UNIVERSITY OF SOUTHAMPTON

FACULTY OF ENGINEERING, SCIENCE AND MATHEMATICS

School of Civil Engineering and the Environment

The behaviour of modern flexible framed structures undergoing differential settlement

by

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ABSTRACT

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THE BEHAVIOUR OF MODERN FLEXIBLE FRAMED STRUCTURES UNDERGOING DIFFERENTIAL SETTLEMENT

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Modern office buildings are often open plan buildings with a frame consisting of flat RC slabs, RC columns and non-load bearing internal and external partitions and facades. These modern framed structures are more flexible than older conventional buildings with load bearing walls and are less susceptible to differential settlement damage. The use of conventional guidelines for differential settlement on modern flexible framed structures may therefore be over-conservative.

The literature review of the study highlights the factors producing differential settlement, the types of damage caused by differential settlement and conventional guidelines for limiting differential settlement damage. Conventional guidelines focusing on 2D structures lack provision for the 3D deformation of a structure.

To determine the behaviour of a modern flexible framed structure a numerical experiment was performed, which consisted of the design according to British Standards and Eurocodes of a 3D, 5-bay by 5-bay, 6 storey flat slab RC frame with pad foundations on clay. The behaviour of the designed structure undergoing differential settlement was then analysed by means of linear-elastic finite element analyses.

The results show firstly that it is possible to normalise structural behaviour to the soil-structure stiffness ratio, secondly the importance of 3D deformation of the structure and thirdly that stiffer load-displacement responses of foundations may also affect the behaviour of the structure. A stiffer load-displacement response may occur with the reuse of foundations.

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Declaration of Authorship

I, Gerrit Smit, declare that the thesis entitled “The behaviour of modern flexible framed structures undergoing differential settlement” and the work presented in it are my own and have been generated by me as the result of my own original research. I confirm that:

- This work was done wholly or mainly while in candidature for a research degree at this University;
- Where any part of this thesis has previously been submitted for a degree or any other qualification at this University or any other institution, this has been clearly stated;
- Where I have consulted the published work of others, this is always clearly attributed;
- Where I have quoted from the work of others, the source is always given. With the exception of such quotations, this thesis is entirely my own work;
- I have acknowledged all main sources of help;
- Where the thesis is based on work done by myself jointly with others, I have made clear exactly what was done by others and what I have contributed myself;
- None of this work has been published before submission.

Signed:

Date: 06 August 2010

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List of symbols

Symbol	Descriptions
a	raft radius
A	area
b	raft width
E	Young's modulus
E_c	Young's modulus of concrete
E_r	Young's modulus of the raft
E_s	Young's modulus of the soil
G	shear modulus
h	height
H	height
I	second moment of area
K	relative raft stiffness
l	distance
L	length
n	number
p	stress per unit area
P	total vertical load
p_{av}	average surface stress per unit area
q	uniform load
r	distance from raft centre
t	raft thickness
α	angular strain
α_A	area load reduction factor
α_n	storey load reduction factor
α_T	expansion coefficient
β	relative rotation or angular distortion
δ	differential settlement
Δ	relative deflection
ε	strain
v	Poisson's ratio
v'	effective Poisson's ratio
v_r	Poisson's ratio of the raft
v_s	Poisson's ratio of the soil

θ rotation

ρ settlement

ρ_t total settlement

ρ_u immediate settlement

ρ_z vertical displacement

ρ^* relative bending stiffness

σ_z vertical pressure

ω tilt

1 INTRODUCTION

The objective of the research was to investigate the behaviour of modern flexible framed structures undergoing differential settlement. Differential settlement in structures is important because differential settlement of foundations often leads to damage within the structure (Burland *et al.*, 2001a). The impact of differential settlement on a structure has been widely investigated since the late 1940's (Meyerhof, 1947, Chamecki ,1956, Skempton and Macdonald, 1956, Polshin and Tokar, 1957, Jennings and Kerrich, 1962, Brown, 1969a, 1969b, Grant *et al.*, 1974, Burland and Wroth, 1975, Burland *et al.*, 1977, Jardine *et al.*, 1986, Boscarding and Cording, 1989, Boone, 1996, Potts and Addenbrooke, 1997, Potts *et al.*, 1998, Burland *et al.*, 2001a). Although widely researched, progress was often hampered by:

- the lack of rigorous methods to describe foundation movement,
- the lack of rigorous methods to describe the type of structure,
- the lack of rigorous methods to describe the damage to the structure; and
- different methods used to predict differential settlement and propose guidelines.

This section briefly describes the current state of the art and the shortcomings it has with respect to modern flexible framed structures. The following state of the art and the shortcomings topics are discussed:

- Description of foundation movement.
- The type of structure.
- Description of damage; and
- Methods and guidelines used to predict differential settlement.

To define foundation movement Terzaghi (1935) stated for an adequate description of foundation movement precise levels needs to be taken of at least 15 or 20 points scattered over the entire area occupied by the building. This will give a 3D representation of the building deformation. Skempton and MacDonald (1956) used a simplified 2D approach and defined 'angular distortion' as the ratio of differential settlement and the distance between two points after eliminating the influence of tilt on the building to describe differential settlement. Polshin and Tokar (1957) defined 'slope' as the difference of settlement of two adjacent supports relative to the distance between them (similar to 'angular distortion') and 'relative deflection' as the ratio of deflection to the length of the deflected part to describe differential settlement. In 1974

Burland and Wroth (1975) suggested a whole set of definitions to define differential settlements in 2D. The definitions are often used in later papers to describe differential settlement (Burland *et al.*, 1977, Burland *et al.*, 2001a). These definitions have the limitation that they only describe movement in a plane (2D) and are useful for describing the behaviour of 2D frames and 3D buildings with minimal lateral deformation (which is often the case with long buildings and where settlement occurs due to tunnelling perpendicular to the building). However square buildings undergoing differential settlement due to self weight may have significant deformation in the corners, which is difficult to describe in 2D. The research presented in this report investigated the behaviour of modern flexible 3D structures and the results showed that description of deformation only in 2D is insufficient.

The type of structure influences the response to differential settlement. Skempton and Macdonald's (1956) guidelines were limited to traditional steel and concrete frame buildings and structures with load bearing walls. The 1955 Building Code of the USSR treated framed structures separately from load bearing structures with much stricter criteria being laid down for load bearing brick buildings. Meyerhof (1956) also treated framed panels and load bearing brick walls separately. Guidelines are often presented in text books or design recommendations without emphasising the type of structure the guidelines are applicable to (Burland *et al.*, 1977). Current state of the art guidelines are focussed on relatively stiff load bearing brick buildings. The use of these guidelines on modern flexible framed structures may therefore be over conservative. The research presented in this report investigated modern flexible framed structures and confirmed that these types of buildings are less susceptible to differential settlement damage.

Damage to a structure is very subjective and depends on both the function of the building and the reaction of the users and is difficult to quantify (Burland and Wroth, 1975). Peck *et al.* (1956) have shown that a certain amount of cracking is unavoidable if the building is to be economical. Little (1969) has estimated that for a particular type of building the cost to prevent cracking may exceed 10% of the total building cost. Therefore it will be likely that some damage will occur and the level of acceptable damage needs to be defined. Skempton and Macdonald (1956) divided damage into the following 3 categories:

- 'Structural' involving only the frame, i.e. stanchions and beams.
- 'Architectural' involving only the panel walls, floors or finishes.
- Combined structural and architectural damage.

They also stated that architectural damage such as cracking of wall panels, is likely to occur at smaller distortions of the building than for structural damage. This may be true for conventional buildings, however it does not necessarily hold for modern flexible structures. Skempton and Macdonald (1956) also note that the limits of allowable settlement may be due to visual effects; notably the tilt or lean of a building. Burland *et al.* (1977) suggested a classification system to quantify building damage objectively. The damage classification system is based on the ease of repair of visible damage and is based on the work of Jennings and Kerrich (1962), the UK National Coal Board (1975) and Macleod and Littlejohn (1975). Since then it has been adopted with only slight modifications by BRE (1981, 1990), the Institution of Structural Engineers, London (1978, 1989, 1994, 2000) and BRE again in Freeman *et al.* (1994). Burland *et al.* (2001a) point out that the classification system was developed for brickwork and blockwork or stone masonry. It could be adapted for other forms of cladding and it is not intended to apply to reinforced concrete structural elements. These guidelines may therefore not be directly applicable to modern flexible framed structures. The research presented in this report investigates which type of damage is likely to occur first in modern flexible framed structures. In conventional load bearing structures cracking of facades or infill panels is usually the limiting factor.

Initially two approaches were followed to address the impact of differential settlements on structures. Meyerhof (1947) analysed the interaction between a 2D frame and the soil and calculated the effect it has on the stress within the structure. Skempton and MacDonald (1956) and Grant *et al.* (1974) followed an observational approach based on case studies. They measured the differential settlement and corresponding damage on existing buildings and suggested differential settlement limits based on the study. The study showed that an angular distortion of greater than 1/150 will cause structural damage and an angular distortion of greater than 1/300 will cause cracking in walls and partitions and they recommended that angular distortions greater than 1/500 should be avoided if it was important to avoid differential settlement damage.

In contrast Meyerhof's (1947) frame analysis showed that a lesser angular distortion of 1/950 will cause an increase of 74% in the bending moment in the beam which was subjected to the largest bending moment prior to differential settlement. Skempton and MacDonald (1956) argued that an angular distortion of 1/950 did not result in damage in the observed structures. They suggested the reason for it may be that the live loads assumed in the design are conservative compared to the actual average loads in buildings and the composite action of the frame floors and walls may reduce the stresses and deflections within the building.

Burland and Wroth (1975) and Burland *et al.* (1977) used the concept of a tensile strain limit to study the development of cracking in weightless elastic beams undergoing deflections, which is a simplified representation of a building. Studies done by Polshin and Tokar (1957) and Burland and Wroth (1975) showed that cracking in walls and finishes usually results from tensile strain and that for a given material visible cracking is associated with a reasonably well defined value of strain that is insensitive to the mode of deformation. Boscardin and Cording (1989) associated tensile strain values with the damage categories as proposed by Burland *et al.* (1977). Strain distribution within the simple beam depends on the mode of deformation and two extreme modes of bending only about a neutral axis and shear only were analysed. Both modes occur simultaneously and need to be calculated to determine whether bending or diagonal strain is limiting. Using an expression for midspan deflection of a centrally loaded beam having both bending and shear stiffness (Timoshenko, 1957), a limiting value of deflection can be calculated for a given tensile strain limit by taking the building length and height, Young's modulus and shear modulus and the position of the neutral axis into account. Boscardin and Cording (1989) expanded the above analysis to include horizontal strain which is associated with tunnelling.

The above method was used successfully on the Jubilee Line extension (Burland *et al.*, 2001a) and its simplicity is a big advantage when it comes to the analyses of structures, however it is important to realise the simplifications and limitations of the method. The method assumes that the deformation of the structure is predominantly 2D and that the structure behaves like a beam. These assumptions may be true for conventional buildings; however modern flexible framed structures may behave differently.

The advance in modern computers and modern finite element software packages allows for the analyses of 3D models with increased complexity. Simplified 2D models may provide valuable insight and use fewer resources; however full 3D models can include more detail and show the shortcomings of simplified 2D models. The research presented in this report makes use of the capability to analyse a 3D structure to investigate the behaviour of modern flexible framed structures.

In recent years the need for more sustainable development has become an important factor in the design of new buildings. In areas of frequent redevelopment the reuse of foundations of the demolished buildings is encouraged (Chapman *et al.*, 2001, Chow *et al.*, 2002, Cameron and Chapman, 2004) to save resources. However in practice old foundations are often discarded in favour of new foundations at extra cost even when the old foundations are located in the correct position, in good condition and capable of withstanding the design loads. This is due to the uncertainty involved in how the structure will behave if older preloaded foundations with a stiffer response are combined with new foundations. Understanding the behaviour of modern flexible structures undergoing differential settlement will pave the way for the reuse of foundations. It will also be valuable in assessing the risk of foundation (old or new) failure on newly constructed buildings.

The thesis first reviews and discusses the factors producing differential settlement and the current state of the art on the behaviour of structures undergoing differential settlement in Chapter 2. A linear-elastic model is developed and verified in Chapter 3 and typical results are presented. Chapter 4 discusses the behaviour of structures undergoing differential settlement. The discussion is based on the results. Chapter 5 presents the conclusions of the literature review, the development of the numerical model and the discussion of the results, as well as suggestions for future work.

2 LITERATURE REVIEW

The literature review firstly defines differential settlement. Secondly the factors producing differential settlement are discussed with specific reference to soil variability, loads and their variability, foundation load-displacement response and structure stiffness. Thirdly the difference between conventional old buildings and new flexible framed buildings and the reuse of foundations are discussed. Fourthly the damage resulting from differential settlement is discussed and at the end of Chapter 2 the conclusions from the discussion of the literature are summarised.

2.1 Defining differential settlement

The complete description of the settlement of a structure requires a large number of observation points so that detailed profiles of foundation movement can be plotted (Terzaghi, 1935). Differential settlement is a general term used to describe the differences in vertical displacement of foundations. However differential settlement on its own does not give any indication of the spatial variation. It is the magnitude of differential settlement combined with the spatial variation that influences the behaviour of the structure (Skempton and Macdonald, 1956, Burland and Wroth, 1975). To be able to describe the movement of foundations more rigorously, definitions are needed. This section considers definitions of differential settlement and their shortcomings.

Skempton and Macdonald (1956) suggested the use of angular distortion to describe differential settlement. They defined angular distortion as the ratio of the differential settlement (δ) and the distance (l) between two points after eliminating the effect of tilt of the building. Polshin and Tokar (1957) defined a slope, equivalent to angular distortion; and relative deflection as the ratio of deflection to the length of the deflected part. Subsequently similar definitions have been defined by a number of authors (Fjeld, 1963, Grant *et al.*, 1974, Burland and Wroth, 1975, Wahls, 1981, Burland *et al.*, 2001a).

Burland and Wroth (1975) proposed a consistent set of definitions based on the displacement of a number of discrete points on the foundation of a building. The definitions have been widely accepted, are illustrated in Figure 2.1 and defined as follows (Burland *et al.*, 2001a):

- *Rotation* or *slope* θ is the change in gradient of a line joining two reference points (e.g. AB in Figure 2.1(a)).
- The *angular strain* α is defined in Figure 2.1(a). It is positive for upward concavity (sagging) and negative for downward concavity (hogging).
- *Relative deflection* Δ is the displacement of a point relative to the line connecting two reference points on either side (see Figure 2.1(b)). The sign convention is as for angular strain.
- *Deflection ratio* (sagging ratio or hogging ratio) is denoted by Δ/L where L is the distance between the two reference points defining Δ. The sign convention is as angular strain.
- *Tilt* ω describes the rigid body rotation of the structure or a well defined part of it. See Figure 2.1(c).
- *Relative rotation* (angular distortion) β is the rotation of the line joining two points, relative to the tilt ω. See Figure 2.1(c). It is not always straightforward to identify the tilt and the evaluation of β can sometimes be difficult. It is also very important not to confuse relative rotation β with angular strain α. For these reasons Burland and Wroth (1975) preferred the use of deflection ratio Δ/L as a measure of building distortion.

The above set of definitions provides a way of describing differential settlement, however in practice it is often difficult to know the precise deformed shape between observation points (Burland and Wroth, 1975). Therefore care should be taken in defining suitable observation points.

The definitions provide only a description of in plane movement and no attempt is made to describe 3D deformation of a structure. The definitions are therefore suitable for structures that deform primarily in plane, however 3D deformation needs to be described for structures with 3D behaviour. Describing 3D deformation is more complex than 2D deformation and no straightforward definitions exist.

2.2 Factors producing differential settlement

Differential settlement may cause damage to structures. A good understanding of mechanisms and factors producing differential settlement will result in a better understanding of the behaviour of the structure and will therefore allow for a more optimal design. The following sections will discuss the impact of soil variability, loads and their variability, foundation load-displacement response and building stiffness on differential settlement.

2.2.1 Soil variability

This section considers soil variability, since varying stiffness and strength of soil beneath foundations is a potential cause of differential settlement. Inherent soil variability and the parameters describing it are considered, as are uncertainties in measurement and transformation models of soil properties. Typical values of variation are also discussed.

Soils are heterogeneous materials created by complex geological processes. Soil properties vary from point to point, even in the same strata. Terzaghi (1955) discussed how soil variability can be linked to complex depositional conditions. Fookes (1997) discusses the value of a comprehensive geological model in understanding soil conditions on a site, but also states that, regardless of the detail and the amount of work involved, the geological model is unlikely to achieve the same qualitative accuracy as the structural engineering design because of the inherent complexity and inhomogeneity of the soil. Other researchers have investigated and quantified the spatial variability of natural soils (Phoon and Kulhawy, 1999, Bourdeau and Amandaray, 2005, El Gonnouni *et al.*, 2005). Table 2.1 (Phoon and Kulhawy, 1999) shows a summary of inherent variability of strength properties of various soils. Table 2.2 (Phoon and Kulhawy, 1999) shows inherent variability of the index parameters of various soils. The Coefficient of Variation (COV) is the Standard Deviation normalised to the mean soil property value. Another parameter needed to describe the variability of soil is the distance of the variability change i.e. the scale of fluctuation. Figure 2.2 shows the scale of fluctuation graphically. Table 2.3 (Phoon and Kulahawy, 1999) summarises vertical and horizontal fluctuations of some geotechnical properties. From the data it is evident that the scale of fluctuation in a vertical direction is smaller than for a horizontal fluctuation. Horizontal fluctuations typically range from 3.0 m to 80.0 m, which means within the footprint of structure, the soil properties can vary significantly.

Inherent soil variation needs to be distinguished from variation in measured values due to inaccurate measurement and transformation models (Phoon and Kulhawy, 1999).

Measurements of soil properties are dependent on the test method, apparatus used and operator expertise. Testing of an inherently homogeneous soil will result in a measured variation in soil properties. Therefore soil tests need to be standardised to minimise variation due to measurement error. It is widely recognised that the SPT is associated with greater testing uncertainty than the CPT (Jaksa *et al.*, 2005). Lee *et al.* (1983) suggested that the COV for SPT varies between 27% and 85% and Phoon and Kulhawy (1999) suggested it varies between 25% and 50%. Orchant *et al.* (1988) suggested that the COV for CPT varies between 7% and 12% and Phoon and Kulhawy (1999) suggested it varies between 5% and 40% in clays.

Variations also occur due to different transformation models. For example soil properties can be derived from a standard penetration test or a cone penetration test. Young's modulus is often used as a design or analysis parameter. The pressuremeter test or the dilatometer test provides a direct measurement of soil modulus. Phoon and Kulhawy (1999) provide the following data on the COV of soils. The inherent variability COV for the pressuremeter test in sand is estimated between 15% and 65%. The measurement error COV was estimated between 10% and 20%. Using this numerical data the total COV for the pressuremeter in sand is between 18% and 68%. For dilatometer test the total COV is estimated between 16% and 67% which is similar to the pressuremeter.

For soil variability to have an effect on settlement, it must be within the stress influence zone of the foundation. For a flexible square foundation, the vertical stress at a depth of 3B, where B equals the width of the foundation, is less than 6% of the surface stress (Atkinson, 1993). Therefore most soil compression will occur within a 3B depth and the focus should be on soil variations within this zone.

2.2.2 Loads and their variability

This section considers building loads, since variation of loads across foundations is a potential cause of differential settlement. This section discusses the following:

- Live loads with reference to:
    - Measured live loads in offices.
    - Measured construction loads.

- Wind loads.
- Dead loads with reference to:
    - Concrete frames.
    - Steel frames.
- Code recommendations with reference to:
    - British Standards Institution.
    - Eurocodes.
- The magnitude of thermal loads.
- Summary and discussion of the live loads and dead loads of both steel and concrete buildings.

Live loads on floors comprise all non-permanent gravity loads including furniture. This includes desks, chairs, computers, safes, cupboards, etc., their contents, movable internal partition walls with partial height and loading due to personnel.

Table 2.4 provides a summary of studies on sustained live loads in office buildings in various countries, from Kumar (2002), Ruiz and Soriano (1997), Choi (1992), Culver (1976); and Mitchell and Woodgate (1971). The difference between the live load survey results may be attributed to cultural backgrounds, the habit of using office appliances, difference in methodology, time interval between surveys and the sample size of surveys (Kumar, 2002). The mean values range from 0.31 to 0.83 kN/m² with an average of 0.54 kN/m². The standard deviations range from 0.15 to 0.82 kN/m² with an average of 0.40 kN/m². From the tabulated data it is evident that an increase in room floor area usually leads to a decrease in sustained live load. Kumar (2002) found in the Indian study that the personnel load contributed to 30.5% of the live loads in office buildings. The maximum load measured in the survey in India (Kumar, 2002) was 2.05 kN/m² in a store room. Although this is approximately 4 times the average sustained load, it was localised to one room.

Construction loads on floors consist of construction workers, construction machinery, stacking of construction materials and loads due to props supporting the next floor slab. Ayoub and Karshenas (1994) surveyed live loads on newly poured slabs and suggested a mean equivalent uniformly distributed construction live load of 0.3 kN/m² with a COV of 0.32 kN/m² on newly poured slabs (i.e. uppermost floor). Beeby (2001) measured loads in backprops in a case study by BRE at Cardington. The spacing of columns was 7.5 m and the floor slab had a thickness of 250 mm throughout. Table 2.5

is a summary of the structural design values and measured floor loads. The maximum imposed load on a floor slab was 10.57 kN/m². The high level of load on the floor slab is due to the load of the backprops on the slab supporting the formwork and casting of the above slab. It is important to note that not all the floor slabs will be subjected to the maximum live load simultaneously. As construction progresses and props are removed live loads will change to dead loads as each floor slab supports itself.

Krishna (1995) describes the complexities in measuring wind loads on buildings. Wind loads on buildings depend on wind strength, direction of the wind with respect to the building, the surrounding area i.e. other buildings; and the geometry of the building. Meecham (1992) has reported that for a hip roof, peak pressures are reduced by as much as 50% compared with those of a gable roof. Likewise Blackmore (1988) has reported on the effect of chamfering building edges at different angles. He has reported that roof loads reduce with increase in chamfer angle. Reductions as high as 70% in average load on a corner panel and 30% in overall design load are observed. It is therefore extremely difficult to generalise average wind load and duration on buildings. Extreme wind loads are usually of a limited duration and are often taken into account for strength and serviceability calculations i.e. vibration of panels. Due to the short duration it normally does not affect settlement of foundations.

Dead loads comprise all permanent gravity loads including the floors, walls, columns, services and finishes. The dead load depends on the materials used within the structure. A concrete structure is usually heavier than a steel structure as shown in the following examples of typical dead loads.

A typical dead load of a concrete frame for an office building may be summarised as follows. (Refer to Chapter 3 for details of the sizing of the elements according to the structural design. Densities derived from BSI 6399-1: 1996.)

- A 300 mm concrete floor slab with a concrete density of 25 kN/m³ results in a distributed load per floor of 7.50 kN/m².
- Concrete columns (2.7 m x 450 mm x 450 mm) spaced at 7.5 m centre to centre will increase the average floor load by 0.23 kN/m².
- Combining these gives an indication of magnitude of expected dead load for the typical concrete frame of 7.73 kN/m² per storey.

A typical dead load of a steel frame for an office building may be summarised as follows. (The loads are derived from worked examples for the design of steel structures as prepared by the Building Research Establishment, The Steel Construction Institute and Ove Arup & Partners (Building Research Establishment, 1994)

- A raised floor on 130 mm lightweight concrete on profiled metal decking results in a distributed load per floor of 2.70 kN/m².
- Beams (406 x 140 x 46 UB) at 2.5 m spacing in the x-direction and beams (610 x 229 x 101 UB) at 7.5 m spacing in the y-direction results in a distributed load per floor of 0.31 kN/m².
- Steel columns (254 x 254 x 73 UC) spaced at 7.5 m centre to centre will increase the average floor load by 0.03 kN/m².
- Combining these gives an indication of magnitude of expected dead load for a typical steel frame of 3.04 kN/m² per storey.

From the above examples it is evident that the typical load of a concrete frame (7.73 kN/m² per storey) is more than twice the load of a similar steel frame (3.04 kN/m² per storey).

The British Standards Codes and the Eurocodes are currently acceptable design codes for the UK and the suggested design loads will therefore be compared. The current suite of British Standards Codes, will in due course be almost entirely replaced by the system of Eurocodes and it is expected that the replacement will be complete by about 2010 (Department for Communities and Local Government, 2006).

Eurocode 7 (EN 1997-1: 2004) offers a choice (or combination) of 4 methods for geotechnical design:

- Using ultimate limit state design calculations for ULS and SLS.
- Using prescriptive measures. Prescriptive measures involve conventional and generally conservative rules in the design and usually involve the application of charts, tables and procedures that have been established from comparable experience.
- Using tests. Designs may be based on the results of load tests; or
- Using the Observational Method. The Observational Method is a continuous, managed, integrated process of design, construction control, monitoring and review that enables previous modifications to be incorporated during or after construction as appropriate.

The discussion of recommended Eurocodes load values will focus on the limit states design (ULS and SLS).

The Codes recommend the following values. References to the specific codes are British Standards Institution (BSI), Eurocodes (EU) and the UK National Annex to Eurocode (UK).

- **Structure design for Ultimate Limit State (ULS).**

    - *Live loads* (BSI 6339-1: 1996, EN 1990: 2002, NA to EN 1990: 2002, EN 1991-1-1: 2002, NA to EN 1991-1-1: 2002, EN 1991-1-3: 2003, NA to EN 1991-1-3: 2003).

      Live loads for general use offices consist of an imposed uniform distributed load (UDL) of 2.5 kN/m² (BSI), 1.5 to 2.0 kN/m² (EU) and 2.5 kN/m² (UK) on floors.

      Movable partitions should be taken as an additional imposed UDL of not less than 1.0 kN/m² (BS). The Eurocodes (EU and UK) distinguish between movable partitions with different self-weights and recommend for self weights ≤ 1.0 kN/m a UDL of 0.5 kN/m², for self weights ≤ 2.0 kN/m a UDL of 0.8 kN/m² and for self weights ≤ 3.0 kN/m a UDL of 1.2 kN/m².

      For roofs with only maintenance access, minimum UDLs of 1.5 kN/m² (BSI), 0.4 kN/m² (EU) and 0.6 kN/m² (UK) are recommended.

      Snow loads need to be considered, however recommended values are site and structure specific and will therefore not be discussed.

      For beam design, reduction of live loads on beams is allowed based on the floor area supported. The British Code (BSI) and UK Annex (UK) recommend a reduction factor (α_A) calculated from:

      $$\alpha_A = 1.0 - \frac{A}{1000} \leq 0.75 \qquad \textbf{Equation 2.1}$$

      Where A is the loaded area (m²) supported.

Eurocodes (EU) recommends a reduction factor (α_A) calculated from:

$$\alpha_A = 0.5 + \frac{10}{A} \leq 1.0 \qquad \textbf{Equation 2.2}$$

Where A is the loaded area (m²) supported.

Figure 2.3 gives a graphical presentation of the area reduction factors. It is evident that the load reduction factor (α_A) for the Eurocodes is approximately 20% lower than the British Standards and UK Annex for areas above 50 m².

For column design, reduction of floor live loads is allowed based on the number of storeys supported by the column under consideration. The British Code (BSI) and UK Annex (UK) recommend a reduction factor (α_n) calculated from:

$$\begin{aligned} \alpha_n &= 1.1 - \frac{n}{10} && \mathit{for}\ 1 \leq n \leq 5 \\ \alpha_n &= 0.6 && \mathit{for}\ 5 < n \leq 10 \qquad \textbf{Equation 2.3} \\ \alpha_n &= 0.5 && \mathit{for}\ n > 10 \end{aligned}$$

Where n is the number of storeys above the loaded structural elements.

Eurocodes (EU) recommends a reduction factor (α_A) calculated from:

$$\alpha_n = \frac{2 + (n-2)0.7}{n} \qquad \textbf{Equation 2.4}$$

Where n is the number of storeys (greater than 2) above the loaded structural elements.

Figure 2.4 gives a graphical presentation of the storey reduction factors and it is evident that the load reduction factor (α_A) for the British Standards and UK Annex is approximately 20% lower than the Eurocodes for more than 3 floors. Combining load reduction factors for both area and number of supported storeys should lead to a smaller difference in predicted column live loads between the codes.

- *Dead loads* (BSI 6399-1: 1996, EN 1991-1-1: 2002)

  Dead loads are calculated using the measured volumes and densities of building materials used. The codes suggest densities for various construction materials. Dead loads include floors, walls, columns, services and finishes.

- *Wind loads* (BSI 6399-2: 1997, EN 1991-1-4: 2005)
  Wind loads are building and site dependent and are influenced by building location, altitude, topography, surrounding terrain (buildings, trees) and building geometry.

- *Partial safety factors* (BSI 8110-1: 1997, EN 1990: 2002, NA to EN 1990: 2002)
  Partial safety factors are applied to above loads for ULS calculations. Combinations of partial factors according to British Standards are summarised in Table 2.6. The Eurocodes distinguish between the following ULS and the appropriate partial factors are summarised in Table 2.7.

  EQU: Loss of static equilibrium of the structure or any part of it considered as a rigid body, where the strength of structural materials and the ground are insignificant in providing resistance.

  STR: Internal failure or excessive deformation of the structure or structural members, including footings, piles, and basement walls, etc., where the strength of structural materials is significant in providing resistance.

  GEO: Failure or excessive deformation of the ground where strength of soils or rock are significant in providing resistance, (e.g. overall stability, bearing resistance of spread foundations or pile foundations).

- **Structure design for Serviceability Limit State (SLS)**

  - *General* (BSI 8110-2: 1985, EN 1990: 2002, NA to EN 1990: 2002)
    For serviceability calculations the codes state that it is necessary to make sure the assumptions made regarding loads are compatible with the way results will be used. If a best estimate of the expected behaviour is required then the expected or most likely values should be

used. However, to satisfy a serviceability limit state it may be necessary to take a more conservative value depending on the severity of the serviceability limit state under consideration, i.e. the consequences of failure (with regard to serviceability limit state).

- *Live loads* (BSI 8110-2: 1985, EN 1990: 2002, NA to EN 1990: 2002)
  Live loads should in general be the characteristic values (as calculated for ULS, with a partial safety factor of one), however when calculating deflections, it is necessary to determine how much of the load is permanent and how much transitory. The British Standards suggest for normal domestic or office occupancy, 25 % of the live load should be considered as permanent and for structures used for storage, at least 75% should be considered permanent when the upper limit to the deflection is being assessed.

- *Dead loads* (BSI 8110-2: 1985, EN 1990: 2002, NA to EN 1990: 2002)
  Dead loads should be the characteristic value (as calculated for ULS, with a partial safety factor of one).

- **Foundation design according to British Standards (allowable bearing pressure)**

  - *Dead and live loads* (BSI 8004: 1986)
    Dead and live loads should be the characteristic value (as calculated for ULS, with a partial safety factor of one). Dead load should include the weight of foundations and any backfill above the foundations.

  - *Wind loads* (BSI 8004: 1986)
    Wind loads resulting in loads on foundations that are less than 25% of the loadings due to dead and live loads may be ignored. Where this ratio exceeds 25% foundations may be so proportioned that the pressure due to combined dead, live and wind loads does not exceed the allowable bearing pressure by more than 25%.

- **Thermal actions**

  The investigation of thermal actions on buildings due to climatic and operational temperature changes falls outside the scope of this research, however it should be considered in the design of buildings where there is a possibility of the ULS or SLS being exceeded due to thermal movement and/or stresses. This section illustrates the possible effect of temperature on a building. Eurocode 1 (EN 1991-1-5: 2003) and the UK Annex (NA to EN 1991-1-5: 2003) provide guidance on calculating temperature changes in buildings.

  The Eurocode suggests inside building temperatures of 20 °C during summer and 25 °C during winter. Outside building temperatures depend on shade air temperature and the type of surface. In the UK minimum shade air temperatures range from -21 °C to -9 °C and maximum shade air temperatures range from 26 °C to 35 °C, depending on location. Table 2.8 shows the surface temperatures to be used for design calculations. For example, according to the Eurocode, in the summer buildings in Southampton will experience an inside temperature of 20 °C, while dark surfaces outside on North-East facing elements will experience 37 °C and dark surfaces South-West or horizontal elements 75 °C, resulting in a 55 °C temperature difference in the building.

  Table 2.9 (EN 1991-1-5: 2003, EN 572-1: 2004) lists coefficients of linear expansion for construction materials. The following example illustrates the impact of thermal expansion on a structure. Assume a structure:

  - with a thermal expansion coefficient of 10 x 10^{-6}/°C for both the internal frame and external facades
  - located in Southampton,
  - during summer; and
  - dark surfaces on facades.

  On West-South sides the facades will be 55 °C warmer than the internal frame. Thermal expansion in an unrestrained 3 m facade will therefore lead to 1.65 mm differential movement of the facade with respect to the frame. Thermal expansion in a totally restrained facade with a Young's modulus of 70 GPa (aluminium or glass (EN 1991-1-1: 2002, EN 572-1: 2004)) will increase the stress in the facade by 38.5 MPa.

It is important to note that not only the magnitude, but also the duration, of load affects the settlement of the building. The impact of load duration on settlement will be discussed under foundation-load displacement response.

Table 2.10 provides a summary of the measured and proposed loads as listed above. The average of the measured live loads is 0.54 kN/m². The codes recommend unfactored values, excluding floor area and storey reductions ranging from 1.50 kN/m² to 4.70 kN/m². The measured loads are significantly lower than the values suggested by the codes. The maximum load in one room measured by Kumar (2002) of 2.05 kN/m² is within the lower end of the range of the codes' recommended live loads. The dead load of the concrete frame in the worked example (7.73 kN/m^2) is more than twice the load for the steel frame (3.04 kN/m^2). If a short term live load of 3.00 kN/m^2 is assumed, it will be approximately 100% of the dead load of the steel frame; however it will only be approximately 40% of the dead load of the concrete frame. The live load on a steel framed structure is therefore a more significant part of the total load in comparison to a concrete framed structure.

2.2.3 Foundation load-displacement response

This section considers foundation response, since varying load-settlement response is a potential cause of differential settlement. Foundation load-displacement response varies significantly and depends on the structural loading on the foundations, the foundation geometry and the soil supporting the foundation. These aspects in combination with measured load-displacement responses are discussed.

Structural loading on a foundation results in settlement of the foundation. An increase in load (while other properties remain constant) will cause an increase in settlement; however the stiffness will usually decrease with an increase of load as seen in the measured load-displacement response of a number of piles in Figure 2.5 and 2.6 (Whitaker and Cooke, 1966, De Beer *et al.*, 1979, Fleming, 1992). Load-displacement response is also dependent on the load history, i.e. reloading on a foundation will usually result in a stiffer load-displacement response than virgin loading (Whitaker and Cooke, 1966). Whitaker and Cooke (1966) measured load-displacements on a number of piles in London Clay and Figure 2.7 shows the load-displacement response of a bored pile with an enlarged base (12 m length, 0.8 m diameter shaft and 1.7 m diameter base). The first part of the test (until reloading) was a maintained load test with incremental steps and the second part (after reloading) a constant rate of penetration test. Although the difference in test methods may slightly influence the measured stiffness, the data still show a significant increase in stiffness on reloading. In the range of 1260 kN to 3620 kN the stiffness of the virgin loading was 40 kN/mm and on the reload 385 kN/mm showing an increase in stiffness of approximately 10 times. Therefore the increase in stiffness on reloading needs to be taken into account when piles are being reused and combined with new piles. Reuse of piles will become more common practice due to the drive for more sustainable development in large cities where frequent redevelopment leads to the soil being filled with old foundations, restricting the installation of new piles (Chapman *et al.*, 2001, Chow *et al.*, 2002).

The resistance of a foundation to vertical movements under loads is caused by end bearing resistance on the horizontal contact surfaces and friction on the vertical surfaces. Pad and raft foundations depend mainly on end bearing resistance, while the load capacity of pile foundations are from either or a combination of end bearing and shaft friction, depending on the type of pile. End bearing resistance in pads or rafts will increase with settlement until the foundation fails with regard to stability (i.e. tilting). However, damage to or unacceptable response from the superstructure due to excessive differential settlement is likely to be the limiting factor and not the ultimate capacity of the foundation (Chan, 1997). Two pad foundations tested on soft clay at Bothekennar (Jardine *et al.*, 1995, Lehane and Jardine, 2003) failed due to tilting at 160 mm and 220 mm vertical displacement respectively; however this settlement will be unacceptable for most structures.

Whitaker and Cooke (1966) instrumented bored piles with load cells and the results showed that shaft friction and end bearing capacity are mobilised at different rates of settlement. Frictional resistance develops rapidly with settlement and is generally fully mobilised when the settlement is about 0.5 % of the pile diameter. On the other hand, base resistance is seldom fully mobilised until the pile settlement reaches 10% to 20% of the base diameter. The shape of a bored pile load-displacement graph depends on the relative contribution of shaft and the base (Burland and Cooke, 1974). Figure 2.8(a) shows a typical load settlement curve for long straight shafted piles and Figure 2.8(b) shows the behaviour of relatively short piles with large under reams. Whitaker and Cooke (1974) measured the load distribution of the shaft and the base of an under reamed pile. Figure 2.7 shows the results and it is evident that the skin friction for this pile is fully mobilised at approximately 10 mm (1% of pile diameter) of settlement, whereas the base load was fully mobilised at approximately 100 mm (6% of base diameter) of settlement.

From the discussion above it is evident that a foundation load-displacement is not linear elastic. Burland and Wroth (1975) point out that for reasonably small stress changes overconsolidated clay behaves as an elastic material in contrast to normally consolidated clays which deform plastic.

Foundation settlement is time dependent and the response depends on the type of supporting soil. On sands settlement will usually occur immediately, however collapsible sand may show significant settlement at a later stage due to a rising water table (Wiseman and Lavie, 1983, Alawaji, 1997). In saturated fine grained soils with a low permeability, consolidation will take place and the short and long term settlement will differ significantly.

For a uniform circular load on overconsolidated clay, Burland and Wroth (1975) calculated the ratio of immediate to total settlement as:

$$\frac{\rho_u}{\rho_t} = \frac{1}{2(1-v')}$$ **Equation 2.5**

where ρ_u is immediate settlement, ρ_t is total settlement and v' is effective Poisson's ratio. For overconsolidated clays the likely range for v' is 0.1 to 0.33. Therefore ρ_u / ρ_t lies in the range of 0.55 to 0.75. This simple elastic analysis suggests (assuming no consolidation occurs during the construction period) that the ratio of immediate to total settlement of a foundation on overconsolidated clay will be in the range of 0.55 to 0.75.

Burland and Wroth (1975) similarly calculated that for overconsolidated clay, with a linear increase of Young's modulus with depth, the ratio of ρ_u / ρ_t is reduced and suggests a range 0.35 to 0.55.

Morton and Au (1975) analysed 8 case records of buildings on London Clay and suggested a ratio of ρ_u / ρ_t in the range of 0.40 to 0.82 with an average of 0.63.

Simons and Som (1970) analysed 12 case records of buildings on overconsolidated clays and 9 case records on normally consolidated clays. For overconsolidated clay they suggest a ratio of ρ_u / ρ_t in the range of 0.315 to 0.735 with an average of 0.575 and for normally consolidated clays a range of 0.077 to 0.212 with an average of 0.156. Table 2.11 provides a summary of immediate to long term settlements.

2.2.4 Structure stiffness

This section discusses the structural stiffness and the factors affecting it. Structural stiffness is determined by the stiffness of the materials used and the geometry. Firstly the stiffness of reinforced concrete and masonry will be discussed and secondly the effect of the geometry on the structural stiffness.

The stiffness of construction materials are strain and time dependent, however under normal operating conditions a typical stiffness may be determined. Reinforced concrete, masonry, dry wall partitions and glass facades are often used for construction. Dry wall partitions and glass facades are usually fastened in such a way that the contribution to the overall structure stiffness is insignificant and this will therefore not be discussed. Reinforced concrete consists of reinforcement steel and concrete. The Young's modulus of reinforcement steel ranges from 190 to 210 GPa (Gere and Timoshenko, 1991). British Standards Institution (BSI 8110-1:1997) suggests the use of a steel stiffness of 200 GPa in the elastic zone. The stiffness of concrete is time dependent and influenced (Neville, 1981) by:

- The strength of the concrete (which increases with age).
- Applied stress (which influences the strain)
- Moisture condition.
- Type of aggregate, and
- Mix ratios.

Figure 2.9 (Neville, 1981) shows the relationship between stress/strength ratio and strain for concrete of different strengths. It is evident that stiffness decreases with an increase of stress and that a high strength concrete has a higher stiffness than a low strength concrete at an equivalent strain. Concrete strength increases with time as shown in Figure 2.10 (Wood, 1991) which shows the development of strength in 150 mm concrete cubes over 20 years. Figure 2.11 (Neville, 1981) shows the influence of the moisture condition on the modulus of elasticity at a stress of 5.5 MPa of concrete at different ages. The concrete stiffness increased by 4 GPa to 5 GPa in a wet sample. Figure 2.12 (Neville, 1981) shows the variation of stiffness between cement paste, aggregate and concrete. The stiffness of the aggregate and cement paste is linear with concrete being nonlinear. The stiffness of the aggregate is approximately 4 times the stiffness of cement paste and an increase in aggregate in the mix will therefore increase the stiffness of the concrete. A stiff aggregate can approximately double the stiffness of the concrete in comparison to a low stiffness aggregate.

Shrinkage, creep and cracking affect the stress within the concrete member and therefore also the stiffness of the concrete member. Table 2.12 (Neville, 1981) shows the effect of aggregate/cement and water/cement ratio on shrinkage of concrete. An increase in the aggregate/cement ratio leads to a decrease in shrinkage. Using an aggregate/cement ratio of 3 instead of 7 will increase the shrinkage by 4 times. An increase in water/cement ratio will lead to an increase in shrinkage. Using a water cement ratio of 0.7 instead of 0.4 will approximately double the shrinkage. Figure 2.13 (Neville, 1981) shows the effect of stiffness on shrinkage. An increase in stiffness will lead to a decrease in shrinkage. Concrete with a Young’s modulus of 35 GPa will experience approximately half the shrinkage of a 15 GPa concrete. Figure 2.14 (Neville, 1981) shows the effect of different types of aggregate on shrinkage over time. Using sandstone instead of quartz aggregate can double the amount of shrinkage. Figure 2.15 (Neville, 1981) shows the effect of relative humidity on shrinkage of concrete. A decrease in relative humidity leads to an increase of shrinkage over time. Shrinkage for concrete typically ranges from 0 to 1.2×10^{-3}.

Creep in concrete can be defined as the increase in strain under sustained stress, or as a decrease in stress within the member under constant strain. In most structures creep and shrinkage occur simultaneously. Creep depends on aggregate content, type of aggregate, type of cement, applied stress, concrete strength, humidity, size of specimen, temperature and time (Neville, 1995). Figure 2.16 (Neville, 1981) shows the effect of creep on stress over time at a constant strain. The creep resulted in a 50% decrease in stress within 80 days for this specific sample. Figure 2.17 (Neville, 1981) shows the effect of aggregate type on creep. Using sandstone aggregate instead of limestone aggregate can double the amount of creep. Figure 2.18 (Neville, 1981) shows the effect of admixtures on the creep in concrete over time. Certain admixtures can increase creep by up to 30%. Figure 2.19 (Neville, 1981) shows the effect of relative humidity on creep over time. A humidity of 50% can increase creep with 150% in comparison to

100% humidity. Figure 2.20 (Neville, 1981) shows the range of creep-time curves for different concretes stored at various relative humidities. Creep in concrete typically ranges from 0 to 2000 x 10^{-3}.

From the discussion above it is evident that the stiffness of concrete can range significantly depending on the circumstances. The British Standards Institution (BSI 8110-2: 1985) suggest in the absence of better information the use of the following equations to determine the Young's modulus for serviceability limit state calculations:

$$E_{c,28} = 20 + 0.2 f_{cu,28} \qquad \textbf{Equation 2.6}$$

Where $E_{c.28}$ is the concrete modulus at 28 days in GPa and $f_{cu,28}$ is concrete strength at 28 days in MPa. A class 25/30 concrete will therefore have Young's Modulus of 26 GPa at 28 days. British Standards Institution (BSI 8110-2: 1985) suggests using the following equation for the Young's modulus at an age t:

$$E_{c,t} = E_{c,28}\left(0.4 + 0.6\, f_{cu,t} / f_{cu,28}\right) \qquad \textbf{Equation 2.7}$$

Masonry infill in reinforced concrete frames may have a significant impact on the stiffness of the structure. The behaviour of masonry infilled frames has been investigated by a number of researchers. Holmes (1961), Stafford Smith (1962, 1966, 1967), and Mainstone and Weeks (1970) have conducted experimental and analytical investigations on the lateral stiffness and strength of steel frames infilled with mortar and concrete panels. The behaviour of masonry infilled reinforced concrete frames is generally more complicated than of steel infilled frames and has been examined by Kahn and Hanson (1979), Bertero and Brokken (1983), Mehrabi *et al.* (1996) and Mehrabi and Shing (1997). Mehrabi *et al.* (1996) tested twelve 1/2-scale single-storey, single bay frames of which eleven were infilled with masonry. The experimental results showed that the masonry infill can significantly influence the stiffness of reinforced concrete frames. The Young's modulus of the eleven masonry infill panels ranged from 3.1 to 9.6 GPa with an average of 6.7 GPa. The Young's modulus of the masonry is therefore approximately ¼ of the stiffness of a class 25/30 concrete at 28 days.

Differential settlement depends on both the stiffness of the structure as well as the stiffness of the supporting soil. Theoretically the stiffness of the structure with respect to the soil can range from perfectly flexible to perfectly rigid. A real structure's stiffness will be within the two extremes. To gain a better understanding of the behaviour of a structure the two theoretical extremes are discussed.

Poulos and Davis (1974) provided standard elastic solutions for surface displacement and stress due to a circular uniform load on a semi-infinite mass as shown in Figure 2.21. Comparing settlement ratios and stress patterns for perfectly flexible and perfectly rigid loads illustrates the behaviour of a structure at opposite extremes. Circular loads are a simplified way to illustrate the effect of structural stiffness without dealing with the added complexity of rectangular foundations which are present in most structures.

The contact stress distribution under a flexible circular uniform loading is the uniform loading whereas the contact stress distribution under a rigid circular loading (Schiffmann and Aggarwala, 1961, cited in Poulos and Davis, 1974) can be calculated from:

$$\sigma_z = \frac{p_{av}}{2\sqrt{1-\frac{r^2}{a^2}}}$$ **Equation 2.8**

The vertical surface displacement ρ_z under a flexible circular uniform loading (Ahlvin and Ulery, 1962, cited in Poulos and Davis, 1974) can be calculated from:

$$\rho_z = p\frac{(1-v^2)}{E}aH$$ **Equation 2.9**

where H is a function of *r/a*. Ahlvin and Ulery provided tabulated values for *H*.

The vertical surface displacement under a rigid circular uniform loading (Schiffmann and Aggarwala, 1961, cited in Poulos and Davis, 1974) can be calculated from:

$$\rho_z = \frac{\pi}{2}p_{av}\frac{(1-v^2)}{E}a$$ **Equation 2.10**

Using Equations 2.8, 2.9 and 2.10, ratios of contact pressure and surface settlement were calculated and are shown graphically in Figure 2.21. The contact stress beneath the centre of a rigid uniform loading is 50% less than for a flexible uniform loading with the same load and area. The contact stress at the edge of the rigid uniform loading on an elastic halfspace is infinite. Even though a circular load on an elastic halfspace is a simplification of real foundation behaviour the same trends will be seen until the soil fails due to excessive stress. A rigid structure on a raft foundation will therefore have higher contact pressure at the edges up to the point of soil failure.

Surface settlement due a flexible uniform loading is 19 % less at the edge and 27 % more at the centre in comparison to that of a rigid uniform loading. The stiffness of a structure and soil will influence the amount and distribution of differential settlement.

From Equations 2.9 and 2.10 it is evident that for a constant:

- area, settlement will increase linearly with an increase of applied stress ($\rho_z \propto p$).
- applied stress, settlement will increase with an increase in area ($\rho_z \propto A^2$); and
- applied total load, settlement will increase with a decrease in area ($\rho_z \propto A^{-0.5}$).

It may be argued that differential settlement may be eliminated by sizing the foundations according to the loads carried, resulting in uniform settlement. However to achieve this, accurate predictions of the loads within the structure and load-settlement responses of the foundations are needed. Predicting load and load-settlement response accurately is difficult due to the variation in load and soil conditions.

The stiffness of structures is between perfectly flexible and perfectly rigid. Brown (1969a, 1969b) investigated the effect of raft stiffness on the behaviour of the raft. The study was based on the numerical analyses of uniform loaded circular rafts of any flexibility which rest on an isotropic elastic foundation layer. He found that the behaviour of the raft depends on the soil raft stiffness ratio, and defined the relative raft stiffness *K* as:

$$K = \frac{E_r\left(1-\upsilon_r^2\right)t^3}{E_s a^3}$$ **Equation 2.11**

Where E_r denotes Young's modulus of the raft, E_s denotes Young's modulus of the foundation material, υ_r denotes Poisson's ratio of the foundation material, *t* is the raft thickness and *a* is the raft radius.

Fraser and Wardle (1976) defined relative raft stiffness K as:

$$K = \frac{4E_r\left(1-\upsilon_r^2\right).t^3}{3E_s\left(1-\upsilon_s^2\right).b^3}$$ **Equation 2.12**

Where υ_r denotes Poisson's ratio of the soil and b is the raft width.

Hain and Lee (1978) defined relative raft stiffness K as:

$$K = \frac{4E_r\left(1-\upsilon_r^2\right)bt^3}{3\pi E_s a^4}$$ **Equation 2.13**

Although Equations 2.11, 2.12 and 2.13 vary, it should be noted that results against relative raft stiffness K are usually plotted on a log scale; therefore the Poisson's ratio will not have a significant impact.

In the Equation 2.13 by Hain and Lee the effect of the geometry of the raft is described by term bt^3/a^4. For a complete building this term needs to describe the effective building geometry. Potts and Addenbrooke (1997) defined relative bending stiffness ρ* of a building as:

$$\rho^* = \frac{EI}{E_s H^4}$$ **Equation 2.14**

Where H is the half width of the building in the plane of deformation and EI the bending stiffness of the structure. Potts and Addenbrook state that two approaches can be taken to calculate the EI of a structure. The first approach employs the parallel axis theorem to define the structural stiffness about the neutral axis as shown in Equation 2.15:

$$(EI)_{Stiffstruct} = E\sum_1^n (I_{slab} + A_{slab}h^2)$$ **Equation 2.15**

Where n is the number of storeys. This can be considered to be an overestimate of the building stiffness as only a rigidly framed structure would approach such mode of deformation. An alternative method was used to obtain the bending stiffness by summing the independent EI values for each slab as shown in Equation 2.16. This implies that the walls and columns transfer the same deformed shape to each storey.

$$(EI)_{Flexstruct} = E\sum_1^n I_{slab}$$ **Equation 2.16**

From equation 2.15 and 2.16 it is evident that stiff infill panels will have a significant effect on the bending stiffness. Currently the bending stiffness calculations are based on these simple assumptions. Researching the real bending stiffness of a variety of buildings will therefore be valuable to predict accurate differential settlement behaviour of structures.

It is important to note that most studies of the interaction between a structure and the supporting soil assumed that the building frame is complete before loading commences. In practice much of the loading is applied progressively during construction (Brown and Yu, 1986). Heil (1969) and Goschy (1978) both analysed progressive loading, but did not attempt to quantify the differences between the effect of progressive loading and the loading of the completed frame. Brown and Yu (1986) analysed a 3-bay by 3-bay four storey steel framed office building with precast floor and roof slabs on a raft foundation resting on a deep homogeneous linear elastic soil mass. They also analysed a plane frame representing the midspan of the structure. Both the plane and space frame analysis showed that the effective stiffness for interaction purposes of a building that is loaded progressively during construction is about half the stiffness of the completed building.

2.3 Difference between modern and old buildings

The design of modern office buildings is often significantly different from the design of older buildings, resulting in a more flexible structure. This section discusses the case studies of the Jubilee Line Extension with reference to the type of structure, construction date and building stiffness. Lastly the tendency and the effect of reuse in modern buildings are discussed.

Table 2.13 shows a summary of the descriptions of the case study buildings for the Jubilee Line Extension (Burland *et al*., 2001b). The case studies provide detail information on the type of structures. From the list it is evident that load bearing brickwork was mostly used in the earlier buildings (1750 to 1950) and in the lower (5-storeys or less) buildings of 1960 to 1990. The steel framed buildings date from 1900 to 1930 and one from 1978. The reinforced concrete frame buildings date from 1915 to 1990. Even though the list is limited, it shows a tendency to move away from load bearing brick infill (especially in higher buildings) and use reinforced concrete frames instead.

The case studies listed in Table 2.13 provides detailed settlement movements of the buildings during and after the construction of the Jubilee Line Extension. The stiffness of the building influences the settlement of the structure (Burland *et al*., 2001a), however it is difficult to back analyse the building stiffness based on the settlement due to the complex factors which also influence the settlement. These factors include:

- The size of the tunnel.
- The location of the tunnel relative to the structure.
- Tunnel construction method.
- Compensation grouting.
- Time dependency.

Settlement predictions were made for Elizabeth, Neptune, Murdoch and Clegg Houses. Elizabeth House is a 10-storey reinforced concrete frame building constructed in the 1960s with two levels of basement and founded on a 1.4 m thick concrete raft. Neptune, Murdoch and Clegg Houses are of similar construction, being 3-storey brick load bearing structures.

The relative bending stiffnesses of the structures were calculated based on the relative bending stiffness ρ^* (Equation 2.14) as defined by Potts and Addenbrooke (1997).

For Elizabeth House (reinforced concrete frame) the bending stiffness of the structure (EI) was calculated from:

- $E_{concrete}$ was assumed to be 23 x 10^6 kPa
- $IE_{flexstruct}$ was calculated based on Equation 2.16, i.e. the sum of the individual slab stiffness and any AH^2 terms (Equation 2.15) were ignored.

The calculated EI for the 10-storey reinforced concrete building is approximately 6 x 10^6 kN-m.

For Neptune, Murdoch and Clegg Houses the bending stiffness of the structure (EI) was calculated from:

- E_{brick} was assumed to be 7.5 x 10^6 kPa
- $IE_{stiffstruct}$ was calculated based on H^3 x E_{brick} where H is the height of the building.

The calculated EI for the 3-storey load bearing brickwork building is approximately 203 x 10^6 kN-m.

From the values above it is evident that the bending stiffness of the 3-storey load bearing brickwork building is approximately 34 times stiffer than the 10-storey reinforced concrete building, even if the stiffness of the brickwork is approximately a third of the concrete stiffness. Modern reinforced concrete buildings are therefore significantly more flexible than older load bearing brickwork buildings.

Historically most structures fell in the category of load bearing brickwork structures and research was focused on this type of structure (Skempton and MacDonald, 1956, Polshin and Tokar, 1957, Grant *et al.*, 1974, Burland and Wroth, 1975, Burland *et al.*, 2001a). Modern reinforced concrete frame structures with no infill or danger of damage to the cladding are less susceptible to damage due to differential settlement. Applying the stricter criteria applicable to structures with load bearing brickwork to modern reinforced concrete framed structures may be overly conservative.

The drive for sustainable development has an impact on the design of modern structures. Reuse of foundations reduces the consumption of resources (energy and materials) and leads to a more sustainable development (Chapman *et al.*, 2001, Chow *et al.*, 2002, Cameron and Chapman, 2004). In areas of frequent development the soil becomes polluted with old discarded foundations, leaving no space for new foundations. Existing services like tunnels, pipelines and cables may also affect the availability of space for new foundations. To create space for new foundations, old foundations need to be removed. Deep foundations, for example piles, are expensive to remove and cause disturbance to soil. Therefore the reuse of foundations may be a more suitable alternative.

To reuse old foundations the foundation must be suitable for supporting the new structure. Old foundations are usually not used due to the following reasons (Chapman *et al.*, 2001, Chow *et al.*, 2002, Cameron and Chapman, 2004):

- Unsuitable load capacity of the foundation.
- Unsuitable location of the foundation.
- Uncertainty of the dimensions of the foundation. Good records and in-situ testing may eliminate it.
- Uncertainty of the integrity of the foundations. Previous loading from load takedown calculations and non destructive testing can give valuable insight into the integrity of foundations.

- Stiffer load-displacement response due to preloading.
- The belief that new foundations are better and pose less risk than old foundations.

Knowledge of the behaviour of modern flexible framed buildings on reused foundations in combination with new foundations will lower the risk involved in the reuse of foundations.

2.4 Damage to buildings

Damage to buildings is very subjective and depends both on the function of the building and the reaction of the users, and is difficult to quantify (Burland and Wroth, 1975). Burland *et al.* (1977) have suggested a classification system to quantify damage. This will be discussed in more detail in the following section. Peck *et al.* (1956) have also shown that a certain amount of cracking is unavoidable if the building is to be economical. Little (1969) has estimated for a particular type of building the cost to prevent cracking may exceed 10% of the total building cost. It is therefore important to educate clients about possible damage management within buildings and the impact of it on construction and running costs. It is also important to note that damage due to movements in buildings can also be attributed to a number of factors such as creep, shrinkage and temperature as well as movement from structural members under working loads, rather than movement of foundations (Burland and Wroth, 1975).

2.4.1 Classification of damage

This section firstly discusses the different types of damage. Secondly a damage classification system based on the severity of damage is presented, and lastly the applicability to modern buildings is discussed.

Building damage can range from minor cracks to total collapse. Skempton and Macdonald (1956) have divided building damage into 3 categories; visual appearance, function and structural.

- Visual damage affects the appearance of structures and is usually related to cracks or separations in panel walls, floors and finishes. Burland *et al.*, (1977) suggested that cracks in plaster walls greater than 0.5 mm wide and cracks in masonry or rough concrete walls greater than 1 mm are representative of a threshold of where damage is noticed.

- Functional damage affects the use of the structure and is related to jammed doors and windows, tilting and deflection of walls and floors, extensive cracking and falling plaster. This type of damage would require non-structural repair to ensure full functionality of the structure.
- Structural damage affects the stability of the structure and is related to cracks or distortions in support elements like beams, columns and load bearing walls.

Burland *et al.* (2001a) state that as foundation movements increase, damage to a building will progress successively from visual damage to functional damage to stability issues.

To quantify building damage objectively, guidelines with descriptions are needed. Burland *et al.* (1977) suggested a classification system based on the work of Jennings and Kerrich (1962), the UK National Coal Board (1975) and MacLeod and Littlejohn (1975). Since then it has been adopted with only slight modifications by BRE (1981 and 1990), the Institution of Structural Engineers, London (1978, 1989, 1994 and 2000) and by the Institution of Civil Engineers and BRE again in Freeman *et al.* (1994). Table 2.14 shows the suggested classification system (after Burland *et al.*, 2001a). Burland *et al.* (2001a) made the following important comments regarding the classification system:

- The classification is based on the ease of repair of the visible damage.
- The classification only relates to visible damage at a given time and not cause or possible progression.
- The temptation to classify the damage solely on crack width must be resisted. The ease of repair is the key factor in the classification.
- The classification was developed for brickwork or blockwork and stone masonry and could be adapted for other forms of cladding. It is not intended to apply to reinforced concrete structural elements.

The above classification system has been used successfully and extensively on the Jubilee Line Extension (Burland *et al.*, 2001a). However it is important to note the limitations of this approach as highlighted by Burland *et al.* (2001a). This is aimed at structures with load bearing walls. These structures will usually first show visual damage, then functional damage and lastly structural damage as differential settlement increase. Therefore maximum allowable differential settlement will be limited by visual damage. However the design of facades and internal partitions in modern framed structures often allows for movement by either leaving gaps or the connection detail. This results in the capability to accommodate more differential settlement without damage in comparison to older load bearing structures. This leads to the following questions about modern open plan structures:

- How much differential settlement can the facades / partitions on modern structures withstand without signs of damage?
- Will visible cracking or functional damage be the limiting factor?
- How much more differential settlement can a modern structure withstand in comparison to an old structure with load bearing walls?
- Is it sensible to apply established approaches for old structures with load bearing walls to modern open plan structures? If not, what approach needs to be followed?

