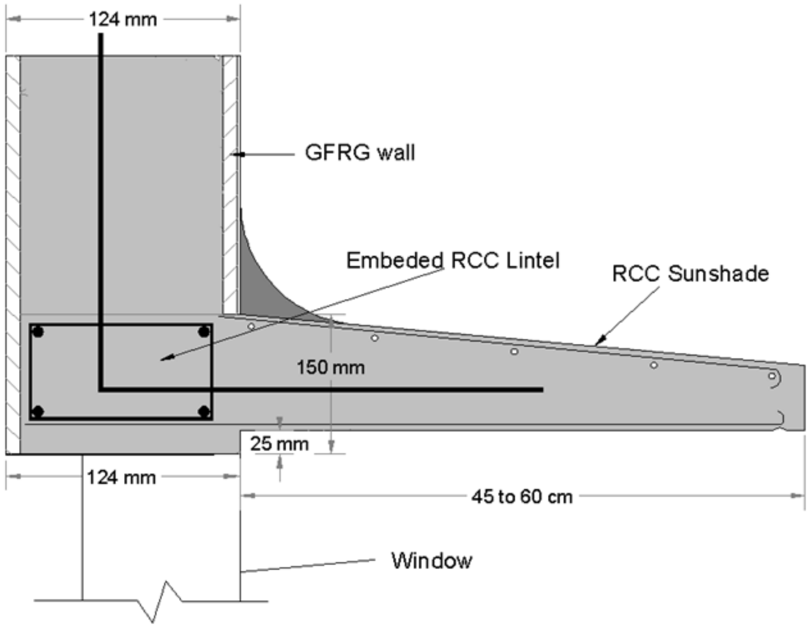


Fig. A5.3 Connectivity between lintel, sunshade and GFRG panel

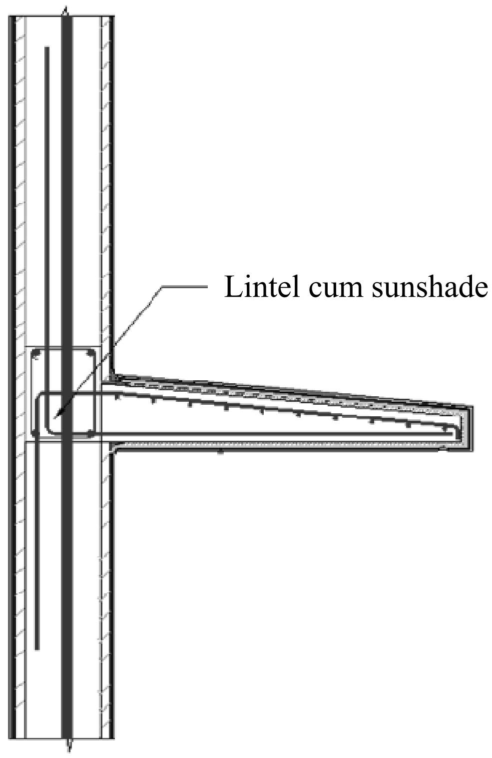


Fig. **A5.4** Lintel, sunshade and GFRG wall

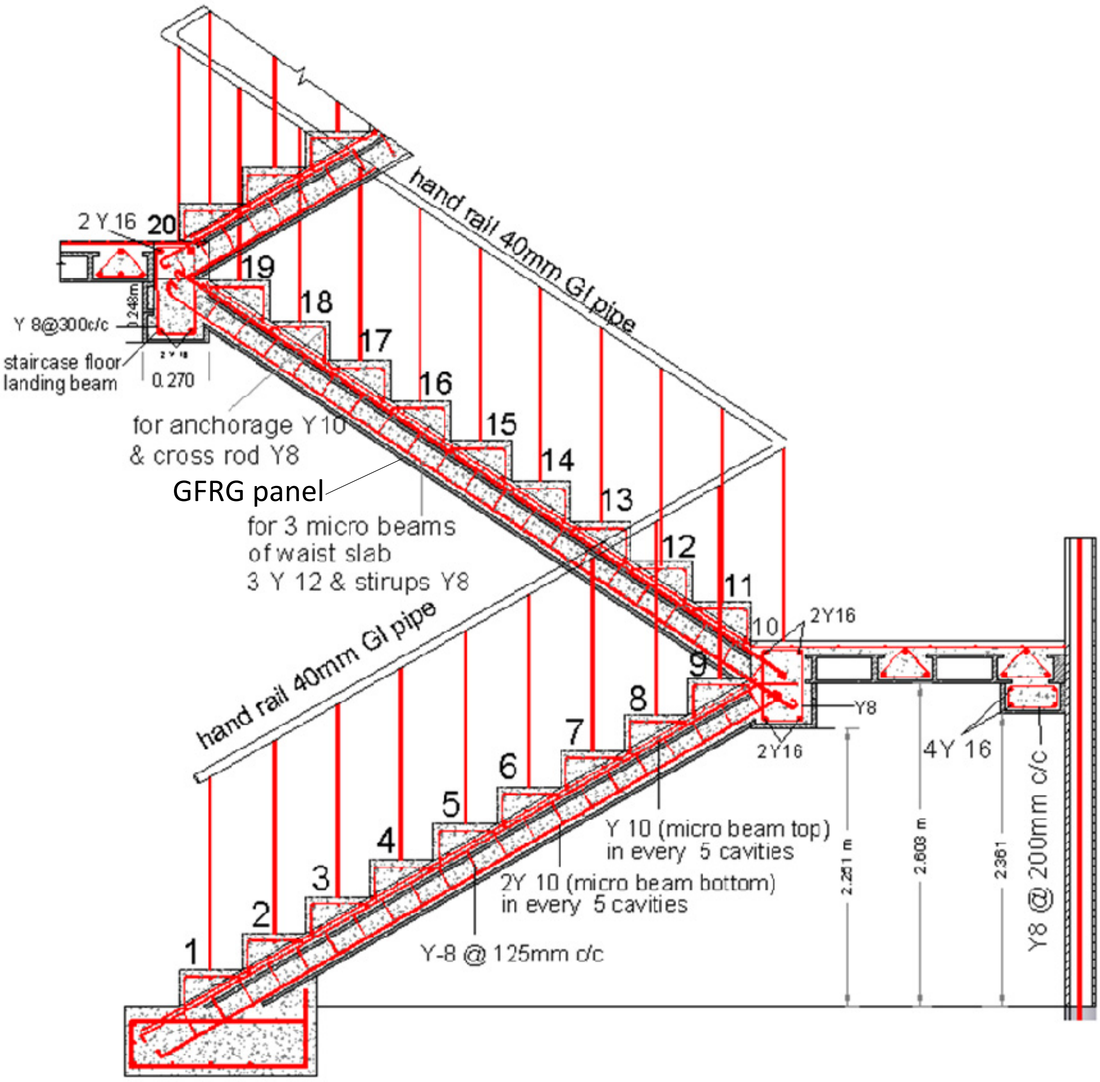


Fig. **A5.5** Cross-section of staircase

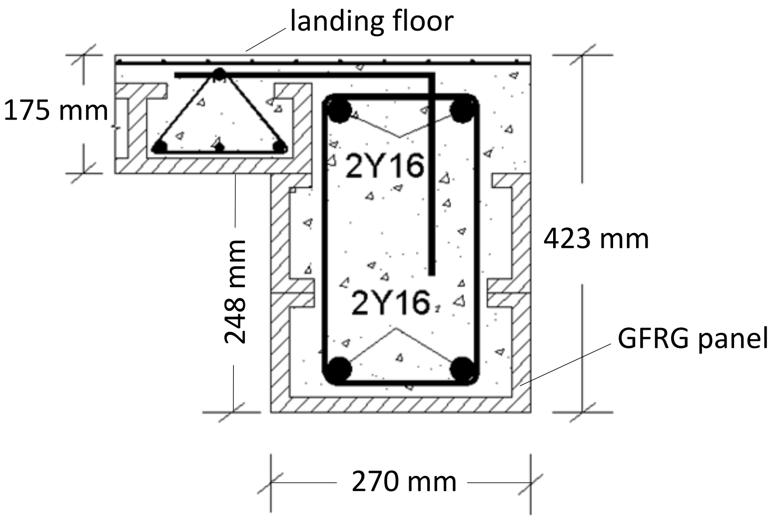