To answer these questions damage needs to be linked to differential settlement. The following section will discuss differential settlement guidelines and the applicability to old and modern structures.

2.4.2 Guidelines to prevent differential settlement damage

This section considers the relationship between differential settlement and damage. Firstly, angular distortion limits to prevent damage based on a simple frame analysis are suggested. Secondly, empirical limits based on a number of case studies are presented and compared to each other as well as to the limits suggested by the frame analysis. Thirdly a fundamental method linking damage to differential settlement is presented through the use of tensile strain as a serviceability parameter and a 2D beam analysis. These limits are compared to case studies. Lastly the limitations of the current methods are discussed.

Meyerhof (1947) analysed a five storey three bay reinforced concrete frame and found that an angular distortion of 1/950 caused an increase in bending moment of 74% in the beam with the largest bending moment prior to differential settlement. A beam already at working stress before differential settlement with a 74 % increase in bending moment would be expected to show damage in the form of cracking.

Skempton and MacDonald (1956) however followed another approach and summarised settlement and damage observations on 98 buildings, 40 of which showed signs of damage. The buildings studied were mostly steel and reinforced concrete structures with load bearing walls. No useable data was available for framed structures with curtain walling and dry-construction partitions. Angular distortion was used to define the differential settlement on buildings and was expressed as the ratio of differential settlement δ and the distance l between two points after eliminating the effect of tilt. It was concluded from the data that an angular distortion greater than 1/150 will cause structural damage and an angular distortion greater than 1/300 will cause cracking in walls and partitions. They recommended that angular distortions greater than 1/500 should be avoided if it was important to prevent differential settlement damage.

Skempton and MacDonald (1956) argued that an angular distortion of 1/950 as used by Meyerhof (1956) in the example did not result in damage in the observed structures. They gave the following possible explanations:

- The live loads assumed in the design are conservative compared to the actual average loads in buildings, and
- The composite action of the frame, floors and walls may reduce the stresses and deflections within the building.

In response to the study of Skempton and Macdonald, Meyerhof (1956) used laboratory results of panel and load bearing brick walls failure and combined them with different factors of safety which are based on the type of damage. He suggested angular distortion limits of 1/300 for open frames, 1/1000 for panel walls of brick masonry and 1/2000 for load bearing walls.

Burland *et al.* (1977) commented on Skempton and MacDonald's work. They recognised it as a milestone; however, they also warned of the danger of following the guidelines blindly, without taking the limited range of structures studied or the criteria that were used into account. The following points were noted:

- The studies were limited to traditional steel and reinforced concrete frame buildings and a few load bearing brick wall buildings. The direct evidence is based on seven frame buildings of which two were damaged and seven load bearing brick wall buildings of which only one was damaged. The remaining data was based on indirect evidence, being where settlement damage was reported without specifying detail or where, so far as is known, no settlement damage had occurred. Skempton and Macdonald emphasised this, however it is seldom emphasised in textbooks or design recommendations.

- The criterion used for limiting deformation is angular distortion and this implies that damage results from shear distortion, which is not necessarily the case.
- No classification other than ‘architectural’, ‘functional’ and ‘structural’ damage was used to classify the damage.
- Although it is the cladding and finishes that are usually damaged, the angular distortion values quoted are for the whole structure and not necessarily applied to the cladding or finishes. For load bearing walls the angular distortion values are the relevant values. However for frame buildings the cladding and finishes may be applied after some settlement has occurred. Therefore the angular distortion values of the facades may be significantly less than those for the whole structure.
- The limiting values for angular distortion that leads to structural damage are for frame buildings with structural members of average dimensions. They do not apply to exceptionally large and stiff beams or columns where the limiting values of angular distortion may be much less.

Grant *et al.* (1974) performed a similar study to that of Skempton and Macdonald (1956) analysing 95 additional structures. The structures analysed were newer than those analysed by Skempton and Macdonald (1956) with 68 of the 95 structures built in the 1950’s and 1960’s. The structures analysed consisted of open frames and frames with loadbearing walls. Their conclusions were similar to those of Skempton and MacDonald (1956) and they suggest an angular distortion limit of 1/300 to prevent damage.

Polshin and Tokar (1957) discussed the allowable deformation and presented the limiting values proposed by the 1955 Building Code of the USSR. They recommend angular distortion limits that vary from 1/500 for steel and concrete frame infilled structures to 1/200 where there is no infill or danger of damage to the cladding. These values are in line with those proposed by Skempton and Macdonald (1956). However they laid down much stricter criteria for load bearing walls. For ratios of Length/Height $L/H < 3$, the maximum deflection ratios Δ/L are 0.0003 and 0.0004 for sand and soft clay respectively. For $L/H > 5$ the corresponding values are 0.0005 and 0.0007. Polshin and Tokar introduced two new concepts, taking the L/H ratio into account and the use of a limiting tensile strain of 0.05% for a theoretical approach.

Burland and Wroth (1975), Burland *et al.* (1977) and Burland *et al.* (2001a) followed a more fundamental approach to link damage to differential settlement. Following the work of Polshin and Tokar (1957) they assumed that the onset of visible cracking in a given material may be linked to a limiting tensile strain. Burland and Wroth (1975) took as their starting point the statement that damage due to differential movement is almost always confined to the cladding and finishes rather than the structural members and apart from a few notable exceptions buildings will usually become unserviceable before there is danger of structural collapse.

Drawing on the work of Mainstone (1970), Mainstone and Weeks (1971) and Burhouse (1969), Burland and Wroth (1975) set out to determine critical tensile strains before cracking occurs. Mainstone and Weeks reported on large scale tests of frames with brick infill by the UK Building Research Establishment. It was observed that visible cracking occurs at average diagonal tensile strains between 0.081% and 0.137% with a mean of 0.081%. However, these figures are based on the measurement of displacement of the frame. Detailed observations of the brick infill of one of the frames indicated lower local average strains in the range of 0.05% to 0.10%. Burhouse reported on a study of the composite action between brick walls and supporting beams. In these tests the tensile strain at the onset of visible cracking varied from 0.038% to 0.06%. Polshin and Tokar (1957) suggested that visible cracking occurs in brick walls at 0.05%, which seems to correlate with above results.

Boscardin and Cording (1989) analysed 17 case records of damage due to excavation induced settlement. These case studies showed that the damage given Table 2.13 can be broadly linked to ranges of limiting tensile strain. These ranges are tabulated in Table 2.15.

Using a limiting tensile strain as serviceability parameter Burland and Wroth (1975), Burland *et al.* (1977) and Burland *et al.* (2001a) used the following approach to determine the relation between differential settlement and tensile strain. They suggest representing a building with a rectangular beam of length L and height H. Figure 2.22 illustrates the approach. The aim is to calculate the tensile strains in the beam for a given deflected shape and obtain the sagging or hogging ratio Δ/L at which cracking is initiated. The distribution of strains depends on the mode of deformation. Two extreme modes are bending only at a neutral axis at the centre (Figure 2.22c) and shearing only (Figure 2.22d). In bending only, the maximum tensile strain occurs in the extreme bottom fibre, where cracking will start. For shear only the maximum tensile strains are inclined at 45 degrees, initiating diagonal cracking. Both modes of deformation will usually occur simultaneously. Therefore both will need to be calculated to determine which one is limiting.

Timoshenko (1957) gives the equation for total midspan deflection Δ of a centrally loaded beam having both bending and shear stiffness as:

$$\Delta = \frac{PL^3}{48EI}\left(1 + \frac{18EI}{L^2HG}\right) \qquad \textbf{Equation 2.17}$$

Equation 2.17 can be rewritten in terms of the maximum extreme fibre strain ε_{bmax} as follows:

$$\frac{\Delta}{L} = \left(\frac{L}{12t} + \frac{3I}{2yLH} \times \frac{E}{G}\right)\varepsilon_{b\max} \qquad \textbf{Equation 2.18}$$

Similarly Equation 2.17 can be rewritten for maximum diagonal strain ε_{dmax} as follows:

$$\frac{\Delta}{L} = \left(1 + \frac{hL^2}{18I} \times \frac{G}{E}\right)\varepsilon_{d\max} \qquad \textbf{Equation 2.19}$$

It can be shown that for a given deflection Δ the maximum tensile strains are not very sensitive to the precise form of loading (Burland and Wroth, 1975).

By setting $\varepsilon_{max} = \varepsilon_{lim}$ Equation 2.18 and 2.19 define the limiting values of Δ/L for cracking of simple beams in bending and shear. It is evident that for a given value of ε_{lim} the limiting value Δ/L (whichever is the lowest from Equations 2.18 and 2.19) depends on L/H, E/G and the position of the neutral axis. Figure 2.23 shows the relationship between Δ/L normalised by ε_{lim} and L/H for an isotropic (E/G =2.6) rectangular beam with the neutral axis at the bottom edge. For values of L/H < 1.5 the diagonal strains dominate, whereas for L/H >1.5 bending strains dominate.

Burland *et al.* (1977) compared the beam approach to damage from case studies. Figure 2.24 show the results for frame buildings and load bearing walls. A limiting tensile strain of 0.075% and a neutral axis at the centre were used for both cases. E/G ratios of 12.5 and 2.5 were used respectively for the framed buildings and load bearing walls. Also shown are an angular distortion of 1/300 and the relationship proposed by Polshin and Tokar (1957). The beam approach is in good correlation to the case studies, in spite of its simplicity. The data also shows that frame buildings can tolerate more settlement than load bearing walls.

Boscardin and Cording (1989) included horizontal strain in the above analysis using superposition. This will not be discussed because horizontal strains are usually associated with tunnelling and exaction which is outside the scope of this report.

The above analyses assume that the behaviour of a 3D structure undergoing differential settlement can be represented by a simplified 2D beam undergoing bending and shear deformation. For damage prediction, the deformation and maximum tensile strain within the beam are calculated and compared to known critical tensile strains for damage in infill panels. For this method to be valid the following criteria need to be satisfied:

- Insignificant differential settlement must occur perpendicular to the plane of bending. This may happen where differential settlement occurs due to tunnelling or open excavations parallel to the structure, however differential settlement driven by the self weight of the building results in 3 dimensional deformation of the structure.
- The complete building (including facades and partitions) is constructed before any differential settlement occurs. This may be a justifiable assumption if the differential settlement is caused by adjacent excavation after the completion of the building. However, for differential settlement driven by the self weight of the building the change of stiffness of the structure and the settlement that occurs as construction progresses, needs to be taken into account.
- The facades and partitions are fixed to the frame and no allowance is made for differential settlement. Gaps or brackets allowing for movement will reduce the strain in facades and partitions.

The current state of the art as presented above gives valuable insight into damage due to differential settlement and the works presented here can be summarised as follows:

- The frame analysis by Meyerhof (1956) resulted in a conservative angular distortion value of 1/950 for an open frame when compared to case studies. This is most likely due to overestimated live load and the stiffness effect of walls and partitions in the case studies.
- Structures can be divided into 3 categories with the following suggested angular distortion limits:

  - Structural damage on frames without cladding: 1/150 to 1/300.
  - Facade cracking on frames with non load bearing facades: 1/300 to 1/1000, and
  - Damage to load bearing walls: 1/300 to 1/2000.
- A method to predict deformation limits was presented. Tensile strain was used as a serviceability parameter and a 2D beam analysis to link deflection to damage. This approach showed a good correlation with data from case studies.
- The data presented are for specific structures and the limits must not be applied blindly to all structures.
- The guidelines are based on the overall distortion of the structure; however the distortion of the facades may be less than the whole structure. For load bearing walls the overall distortion and the facade distortion are the same.
- The L/H ratio of a building affects the deformation limit. The higher the L/H ratio, the more differential settlement can be accommodated.

Although sometimes forgotten, a number of authors clearly state the type of buildings the data of their analyses are related to. The question arises whether these approaches are applicable to modern framed structures with facades designed to allow for movements? These approaches also simplify the buildings to 2D structures without any reference to the 3D behaviour of the buildings. Burland *et al.* (1977) state regarding this method: “Clearly there is scope for more realistic analysis of actual structures using numerical methods of analysis. It is hoped that the success of the present over-simplified approach will stimulate further work along these lines.” Approximately 20 years later the same simplified beam approach has been used on the Jubilee Line Extension (Burland *et al.*, 2001a). Although a major advantage of the approach is the simplicity, a deeper insight into the behaviour of 3D structures as well as modern structures is needed.

2.5 Conclusions

The following is a summary of the conclusions of the literature review:

- Current deformation definitions describe only in plane deformations and no attempt is made to define 3D behaviour.
- Spatial variation of soil properties are often inaccurately predicted due to a lack of understanding of the inherent soil variability, uncertainties in measurement and transformation models of soil properties.

- Ultimate limit state design loads are significantly higher (up to 8 times) than expected loads during normal use.
- Although foundation load-displacement response is non-linear, the load-displacement behaviour of foundations loaded within the design loads on overconsolidated clays is close to linear elastic.
- Reused foundations have been preloaded and will have a stiffer load-displacement response than a similar new foundation. The increase in stiffness of the load-displacement response depends on the loading and soil type.
- The ratio of immediate to total settlement of a foundation on overconsolidated clay will be in the range of 0.35 to 0.75.
- The stiffness of a building that is loaded progressively during construction is about half the stiffness of the completed building.
- The need exists for accurate building stiffness predictions and the normalisation of soil-structure stiffness.
- On structures with load bearing walls, the first damage to occur due to differential settlement will in most cases be visual damage. Structural damage will only occur at larger differential settlements.
- The current state of the art guidelines to prevent differential settlement damage are based on the assumption that the behaviour of the 3D structure can be represented by a simplified 2D structure.
- The current state of the art guidelines to prevent differential settlement damage are focussed on older, rigid, masonry infill buildings. These acknowledge that the behaviour of flexible structures may be different and the guidelines are only applicable to rigid structures, however minimal guidance is given on the behaviour of 3D flexible structures.

From the literature it is evident that the current state of the art may not be applicable to modern flexible structures. The need exists to investigate the full 3D behaviour of modern flexible buildings with or without the reuse of foundations.

Table 2.1: **Summary of inherent variability of strength parameters (Phoon and Kulhawy, 1999)**

Property[a]	Soil type	No. of data groups	No. of tests per group		Property value		Property COV (%)	
			Range	Mean	Range	Mean	Range	Mean
S_u (UC) (kN/m^2)	Fine grained	38	2-538	101	6-412	100	6-56	33
S_u (UU) (kN/m^2)	Clay, silt	13	14-82	33	15-363	276	11-49	22
S_u (CIUC) (kN/m^2)	Clay	10	12-86	47	130-713	405	18-42	32
S_u (kN/m2)[b]	Clay	42	24-124	48	8-638	112	6-80	32
Ø (°)	Sand	7	29-136	62	35-41	37.6	5-11	9
Ø (°)	Clay, silt	12	5-51	16	9-33	15.3	10-50	21
Ø (°)	Clay, silt	9	-	-	17-41	33.3	4-12	9
tan Ø (TC)	Clay, silt	4	-	-	0.24-0.69	0.509	6-46	20
tan Ø (DS)	Clay, silt	3	-	-	-	0.615	6-46	23
tan Ø [b]	Sand	13	6-111	45	0.65-0.92	0.744	5-14	9

a S_u, undrained shear strength; Ø, effective stress friction angle; TC, triaxial compression test; UC, unconfined compression test; UU, unconsolidated-undrained triaxial compression test; CIUC, consolidated isotropic undrained triaxial compression test; DS, direct shear test.

b Laboratory test type not reported

Table 2.2: Summary of inherent variability of index parameters (Phoon and Kulhawy, 1999)

Property[a]	Soil type[b]	No. of data groups	No. of tests per group		Property value		Property COV (%)	
			Range	Mean	Range	Mean	Range	Mean
W_n (%)	Fine grained	40	17-439	252	13-105	29	7-46	18
W_L (%)	Fine grained	38	15-299	129	27-89	51	7-39	18
W_p (%)	Fine grained	23	32-299	201	14-27	22	6-34	16
PI (%)	Fine grained	33	15-299	120	12-44	25	9-57	29
LI	Clay, silt	2	32-118	75	-	0.094	60-88	74
γ (kN/m³)	Fine grained	6	5-3200	564	14-20	17.5	3-20	9
γ_d(kN/m³)	Fine grained	8	4-315	122	13-18	15.7	2-13	7
D_r (%)[c]	Sand	5	-	-	30-70	50	11-36	19
D_r (%)[d]	Sand	5	-	-	30-70	50	49-74	61

[a] W_n natural water content, W_L liquid limit, W_p plastic limit, PI plasticity index, LI liquidity index**,** γ total unit weight, γ_d dry unit weight, D_r relative density**.**

[b] Fine-grained materials derived from a variety of geologic origins, e.g. glacial deposits, tropical soils and loess.

[c] Total variability of direct method of determination

[d] Total variability for indirect determination using standard penetration test (SPT) values.

Table 2.3: Scale of fluctuation of some geotechnical parameters (Phoon and Kulhawy, 1999)

Property[a]	Soil type	No. of studies	Scale of Fluctuation (m)	
			Range	Mean
Vertical fluctuation				
s_u	Clay	5	0.8-6.1	2.5
q_c	Sand, clay	7	0.1-2.2	0.9
q_T	Clay	10	0.2-0.5	0.3
s_u (VST)	Clay	6	2.0-6.2	3.8
N	Sand	1	-	2.4
w_n	Clay, loam	3	1.6-12.7	5.7
w_L	Clay, loam	2	1.6-8.7	5.2
γ	Clay, loam	2	2.4-7.9	5.2
Horizontal fluctuation				
q_c	Sand, clay	11	3.0-80.0	47.9
q_T	Clay	2	23.0-66.0	44.5
s_u (VST)	Clay	3	46.0-60	50.7
w_n	Clay	1	-	170

[a] s_u and s_u (VST), undrained shear strength from laboratory test and vane shear tests respectively.

Table 2.4: Summary of live loads

Survey	Location	Survey date	Number of buildings	Total area surveyed (m^2)	Room Floor area *A* (m^2)	Live load Mean (kN/m^2)	Live load Standard Deviation (kN/m^2)
Kumar (2002)	Kandur, India	1992-1993	8	11 720	All	0.458	0.278
					$A \le 8$	0.68	0.41
					$8< A \le 16$	0.60	0.32
					$16< A \le 24$	0.50	0.36
					$24< A \le 32$	0.50	0.29
					$32< A \le 40$	0.47	0.26
					$40< A \le 48$	0.45	0.24
					$48< A \le 56$	0.45	0.25
					$64< A \le 72$	0.46	0.15
					$72< A \le 80$	0.46	0.19
					$A >80$	0.31	0.20
Ruiz and Soriano (1997)	Mexico city, Mexico		5	14 890	All	0.734	$\sqrt{0.0394+\frac{0.4473}{A}}$
Choi (1992)	Sydney, Australia	1972-1979	11	144 136	All	0.62	0.60
					$A \le 5$	0.50	0.66
					$5< A \le 10$	0.62	0.64
					$10< A \le 20$	0.55	0.47
					$20< A \le 40$	0.45	0.53
					$40< A \le 80$	0.43	0.45
					$A >80$	0.51	0.41
Culver (1976)	USA		23		$A \le 4.7$	0.83	0.82
					$4.7< A \le 9.3$	0.63	0.60
					$9.3< A \le 27.9$	0.44	0.31
					$A >27.9$	0.42	0.43
Mitchell and Woodgate (1971)	UK				2.4	0.66	0.65
					5.2	0.64	0.53
					14.0	0.62	0.43
					31.2	0.61	0.34
					58.0	0.59	0.30
					111.3	0.58	0.26
					192.4	0.56	0.21

Table 2.5: Summary of design and measured construction loads (Beeby, 2001)

Construction stage	Floor	Design floor loads: kN/m²	Measured floor load: kN/m²
Formwork to first floor struck	1	7.00	6.00
Second floor concreted	1	8.25	?
Formwork to second floor struck	2	7.00	6.00
Third floor concreted	1	10.50	8.20
	2	11.75	10.32
Formwork to third floor struck	3	7.50	6.00
Fourth floor concreted	1	9.33	6.90
	2	9.33	7.35
	3	10.58	10.57
Formwork to fourth floor struck	4	7.00	6.00
Fifth floor concreted	3	10.50	8.50
	4	11.75	9.90
Formwork to fifth floor struck	5	7.00	6.00
Sixth floor concreted	4	10.50	8.20
	5	11.75	10.32
Formwork to sixth floor struck	6	7.50	6.50
Roof concreted	5	9.70	7.80
	6	12.69	9.50
Backprops between fifth and sixth	6	15.25	10.10
floor struck	7	0.00	1.20
Formwork to roof struck	7	7.00	6.00

Table 2.6: Load combinations and partial factors for Ultimate Limit State design according to British Standards Institution (BSI 8110-1: 1997)

Load combination	Load type				
	Dead		Live		Wind
	Adverse	Beneficial	Adverse	Beneficial	
Dead and live	1.4	1.0	1.6	0	-
Dead and wind	1.4	1.0	-	-	1.4
Dead, live and wind	1.2	1.2	1.2	1.2	1.2

Table 2.7: Load combinations and partial factors for Ultimate Limit State design according to Eurocode (EN 1990: 2002, NA to EN 1990: 2002)

	Permanent action		**Variable action**	
	Unfavourable	**Favourable**	**Unfavourable**	**Favourable**
A) *EQU	1.05 (1.1 UK)	0.95 (0.9 UK)	1.5	0
B) *STR/GEO	1.35	1.00	1.5	0
C) *STR/GEO	1.00	1.00	1.3	0

* Notes: Static equilibrium should be verified using the design values of actions 'A'

Design of structural members not involving geotechnical actions should be verified using the design values of 'B'

Design of structural members (footings, piles, etc.) involving geotechnical actions should be verified using one of the following:

Approach 1: Applying in separate calculations design values from 'B' and 'C' to the geotechnical actions as well as the actions on / from the structure;

Approach 2: Apply the design values of actions from 'B' to the geotechnical actions as well as actions on / from the structure;

Approach 3: Applying design values of actions from 'C' to the geotechnical actions and, simultaneously, applying design values from actions from 'B' to the actions on / from the structure.

UK Annex recommends Approach 1.

Table 2.8: Thermal surface temperatures (EN 1991-1-5: 2003, NA to EN 1991-1-5: 2003)

Season	Surface	Surface temperature in °C	
		North-East facing	**South-West or horizontal facing**
Summer	Bright light surface	$T_{max} + 0$	$T_{max} + 18$
	Light coloured surface	$T_{max} + 2$	$T_{max} + 30$
	Dark surface	$T_{max} + 4$	$T_{max} + 42$
Winter		T_{min}	T_{min}

Note: T_{max} is the maximum shade air temperature and T_{min} the minimum air temperatures

Table 2.9: Coefficients of linear expansion (EN 1991-1-5: 2003, EN 572-1: 2004)

Material	**α_T (x 10^{-6}/°C)**
Aluminium, aluminium alloy	24
Stainless steel	16
Concrete	12
Masonry	6-10
Glass	9

Table 2.10: Summary of measured and proposed loads

Source	Load type	Uniform distributed load (kN/m²)
Measured		
Kumar (2002)	Live load (all)	0.46
Kumar (2002)	Live load (maximum in one room)	2.05
Ruiz and Soriano (1997)	Live load (all)	0.73
Choi (1994)	Live load (all)	0.62
Beeby (2001)	Live load - Construction (including slab support)	10.57
Proposed		
Ayoub and Karshenas (1994)	Live load - Construction	0.30
British Standards	Live load (excluding partitions, unfactored)	2.50
British Standards	Live load (including partitions, unfactored)	3.50
Eurocode	Live load (excluding partitions, unfactored)	1.50 - 2.00
Eurocode	Live load (including partitions, unfactored)	2.00 - 3.20
UK Annex	Live load (excluding partitions, unfactored)	2.50
UK Annex	Live load (including partitions, unfactored)	3.00 - 4.70
Worked example concrete frame	Dead load (frame only)	7.73
Worked example steel frame	Dead load (frame only)	3.04

Table 2.11: Summary of immediate to long term settlement ratios

Reference	**Clay type**	**Based on**	**Ratio (min.-max.)**
Burland and Wroth (1975)	Overconsolidated	Calculations	0.55-0.75
Burland and Wroth (1975)	Overconsolidated, increased Young's modulus with depth	Calculations	0.35-0.55
Morton and Au (1975)	London Clay	Case studies	0.40-0.82
Simons and Som (1970)	Overconsolidated	Case studies	0.315-0.735
Simons and Som (1970)	Normally consolidated	Case studies	0.315-0.735

Table 2.12: Effect of aggregate/cement and water/cement ratio on shrinkage

Aggregate/cement ratio	Shrinkage after six months (10^{-6}) for water/cement ratio of:			
	0.4	**0.5**	**0.6**	**0.7**
3	800	1200	-	-
4	550	850	1050	-
5	400	600	750	850
6	300	400	550	650
7	200	300	400	500

Table 2.13: Description of buildings analysed for the Jubilee Line Extension (Burland *et al.*, 2001b)

Building	Storeys	Load bearing brickwork	Steel frame	Reinforced concrete frame	Construction date
RICS, Great George Street	6	x			1756 & 1896
128-130 Jamaica Road	3	x			unknown, probably early 1800s
London Bridge Post Office	5	x			early 1840s
St. Stephen's (Clock) Tower, Palace of Westminster	92 m	x			1858
Treasury	5	x			1898-1912
Ritz Hotel	8		x		1906
RAC Building	5		x		1911
ICE, Great George Street	8		x		1912
Telephone House	7	x	x	x	1915
PHED, Great George Street	9		x		1927
Murdoch, Clegg and Neptune Houses	3	x			1931
Blick House	5	x			1950
Fielden House	7			x	1952
Elizabeth House	7 & 10			x	1960
182-210 Jamaica Road	2			x	1960s
Niagara Court	4	x			1960s
Regina and Columbia Points	21			x	1961-1962
Tenants' Hall and Boiler House, Canada Estate	1	x			1961-1962
BT Building	6			x	mid 1970s
RICS extension, Great George Street	6		x		1978
Keeton's Estate	3	x			early 1980s
1-7 St, Thomas Street	4			x	late 1980s

Table 2.14: Classification of visible damage to walls with particular reference to ease of repair of plaster and brickwork or masonry (Burland *et al.*, 2001a)

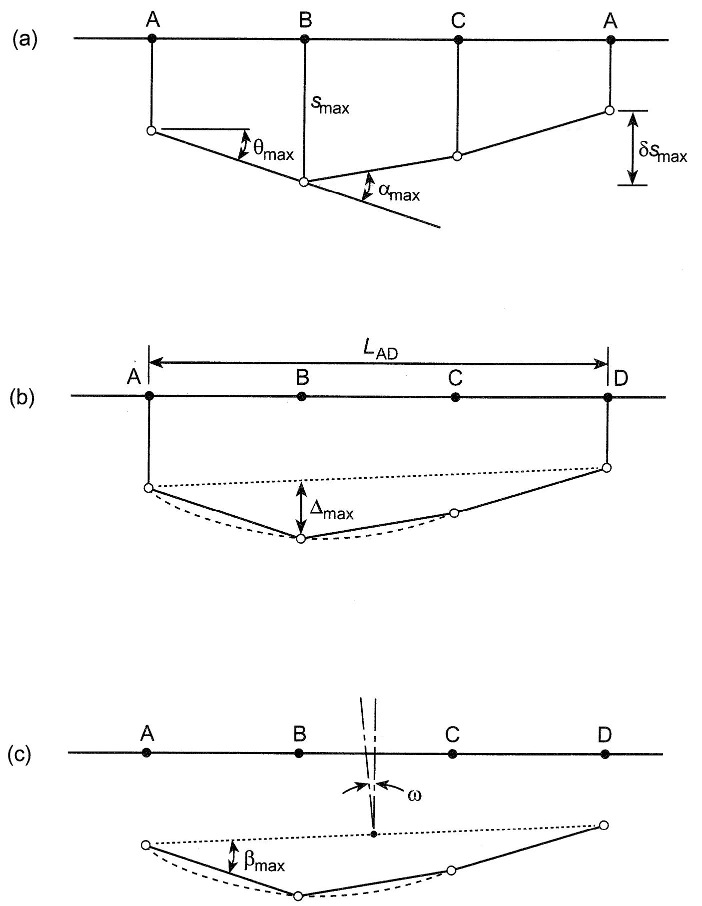
Category of damage	Normal degree of severity	**Description of typical damage** (ease of repair is in bold type) *Note: Crack width is only one factor in assessing category of damage and should not be used on its own as a direct measure of it.*
0	Negligible	Hairline cracks less than about 0.1 mm wide.
1	Very slight	**Fine cracks that are easily treated during normal decoration.** Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickwork or masonry. Typical crack widths up to 1 mm.
2	Slight	**Cracks easily filled. Redecoration probably required. Recurrent cracks can be masked by suitable linings.** Cracks may be visible externally and **some repointing may be required to ensure weather-tightness.** Doors and windows may stick slightly. Typical crack widths up to 5 mm.
3	Moderate	**The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced.** Doors and windows sticking. Service pipes may fracture. Weather tightness often impaired. Typical crack widths are 5-15 mm or several > 3 mm.
4	Severe	**Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows.** Windows and door frames distorted, floor sloping noticeably[1]. Walls leaning[1] or bulging noticeably, some loss of bearing in beams. Service pipes disrupted. Typical crack widths are 15-25 mm, but also depends on the number of cracks.
5	Very severe	**This requires a major repair job involving partial or complete rebuilding.** Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability. Typical crack widths are greater than 25 mm, but depends on the number of cracks.

[1]Note: Local deviation of slope, from the horizontal or vertical, of more than 1/100 will normally be clearly visible. Overall deviations in excess of 1/150 are undesirable.

Table 2.15: Relationship between category of damage and limiting tensile strain (Burland *et al.*, 2001a)

Category of damage	**Normal degree of severity**	**Limiting tensile strain (%)**
0	Negligible	0-0.05
1	Very slight	0.05-0.075
2	Slight	0.075-0.15
3	Moderate*	0.15-0.3
4 to 5	Severe to very severe	> 0.3

*Note: Boscarding and Cording (1989) describe the damage corresponding to limiting tensile strain in the range of 0.15 to 0.3 per cent as “moderate to severe”. However none of these cases quoted by them exhibits severe damage for this range of strains. There is therefore no evidence to suggest that tensile strains up to 0.3 per cent will result in severe damage.



Definitions of ground and foundation movement
(a) settlement s, relative settlement δs*, rotation* θ*, and angular strain* α*:*
(b) relative deflection Δ *and deflection ratio* Δ/L
(c) tilt ω *and relative rotation (angular distortion)* β

Figure 2.1: Definitions of ground and foundation movement (Burland *et al.*, 2001a)

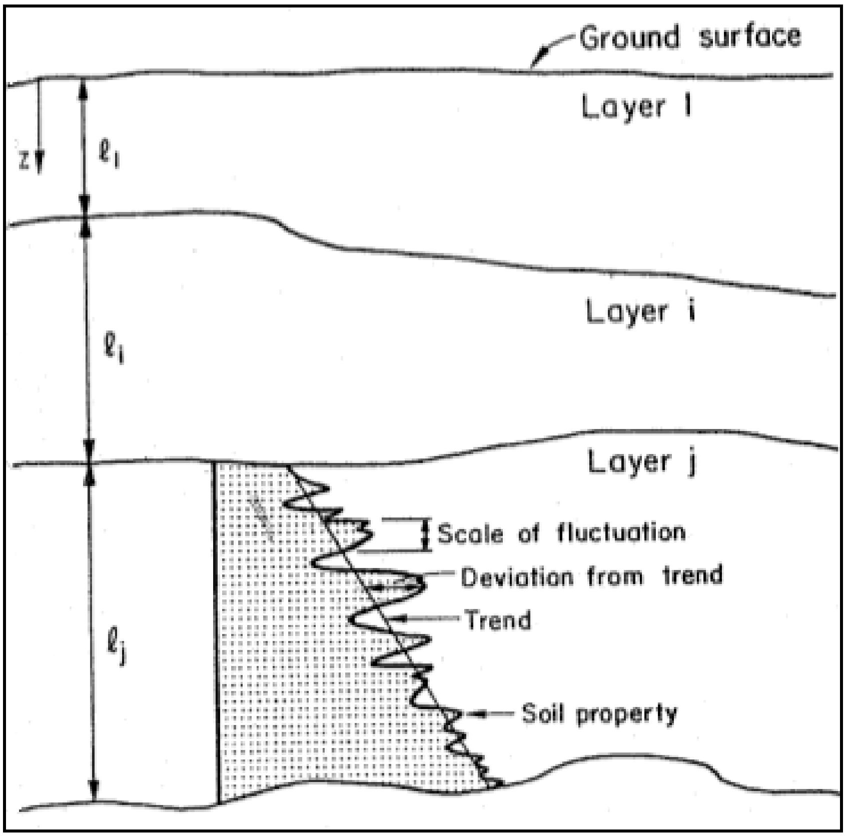


Figure 2.2: Inherent soil variability and scale of fluctuation (Phoon and Kulhawy, 1999)

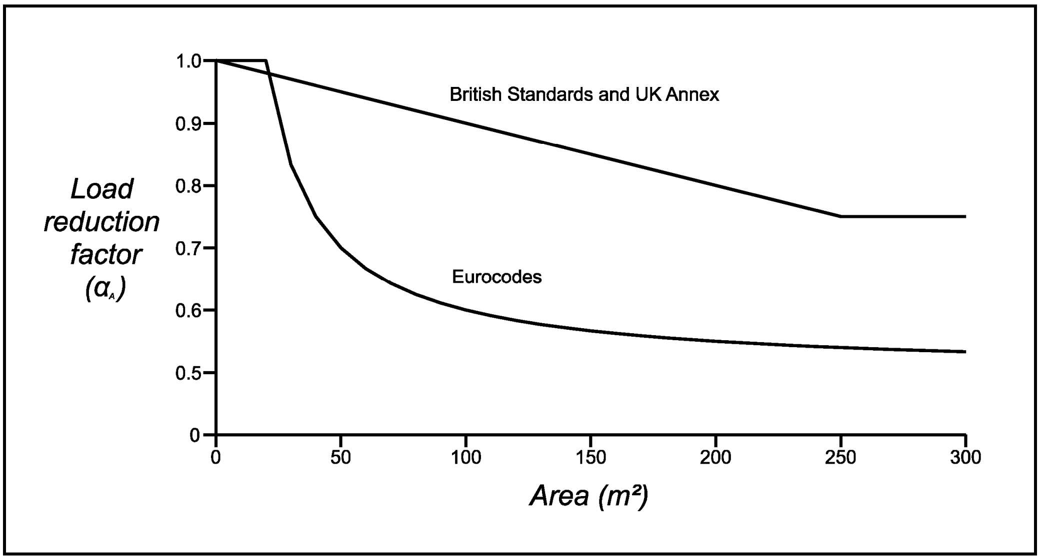


Figure 2.3: Load reduction factors according to area supported

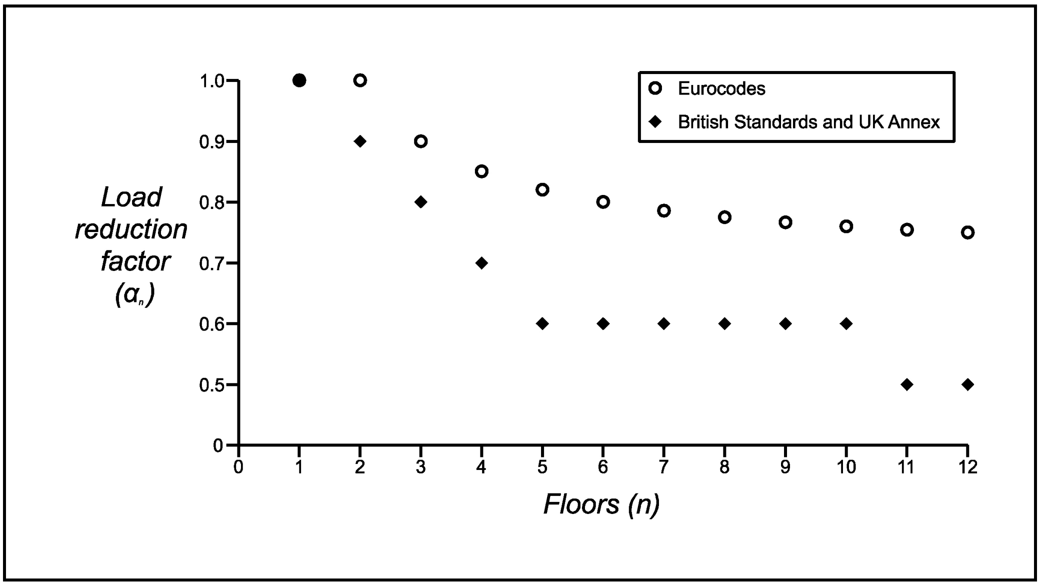


Figure 2.4: **Load reduction factors for number of floors supported**

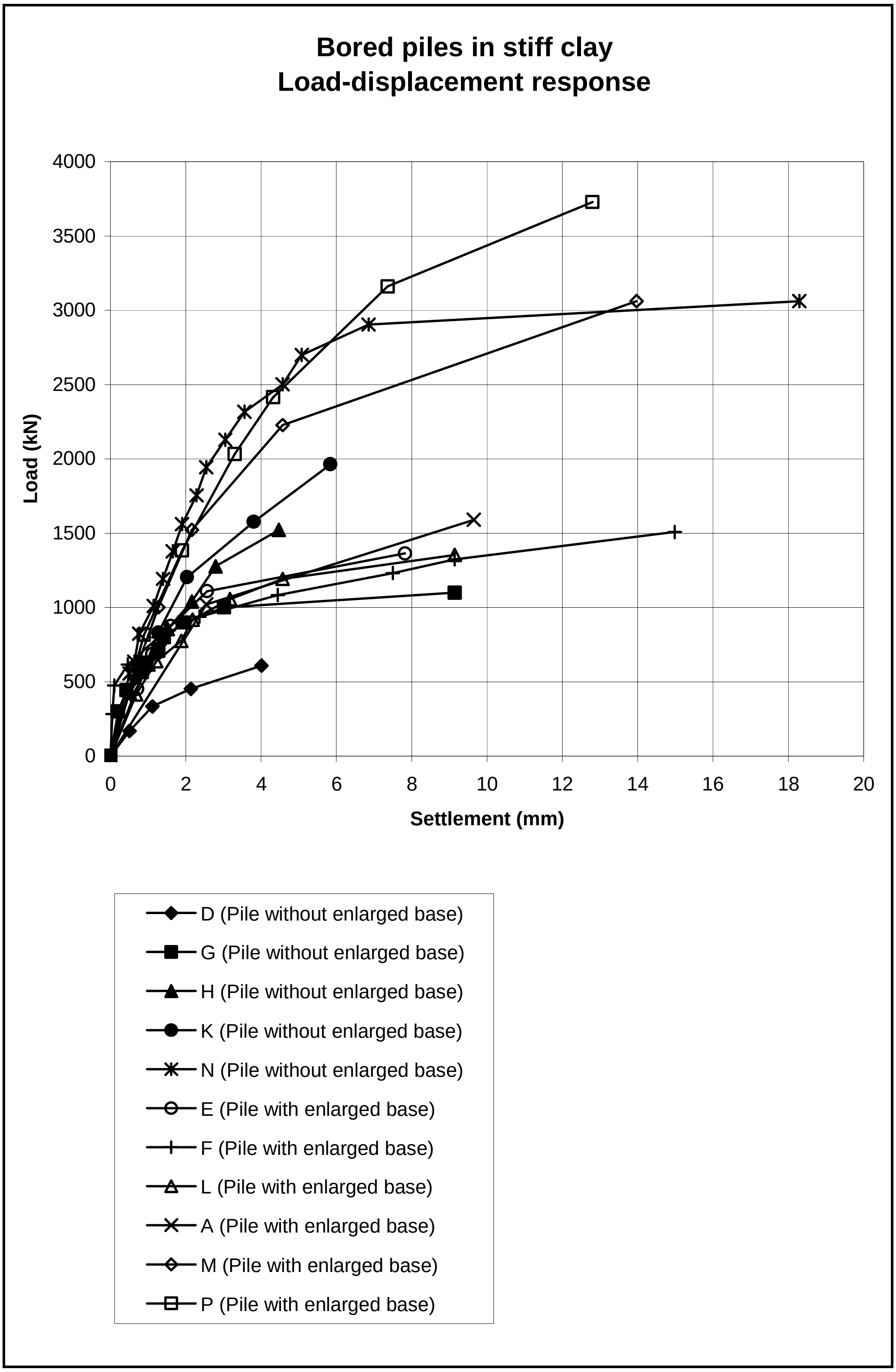


Figure 2.5: **Load-displacement of bored piles in stiff clay (Whitaker and Cooke, 1966)**

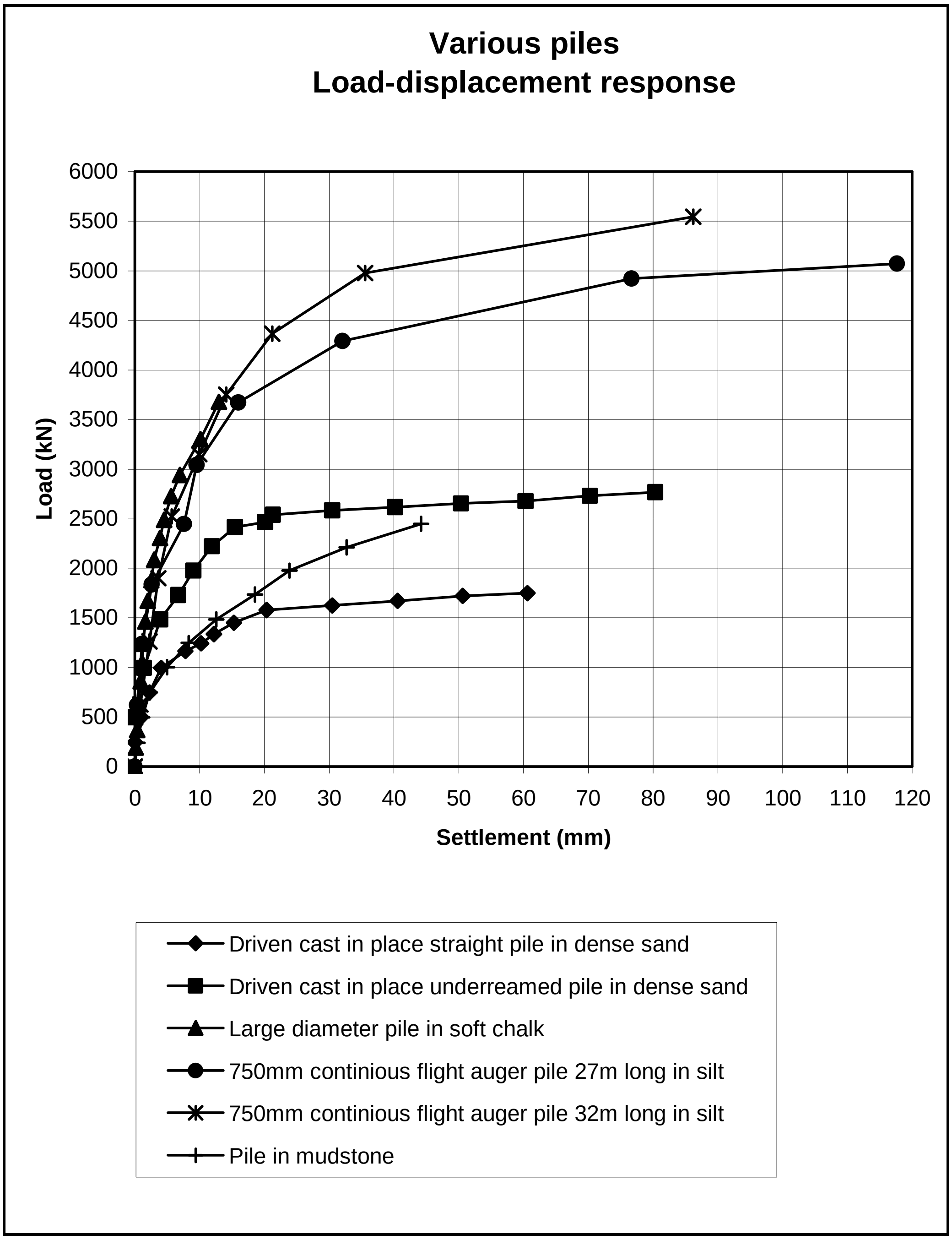


Figure 2.6: **Load-displacement response of various piles (De Beer *et al.*, 1979, Fleming, 1992)**

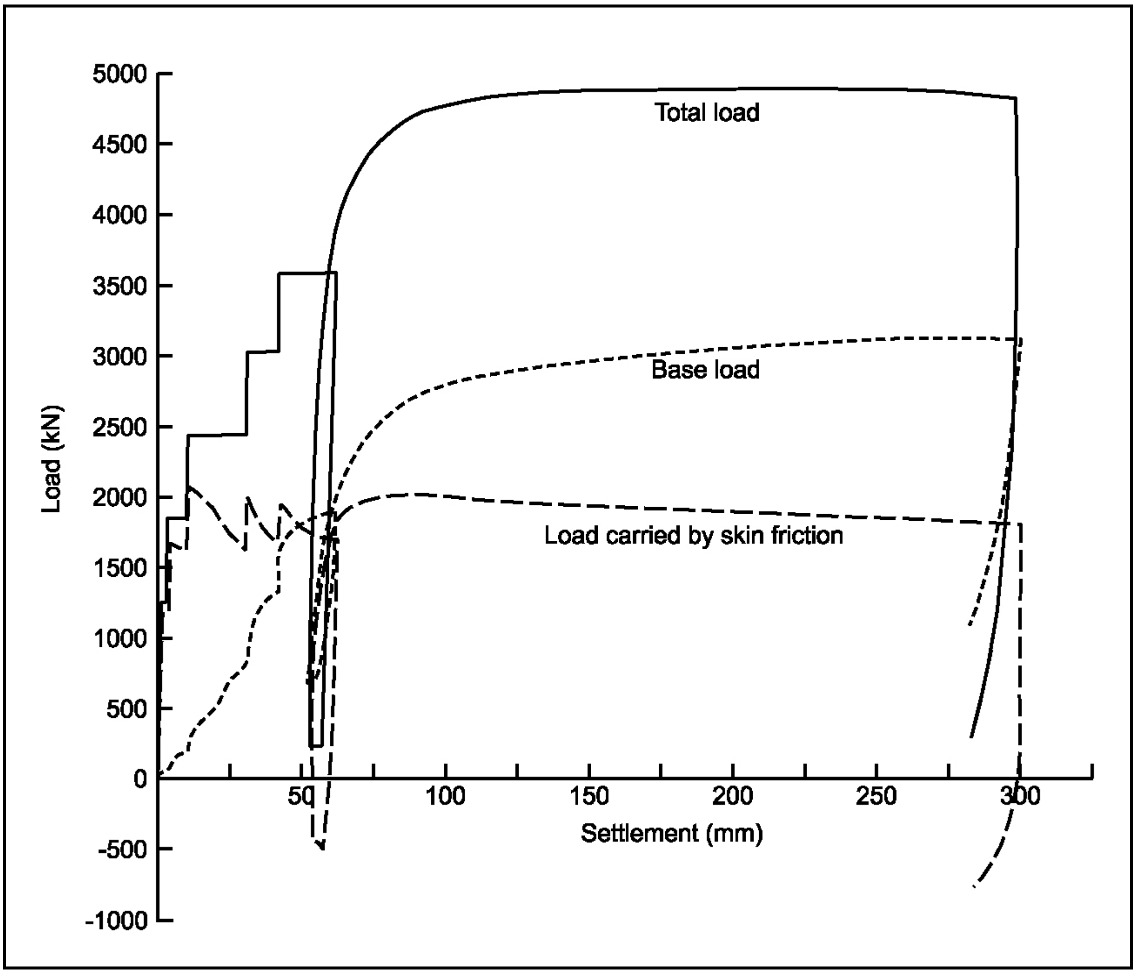


Figure 2.7: **Measured load distribution of the shaft and base of an under reamed pile (Whitaker and Cooke, 1966)**

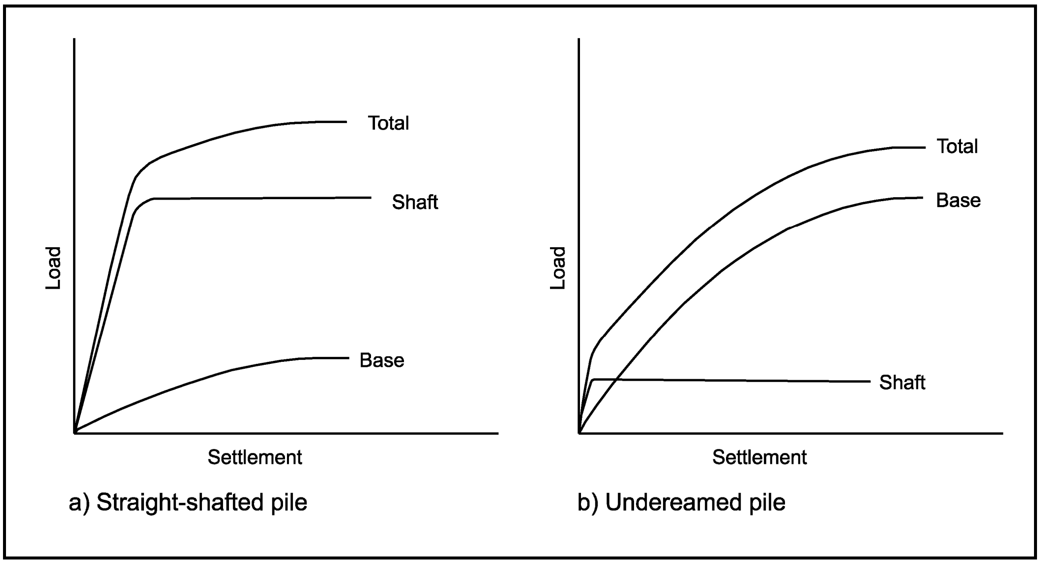


Figure 2.8: Typical load settlement curves (Burland and Cooke, 1974)

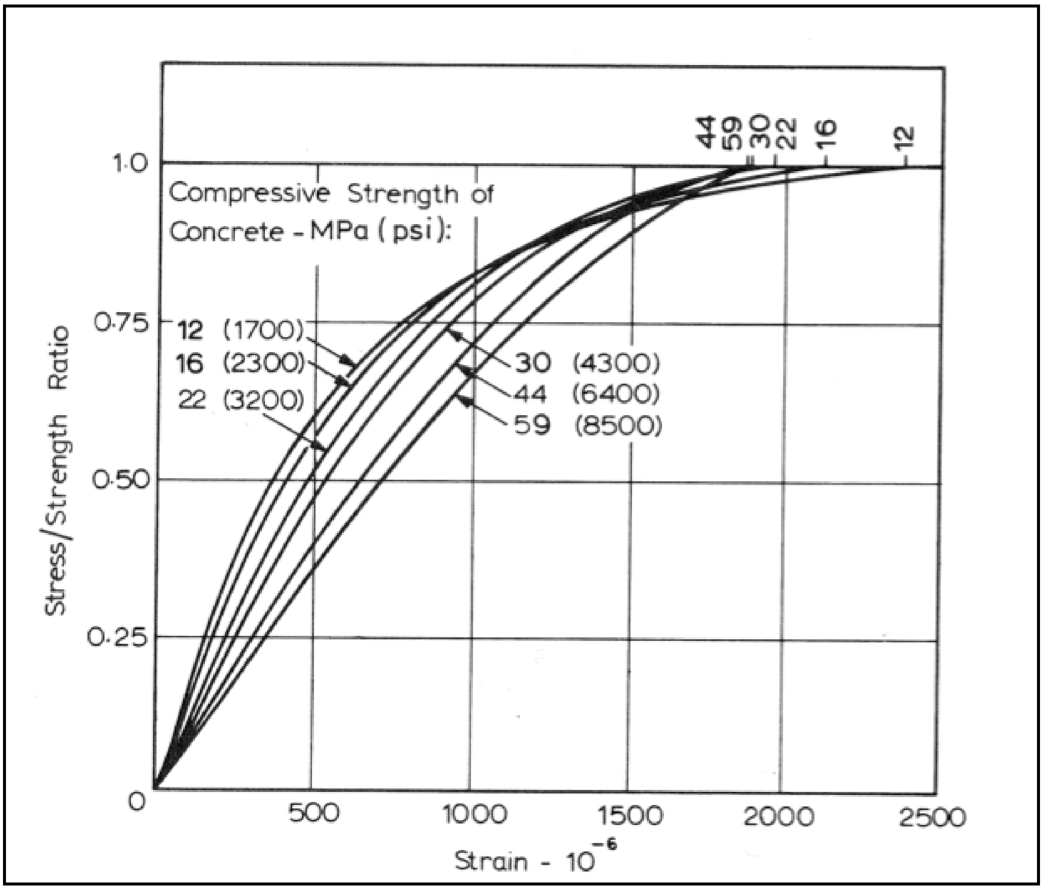


Figure 2.9: Relation between stress/strength ratio and strain for concretes of different strengths (Neville, 1981)

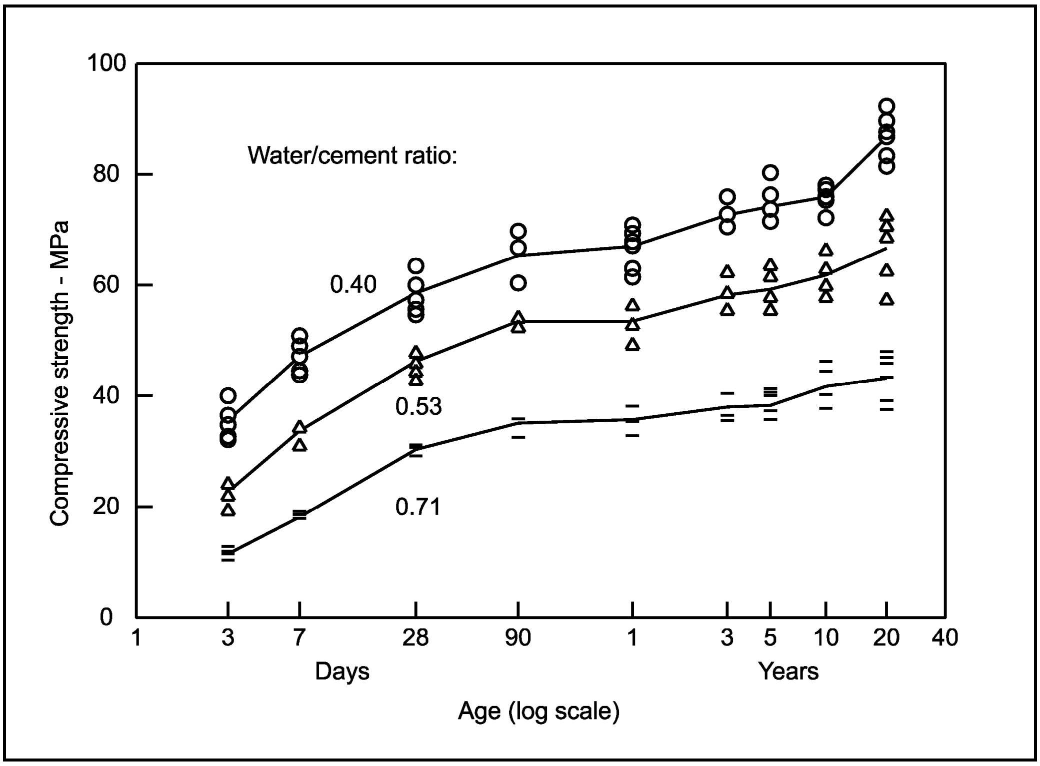


Figure 2.10: **Development of strength of concrete (determined by 150 mm cubes), stored under moist conditions (Wood, 1991)**

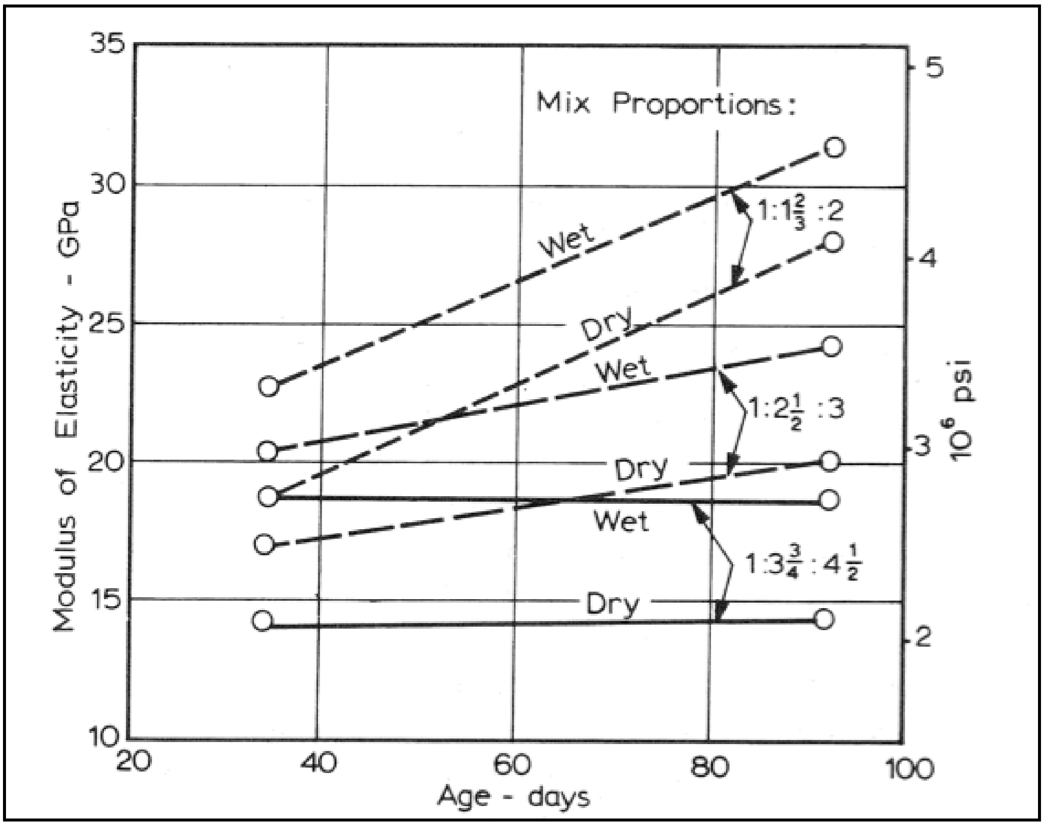


Figure 2.11: **Influence of moisture condition on the modulus of elasticity at 5.5 MPa (Neville, 1981)**