Fig. **A5.6** Staircase floor landing beam

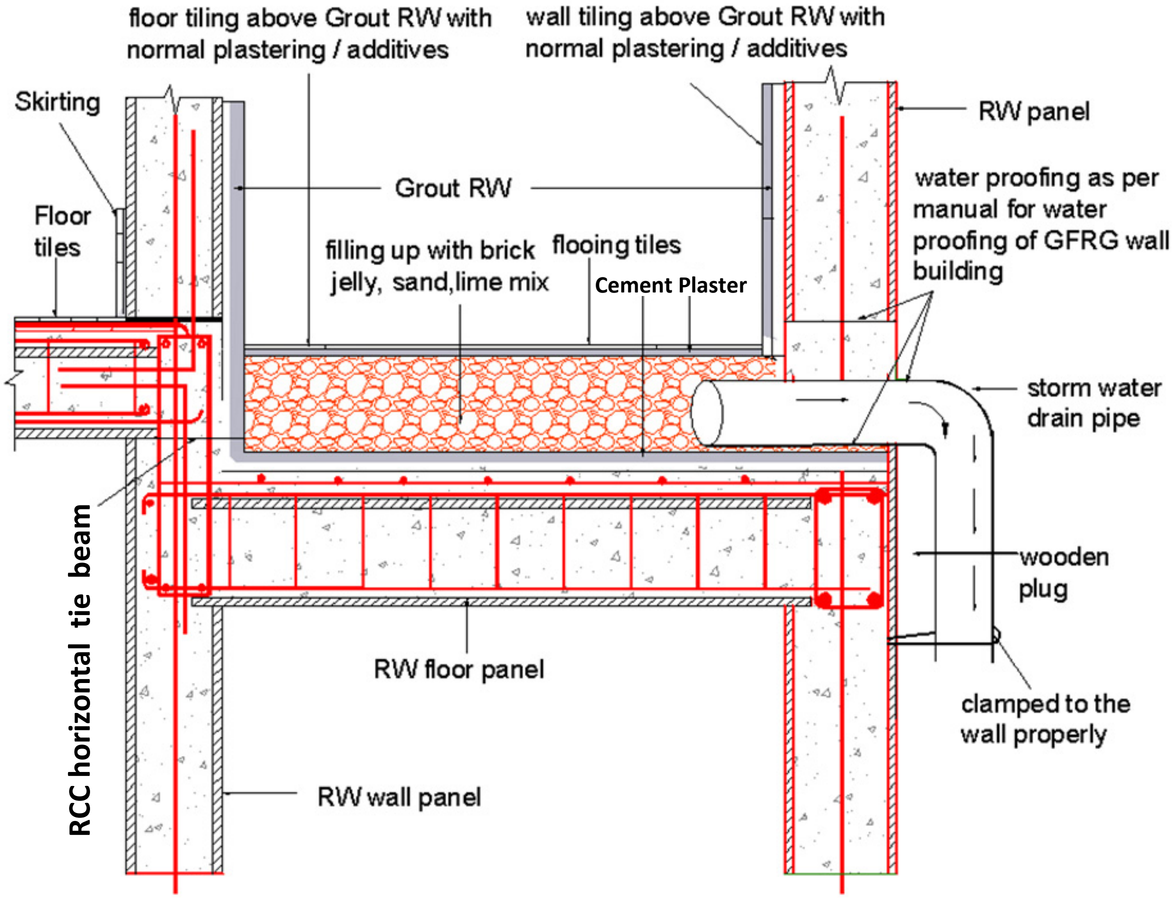


Fig. A5.7 Cross-section of sunken floor (to be used in bath/toilet/wet area)