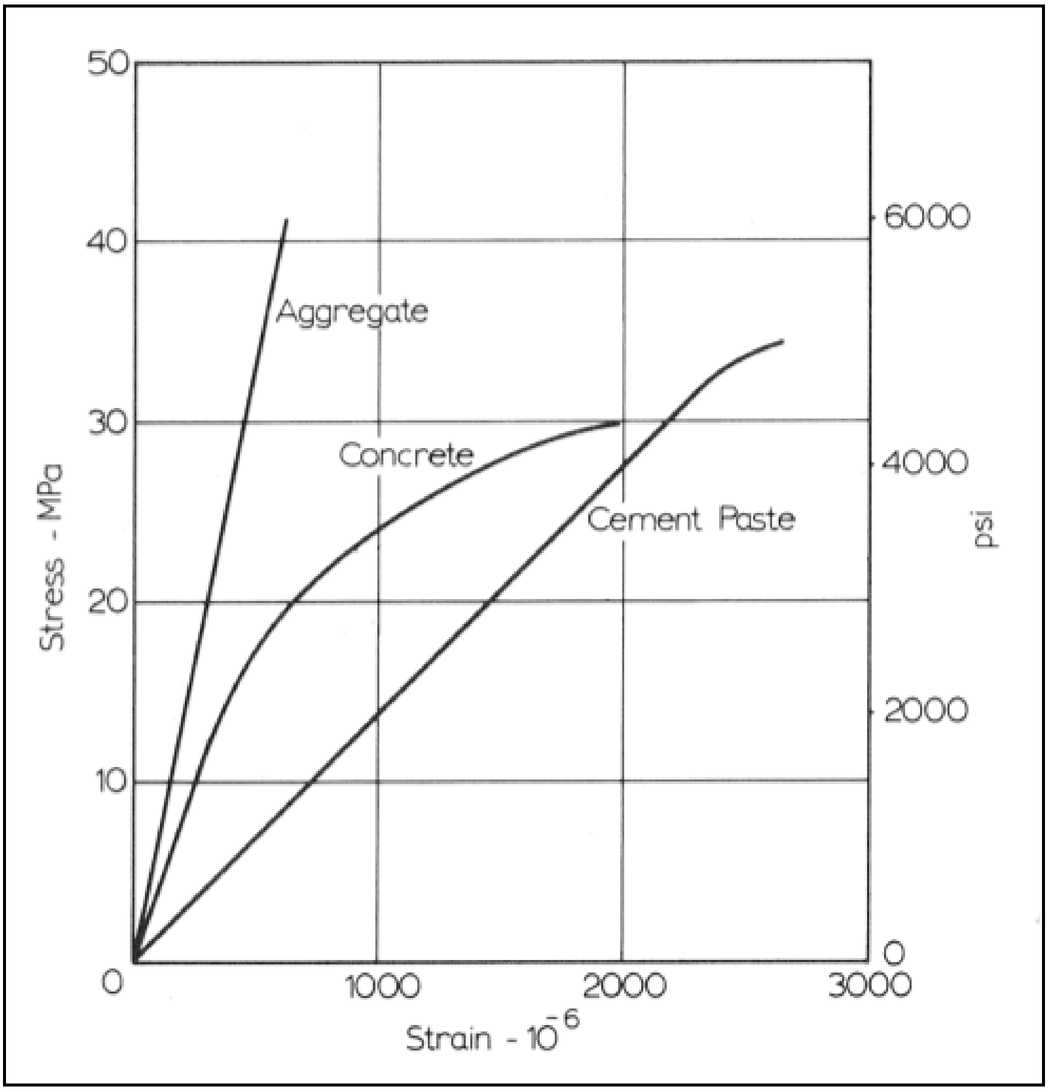


Figure 2.12: **Stress-strain relations for cement paste, aggregate and concrete (Neville, 1981)**

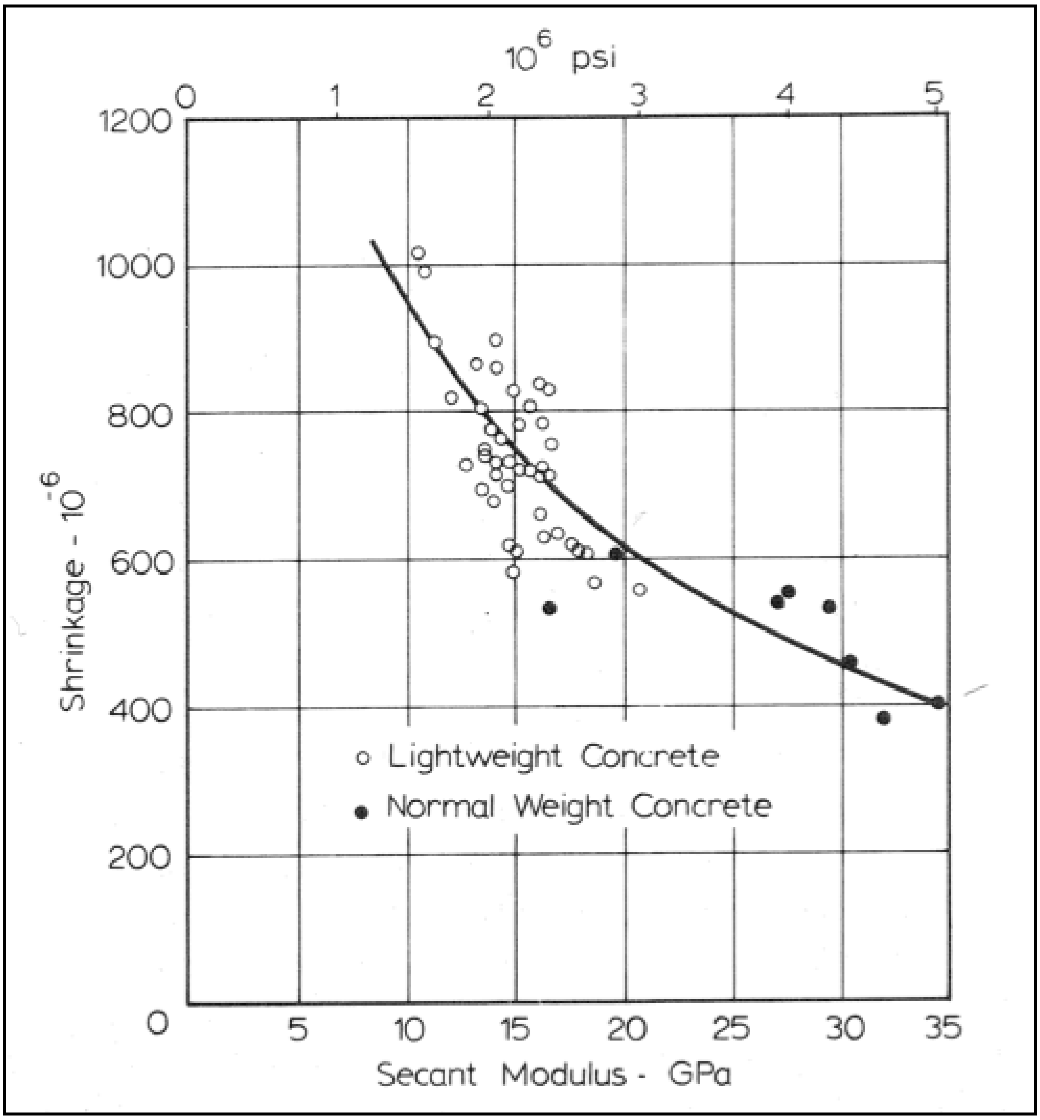


Figure 2.13: **Relation between drying shrinkage after 2 years and modulus of elasticity at a stress/strength ratio of 0.4 at 28 days (Neville, 1981)**

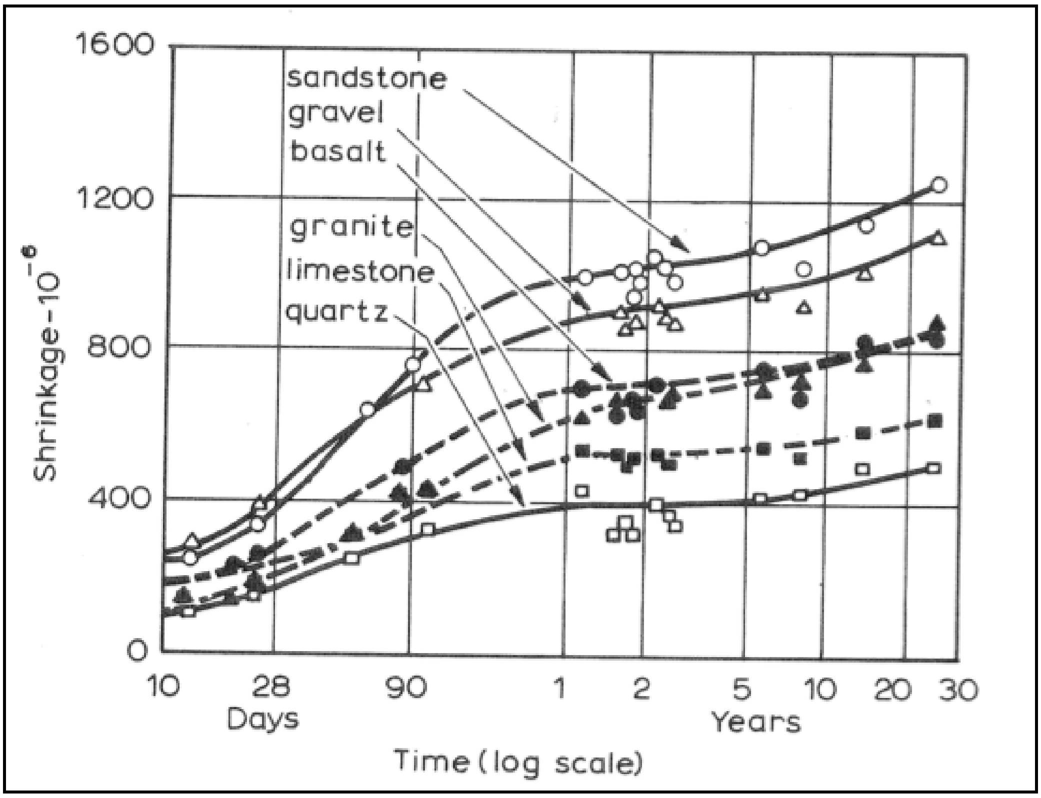


Figure 2.14: **Shrinkage of concretes of fixed mix proportions but made with different aggregates and stored in air at 21°C and a relative humidity of 50% (Neville, 1981)**

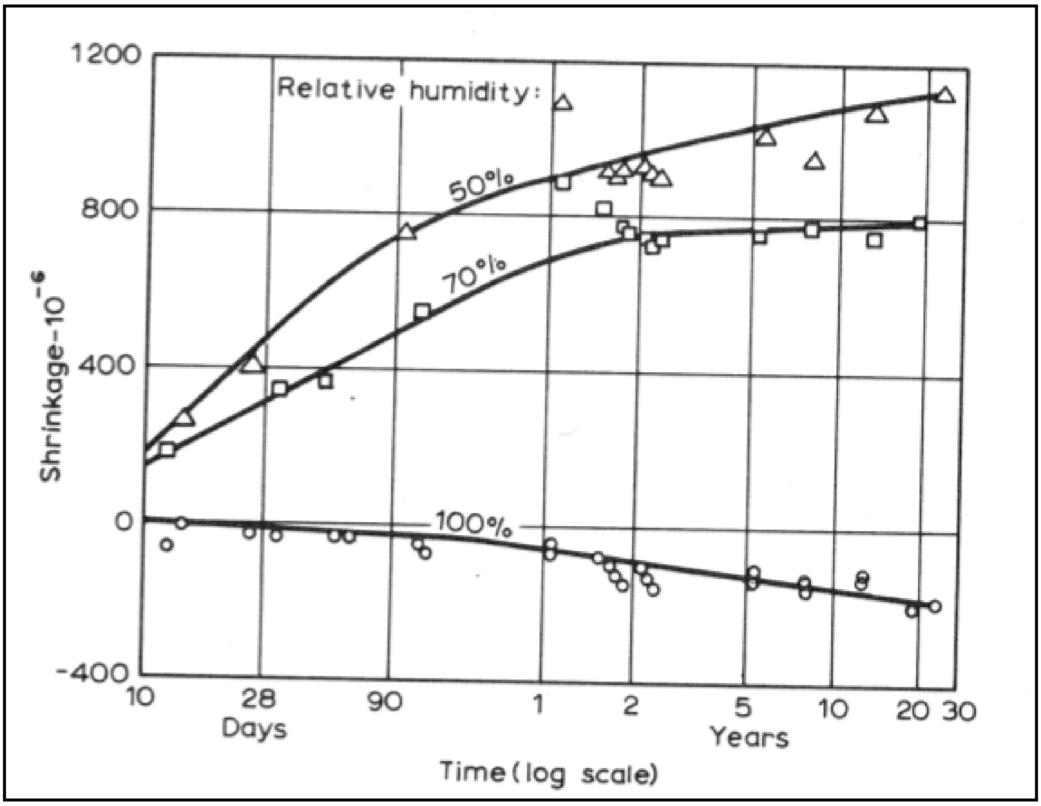


Figure 2.15: **Relation between shrinkage and time for concrete stored at different relative humidities (Neville, 1981)**

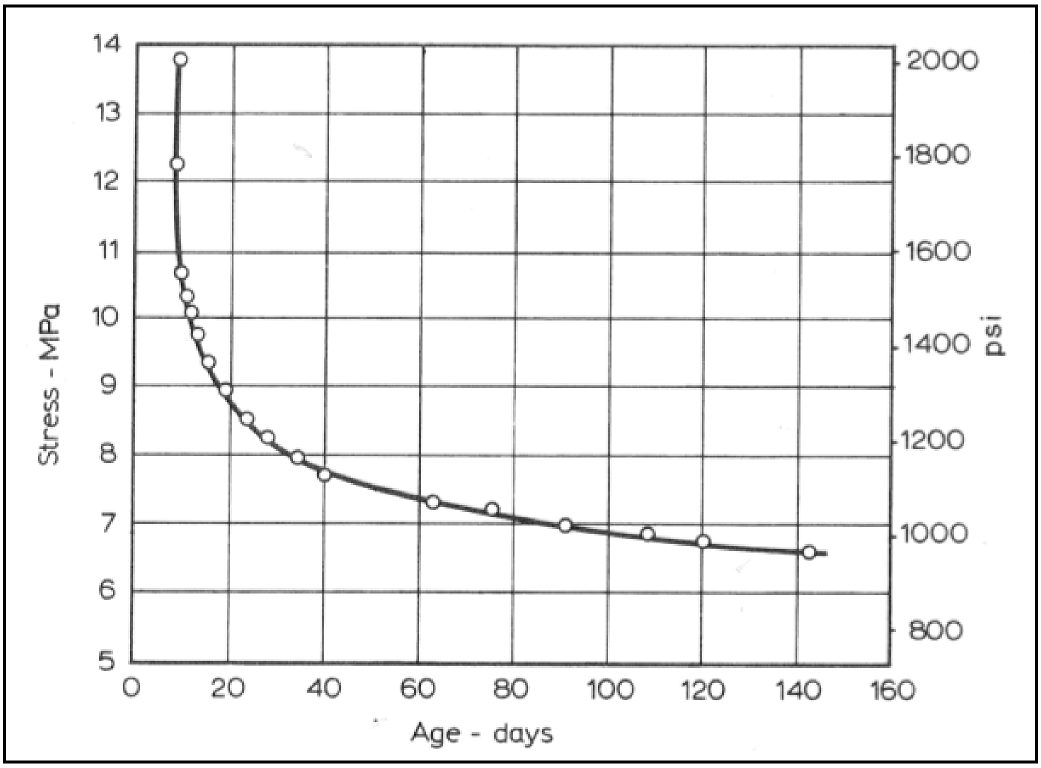


Figure 2.16: Relaxation of stress under a constant strain of 360×10^{-6} (Neville, 1981)

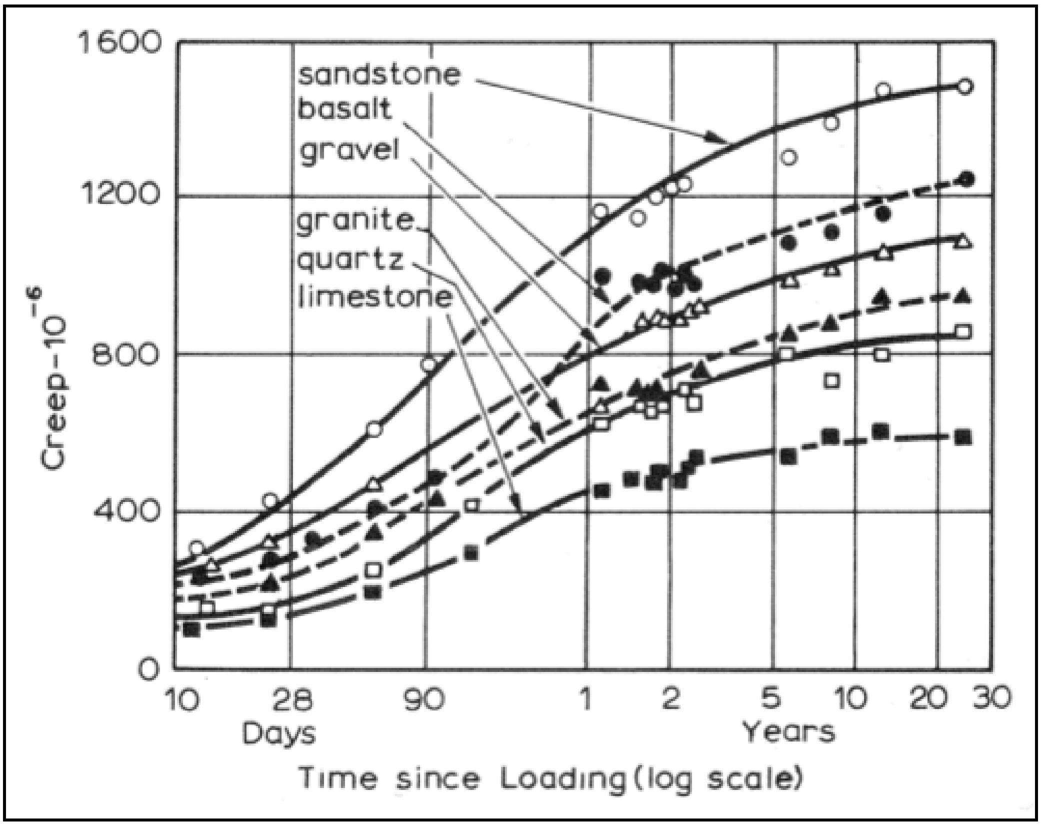


Figure 2.17: **Creep of concretes of fixed proportions but made with different aggregates, loaded at the age of 28 days and stored in air at 21°C and a relative humidity of 50% (Neville, 1981)**

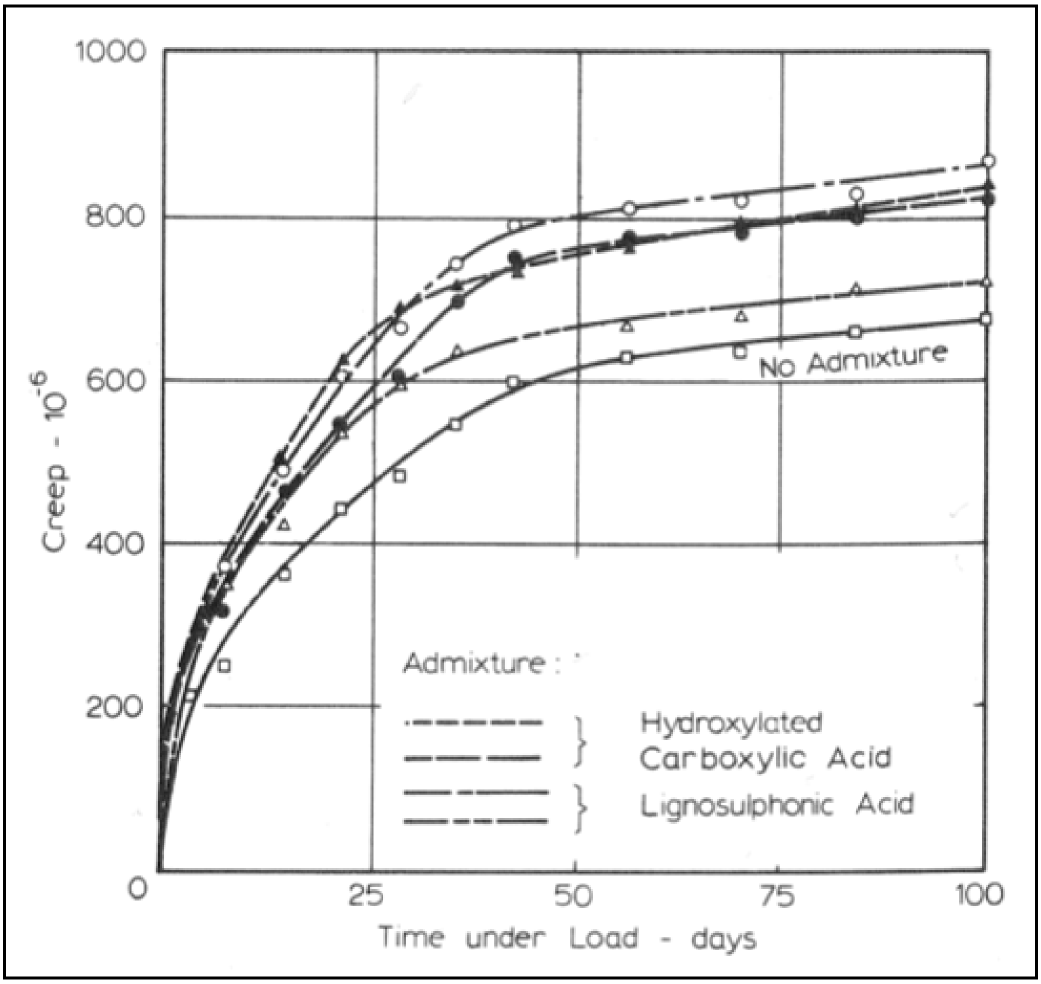


Figure 2.18: **Influence of water reducing and set retarding admixtures on creep of concrete (water/cement ratio = 0.65; age at loading = 28 days; relative humidity of storage = 94%) (Neville, 1981)**

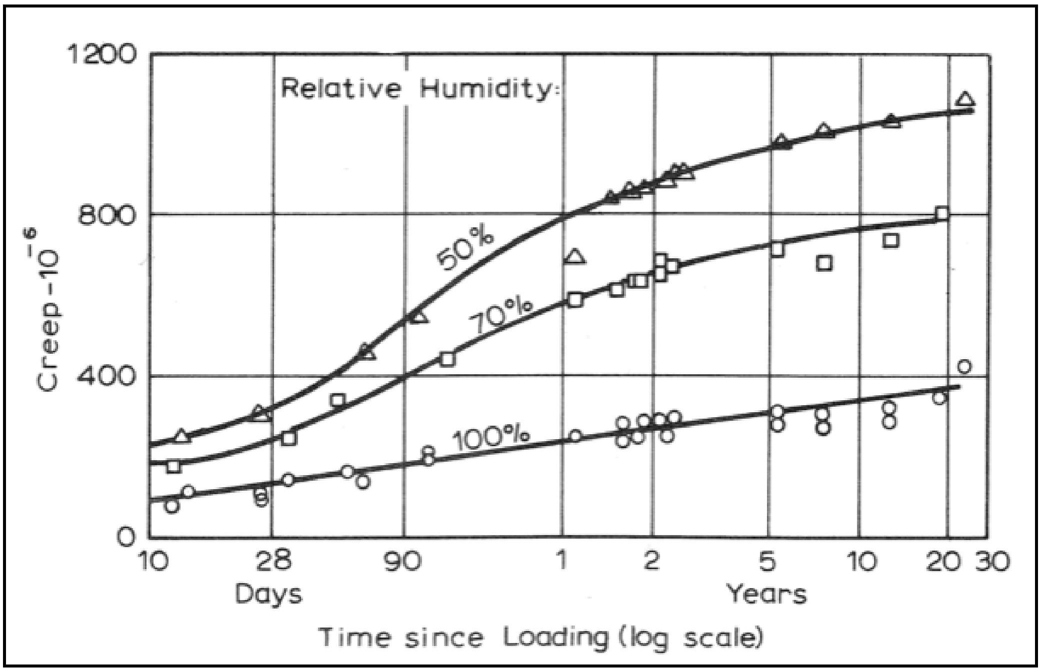


Figure 2.19: **Creep of concrete cured in fog for 28 days, then loaded and stored at different humidities (Neville, 1981)**

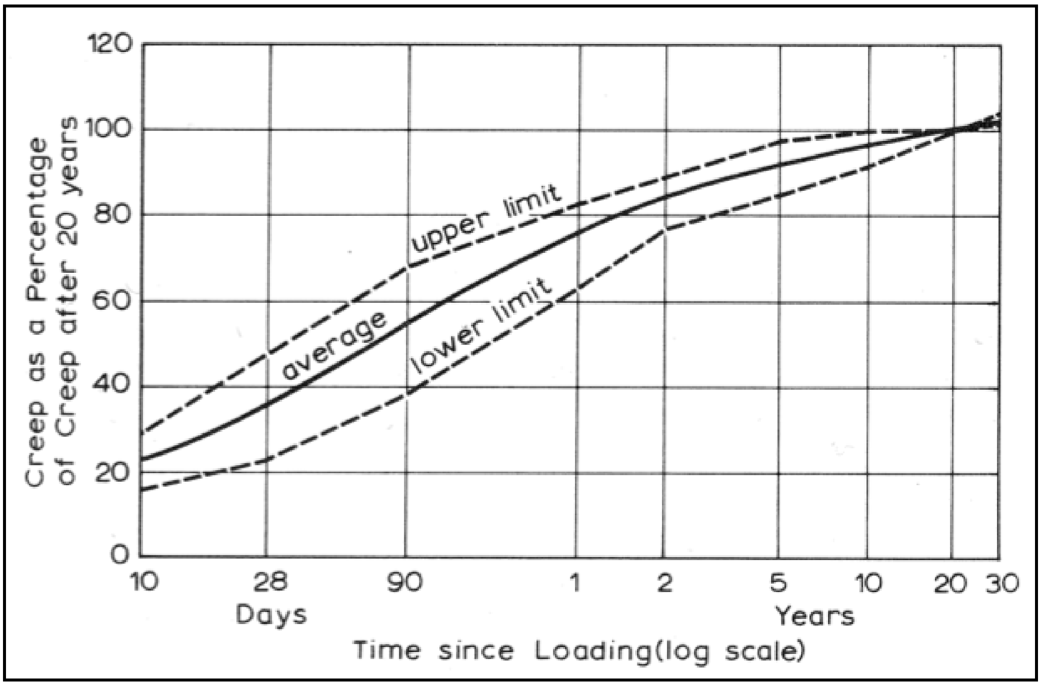


Figure 2.20: Range of creep-time curves for different concretes stored at various relative humidities (Neville, 1981)

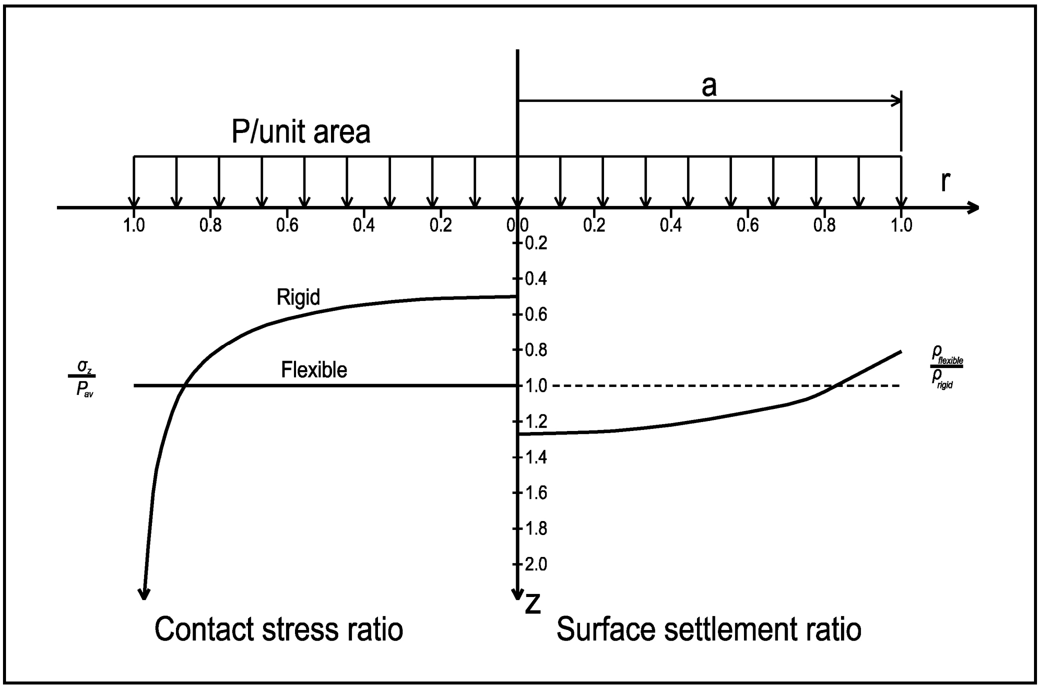


Figure 2.21: **Circular uniform distributed load on semi-infinite mass (Poulos and Davis, 1974)**

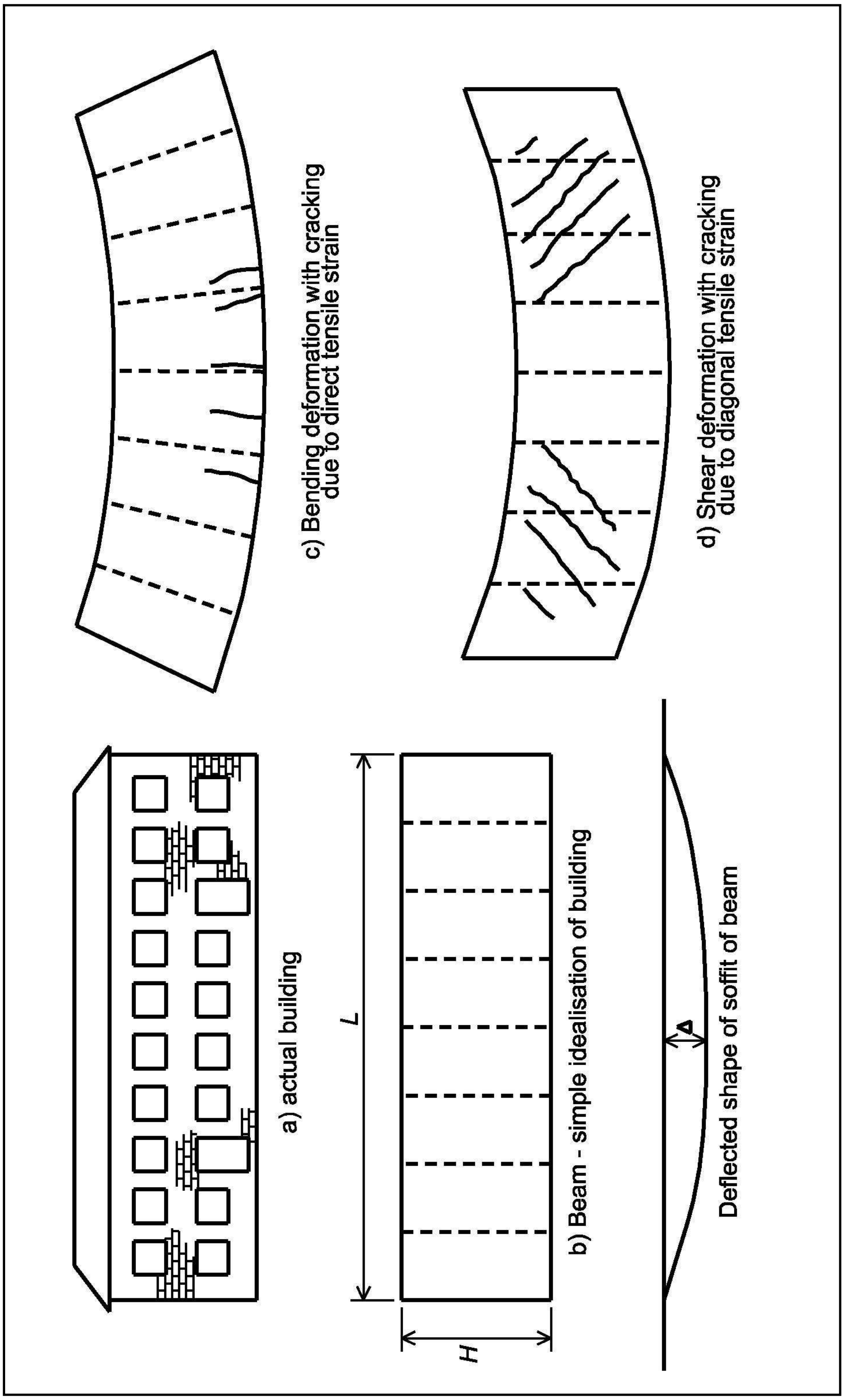


Figure 2.22: Cracking of a simple beam in bending and shear (Burland, 1975)

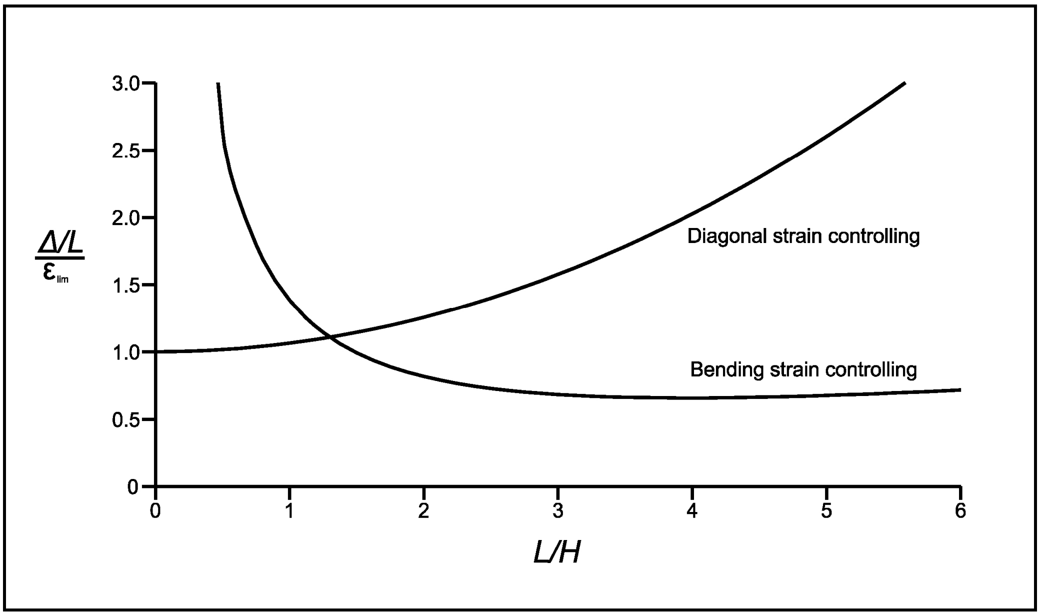


Figure 2.23: **Relationship between (Δ/*L*)/ε_{lim} and *L*/*H* for rectangular isotropic beams with the neutral axis at the bottom edge (Burland *et al.,* 2001a)**

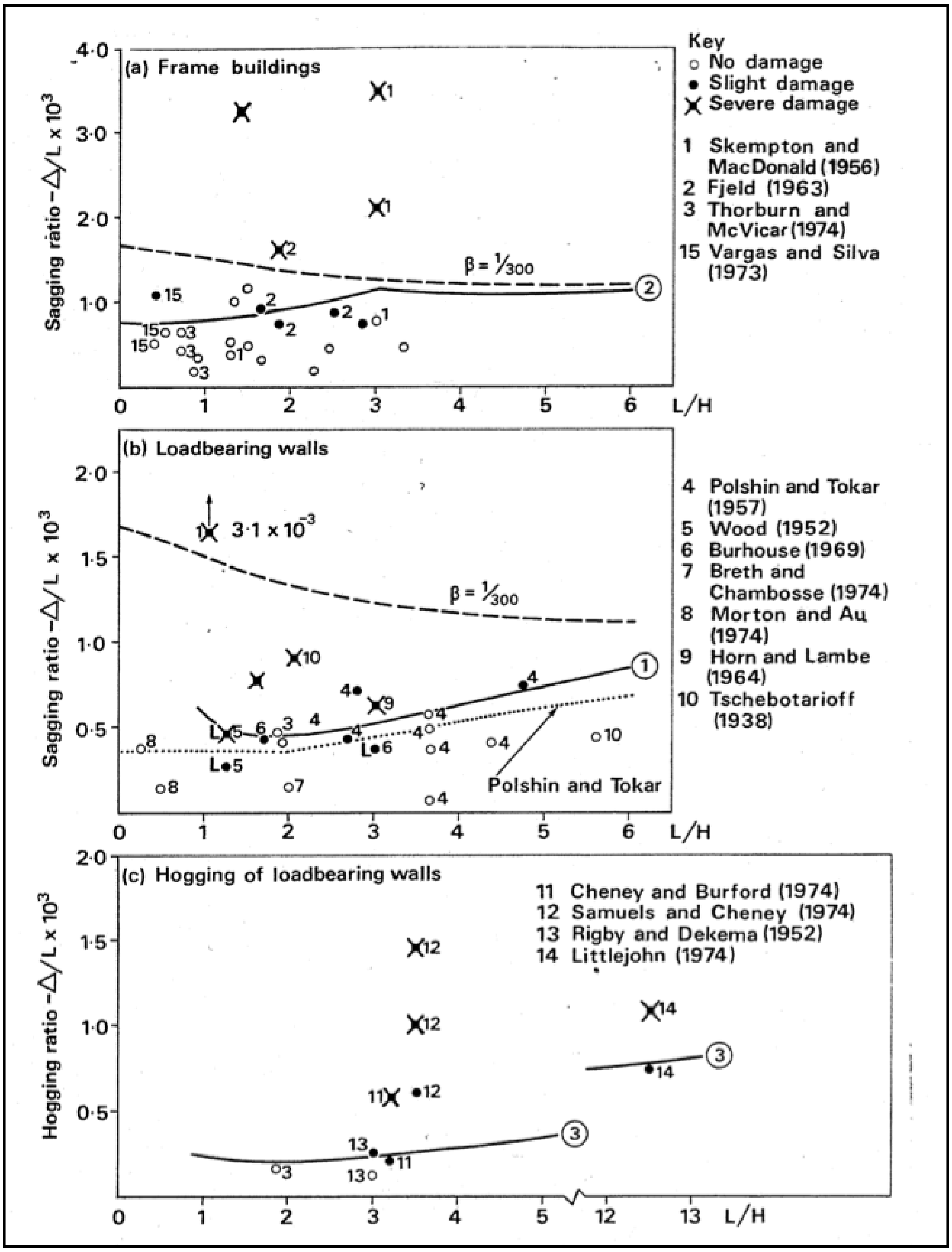


Figure 2.24: **Relationship between Δ/L and L/H for buildings showing various degrees of damage (Burland *et al.*, 1977)**

3 RESEARCH METHODOLOGY

Finite element analysis was used to investigate the behaviour of modern flexible framed structures undergoing differential settlement. Modern office buildings vary in design and to minimise the numerical modelling a “typical” modern structure was investigated. The structure investigated was a reinforced concrete structure with six storeys and five bays in both directions, supported by pad foundations. This structure was designed according to British Standards and Eurocode 7 to determine the member sizes for the finite element model. Using the structural sizing from the design the finite element model was created in phases, gradually expanding the geometry. Each phase of the model was analysed, the behaviour verified and problems corrected before commencing with the next phase. With the model completely built and verified the effect of soil-structure stiffness was investigated by changing the soil or concrete stiffness by varying the Young’s modulus of the material. The following analyses sets were run:

- For the 5 bay structure the soil stiffness for the whole halfspace was varied from a Young’s modulus of 100 Pa to 1 000 GPa, increasing the stiffness each time with an order of magnitude to determine the effect of soil stiffness on column loads, column displacements and bending moments in the slabs.
- For the 5 bay structure the Young’s modulus of the concrete was increased from 13 GPa to 13 000 GPa to determine the effect of a stiffer structure on column loads. 13 000 GPa is not a realistic stiffness, however it was used to increase the superstructure stiffness without any change in the geometry. The soil stiffness was varied as before.
- The structure geometry was reduced from 5 by 5 bays to 5 by 4 bays and 5 by 3 bays to determine the effect of structure geometry on column loads.
- The 5 by 5 bay superstructure was replaced by a slab with a bending stiffness equivalent to the individual slabs to determine the possibility of modelling an equivalent slab instead of the whole structure; and,
- The soil stiffness under individual pad foundations in the 5 by 5 bay structure was varied to determine the effect of soft or hard spots under foundations.

This chapter firstly discusses the layout of the structure and presents the structural design of the frame which was used to determine member sizes. Secondly the numerical model is presented. Thirdly the verification of the model is discussed and lastly typical results are presented. Chapter 4 contains a discussion of the analyses.

3.1 Structural design

3.1.1 Layout

To investigate the behaviour of a modern flexible frame structure the layout of the structure under investigation needed to be defined. For a representative structure the following assumptions were made:

- The structure was a 6 storey frame with 5 bays in each direction. This structure was chosen to provide a sufficiently large structure to evaluate the change in behaviour from external to internal bays and for subsequent levels. This was also the upper limit for the number of foundations and surrounding soil which the finite element analysis software could mesh adequately, given the computer software that was available.
- Floor and roof slabs were reinforced flat concrete slabs, i.e. no beams. Modern office buildings often use flat slab construction. The use of flat slabs also simplifies the geometry of a finite element model.
- Floor spacing was 3 m centre to centre which is typical of modern buildings.
- Columns were reinforced concrete with the same dimensions throughout the structure, i.e. sized according to the largest column load which is an internal ground floor column. This simplified the design and the finite element modelling.
- Column spacing was 7.5 m centre to centre which is typical for current flat slab construction.
- External facades and internal partitions were assumed not to alter the frame stiffness, i.e. movement was allowed between the facade or partition and frame.
- Ground floor columns were supported by individual pad foundations at 5 m below ground level. The 5 m depth ensured enough bearing capacity on an assumed London Clay for individual pad foundations for each column. Three different foundation sizes were used for corner, edge and internal foundations.
- The ground floor slab was supported by the underlying soil and not connected to the structure.

The complete structure layout was based on the above assumptions as shown in Figure 3.1.

3.1.2 Structural sizing

The structure as shown in Figure 3.1 was designed according to British Standards to determine member sizes. Eurocode 7 was used to determine the foundation footprint.

The design was based on the following characteristic loads derived from the British Standards.

- A uniform distributed imposed floor load of 2.5 kN/m² and a concentrated point load of 2.7 kN (BSI 6399-1: 1996).
- A uniform distributed imposed roof load of 1.5 kN/m² (BSI 6399-3: 1988).
- A horizontal uniform distributed wind load of 1.4 kN/m² (BSI 6399-2: 1997).
- A uniform distributed dead load of 0.419 kN/m² on the roof represented the asphalt waterproofing.
- A uniform distributed dead load of 0.566 kN/m² on the floor represented the floor finish; and
- A line load of 7.241 kN/m on the edge of each floor slab, represented facades (BS 648: 1964).

Note the above loads were for the ultimate limit state design and the live loads were significantly higher than the expected long term loads during the normal use of the structure. For the subsequent finite element analysis of the behaviour of the structure an expected uniform distributed imposed floor load of 0.5 kN/m² with no imposed roof or wind load was used.

Design loads were calculated using combinations of partial safety factors as described in BSI 8110-1 (1997). Appendix A shows the design load calculations. The following partial safety factors were used:

- Adverse dead load: 1.4.
- Beneficial dead load: 1.0.
- Adverse imposed load: 1.6, and
- Beneficial imposed load was neglected.

A C25/30 concrete as suggested by the BSI 8500-1 (2006) for office buildings and reinforcing steel with a 500 MPa tensile strength (BSI 8110-1: 1997) were used for the design.

The structural design was carried out using standard design Excel spreadsheets from RCC-2000. RCC-2000 are design spreadsheets based on BSI 8119-1 (1997) and were published by the British Cement Association on behalf of the industry sponsors of the Reinforced Concrete Council. The slab design calculations are shown in Appendix B and the column design calculations in Appendix C. Flat slab and column design were done according to BSI 8110-1 (1997).

The design showed that 300 mm flat floor slabs, a 250 mm flat roof slab and 450 mm x 450 mm columns were adequate. Although smaller columns may be used on the higher floors, 450 mm x 450 mm columns were used throughout the structure to simplify the design. A 200 mm ground floor slab was assumed. The ground floor slab was not connected to the columns and was supported by the underlying soil.

The following vertical characteristic column loads at ground level were calculated for foundation design. Live loads were reduced based on the number of floors supported (BSI 8110-1: 1997).

- Corner columns, dead load 1 050 kN, imposed load 160 kN.
- Edge columns, dead load 1 688 kN, imposed load 321 kN, and
- Internal columns, dead load 2 636 kN, imposed load 641 kN.

The above loads in combination with the foundation footprint area were used to calculate appropriate pad foundation thickness according to BSI 8110-1 (1997). The design calculations are shown in Appendix D. The following foundation thicknesses are adequate for the imposed loads.

- Corner pad, 0.50 m.
- Edge pad, 0.65 m, and
- Internal pad, 0.85 m.

Foundation footprint sizing was done according to Eurocode 7. The design calculations are shown in Appendix E.

The undrained shear strength (S_u) of the soil was assumed to be 90 kN/m² and saturated soil bulk unit weight 20 kN/m³. The undrained shear strength was based on the data collected by Burland *et al.* (2001a) on London Clay during the Jubilee Line Extension in London as shown in Figure 3.2. They recommend a design line of S_u = 50 + 8z

where z is the depth below the top of the London Clay. All the foundations were founded 5 m below ground level. The following pad foundation sizes and factors of safety regarding settlement were calculated according to Eurocode 7:

- Corner foundation 3.3 m x 3.3 m x 0.50 m thick (FOS 3.06)
- Edge foundation 4.2 m x 4.2 m x 0.65 m thick (FOS 3.02), and
- Internal foundation 5.4 m x 5.4 m x 0.85 m thick (FOS 3.04).

3.2 Finite element modelling

The LUSAS version 14.3 finite element analysis software package was used to perform the numerical modelling. LUSAS is the trading name of Finite Element Analysis Ltd whose headquarters are located in the UK. LUSAS is supported around the world by a number of LUSAS regional offices as well as by a network of LUSAS Distributors. LUSAS is a commercial package often used by structural engineers due to the ease with which superstructures can be modelled with standard element libraries. Other finite element analysis software packages with advanced soil models may be more suitable for geotechnical modelling, however for this modelling the decision was made to model the supporting halfspace as a linear elastic material, making LUSAS a suitable package. A 2.4 GHz Intel Core2 Duo PC with a 32-bit platform and 3 GB of RAM was used to run LUSAS.

Modelling the soil as a linear elastic material led to a significant reduction in the computational effort to run the model. Modelling of the soil as a linear elastic material was based on the underlying assumption that at working loads the soil mass was behaving in a linear elastic way. To validate this assumption the linear elastic model was compared to a model on non-linear soil; see the discussion in Chapter 4, discussion of the analyses.

The superstructure was also modelled as a linear elastic material to reduce computational effort. Although stress-strain behaviour in reinforced concrete within a structure is complex and influenced by creep, shrinkage and cracking, the use of a linear elastic material still provided valuable insight into the change of stress within the structure, although it may not represent the actual stress within a real structure. To validate this assumption the elastic bending moments in the slabs were compared to the design bending moments, as discussed in Chapter 4, discussion of the analyses.

3.2.1 Structural geometry

The structural geometry of the numerical model was based on the member dimensions as calculated in the structural design. Figure 3.1 shows the structural model and the geometry can be summarised as follows:

- five 0.3 m thick flat floor slabs spaced 3 m centre to centre,
- a 0.25 m thick roof slab, 3 m above the top floor,
- thirty-six 0.45 m x 0.45 m columns, (six in both spans) with a 7.5 m spacing,
- four corner pad foundations, 3.3 m x 3.3 m x 0.5 m,
- sixteen edge foundations, 4.2 m x 4.2 m x 0.65 m,
- sixteen internal foundations, 5.4 m x 5.4 mm x 0.85 m; and,
- all the foundations were founded at 5 m below ground level.

The following variations of the above model were analysed and are discussed in more detail in the subsequent paragraphs:

- The structure geometry was reduced from five by five bays to a five by four and a five by three bay structure.
- The structure above ground level was replaced with a single slab.

The structure's geometry was reduced to 5 by 4 and 5 by 3 bays to determine the effect of the structure's aspect ratio on the behaviour. The structure with the reduced geometry had the same size corner, edge and internal foundations as the original structure.

To replace the structure with an equivalent slab the thickness of the slab should be such that the bending stiffness (EI) of the single slab is equivalent to that of the structure. Two approaches were used to calculate an equivalent EI. The first approach employed the parallel axis theorem to define the structure's stiffness about the neutral axis as shown in Equation 3.1

$$(E_c I)_{Stiffstruct} = E_c \sum_{1}^{n} (I_{slab} + A_{slab} H^2) \qquad \textbf{Equation 3.1}$$

Where n is the number of storeys. For this structure it gave an equivalent slab thickness of 14.0 m. This could be considered to be an overestimate of the building stiffness as only a rigidly framed structure would approach such mode of deformation.

An alternative assumption was used to obtain the bending stiffness summing the independent EI values for each slab as shown in Equation 3.2

$$(E_c I)_{Flexstruct} = E_c \sum_{1}^{n} I_{slab} \qquad \textbf{Equation 3.2}$$

This implied that the columns transform the same deformed shape to each storey. For this structure it gives an equivalent slab thickness of 532 mm. This approach was used for the plate model. The loading on the replacement slab was equal to that of the whole structure. This was done by a distributed load to compensate for the live loads on the slabs and the self weight of the slabs and point loads for the columns.

3.2.2 Model discretisation and element types

Columns were modelled as 3D thick beam elements with restrained translation and rotation at end nodes. Each 3 m column and the columns below ground level (with a length of either 4.5 m, 4.35 m or 4.15 m due to a variation in foundation thickness) were discretised in 8 equal divisions.

The floor and roof slabs were modelled as thin shell 8 node quadrilateral elements taking membrane and flexural deformations into account. Both translation and rotation were restrained at nodes. Each bay was divided into 8 equal divisions in both the x and y direction.

The foundations were modelled with 10 node tetrahedral 3D continuum elements capable of modelling curved element boundaries. The nodes were free to rotate and only translation was restrained. The foundations were discretised with an irregular mesh with a maximum element size of a quarter of the foundation length. Figure 3.3 shows the mesh of foundations and the underlying soil.

The halfspace was modelled with the same 10 node tetrahedral 3D continuum elements as for the foundations. An irregular mesh with variable element sizes was used. Although LUSAS can vary the mesh size gradually, it was unable to mesh the model with 36 pad foundations with a gradually variable mesh. The halfspace was therefore divided into zones each with an assigned maximum element size. The choice of internal zones, element sizes and boundary location is discussed in detail in Chapter 3 under the verification of the finite element model. Figure 3.4 shows the discretisation zones and Table 3.1 lists the maximum element size in each zone.

The connections between the columns and slabs or foundations were fixed. The bases of the foundations were connected to the halfspace and the other sides (top and edge) of the foundations and the subsoil columns were disconnected from the halfspace allowing freedom of movement.

3.2.3 Material properties

The whole structure was designed based on RC 25/30 reinforced concrete. The following values were used for concrete properties in the numerical analysis:

- A Poisson's ratio of 0.2 (BS 8110-1: 1997).
- A Young's modulus of 13 GPa. British Standards Institution (BSI 8110-2: 1985) suggests a Young's modulus of 26 GPa for a RC 25/30 concrete at 28 days. Half of the recommended value was used in line with Brown and Yu's (1986) recommendation due to the lack of progressive modelling of the loading during construction; and,
- A concrete weight of 24 kN/m^3 (BS 648: 1964)

The soil was modelled as an elastic, isotropic material with a Poisson's ratio of 0.25. A range of Young's modulus values of 100 Pa to 10 GPa was used to display the effect of soil stiffness. This range of values is unrealistic for soil; however it is important to note that the behaviour of the structure depends on the relative bending stiffness ratio and not the absolute stiffness value of the soil. Figure 3.5 shows the envelope of undrained Young's modulus / undrained shear strength ratios with axial strain for a London Clay. The envelope is a result from triaxial tests performed on London Clay for the Jubilee Line Extension (Burland *et al.*, 2001a). For 0.1% local axial strain the undrained Young's modulus was between 250 and a 1000 times the undrained shear strength. For the design an undrained shear strength of 90 kPa was assumed and the corresponding Young's modulus at 0.1% strain will be in the range of 22.5 to 90 MPa.

3.2.4 Loading and restraints

The following loads were imposed on the model:

- A vertical acceleration of 9.81 m/s^2 on the concrete.
- A distributed load of 0.5 kN/m^2 on floor slabs represented the live loads.
- A distributed load of 0.566 kN/m^2 on floor slabs represented the finishes.
- A distributed load of 0.419 kN/m^2 on roof slabs represented the waterproofing; and,
- A line load of 7.241 kN/m on the edge of the floor slabs represented the facade.

The boundaries of the halfspace were modelled as fixed, except for the top surface which was free.

3.3 Verification of finite element model

The finite element model was built in stages, with each stage introducing a new aspect of the model. In each stage the behaviour of the model was verified before introducing more complexity in the following stage. The following stages were analysed and are discussed in the subsequent paragraphs:

- A single bay slab on pinned supports.
- A single bay slab with fixed support on the edges.
- A continuous slab with five spans in both directions on pinned supports.
- The 6 storey superstructure (slabs and columns) with fixed supports at ground level.
- A single pad foundation supported by an elastic halfspace.
- The superstructure founded on 36 pad foundations on an elastic halfspace.

Figure 3.6 shows the vertical displacement of a 7.5 m x 7.5 m x 300 mm concrete slab with self weight and an imposed load of 1.066 kN/m² (floor finish and live load) supported by four hinged supports. The maximum vertical displacement was 22.3 mm at the centre. The same slab modelled with fixed support on the edges had a maximum vertical displacement of 1.1 mm at the centre, which showed the clamping of the edges had a significant effect on the deflection.

Timoshenko (1959) gave the following solution for the bending of square plates under uniform loading:

$$y_{\max} = \alpha \frac{qb^4}{Et^3} \qquad \textbf{Equation 3.3}$$

Where y_{max} is the maximum displacement, α is a factor for the type of edge support, q is the uniform loading, b the length of the plate, E is the Young's modulus and t the thickness of the plate. For a plate with simply supported and clamped edges α is 0.0443 and 0.0138 respectively. The maximum displacements for the slab calculated with Timoshenko's solution for the simply supported and clamped edges were 3.3 mm and 1.0 mm respectively.

The maximum displacement (1.1 mm) of the modelled slab with clamped edges correlated well with the solution by Timoshenko (1.0 mm). The maximum displacement on the modelled slab (22.3 mm) was significantly more than the displacement of the slab that was simply supported on the edges (3.3 mm). This showed that line supports reduced the displacement significantly in comparison to pinned point supports.

Figure 3.7 shows the vertical displacement of a complete level with 25 bays on pinned supports. Loading was the same as for the single bay except for the added line load of 7.241 kN/m at the edge of the floor slab which represented the weight of the external facade. The maximum vertical displacement for the corner slabs was 13.8 mm. For the edge slabs it ranged from 8.7 mm to 9.6 mm, and for the internal slabs it ranged from 3.4 mm to 5.9 mm. This clearly showed the effect adjacent bays had on the reduction of the deflection of the slabs.

Figure 3.8 shows the vertical displacement of the complete six storey superstructure on fixed supports at ground level. It is important to note that the top level was the roof slab with a thickness of 250 mm with no live load and therefore the vertical displacement will vary from a typical floor slab. The maximum vertical displacements of the top floor level (not roof) are summarised in Table 3.2 (column 2). It is clear the vertical displacements were significantly higher than for the single 25 bay slab (column 1). This was due to the shortening of the columns under load. Normalising the slab vertical displacement by subtracting the average vertical column displacement for the specific bay gave the displacements as shown in column 3. These values correlated well with the values of the single 25 bay slab. Column 4 gives the values of column displacement at the top floor calculated by the assumption that each column supported 25% of each adjacent span. The values correlated well with the values from the numerical model.

The sum of the vertical reactions at the supports in the model was within 2% of the total load takedown determined by hand calculations. Assuming each ground level support carried the columns above and 25% of the adjacent bays, an estimation of the expected column loads was made. Table 3.3 shows the comparison between column loads from the numerical model and hand calculation. All of the column loads were within 10% of the estimated values, which shows that the model behaved as expected.

The next step was to add foundations and the elastic halfspace to the model. It was critical to get a reasonable load-displacement response for the foundations while keeping the number of meshed elements to the minimum. Due to the scale of the model, a small increase in the mesh density had a large effect on the number of elements in the model, which may result in the inability to mesh to the model. To investigate the load-displacement response of the foundations, a single rigid foundation was modelled on an elastic halfspace and the results compared to a known elastic solution from Poulos and Davis (1974). The mesh, element type and halfspace boundaries of the halfspace supporting the single foundation were varied to optimise the mesh, element type and halfspace size.

A regular mesh for the halfspace proved to be unsuitable for the large model. It either created too many elements or unacceptably distorted the aspect ratio of the elements. An irregular mesh with tetrahedral elements was therefore used. Although LUSAS was capable of generating an irregular mesh with a gradually changing mesh density, it was not capable of applying the automated process to the large model with multiple foundations. To overcome this problem, zones with different mesh densities were manually assigned to the model. The single foundation model was used to optimise the zones, mesh density, and the boundary locations. The aim was to get an acceptable foundation load-displacement response with the smallest number of elements. The size of the linear elastic model was limited by the discretisation of the foundations and supporting halfspace and not by the solver. LUSAS is only available as a 32-bit program and can therefore only use a maximum of 3 GB of RAM.

The single foundation model consisted of a rigid 5.4 m x 5.4 m pad foundation on an elastic halfspace with a Young's modulus of 10 MPa and a Poisson's ratio of 0.25. Symmetry was used, to reduce the model size to a quarter. A 100 mm displacement was applied to the foundation and the reaction force calculated. The reaction force was compared to the following approximate elastic solution of a rectangular rigid loaded area on a semi infinite mass (Whitman and Richart, 1967, cited in Poulos and Davis, 1974):

$$\rho_z = \frac{P(1-\upsilon^2)}{1.2\times bE} \qquad \textbf{Equation 3.4}$$

More than 50 model load-displacement responses were analysed to determine an optimal model. Figure 3.9 shows the dimensions of the optimised zones with respect to the foundation width. Table 3.4 shows the effect of halfspace size, element type and element size on the load-displacement response. The type of halfspace element had a significant effect on the accuracy of the solutions. A TH4 element was a 4 node tetrahedral element and a TH10 was a 10 node tetrahedral element capable of modelling curved boundaries. Changing from a TH4 to a TH10 element reduced the error from 35.9% to 7.0%. An increased halfspace size and the use of a finer mesh discretisation reduced the error. Model 2 was chosen as a basis for the discretisation of the large model. Although this model was not the most accurate model, having a 7.0% error, it used fewer elements than the more accurate models and therefore significantly reduced the number of elements in the whole model. The ratios of the zone dimensions and the element sizes from the single pad foundation were applied separately to each foundation in the large model, as well as to the overall footprint of the building. In overlapping areas, the finer mesh was used. The layout from Model 2 was the upper limit with respect to elements that could be modelled successfully for the complete geometry.

Using the above discretisation the structure with foundations on an elastic halfspace with a Young's modulus of 10 MPa was modelled. A comparison of the column loads at ground level showed that the total load in the model was within 1 % of the total load takedown. This model was used to verify the suitability of the slab discretisation and element type. The columns loads were calculated and compared for both 4 noded and 8 noded quadrilateral elements. Each bay was divided into either 4, 8, 12, 16 or 20 divisions. Figure 3.10 shows the column loads of the models normalised to the column load of the 12 divisions, 8 noded elements (which was used for further modelling). It was evident that the 8 noded element resulted in more accurate predictions than the 4 noded element. With the 8 noded element the structure was insensitive to the number of divisions with the difference in column load between 2 and 12 divisions being less than 3%. The suitability of the column mesh was tested by comparing the ground level column loads for 4, 8 and 12 column mesh divisions. The change of column loads were within 0.1% of each other and therefore the difference between using 4, 8 or 12 divisions was insignificant. 8 column divisions were used for the final model.

3.4 Typical model results

In this section typical modelling results are presented. Specific aspects are discussed in detail in Chapter 4, discussion of results.

For the design, an undrained soil strength of 90 kPa was assumed. Based on Figure 3.5 the stiffness of clay with an undrained shear strength of 90 kPa at 0.1% strain typically ranged from 22.5 MPa to 90 MPa. A stiffness value of 50 MPa was used for the model. Figure 3.11 shows the vertical strain under the pad foundations. The vertical strain for the internal foundations typically ranged from -0.1% to -0.2% and for the edge and corner foundations from -0.075% to -0.15%. Further away from the foundation base the strain was lower, which may have resulted in a higher stiffness.

Figure 3.12 shows the vertical displacement at foundation level. Foundation displacements ranged from 20 mm to 45 mm. Equation 3.4 was used to calculate single rigid foundation settlement for comparison. Using a load takedown (based on the assumption that each column supports half of the span) foundation loads were calculated. With the foundation load, a soil stiffness with Young's Modulus of 50 MPa and a Poisson's ratio of 0.2, the following rigid foundation settlements were calculated according to Equation 3.4:

- Single corner foundation: 5.6 mm.
- Single edge foundation: 7.4 mm.
- Single internal foundation: 9.8 mm.
- Total superstructure footprint (37.5 m x 37.5 m): 37.9 mm.
- Total 'foundation' footprint (41.7 m x 41.7 m): 34.1 mm.

The single rigid foundations had significantly less settlement than the combined foundations in the finite element model. This was due to the interaction effect of the adjacent foundations. The total rigid 'foundation' footprint settlement correlated well with the finite element model. As expected the rigid settlement was within the range of the settlement of the model with a flexible structure.

Figure 3.13 shows the vertical stress beneath the foundations. The vertical stress under the foundations ranged from 40 kPa to 110 kPa. The total building load (including the foundations) divided by the total foundation area gave an average vertical stress of 115 kPa, which was higher than the range of 40 kPa to 110 kPa. The stress contours in this figure are ‘smoothed’ and based on averaged nodal values derived from extrapolation of the values in the integration points.

Figure 3.14 shows the vertical displacement of the structure. The maximum vertical displacement was 67.3 mm at the midspan of a roof slab. This displacement was due to foundation settlement, column compression and slab deflection.

Figure 3.15 shows the pattern of the bending moment on the first floor slab. The bending moments in the floor slabs are discussed in detail in Chapter 4, discussion of analyses.

Table 3.1: Maximum element size in mesh discretisation zones

Zone	Element size
1	0.25B
2	0.5B
3	1B
4	2B
5	4B
6	4B

Table 3.2: Summary of vertical displacements

	25 bay slab (mm)	Superstructure top floor (mm)	Superstructure normalised (mm)	Hand calculation top floor (mm)
Corner slab	13.8	21.6	14.7	NA
Edge slab	8.7-9.6	18.2-18.4	9.6-9.9	NA
Internal slab	3.4-5.9	15.2-15.7	4.4-5.1	NA
Corner column	NA	3.0	NA	3.7
Edge column	NA	5.5-6.1	NA	6.2
Internal column	NA	10.0-12.5	NA	10.3

Table 3.3: Column load comparison

Column	FE Model (kN)	Hand Calculation (kN)	Difference (%)
Corner	947	1030	9
Edge	1632 to 1708	1702	0 to 4
Internal	2763 to 3 025	2773	0 to -8

Table 3.4: Optimisation of single foundation model

Model	Element	Halfspace size (D, H/2 with respect to B)	Maximum element size with respect to B in zone						Total elements No.*	Difference from solution %
			1	2	3	4	5	6		
1	TH4	10	0.25	0.5	1	2	4	NA	761	35.9
2	**TH10**	**10**	**0.25**	**0.5**	**1**	**2**	**4**	**NA**	**749**	**7.0**
3	TH10	5	0.25	0.5	1	2	NA	NA	789	12.8
2	**TH10**	**10**	**0.25**	**0.5**	**1**	**2**	**4**	**NA**	**749**	**7.0**
4	TH10	20	0.25	0.5	1	2	4	4	1484	3.9
5	TH10	10	0.5	1	2	4	8	NA	248	11.8
2	**TH10**	**10**	**0.25**	**0.5**	**1**	**2**	**4**	**NA**	**749**	**7.0**
6	TH10	10	0.125	0.25	0.5	1	2	NA	4275	1.8
7	TH10	10	0.0625	0.125	0.25	0.5	1	NA	26348	0.1

* The number of elements if symmetry is used. Without the use of symmetry it would be approximately four times more.

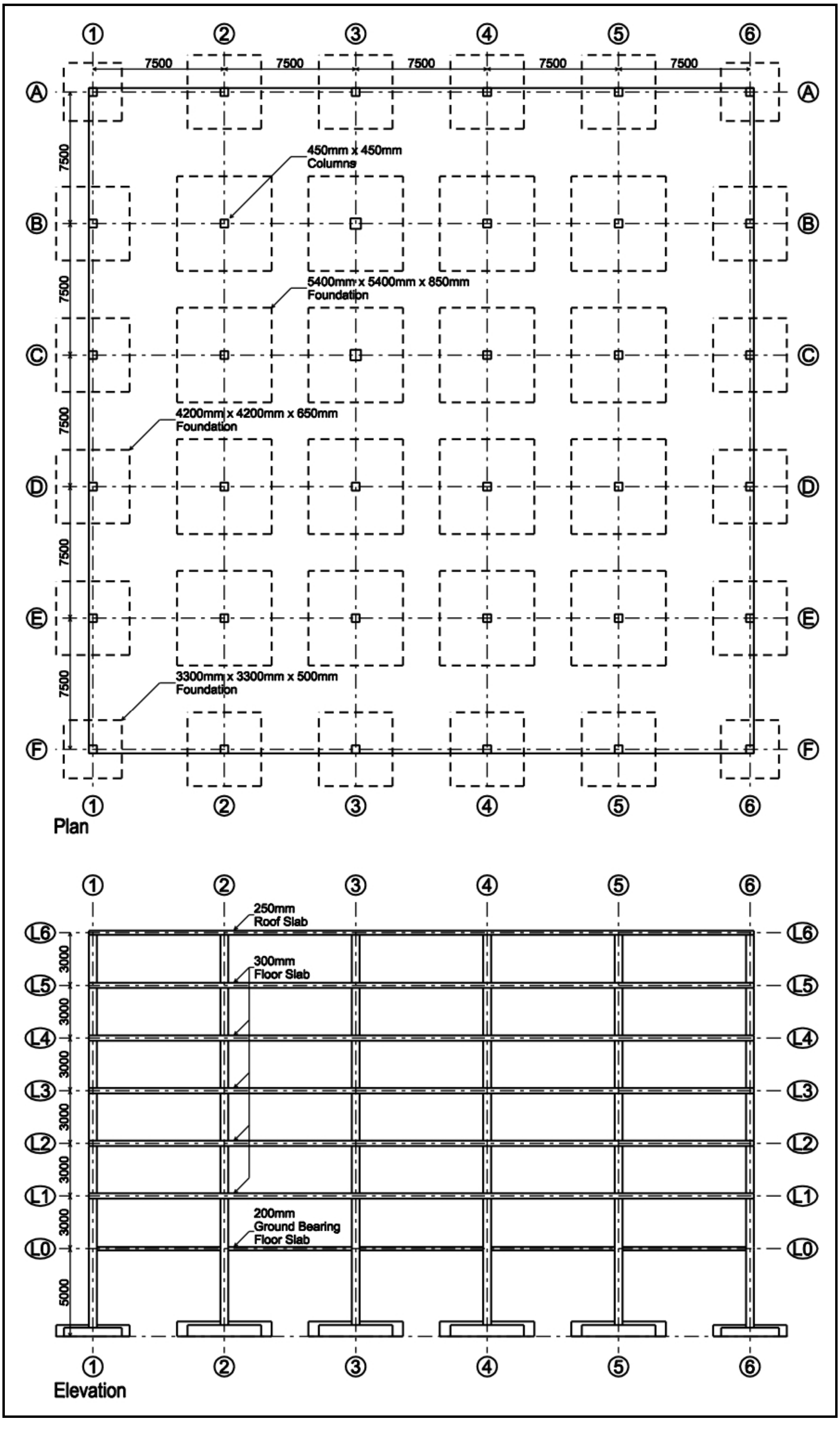


Figure 3.1: Building layout

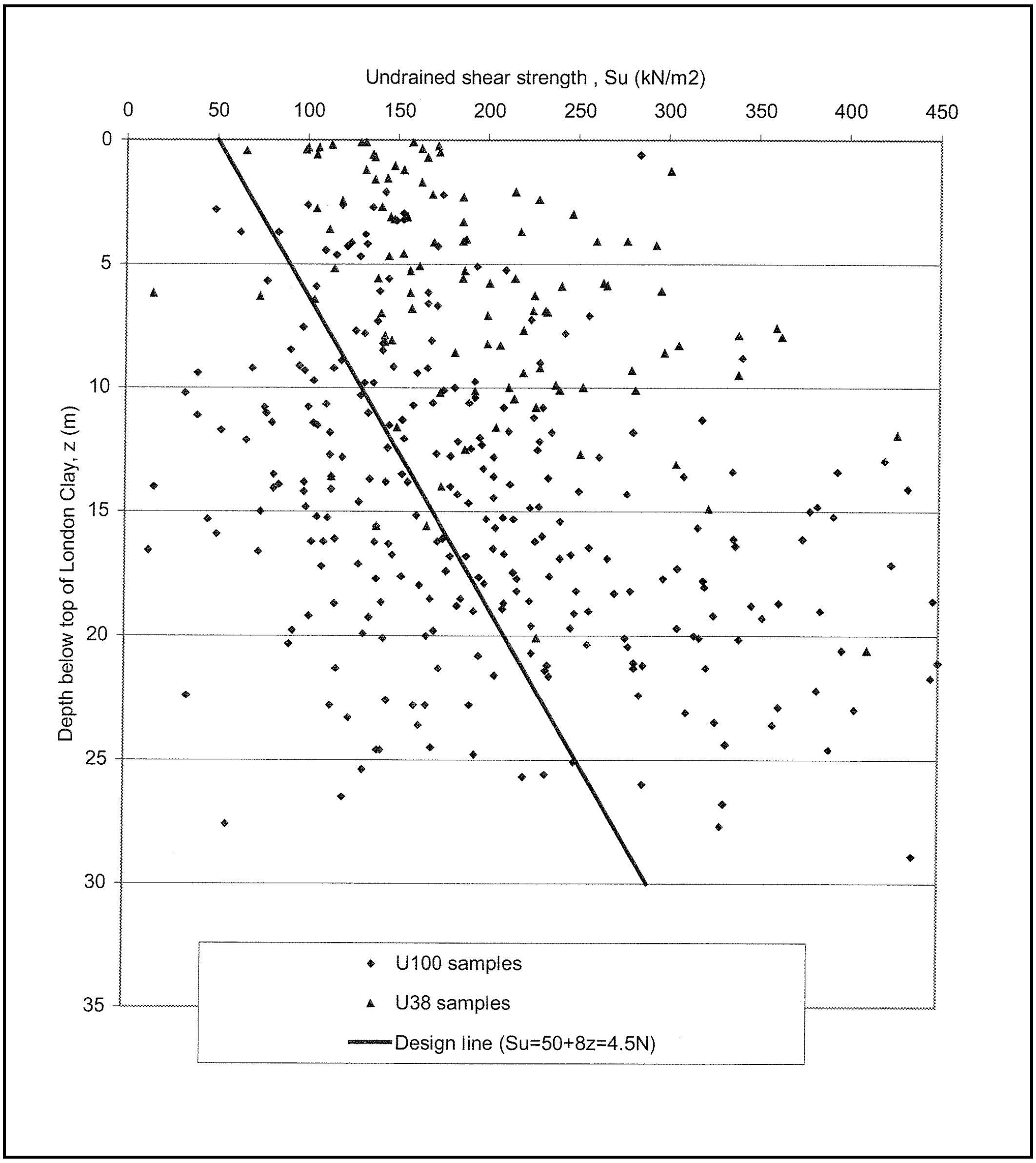


Figure 3.2: **Undrained shear strength of London Clay (Burland *et al.*, 2001a)**

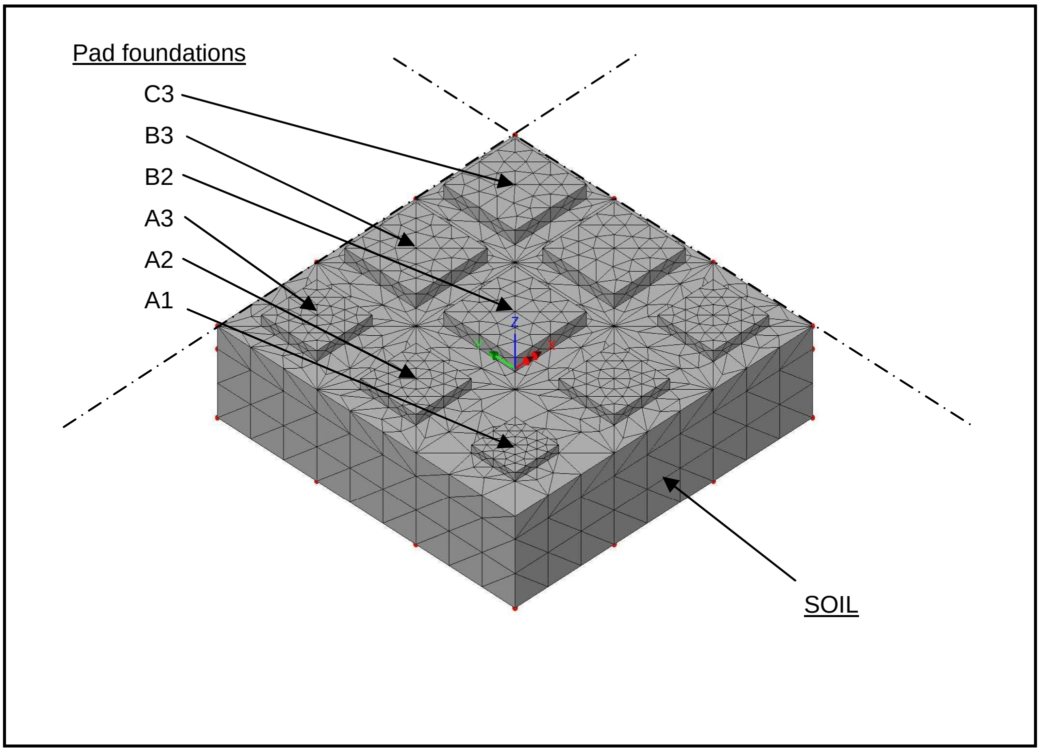


Figure 3.3: **Meshed foundations on supporting soil (quarter of model shown)**

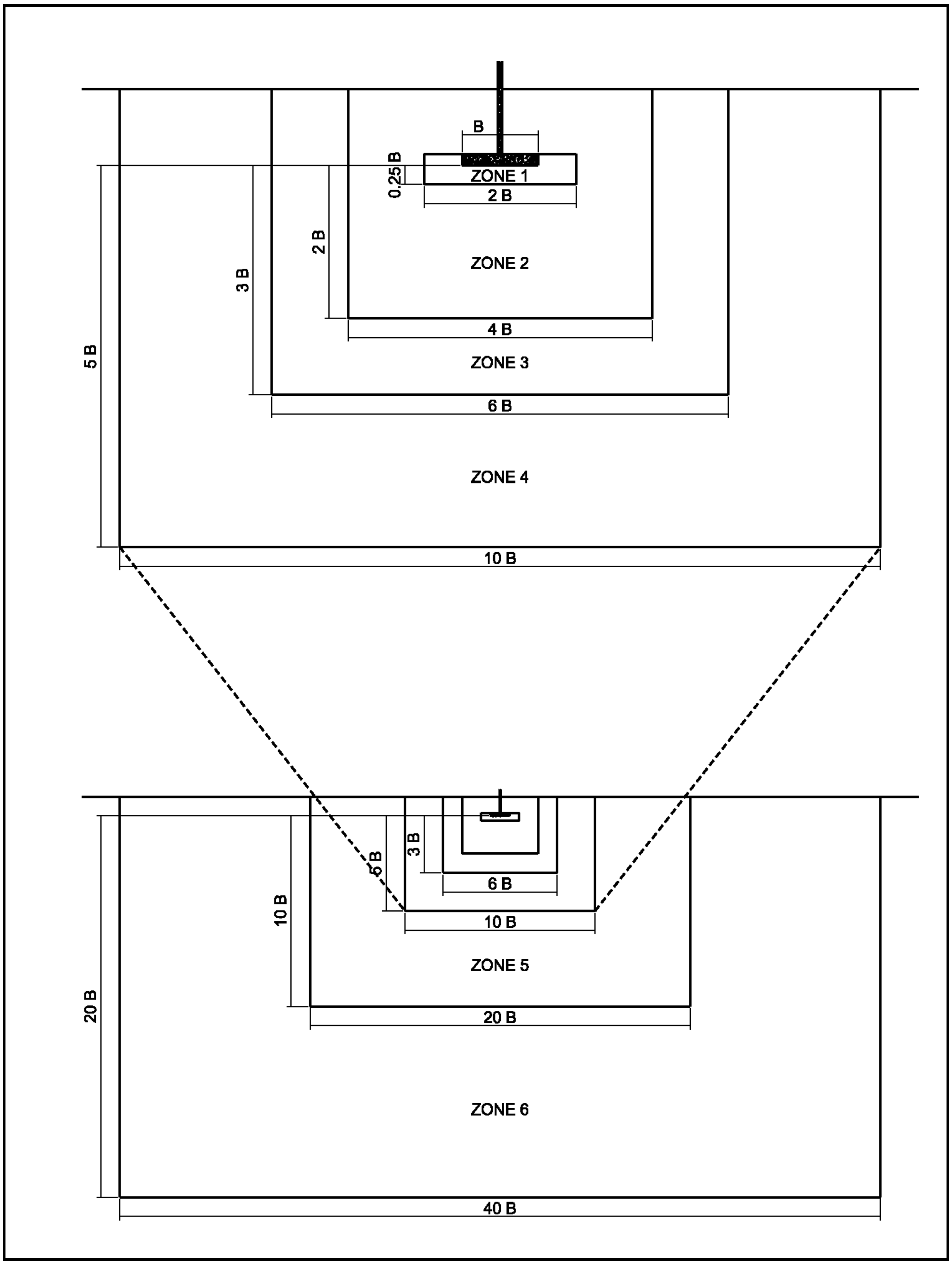


Figure 3.4: **Discretisation zones**

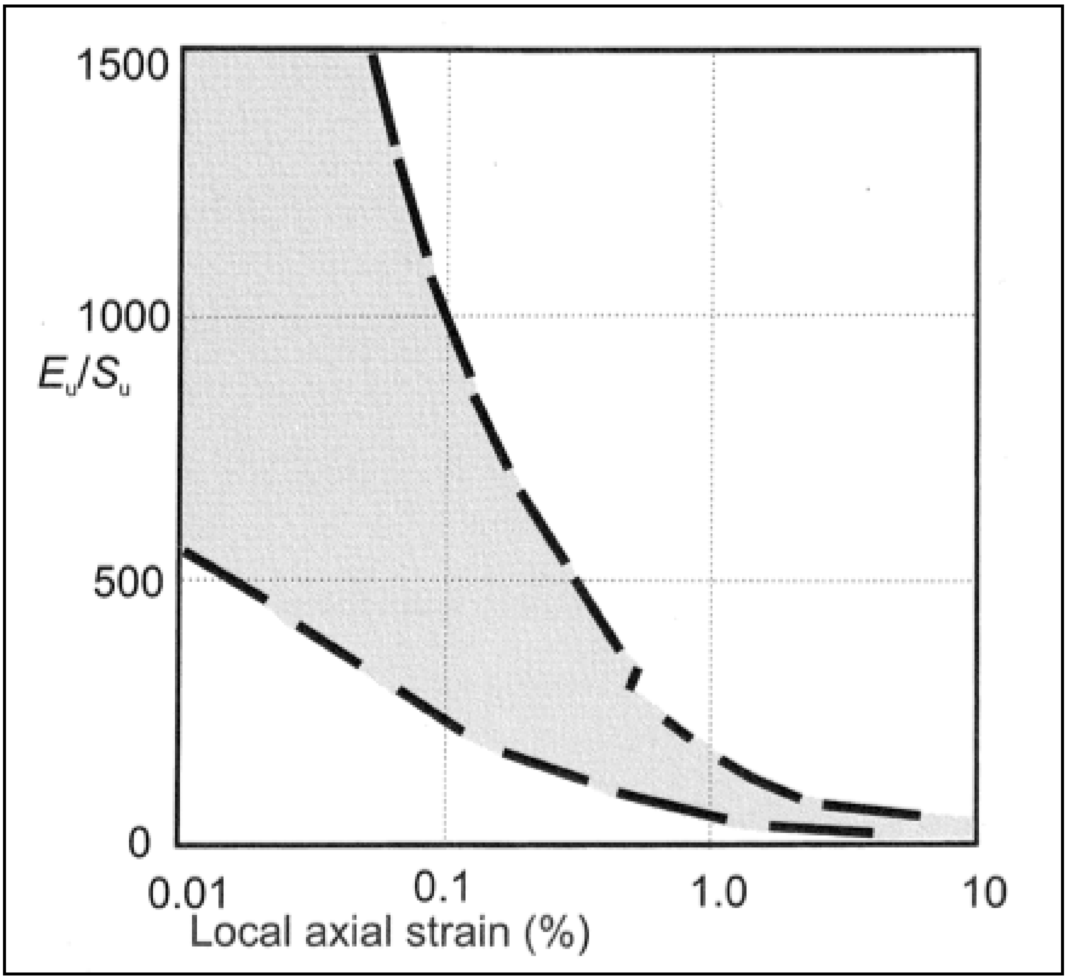


Figure 3.5: **Envelope of undrained Young's modulus / undrained shear strength with axial strain for London Clay (Burland *et al.,* 2001a)**

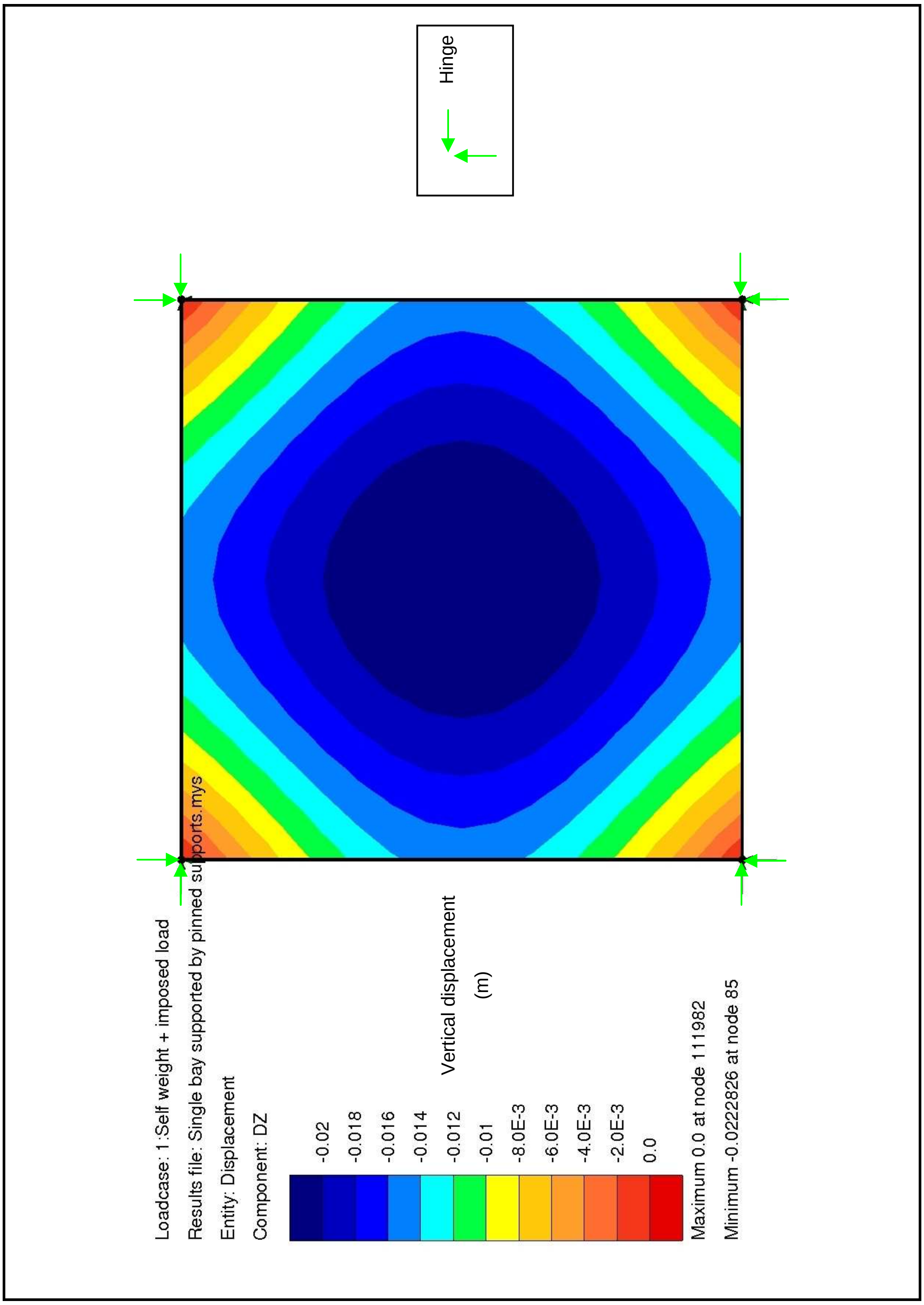


Figure 3.6: **Vertical displacements of single bay slab supported by pinned support**

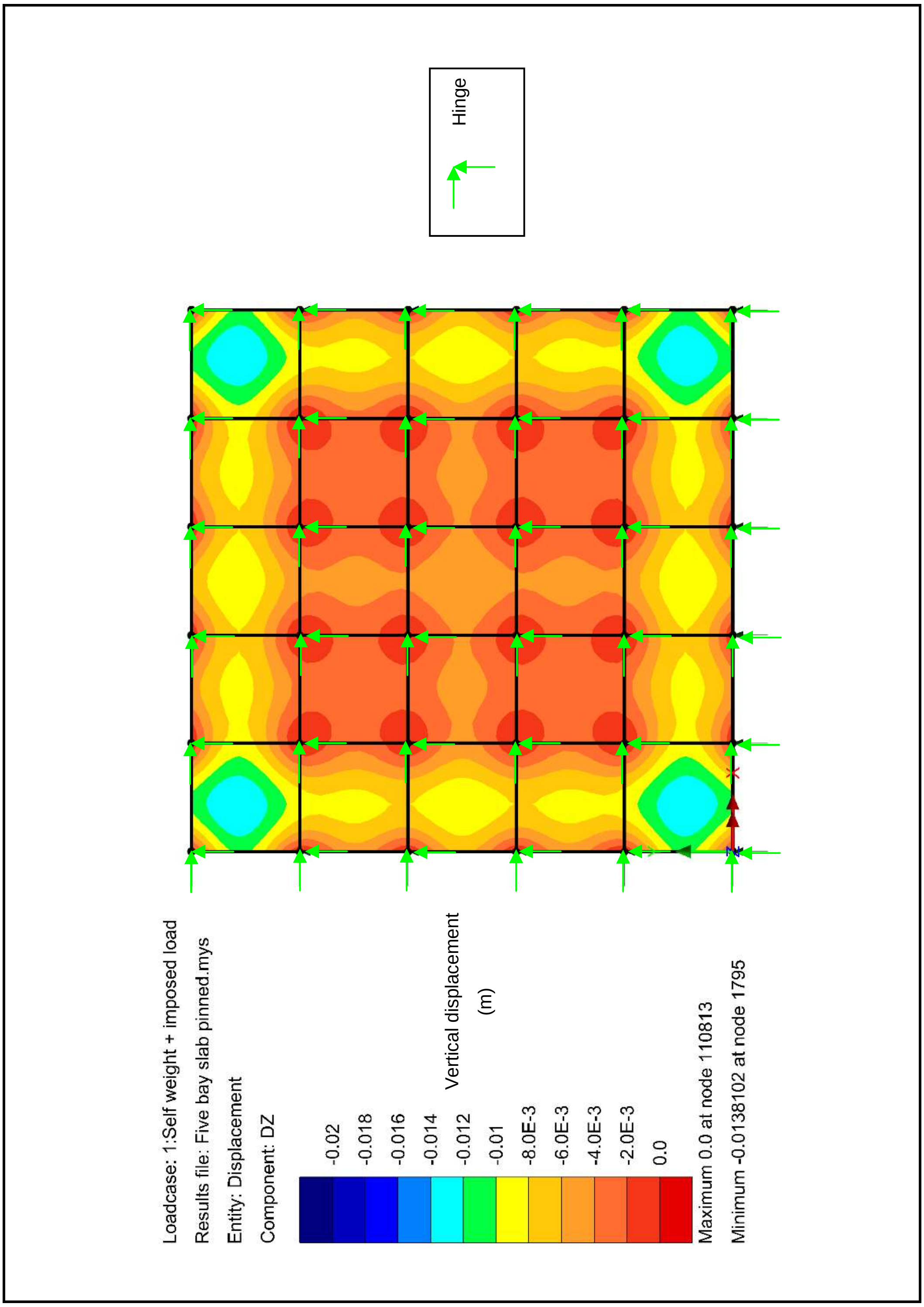


Figure 3.7: **Vertical displacements of a 25 bay slab**

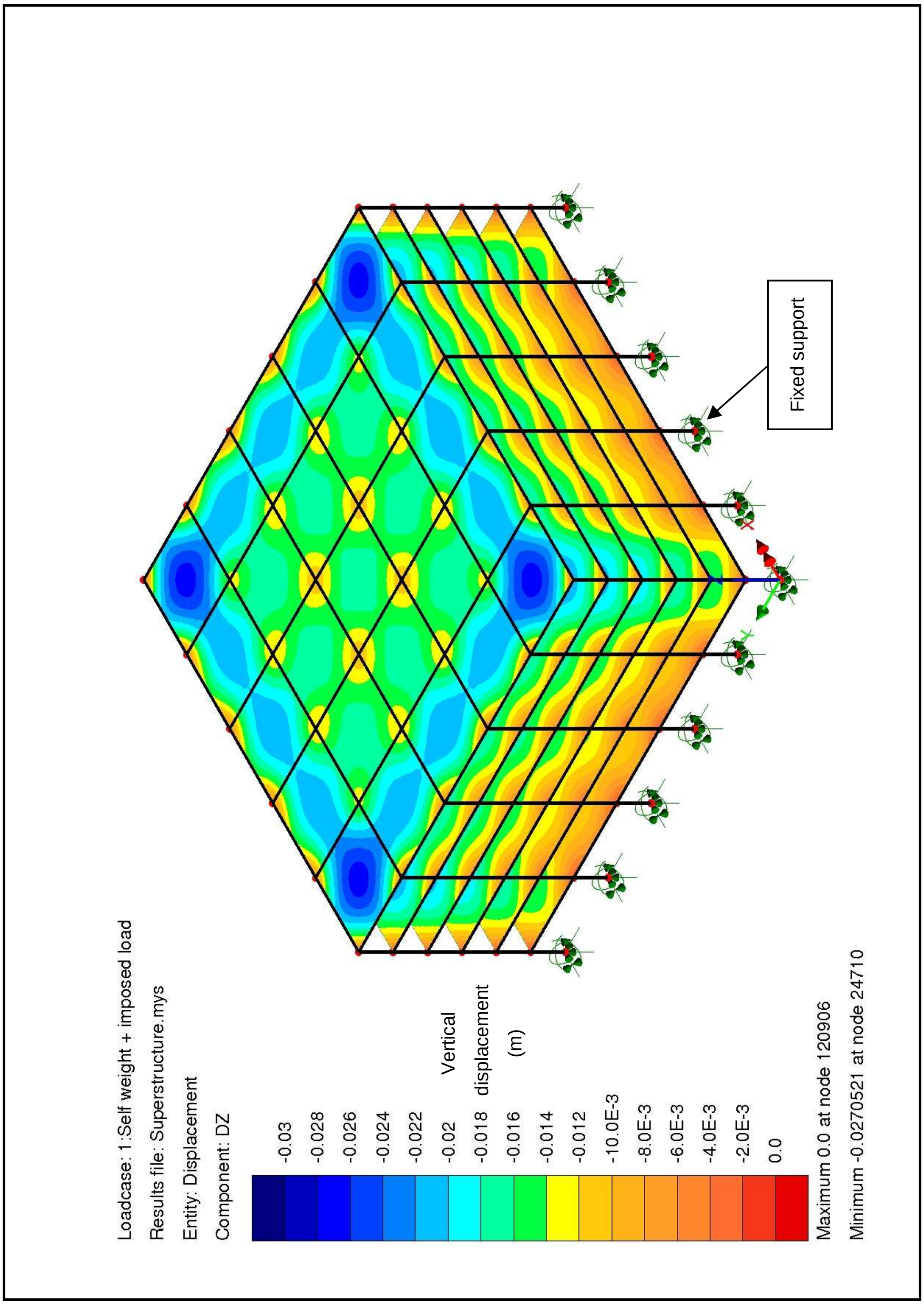


Figure 3.8: Vertical displacements of the superstructure on fixed supports

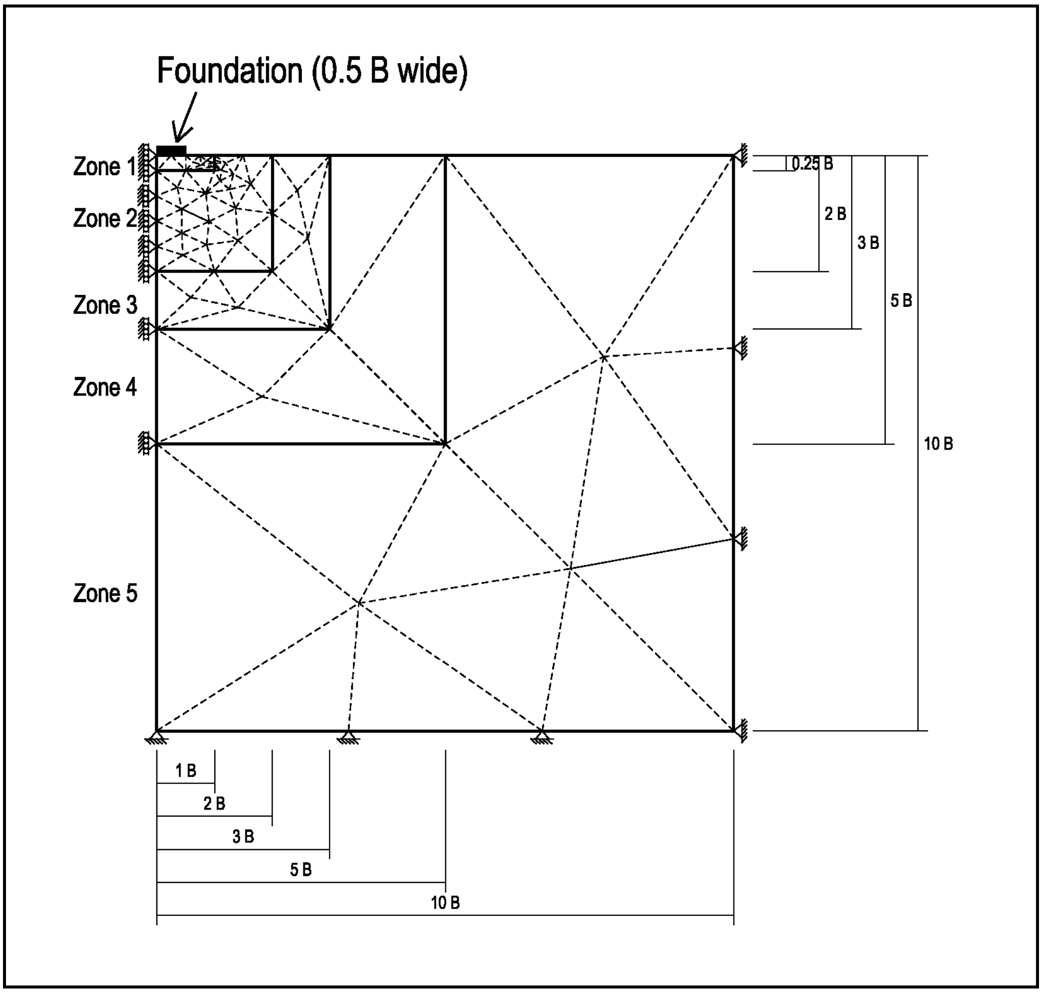


Figure 3.9: **Mesh for single foundation model**

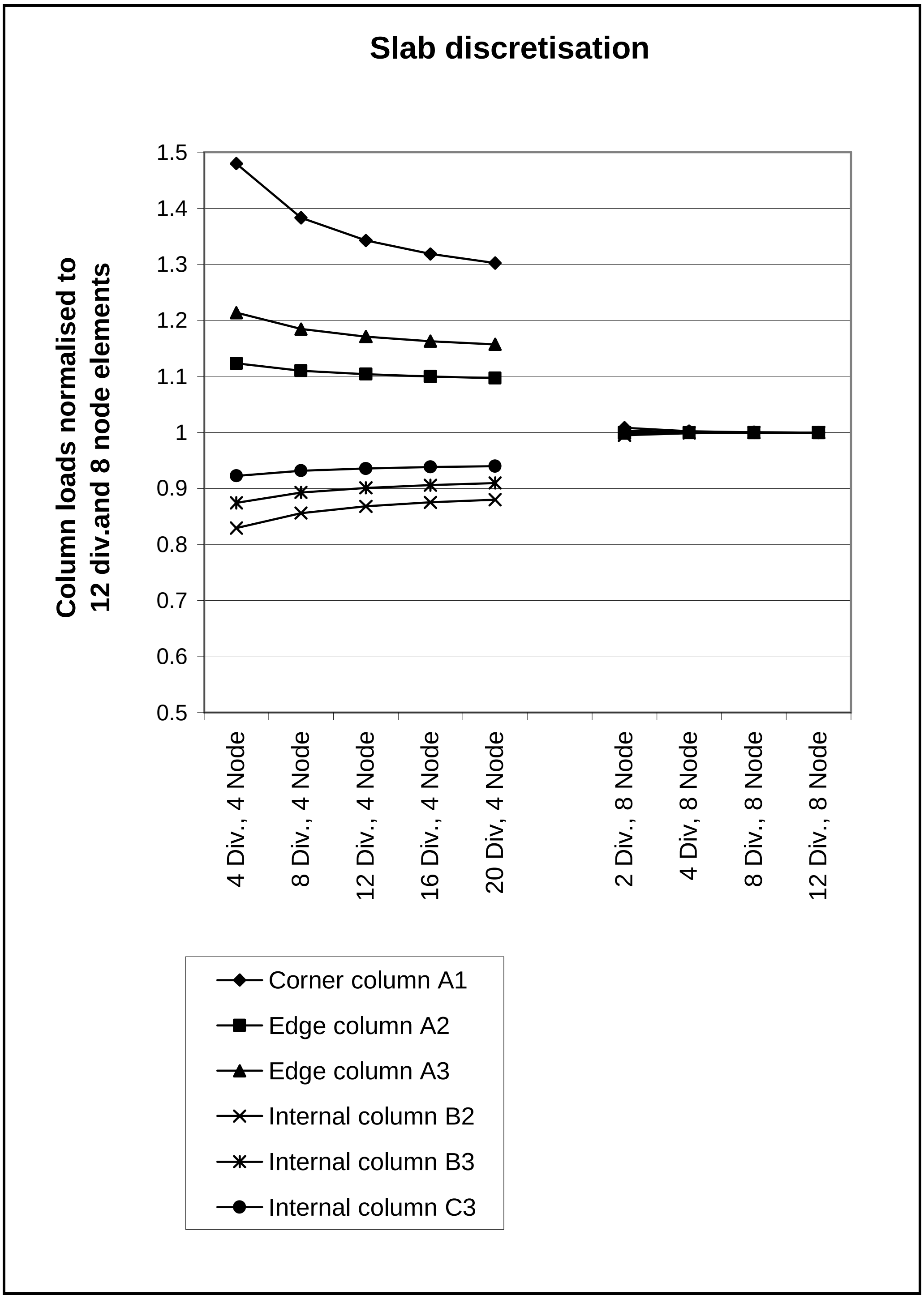


Figure 3.10: **Slab discretisation**

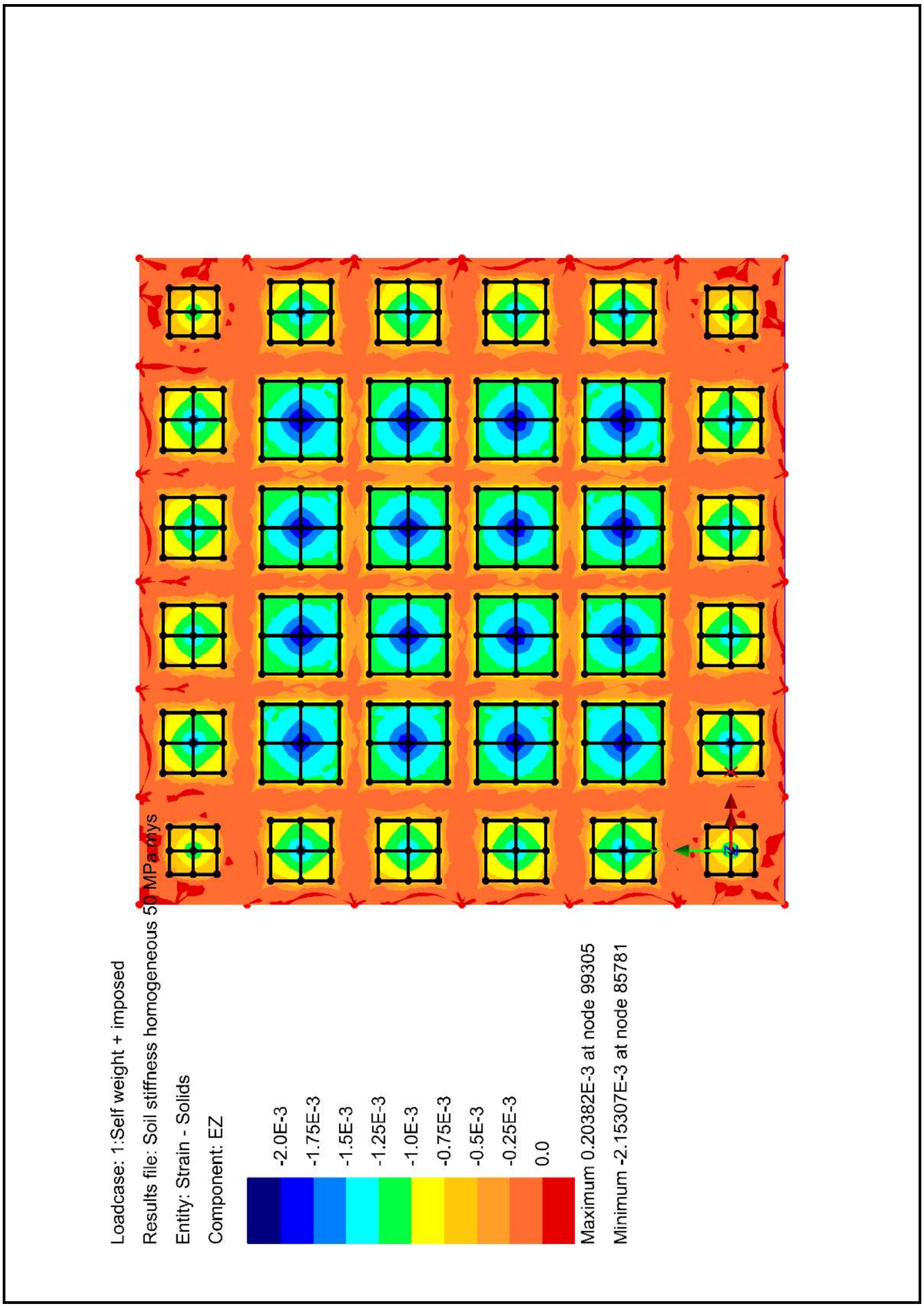


Figure 3.11: Vertical strains under pad foundations for soil with E = 50 MPa

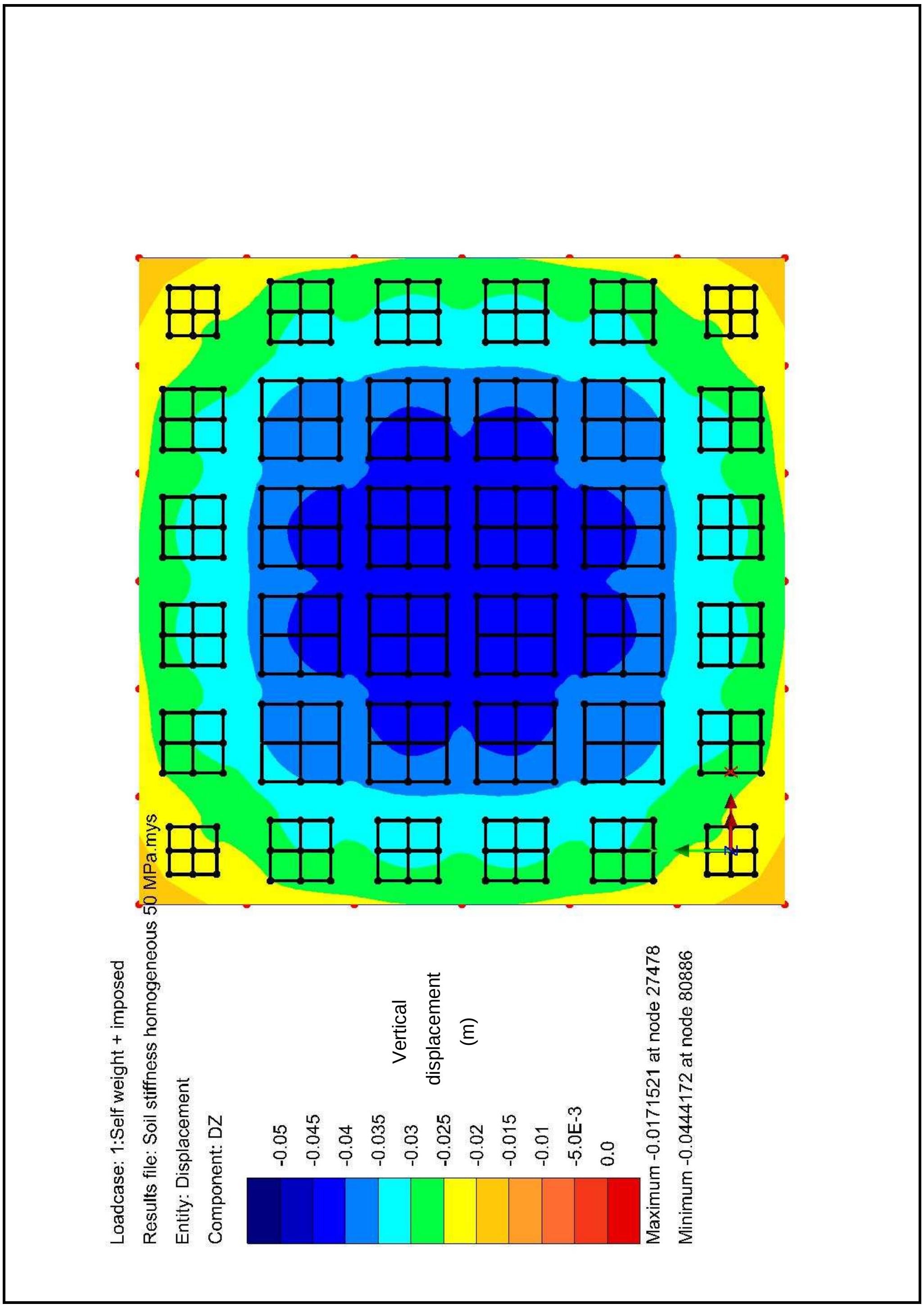


Figure 3.12: **Vertical displacements of halfspace at foundation level**

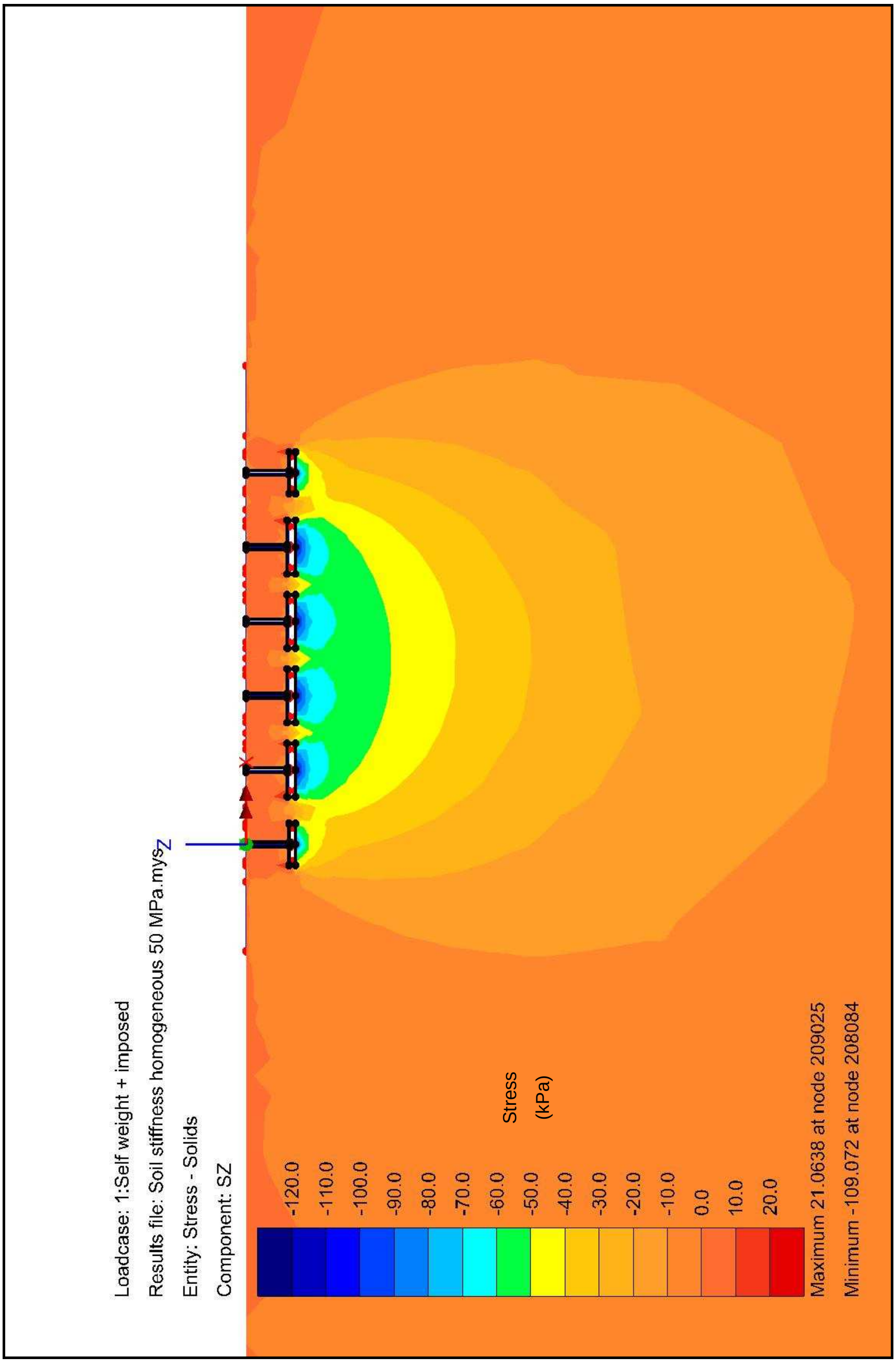


Figure 3.13: Vertical stresses on section through foundations and halfspace

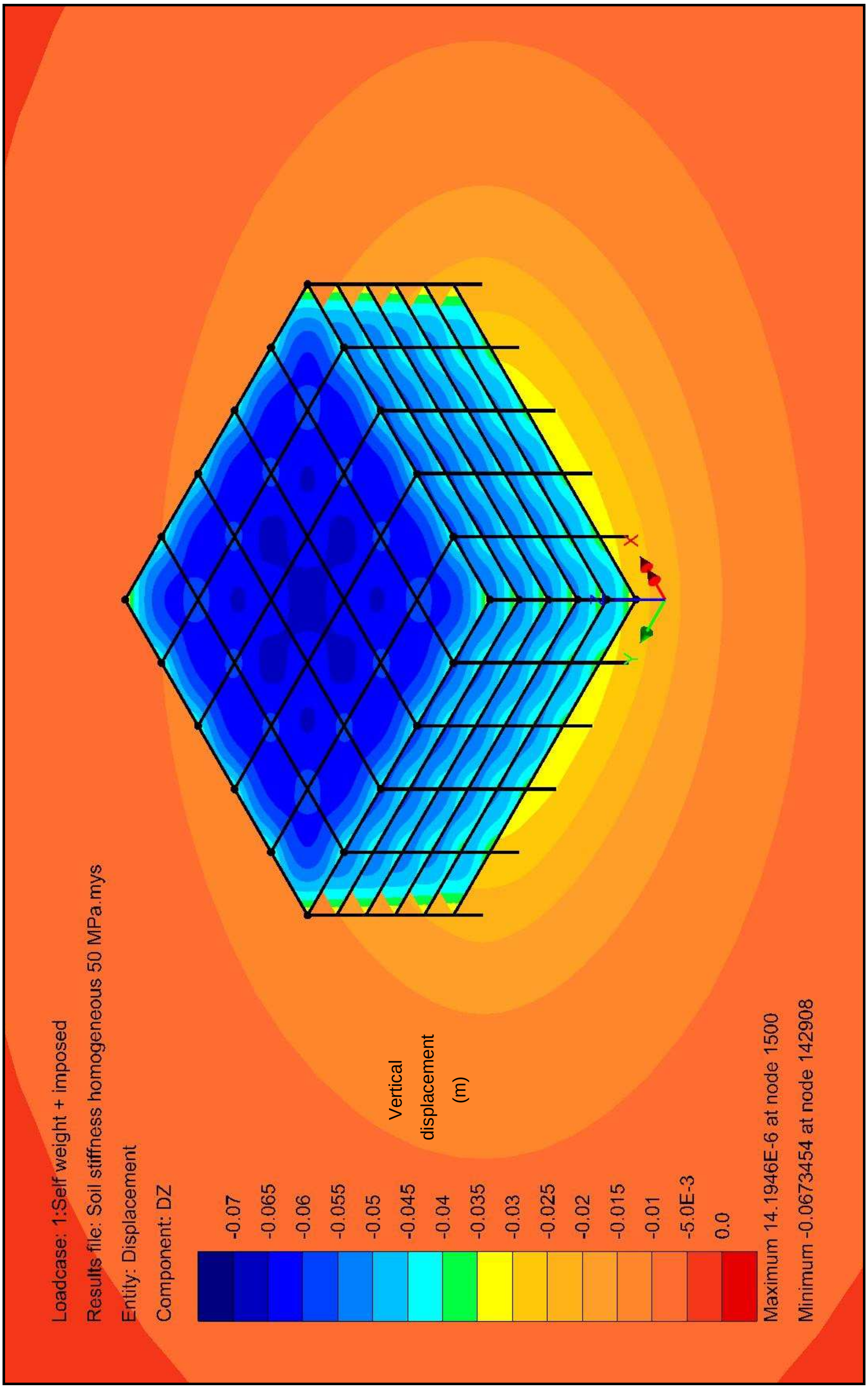


Figure 3.14: **Vertical displacements of superstructure and halfspace surface**

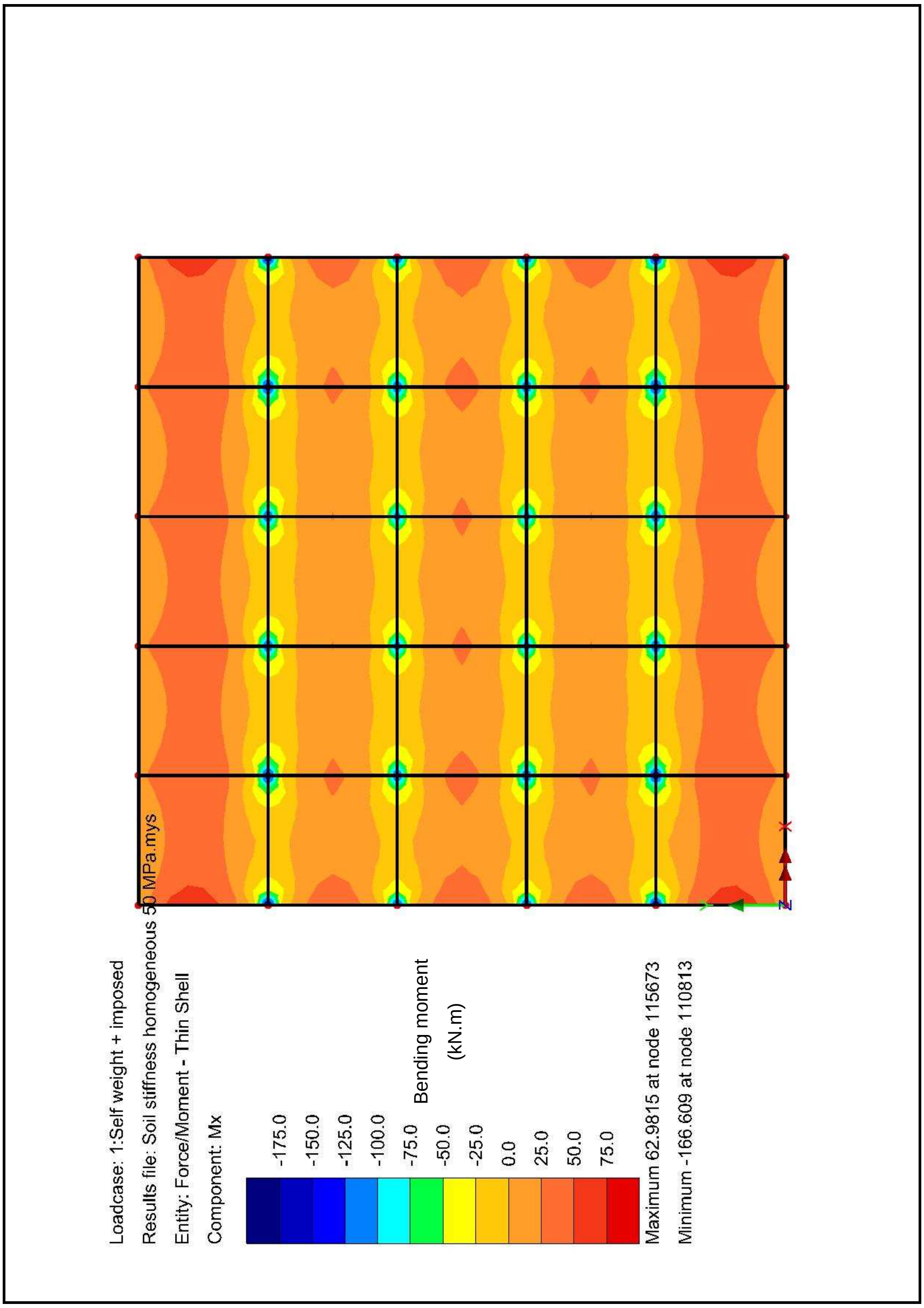


Figure 3.15: Bending moments on first floor slab

4 DISCUSSION OF ANALYSES

This chapter discusses the results from the finite element analyses. Firstly the normalisation of the data is discussed. The normalisation of the data allows for the data of a limited number of models to be applied to a wider range of models, based on the structure's relative bending stiffness. Secondly the loads generated within the model are compared to the strength of the materials to determine possible failure. Thirdly the structural deformation is compared with suggested limits from the literature review. Fourthly the effect of variation in foundation load-displacement response is discussed.

4.1 Normalisation of data

The behaviour of a structure undergoing differential settlement is determined by the relative bending stiffness. The relative bending stiffness depends on the:

- stiffness of the building materials,
- geometry of the building,
- geometry of the foundation; and
- soil stiffness.

By normalising the relative bending stiffness the behaviour of a few specific models can be applied to a wider range of structures, based on the relative bending stiffness.