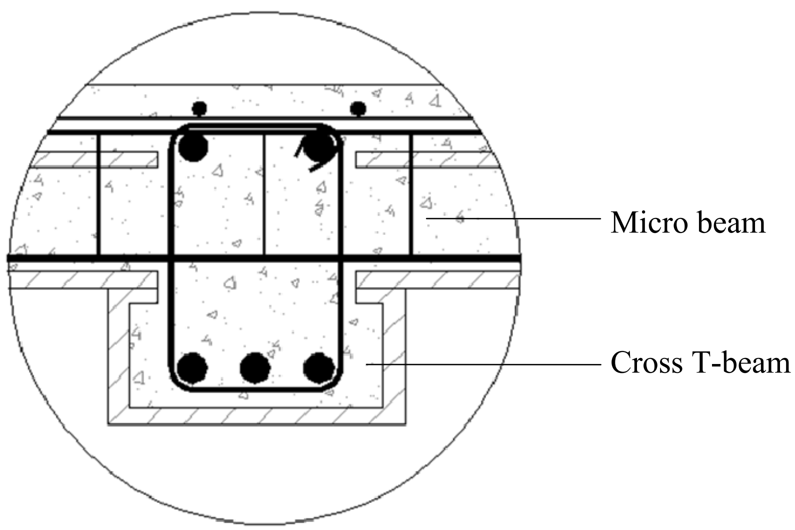


Fig. A5.8 GFRG-RC composite embedded cross beam in long span floors

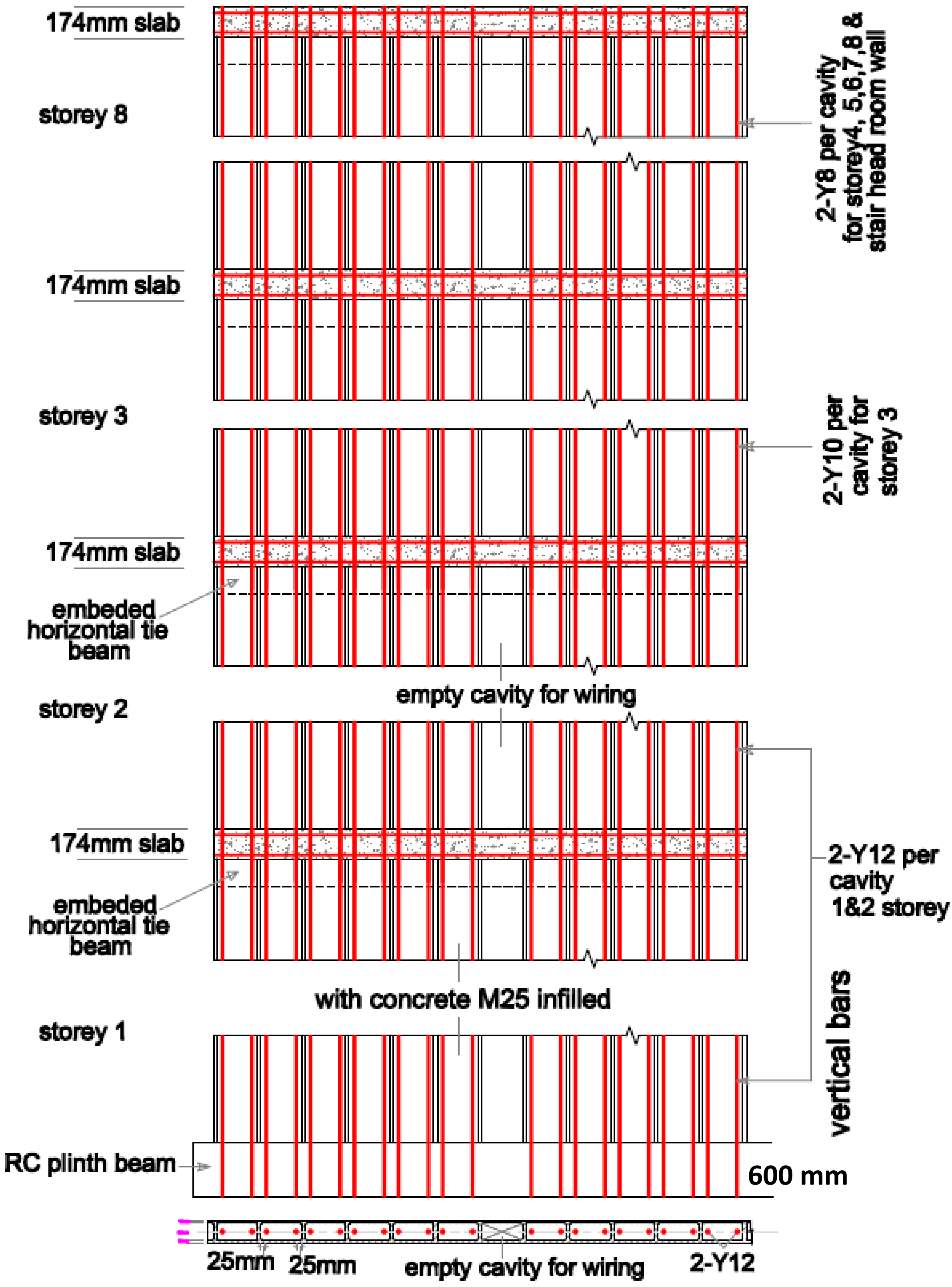


Fig. **A5.9** Longitudinal sectional view of walls showing vertical reinforcement details