The magnitudes of column loads at ground level were used as an indicator of the behaviour of the structure. Due to symmetry within the structure only six column loads needed to be used. Figure 4.1 shows the location of the columns. Column A1 is the corner column, A2 and A3 are the two edge columns, B2, B3 and C3 are internal columns with C3 being the nearest to the centre of the structure. Figure 4.2 shows the column loads for the 5 bay structure with an imposed load of 0.5 kN/m² on the floors, an imposed line load on the edges of the floor slabs of 7.241 kN/m representing the facades, a concrete stiffness of 13 GPa and a soil stiffness that ranges from 100 Pa to 1000 GPa. The wide range of soil stiffness is unrealistic for real soils; however it provides valuable insights into the theoretical structural behaviour. From Figure 4.2 it is evident that the column loads at ground level are approximately constant for soil stiffnesses larger than 100 MPa. The column loads at ground level vary for a stiffness range from 0.01 MPa to 100 MPa. For soil stiffnesses lower than 0.01 MPa the column loads are also constant. The change in columns loads and the ultimate strength of the structure are discussed in detail in the following section.

Figure 4.3 shows column loads for the same structure where the stiffness of the concrete in the structure was increased by three orders of magnitude (from 13 GPa to 13 000 GPa). From the results in Figure 4.2 and 4.3 it is evident that an increase of 3 orders of magnitude in structural stiffness is equivalent to a decrease of 3 orders of magnitude in the stiffness of the soil. It is therefore the relative bending stiffness and not the absolute values that determine the behaviour of the structure.

To determine the effect of the geometry of the building on the relative bending stiffness an 'equivalent' single slab with a similar stiffness was calculated. Potts and Addenbrook (1997) have suggested two possible approaches to calculate the stiffness (E_cI) of a structure. The first approach employs the parallel axis theorem to define the structural stiffness about the neutral axis as shown in Equation 4.1:

$$(E_c I)_{Stiffstruct} = E_c \sum_{1}^{n} (I_{slab} + A_{slab} h^2) \qquad \textbf{Equation 4.1}$$

Where n is the number of storeys. Using Equation 4.1 an equivalent slab thickness of 14.0 m was calculated for the model. Finite element analysis carried out, replacing the superstructure with a single 14.0 m thick slab at ground level, with no soil contact, supported by the subsoil columns and foundations, with the same loading as the original structure, showed the replacement slab to be significantly stiffer than the structure. This can therefore be considered to be an overestimate of the building stiffness. Only a rigidly framed structure with bracing would approach such a mode of deformation.

The alternative method was used to obtain the bending stiffness by adding the independent EI values of each slab as shown in Equation 4.2. This implies that the walls and columns transfer the same deformed shape to each storey.

$$(E_c I)_{Flexstruct} = E_c \sum_{1}^{n} I_{slab} \qquad \textbf{Equation 4.2}$$

Using Equation 4.2 an equivalent slab thickness of 532 mm was calculated for the model. The structural bending stiffness based on Equation 4.1 is approximately 18 000 times stiffer than the bending stiffness based on Equation 4.2. Figure 4.4 compares the column loads at ground level for the 532 mm slab and the structure. From the graph it is evident that the stiffness of the single 532 mm slab is a good approximation of the stiffness of the structure.

Based on a plane strain analysis (only) Potts and Addenbrooke (1997) defined relative bending stiffness ρ^* of a building as:

$$\rho^* = \frac{EI}{E_s H^4} \qquad \textbf{Equation 4.3}$$

Where EI is the bending stiffness of the superstructure, E_s is a representative soil stiffness and H is half the width (in the plane of deformation) of the superstructure. From the equation it is evident that for a fixed building and foundation geometry $\rho^* \propto \frac{E}{E_s}$ which coincides with the data given in Figure 4.2 and 4.3. From Figure 4.4 it is evident that EI for this flexible structure without bracing or stiffening due to facades can be calculated with Equation 4.2. To determine the effect of building width a 5 bay x 4 bay and a 5 bay x 3 bay model were analysed. Both models were produced by removing either 1 or 2 of the internal bays of the 5 bay model, which resulted in an identical line load on the edges of the floor slabs and identical edge and corner foundations for the structures.

Figure 4.5 shows the normalised ground level column loads of the 5, 4 and 3 bay structures normalised using ρ^* from Equation 4.3 where H is half the length of the structure. Note that due to the formulation of ρ^* the ‘stiffer soil’ is on the left of the horizontal axis, in contrast to the previous graphs where the stiffer soil is on the right. The column loads were normalised to the column load in the specific column without any soil-structure interaction effect (i.e. founded on an infinitely stiff soil).

Figure 4.5 shows that the corner column loads in the linear elastic finite element model for a ‘rigid’ structure may be up to 5 times greater than for the flexible structure. A basic load takedown to determine column loads could be used for the flexible structure; however a load takedown would underestimate the column loads for the linear elastic ‘rigid’ structure model. It is important to note that the results in Figure 4.5 are based on a linear elastic model. Failure of concrete in the columns, floor slabs or soil failure may reduce the column loads.

Three distinct zones of behaviour within the soil structure stiffness range can be identified within Figure 4.5:

- *Zone 1 'Flexible structure'* is the zone of relative bending stiffness (ρ^*) where the structure is flexible in comparison with the soil. ρ^* is typically less than 1×10^{-4} in *Zone 1*. The structural loads in *Zone 1* can be determined without taking differential settlement into account.
- *Zone 2 'Intermediate structure'* is the intermediate zone where the loads in the edge and corner columns increase and the loads in the internal columns decrease with an increase of relative bending stiffness. ρ^* typically ranges from 1×10^{-4} to 1×10^{-1} in *Zone 2*.
- *Zone 3 'Rigid structure'* is the zone of relative bending stiffness (ρ^*) where the structure is rigid in comparison with the soil. In zone 3 the loads, stresses and differential movements within the structure are constant, independent of the relative bending stiffness. ρ^* is typically larger than 1×10^{-1} in *Zone 3*.

The designed 5 bay structure on a typical London Clay with an undrained shear strength (S_u) of 90 kPa results in an approximate relative bending stiffness (ρ^*) of 2.2×10^{-6}, which falls in Zone 1. This relative bending stiffness was based on a concrete stiffness of 13 GPa, a bending stiffness based on Equation 4.2, i.e. the sum of independent *EI* values of each slab and a soil stiffness of 600 MPa (the soil stiffness are discussed in detail in section 4.2). The 'typical' structure modelled in this thesis by finite element analysis will therefore behave flexibly.

The 'typical' structure modelled in this thesis by way of finite element analysis was modelled without any internal walls or bracing to reduce the complexity. To investigate the effect of a stiffer structure the concrete stiffness was increased by orders of magnitude instead of adding internal walls and bracing.

Internal walls and bracing within structures will increase the bending stiffness of the structure. The stiffness, location and fitment details of the walls and bracing will affect the bending stiffness of the structure. The bending stiffness can be expected to be between the lower bound calculated by Equation 4.2 and the upper bound calculated by Equation 4.1. For the 'typical' structure modelled in this thesis the bending stiffness calculated by Equation 4.2 is approximately 4 orders of magnitude larger than the bending stiffness from Equation 4.1.

4.2 Structural strength

This section discusses the loads within the numerical linear-elastic models and compares them to the strength of the members. The strengths of the members were then compared to the loads from the linear-elastic model to indicate possible concrete or soil failure.

Concrete, reinforcement steel and soil have non-linear stress-strain characteristics. Modelling this behaviour numerically on a full scale structure is complex and requires significant computing power; therefore a simplified linear elastic model was used to model the behaviour of the structure.

Because linear-elastic numerical models were used to determine the effect of differential settlement on the loads within the structure, the maximum load in the model could be infinitely high (depending on the deformation), whereas in a real building the material may fail, limiting the load. Under normal operating conditions the structural members are not intended to be stressed to failure; therefore comparing the load in the linear-elastic model to the strength gives an indication of the performance of the structure.

The structural strength of reinforced concrete members varies and is influenced by the strength of the individual materials and production controls. To ensure adequate member strength the British Standards Institution (BSI 8110-1: 1997) design code recommends the use of partial safety factors for ultimate limit state design. A characteristic material strength is defined as the strength of the material at which less than 5% of all possible test results are expected to fail. The characteristic material strength is then reduced by a partial safety factor which depends on the type of material and application to calculate design strength. For example, the characteristic strength of reinforcement steel is divided by a factor of 1.15 and concrete in flexure or axial load by a factor of 1.50. The same principle applies for loads. Characteristic loads are calculated and increased by a partial safety factor depending on the load type and application. In the design, the design load should be less than the design strength, to ensure a safe structure. Under normal operating conditions the actual load will be significantly less than the design load and the element strength will be higher than the design strength.

The loads in the numerical models are based on expected loads and not ultimate design loads and are therefore significantly lower. The imposed live load on the model is 0.5 kN/m² on all the floors with no roof load. In contrast the ultimate design live load for the floor slabs ranges from 0 to 4.0 kN/m² depending on the load combination. Dead loads on the model are the characteristic loads i.e. a partial safety factor of 1.0 in contrast to the design dead load where a partial safety factor of 1.0 to 1.4 is used depending on the load combination. The loads in the structure under normal operating conditions should therefore be significantly lower than the strength of the members.

The following sections will compare the column loads, foundation loads and bending moments in the slabs from the finite element models with the strength of the members.

4.2.1 Column loads

Column strength depends on the geometric and material properties of the column. To simplify the design, the same column dimensions (450 mm x 450 mm) were used throughout the structure. The column design was based on the maximum load which occurred at an internal column at ground level. This column was used to calculate the column strength for comparison. The column was a 450 mm x 450 mm reinforced column with 9 Y40 reinforcement bars (3 on each side) with 40 mm cover. A 30 MPa actual strength of the concrete was assumed. In a structure the actual concrete strength will vary and will depend on site quality control.

A concrete column fails due to a critical combination of axial load and biaxial bending. To calculate and present this 3-dimensional envelope is complex and therefore the simplification as suggested in BSI 8110-1 (1997) was used. The code suggests the use of the following equations to compare biaxial bending to uniaxial bending:

For $M_x / h' \geq M_y / b', M_x' = M_x + \beta \frac{h'}{b'} M_y$ **Equation 4.4**

For $M_x / h' < M_y / b', M_y' = M_y + \beta \frac{h'}{b'} M_x$ **Equation 4.5**

Where *h'* and *b'* are the depth of the reinforcing steel and β is a coefficient based on the axial force on the column, the dimensions and concrete strength.

The uniaxial column strengths under combined axial load and bending moments are shown in Figure 4.6. Table 4.1 shows the axial column loads with the equivalent uniaxial bending moment derived from the biaxial bending moments and the column loads from the model using Equations 4.4 and 4.5. As the relative bending stiffness increased in the linear elastic model the axial loads in the corner columns increased by approximately 5 times, the loads in the edge columns approximately doubled and the loads in the internal columns reduced to approximately 1/3 in comparison to the column loads within a flexible structure. The cells highlighted in grey show where the loads from the linear elastic numerical model exceed the strength of the column. These column loads are based on a linear elastic model, i.e. no slab or foundation failure occurs. Yielding of foundations may protect the structure from damage to the columns. Foundation failure is discussed in section 4.2.2. Slab failure is discussed in Section 4.2.3. From Table 4.1 it is evident that (without foundation or slab failure) column failure may occur for a relative bending stiffness (ρ*) higher than approximately 1.32 x 10^{-3} which is in Zone 2, intermediate stiffness. If the columns were sized according to the expected loads (i.e. a corner column smaller than an internal column), failure in the corner and edge columns may occur at a lower relative bending stiffness. Column failure will result in moment redistribution within the structure or instability of the structure.

4.2.2 Foundation loads

Foundation loads depend on the load-displacement response and the displacement of the foundation. This section firstly discusses foundation loads with a linear foundation load-displacement response and secondly foundation loads with a non-linear load foundation response.

Figure 4.7 shows the foundation loads (not column loads at ground level) from the linear-elastic model normalised to relative bending stiffness (ρ*). The foundation loads are constant in Zone 1, Flexible structure (ρ*<1 x 10^{-4}). In Zone 2, Intermediate stiffness (1 x 10^{-4} < ρ*<1 x 10^{-1}) the loads in the corner foundation (A1) and edge foundations (A2 and A3) increase while the loads in the internal foundations (B2, B3 and C3) decrease. In Zone 3, Rigid structure (ρ*>1 x 10^{-1}) the foundation loads are constant again. The load in the corner foundation is approximately 2.5 times greater in Zone 3 than in Zone 1; however these values are based on foundations on a linear elastic soil resulting in linear load-displacement response. Foundation failure (non-linear load-displacement response) may reduce the loads in the foundations.

To determine the effect of non-linear load-displacement response a 'non-linear' model was analysed which consisted of changing the stiffness under foundations according to the settlement. For the 'non-linear' model, a structure with a relative bending stiffness (ρ^*) of 1.32 x 10^{-1} (Zone 3, Rigid structure) was used to evaluate the effect of non-linear load-displacement response. In Zone 3 the structure is rigid in comparison with the soil and foundation failure may be expected. ρ^* is a function of both the soil and superstructure stiffness and for the model, firstly an appropriate soil stiffness and secondly the superstructure stiffness were determined for a relative bending stiffness (ρ^*) of 1.32 x 10^{-1}.

Soil stiffness is a function of the strain within as well as the strength of the soil (Atkinson, 2000). Figure 4.8 (Atkinson, 2000) shows the relationship between the soil secant stiffness divided by initial soil secant stiffness vs. the foundation settlement divided by the foundation width. The graph was based on data from settlement of foundations on London Clay. To determine a stiffness profile for the model, a tangent stiffness was chosen for each interval of ρ/B and the corresponding secant stiffness was calculated (shown on Figure 4.9). The input tangent stiffnesses were changed until the calculated secant stiffness corresponded with the secant stiffness degradation as suggested by Atkinson (2000). With the tangent stiffness, ρ/B intervals and an initial small strain soil stiffness a stress strain curve was calculated. Soil stiffness is related to the soil strength (Atkinson, 2000), therefore the initial soil stiffness must be based on the soil strength. For the designed structure an undrained soil shear strength (C_u) of 90 kPa was used. The following equation (Eurocode 7) gives the ultimate bearing stress for a pad foundation:

$$Q_{ult} = (\pi + 2)1.2C_u + q \qquad \textbf{Equation 4.6}$$

Where Q_{ult} is the ultimate bearing stress, C_u the undrained shear strength, and q the surcharge. Assuming the surcharge (q) is approximately equal to the self weight of the foundation and the soil above it, Q_{ult} = 6.17 C_u. For an undrained shear strength of 90 kPa, the ultimate bearing capacity is therefore 555 kPa. Figure 4.10 shows the applied stress vs. settlement graph based on Atkinson's stiffness degradation curve and an initial small strain stiffness of 600 MPa. The graph shows an initial small strain stiffness of 600 MPa, which relates to an ultimate bearing stress of approximately 555 kPa.

For a relative bending stiffness (ρ^*) of 1.32 x 10^{-1} and a soil stiffness of 600 MPa, a structural stiffness (E_cI) of 9.79 x 10^9 kN.m^2 was needed. For the required structural stiffness, the concrete stiffness (E_c) was increased instead of the second moment of inertia (I). This resulted in an unrealistically high concrete stiffness (780 000 GPa), however it is easier to change the concrete stiffness than the geometry, and it has the same effect on the structural stiffness. The high concrete stiffness (780 000 GPa) was assigned to the superstructure (slabs and columns) and the foundations were assigned a normal concrete stiffness of 13 GPa.

To analyse the 'non-linear' model a concrete stiffness of 780 000 GPa and an initial soil stiffness of 600 MPa were used (which relates to relative bending stiffness of (ρ^*) of 1.32×10^{-1} as shown above). Below each foundation a soil block was defined to which an individual stiffness was assigned. The soil blocks were 7.5 m x 7.5 m wide and 10.4 m deep (5.4 m below foundation level) to coincide with the mesh boundaries. For the first iteration 10% of the structural load was assigned to the structure and a 600 MPa soil stiffness was assigned to the blocks and the surrounding soil. The settlement of each foundation was extracted from the model and a new soil stiffness calculated based from the stiffness degradation as shown in Figure 4.8. The calculated stiffnesses were assigned to each soil block after which the model was analysed again. These steps were repeated until the change in stiffness in each block was less than 0.1 MPa, after which the load was increased by 10% and the process repeated. Table 4.2 shows the soil stiffness degradation with each iteration for the model with a concrete stiffness of 780 000 GPa for the superstructure and 13 GPa for the foundations. Table 4.3 shows the soil stiffness degradation with each iteration for the model with a concrete stiffness of 780 000 GPa for both the superstructure and the foundations. The final soil stiffnesses at 100% load for the model with the 13 GPa foundations were between 75 MPa and 100 MPa (between 0.11% and 0.20% strain) and for the model with 780 000 GPa foundations between 94 MPa and 123 MPa (between 0.07% and 0.13% strain). The stiffer foundation resulted in less soil stiffness degradation.

Table 4.4 shows the column loads of four models with the same relative bending stiffness ($\rho^* = 1.32 \times 10^{-1}$) namely:

- The 'non-linear' model described above with a concrete stiffness of 780 000 GPa for the superstructure and 13 GPa for the foundations.
- A linear model with a concrete stiffness of 780 000 GPa for the superstructure and 13 GPa for the foundations.
- The 'non-linear' model as described above with a concrete stiffness of 780 000 GPa for both the superstructure and the foundations.

- A linear model with a concrete stiffness of 780 000 GPa for both the superstructure and the foundations.

The results show that the stiffness of the foundations has an influence on the column loads:

- The load in the corner column (A1) was reduced by 14% in the non-linear model with a 13 GPa foundation concrete stiffness and 34% in the non-linear model with a 780 000 GPa concrete stiffness.
- The loads in the edge columns (A2, A3) were reduced by between 1% and 4% in the non-linear model with a 13 GPa foundation concrete stiffness and 11% in the non-linear model with a 780 000 GPa concrete stiffness.
- The loads in the internal columns (B2, B3, C3) ranged from a reduction of 5% to an increase of 9% in the non-linear model with a 13 GPa foundation concrete stiffness and increase of between 56% and 65% in the non-linear model with a 780 000 GPa concrete stiffness.

It is evident that the foundation and soil stiffness have a significant effect on the column loads and yielding of foundations may protect the structure against column failure.

4.2.3 Slab bending moment

For a 'rigid' structure (i.e. in Zone 3) redistribution of column loads may be possible by bending of the slabs. Increased bending moments resulting in column load redistribution may lead to stresses which exceed the available strength leading to excessive deformation, shear failure or the formation of plastic hinges and structural damage. It is therefore important to check the implied bending stresses for the finite element analysis with the actual strength of the concrete.

For the design of the structure a simplified 2D frame analysis was used to determine the design bending moments in the slabs. These design bending moments were compared to the bending moments in the slabs calculated by the linear elastic 3D finite element analysis to determine possible failure of the structure. To compare the bending moment of the 3D finite element model with those of the design 2D frame, the 3D model was divided into a series of 2D frames. The frames are shown in Figure 4.11. The bending moment for the frame was calculated by averaging the bending moments of the integration points for each row of elements perpendicular to the 2D plane.

Figures 4.12 and 4.13 compare the bending moment in the first floor level slab from the linear elastic finite element analysis model with the design strength envelope. Figure 4.12 is for an internal span and Figure 4.13 for an edge span. From Figure 4.12 and 4.13 it is evident that the bending moments in a structure with a relative bending stiffness larger than 1.32 x 10^{-4} fall outside the ultimate limit state load envelope, and that cracking with load redistribution will occur within the slab which may lead to failure of the structure. Yielding of foundations may prohibit slab failure. Slab failure depends on the geometry and the load redistribution within the structure, however it is most likely that the slabs will firstly fail due to unacceptable deflection, secondly collapse due to shear failure (the plastic deformation of the reinforcing and the opening of cracks reduce the shear resistance) and lastly collapse due to the development of enough plastic hinges for the structure to become unstable.

4.3 Structural deformation

Excessive deformation within a structure may have an impact on the serviceability of the structure. The following two sections compare the deformation within the finite element model with guidelines from the literature.

4.3.1 Tilt

Excessive tilt in buildings has an impact on visual appearance and may therefore be a limiting criterion with respect to differential settlement. The British Standards Institution (BSI 8110-2: 1985) provides guidelines on deflection limits; however no guidance is given for tilt. Burland *et al.* (2001a) note that a deviation of slope, from the horizontal or vertical, of more than 1:100 will normally be clearly visible and overall deviations in excess of 1:150 are undesirable.

Table 4.5 shows the maximum column tilt within each structure with respect to the structure's relative bending stiffness (ρ^*). The first part of the table is for the structure with a concrete stiffness of 13 GPa on a soil with a stiffness ranging from 1 x 10^{-4} to 1 x 10^{4} MPa. The second part of the table is for the structure with a concrete stiffness of 13 000 GPa on the same soil stiffness range. A concrete stiffness of 13 000 GPa is unrealistic, however it simulates the effect of a stiffer structure as would be the case with bracing and internal walls. From the results in Table 4.5 it is evident that the maximum tilt does not relate to relative bending stiffness; it is a function of both the actual soil stiffness and the structure stiffness. A decrease in soil stiffness results in an increase of tilt. A decrease of structural stiffness also results in an increase of tilt; therefore a flexible structure is more likely to suffer from tilt.

If a maximum acceptable tilt of 1:100 is assumed (i.e. 30 mm movement on a 3 m column), the structure with a 13 GPa concrete stiffness on a soil stiffness softer than 10 MPa may experience excessive tilt. The structure with a 13 000 GPa concrete stiffness on a soil stiffness softer than 1 MPa may experience excessive tilt.

Spatial variation in soil stiffness may cause more tilt than what will occur on a homogeneous soil. The above data is based on homogeneous soil stiffness.

4.3.2 Deflection

Excessive deflection has an impact on the performance of a building. Deflection within a building is caused by deflection within each slab or by the deflection of multiple spans due to column moment. Deflection of individual slabs is usually decreased by an increase in the stiffness (thickness) of the slab.

The British Standards Institution (BSI 8110-2: 1985) recommends a deflection limit of 1:250 for visual appearances. To limit damage to non-structural elements (unless they are specifically detailed to allow for deflections and then the 1:250 visual limit will be applicable) the British Standards Institution recommends the following limits:

- 1:500 or 20 mm, whichever is the lesser, for brittle materials; and
- 1:350 or 20 mm, whichever is the lesser, for non brittle partitions or finishes.

The literature review suggested the following angular distortion (not deflection) limits:

- 1:150 to 1:300 for structural damage of frames without cladding,
- 1:300 to 1:1 000 for facade cracking of frames with non load bearing walls; and,
- 1:300 to 1:2 000 for damage to load bearing walls.

The type of structure analysed was an open plan office with external facades detailed to allow for deflections. Therefore the angular distortion guideline of 1:150 to 1:300 would be considered acceptable. Assuming angular distortion guidelines do not include any tilt of the building, the deflection ratio is twice the angular distortion, i.e.

1:300 to 1:600. These values are stricter than the 1:250 suggested by the British Standards.

Table 4.6 shows the deflection of sections through three adjacent foundations parallel to the facade against soil and structure stiffness. Table 4.7 shows the deflection of sections diagonal to the facade. Positive values show a sagging deflection and negative values a hogging deflection. The first part of the tables is for the structure with a concrete stiffness of 13 GPa on a soil with a stiffness ranging from 1 x 10^{-4} to 1 x 10^{4} MPa. The second part of the table is for the structure with a concrete stiffness of 13 000 GPa on the same soil stiffness range. A concrete stiffness of 13 000 GPa is unrealistic, however it simulates the effect of a stiffer structure as would be the case with bracing and internal walls.

From the results in Table 4.6 and Table 4.7 it is evident that the deflection ratio at ground level does not relate to relative bending stiffness; it is a function of both the actual soil stiffness and the structure stiffness. A decrease in soil stiffness generally results in an increase in deflection. A decrease of structural stiffness also generally results in an increase in deflection; therefore a flexible structure is more likely to suffer from excessive deflection.

For the structure with a concrete stiffness of 13 GPa unacceptable deflection is likely to occur on soil stiffnesses softer than 10 MPa. For the stiffer structure with a concrete stiffness of 13 000 GPa unacceptable deflection is likely to occur on soil stiffnesses softer than 0.01 MPa.

It is interesting to note that in the model with a concrete stiffness of 13 GPa and a soil stiffness of 1 MPa the maximum deflection on the edge is 1:230, on the internal section parallel to the edge it is less (1:183) than for the edge section and on the diagonal section it is also less (1:134) than for the edge section. For the model with a concrete stiffness of 13 000 GPa and a soil stiffness of 0.001 MPa the maximum deflection on the edge is 1:702 and for the internal section parallel to the edge it is less (1:318). These values show that the measurement of deflection on the facade of the building (i.e. assuming that the building deformation is primarily in 2D) will result in an underestimate of the maximum deflection. It shows the necessity to take into account the 3D behaviour when analysing for differential settlement damage.

Burland *et al.* (2001a) suggested the use of a simplified beam model to determine the deflection limit at which damage will occur. They used a strain limit which is linked to a damage category as discussed in the literature review. This method is aimed at structures with masonry load bearing walls and is not necessarily applicable to modern open plan offices. However it is interesting to see how it compares to modern open plan offices.

For an isotropic beam (E/G = 2.6) with the neutral axis at the bottom and an L/H ratio of 2 (based on the structure's geometry) the following deflection/strain ratio limit applies (from Figure 2:22):

$$\frac{\Delta/L}{\varepsilon_{\text{lim}}} = 0.82 \qquad \textbf{Equation 4.7}$$

Table 4.8 shows the deflection limits against damage category, based on Equation 4.7. Refer to Table 2.13 in the literature review for the damage category description. The last column in Table 4.6 and 4.7 shows the damage category according to Burland *et al.* (2001a) guidelines. It is evident that the stiffer structure (E_c = 13 000 GPa) is more resistant to excessive deflections.

4.4 Variation in foundation-load displacement response

The previous models which have been discussed were founded on a halfspace with homogeneous soil stiffness. Buildings are often founded on soils with a spatial variation of stiffness which will cause different foundation load-displacement responses for similar foundations. This section summarises from the literature the spatial variation of soil stiffness, the load-displacement response of piles and the effect it has on a structure.

Phoon and Kulhawy (1999) showed that the coefficient of variation (COV) for undrained shear strength may be up to 80%; however the typical mean COV for the groups range from 22% to 33% (Table 2.2). It is therefore expected that the soil stiffness may also vary in a similar order of magnitude. Phoon and Kulhawy also showed that the horizontal fluctuation of soil properties may be as short as 3 m, therefore it is possible to have different soil conditions under adjacent foundations. Soft soils may cause excessive foundation settlement and hard spots beneath foundations may cause a stiffer load-displacement response of the foundation.

Reused foundations often have a stiffer load-displacement response due to the preloading from the previous structure. Whitaker and Cooke (1966) showed that the reload load-displacement response of a pile may be ten times stiffer than the virgin load-displacement response.

Soil stiffness can vary significantly under a structure and to fully investigate the effect of soil variation on a structure a significant number of models need to be analysed. Due to limited resources only two sets of models were modelled.

The first set consisted of a superstructure with a concrete stiffness of 13 GPa on a soil with a 10 MPa stiffness ($\rho^* = 1.32 \times 10^{-4}$). The soil stiffness under foundation B2 (see the figure in Table 4.6 for the location of B2) was changed to 0, 1, 5, 10, 20 and 100 MPa. As discussed in the previous paragraphs the typical COV of the soil's stiffness will be approximately 22% to 33% with an extreme case being 80%. The extreme case correlates with 2 MPa and 18 MPa soil stiffness for this model. The reuse of foundations combined with new (virgin) foundations however may result in a more significant variation of load-displacement stiffness response of up to 10 times. This correlates to a 1 MPa and 100 MPa soil stiffness for this specific model.

The second set consisted of a superstructure with a concrete stiffness of 13 GPa on a soil with a 100 MPa stiffness ($\rho^* = 1.32 \times 10^{-5}$). The soil stiffness under foundation B2 was changed to 0, 10, 50, 100, 200 and 1 000 MPa. To change the stiffness below foundation B2 a soil block which surrounded the foundation was defined to which the new stiffness was assigned. The block was 7.5 m x 7.5 m wide (equal to the column spacing) and 10.4 m deep (5.4 m below foundation level). The value of 10.4 m was chosen because it coincided with a mesh boundary.

Table 4.9 shows the change in column loads at ground level compared to those calculated for the models with homogeneous soil stiffnesses.

On the 10 MPa model ($\rho^* = 1.32 \times 10^{-4}$) with 1 MPa stiffness below foundation B2 the loads in the adjacent columns increased by either 36 % (internal columns) or 42 % (edge columns) and the load in column B2 decreased by 112%, which is more than 100%, a tensile stress due to the foundation and the soil above the foundation being supported by the superstructure and not the soil below it. Increasing the soil stiffness under foundation B2 to 100 MPa increased the load on foundation B2 by 14 %. From the results it is evident that not only softer ‘failed’ foundations, but also a harder spot under a foundation (or stiffer foundation load-displacement response due to a reused foundation), may have an effect on structural loads within a structure.

On the 100 MPa model ($\rho^* = 1.32 \times 10^{-5}$) with 10 MPa stiffness below foundation B2 the loads in the adjacent columns increased by either 5 % (internal columns) or 7 % (edge columns) and the load in column B2 decreased by 14%. Increasing the soil stiffness under foundation B2 to 1 000 MPa increased the load on foundation B2 by 4 %. The results showed that the higher the overall stiffness of the soil, the less impact a stiffer load-displacement response of foundation has on the structure.

Table 4.1: Predicted column failure at ground level due to axial load and bending moment

	Zone 1 Flexible Structure			Zone 2 Intermediate stiffness			Zone 3 'Rigid' structure		
ρ*	**1.32e-7**	**1.32e-6**	**1.32e-5**	**1.32e-4**	**1.32e-3**	**1.32e-2**	**1.32e-1**	**1.32e+0**	**1.32e+1**
Axial load in A1 (kN)	778	778	781	799	1086	2766	3931	4104	4122
Bending moment in A1 (kN.m)	0	0	4	37	334	1870	4508	5534	5667
Axial load in A2 (kN)	1596	1597	1605	1661	1921	2587	2954	3006	3011
Bending moment in A2 (kN.m)	0	0	3	35	316	1991	5406	6833	7035
Axial load in A3 (kN)	1438	1441	1456	1573	2155	2691	2722	2717	2716
Bending moment in A3 (kN.m)	0	0	3	30	274	1777	4935	6225	6404
Axial load in B2 (kN)	3310	3303	3267	3047	2251	1342	987	943	939
Bending moment in B2 (kN.m)	0	0	3	26	264	1730	3763	3918	3895
Axial load in column B3 (kN)	3020	3019	3007	2912	2414	1307	901	834	827
Bending moment in B3 (kN.m)	0	0	2	23	210	1262	2522	2453	2406
Axial load in C3 (kN)	2733	2735	2747	2792	2615	1473	860	773	763
Bending moment in C3 (kN.m)	0	0	1	10	93	582	1015	845	805

Legend	Predicted column failure

Table 4.2: Soil stiffness degradation (780 000 GPa superstructure, 13 GPa foundations)

Iteration	Load on model (%)	Soil stiffness under foundation (MPa)					
		A1 (Corner)	A2 (Edge)	A3 (Edge)	B2 (Internal)	B3 (Internal)	C3 (Internal)
1	10	600.0	600.0	600.0	600.0	600.0	600.0
2	10	293.7	319.0	318.7	347.7	347.5	347.3
3	10	278.1	305.4	305.4	333.2	333.2	333.3
4	10	277.1	304.2	304.3	331.9	332.0	332.1
5	20	277.0	304.1	304.2	331.9	332.0	332.0
6	20	204.1	229.3	229.3	255.2	255.3	255.4
7	20	198.3	221.4	221.6	247.0	247.3	247.5
8	20	197.4	220.1	220.4	245.8	246.1	246.3
9	30	197.4	220.2	220.4	245.8	246.1	246.4
10	30	163.9	183.1	183.3	204.6	204.9	205.1
11	30	159.6	177.7	178.1	200.3	200.6	200.9
12	30	159.0	177.0	177.3	199.7	200.0	200.3
13	40	158.9	176.9	177.2	199.6	199.9	200.2
14	40	138.7	155.4	155.6	174.6	174.9	175.2
15	40	135.8	152.8	153.1	171.8	172.1	172.5
16	40	135.4	152.5	152.8	171.4	171.8	172.1
17	50	135.4	152.4	152.8	171.4	171.7	172.1
18	50	120.9	136.9	137.2	154.5	154.8	155.1
19	50	118.8	134.4	134.8	152.3	152.7	153.0
20	50	118.5	134.0	134.4	151.9	152.3	152.7
21	60	118.4	133.9	134.3	151.9	152.3	152.7
22	60	107.8	122.1	122.4	139.1	139.5	139.9
23	60	106.1	120.3	120.7	137.0	137.4	137.9
24	60	105.9	120.0	120.4	136.6	137.1	137.6
25	70	105.8	119.9	120.3	136.6	137.0	137.5
26	70	96.9	110.6	110.9	125.6	126.0	126.4
27	70	95.7	109.1	109.5	124.1	124.5	124.9
28	70	95.5	108.9	109.2	123.8	124.2	124.7
29	80	95.5	108.8	109.2	123.8	124.2	124.6
30	80	89.8	101.5	102.0	116.0	116.5	117.0
31	80	88.7	100.1	100.5	114.5	115.0	115.5
32	80	88.4	99.8	100.2	114.2	114.7	115.2
33	90	88.4	99.7	100.2	114.1	114.6	115.2
34	90	82.5	94.1	94.4	107.6	108.0	108.5
35	90	81.3	93.1	93.5	106.4	106.9	107.3
36	90	81.0	92.9	93.3	106.2	106.7	107.1
37	100	81.0	92.9	93.3	106.1	106.6	107.1
38	100	76.4	88.3	88.7	100.3	100.8	101.3
39	100	75.7	87.3	87.7	99.0	99.6	100.2
40	100	75.6	87.1	87.5	98.7	99.3	99.9
41	100	75.5	87.0	87.5	98.7	99.3	99.8

Table 4.3: **Soil stiffness degradation (780 000 GPa superstructure and foundations)**

Iteration	Load on model (%)	Soil stiffness under foundation (MPa)					
		A1 (Corner)	A2 (Edge)	A3 (Edge)	B2 (Internal)	B3 (Internal)	C3 (Internal)
1	10	600.0	600.0	600.0	600.0	600.0	600.0
2	10	335.0	360.8	360.0	388.7	387.9	387.2
3	10	322.6	349.4	349.0	376.6	376.2	375.9
4	10	321.6	348.5	348.2	375.7	375.3	375.0
5	20	321.5	348.5	348.1	375.6	375.2	375.0
6	20	245.3	271.8	271.5	298.6	298.3	298.1
7	20	238.4	265.1	264.9	291.5	291.4	291.4
8	20	237.7	264.3	264.2	290.7	290.6	290.6
9	30	237.7	264.2	264.1	290.6	290.5	290.5
10	30	198.2	220.7	220.6	245.9	245.9	245.8
11	30	194.0	215.1	215.2	240.3	240.4	240.5
12	30	193.4	214.5	214.5	239.7	239.8	239.9
13	40	193.3	214.4	214.5	239.6	239.7	239.9
14	40	168.9	189.1	189.2	210.8	210.9	211.1
15	40	165.7	185.3	185.5	206.8	207.1	207.3
16	40	165.2	184.7	184.9	206.1	206.4	206.7
17	50	165.1	184.6	184.8	206.0	206.3	206.6
18	50	149.1	166.4	166.6	187.3	187.5	187.8
19	50	146.6	163.8	164.1	184.0	184.4	184.8
20	50	146.2	163.4	163.6	183.5	183.9	184.3
21	60	146.1	163.3	163.6	183.4	183.8	184.2
22	60	133.0	150.2	150.4	168.6	168.9	169.3
23	60	130.6	148.1	148.4	166.3	166.7	167.1
24	60	130.2	147.7	148.0	166.0	166.3	166.7
25	70	130.1	147.6	148.0	165.9	166.3	166.7
26	70	120.8	136.6	137.0	154.2	154.5	154.8
27	70	119.0	134.6	135.0	152.4	152.8	153.2
28	70	118.7	134.2	134.6	152.1	152.5	152.9
29	80	118.6	134.1	134.5	152.0	152.4	152.8
30	80	110.5	124.9	125.2	143.1	143.3	143.7
31	80	109.1	123.4	123.8	140.8	141.4	141.9
32	80	108.8	123.1	123.4	140.4	140.9	141.5
33	90	108.8	123.1	123.4	140.4	140.9	141.5
34	90	102.4	116.2	116.6	132.0	132.5	133.1
35	90	100.9	114.6	115.1	130.2	130.8	131.4
36	90	100.6	114.3	114.8	129.8	130.4	131.0
37	100	100.5	114.2	114.7	129.7	130.3	131.0
38	100	95.2	108.4	108.8	123.3	123.7	124.2
39	100	94.2	107.2	107.6	122.1	122.6	123.1
40	100	94.0	107.0	107.4	121.8	122.3	122.8
41	100	94.0	106.9	107.3	121.8	122.3	122.8

Table 4.4: Column loads in linear and non-linear models

	Model 1	Model 2	Model 3	Model 4
Type	Non-linear	Linear	Non-Linear	Linear
Superstructure concrete stiffness (GPa)	780 000	780 000	780 000	780 000
Foundations concrete stiffness (GPa)	13	13	780 000	780 000
	Column load (kN)			
A1 (Corner)	1863	2158	2595	3931
A2 (Edge)	2210	2292	2628	2954
A3 (Edge)	2091	2118	2424	2722
B2 (Internal)	2160	2285	1560	986
B3 (Internal)	2033	1988	1481	900
C3 (Internal)	1837	1692	1339	860
	Change with respect to model 2 (%)		**Change with respect to model 4 (%)**	
A1 (Corner)	-14	0	-34	0
A2 (Edge)	-4	0	-11	0
A3 (Edge)	-1	0	-11	0
B2 (Internal)	-5	0	58	0
B3 (Internal)	2	0	65	0
C3 (Internal)	9	0	56	0

Table 4.5: Column tilt in structure

E_s (MPa)	E_c * (GPa)	Relative bending stiffness (ρ*)	Maximum column tilt in structure
		Zone 1, Flexible structure	
1.00×10^{4}	13	1.32×10^{-7}	1:130 064
1.00×10^{3}	13	1.32×10^{-6}	1:50 884
1.00×10^{2}	13	1.32×10^{-5}	1:6 905
		Zone 2, Intermediate stiffness	
1.00×10^{1}	13	1.32×10^{-4}	1:707
1.00×10^{0}	13	1.32×10^{-3}	1:76
1.00×10^{-1}	13	1.32×10^{-2}	1:11
		Zone 3, Rigid structure	
1.00×10^{-2}	13	1.32×10^{-1}	1:3
1.00×10^{-3}	13	1.32×10^{0}	1:0.5
1.00×10^{-4}	13	1.32×10^{1}	1:0.05
		Zone 2, Intermediate stiffness	
1.00×10^{4}	13 000	1.32×10^{-4}	1:714 420
1.00×10^{3}	13 000	1.32×10^{-3}	1:76 843
1.00×10^{2}	13 000	1.32×10^{-2}	1:11 775
		Zone 3, Rigid structure	
1.00×10^{1}	13 000	1.32×10^{-1}	1:2 872
1.00×10^{0}	13 000	1.32×10^{0}	1:481
1.00×10^{-1}	13 000	1.32×10^{1}	1:49
1.00×10^{-2}	13 000	1.32×10^{2}	1:5
1.00×10^{-3}	13 000	1.32×10^{3}	1:0.5
1.00×10^{-4}	13 000	1.32×10^{4}	1:0.03

*Note: Second moment of area (I) is constant

Table 4.6: Deflection ratio at ground level (parallel to facade)

E_s (MPa)	E_c * (GPa)	Relative bending stiffness (ρ*)	Deflection ratio at ground level (Δ:L)						Damage category
			Bay 1-3			Bay 2-4			
			A1-A3	B1-B3	C1-C3	A2-A4	B2-B4	C2-C4	
		Zone 1 Flexible structure							
1.00×10^4	13	1.32×10^{-7}	1: 16 748	1: 9 042	1: 9 596	-1: 102 827	-1: 61 378	-1: 60 704	0
1.00×10^3	13	1.32×10^{-6}	1: 14 478	1: 8 096	1: 8 492	-1: 131 217	-1: 76 560	-1: 78 510	0
1.00×10^2	13	1.32×10^{-5}	1: 7 766	1: 4 875	1: 4 809	1: 52 612	1: 33 749	1: 26 706	0
		Zone 2 Intermediate stiffness							
1.00×10^1	13	1.32×10^{-4}	1: 1 605	1: 1 234	1: 1 128	1: 2 853	1: 1 977	1: 1 708	2
1.00×10^0	13	1.32×10^{-3}	1: 230	1: 225	1: 210	1: 235	1: 199	1: 183	4 to 5
1.00×10^{-1}	13	1.32×10^{-2}	1: 60	1: 70	1: 74	1: 43	1: 46	1: 47	4 to 5
		Zone 3 Rigid structure							
1.00×10^{-2}	13	1.32×10^{-1}	1: 42	1: 51	1: 56	1: 28	1: 31	1: 34	4 to 5
1.00×10^{-3}	13	1.32×10^0	1: 40	1: 57	1: 55	1: 26	1: 29	1: 34	4 to 5
1.00×10^{-4}	13	1.32×10^1	1: 40	1: 78	1: 54	1: 19	1: 21	1: 89	4 to 5
		Zone 2 Intermediate stiffness							
1.00×10^4	13 000	1.32×10^{-4}	1: 1 603 114	1: 1 239 342	1: 1 138 117	1: 2 765 836	1: 1 970 044	1: 1 718 548	0
1.00×10^3	13 000	1.32×10^{-3}	1: 229 782	1: 225 779	1: 211 798	1: 231 532	1: 198 646	1: 183 260	0
1.00×10^2	13 000	1.32×10^{-2}	1: 60 476	1: 69 670	1: 74 789	1: 42 509	1: 45 791	1: 47 315	0
		Zone 3 Rigid structure							
1.00×10^1	13 000	1.32×10^{-1}	1: 42 180	1: 50 939	1: 56 454	1: 27 742	1: 31 363	1: 33 739	0
1.00×10^0	13 000	1.32×10^0	1: 40 429	1: 57 484	1: 54 619	1: 25 586	1: 28 957	1: 34 398	0
1.00×10^{-1}	13 000	1.32×10^1	1: 40 252	-1: 78 084	1: 54 437	1: 19 371	1: 21 262	1: 88 757	0
1.00×10^{-2}	13 000	1.32×10^2	1: 40 214	-1: 3 211	1: 54 348	1: 5 752	1: 5 907	-1: 6 058	0
1.00×10^{-3}	13 000	1.32×10^3	1: 40 000	-1: 318	1: 54 545	1: 702	1: 705	-1: 570	4 to 5
1.00×10^{-4}	13 000	1.32×10^4	1: 37 500	-1: 29	1: 50 000	1: 58	1: 58	-1: 60	4 to 5

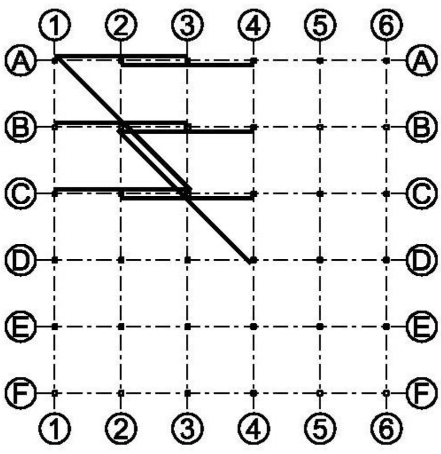


Table 4.7: Deflection ratio at ground level (diagonal to facade)

E_s (MPa)	E_c * (GPa)	Relative bending stiffness (ρ*)	Deflection ratio at ground level – Diagonal – A1-C3	Deflection ratio at ground level – Diagonal – B2-D4	Damage category (taking Table 4.5 into account)
		Zone 1 Flexible structure			
1.00×10^4	13	1.32×10^{-7}	1: 7 987	-1: 43 161	0
1.00×10^3	13	1.32×10^{-6}	1: 7153	-1: 54 817	0
1.00×10^2	13	1.32×10^{-5}	1: 4 483	1: 21 084	0
		Zone 2 Intermediate stiffness			
1.00×10^1	13	1.32×10^{-4}	1: 1 180	1: 1 296	2
1.00×10^0	13	1.32×10^{-3}	1: 187	1: 135	4 to 5
1.00×10^{-1}	13	1.32×10^{-2}	1: 43	1: 33	4 to 5
		Zone 3 Rigid structure			
1.00×10^{-2}	13	1.32×10^{-1}	1: 28	1: 23	4 to 5
1.00×10^{-3}	13	1.32×10^0	1: 27	1: 22	4 to 5
1.00×10^{-4}	13	1.32×10^1	1: 27	1: 24	4 to 5
		Zone 2 Intermediate stiffness			
1.00×10^4	13 000	1.32×10^{-4}	1: 1 168 449	1:1 298 051	0
1.00×10^3	13 000	1.32×10^{-3}	1: 185 024	1: 134 805	0
1.00×10^2	13 000	1.32×10^{-2}	1: 42 497	1: 32 909	0
		Zone 3 Rigid structure			
1.00×10^1	13 000	1.32×10^{-1}	1: 28 428	1: 22 986	0
1.00×10^0	13 000	1.32×10^0	1: 27 121	1: 22 234	0
1.00×10^{-1}	13 000	1.32×10^1	1: 26 992	1: 24 258	0
1.00×10^{-2}	13 000	1.32×10^2	1: 26 989	1: 334 066	0
1.00×10^{-3}	13 000	1.32×10^3	1: 26 852	-1: 4 238	4 to 5
1.00×10^{-4}	13 000	1.32×10^4	1: 26 517	1: 2 847	4 to 5

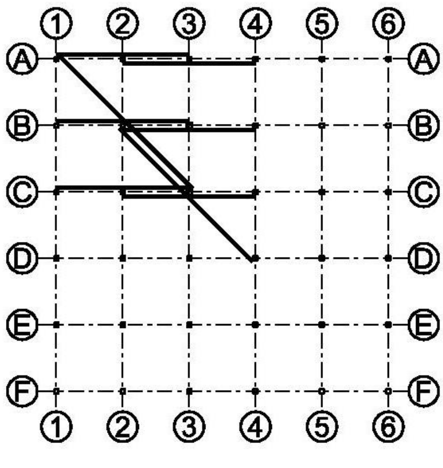


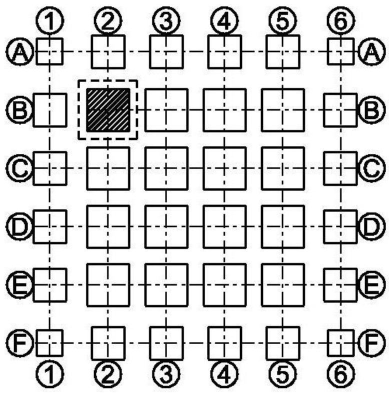
Table 4.8: Deflection limits based on Burland et al. (2001a)

Category of damage	Normal degree of severity	Limiting tensile strain (%)	Deflection limit
0	Negligible	0-0.05	
			1:2 439
1	Very slight	0.05-0.075	
			1:1 626
2	Slight	0.075-0.15	
			1:813
3	Moderate	0.15-0.3	
			1:407
4 to 5	Severe to very severe	> 0.3	

Table 4.9: Column loads with soil stiffness variation at foundation B2

	Load difference at ground level with respect to 10 MPa soil stiffness (%)					
Soil stiffness under foundation B2 (MPa)	**0**	**1**	**5**	**10**	**20**	**100**
Soil stiffness rest of model (MPa)	**10**	**10**	**10**	**10**	**10**	**10**
Column B2	-122	-112	-62	0	10	14
Column A2 & B1 (adjacent, edge)	45	42	23	0	-4	-5
Column B3 & C2 (adjacent, internal)	39	36	20	0	-3	-4
Column A1, A3, C1, C3 (adjacent, diagonal)	-18 to 6	-9 to 5	-5 to 3	0	0 to 1	-1 to 1
Other columns	-7 to 0	-1 to 0	0	0	0	0

	Load difference at ground level with respect to 100 MPa soil stiffness (%)					
Soil stiffness under foundation B2 (MPa)	**0**	**10**	**50**	**100**	**200**	**1000**
Soil stiffness rest of model (MPa)	**100**	**100**	**100**	**100**	**100**	**100**
Column B2	-119	-14	-2	0	2	4
Column A2 & B1 (adjacent, edge)	55	7	1	0	-1	-2
Column B3 & C2 (adjacent, internal)	46	5	1	0	0	-1
Column A1, A3, C1, C3 (adjacent, diagonal)	-19 to 7	-1 to 1	0	0	0	0
Other columns	-7 to 0	-1 to 0	0	0	0	0



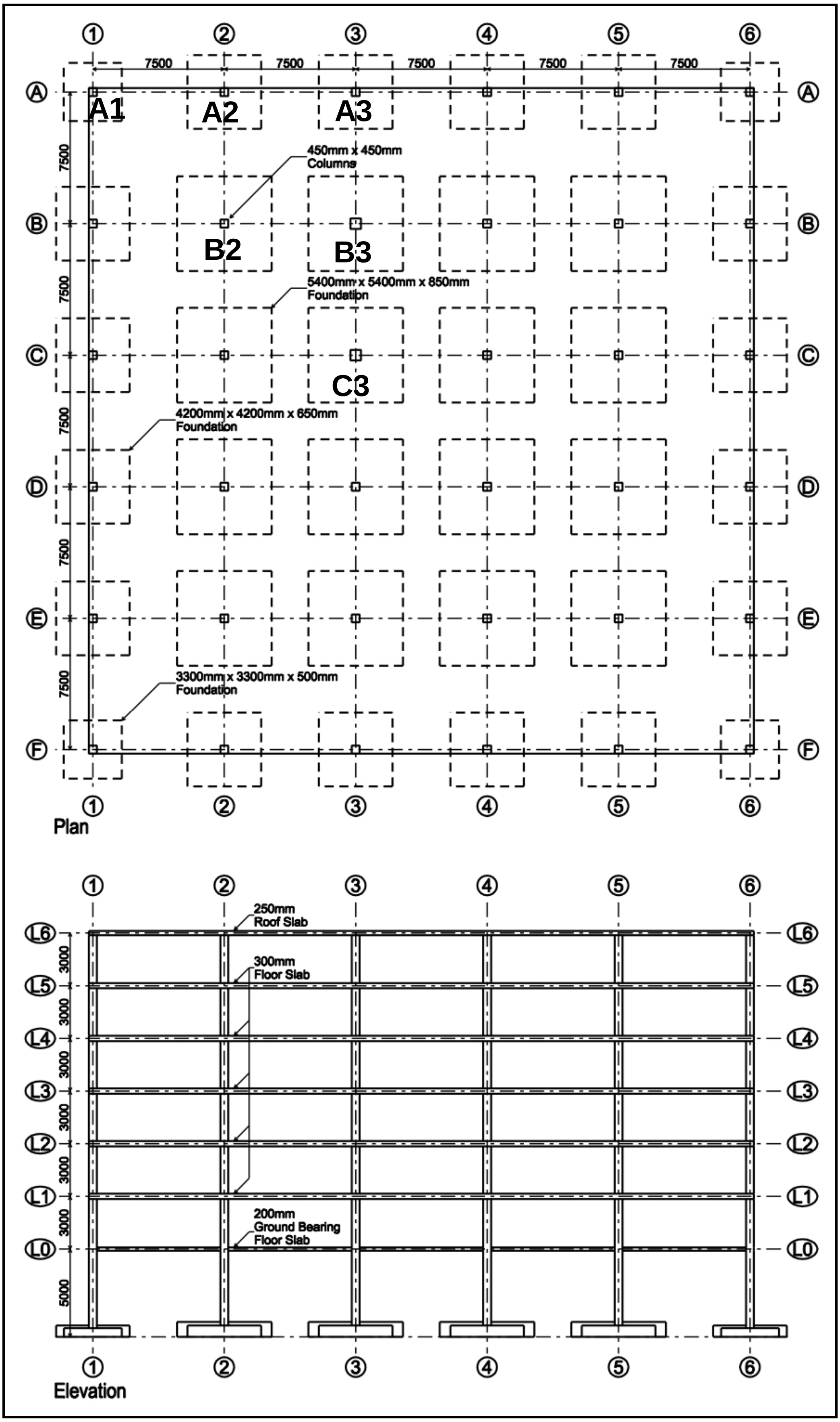


Figure 4.1: **Building layout**

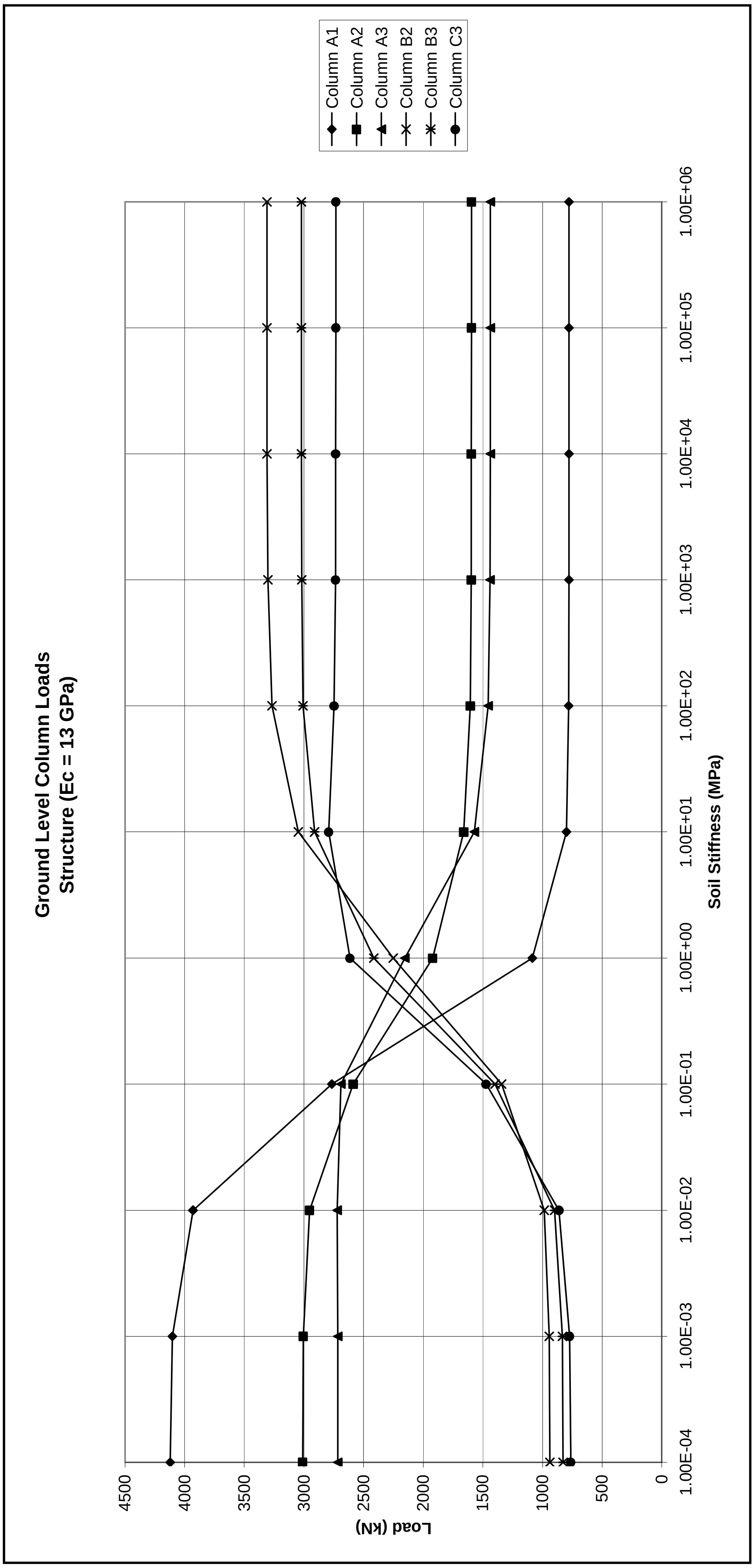


Figure 4.2: **Ground level column loads 5 bay structure (Ec = 13 GPa)**

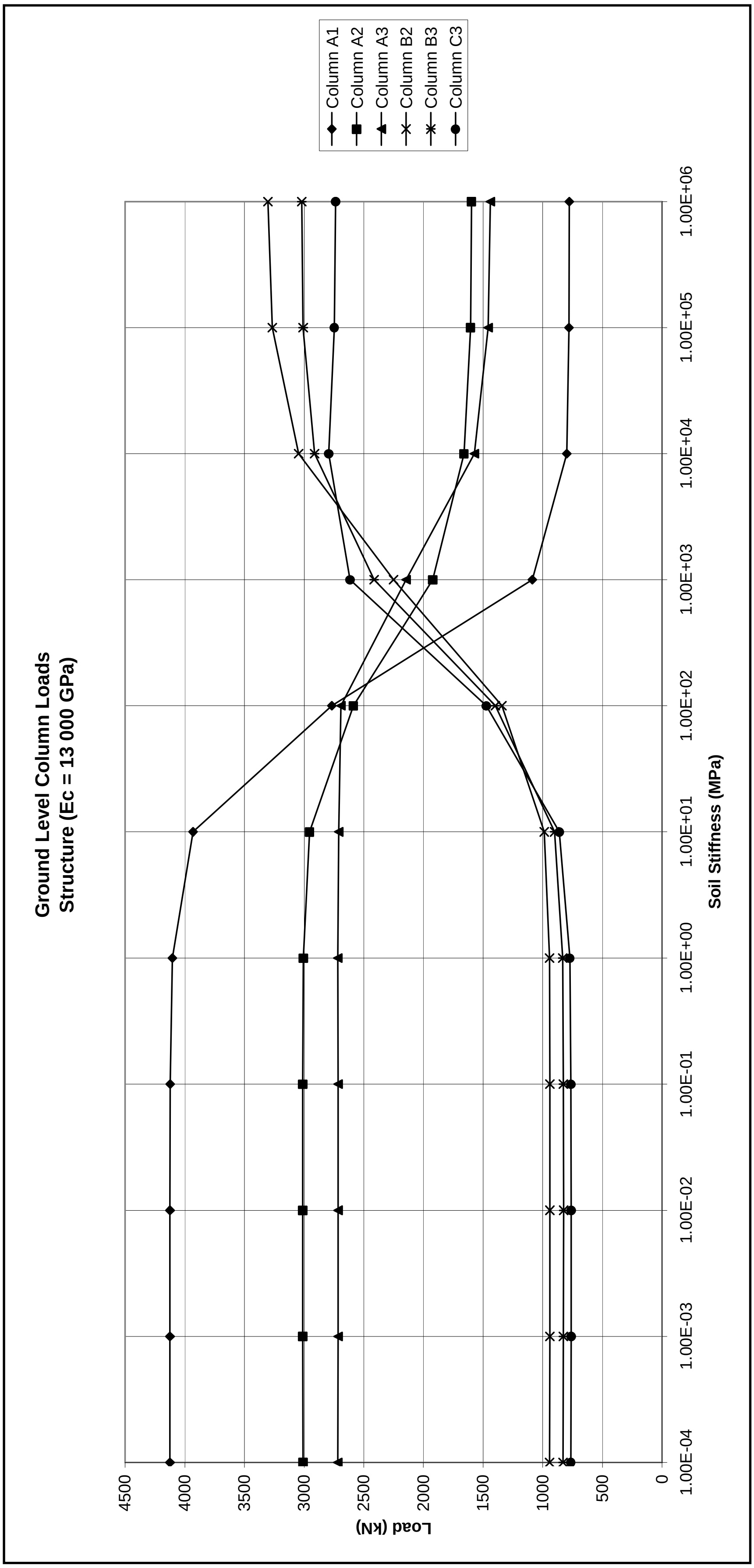


Figure 4.3: Ground level column loads (Ec = 13 000 GPa)

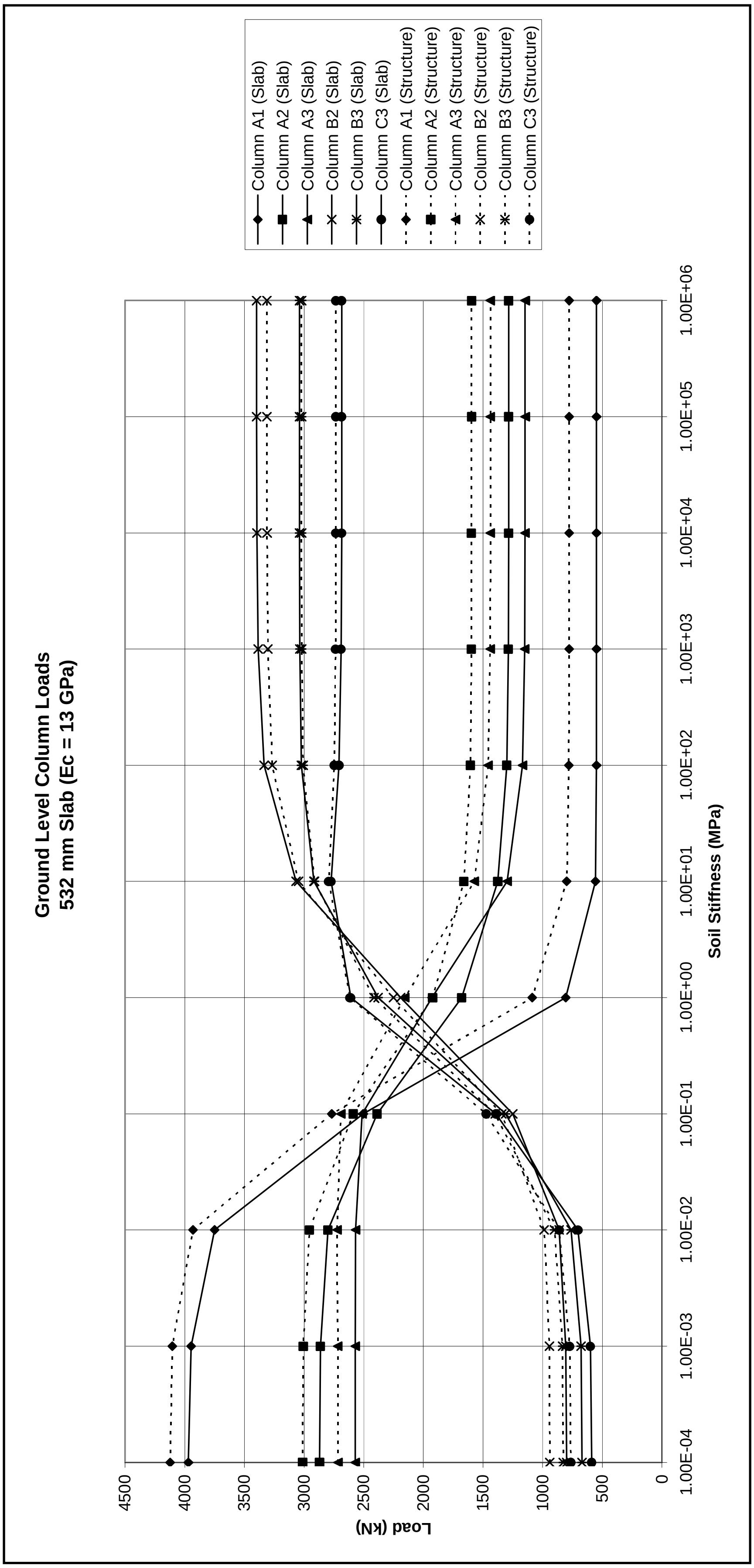


Figure 4.4: Ground level column loads 532 mm slab (Ec = 13 GPa)

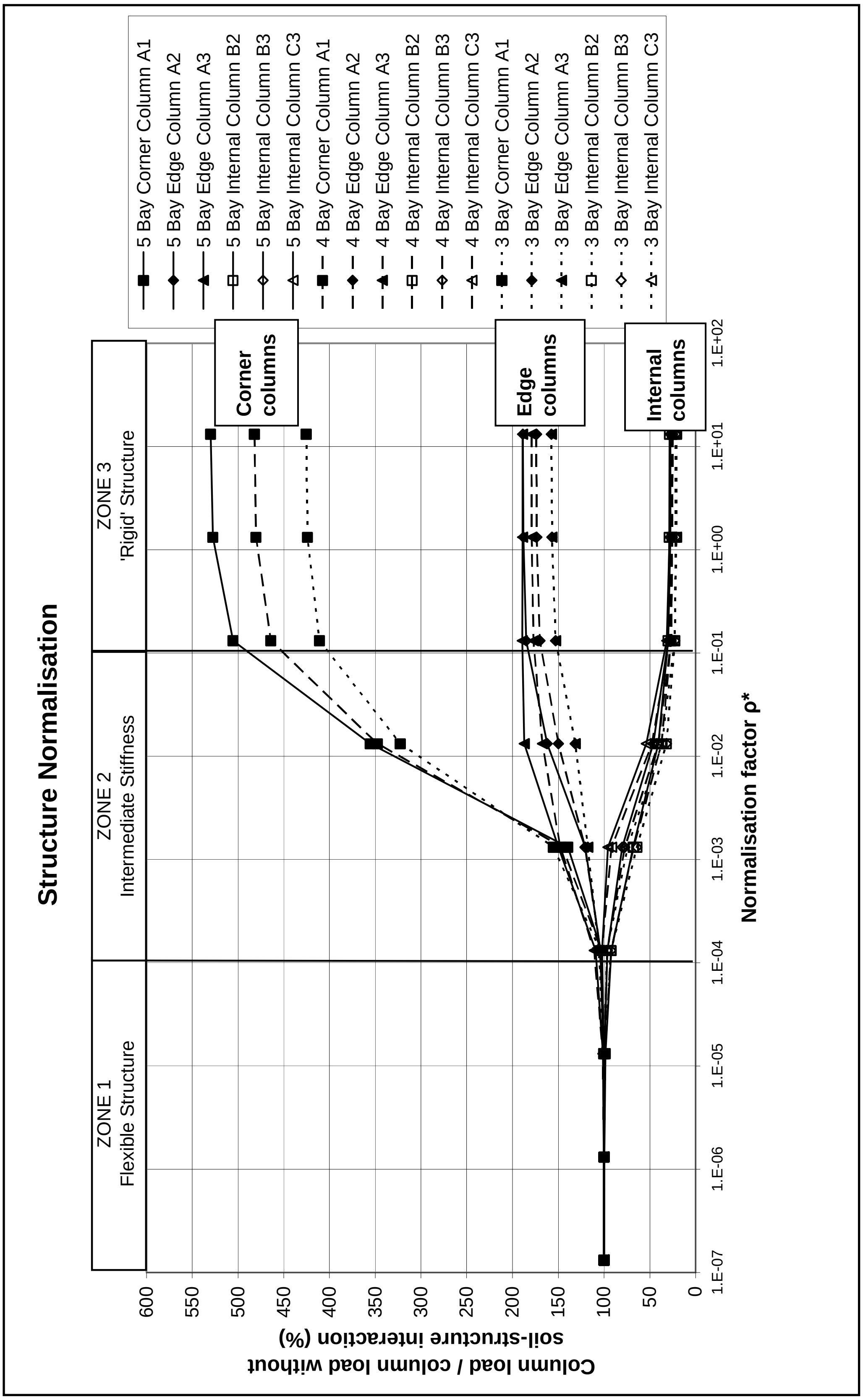


Figure 4.5: **Structure normalisation**

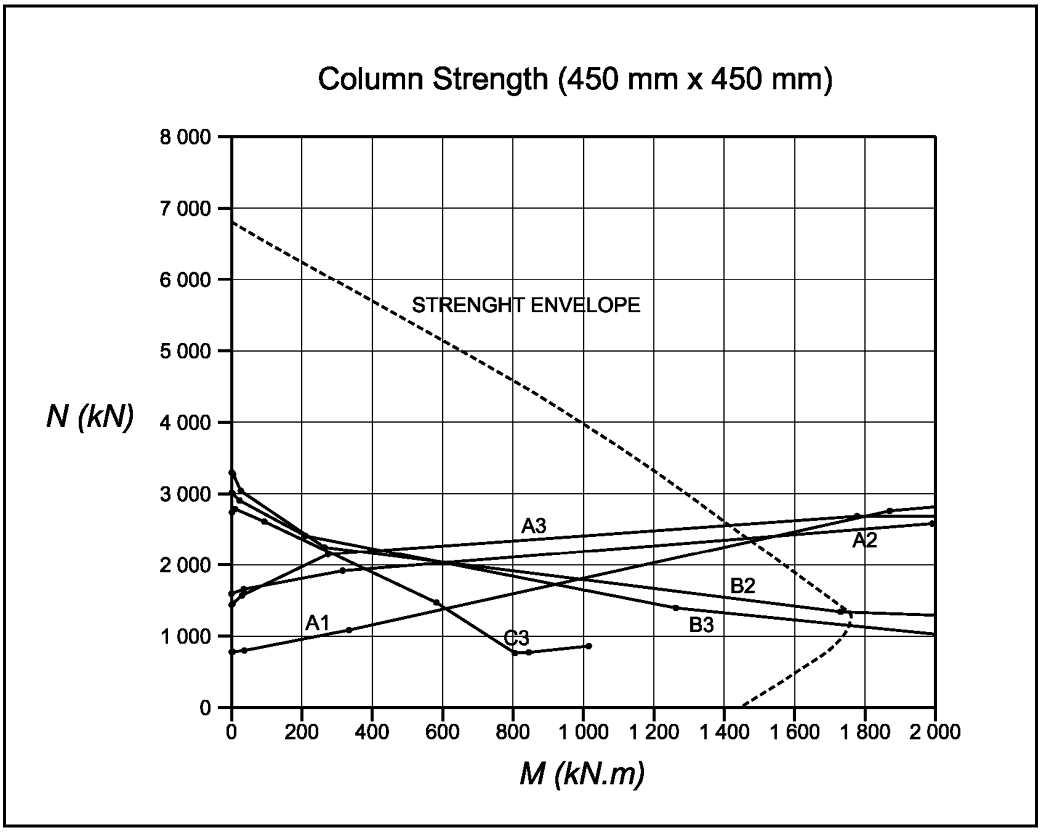


Figure 4.6: Column strength

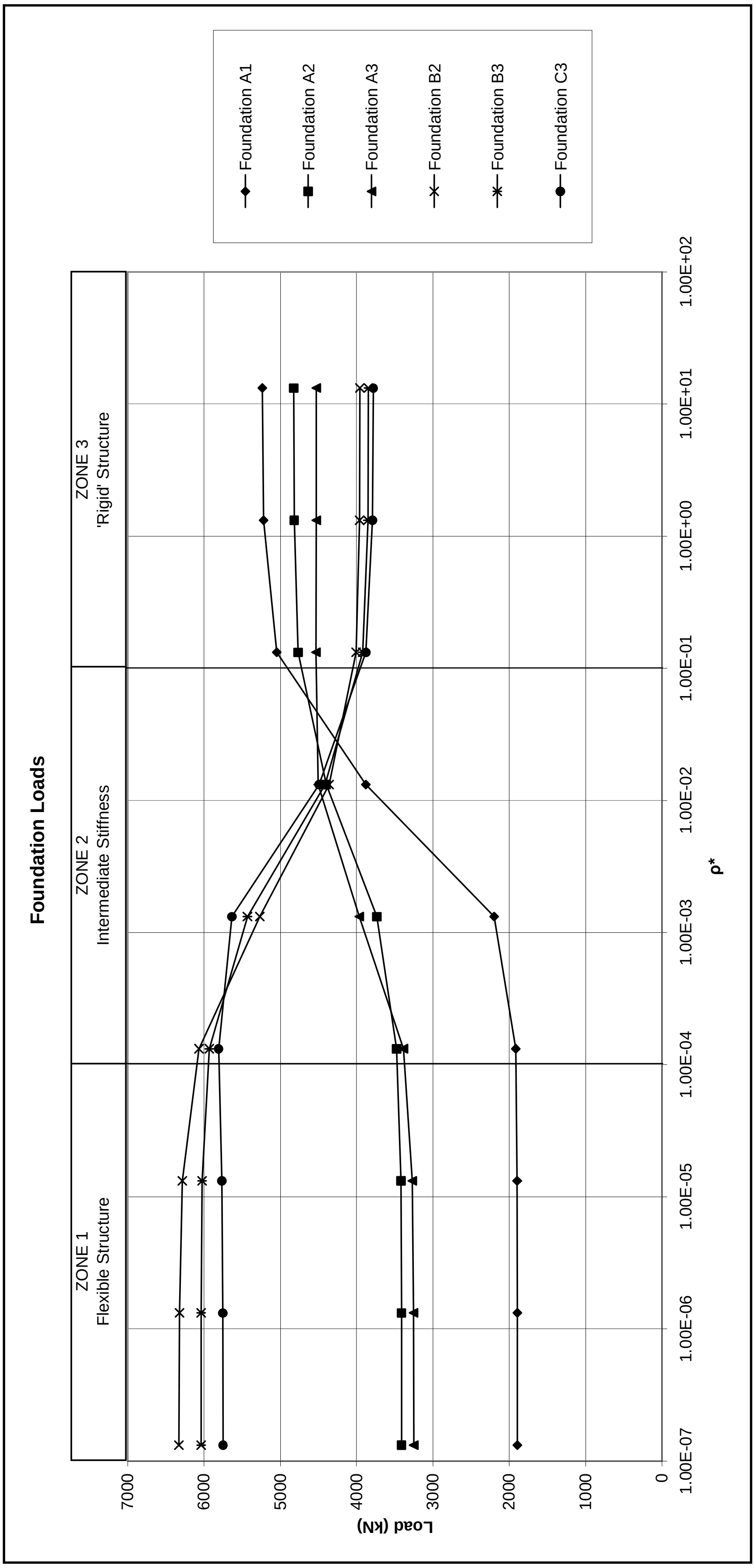


Figure 4.7: Normalised elastic foundation loads and bearing resistance

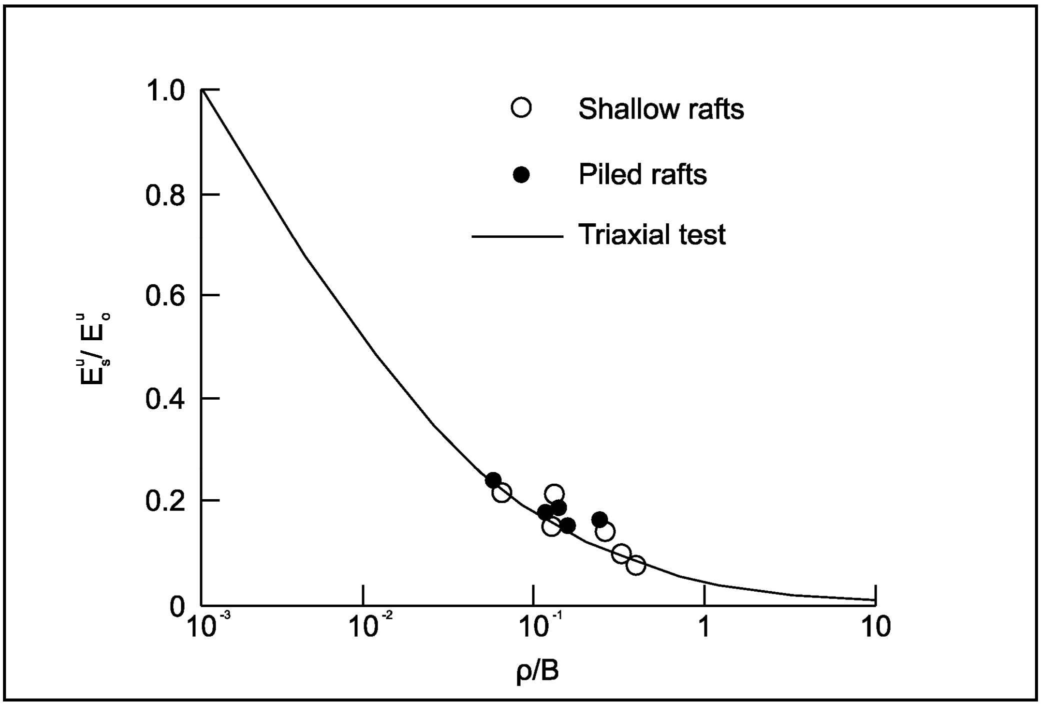


Figure 4.8: **Settlement of foundations on London Clay (Atkinson, 2000)**

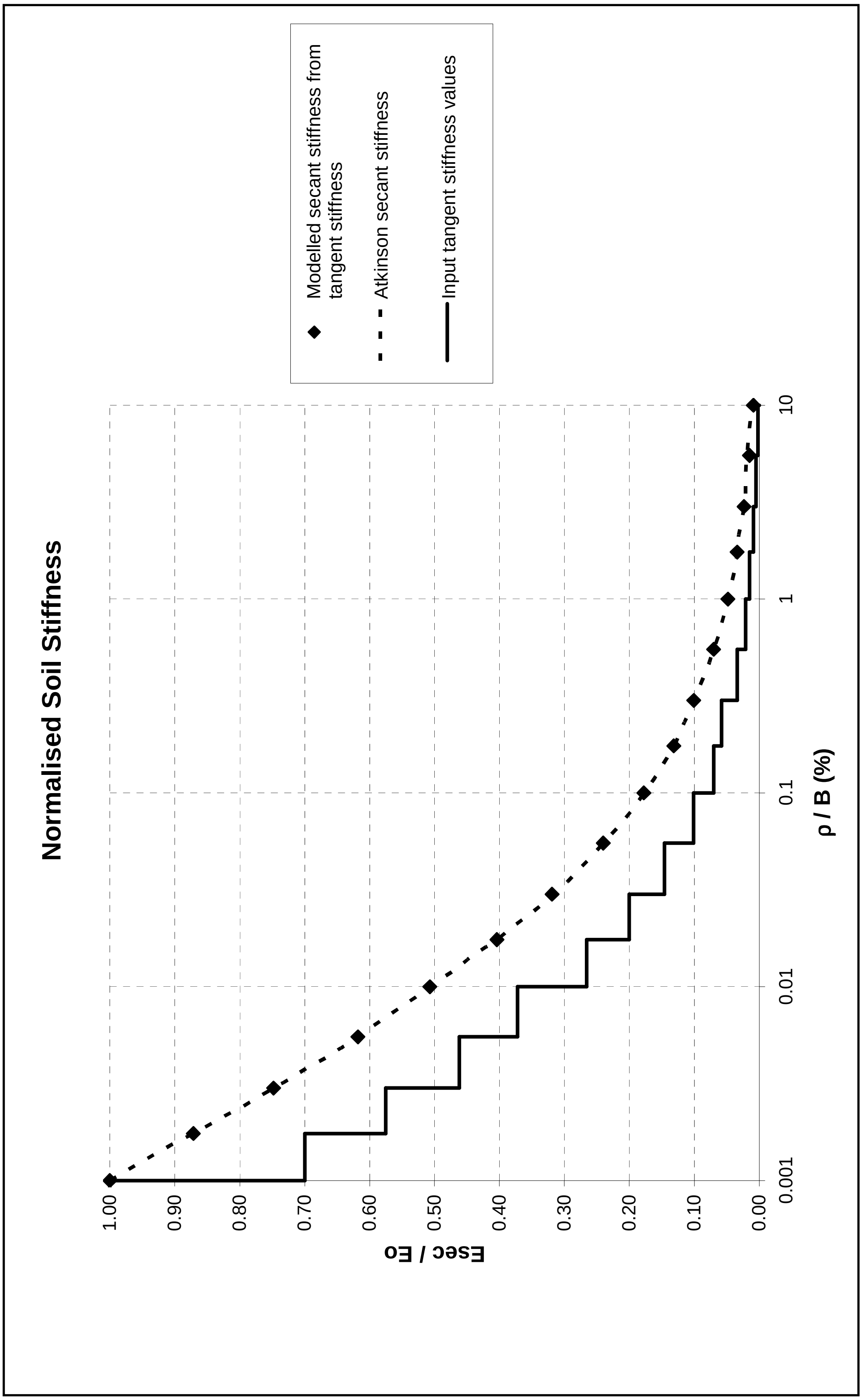


Figure 4.9: Normalised soil stiffness

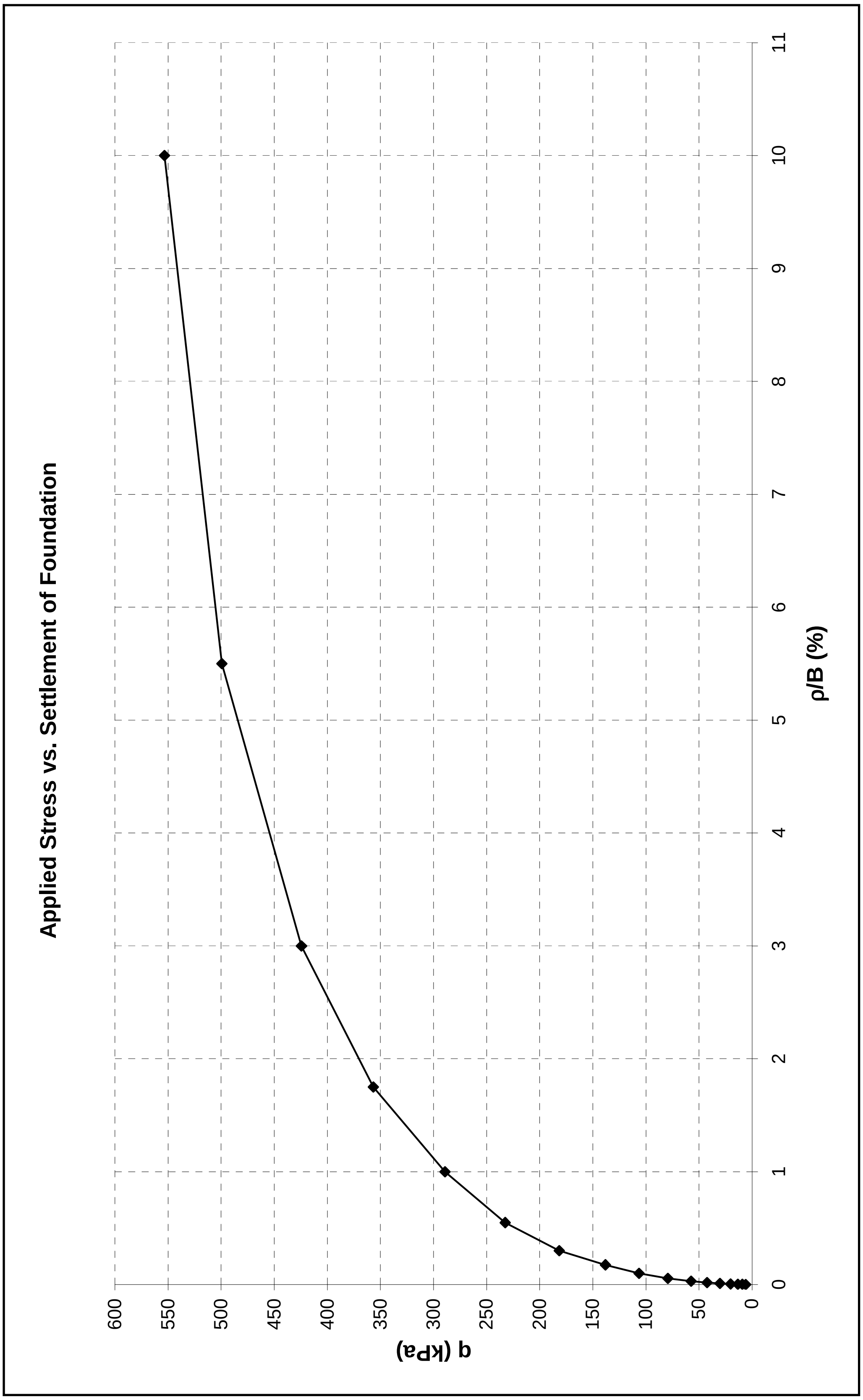


Figure 4.10: **Applied stress vs. settlement of foundation**

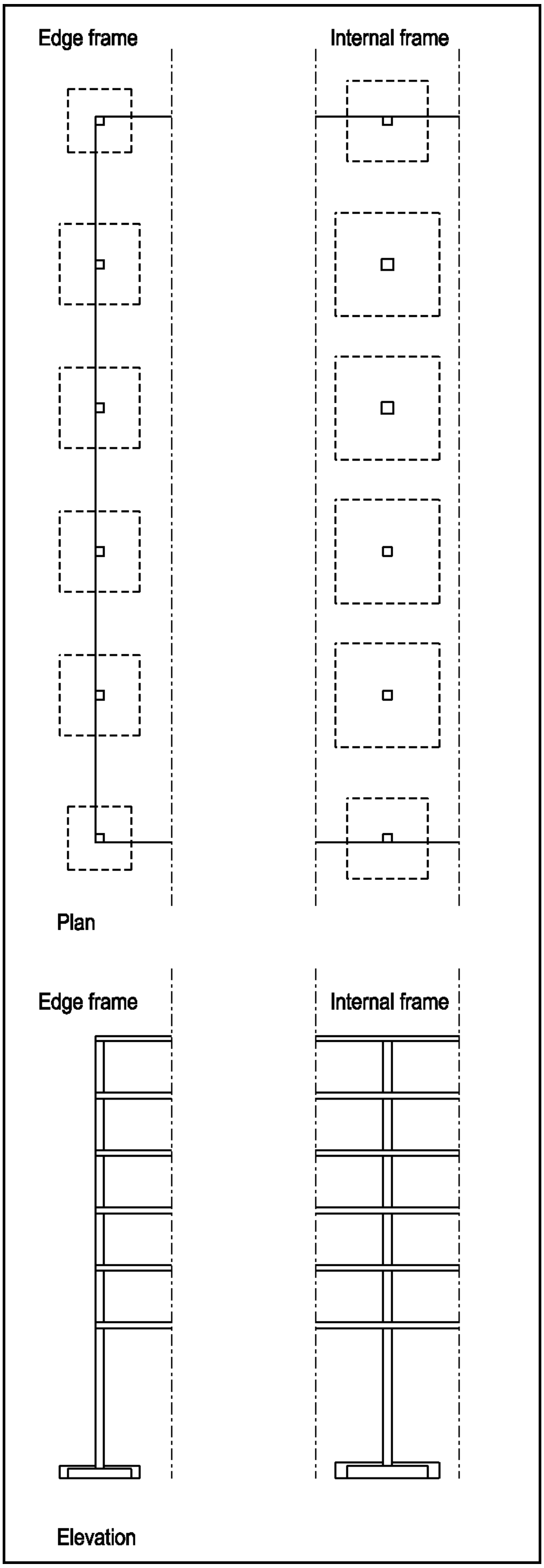


Figure 4.11: 2D frames for bending moment comparisons

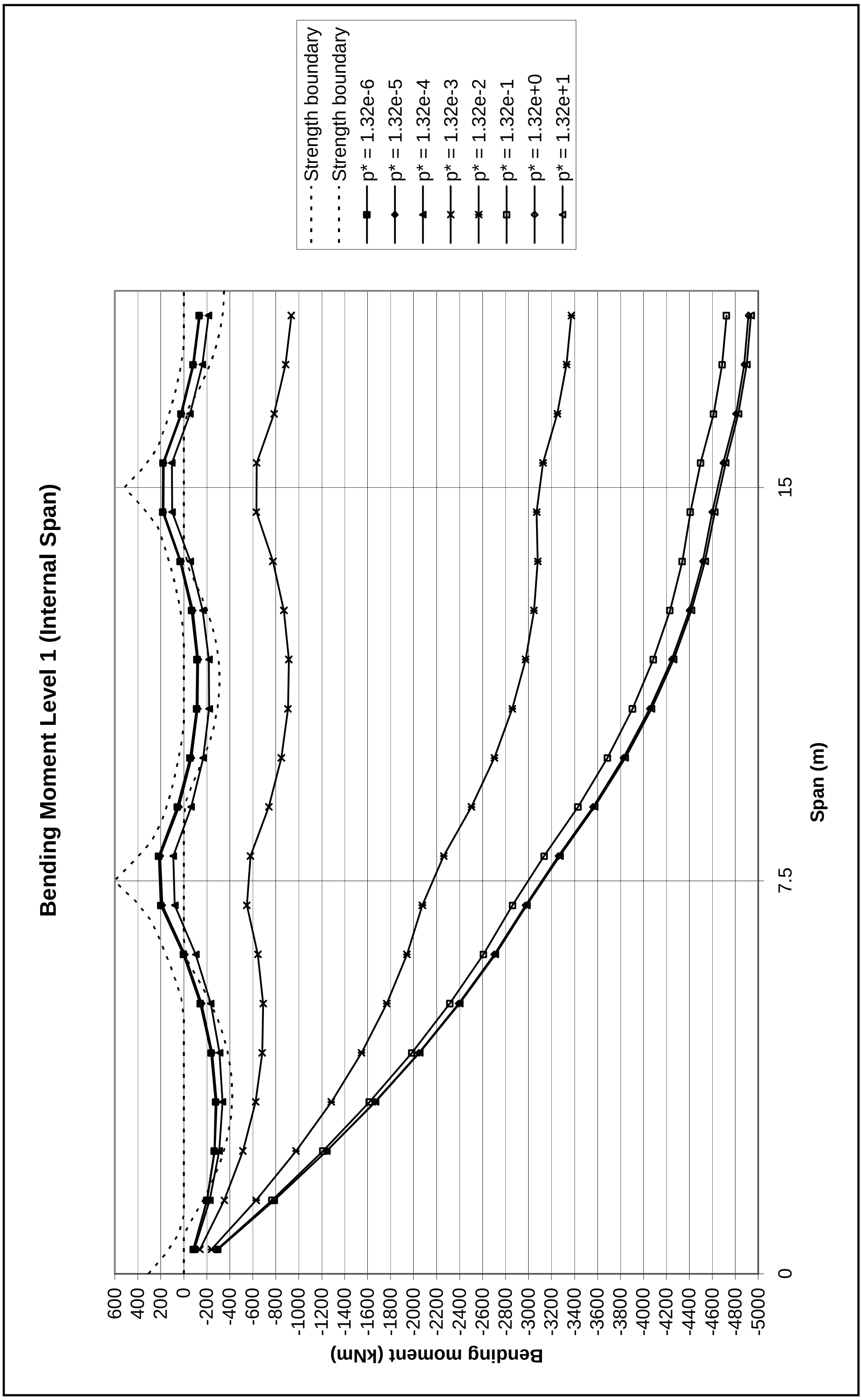


Figure 4.12: **Bending moment level 1, internal span**

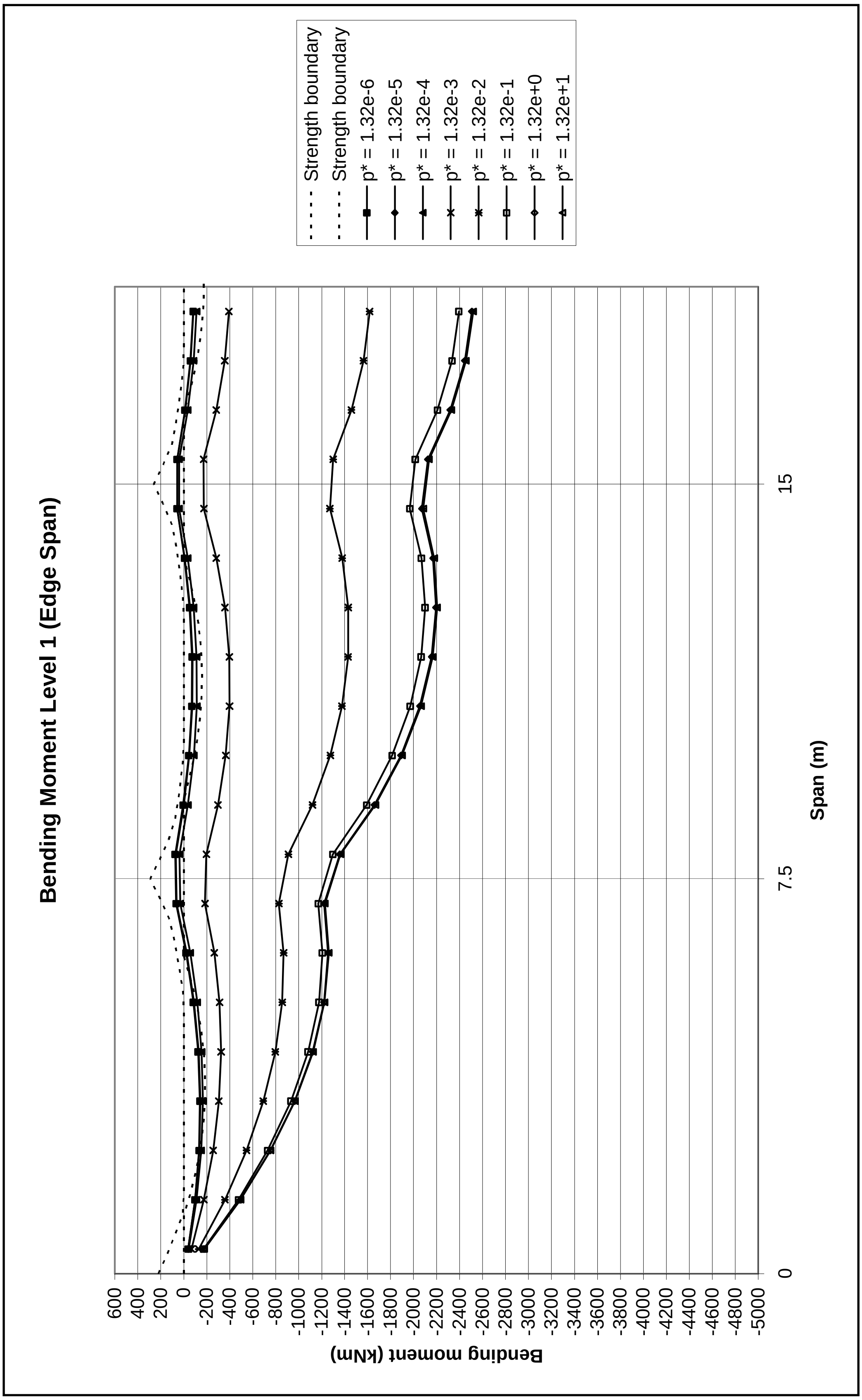


Figure 4.13: **Bending moment level 1, edge span**

5 CONCLUSIONS AND SUGGESTIONS FOR FURTHER WORK

5.1 Conclusions

5.1.1 Conclusions from the literature

- The literature review shows that soil-structure interaction is important. Soil-structure interaction causes transfer of loads within the structure and deformations. Transfer of load within a structure will cause different loads to those predicted without soil-structure interaction which may lead to structural damage or failure. Deformations in the structure may cause visual damage i.e. cracking of facades or partitions; loss of functionality, i.e. doors and windows that get stuck; and unacceptable visual deformations i.e. deflected slabs and tilting of columns (Section 1 & 2.4).
- Movements within the superstructure of a building are not always due to differential settlement of foundations. Significant movements within the superstructure may occur due to the movement of the members under imposed loads and variation in temperature (Section 2.2.2 & 2.4).
- To date definitions to describe differential settlement have been defined for 2D deformation without provision for the 3D deformation characteristics of structures. This thesis considers the 3D nature of modern flexible framed structures undergoing differential settlement (Section 2.1)
- The current state of the art assumes that damage to a building due to differential settlement is confined to cladding and finishes, rather than the structural members. This may be valid for older conventional buildings with brick infill panels, however modern flexible framed buildings with facades that allow for differential settlement may suffer from unacceptable aesthetical deflections, tilting or structural damage before damage to the facades or finishes occur (Section 2.3 & 2.4).

- The current state of the art assumes that the behaviour of a 3D structure undergoing differential settlement can be represented by a simplified 2D beam undergoing bending and shear deformation. For damage prediction, the deformation and maximum tensile strain within the beam are calculated and compared to known critical tensile strains for damage in infill panels. For this method to be valid the following criteria need to be satisfied:
    - Insignificant differential settlement must occur perpendicular to the plane of bending. This may happen where differential settlement occurs due to tunnelling or open excavations parallel to the structure, however differential settlement driven by the self weight of the building results in 3 dimensional deformation of the structure (Section 2.4.1).
    - The complete building (including facades and partitions) is constructed before any differential settlement occurs. This may be a justifiable assumption if the differential settlement is caused by adjacent excavation after the completion of the building. However, for differential settlement driven by the self weight of the building the change of stiffness of the structure and the settlement that occurs as construction progresses, need to be taken into account (Section 2.4.1).
    - The facades and partitions are fixed to the frame and no allowance is made for differential settlement. Gaps or brackets allowing for movement will reduce the strain in facades and partitions (Section 2.4.1).
    - The equivalent bending and shear stiffnesses of the structure need to be predicted. Potts and Addenbrooke (1997) calculated the bending stiffness of the superstructure by using the parallel axis theorem, which in their view is an overestimate of building stiffness as only a rigidly framed structure would approach such modes of deformation. They also suggest an alternative approach by which the bending stiffness of the superstructure is obtained by summing the independent *EI* values for each storey, which implies that the walls and columns transfer the same deformed shape to each storey. No further guidance is given in the literature on the stiffness of a real structure (Section 2.2.4).

- Extensive codes in the form of Eurocodes and British Standards exist with guidelines on how to design a building. In contrast, there is little guidance in the literature on how to perform a numerical analysis of a structure and its foundations, with the aim of providing the loads, deformations, shear forces and bending moments that will occur in reality.

5.1.2 Conclusions from the methodology

- The methodology involved the design of a structure to determine the member sizes for the subsequent finite element analysis (Section 3.1.2).
- The design was done using British Standards and the Eurocodes. Standard design Excel spreadsheets from RCC-2000 were used for the design. RCC-2000 are design spreadsheets based on BSI 8119-1 (1997) and were published by the British Cement Association on behalf of the industry sponsors of the Reinforced Concrete Council (Section 3.1.2).
- A finite element analysis of a complete building on pad foundations was carried out using LUSAS on a 2.4 GHz Intel Core2 Duo PC with a 32-bit platform and 3 GB of RAM (Section 3.1.2).
- The size of the linear elastic model in LUSAS was limited by the discretisation of the foundations and supporting halfspace and not by the solver. LUSAS is only available as a 32-bit program and can therefore only use a maximum of 3 GB of RAM (Section 3.3).
- In the finite element model, column loads at ground level supported by a 'rigid' halfspace were within 10% of the load takedown, calculated assuming each column supports half of the span. The total load from the model was within 2% of the total load from the load takedown (Section 3.3).
- To determine the optimal element type, discretisation and halfspace boundaries, a single rigid foundation on a halfspace was modelled and the load displacement response compared to standard elastic solutions. The load-displacement response of a single rigid pad foundation could be calculated to within 1% of the approximate elastic solution. However the mesh size needed for this accuracy proved to be too fine for the complete model to work, given the available computer resource. Reducing the accuracy of the single foundation load-displacement ratio to within 7% allowed a usable mesh density for the complete model (Section 3.3).

- The use of 10 node tetrahedral continuum elements for the foundations and supported halfspace gave better results in LUSAS than the same number of 4 node tetrahedral continuum elements. The total number of elements that could be discretised in the model was limited by the PC memory and was the same for 4 and 10 node elements (Section 3.3).
- An irregular mesh with variable density proved to be more suitable than a regular mesh for the supporting halfspace (Section 3.3).

5.1.3 Conclusions from the analyses

Normalisation of data

- For an elastic structure supported by a linear elastic soil it is possible to normalise the relative bending stiffness with ρ^* as:

$$\rho^* = \frac{EI}{E_s H^4}$$

  Where E is the Young's modulus of the concrete, I is the second moment of inertia of the structure, E_s is the Young's modulus of the soil and H is the half length of the building (Section 4.1).
- Three distinct zones of behaviour within the relative bending stiffness range can be identified (Section 4.1).
  - *Zone 1 'Flexible structure'* is the zone of relative bending stiffness (ρ^*) where the structure is flexible in comparison to the soil. ρ^* is typically less than 1 x 10^{-4} in *Zone 1*. Behaviour in *Zone 1* can be determined without taking soil-structure interaction into account.
  - *Zone 2 'Intermediate structure'* is the intermediate zone where the loads, shear forces and bending moments within the structure change as the relative bending stiffness increases. ρ^* typically ranges from 1 x 10^{-4} to 1 x 10^{-1} in *Zone 2*.
  - *Zone 3 'Rigid structure'* is the zone of relative bending stiffness (ρ^*) where the structure is rigid in comparison to the soil. In *Zone 3* the loads, shear forces and bending moments within the structure are again constant, independent of the relative bending stiffness, but they may be very different from that in *Zone 1*. ρ^* is typically larger than 1 x 10^{-1} in *Zone 3*.

- An equivalent slab, with a thickness resulting in a second moment of area equal to the sum of the second moment of area of the individual slabs (around their individual neutral axes) within the structure, exhibited a similar soil-structure behaviour as the 6 storey structure. The equivalent slab was suspended and supported by the subsoil columns and foundations. For this specific structure the equivalent slab thickness was 532 mm. An alternative approach using an equivalent slab with a thickness based on the parallel axis theorem resulted in a slab thickness of 14.0 m which was an overestimate of the structural bending stiffness. The bending stiffness of the alternative method is approximately 18 000 times stiffer (Section 4.1)
- The designed 'typical' 5 bay structure on a soil with an undrained shear strength of 90 kPa and a subsequent Young's modulus of 600 MPa resulted in an approximate relative bending stiffness (ρ*) of 2.2 x 10^{-6}, which falls in Zone 1. The structure will therefore behave flexibly (Section 4.1).

Structural strength

- As the relative bending stiffness increased in the linear elastic model the axial loads in the corner columns increased by approximately 5 times, the loads in the edge columns approximately doubled and the loads in the internal columns reduced to approximately 1/3 in comparison to the column loads within a flexible structure. Yielding of foundations or the structure may reduce the column loads (Section 4.2.1)
- The linear-elastic numerical model showed the column loads exceeded the strength of the columns where the relative bending stiffness (ρ*) is larger than 1.32 x 10^{-3}. Yielding of foundations may reduce the column loads and protect against column failure (Section 4.2.1).
- Atkinson (2000) related soil strength to stiffness. A soil with an undrained shear strength of 90 kPa will have an expected small strain stiffness of 600 MPa and an operational stiffness of between 75 and 123 MPa (between 0.07% and 0.20% strain) and this is dependent on the foundation stiffness (Section 4.2.2).

- Foundation stiffness and non-linear soil stiffness influenced the column loads. For a structure with a relative bending stiffness (ρ*) of 1.32 x 10^{-1}, superstructure concrete stiffness of 780 000 GPa and soil stiffness of 600 MPa (Section 4.2.2):
    - The load in the corner column (A1) was reduced by 14% in the non-linear model with a 13 GPa foundation concrete stiffness and 34% in the non-linear model with a 780 000 GPa concrete stiffness.
    - The loads in the edge columns (A2, A3) were reduced by between 1% and 4% in the non-linear model with a 13 GPa foundation concrete stiffness and 11% in the non-linear model with a 780 000 GPa concrete stiffness.
    - The loads in the internal columns (B2, B3, C3) ranged from a reduction of 5% to an increase of 9% in the non-linear model with a 13 GPa foundation concrete stiffness and increase of between 56% and 65% in the non-linear model with a 780 000 GPa concrete stiffness.
- The linear-elastic numerical model showed the bending moments in the slabs exceeded the strength of the slabs where the relative bending stiffness (ρ*) is larger than 1.32 x 10^{-4}. Yielding of foundations may reduce the column loads and protect against column failure (Section 4.2.3).

Structural deformation

- Tilt does not relate to relative bending stiffness; it is a function of both the actual soil stiffness and the structure stiffness. A decrease in soil stiffness results in an increase of tilt. A decrease of structural stiffness also results in an increase of tilt, therefore a flexible structure is more likely to suffer from tilt (Section 4.3.1).
- For structures without facades or internal partitions or where the facades and partitions are specifically designed to allow movement, the literature suggested deflection limits that ranged from 1:250 to 1:600 (Section 4.3.2).
- The deflection limits based on the beam model proposed by Burland *et al.* (2001a) depend on the assumed class of damage and are much stricter. These limits are applicable to masonry infill structures where damage will most likely occur to the facades. A deflection limit of 1: 2 439 is proposed for negligible damage. Based on this limit a relative bending stiffness of $\rho* > 1$ x 10^{-4} will result in damage (Section 4.3.2).

- The researched structure deformed in 3D and the facade deflection was significantly less than the maximum (diagonal) deflection. It is therefore important to take the 3D deformation of a structure into account when analysing a structure for differential settlement damage (section 4.3.2).

Variation in foundation load-displacement response

- A structure's response to soil variation depends on the structure's overall relative bending stiffness (Section 4.4).
- For a relative bending stiffness of $\rho^* = 1.32 \times 10^{-4}$, a foundation on a soil stiffness of one order of magnitude less increased the loads in adjacent columns between 36% and 42%. Increasing the soil stiffness under the foundation one order of magnitude resulted in an increase of load of 14%. A foundation with a stiffer response may therefore have an effect on foundation load. This is significant for the reuse of foundations which have a stiffer (up to 10 times) load-displacement response due to preloading (Section 4.4).
- For an overall relative bending stiffness of $\rho^* = 1.32 \times 10^{-5}$, a foundation on a soil stiffness of one order of magnitude less increased the loads in adjacent columns between 5% and 7%. Increasing the stiffness under the foundation one order of magnitude resulted in a maximum load variation of 4%. The results showed that the higher the overall stiffness of the soil, the less impact a harder spot under a foundation or a stiffer load-displacement response has on the structure (Section 4.4).

5.2 Suggestions for further work

Suggestions for future work include:

- Model the behaviour of modern flexible framed structures on a non-linear soil model with yielding. The presented research was based on a linear-elastic model with a manual iteration process to show the effect of non-linear soil response. The incorporation of non-linear soil model with yielding into the finite element model may produce more accurate answers.
- Model the effect of single piles and pile groups (instead of pad foundations) on the behaviour of modern flexible framed structures.
- Model the variation in soil stiffness more rigorously, i.e. investigate the effect of spatial soil stiffness variation on the corner, edge and internal foundations.

- Determine and model the actual bending stiffness of concrete structures taking into account:
    - Stiffness and fitment details of the facades. Suggested facades include glass panels, light weight concrete blocks and brickwork.
    - Internal walls made out of plasterboard partitions, lightweight concrete blocks or brickwork.
    - Lift shafts and other methods of providing racking stiffness.
    - Creep, shrinkage and cracking of concrete.
    - The variation in stiffness during construction.
- Model the behaviour of steel framed structures and compare it to the flat slab reinforced concrete structure.
- Model the behaviour of reinforced concrete structures with beams and compare it to the flat slab structure.
- Model the effect of construction sequence on differential settlement, taking into account the increase in structural stiffness and load during construction and the time dependency of settlement. It is suggested to model the construction sequence for both steel framed and reinforced concrete framed structures.
- Model the effect of temperature variation within steel framed and concrete framed structures on the soil-structure interaction.
- Develop rigorous definitions to describe 3D deformations.

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APPENDIX A DESIGN LOADS

Imposed Loads (BS 6399-1:1996)

Floors

	Uniformly distributed load (kN/m²)	Concentrated load (kN)	Comments
Floor	2.5	2.7	Table 1, Category B, Offices for general use
Partitioning	1		Paragraph 5.1.4 Partitions
Total	**3.5**	**2.7**	

Columns

Reduction in total imposed distributed load on columns (Table 2)

Floors carried by column	Load reduction (%)
1	0
2	10
3	20
4	30
5 to 10	40

Roof Loads (BS 6399-3:1998)

Site is 100m a.m.s.l.
Located in Southampton
Basic snow load on ground (Fig. 1): 0.5 kN/m2
Snow load shape coefficeint (Fig. 2) : 0.8
Snow load: 0.4 kN/m
Minimum imposed load on roof with access (Paragraph 4.2): **1.5kN/m2** > Snow load

Wind loads (BS 6399-2:1997)

Code applicable (Figure 3)
Basic wind speed (Southampton) (Paragraph 2.2.1): 22m/s
Altitude factor (100m a.b.s.l.) (Paragraph 2.2.2.2): 1.1
Direction factor (Paragraph 2.2.2.3.) : 1.0
Seasonal factor (Paragraph 2.2.2.4): 1.0
Probability factor (Paragraph 2.2.2.5): 1.0
Site wind speed (Equation 8): 24.2 m/s
Effective building height:
18m
Building in town, 2km from sea
Terrain and building factor (Paragraph 2.2.3.3): 1.88
Effective wind speed (Equation 12): 45.5
m/s
Dynamic pressure (Equation 1): 1269 Pa
D/H 37.5/18=2.1
External pressure coefficient (Table 5): +0.758 and -0.5
Size effect factor (Figure 4): 0.87
Pressure acting on external surface (Equation 2): **+0.837 and -0.552 kN/m2**

Horizontal force of 1.5% of 1 floor weigh and colums:
Floor weight (7.775x23.1x23.1): 4148.8 kN
Columns weight (0.6x0.6x2.7x36x24.03): 23.4 kN
External wall (23.1x4x7.241): 669.1 kN
Total: 4841.2 kN
1.5 % of weight: 72.62 kN
Distibuted load (72.62/(3x23.1)): 1.047 kN/m2 < 0.837 + 0.552 **OK**

Dead Loads

Materials Weights(BS 648:1964)
Reinforced concrete: 24.03 kN/m3
Screed (0.5 inch): 0.293 kN/m2
Clay tiles (0.5 inch): 0.273 kN/m2
Roof asphalt (0.75 inch): 0.419 kN/m
Brickwork: 21.535 kN/m3
Glazing: 0.195 kN/m2

Floors

Reinforced concrete (300mm)	7.209	kN/m2
Screed	0.293	kN/m2
Clay tiles	0.273	kN/m2
Total	**7.775**	**kN/m2**

Roof

Reinforced Concrete (250mm)	6.008	kN/m2
Roof Asphalt	0.419	kN/m2
Total	**6.427**	**kN/m2**

External walls

50% Glazing, 50% Brickwork		
Brickwork (240mmx2700mm)	13.955	kN/m
Glazing (2700mm)	0.527	kN/m
Weighted Average	**7.241**	**kN/m**

Design loads (ULS) (BS 8810-1:1997)

Partial safety factors for loads
Dead and imposed load combination

Dead adverse	1.4
Dead beneficial	1.0
Imposed adverse	1.6
Imposed beneficial	0.0

	DL	IL
Roof (kN/m2)	6.43	1.50
Floor (kN/m2)	7.78	3.50
Walls (kN/m)	7.24	-

Factored	DL	IL
Roof (kN/m2)	9.00	2.40
Floor (kN/m2)	10.89	5.60
Walls (kN/m)	10.14	-

APPENDIX B SLAB DESIGN

Project	RC Structure				REINFORCED CONCRETE COUNCIL		
Client					Made by	Date	Page
Location	Internal Roof (xx)	from grids 1	to 6		GS	26-Jan-09	1
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997				Checked	Revision	Job No
	Originated from RCC33.xls on CD	© 1999 BCA for RCC			GS	-	

MATERIALS							COVERS	mm	TO LAYER
	fcu	30	N/mm²	h agg	20	mm	Top cover	25	1
	fyl	500	N/mm²	γs	1.15	steel	Btm cover	25	1
	fyv	500	N/mm²	γc	1.50	concrete			

SPANS	L (m)	GEOMETRY		
SPAN 1	7.500	Bay type	INTERNAL	
SPAN 2	7.500	Slab depth, h	250	mm
SPAN 3	7.500	Panel width, b	7500	mm
SPAN 4	7.500			
SPAN 5	7.500	End distance	225	from supt 1
SPAN 6		End distance	225	from supt 6

PERIMETER LOADS *characteristic*

kN/m outside supports 1 & 6

LOADING PATTERN	min	max
DEAD	1.0	1.4
IMPOSED		1.6

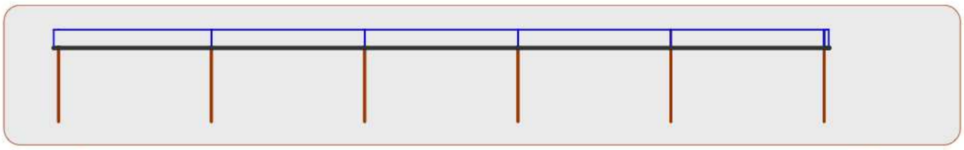
SUPPORTS	ABOVE (m)	H (mm)	B (mm)	End Cond	BELOW (m)	H (mm)	B (mm)	End Cond
Support 1					2.700	450	450	F
Support 2					2.700	450	450	F
Support 3					2.700	450	450	F
Support 4					2.700	450	450	F
Support 5					2.700	450	450	F
Support 6					2.700	450	450	F
Support 7								

LOADING *UDLs (kN/m²) PLs (kN/m) Position (m)*

	Dead Load	Imposed Load	Position from left	Loaded Length
Span 1				
UDL	6.43	3.50	~~~~~	~~~~~
PL 1				~~~~~
PL 2				~~~~~
Part UDL				
Span 2				
UDL	6.43	3.50	~~~~~	~~~~~
PL 1				~~~~~
PL 2				~~~~~
Part UDL				
Span 3				
UDL	6.43	3.50	~~~~~	~~~~~
PL 1				~~~~~
PL 2				~~~~~
Part UDL				

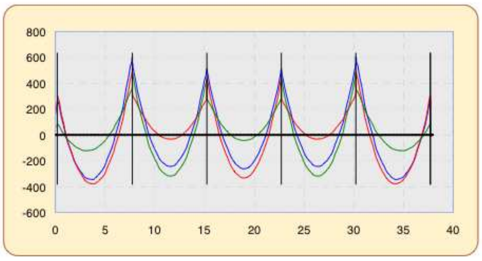
	Dead Load	Imposed Load	Position from left	Loaded Length
Span 4				
UDL	6.43	3.50	~~~~~	~~~~~
PL 1				~~~~~
PL 2				~~~~~
Part UDL				
Span 5				
UDL	6.43	3.50	~~~~~	~~~~~
PL 1				~~~~~
PL 2				~~~~~
Part UDL				
Span 6				
UDL			~~~~~	~~~~~
PL 1				~~~~~
PL 2				~~~~~
Part UDL				

LOADING DIAGRAM

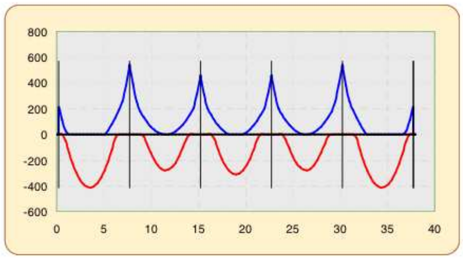


1 6

BENDING MOMENT DIAGRAMS (kNm)



Elastic Moments

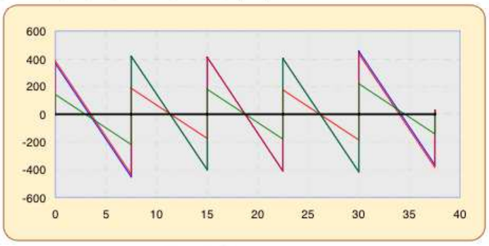


Redistributed Envelope

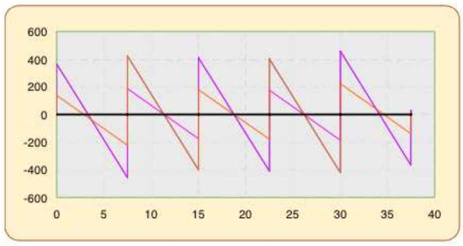
SUPPORT No	1	2	3	4	5	6	
Elastic M	296.2	601.4	508.4	508.4	601.4	296.2	~
Redistributed M	193.5	541.3	457.6	457.6	541.3	193.5	~
βb	0.653	0.900	0.900	0.900	0.900	0.653	~
Redistribution		10.0%	10.0%	10.0%	10.0%		
End support reinf. Ø mm	16 *					16 *	

SPAN No	1	2	3	4	5	
Elastic M	378.91	317.49	332.26	317.49	378.91	~
Redistributed M	413.25	279.20	312.23	279.20	413.25	~
βb	1.091	0.879	0.940	0.879	1.091	~

SHEARS FORCE DIAGRAMS (kN)



Elastic Shears



Redistributed Shears

SPAN No	1		2		3	
Elastic V	384.5	455.3	418.1	404.1	410.6	410.6
Redistributed V	364.2	457.1	421.7	401.7	410.6	410.6

SPAN No	4		5			
Elastic V	404.1	418.1	455.3	384.5	~	~
Redistributed V	401.7	421.7	457.1	364.2	~	~

REACTIONS (kN)

SUPPORT	1	2	3	4	5	6
ALL SPANS LOADED	388.8	878.7	810.0	810.0	878.7	388.8
ODD SPANS LOADED	388.7	642.7	586.5	586.5	642.7	388.7
EVEN SPANS LOADED	148.3	643.6	582.4	582.4	643.6	148.3
Veff for punching	486.0	923.3	819.0	819.0	923.3	486.0
Characteristic Dead	171.9	384.5	358.2	358.2	384.5	171.9
Characteristic Imposed	92.6	212.7	192.8	192.8	212.7	92.6

COLUMN MOMENTS (kNm)		1	2	3	4	5	6
ALL SPANS	Above						
LOADED	Below	263.2	-45.8	9.2	-9.2	45.8	-263.2
ODD SPANS	Above						
LOADED	Below	295.0	-168.2	146.9	-146.9	168.2	-295.0
EVEN SPANS	Above						
LOADED	Below	84.1	102.3	-133.6	133.6	-102.3	-84.1

Project	RC Structure		REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	Internal Roof (xx), from grids 1 to 6		GS	Jan-2009	3
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		Checked	Revision	Job No
	Originated from RCC33.xls on CD	© 1999 BCA for RCC	GS	-	

SPAN 1			LEFT	CENTRE	RIGHT
ACTIONS	ßb		1.000	1.091	0.900
	Be		900		3750
	Total M	kNm	114.3	413.2	449.2
	Mt max	kNm	190.7		794.6
MIDDLE STRIP	Width	mm	6600	3750	3750
	M	kNm	2.4	186.0	112.3
	d	mm	219.0	215.0	217.0
	As	mm²/m	4	558	334
	As deflection	mm²/m		629	334
			Provide Y12 @ 325 T1	Provide Y20 @ 450 B1	Provide Y16 @ 600 T1
	As prov	mm²/m	348	698	335
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /215.0 = 34.884 < 26.0 x 1.532 x 1.050 x 0.9 = 37.658		OK
			(As increased by 12.7 % for deflection)		
COLUMN STRIP	Width	mm	900	3750	3750
	M	kNm	114.3	227.3	336.9
	d	mm	217.0	215.0	217.0
	As	mm²/m	1517	683	1031
	As deflection	mm²/m		848	1031
			Provide Y16 @ 125 T1	Provide Y20 @ 350 B1	Provide Y16 @ 125:250 T1
	As prov	mm²/m	1608	898	1206
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /215.0 = 34.884 < 26.0 x 1.471 x 1.050 x 0.9 = 36.146		OK
			(As increased by 24.1 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 2			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.879	0.900
	Be		3750		3750
	Total M	kNm	449.2	279.2	370.5
	Mt max	kNm	794.6		794.6
MIDDLE STRIP	Width	mm	3750	3750	3750
	M	kNm	112.3	125.6	92.6
	d	mm	217.0	217.0	219.0
	As	mm²/m	334	374	273
	As deflection	mm²/m	334	455	
			Provide Y16 @ 600 T1	Provide Y16 @ 400 B1	Provide Y12 @ 325 T1
	As prov	mm²/m	335	503	348
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /217.0 = 34.562 < 26.0 x 1.559 x 1.050 x 0.9 = 38.315		OK
			(As increased by 21.7 % for deflection)		
COLUMN STRIP	Width	mm	3750	3750	3750
	M	kNm	336.9	153.6	277.9
	d	mm	217.0	215.0	217.0
	As	mm²/m	1031	461	837
	As deflection	mm²/m	1031	601	
			Provide Y16 @ 125:250 T1	Provide Y20 @ 500 B1	Provide Y16 @ 175:350 T1
	As prov	mm²/m	1206	628	862
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /215.0 = 34.884 < 26.0 x 1.478 x 1.050 x 0.9 = 36.319		OK
			(As increased by 30.4 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure		REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	Internal Roof (xx), from grids 1 to 6		GS	Jan-2009	4
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		Checked	Revision	Job No
	Originated from RCC33.xls on CD	© 1999 BCA for RCC	GS	-	