REFERENCES

1. RAPIDWALL® (2002) Engineering Design Guidelines, compiled by Ms Dare Sutton Clarke Engineers, Adelaide, Australia.
2. IS: 456 (2000), Code of practice for plain and reinforced concrete for general building construction, Bureau of Indian Standards, New Delhi.
3. IS: 1905 (1987), Code of practice for structural use of unreinforced masonry, Bureau of Indian Standards, New Delhi.
4. SERC (2002), Evaluation of seismic performance of gypcrete building panels, Structural Engineering Research Centre, Chennai, India; August 2002. Project no. CNP 053241/2.
5. SERC (2002), Investigation on the behaviour of gypcrete panels and blocks under static loading, Structural Engineering Research Centre, Chennai, India; August 2002. Project no. CNP 053241/1.
6. IITM (2002), Material properties and assessment of gypcrete building panels, Indian Institute of Technology, Madras, India; September 2002. Project no. CE/BTCM/2557/2002.
7. Sreenivasa, R. L (2010), Strength and behaviour of glass fibre reinforced gypsum wall panels, Indian Institute of Technology Madras, PhD Thesis.
8. Janardhana, M.(2010), Cyclic behaviour of glass fibre reinforced gypsum wall panels, Indian Institute of Technology Madras, PhD Thesis.
9. Wu, Y. F. (2009), The structural behaviour and design methodology for a new building system consisting of glass fibre reinforced gypsum panels, Construction and Building Materials, Volume 23, 2905–2913.
10. Wu, Y. F. and M. P. Dare (2004), Axial and shear behaviour of glass fibre reinforced gypsum wall panels: tests. Journal of Composites for Construction ASCE, 8(6), 569–78.
11. Wu, Y. F.(2004), The effect of longitudinal reinforcement on the cyclic shear behaviour of glass fibre reinforced gypsum wall panels: tests. Engineering Structures, 26(11): 1633–46.

12. Wu, Y.F., and M. P. Dare.(2006), Flexural and shear strength of composite lintels in glass fibre reinforced gypsum wall constructions. Journal of Materials in Civil Engineering ASCE, 18(3), 415–23.
13. Liu, K., Wu, Y. F. and X. Jiang (2008), Shear strength of concrete filled glass fibre reinforced gypsum walls. Material and Structures, 41(4), 649–62.
14. IS: 875 (1987), Imposed Loads (second revision). Bureau of Indian Standards, Part –2, New Delhi.
15. IS: 875 (1987), Wind Load (second revision), Bureau of Indian Standards, Part –3, New Delhi.
16. IS: 875 (1987), Snow Load (second revision), Bureau of Indian Standards, Part –4, New Delhi.
17. IS: 875 (1987), Special loads and load combinations (second revision), Bureau of Indian Standards, Part –5, New Delhi.
18. IS: 1895 (2002), Criteria for Earthquake Resistant Design of Structures-Bureau of Indian Standards, Part I, New Delhi.

About BMTPC

The Building Materials & Technology Promotion Council (BMTPC) under the aegis of the Ministry of Housing & Urban Poverty Alleviation strives to propagate cost effective, energy efficient, eco-friendly and disaster resistant construction technologies for field level applications. Over the years, BMTPC has successfully transferred many alternate building materials & construction systems, developed standards & specifications and brought out meaningful publications, brochures, guidelines for better advocacy and outreach. However, in the recent years in the backdrop of acute housing shortage, it has been realised that potential emerging technologies for social mass housing is the need of the hour and therefore, BMTPC is making concerted efforts so as to identify, study and propagate new technologies. In the process, BMTPC has successfully identified number of technologies and the same are being studied for implementation in Indian conditions through Performance Appraisal Certification Scheme (PACS) being operated by BMTPC. These emerging technologies are being studied so as to bring speed, quality, economy and safety against natural hazards over the conventional way of construction. With fast depleting natural resources; need for environment protection to protect greenhouse effect; need for bringing more speed, durability and quality in construction; it is prudent to bring alternate technologies from within and outside the country.