SPAN 3			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.940	0.900
	Be		3750		3750
	Total M	kNm	370.5	312.2	370.5
	Mt max	kNm	794.6		794.6
MIDDLE STRIP	Width	mm	3750	3750	3750
	M	kNm	92.6	140.5	92.6
	d	mm	219.0	217.0	219.0
	As	mm²/m	273	418	273
	As deflection	mm²/m	273	490	273
			Provide Y12 @ 325 T1	Provide Y16 @ 400 B1	Provide Y12 @ 325 T1
	As prov	mm²/m	348	503	348
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /217.0 = 34.562 < 26.0 x 1.444 x 1.050 x 0.9 = 35.501		OK
			(As increased by 17.2 % for deflection)		
COLUMN STRIP	Width	mm	3750	3750	3750
	M	kNm	277.9	171.7	277.9
	d	mm	217.0	215.0	217.0
	As	mm²/m	837	516	837
	As deflection	mm²/m	837	654	837
			Provide Y16 @ 175:350 T1	Provide Y20 @ 450 B1	Provide Y16 @ 175:350 T1
	As prov	mm²/m	862	698	862
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /215.0 = 34.884 < 26.0 x 1.498 x 1.050 x 0.9 = 36.806		OK
			(As increased by 26.8 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 4			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.879	0.900
	Be		3750		3750
	Total M	kNm	370.5	279.2	449.2
	Mt max	kNm	794.6		794.6
MIDDLE STRIP	Width	mm	3750	3750	3750
	M	kNm	92.6	125.6	112.3
	d	mm	219.0	217.0	217.0
	As	mm²/m	273	374	334
	As deflection	mm²/m	273	455	
			Provide Y12 @ 325 T1	Provide Y16 @ 400 B1	Provide Y16 @ 600 T1
	As prov	mm²/m	348	503	335
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /217.0 = 34.562 < 26.0 x 1.559 x 1.050 x 0.9 = 38.315		OK
			(As increased by 21.7 % for deflection)		
COLUMN STRIP	Width	mm	3750	3750	3750
	M	kNm	277.9	153.6	336.9
	d	mm	217.0	215.0	217.0
	As	mm²/m	837	461	1031
	As deflection	mm²/m	837	601	
			Provide Y16 @ 175:350 T1	Provide Y20 @ 500 B1	Provide Y16 @ 125:250 T1
	As prov	mm²/m	862	628	1206
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /215.0 = 34.884 < 26.0 x 1.478 x 1.050 x 0.9 = 36.319		OK
			(As increased by 30.4 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure	**REINFORCED CONCRETE COUNCIL**		
Client		Made by	Date	Page
Location	Internal Roof (xx), from grids 1 to 6	GS	Jan-2009	5
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked	Revision	Job No
	Originated from RCC33.xls on CD © 1999 BCA for RCC	GS	-	

SPAN 5			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	1.091	1.000
	Be		3750		900
	Total M	kNm	449.2	413.2	114.3
	Mt max	kNm	794.6		190.7
MIDDLE STRIP	Width	mm	3750	3750	6600
	M	kNm	112.3	186.0	2.4
	d	mm	217.0	215.0	219.0
	As	mm²/m	334	558	4
	As deflection	mm²/m	334	629	
			Provide Y16 @ 600 T1	Provide Y20 @ 450 B1	Provide Y12 @ 325 T1
	As prov	mm²/m	335	698	348
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /215.0 = 34.884 < 26.0 x 1.532 x 1.050 x 0.9 = 37.658		OK
			(As increased by 12.7 % for deflection)		
COLUMN STRIP	Width	mm	3750	3750	900
	M	kNm	336.9	227.3	114.3
	d	mm	217.0	215.0	217.0
	As	mm²/m	1031	683	1517
	As deflection	mm²/m	1031	848	
			Provide Y16 @ 125:250 T1	Provide Y20 @ 350 B1	Provide Y16 @ 125 T1
	As prov	mm²/m	1206	898	1608
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /215.0 = 34.884 < 26.0 x 1.471 x 1.050 x 0.9 = 36.146		OK
			(As increased by 24.1 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	Internal Roof (xx), from grids 1 to 6	GS	26-Jan-09	6
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WEIGHT of REINFORCEME		No Mid Strip	No Col Strip	Type	Dia	Length	Unit wt	Weight
TOP STEEL	Support 1	21		T	12	2550	0.888	47.5
			9	T	16	2550	1.578	36.2
	Span 1	12		T	12	5100	0.888	54.3
			12	T	12	5100	0.888	54.3
	Support 2	7		T	16	3750	1.578	41.4
			23	T	16	3750	1.578	136.1
	Span 2	12		T	12	5100	0.888	54.3
			12	T	12	5100	0.888	54.3
	Support 3	12		T	12	3750	0.888	40.0
			17	T	16	3750	1.578	100.6
	Span 3	12		T	12	5100	0.888	54.3
			12	T	12	5100	0.888	54.3
	Support 4	12		T	12	3750	0.888	40.0
			17	T	16	3750	1.578	100.6
	Span 4	12		T	12	5100	0.888	54.3
			12	T	12	5100	0.888	54.3
	Support 5	7		T	16	3750	1.578	41.4
			23	T	16	3750	1.578	136.1
	Span 5	12		T	12	5100	0.888	54.3
			12	T	12	5100	0.888	54.3
	Support 6	24		T	12	2550	0.888	54.3
			9	T	16	2550	1.578	36.2
BTM STEEL	Span 1	9		T	20	6925	2.466	153.7
			11	T	20	7850	2.466	213.0
	Span 2	10		T	16	6825	1.578	107.7
			8	T	20	7650	2.466	150.9
	Span 3	10		T	16	6825	1.578	107.7
			9	T	20	7650	2.466	169.8
	Span 4	10		T	16	6825	1.578	107.7
			8	T	20	7650	2.466	150.9
	Span 5	9		T	20	6925	2.466	153.7
			11	T	20	7850	2.466	213.0

SUMMARY *Rebar for single direction only. All figures approximate - see User Guide.*

TOTAL REINFORCEMENT IN BAY (kg) **2882** REINFORCEMENT DENSITY (kg/m^3) **40.5**

Project	RC Structure			REINFORCED CONCRETE COUNCIL		
Client				Made by	Date	Page
Location	Internal Roof (yy)	from grids 1	to 6	GS	26-Jan-09	1
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997			Checked	Revision	Job No
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MATERIALS

						COVERS	mm	TO LAYER
fcu	30	N/mm²	h agg	20	mm			
fyl	500	N/mm²	γs	1.15	steel	Top cover	45	2
fyv	500	N/mm²	γc	1.50	concrete	Btm cover	45	2

SPANS

	L (m)
SPAN 1	7.500
SPAN 2	7.500
SPAN 3	7.500
SPAN 4	7.500
SPAN 5	7.500
SPAN 6	

GEOMETRY

Bay type	INTERNAL	
Slab depth, h	250	mm
Panel width, b	7500	mm
End distance	225	from supt 1
End distance	225	from supt 6

PERIMETER LOADS *characteristic*

kN/m outside supports 1 & 6

LOADING PATTERN

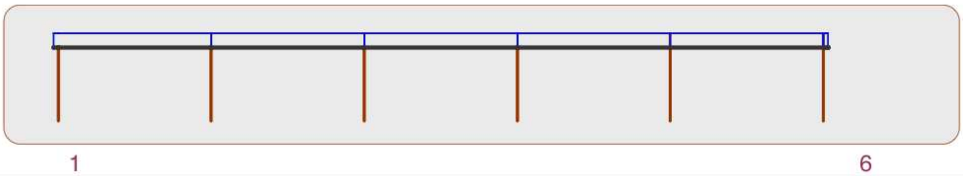
	min	max
DEAD	1.0	1.4
IMPOSED		1.6

SUPPORTS	ABOVE (m)	H (mm)	B (mm)	End Cond	BELOW (m)	H (mm)	B (mm)	End Cond
Support 1					2.700	450	450	F
Support 2					2.700	450	450	F
Support 3					2.700	450	450	F
Support 4					2.700	450	450	F
Support 5					2.700	450	450	F
Support 6					2.700	450	450	F
Support 7								

LOADING *UDLs (kN/m²) PLs (kN/m) Position (m)*

	Dead Load	Imposed Load	Position from left	Loaded Length		Dead Load	Imposed Load	Position from left	Loaded Length
Span 1					Span 4				
UDL	6.43	1.50	~~~~~	~~~~~	UDL	6.43	1.50	~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 2					Span 5				
UDL	6.43	1.50	~~~~~	~~~~~	UDL	6.43	1.50	~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 3					Span 6				
UDL	6.43	1.50	~~~~~	~~~~~	UDL			~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				

LOADING DIAGRAM



Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	Internal Roof (yy), from grids 1 to 6	GS	26-Jan-09	2
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked	Revision	Job No
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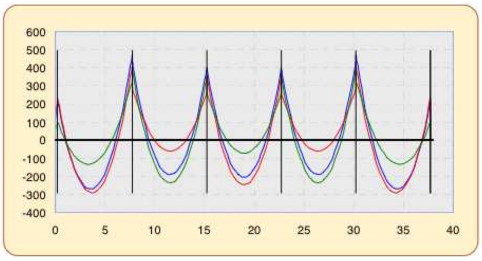
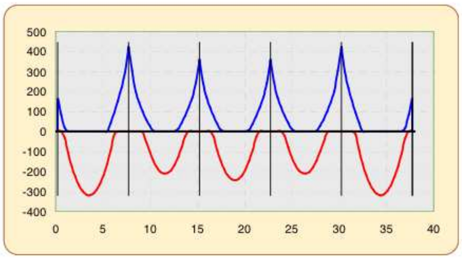
BENDING MOMENT DIAGRAMS (kNm)



Elastic Moments

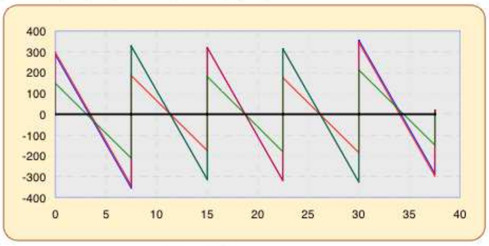


Redistributed Envelope

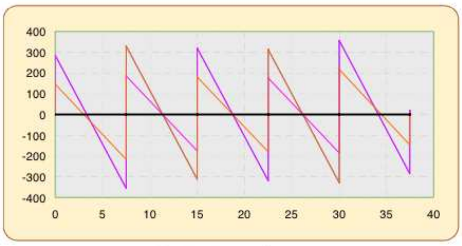
SUPPORT No	1	2	3	4	5	6	
Elastic M	226.1	469.6	397.0	397.0	469.6	226.1	~
Redistributed M	159.3	422.6	357.3	357.3	422.6	159.3	~
ßb	0.705	0.900	0.900	0.900	0.900	0.705	~
Redistribution		10.0%	10.0%	10.0%	10.0%		
End support reinf. Ø mm	16 *					16 *	

SPAN No	1	2	3	4	5	
Elastic M	289.83	234.99	247.21	234.99	289.83	~
Redistributed M	317.80	211.55	243.79	211.55	317.80	~
ßb	1.097	0.900	0.986	0.900	1.097	~

SHEARS FORCE DIAGRAMS (kN)



Elastic Shears



Redistributed Shears

SPAN No	1		2		3	
Elastic V	297.0	355.5	326.4	315.3	320.6	320.6
Redistributed V	285.5	355.8	329.3	311.8	320.6	320.6

SPAN No	4		5			
Elastic V	315.3	326.4	355.5	297.0	~	~
Redistributed V	311.8	329.3	355.8	285.5	~	~

REACTIONS (kN)

SUPPORT	1	2	3	4	5	6
ALL SPANS LOADED	304.7	684.9	632.4	632.4	684.9	304.7
ODD SPANS LOADED	304.6	540.9	497.0	497.0	540.9	304.6
EVEN SPANS LOADED	156.1	545.5	492.6	492.6	545.5	156.1
Veff for punching	380.9	720.2	639.5	639.5	720.2	380.9
Characteristic Dead	171.9	384.5	358.2	358.2	384.5	171.9
Characteristic Imposed	40.0	91.6	81.8	81.8	91.6	40.0

COLUMN MOMENTS (kNm)		1	2	3	4	5	6
ALL SPANS	Above						
LOADED	Below	205.5	-35.8	7.2	-7.2	35.8	-205.5
ODD SPANS	Above						
LOADED	Below	224.8	-110.3	91.0	-91.0	110.3	-224.8
EVEN SPANS	Above						
LOADED	Below	96.5	54.3	-79.7	79.7	-54.3	-96.5

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	Internal Roof (yy), from grids 1 to 6	GS	Jan-2009	3
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	Originated from RCC33.xls on CD © 1999 BCA for RCC	GS	-	

SPAN 1			LEFT	CENTRE	RIGHT
ACTIONS	ßb		1.000	1.097	0.900
	Be		900		3750
	Total M	kNm	97.3	317.8	350.7
	Mt max	kNm	157.2		654.9
MIDDLE STRIP	Width	mm	6600	3750	3750
	M	kNm	1.9	143.0	87.7
	d	mm	199.0	195.0	199.0
	As	mm²/m	4	473	284
	As deflection	mm²/m		583	284
			Provide Y12 @ 325 T2	Provide Y20 @ 500 B2	Provide Y12 @ 325 T2
	As prov	mm²/m	348	628	348
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /195.0 = 38.462 < 26.0 x 1.636 x 1.055 x 0.9 = 40.383		OK
			(As increased by 23.1 % for deflection)		
COLUMN STRIP	Width	mm	900	3750	3750
	M	kNm	97.3	174.8	263.0
	d	mm	197.0	195.0	197.0
	As	mm²/m	1429	579	883
	As deflection	mm²/m		800	883
			Provide Y16 @ 125 T2	Provide Y20 @ 375 B2	Provide Y16 @ 150:300 T2
	As prov	mm²/m	1608	838	1005
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /195.0 = 38.462 < 26.0 x 1.597 x 1.055 x 0.9 = 39.420		OK
			(As increased by 38.2 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 2			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.900	0.900
	Be		3750		3750
	Total M	kNm	350.7	211.6	289.3
	Mt max	kNm	654.9		654.9
MIDDLE STRIP	Width	mm	3750	3750	3750
	M	kNm	87.7	95.2	72.3
	d	mm	199.0	197.0	199.0
	As	mm²/m	284	312	235
	As deflection	mm²/m	284	396	
			Provide Y12 @ 325 T2	Provide Y16 @ 500 B2	Provide Y12 @ 325 T2
	As prov	mm²/m	348	402	348
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.567 x 1.055 x 0.9 = 38.696		OK
			(As increased by 26.8 % for deflection)		
COLUMN STRIP	Width	mm	3750	3750	3750
	M	kNm	263.0	116.4	216.9
	d	mm	197.0	197.0	197.0
	As	mm²/m	883	381	718
	As deflection	mm²/m	883	514	
			Provide Y16 @ 150:300 T2	Provide Y16 @ 375 B2	Provide Y16 @ 200:400 T2
	As prov	mm²/m	1005	536	754
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.598 x 1.055 x 0.9 = 39.446		OK
			(As increased by 34.8 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 3			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.986	0.900
	Be		3750		3750
	Total M	kNm	289.3	243.8	289.3
	Mt max	kNm	654.9		654.9
MIDDLE STRIP	Width	mm	3750	3750	3750
	M	kNm	72.3	109.7	72.3
	d	mm	199.0	197.0	199.0
	As	mm²/m	235	360	235
	As deflection	mm²/m	235	434	235
			Provide Y12 @ 325 T2	Provide Y16 @ 450 B2	Provide Y12 @ 325 T2
	As prov	mm²/m	348	447	348
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.583 x 1.055 x 0.9 = 39.084		OK
			(As increased by 20.7 % for deflection)		
COLUMN STRIP	Width	mm	3750	3750	3750
	M	kNm	216.9	134.1	216.9
	d	mm	197.0	195.0	197.0
	As	mm²/m	718	444	718
	As deflection	mm²/m	718	590	
			Provide Y16 @ 200:400 T2	Provide Y20 @ 500 B2	Provide Y16 @ 200:400 T2
	As prov	mm²/m	754	628	754
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /195.0 = 38.462 < 26.0 x 1.629 x 1.055 x 0.9 = 40.207		OK
			(As increased by 32.8 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 4			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.900	0.900
	Be		3750		3750
	Total M	kNm	289.3	211.6	350.7
	Mt max	kNm	654.9		654.9
MIDDLE STRIP	Width	mm	3750	3750	3750
	M	kNm	72.3	95.2	87.7
	d	mm	199.0	197.0	199.0
	As	mm²/m	235	312	284
	As deflection	mm²/m	235	396	
			Provide Y12 @ 325 T2	Provide Y16 @ 500 B2	Provide Y12 @ 325 T2
	As prov	mm²/m	348	402	348
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.567 x 1.055 x 0.9 = 38.696		OK
			(As increased by 26.8 % for deflection)		
COLUMN STRIP	Width	mm	3750	3750	3750
	M	kNm	216.9	116.4	263.0
	d	mm	197.0	197.0	197.0
	As	mm²/m	718	381	883
	As deflection	mm²/m	718	514	
			Provide Y16 @ 200:400 T2	Provide Y16 @ 375 B2	Provide Y16 @ 150:300 T2
	As prov	mm²/m	754	536	1005
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.598 x 1.055 x 0.9 = 39.446		OK
			(As increased by 34.8 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure	REINFORCED CONCRETE COUNCIL	REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	Internal Roof (yy), from grids 1 to 6		GS	Jan-2009	5
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		Checked	Revision	Job No
	Originated from RCC33.xls on CD © 1999 BCA for RCC		GS	-	

SPAN 5			LEFT	CENTRE	RIGHT	
ACTIONS	ßb		0.900	1.097	1.000	
	Be		3750		900	
	Total M	kNm	350.7	317.8	97.3	
	Mt max	kNm	654.9		157.2	
MIDDLE STRIP	Width	mm	3750	3750	6600	
	M	kNm	87.7	143.0	1.9	
	d	mm	199.0	195.0	199.0	
	As	mm²/m	284	473	4	
	As deflection	mm²/m	284	583		
			Provide Y12 @ 325 T2	Provide Y20 @ 500 B2	Provide Y12 @ 325 T2	
	As prov	mm²/m	348	628	348	
	Top steel			Provide Y12 @ 325 T2		
	Deflection		L/d = 7,500 /195.0 = 38.462 < 26.0 x 1.636 x 1.055 x 0.9 = 40.383			OK
			(As increased by 23.1 % for deflection)			
COLUMN STRIP	Width	mm	3750	3750	900	
	M	kNm	263.0	174.8	97.3	
	d	mm	197.0	195.0	197.0	
	As	mm²/m	883	579	1429	
	As deflection	mm²/m	883	800		
			Provide Y16 @ 150:300 T2	Provide Y20 @ 375 B2	Provide Y16 @ 125 T2	
	As prov	mm²/m	1005	838	1608	
	Top steel			Provide Y12 @ 325 T2		
	Deflection		L/d = 7,500 /195.0 = 38.462 < 26.0 x 1.597 x 1.055 x 0.9 = 39.420			OK
			(As increased by 38.2 % for deflection)			
CHECKS	% As		ok	ok	ok	
	Singly reinforced		ok	ok	ok	
	max S		ok	ok	ok	

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	Internal Roof (yy), from grids 1 to 6	GS	26-Jan-09	6
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked	Revision	Job No
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WEIGHT of REINFORCEME		No Mid Strip	No Col Strip	Type	Dia	Length	Unit wt	Weight
TOP STEEL	Support 1	21		T	12	2525	0.888	47.1
			9	T	16	2550	1.578	36.2
	Span 1	12		T	12	5100	0.888	54.3
			12	T	12	5100	0.888	54.3
	Support 2	12		T	12	3750	0.888	40.0
			19	T	16	3750	1.578	112.5
	Span 2	12		T	12	5100	0.888	54.3
			12	T	12	5100	0.888	54.3
	Support 3	12		T	12	3750	0.888	40.0
			15	T	16	3750	1.578	88.8
	Span 3	12		T	12	5100	0.888	54.3
			12	T	12	5100	0.888	54.3
	Support 4	12		T	12	3750	0.888	40.0
			15	T	16	3750	1.578	88.8
	Span 4	12		T	12	5100	0.888	54.3
			12	T	12	5100	0.888	54.3
	Support 5	12		T	12	3750	0.888	40.0
			19	T	16	3750	1.578	112.5
	Span 5	12		T	12	5100	0.888	54.3
			12	T	12	5100	0.888	54.3
	Support 6	24		T	12	2525	0.888	53.8
			9	T	16	2550	1.578	36.2
BTM STEEL	Span 1	8		T	20	6925	2.466	136.6
			10	T	20	7850	2.466	193.6
	Span 2	8		T	16	6825	1.578	86.2
			10	T	16	7650	1.578	120.7
	Span 3	9		T	16	6825	1.578	96.9
			8	T	20	7650	2.466	150.9
	Span 4	8		T	16	6825	1.578	86.2
			10	T	16	7650	1.578	120.7
	Span 5	8		T	20	6925	2.466	136.6
			10	T	20	7850	2.466	193.6

SUMMARY *Rebar for single direction only. All figures approximate - see User Guide.*

TOTAL REINFORCEMENT IN BAY (kg) **2601** REINFORCEMENT DENSITY (kg/m^3) **36.6**

Project	RC Structure		REINFORCED CONCRETE COUNCIL	
Client		Made by	Date	Page
Location	External Roof (xx) from grids 1 to 6	GS	26-Jan-09	1
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked	Revision	Job No
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MATERIALS

						COVERS	mm	TO LAYER
fcu	30	N/mm²	h agg	20	mm	Top cover	25	1
fyl	500	N/mm²	γs	1.15	steel	Btm cover	25	1
fyv	500	N/mm²	γc	1.50	concrete			

SPANS	L (m)	GEOMETRY		
SPAN 1	7.500	Bay type	EDGE	
SPAN 2	7.500	Slab depth, h	250	mm
SPAN 3	7.500	Panel width, b	3750	mm
SPAN 4	7.500	Edge distance	225	mm to C/L
SPAN 5	7.500	End distance	225	from supt 1
SPAN 6		End distance	225	from supt 6

PERIMETER LOADS *characteristic*

kN/m outside supports 1 & 6

kN/m along bay edge

LOADING PATTERN

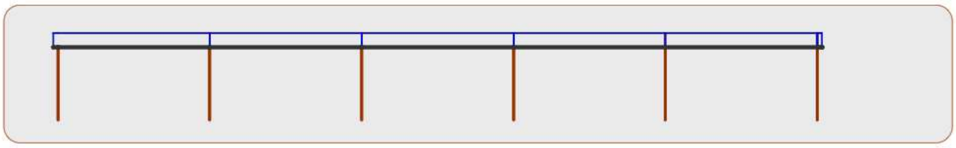
	min	max
DEAD	1.0	1.4
IMPOSED		1.6

SUPPORTS	ABOVE (m)	H (mm)	B (mm)	End Cond	BELOW (m)	H (mm)	B (mm)	End Cond
Support 1					2.700	450	450	F
Support 2					2.700	450	450	F
Support 3					2.700	450	450	F
Support 4					2.700	450	450	F
Support 5					2.700	450	450	F
Support 6					2.700	450	450	F
Support 7								

LOADING *UDLs (kN/m²) PLs (kN/m) Position (m)*

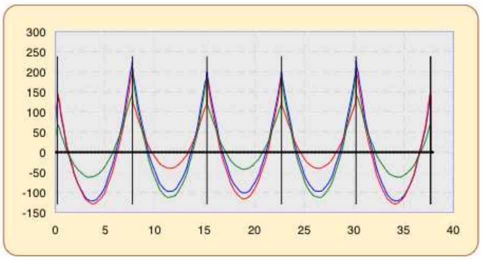
	Dead Load	Imposed Load	Position from left	Loaded Length		Dead Load	Imposed Load	Position from left	Loaded Length
Span 1					Span 4				
UDL	6.43	1.50	~~~~~	~~~~~	UDL	6.43	1.50	~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 2					Span 5				
UDL	6.43	1.50	~~~~~	~~~~~	UDL	6.43	1.50	~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 3					Span 6				
UDL	6.43	1.50	~~~~~	~~~~~	UDL			~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				

LOADING DIAGRAM

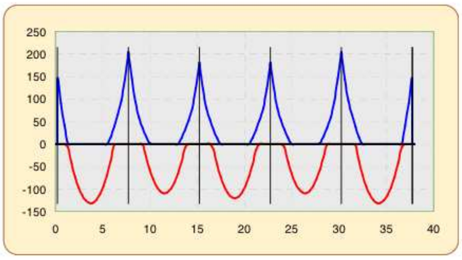


1 6

BENDING MOMENT DIAGRAMS (kNm)



Elastic Moments

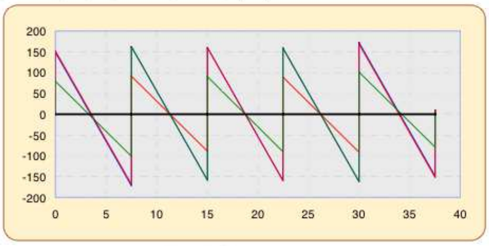


Redistributed Envelope

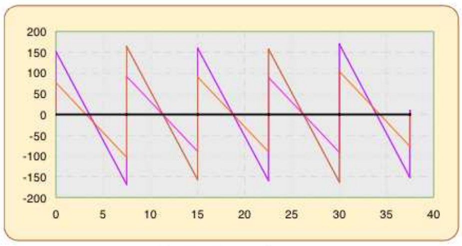
SUPPORT No	1	2	3	4	5	6	
Elastic M	144.2	226.0	199.7	199.7	226.0	144.2	~
Redistributed M	143.6	203.4	179.7	179.7	203.4	143.6	~
ßb	0.996	0.900	0.900	0.900	0.900	0.996	~
Redistribution		10.0%	10.0%	10.0%	10.0%		
End support reinf. Ø mm	16 *					16 *	

SPAN No	1	2	3	4	5	
Elastic M	128.18	112.67	115.43	112.67	128.18	~
Redistributed M	131.97	109.07	120.80	109.07	131.97	~
ßb	1.030	0.968	1.046	0.968	1.030	~

SHEARS FORCE DIAGRAMS (kN)



Elastic Shears



Redistributed Shears

SPAN No	1		2		3	
Elastic V	152.6	172.3	161.8	158.9	160.3	160.3
Redistributed V	152.3	169.3	163.4	157.1	160.3	160.3

SPAN No	4		5			
Elastic V	158.9	161.8	172.3	152.6	~	~
Redistributed V	157.1	163.4	169.3	152.3	~	~

REACTIONS (kN)

SUPPORT	1	2	3	4	5	6
ALL SPANS LOADED	160.9	332.8	317.4	317.4	332.8	160.9
ODD SPANS LOADED	161.9	259.7	249.6	249.6	259.7	161.9
EVEN SPANS LOADED	82.6	267.0	247.5	247.5	267.0	82.6
Veff for punching	202.4	439.8	400.3	400.3	439.8	202.4
Characteristic Dead	89.0	188.4	179.9	179.9	188.4	89.0
Characteristic Imposed	23.3	43.1	41.0	41.0	43.1	23.3

COLUMN MOMENTS (kNm)		1	2	3	4	5	6
ALL SPANS	Above						
LOADED	Below	134.5	-17.4	2.5	-2.5	17.4	-134.5
ODD SPANS	Above						
LOADED	Below	143.6	-68.8	59.3	-59.3	68.8	-143.6
EVEN SPANS	Above						
LOADED	Below	66.8	41.6	-55.4	55.4	-41.6	-66.8

Project	RC Structure	REINFORCED CONCRETE COUNCIL	REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	External Roof (xx), from grids 1 to 6		GS	Jan-2009	3
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		Checked	Revision	Job No
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SPAN 1

			LEFT	CENTRE	RIGHT
ACTIONS	ßb		1.000	1.030	0.900
	Be		675		2100
	Total M	kNm	110.5	132.0	167.7
	Mt max	kNm	143.0		445.0
MIDDLE STRIP	Width	mm	3300	1875	1875
	M	kNm	1.0	59.4	41.9
	d	mm	219.0	217.0	219.0
	As	mm²/m	3	353	247
	As deflection	mm²/m		363	247
			Provide Y12 @ 325 T1	Provide Y16 @ 550 B1	Provide Y12 @ 325 T1
	As prov	mm²/m	348	366	348
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /217.0 = 34.562 < 26.0 x 1.419 x 1.050 x 0.9 = 34.881		OK
			(As increased by 2.6 % for deflection)		
COLUMN STRIP	Width	mm	675	2100	2100
	M	kNm	110.5	72.6	125.8
	d	mm	217.0	217.0	217.0
	As	mm²/m	2045	386	668
	As deflection	mm²/m		404	668
			Provide Y16 @ 75 T1	Provide Y16 @ 450 B1	Provide Y16 @ 225:450 T1
	As prov	mm²/m	2681	447	670
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /217.0 = 34.562 < 26.0 x 1.558 x 1.050 x 0.9 = 38.281		OK
			(As increased by 4.7 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 2

			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.968	0.900
	Be		2100		2100
	Total M	kNm	167.7	109.1	145.5
	Mt max	kNm	445.0		436.8
MIDDLE STRIP	Width	mm	1875	1875	1875
	M	kNm	41.9	49.1	36.4
	d	mm	219.0	219.0	219.0
	As	mm²/m	247	289	214
	As deflection	mm²/m	247	301	
			Provide Y12 @ 325 T1	Provide Y12 @ 325 B1	Provide Y12 @ 325 T1
	As prov	mm²/m	348	348	348
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /219.0 = 34.247 < 26.0 x 1.649 x 1.050 x 0.9 = 40.525		OK
			(As increased by 4.1 % for deflection)		
COLUMN STRIP	Width	mm	2100	2100	2100
	M	kNm	125.8	60.0	109.1
	d	mm	217.0	217.0	215.0
	As	mm²/m	668	319	585
	As deflection	mm²/m	668	341	
			Provide Y16 @ 225:450 T1	Provide Y16 @ 550 B1	Provide Y20 @ 400:800 T1
	As prov	mm²/m	670	366	589
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /217.0 = 34.562 < 26.0 x 1.528 x 1.050 x 0.9 = 37.551		OK
			(As increased by 6.9 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 3			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	1.046	0.900
	Be		2100		2100
	Total M	kNm	145.5	120.8	145.5
	Mt max	kNm	436.8		436.8
MIDDLE STRIP	Width	mm	1875	1875	1875
	M	kNm	36.4	54.4	36.4
	d	mm	219.0	219.0	219.0
	As	mm²/m	214	320	214
	As deflection	mm²/m	214	320	214
			Provide Y12 @ 325 T1	Provide Y12 @ 325 B1	Provide Y12 @ 325 T1
	As prov	mm²/m	348	348	348
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /219.0 = 34.247 < 26.0 x 1.567 x 1.050 x 0.9 = 38.516		OK
			(As increased by 0.0 % for deflection)		
COLUMN STRIP	Width	mm	2100	2100	2100
	M	kNm	109.1	66.4	109.1
	d	mm	215.0	217.0	215.0
	As	mm²/m	585	353	585
	As deflection	mm²/m	585	356	
			Provide Y20 @ 400:800 T1	Provide Y16 @ 550 B1	Provide Y20 @ 400:800 T1
	As prov	mm²/m	589	366	589
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /217.0 = 34.562 < 26.0 x 1.448 x 1.050 x 0.9 = 35.595		OK
			(As increased by 1.0 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 4			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.968	0.900
	Be		2100		2100
	Total M	kNm	145.5	109.1	167.7
	Mt max	kNm	436.8		445.0
MIDDLE STRIP	Width	mm	1875	1875	1875
	M	kNm	36.4	49.1	41.9
	d	mm	219.0	219.0	219.0
	As	mm²/m	214	289	247
	As deflection	mm²/m	214	301	
			Provide Y12 @ 325 T1	Provide Y12 @ 325 B1	Provide Y12 @ 325 T1
	As prov	mm²/m	348	348	348
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /219.0 = 34.247 < 26.0 x 1.649 x 1.050 x 0.9 = 40.525		OK
			(As increased by 4.1 % for deflection)		
COLUMN STRIP	Width	mm	2100	2100	2100
	M	kNm	109.1	60.0	125.8
	d	mm	215.0	217.0	217.0
	As	mm²/m	585	319	668
	As deflection	mm²/m	585	341	
			Provide Y20 @ 400:800 T1	Provide Y16 @ 550 B1	Provide Y16 @ 225:450 T1
	As prov	mm²/m	589	366	670
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /217.0 = 34.562 < 26.0 x 1.528 x 1.050 x 0.9 = 37.551		OK
			(As increased by 6.9 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure		REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	External Roof (xx), from grids 1 to 6		GS	Jan-2009	5
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		Checked	Revision	Job No
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SPAN 5			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	1.030	1.000
	Be		2100		675
	Total M	kNm	167.7	132.0	110.5
	Mt max	kNm	445.0		143.0
MIDDLE STRIP	Width	mm	1875	1875	3300
	M	kNm	41.9	59.4	1.0
	d	mm	219.0	217.0	219.0
	As	mm²/m	247	353	3
	As deflection	mm²/m	247	363	
			Provide Y12 @ 325 T1	Provide Y16 @ 550 B1	Provide Y12 @ 325 T1
	As prov	mm²/m	348	366	348
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /217.0 = 34.562 < 26.0 x 1.419 x 1.050 x 0.9 = 34.881		OK
			(As increased by 2.6 % for deflection)		
COLUMN STRIP	Width	mm	2100	2100	675
	M	kNm	125.8	72.6	110.5
	d	mm	217.0	217.0	217.0
	As	mm²/m	668	386	2045
	As deflection	mm²/m	668	404	
			Provide Y16 @ 225:450 T1	Provide Y16 @ 450 B1	Provide Y16 @ 75 T1
	As prov	mm²/m	670	447	2681
	Top steel			Provide Y12 @ 325 T1	
	Deflection		L/d = 7,500 /217.0 = 34.562 < 26.0 x 1.558 x 1.050 x 0.9 = 38.281		OK
			(As increased by 4.7 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	External Roof (xx), from grids 1 to 6	GS	26-Jan-09	6
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked	Revision	Job No
	Originated from RCC33.xls on CD © 1999 BCA for RCC	GS	-	

WEIGHT of REINFORCEME		No Mid Strip	Col Strip	Type	Dia	Length	Unit wt	Weight
TOP STEEL	Support 1	11		T	12	2550	0.888	24.9
			10	T	16	2550	1.578	40.2
	Span 1	6		T	12	5100	0.888	27.2
			7	T	12	5100	0.888	31.7
	Support 2	6		T	12	3750	0.888	20.0
			7	T	16	3750	1.578	41.4
	Span 2	6		T	12	5100	0.888	27.2
			7	T	12	5100	0.888	31.7
	Support 3	6		T	12	3750	0.888	20.0
			4	T	20	3750	2.466	37.0
	Span 3	6		T	12	5100	0.888	27.2
			7	T	12	5100	0.888	31.7
	Support 4	6		T	12	3750	0.888	20.0
			4	T	20	3750	2.466	37.0
	Span 4	6		T	12	5100	0.888	27.2
			7	T	12	5100	0.888	31.7
	Support 5	6		T	12	3750	0.888	20.0
			7	T	16	3750	1.578	41.4
	Span 5	6		T	12	5100	0.888	27.2
			7	T	12	5100	0.888	31.7
	Support 6	12		T	12	2550	0.888	27.2
			10	T	16	2550	1.578	40.2
BTM STEEL	Span 1	4		T	16	6925	1.578	43.7
			5	T	16	7850	1.578	61.9
	Span 2	6		T	12	6825	0.888	36.4
			4	T	16	7650	1.578	48.3
	Span 3	6		T	12	6825	0.888	36.4
			4	T	16	7650	1.578	48.3
	Span 4	6		T	12	6825	0.888	36.4
			4	T	16	7650	1.578	48.3
	Span 5	4		T	16	6925	1.578	43.7
			5	T	16	7850	1.578	61.9

SUMMARY *Rebar for single direction only. All figures approximate - see User Guide.*

TOTAL REINFORCEMENT IN BAY (kg) **1129** REINFORCEMENT DENSITY (kg/m^3) **31.7**

Project	RC Structure		REINFORCED CONCRETE COUNCIL
Client			Made by: GS · Date: 26-Jan-09 · Page: 1
Location	External Roof (yy)	from grids 1 to 6	
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		Checked: GS · Revision: - · Job No
	Originated from RCC33.xls on CD	© 1999 BCA for RCC	

MATERIALS

fcu	30	N/mm²	h agg	20	mm
fyl	500	N/mm²	γs	1.15	steel
fyv	500	N/mm²	γc	1.50	concrete

COVERS

	mm	TO LAYER
Top cover	45	2
Btm cover	45	2

SPANS

	L (m)
SPAN 1	7.500
SPAN 2	7.500
SPAN 3	7.500
SPAN 4	7.500
SPAN 5	7.500
SPAN 6	

GEOMETRY

Bay type	EDGE	
Slab depth, h	250	mm
Panel width, b	3750	mm
Edge distance	225	mm to C/L
End distance	225	from supt 1
End distance	225	from supt 6

PERIMETER LOADS *characteristic*

kN/m outside supports 1 & 6
kN/m along bay edge

LOADING PATTERN

	min	max
DEAD	1.0	1.4
IMPOSED		1.6

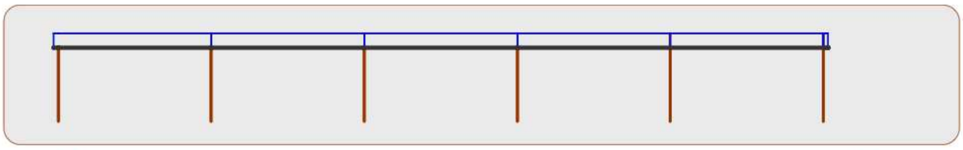
SUPPORTS

	ABOVE (m)	H (mm)	B (mm)	End Cond	BELOW (m)	H (mm)	B (mm)	End Cond
Support 1					2.700	450	450	F
Support 2					2.700	450	450	F
Support 3					2.700	450	450	F
Support 4					2.700	450	450	F
Support 5					2.700	450	450	F
Support 6					2.700	450	450	F
Support 7								

LOADING *UDLs (kN/m²) PLs (kN/m) Position (m)*

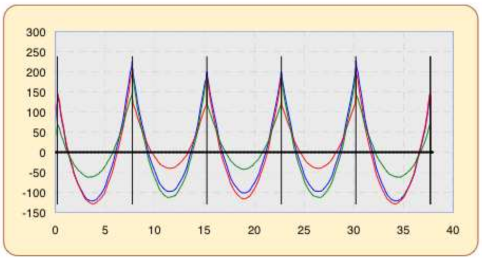
	Dead Load	Imposed Load	Position from left	Loaded Length
Span 1				
UDL	6.43	1.50	~~~~~	~~~~~
PL 1				~~~~~
PL 2				~~~~~
Part UDL				
Span 2				
UDL	6.43	1.50	~~~~~	~~~~~
PL 1				~~~~~
PL 2				~~~~~
Part UDL				
Span 3				
UDL	6.43	1.50	~~~~~	~~~~~
PL 1				~~~~~
PL 2				~~~~~
Part UDL				
Span 4				
UDL	6.43	1.50	~~~~~	~~~~~
PL 1				~~~~~
PL 2				~~~~~
Part UDL				
Span 5				
UDL	6.43	1.50	~~~~~	~~~~~
PL 1				~~~~~
PL 2				~~~~~
Part UDL				
Span 6				
UDL			~~~~~	~~~~~
PL 1				~~~~~
PL 2				~~~~~
Part UDL				

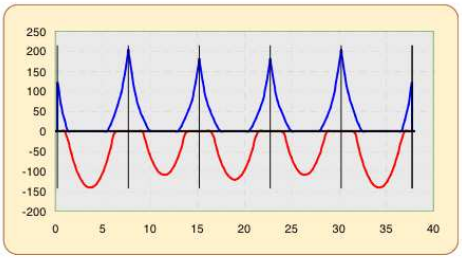
LOADING DIAGRAM



Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	External Roof (yy), from grids 1 to 6	GS	26-Jan-09	2
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked	Revision	Job No
	Originated from RCC33.xls on CD © 1999 BCA for RCC	GS	-	

BENDING MOMENT DIAGRAMS (kNm)





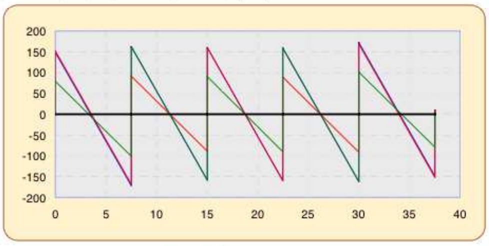
Elastic Moments

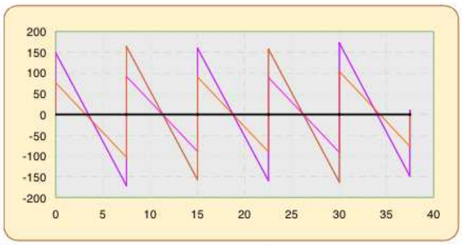
Redistributed Envelope

SUPPORT No	1	2	3	4	5	6	
Elastic M	144.2	226.0	199.7	199.7	226.0	144.2	~
Redistributed M	119.0	203.4	179.7	179.7	203.4	119.0	~
ßb	0.825	0.900	0.900	0.900	0.900	0.825	~
Redistribution		10.0%	10.0%	10.0%	10.0%		
End support reinf. Ø mm	16 *					16 *	

SPAN No	1	2	3	4	5	
Elastic M	128.18	112.67	115.43	112.67	128.18	~
Redistributed M	141.07	109.07	120.80	109.07	141.07	~
ßb	1.101	0.968	1.046	0.968	1.101	~

SHEARS FORCE DIAGRAMS (kN)





Elastic Shears

Redistributed Shears

SPAN No	1		2		3	
Elastic V	152.6	172.3	161.8	158.9	160.3	160.3
Redistributed V	149.0	171.6	163.4	157.1	160.3	160.3

SPAN No	4		5			
Elastic V	158.9	161.8	172.3	152.6	~	~
Redistributed V	157.1	163.4	171.6	149.0	~	~

REACTIONS (kN)

SUPPORT	1	2	3	4	5	6
ALL SPANS LOADED	158.6	335.0	317.4	317.4	335.0	158.6
ODD SPANS LOADED	158.6	263.1	249.6	249.6	263.1	158.6
EVEN SPANS LOADED	82.6	267.0	247.5	247.5	267.0	82.6
Veff for punching	198.3	442.8	400.3	400.3	442.8	198.3
Characteristic Dead	89.0	188.4	179.9	179.9	188.4	89.0
Characteristic Imposed	21.3	44.5	41.0	41.0	44.5	21.3

COLUMN MOMENTS (kNm)		1	2	3	4	5	6
ALL SPANS	Above						
LOADED	Below	134.5	-17.4	2.5	-2.5	17.4	-134.5
ODD SPANS	Above						
LOADED	Below	143.6	-68.8	59.3	-59.3	68.8	-143.6
EVEN SPANS	Above						
LOADED	Below	66.8	41.6	-55.4	55.4	-41.6	-66.8

Project	RC Structure	REINFORCED CONCRETE COUNCIL	
		Made by	Date	Page
Client		GS	Jan-2009	3
Location	External Roof (yy), from grids 1 to 6	Checked	Revision	Job No
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	GS	-	
	Originated from RCC33.xls on CD © 1999 BCA for RCC			

SPAN 1			LEFT	CENTRE	RIGHT
ACTIONS	ßb		1.000	1.101	0.900
	Be		675		2100
	Total M	kNm	86.5	141.1	167.7
	Mt max	kNm	117.9		366.7
MIDDLE STRIP	Width	mm	3300	1875	1875
	M	kNm	1.0	63.5	41.9
	d	mm	199.0	197.0	199.0
	As	mm²/m	4	416	272
	As deflection	mm²/m	474		272
			Provide Y12 @ 325 T2	Provide Y16 @ 400 B2	Provide Y12 @ 325 T2
	As prov	mm²/m	348	503	348
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.614 x 1.055 x 0.9 = 39.846		OK
			(As increased by 13.9 % for deflection)		
COLUMN STRIP	Width	mm	675	2100	2100
	M	kNm	86.5	77.6	125.8
	d	mm	197.0	195.0	197.0
	As	mm²/m	1745	459	745
	As deflection	mm²/m		554	745
			Provide Y16 @ 100 T2	Provide Y20 @ 550 B2	Provide Y16 @ 200:400 T2
	As prov	mm²/m	2011	571	754
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /195.0 = 38.462 < 26.0 x 1.591 x 1.055 x 0.9 = 39.275		OK
			(As increased by 20.8 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 2			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.968	0.900
	Be		2100		2100
	Total M	kNm	167.7	109.1	145.5
	Mt max	kNm	366.7		366.7
MIDDLE STRIP	Width	mm	1875	1875	1875
	M	kNm	41.9	49.1	36.4
	d	mm	199.0	197.0	199.0
	As	mm²/m	272	322	236
	As deflection	mm²/m	272	383	
			Provide Y12 @ 325 T2	Provide Y16 @ 500 B2	Provide Y12 @ 325 T2
	As prov	mm²/m	348	402	348
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.617 x 1.055 x 0.9 = 39.912		OK
			(As increased by 18.9 % for deflection)		
COLUMN STRIP	Width	mm	2100	2100	2100
	M	kNm	125.8	60.0	109.1
	d	mm	197.0	197.0	197.0
	As	mm²/m	745	351	640
	As deflection	mm²/m	745	428	
			Provide Y16 @ 200:400 T2	Provide Y16 @ 450 B2	Provide Y16 @ 225:450 T2
	As prov	mm²/m	754	447	670
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.602 x 1.055 x 0.9 = 39.540		OK
			(As increased by 22.0 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure		REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	External Roof (yy), from grids 1 to 6		GS	Jan-2009	4
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		Checked	Revision	Job No
	Originated from RCC33.xls on CD	© 1999 BCA for RCC	GS	-	

SPAN 3			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	1.046	0.900
	Be		2100		2100
	Total M	kNm	145.5	120.8	145.5
	Mt max	kNm	366.7		366.7
MIDDLE STRIP	Width	mm	1875	1875	1875
	M	kNm	36.4	54.4	36.4
	d	mm	199.0	197.0	199.0
	As	mm²/m	236	356	236
	As deflection	mm²/m	236	404	
			Provide Y12 @ 325 T2	Provide Y16 @ 450 B2	Provide Y12 @ 325 T2
	As prov	mm²/m	348	447	348
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.678 x 1.055 x 0.9 = 41.435		OK
			(As increased by 13.4 % for deflection)		
COLUMN STRIP	Width	mm	2100	2100	2100
	M	kNm	109.1	66.4	109.1
	d	mm	197.0	197.0	197.0
	As	mm²/m	640	389	640
	As deflection	mm²/m	640	454	
			Provide Y16 @ 225:450 T2	Provide Y16 @ 400 B2	Provide Y16 @ 225:450 T2
	As prov	mm²/m	670	503	670
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.670 x 1.055 x 0.9 = 41.241		OK
			(As increased by 16.8 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 4			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.968	0.900
	Be		2100		2100
	Total M	kNm	145.5	109.1	167.7
	Mt max	kNm	366.7		366.7
MIDDLE STRIP	Width	mm	1875	1875	1875
	M	kNm	36.4	49.1	41.9
	d	mm	199.0	197.0	199.0
	As	mm²/m	236	322	272
	As deflection	mm²/m	236	383	
			Provide Y12 @ 325 T2	Provide Y16 @ 500 B2	Provide Y12 @ 325 T2
	As prov	mm²/m	348	402	348
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.617 x 1.055 x 0.9 = 39.912		OK
			(As increased by 18.9 % for deflection)		
COLUMN STRIP	Width	mm	2100	2100	2100
	M	kNm	109.1	60.0	125.8
	d	mm	197.0	197.0	197.0
	As	mm²/m	640	351	745
	As deflection	mm²/m	640	428	
			Provide Y16 @ 225:450 T2	Provide Y16 @ 450 B2	Provide Y16 @ 200:400 T2
	As prov	mm²/m	670	447	754
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.602 x 1.055 x 0.9 = 39.540		OK
			(As increased by 22.0 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure		REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	External Roof (yy), from grids 1 to 6		GS	Jan-2009	5
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		Checked	Revision	Job No
	Originated from RCC33.xls on CD	© 1999 BCA for RCC	GS	-	

SPAN 5			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	1.101	1.000
	Be		2100		675
	Total M	kNm	167.7	141.1	86.5
	Mt max	kNm	366.7		117.9
MIDDLE STRIP	Width	mm	1875	1875	3300
	M	kNm	41.9	63.5	1.0
	d	mm	199.0	197.0	199.0
	As	mm²/m	272	416	4
	As deflection	mm²/m	272	474	
			Provide Y12 @ 325 T2	Provide Y16 @ 400 B2	Provide Y12 @ 325 T2
	As prov	mm²/m	348	503	348
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /197.0 = 38.071 < 26.0 x 1.614 x 1.055 x 0.9 = 39.846		OK
			(As increased by 13.9 % for deflection)		
COLUMN STRIP	Width	mm	2100	2100	675
	M	kNm	125.8	77.6	86.5
	d	mm	197.0	195.0	197.0
	As	mm²/m	745	459	1745
	As deflection	mm²/m	745	554	
			Provide Y16 @ 200:400 T2	Provide Y20 @ 550 B2	Provide Y16 @ 100 T2
	As prov	mm²/m	754	571	2011
	Top steel			Provide Y12 @ 325 T2	
	Deflection		L/d = 7,500 /195.0 = 38.462 < 26.0 x 1.591 x 1.055 x 0.9 = 39.275		OK
			(As increased by 20.8 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	External Roof (yy), from grids 1 to 6	GS	26-Jan-09	6
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked	Revision	Job No
	Originated from RCC33.xls on CD © 1999 BCA for RCC	GS	-	

WEIGHT of REINFORCEME		No Mid Strip	Col Strip	Type	Dia	Length	Unit wt	Weight
TOP STEEL	Support 1	11		T	12	2525	0.888	24.7
			8	T	16	2550	1.578	32.2
	Span 1	6		T	12	5100	0.888	27.2
			7	T	12	5100	0.888	31.7
	Support 2	6		T	12	3750	0.888	20.0
			8	T	16	3750	1.578	47.4
	Span 2	6		T	12	5100	0.888	27.2
			7	T	12	5100	0.888	31.7
	Support 3	6		T	12	3750	0.888	20.0
			7	T	16	3750	1.578	41.4
	Span 3	6		T	12	5100	0.888	27.2
			7	T	12	5100	0.888	31.7
	Support 4	6		T	12	3750	0.888	20.0
			7	T	16	3750	1.578	41.4
	Span 4	6		T	12	5100	0.888	27.2
			7	T	12	5100	0.888	31.7
	Support 5	6		T	12	3750	0.888	20.0
			8	T	16	3750	1.578	47.4
	Span 5	6		T	12	5100	0.888	27.2
			7	T	12	5100	0.888	31.7
	Support 6	12		T	12	2525	0.888	26.9
			8	T	16	2550	1.578	32.2
BTM STEEL	Span 1	5		T	16	6925	1.578	54.6
			4	T	20	7850	2.466	77.4
	Span 2	4		T	16	6825	1.578	43.1
			5	T	16	7650	1.578	60.4
	Span 3	5		T	16	6825	1.578	53.9
			6	T	16	7650	1.578	72.4
	Span 4	4		T	16	6825	1.578	43.1
			5	T	16	7650	1.578	60.4
	Span 5	5		T	16	6925	1.578	54.6
			4	T	20	7850	2.466	77.4

SUMMARY *Rebar for single direction only. All figures approximate - see User Guide.*

TOTAL REINFORCEMENT IN BAY (kg) **1265** REINFORCEMENT DENSITY (kg/m^3) 35.6

Project	RC Structure		REINFORCED CONCRETE COUNCIL
Client		Made by GS	Date 26-Jan-09	Page 1
Location	Internal Floor (xx) from grids 1 to 6		
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked GS	Revision -	Job No
	Originated from RCC33.xls on CD © 1999 BCA for RCC		

MATERIALS

							COVERS	mm	TO LAYER
fcu	30	N/mm²	h agg	20	mm		Top cover	25	1
fyl	500	N/mm²	γ_s	1.15	steel		Btm cover	25	1
fyv	500	N/mm²	γ_c	1.50	concrete				

SPANS	L (m)
SPAN 1	7.500
SPAN 2	7.500
SPAN 3	7.500
SPAN 4	7.500
SPAN 5	7.500
SPAN 6	

GEOMETRY

Bay type	INTERNAL	
Slab depth, h	300	mm
Panel width, b	7500	mm
End distance	225	from supt 1
End distance	225	from supt 6

PERIMETER LOADS *characteristic*

7.24 *kN/m outside supports 1 & 6*

LOADING PATTERN

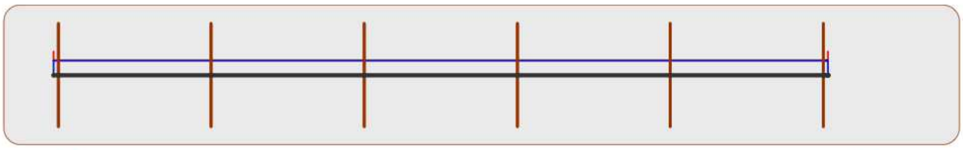
	min	max
DEAD	1.0	1.4
IMPOSED		1.6

SUPPORTS	ABOVE (m)	H (mm)	B (mm)	End Cond	BELOW (m)	H (mm)	B (mm)	End Cond
Support 1	2.700	450	450	F	2.700	450	450	F
Support 2	2.700	450	450	F	2.700	450	450	F
Support 3	2.700	450	450	F	2.700	450	450	F
Support 4	2.700	450	450	F	2.700	450	450	F
Support 5	2.700	450	450	F	2.700	450	450	F
Support 6	2.700	450	450	F	2.700	450	450	F
Support 7								

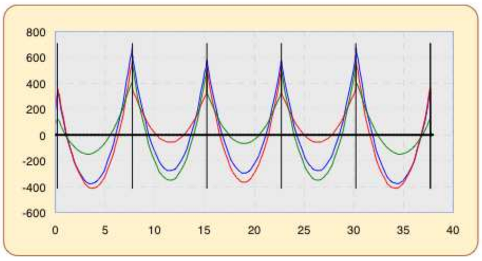
LOADING *UDLs (kN/m²) PLs (kN/m) Position (m)*

	Dead Load	Imposed Load	Position from left	Loaded Length		Dead Load	Imposed Load	Position from left	Loaded Length
Span 1					Span 4				
UDL	7.78	3.50	~~~~~	~~~~~	UDL	7.78	3.50	~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 2					Span 5				
UDL	7.78	3.50	~~~~~	~~~~~	UDL	7.78	3.50	~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 3					Span 6				
UDL	7.78	3.50	~~~~~	~~~~~	UDL			~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				

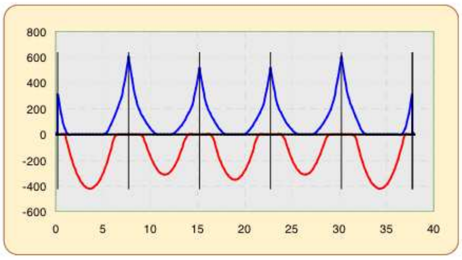
LOADING DIAGRAM



BENDING MOMENT DIAGRAMS (kNm)



Elastic Moments

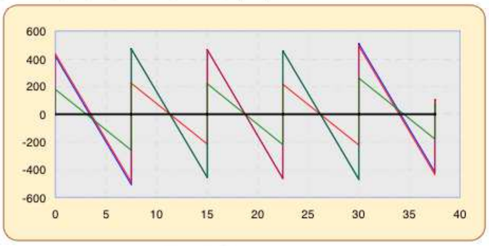


Redistributed Envelope

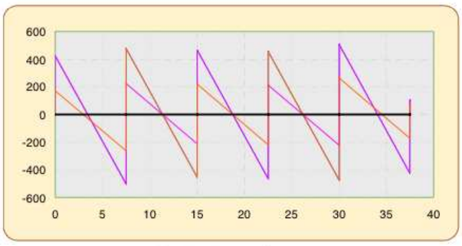
SUPPORT No	1	2	3	4	5	6	
Elastic M	358.5	671.3	575.2	575.2	671.4	358.6	~
Redistributed M	309.0	604.1	517.7	517.7	604.3	309.0	~
ßb	0.862	0.900	0.900	0.900	0.900	0.862	~
Redistribution		10.0%	10.0%	10.0%	10.0%		
End support reinf. Ø mm	16 *					16 *	

SPAN No	1	2	3	4	5	
Elastic M	412.12	350.29	364.41	350.27	412.24	~
Redistributed M	422.59	312.22	351.62	312.20	422.76	~
ßb	1.025	0.891	0.965	0.891	1.026	~

SHEARS FORCE DIAGRAMS (kN)



Elastic Shears



Redistributed Shears

SPAN No	1		2		3	
Elastic V	436.5	509.4	471.0	457.3	463.6	463.6
Redistributed V	424.3	503.9	475.2	453.0	463.6	463.6

SPAN No	4		5			
Elastic V	457.3	471.0	509.5	436.6	~	~
Redistributed V	453.0	475.2	504.0	424.4	~	~

REACTIONS (kN)

SUPPORT	1	2	3	4	5	6
ALL SPANS LOADED	528.1	978.2	915.8	915.7	978.3	528.2
ODD SPANS LOADED	527.3	727.3	677.5	677.5	727.5	527.4
EVEN SPANS LOADED	240.6	738.4	671.7	671.7	738.5	240.7
Veff for punching	660.2	1022.1	924.1	924.1	1022.3	660.3
Characteristic Dead	264.9	462.0	433.9	433.9	462.1	265.0
Characteristic Imposed	98.3	207.1	192.6	192.6	207.1	98.3

COLUMN MOMENTS (kNm)		1	2	3	4	5	6
ALL SPANS	Above	154.0	-25.4	4.8	-4.8	25.4	-154.1
LOADED	Below	154.0	-25.4	4.8	-4.8	25.4	-154.1
ODD SPANS	Above	172.4	-96.1	83.9	-83.9	96.1	-172.4
LOADED	Below	172.4	-96.1	83.9	-83.9	96.1	-172.4
EVEN SPANS	Above	53.1	58.9	-76.8	76.8	-58.9	-53.2
LOADED	Below	53.1	58.9	-76.8	76.8	-58.9	-53.2

Project	RC Structure	REINFORCED CONCRETE COUNCIL	REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	Internal Floor (xx), from grids 1 to 6		GS	Jan-2009	3
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		Checked	Revision	Job No
	Originated from RCC33.xls on CD © 1999 BCA for RCC		GS	-	

SPAN 1			LEFT	CENTRE	RIGHT
ACTIONS	ßb		1.000	1.025	0.900
	Be		900		3750
	Total M	kNm	216.6	422.6	500.4
	Mt max	kNm	288.7		1185.0
MIDDLE STRIP	Width	mm	6600	3750	3750
	M	kNm	4.8	190.2	125.1
	d	mm	267.0	267.0	267.0
	As	mm²/m	7	460	302
	As deflection	mm²/m		460	302
			Provide Y16 @ 500 T1	Provide Y16 @ 400 B1	Provide Y16 @ 500 T1
	As prov	mm²/m	402	503	402
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.479 x 1.048 x 0.9 = 36.261		OK
COLUMN STRIP	Width	mm	900	3750	3750
	M	kNm	216.6	232.4	375.3
	d	mm	267.0	265.0	265.0
	As	mm²/m	2429	566	920
	As deflection	mm²/m		566	920
			Provide Y16 @ 75 T1	Provide Y20 @ 550 B1	Provide Y20 @ 250:500 T1
	As prov	mm²/m	2681	571	942
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /265.0 = 28.302 < 26.0 x 1.273 x 1.048 x 0.9 = 31.221		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 2			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.891	0.900
	Be		3750		3750
	Total M	kNm	500.4	312.2	419.1
	Mt max	kNm	1185.0		1185.0
MIDDLE STRIP	Width	mm	3750	3750	3750
	M	kNm	125.1	140.5	104.8
	d	mm	267.0	267.0	267.0
	As	mm²/m	302	340	253
	As deflection	mm²/m	302	340	
			Provide Y16 @ 500 T1	Provide Y16 @ 500 B1	Provide Y16 @ 500 T1
	As prov	mm²/m	402	402	402
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.491 x 1.048 x 0.9 = 36.568		OK
			(As increased by 0.0 % for deflection)		
COLUMN STRIP	Width	mm	3750	3750	3750
	M	kNm	375.3	171.7	314.3
	d	mm	265.0	267.0	265.0
	As	mm²/m	920	415	766
	As deflection	mm²/m	920	423	
			Provide Y20 @ 250:500 T1	Provide Y16 @ 450 B1	Provide Y20 @ 300:600 T1
	As prov	mm²/m	942	447	785
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.249 x 1.048 x 0.9 = 30.635		OK
			(As increased by 2.0 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 3			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.965	0.900
	Be		3750		3750
	Total M	kNm	419.1	351.6	419.1
	Mt max	kNm	1185.0		1185.0
MIDDLE STRIP	Width	mm	3750	3750	3750
	M	kNm	104.8	158.2	104.8
	d	mm	267.0	267.0	267.0
	As	mm²/m	253	383	253
	As deflection	mm²/m	253	383	
			Provide Y16 @ 500 T1	Provide Y16 @ 500 B1	Provide Y16 @ 500 T1
	As prov	mm²/m	402	402	402
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.378 x 1.048 x 0.9 = 33.797		OK
COLUMN STRIP	Width	mm	3750	3750	3750
	M	kNm	314.3	193.4	314.3
	d	mm	265.0	267.0	265.0
	As	mm²/m	766	468	766
	As deflection	mm²/m	766	468	
			Provide Y20 @ 300:600 T1	Provide Y16 @ 400 B1	Provide Y20 @ 300:600 T1
	As prov	mm²/m	785	503	785
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.349 x 1.048 x 0.9 = 33.071		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 4			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.891	0.900
	Be		3750		3750
	Total M	kNm	419.1	312.2	500.5
	Mt max	kNm	1185.0		1185.0
MIDDLE STRIP	Width	mm	3750	3750	3750
	M	kNm	104.8	140.5	125.1
	d	mm	267.0	267.0	267.0
	As	mm²/m	253	340	303
	As deflection	mm²/m	253	340	
			Provide Y16 @ 500 T1	Provide Y16 @ 500 B1	Provide Y16 @ 500 T1
	As prov	mm²/m	402	402	402
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.492 x 1.048 x 0.9 = 36.570		OK
			(As increased by 0.0 % for deflection)		
COLUMN STRIP	Width	mm	3750	3750	3750
	M	kNm	314.3	171.7	375.4
	d	mm	265.0	267.0	265.0
	As	mm²/m	766	415	920
	As deflection	mm²/m	766	423	
			Provide Y20 @ 300:600 T1	Provide Y16 @ 450 B1	Provide Y20 @ 250:500 T1
	As prov	mm²/m	785	447	942
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.250 x 1.048 x 0.9 = 30.637		OK
			(As increased by 2.0 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 5			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	1.026	1.000
	Be		3750		900
	Total M	kNm	500.5	422.8	216.6
	Mt max	kNm	1185.0		288.7
MIDDLE STRIP	Width	mm	3750	3750	6600
	M	kNm	125.1	190.2	4.8
	d	mm	267.0	267.0	267.0
	As	mm²/m	303	460	7
	As deflection	mm²/m	303	460	
			Provide Y16 @ 500 T1	Provide Y16 @ 400 B1	Provide Y16 @ 500 T1
	As prov	mm²/m	402	503	402
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.478 x 1.048 x 0.9 = 36.247		OK
COLUMN STRIP	Width	mm	3750	3750	900
	M	kNm	375.4	232.5	216.6
	d	mm	265.0	265.0	267.0
	As	mm²/m	920	566	2429
	As deflection	mm²/m	920	566	
			Provide Y20 @ 250:500 T1	Provide Y20 @ 550 B1	Provide Y16 @ 75 T1
	As prov	mm²/m	942	571	2681
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /265.0 = 28.302 < 26.0 x 1.273 x 1.048 x 0.9 = 31.208		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure		REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	Internal Floor (xx), from grids 1 to 6		GS	26-Jan-09	6
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		Checked	Revision	Job No
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WEIGHT of REINFORCEME		No Mid Strip	Col Strip	Type	Dia	Length	Unit wt	Weight
TOP STEEL	Support 1	14		T	16	2600	1.578	57.5
			13	T	16	2600	1.578	53.3
	Span 1	8		T	16	5550	1.578	70.1
			8	T	16	5550	1.578	70.1
	Support 2	8		T	16	3750	1.578	47.4
			12	T	20	3750	2.466	111.0
	Span 2	8		T	16	5550	1.578	70.1
			8	T	16	5550	1.578	70.1
	Support 3	8		T	16	3750	1.578	47.4
			10	T	20	3750	2.466	92.5
	Span 3	8		T	16	5550	1.578	70.1
			8	T	16	5550	1.578	70.1
	Support 4	8		T	16	3750	1.578	47.4
			10	T	20	3750	2.466	92.5
	Span 4	8		T	16	5550	1.578	70.1
			8	T	16	5550	1.578	70.1
	Support 5	8		T	16	3750	1.578	47.4
			12	T	20	3750	2.466	111.0
	Span 5	8		T	16	5550	1.578	70.1
			8	T	16	5550	1.578	70.1
	Support 6	15		T	16	2600	1.578	61.6
			13	T	16	2600	1.578	53.3
BTM STEEL	Span 1	10		T	16	6925	1.578	109.3
			7	T	20	7850	2.466	135.5
	Span 2	8		T	16	6825	1.578	86.2
			9	T	16	7650	1.578	108.7
	Span 3	8		T	16	6825	1.578	86.2
			10	T	16	7650	1.578	120.7
	Span 4	8		T	16	6825	1.578	86.2
			9	T	16	7650	1.578	108.7
	Span 5	10		T	16	6925	1.578	109.3
			7	T	20	7850	2.466	135.5

SUMMARY *Rebar for single direction only. All figures approximate - see User Guide.*

TOTAL REINFORCEMENT IN BAY (kg) **2609** REINFORCEMENT DENSITY (kg/m^3) **30.6**

Project	RC Structure			REINFORCED CONCRETE COUNCIL		
Client				Made by	Date	Page
Location	Internal Floor (yy)	from grids 1	to 6	GS	26-Jan-09	1
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997			Checked	Revision	Job No
	Originated from RCC33.xls on CD	© 1999 BCA for RCC		GS	-	

MATERIALS							COVERS	mm	TO LAYER
	fcu	30	N/mm²	h agg	20	mm	Top cover	45	2
	fyl	500	N/mm²	γs	1.15	steel	Btm cover	45	2
	fyv	500	N/mm²	γc	1.50	concrete			

SPANS	L (m)
SPAN 1	7.500
SPAN 2	7.500
SPAN 3	7.500
SPAN 4	7.500
SPAN 5	7.500
SPAN 6	

GEOMETRY

Bay type	INTERNAL		
Slab depth, h	300	mm	
Panel width, b	7500	mm	
End distance	225	from supt 1	
End distance	225	from supt 6	

PERIMETER LOADS *characteristic*

7.24 *kN/m outside supports 1 & 6*

LOADING PATTERN

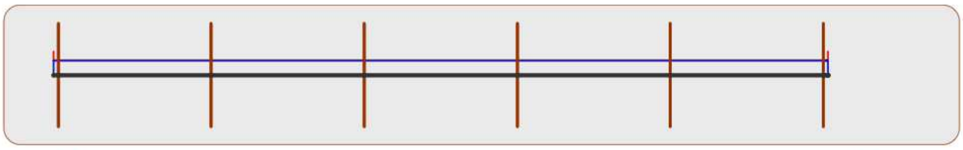
	min	max
DEAD	1.0	1.4
IMPOSED		1.6

SUPPORTS	ABOVE (m)	H (mm)	B (mm)	End Cond	BELOW (m)	H (mm)	B (mm)	End Cond
Support 1	2.700	450	450	F	2.700	450	450	F
Support 2	2.700	450	450	F	2.700	450	450	F
Support 3	2.700	450	450	F	2.700	450	450	F
Support 4	2.700	450	450	F	2.700	450	450	F
Support 5	2.700	450	450	F	2.700	450	450	F
Support 6	2.700	450	450	F	2.700	450	450	F
Support 7								

LOADING *UDLs (kN/m²) PLs (kN/m) Position (m)*

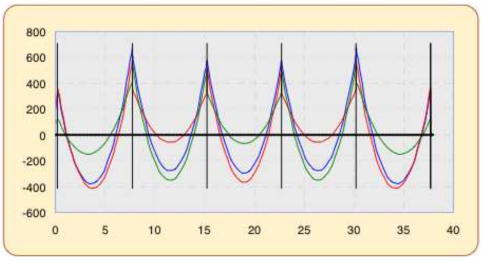
	Dead Load	Imposed Load	Position from left	Loaded Length		Dead Load	Imposed Load	Position from left	Loaded Length
Span 1					Span 4				
UDL	7.78	3.50	~~~~~	~~~~~	UDL	7.78	3.50	~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 2					Span 5				
UDL	7.78	3.50	~~~~~	~~~~~	UDL	7.78	3.50	~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 3					Span 6				
UDL	7.78	3.50	~~~~~	~~~~~	UDL			~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				

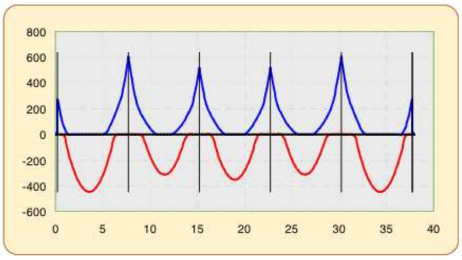
LOADING DIAGRAM



Project	RC Structure		REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	Internal Floor (yy), from grids 1 to 6		GS	26-Jan-09	2
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		Checked	Revision	Job No
	Originated from RCC33.xls on CD	© 1999 BCA for RCC	GS	-	

BENDING MOMENT DIAGRAMS (kNm)





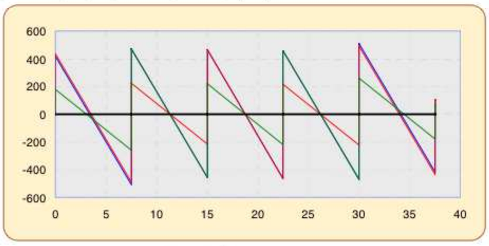
Elastic Moments

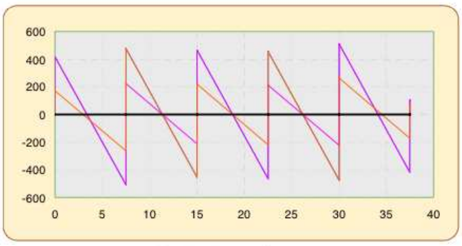
Redistributed Envelope

SUPPORT No	1	2	3	4	5	6	
Elastic M	358.5	671.3	575.2	575.2	671.4	358.6	~
Redistributed M	267.3	604.1	517.7	517.7	604.3	267.3	~
ßb	0.746	0.900	0.900	0.900	0.900	0.746	~
Redistribution		10.0%	10.0%	10.0%	10.0%		
End support reinf. Ø mm	16 *					16 *	

SPAN No	1	2	3	4	5	
Elastic M	412.12	350.29	364.41	350.27	412.24	~
Redistributed M	445.34	312.22	351.62	312.20	445.51	~
ßb	1.081	0.891	0.965	0.891	1.081	~

SHEARS FORCE DIAGRAMS (kN)





Elastic Shears

Redistributed Shears

SPAN No	1		2		3	
Elastic V	436.5	509.4	471.0	457.3	463.6	463.6
Redistributed V	418.7	509.4	475.2	453.0	463.6	463.6

SPAN No	4		5			
Elastic V	457.3	471.0	509.5	436.6	~	~
Redistributed V	453.0	475.2	509.6	418.8	~	~

REACTIONS (kN)

SUPPORT	1	2	3	4	5	6
ALL SPANS LOADED	522.6	983.7	915.8	915.7	983.9	522.7
ODD SPANS LOADED	521.7	732.9	677.5	677.5	733.0	521.8
EVEN SPANS LOADED	240.6	738.4	671.7	671.7	738.5	240.7
Veff for punching	653.2	1027.6	924.1	924.1	1027.8	653.4
Characteristic Dead	264.9	462.0	433.9	433.9	462.1	265.0
Characteristic Imposed	94.9	210.6	192.6	192.6	210.6	94.8

COLUMN MOMENTS (kNm)		1	2	3	4	5	6
ALL SPANS	Above	154.0	-25.4	4.8	-4.8	25.4	-154.1
LOADED	Below	154.0	-25.4	4.8	-4.8	25.4	-154.1
ODD SPANS	Above	172.4	-96.1	83.9	-83.9	96.1	-172.4
LOADED	Below	172.4	-96.1	83.9	-83.9	96.1	-172.4
EVEN SPANS	Above	53.1	58.9	-76.8	76.8	-58.9	-53.2
LOADED	Below	53.1	58.9	-76.8	76.8	-58.9	-53.2

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	Internal Floor (yy), from grids 1 to 6	GS	Jan-2009	3
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked	Revision	Job No
	Originated from RCC33.xls on CD © 1999 BCA for RCC	GS	-	

SPAN 1			LEFT	CENTRE	RIGHT
ACTIONS	ßb		1.000	1.081	0.900
	Be		900		3750
	Total M	kNm	176.2	445.3	500.4
	Mt max	kNm	247.1		1029.5
MIDDLE STRIP	Width	mm	6600	3750	3750
	M	kNm	4.8	200.4	125.1
	d	mm	247.0	245.0	247.0
	As	mm²/m	7	528	327
	As deflection	mm²/m		528	327
			Provide Y16 @ 500 T2	Provide Y20 @ 550 B2	Provide Y16 @ 500 T2
	As prov	mm²/m	402	571	402
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /245.0 = 30.612 < 26.0 x 1.443 x 1.051 x 0.9 = 35.499		OK
COLUMN STRIP	Width	mm	900	3750	3750
	M	kNm	176.2	244.9	375.3
	d	mm	247.0	245.0	247.0
	As	mm²/m	2115	645	997
	As deflection	mm²/m		645	997
			Provide Y16 @ 75 T2	Provide Y20 @ 450 B2	Provide Y16 @ 150:300 T2
	As prov	mm²/m	2681	698	1005
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /245.0 = 30.612 < 26.0 x 1.354 x 1.051 x 0.9 = 33.313		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 2			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.891	0.900
	Be		3750		3750
	Total M	kNm	500.4	312.2	419.1
	Mt max	kNm	1029.5		1029.5
MIDDLE STRIP	Width	mm	3750	3750	3750
	M	kNm	125.1	140.5	104.8
	d	mm	247.0	247.0	247.0
	As	mm²/m	327	367	274
	As deflection	mm²/m	327	389	
			Provide Y16 @ 500 T2	Provide Y16 @ 500 B2	Provide Y16 @ 500 T2
	As prov	mm²/m	402	402	402
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.296 x 1.051 x 0.9 = 31.877		OK
			(As increased by 6.0 % for deflection)		
COLUMN STRIP	Width	mm	3750	3750	3750
	M	kNm	375.3	171.7	314.3
	d	mm	247.0	247.0	247.0
	As	mm²/m	997	449	825
	As deflection	mm²/m	997	492	
			Provide Y16 @ 150:300 T2	Provide Y16 @ 400 B2	Provide Y16 @ 175:350 T2
	As prov	mm²/m	1005	503	862
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.272 x 1.051 x 0.9 = 31.303		OK
			(As increased by 9.5 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
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SPAN 3			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.965	0.900
	Be		3750		3750
	Total M	kNm	419.1	351.6	419.1
	Mt max	kNm	1029.5		1029.5
MIDDLE STRIP	Width	mm	3750	3750	3750
	M	kNm	104.8	158.2	104.8
	d	mm	247.0	247.0	247.0
	As	mm²/m	274	414	274
	As deflection	mm²/m	274	414	
			Provide Y16 @ 500 T2	Provide Y16 @ 450 B2	Provide Y16 @ 500 T2
	As prov	mm²/m	402	447	402
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.373 x 1.051 x 0.9 = 33.787		OK
			(As increased by 0.0 % for deflection)		
COLUMN STRIP	Width	mm	3750	3750	3750
	M	kNm	314.3	193.4	314.3
	d	mm	247.0	245.0	247.0
	As	mm²/m	825	510	825
	As deflection	mm²/m	825	533	
			Provide Y16 @ 175:350 T2	Provide Y20 @ 550 B2	Provide Y16 @ 175:350 T2
	As prov	mm²/m	862	571	862
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /245.0 = 30.612 < 26.0 x 1.350 x 1.051 x 0.9 = 33.205		OK
			(As increased by 4.5 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 4			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.891	0.900
	Be		3750		3750
	Total M	kNm	419.1	312.2	500.5
	Mt max	kNm	1029.5		1029.5
MIDDLE STRIP	Width	mm	3750	3750	3750
	M	kNm	104.8	140.5	125.1
	d	mm	247.0	247.0	247.0
	As	mm²/m	274	367	327
	As deflection	mm²/m	274	389	
			Provide Y16 @ 500 T2	Provide Y16 @ 500 B2	Provide Y16 @ 500 T2
	As prov	mm²/m	402	402	402
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.296 x 1.051 x 0.9 = 31.879		OK
			(As increased by 6.0 % for deflection)		
COLUMN STRIP	Width	mm	3750	3750	3750
	M	kNm	314.3	171.7	375.4
	d	mm	247.0	247.0	247.0
	As	mm²/m	825	449	997
	As deflection	mm²/m	825	492	
			Provide Y16 @ 175:350 T2	Provide Y16 @ 400 B2	Provide Y16 @ 150:300 T2
	As prov	mm²/m	862	503	1005
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.272 x 1.051 x 0.9 = 31.305		OK
			(As increased by 9.5 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	Internal Floor (yy), from grids 1 to 6	GS	Jan-2009	5
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SPAN 5			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	1.081	1.000
	Be		3750		900
	Total M	kNm	500.5	445.5	176.2
	Mt max	kNm	1029.5		247.1
MIDDLE STRIP	Width	mm	3750	3750	6600
	M	kNm	125.1	200.5	4.8
	d	mm	247.0	245.0	247.0
	As	mm²/m	327	528	7
	As deflection	mm²/m	327	528	
			Provide Y16 @ 500 T2	Provide Y20 @ 550 B2	Provide Y16 @ 500 T2
	As prov	mm²/m	402	571	402
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /245.0 = 30.612 < 26.0 x 1.442 x 1.051 x 0.9 = 35.486		OK
COLUMN STRIP	Width	mm	3750	3750	900
	M	kNm	375.4	245.0	176.2
	d	mm	247.0	245.0	247.0
	As	mm²/m	997	646	2115
	As deflection	mm²/m	997	646	
			Provide Y16 @ 150:300 T2	Provide Y20 @ 450 B2	Provide Y16 @ 75 T2
	As prov	mm²/m	1005	698	2681
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /245.0 = 30.612 < 26.0 x 1.353 x 1.051 x 0.9 = 33.301		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure		REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	Internal Floor (yy), from grids 1 to 6		GS	26-Jan-09	6
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WEIGHT of REINFORCEME		No Mid Strip	Col Strip	Type	Dia	Length	Unit wt	Weight
TOP STEEL	Support 1	14		T	16	2575	1.578	56.9
			13	T	16	2575	1.578	52.8
	Span 1	8		T	16	5550	1.578	70.1
			8	T	16	5550	1.578	70.1
	Support 2	8		T	16	3750	1.578	47.4
			19	T	16	3750	1.578	112.5
	Span 2	8		T	16	5550	1.578	70.1
			8	T	16	5550	1.578	70.1
	Support 3	8		T	16	3750	1.578	47.4
			17	T	16	3750	1.578	100.6
	Span 3	8		T	16	5550	1.578	70.1
			8	T	16	5550	1.578	70.1
	Support 4	8		T	16	3750	1.578	47.4
			17	T	16	3750	1.578	100.6
	Span 4	8		T	16	5550	1.578	70.1
			8	T	16	5550	1.578	70.1
	Support 5	8		T	16	3750	1.578	47.4
			19	T	16	3750	1.578	112.5
	Span 5	8		T	16	5550	1.578	70.1
			8	T	16	5550	1.578	70.1
	Support 6	15		T	16	2575	1.578	61.0
			13	T	16	2575	1.578	52.8
BTM STEEL	Span 1	7		T	20	6925	2.466	119.5
			9	T	20	7850	2.466	174.2
	Span 2	8		T	16	6825	1.578	86.2
			10	T	16	7650	1.578	120.7
	Span 3	9		T	16	6825	1.578	96.9
			7	T	20	7650	2.466	132.1
	Span 4	8		T	16	6825	1.578	86.2
			10	T	16	7650	1.578	120.7
	Span 5	7		T	20	6925	2.466	119.5
			9	T	20	7850	2.466	174.2

SUMMARY *Rebar for single direction only. All figures approximate - see User Guide.*

TOTAL REINFORCEMENT IN BAY (kg) **2770** REINFORCEMENT DENSITY (kg/m^3) **32.4**

Project	RC Structure		REINFORCED CONCRETE COUNCIL
Client			
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MATERIALS							COVERS	mm	TO LAYER
fcu	30	N/mm²	h agg	20	mm				
fyl	500	N/mm²	γs	1.15	steel		Top cover	25	1
fyv	500	N/mm²	γc	1.50	concrete		Btm cover	25	1

SPANS	L (m)
SPAN 1	7.500
SPAN 2	7.500
SPAN 3	7.500
SPAN 4	7.500
SPAN 5	7.500
SPAN 6	

GEOMETRY		
Bay type	EDGE	
Slab depth, h	300	mm
Panel width, b	3750	mm
Edge distance	225	mm to C/L
End distance	225	from supt 1
End distance	225	from supt 6

PERIMETER LOADS *characteristic*

7.24	*kN/m outside supports 1 & 6*
7.24	kN/m along bay edge

LOADING PATTERN

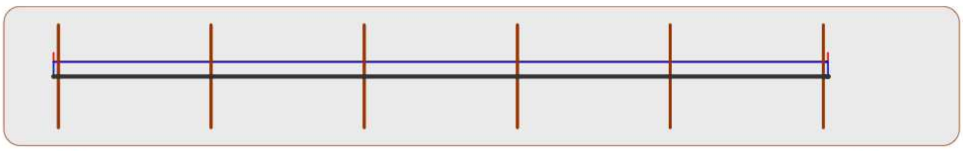
	min	max
DEAD	1.0	1.4
IMPOSED		1.6

SUPPORTS	ABOVE (m)	H (mm)	B (mm)	End Cond	BELOW (m)	H (mm)	B (mm)	End Cond
Support 1	2.700	450	450	F	2.700	450	450	F
Support 2	2.700	450	450	F	2.700	450	450	F
Support 3	2.700	450	450	F	2.700	450	450	F
Support 4	2.700	450	450	F	2.700	450	450	F
Support 5	2.700	450	450	F	2.700	450	450	F
Support 6	2.700	450	450	F	2.700	450	450	F
Support 7								

LOADING *UDLs (kN/m²) PLs (kN/m) Position (m)*

	Dead Load	Imposed Load	Position from left	Loaded Length		Dead Load	Imposed Load	Position from left	Loaded Length
Span 1					Span 4				
UDL	7.78	3.50	~~~~~	~~~~~	UDL	7.78	3.50	~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 2					Span 5				
UDL	7.78	3.50	~~~~~	~~~~~	UDL	7.78	3.50	~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 3					Span 6				
UDL	7.78	3.50	~~~~~	~~~~~	UDL			~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				

LOADING DIAGRAM



1 6

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
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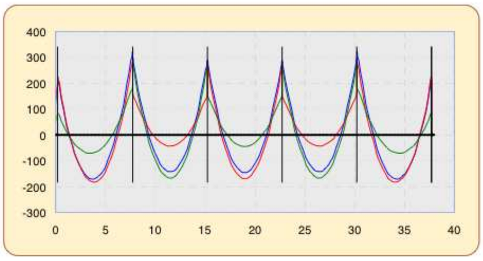
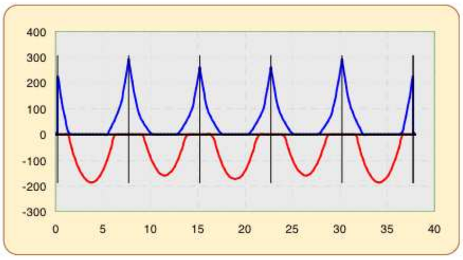
BENDING MOMENT DIAGRAMS (kNm)



Elastic Moments

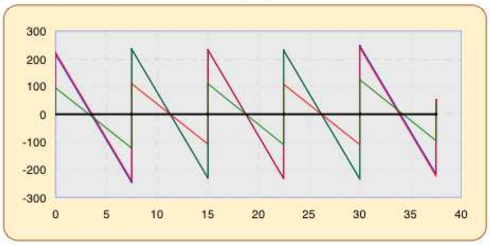


Redistributed Envelope

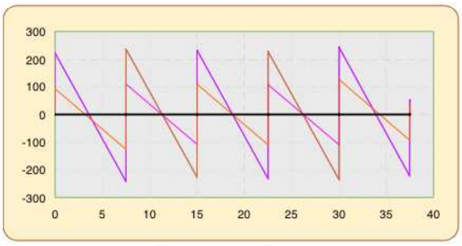
SUPPORT No	1	2	3	4	5	6	
Elastic M	221.0	323.1	289.1	289.1	323.2	221.1	~
Redistributed M	221.0	290.8	260.2	260.2	290.9	221.1	~
ßb	1.000	0.900	0.900	0.900	0.900	1.000	~
Redistribution		10.0%	10.0%	10.0%	10.0%		
End support reinf. Ø mm	16 *					16 *	

SPAN No	1	2	3	4	5	
Elastic M	182.48	165.75	168.83	165.74	182.53	~
Redistributed M	186.58	159.31	174.50	159.28	186.63	~
ßb	1.022	0.961	1.034	0.961	1.022	~

SHEARS FORCE DIAGRAMS (kN)



Elastic Shears



Redistributed Shears

SPAN No	1		2		3	
Elastic V	223.4	247.3	233.6	230.2	231.8	231.8
Redistributed V	222.5	242.9	235.9	227.7	231.8	231.8

SPAN No	4		5			
Elastic V	230.2	233.6	247.3	223.4	~	~
Redistributed V	227.7	235.9	243.0	222.6	~	~

REACTIONS (kN)

SUPPORT	1	2	3	4	5	6
ALL SPANS LOADED	272.6	478.9	459.6	459.5	478.9	272.7
ODD SPANS LOADED	274.4	351.6	340.1	340.1	351.6	274.5
EVEN SPANS LOADED	126.4	361.9	337.1	337.1	362.0	126.4
Veff for punching	343.0	628.3	578.4	578.4	628.4	343.1
Characteristic Dead	135.9	226.7	217.8	217.8	226.7	135.9
Characteristic Imposed	52.6	100.9	96.6	96.6	100.9	52.6

COLUMN MOMENTS (kNm)		1	2	3	4	5	6
ALL SPANS	Above	98.6	-11.8	1.6	-1.6	11.8	-98.6
LOADED	Below	98.6	-11.8	1.6	-1.6	11.8	-98.6
ODD SPANS	Above	107.1	-59.5	53.9	-53.9	59.5	-107.1
LOADED	Below	107.1	-59.5	53.9	-53.9	59.5	-107.1
EVEN SPANS	Above	37.3	42.2	-51.6	51.6	-42.2	-37.3
LOADED	Below	37.3	42.2	-51.6	51.6	-42.2	-37.3

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SPAN 1			LEFT	CENTRE	RIGHT
ACTIONS	ßb		1.000	1.022	0.900
	Be		675		2100
	Total M	kNm	172.5	186.6	239.3
	Mt max	kNm	216.5		663.6
MIDDLE STRIP	Width	mm	3300	1875	1875
	M	kNm	3.4	84.0	59.8
	d	mm	267.0	267.0	267.0
	As	mm²/m	9	406	289
	As deflection	mm²/m		406	289
			Provide Y16 @ 500 T1	Provide Y16 @ 450 B1	Provide Y16 @ 500 T1
	As prov	mm²/m	402	447	402
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.536 x 1.048 x 0.9 = 37.651		OK
COLUMN STRIP	Width	mm	675	2100	2100
	M	kNm	172.5	102.6	179.5
	d	mm	267.0	267.0	265.0
	As	mm²/m	2614	443	781
	As deflection	mm²/m		443	781
			Provide Y16 @ 75 T1	Provide Y16 @ 450 B1	Provide Y20 @ 300:600 T1
	As prov	mm²/m	2681	447	785
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.358 x 1.048 x 0.9 = 33.292		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 2			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.961	0.900
	Be		2100		2100
	Total M	kNm	239.3	159.3	210.5
	Mt max	kNm	663.6		663.6
MIDDLE STRIP	Width	mm	1875	1875	1875
	M	kNm	59.8	71.7	52.6
	d	mm	267.0	267.0	267.0
	As	mm²/m	289	347	254
	As deflection	mm²/m	289	347	
			Provide Y16 @ 500 T1	Provide Y16 @ 500 B1	Provide Y16 @ 500 T1
	As prov	mm²/m	402	402	402
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.583 x 1.048 x 0.9 = 38.807		OK
COLUMN STRIP	Width	mm	2100	2100	2100
	M	kNm	179.5	87.6	157.9
	d	mm	265.0	267.0	265.0
	As	mm²/m	781	378	687
	As deflection	mm²/m	781	378	
			Provide Y20 @ 300:600 T1	Provide Y16 @ 500 B1	Provide Y20 @ 325:650 T1
	As prov	mm²/m	785	402	725
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.396 x 1.048 x 0.9 = 34.218		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 3			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	1.034	0.900
	Be		2100		2100
	Total M	kNm	210.5	174.5	210.5
	Mt max	kNm	663.6		663.6
MIDDLE STRIP	Width	mm	1875	1875	1875
	M	kNm	52.6	78.5	52.6
	d	mm	267.0	267.0	267.0
	As	mm²/m	254	380	254
	As deflection	mm²/m	254	380	
			Provide Y16 @ 500 T1	Provide Y16 @ 500 B1	Provide Y16 @ 500 T1
	As prov	mm²/m	402	402	402
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.516 x 1.048 x 0.9 = 37.173		OK
COLUMN STRIP	Width	mm	2100	2100	2100
	M	kNm	157.9	96.0	157.9
	d	mm	265.0	267.0	265.0
	As	mm²/m	687	414	687
	As deflection	mm²/m	687	414	
			Provide Y20 @ 325:650 T1	Provide Y16 @ 450 B1	Provide Y20 @ 325:650 T1
	As prov	mm²/m	725	447	725
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.512 x 1.048 x 0.9 = 37.070		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 4			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.961	0.900
	Be		2100		2100
	Total M	kNm	210.5	159.3	239.4
	Mt max	kNm	663.6		663.6
MIDDLE STRIP	Width	mm	1875	1875	1875
	M	kNm	52.6	71.7	59.8
	d	mm	267.0	267.0	267.0
	As	mm²/m	254	347	289
	As deflection	mm²/m	254	347	
			Provide Y16 @ 500 T1	Provide Y16 @ 500 B1	Provide Y16 @ 500 T1
	As prov	mm²/m	402	402	402
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.583 x 1.048 x 0.9 = 38.809		OK
COLUMN STRIP	Width	mm	2100	2100	2100
	M	kNm	157.9	87.6	179.5
	d	mm	265.0	267.0	265.0
	As	mm²/m	687	378	781
	As deflection	mm²/m	687	378	
			Provide Y20 @ 325:650 T1	Provide Y16 @ 500 B1	Provide Y20 @ 300:600 T1
	As prov	mm²/m	725	402	785
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.396 x 1.048 x 0.9 = 34.221		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

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	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked	Revision	Job No
	Originated from RCC33.xls on CD © 1999 BCA for RCC	GS	-	

SPAN 5			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	1.022	1.000
	Be		2100		675
	Total M	kNm	239.4	186.6	172.6
	Mt max	kNm	663.6		216.5
MIDDLE STRIP	Width	mm	1875	1875	3300
	M	kNm	59.8	84.0	3.4
	d	mm	267.0	267.0	267.0
	As	mm²/m	289	406	9
	As deflection	mm²/m	289	406	
			Provide Y16 @ 500 T1	Provide Y16 @ 450 B1	Provide Y16 @ 500 T1
	As prov	mm²/m	402	447	402
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.535 x 1.048 x 0.9 = 37.637		OK
COLUMN STRIP	Width	mm	2100	2100	675
	M	kNm	179.5	102.6	172.6
	d	mm	265.0	267.0	267.0
	As	mm²/m	781	443	2614
	As deflection	mm²/m	781	443	
			Provide Y20 @ 300:600 T1	Provide Y16 @ 450 B1	Provide Y16 @ 75 T1
	As prov	mm²/m	785	447	2681
	Top steel			Provide Y16 @ 500 T1	
	Deflection		L/d = 7,500 /267.0 = 28.090 < 26.0 x 1.357 x 1.048 x 0.9 = 33.279		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	External Floor (xx), from grids 1 to 6	GS	26-Jan-09	6
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked	Revision	Job No
	Originated from RCC33.xls on CD © 1999 BCA for RCC	GS	-	

WEIGHT of REINFORCEME

		No Mid Strip	Col Strip	Type	Dia	Length	Unit wt	Weight
TOP STEEL	Support 1	7		T	16	2600	1.578	28.7
			10	T	16	2600	1.578	41.0
	Span 1	4		T	16	5550	1.578	35.0
			5	T	16	5550	1.578	43.8
	Support 2	4		T	16	3750	1.578	23.7
			6	T	20	3750	2.466	55.5
	Span 2	4		T	16	5550	1.578	35.0
			5	T	16	5550	1.578	43.8
	Support 3	4		T	16	3750	1.578	23.7
			5	T	20	3750	2.466	46.2
	Span 3	4		T	16	5550	1.578	35.0
			5	T	16	5550	1.578	43.8
	Support 4	4		T	16	3750	1.578	23.7
			5	T	20	3750	2.466	46.2
	Span 4	4		T	16	5550	1.578	35.0
			5	T	16	5550	1.578	43.8
	Support 5	4		T	16	3750	1.578	23.7
			6	T	20	3750	2.466	55.5
	Span 5	4		T	16	5550	1.578	35.0
			5	T	16	5550	1.578	43.8
	Support 6	8		T	16	2600	1.578	32.8
			10	T	16	2600	1.578	41.0
BTM STEEL	Span 1	5		T	16	6925	1.578	54.6
			5	T	16	7850	1.578	61.9
	Span 2	4		T	16	6825	1.578	43.1
			5	T	16	7650	1.578	60.4
	Span 3	4		T	16	6825	1.578	43.1
			5	T	16	7650	1.578	60.4
	Span 4	4		T	16	6825	1.578	43.1
			5	T	16	7650	1.578	60.4
	Span 5	5		T	16	6925	1.578	54.6
			5	T	16	7850	1.578	61.9

SUMMARY *Rebar for single direction only. All figures approximate - see User Guide.*

TOTAL REINFORCEMENT IN BAY (kg) **1380** REINFORCEMENT DENSITY (kg/m^3) **32.3**

Project	RC Structure		REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	External Floor (yy) from grids 1 to 6		GS	26-Jan-09	1
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		Checked	Revision	Job No
	Originated from RCC33.xls on CD	© 1999 BCA for RCC	GS	-	

MATERIALS

						COVERS	mm	TO LAYER
fcu	30	N/mm²	h agg	20	mm	Top cover	45	2
fyl	500	N/mm²	γs	1.15	steel	Btm cover	45	2
fyv	500	N/mm²	γc	1.50	concrete			

SPANS	L (m)
SPAN 1	7.500
SPAN 2	7.500
SPAN 3	7.500
SPAN 4	7.500
SPAN 5	7.500
SPAN 6	

GEOMETRY

Bay type	EDGE	
Slab depth, h	300	mm
Panel width, b	3750	mm
Edge distance	225	mm to C/L
End distance	225	from supt 1
End distance	225	from supt 6

PERIMETER LOADS *characteristic*

7.24 *kN/m outside supports 1 & 6*

7.24 kN/m along bay edge

LOADING PATTERN

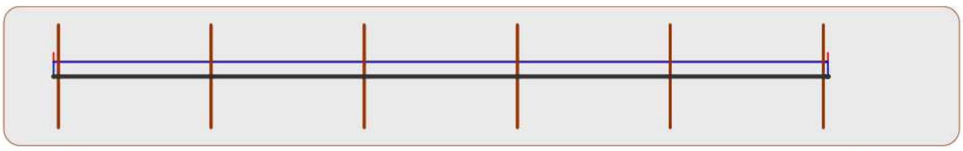
	min	max
DEAD	1.0	1.4
IMPOSED		1.6

SUPPORTS	ABOVE (m)	H (mm)	B (mm)	End Cond	BELOW (m)	H (mm)	B (mm)	End Cond
Support 1	2.700	450	450	F	2.700	450	450	F
Support 2	2.700	450	450	F	2.700	450	450	F
Support 3	2.700	450	450	F	2.700	450	450	F
Support 4	2.700	450	450	F	2.700	450	450	F
Support 5	2.700	450	450	F	2.700	450	450	F
Support 6	2.700	450	450	F	2.700	450	450	F
Support 7								

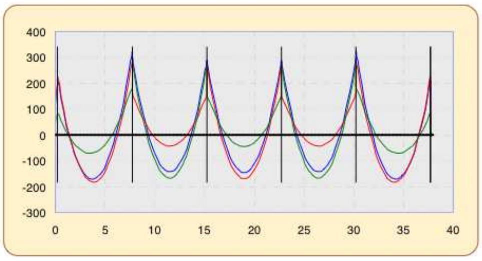
LOADING *UDLs (kN/m²) PLs (kN/m) Position (m)*

	Dead Load	Imposed Load	Position from left	Loaded Length		Dead Load	Imposed Load	Position from left	Loaded Length
Span 1					Span 4				
UDL	7.78	3.50	~~~~~	~~~~~	UDL	7.78	3.50	~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 2					Span 5				
UDL	7.78	3.50	~~~~~	~~~~~	UDL	7.78	3.50	~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 3					Span 6				
UDL	7.78	3.50	~~~~~	~~~~~	UDL			~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				

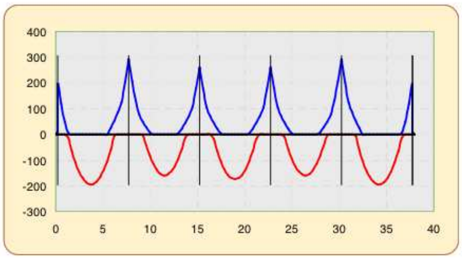
LOADING DIAGRAM



BENDING MOMENT DIAGRAMS (kNm)



Elastic Moments

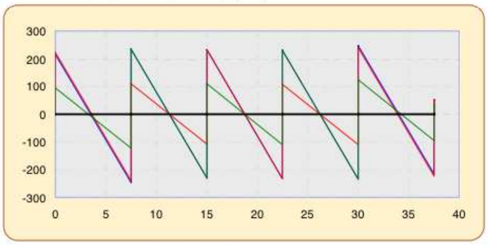


Redistributed Envelope

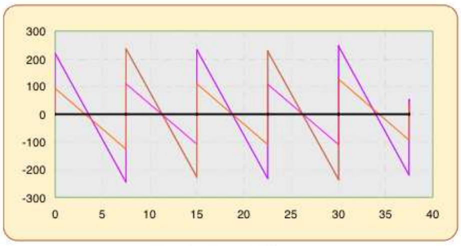
SUPPORT No	1	2	3	4	5	6	
Elastic M	221.0	323.1	289.1	289.1	323.2	221.1	~
Redistributed M	195.4	290.8	260.2	260.2	290.9	195.4	~
ßb	0.884	0.900	0.900	0.900	0.900	0.884	~
Redistribution		10.0%	10.0%	10.0%	10.0%		
End support reinf. Ø mm	16 *					16 *	

SPAN No	1	2	3	4	5	
Elastic M	182.48	165.75	168.83	165.74	182.53	~
Redistributed M	194.57	159.31	174.50	159.28	194.66	~
ßb	1.066	0.961	1.034	0.961	1.066	~

SHEARS FORCE DIAGRAMS (kN)



Elastic Shears



Redistributed Shears

SPAN No	1		2		3	
Elastic V	223.4	247.3	233.6	230.2	231.8	231.8
Redistributed V	219.1	245.0	235.9	227.7	231.8	231.8

SPAN No	4		5			
Elastic V	230.2	233.6	247.3	223.4	~	~
Redistributed V	227.7	235.9	245.0	219.2	~	~

REACTIONS (kN)

SUPPORT	1	2	3	4	5	6
ALL SPANS LOADED	271.0	480.4	459.6	459.5	480.5	271.1
ODD SPANS LOADED	270.6	355.4	340.1	340.1	355.5	270.6
EVEN SPANS LOADED	126.4	361.9	337.1	337.1	362.0	126.4
Veff for punching	338.8	630.3	578.4	578.4	630.4	338.9
Characteristic Dead	135.9	226.7	217.8	217.8	226.7	135.9
Characteristic Imposed	50.5	101.9	96.6	96.6	101.9	50.5

COLUMN MOMENTS (kNm)		1	2	3	4	5	6
ALL SPANS	Above	98.6	-11.8	1.6	-1.6	11.8	-98.6
LOADED	Below	98.6	-11.8	1.6	-1.6	11.8	-98.6
ODD SPANS	Above	107.1	-59.5	53.9	-53.9	59.5	-107.1
LOADED	Below	107.1	-59.5	53.9	-53.9	59.5	-107.1
EVEN SPANS	Above	37.3	42.2	-51.6	51.6	-42.2	-37.3
LOADED	Below	37.3	42.2	-51.6	51.6	-42.2	-37.3

Project	RC Structure	REINFORCED CONCRETE COUNCIL			
			Made by	Date	Page
Client			GS	Jan-2009	3
Location	External Floor (yy), from grids 1 to 6		Checked	Revision	Job No
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997		GS	-	
	Originated from RCC33.xls on CD © 1999 BCA for RCC				

SPAN 1			LEFT	CENTRE	RIGHT
ACTIONS	ßb		1.000	1.066	0.900
	Be		675		2100
	Total M	kNm	147.7	194.6	239.3
	Mt max	kNm	185.3		576.5
MIDDLE STRIP	Width	mm	3300	1875	1875
	M	kNm	3.4	87.6	59.8
	d	mm	247.0	247.0	247.0
	As	mm²/m	10	458	313
	As deflection	mm²/m	10	458	313
			Provide Y16 @ 500 T2	Provide Y16 @ 400 B2	Provide Y16 @ 500 T2
	As prov	mm²/m	402	503	402
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.512 x 1.051 x 0.9 = 37.211		OK
COLUMN STRIP	Width	mm	675	2100	2100
	M	kNm	147.7	107.0	179.5
	d	mm	247.0	247.0	247.0
	As	mm²/m	2419	499	842
	As deflection	mm²/m		499	842
			Provide Y16 @ 75 T2	Provide Y16 @ 400 B2	Provide Y16 @ 175:350 T2
	As prov	mm²/m	2681	503	862
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.349 x 1.051 x 0.9 = 33.188		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 2			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.961	0.900
	Be		2100		2100
	Total M	kNm	239.3	159.3	210.5
	Mt max	kNm	576.5		576.5
MIDDLE STRIP	Width	mm	1875	1875	1875
	M	kNm	59.8	71.7	52.6
	d	mm	247.0	247.0	247.0
	As	mm²/m	313	375	275
	As deflection	mm²/m	313	375	
			Provide Y16 @ 500 T2	Provide Y16 @ 500 B2	Provide Y16 @ 500 T2
	As prov	mm²/m	402	402	402
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.389 x 1.051 x 0.9 = 34.187		OK
COLUMN STRIP	Width	mm	2100	2100	2100
	M	kNm	179.5	87.6	157.9
	d	mm	247.0	247.0	247.0
	As	mm²/m	842	409	737
	As deflection	mm²/m	842	409	
			Provide Y16 @ 175:350 T2	Provide Y16 @ 450 B2	Provide Y16 @ 200:400 T2
	As prov	mm²/m	862	447	754
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.390 x 1.051 x 0.9 = 34.188		OK
			(As increased by 0.0 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 3			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	1.034	0.900
	Be		2100		2100
	Total M	kNm	210.5	174.5	210.5
	Mt max	kNm	576.5		576.5
MIDDLE STRIP	Width	mm	1875	1875	1875
	M	kNm	52.6	78.5	52.6
	d	mm	247.0	247.0	247.0
	As	mm²/m	275	410	275
	As deflection	mm²/m	275	410	
			Provide Y16 @ 500 T2	Provide Y16 @ 450 B2	Provide Y16 @ 500 T2
	As prov	mm²/m	402	447	402
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.499 x 1.051 x 0.9 = 36.887		OK
COLUMN STRIP	Width	mm	2100	2100	2100
	M	kNm	157.9	96.0	157.9
	d	mm	247.0	247.0	247.0
	As	mm²/m	737	448	737
	As deflection	mm²/m	737	448	
			Provide Y16 @ 200:400 T2	Provide Y16 @ 400 B2	Provide Y16 @ 200:400 T2
	As prov	mm²/m	754	503	754
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.508 x 1.051 x 0.9 = 37.105		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

SPAN 4			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	0.961	0.900
	Be		2100		2100
	Total M	kNm	210.5	159.3	239.4
	Mt max	kNm	576.5		576.5
MIDDLE STRIP	Width	mm	1875	1875	1875
	M	kNm	52.6	71.7	59.8
	d	mm	247.0	247.0	247.0
	As	mm²/m	275	375	313
	As deflection	mm²/m	275	375	
			Provide Y16 @ 500 T2	Provide Y16 @ 500 B2	Provide Y16 @ 500 T2
	As prov	mm²/m	402	402	402
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.390 x 1.051 x 0.9 = 34.189		OK
COLUMN STRIP	Width	mm	2100	2100	2100
	M	kNm	157.9	87.6	179.5
	d	mm	247.0	247.0	247.0
	As	mm²/m	737	409	842
	As deflection	mm²/m	737	409	
			Provide Y16 @ 200:400 T2	Provide Y16 @ 450 B2	Provide Y16 @ 175:350 T2
	As prov	mm²/m	754	447	862
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.390 x 1.051 x 0.9 = 34.191		OK
			(As increased by 0.0 % for deflection)		
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	External Floor (yy), from grids 1 to 6	GS	Jan-2009	5
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked	Revision	Job No
	Originated from RCC33.xls on CD © 1999 BCA for RCC	GS	-	

SPAN 5			LEFT	CENTRE	RIGHT
ACTIONS	ßb		0.900	1.066	1.000
	Be		2100		675
	Total M	kNm	239.4	194.7	147.7
	Mt max	kNm	576.5		185.3
MIDDLE STRIP	Width	mm	1875	1875	3300
	M	kNm	59.8	87.6	3.4
	d	mm	247.0	247.0	247.0
	As	mm²/m	313	458	10
	As deflection	mm²/m	313	458	
			Provide Y16 @ 500 T2	Provide Y16 @ 400 B2	Provide Y16 @ 500 T2
	As prov	mm²/m	402	503	402
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.512 x 1.051 x 0.9 = 37.197		OK
COLUMN STRIP	Width	mm	2100	2100	675
	M	kNm	179.5	107.1	147.7
	d	mm	247.0	247.0	247.0
	As	mm²/m	842	500	2419
	As deflection	mm²/m	842	500	
			Provide Y16 @ 175:350 T2	Provide Y16 @ 400 B2	Provide Y16 @ 75 T2
	As prov	mm²/m	862	503	2681
	Top steel			Provide Y16 @ 500 T2	
	Deflection		L/d = 7,500 /247.0 = 30.364 < 26.0 x 1.348 x 1.051 x 0.9 = 33.174		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	max S		ok	ok	ok

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	External Floor (yy), from grids 1 to 6	GS	26-Jan-09	6
	FLAT SLAB ANALYSIS & DESIGN to BS 8110:1997	Checked	Revision	Job No
	Originated from RCC33.xls on CD © 1999 BCA for RCC	GS	-	

WEIGHT of REINFORCEME		No Mid Strip	No Col Strip	Type	Dia	Length	Unit wt	Weight
TOP STEEL	Support 1	7		T	16	2575	1.578	28.4
			10	T	16	2575	1.578	40.6
	Span 1	4		T	16	5550	1.578	35.0
			5	T	16	5550	1.578	43.8
	Support 2	4		T	16	3750	1.578	23.7
			9	T	16	3750	1.578	53.3
	Span 2	4		T	16	5550	1.578	35.0
			5	T	16	5550	1.578	43.8
	Support 3	4		T	16	3750	1.578	23.7
			8	T	16	3750	1.578	47.4
	Span 3	4		T	16	5550	1.578	35.0
			5	T	16	5550	1.578	43.8
	Support 4	4		T	16	3750	1.578	23.7
			8	T	16	3750	1.578	47.4
	Span 4	4		T	16	5550	1.578	35.0
			5	T	16	5550	1.578	43.8
	Support 5	4		T	16	3750	1.578	23.7
			9	T	16	3750	1.578	53.3
	Span 5	4		T	16	5550	1.578	35.0
			5	T	16	5550	1.578	43.8
	Support 6	8		T	16	2575	1.578	32.5
			10	T	16	2575	1.578	40.6
BTM STEEL	Span 1	5		T	16	6925	1.578	54.6
			6	T	16	7850	1.578	74.3
	Span 2	4		T	16	6825	1.578	43.1
			5	T	16	7650	1.578	60.4
	Span 3	5		T	16	6825	1.578	53.9
			6	T	16	7650	1.578	72.4
	Span 4	4		T	16	6825	1.578	43.1
			5	T	16	7650	1.578	60.4
	Span 5	5		T	16	6925	1.578	54.6
			6	T	16	7850	1.578	74.3

SUMMARY *Rebar for single direction only. All figures approximate - see User Guide.*

TOTAL REINFORCEMENT IN BAY (kg) **1424** REINFORCEMENT DENSITY (kg/m^3) **33.3**

APPENDIX C COLUMN DESIGN

Project	RC Structure	REINFORCED CONCRETE COUNCIL	
Client		Made by GS	Date 26-Jan-09	Page 1
Location	Column Corner	Checked GS	Revision -	Job No -

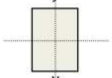
COLUMN LOAD TAKE DOWN & DESIGN FOR SYMMETRICALLY REINFORCED RECT. COLUMNS BENT ABOUT TWO AXES TO BS 8110:1997
Originated from RCC51.xls on CD © 1999 BCA for RCC

INPUT

Location Column Corner

1-2

y

Orientation n/a x  x B-C H

y

n/a

concrete density, kN/m^3 24.0
γ_{fgk} 1.40
γ_{fqk} 1.60

			Level	6	5	4	3	2	1
Dimensions									
Spans	Cl to Cl	n/a	m	0.00					
		B-C	m	7.50	7.50	7.50	7.50	7.50	7.50
		1-2	m	7.50	7.50	7.50	7.50	7.50	7.50
		n/a	m	0.00					
Slab	thickness (solid)		mm	250	300	300	300	300	300
	span direction,(ll to) x, y or b			b	b	b	b	b	b
Beams	width	n/a	mm	0					
	depth o/a	n/a	mm	0					
	width	B-C	mm	0					
	depth o/a	B-C	mm	0					
	width	1-2	mm	0					
	depth o/a	1-2	mm	0					
	width	n/a	mm	0					
Column below			(col above)						
	H (ll to yy)	mm	0	450	450	450	450	450	450
	B (ll to xx)	mm	0	450	450	450	450	450	450
Height (fl. to floor.)		m	0.00	3.00	3.00	3.00	3.00	3.00	3.00

			Level	6	5	4	3	2	1
Loads	(characteristic uno)								
Slab	(inc swt.)	gk	kN/m2	6.43	7.78	7.78	7.78	7.78	7.78
		qk	kN/m2	1.50	3.50	3.50	3.50	3.50	3.50
Beams	(swt.)	gk	kN/m	included	included	included	included	included	included
	line loads (-extra over slab loads and beam self weight)								
	n/a	gk	kN/m	0.0					
		qk	kN/m	0.0					
	B-C	gk	kN/m	7.2	7.2	7.2	7.2	7.2	7.2
		qk	kN/m	0.0					
	1-2	gk	kN/m	7.2	7.2	7.2	7.2	7.2	7.2
		qk	kN/m	0.0					
	n/a	gk	kN/m	0.0					
		qk	kN/m	0.0					
At column position, other applied loads (eg loads from cantilevers)									
		Gk	kN (char)	0.0					
		Qk	kN (char)	0.0					
		Mxx	kNm (ult)	0.0					
		Myy	kNm (ult)	0.0					

Loads per floor								
Floor	Gk	kN	144.7	163.6	163.6	163.6	163.6	163.6
Floor	Qk	kN	21.1	49.2	49.2	49.2	49.2	49.2
Column below	Gk	kN	14.6	14.6	14.6	14.6	14.6	14.6

OUTPUT

			Column level	6 to 5	5 to 4	4 to 3	3 to 2	2 to 1	Below 1
Cumulative loads in column.									
		Gk	kN	159.3	337.5	515.7	694.0	872.2	1050.4
		Qk	kN	21.1	70.3	119.5	168.8	218.0	267.2
		Qk redn	factor	1.0	0.9	0.8	0.7	0.6	0.6
		Qk red*	kN	21.1	63.3	95.6	118.1	130.8	160.3
		N	kN	**257**	**574**	**875**	**1161**	**1430**	**1727**
Moments in column									
about x-x	Mxx	top	kNm	172.3	115.7	115.7	115.7	115.7	115.7
about y-y	Myy	top	kNm	172.3	115.7	115.7	115.7	115.7	115.7
	Mxx	bottom	kNm	115.7	115.7	115.7	115.7	115.7	
	Myy	bottom	kNm	115.7	115.7	115.7	115.7	115.7	

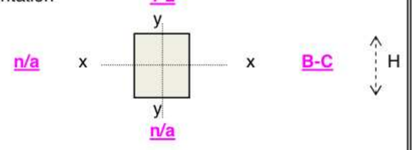
Project	RC Structure		REINFORCED CONCRETE COUNCIL		
Client	0		Made by	Date	Page
Location	Column Corner		GS	26-Jan-09	2
	COLUMN LOAD TAKE DOWN & DESIGN FOR SYMMETRICALLY REINFORCED RECT. COLUMNS BENT ABOUT TWO AXES		Checked	Revision	Job No
	Originated from RCC51.xls on CD	© 1999 BCA for RCC	GS	-	-

INPUT

Location **Column Corner**

Level considered: **Bottom (Max N)**

Orientation 1-2



Dimensions		about x-x	about y-y
h (ll to yy)	mm	450	-
b (ll to xx)	mm	-	450
lo, clear height	mm	2750	2750
ß	value	0.85	0.85
Braced or Unbraced?	B or U	B	B

Loads

Axial	N	kN	1727	-
Moments	top	kNm	115.7	115.7
	bottom	kNm	0.0	0.0

Column properties

fcu		30
fy		500
cover to link	mm	30
Max sized main bar	mm	40
Probable percentage As	%	2.50%
link diameter	mm	10

OUTPUT

Design criteria

N	kNm	**1727**
M	kN	**192.0**
about		**Y-Y**

PROOF

Slenderness

le	mm	2338	2337.5
Slenderness		5.19	5.19
Limit for short column		15.0	15.0
Design column as		Short	
Column is		Short	Short
		about x-x	about y-y

Design moments

Min eccentricity, 0.05 h	kNm	34.5	
Madd	n/a - short column		
d	mm	390	390
Nuz	kN	5138.4	5138.4
Nbal	kN	1316.3	1316.3
K		0.893	0.893
b' or, if slender, h? - b'	mm	390	390
ßa		0.018	0.018
au	mm	7.2	7.2
Madd	kNm	0.0	0.0
Eqns 32-35 ok to use?		ok	
Braced columns M1	kNm	0.0	0.0
Mi	kNm	69.4	69.4
Mi, (Mi=0 if Le/h>20)	kNm	69.4	69.4
Mi, (Mi=0 if b/h>3)		69.4	69.4

Design moments for braced columns

M2	kNm	115.7	115.7
Mi+Madd	kNm	69.4	69.4
M1+Madd/2	kNm	0.0	0.0
eminN	kNm	n/a	34.5
Maximum	kNm	115.7	115.7

Design moments (cont)

Design moments for unbraced columns

		about x-x	about y-y
M2+100% Madd	kNm	n/a	n/a
eminN	kNm	n/a	n/a
Maximum	kNm	n/a	n/a

Design moments

	kNm	**115.7**	**115.7**
		about x-x	about y-y
		Short	
		Braced	Braced

Biaxial bending

Mx/h'	0.297	
My/b		0.297
Critical direction	Y-Y	
N/bhfcu	0.28	
ß	0.66	

Maximum design moment

= 115.7+0.66*390/390*115.7

= kNm - **192.0**

Project RC Structure	REINFORCED CONCRETE COUNCIL	REINFORCED CONCRETE COUNCIL		
Client 0		Made by	Date	Page
Location Column Corner		GS	26-Jan-09	3
COLUMN LOAD TAKE DOWN & DESIGN FOR SYMMETRICALLY REINFORCED RECT. COLUMNS BENT ABOUT TWO AXES		Checked	Revision	Job No
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INPUT

Level designed: **Bottom (Max N)**

Axial load, N	1727	kN	fcu	30	N/mm²
Moment, M	192.0	kNm	fy	500	N/mm²
about	Y-Y	axis	fyv	500	N/mm²
Height, h (ll to yy, L'r to xx)	450	mm	γm	1.05	steel
Breadth, b (ll to xx)	450	mm	γm	1.5	concrete
Max bar diameter	40	mm	Link Ø	10	mm
cover (to link)	30	mm			

CALCULATIONS

from M As = {M - 0.67fcu.b.dc(h/2 - dc/2)}/[(h/2-d').(fsc+fst).gm]

from N As = (N - 0.67fcu.b.dc/gm) / (fsc - fst) As = Ast = Asc: dc=min(h,0.9x)

d' =	60	mm	.67fcu/gm =	13.4	N/mm²
d =	390	mm	fy/gm =	476.2	N/mm²
critical about Y-Y axis:...... h=	450	mm			
b=	340	mm			

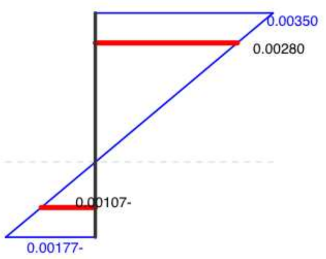
from iteration, neutral axis depth, x, =	299.0	mm	
dc	269.1	mm	
0.67.fcu.b.dc/gm	1622.7	kN	
Steel comp strain	0.00280		
Steel tens strain	0.00107		
Steel stress in comp. face, fsc	476	N/mm2	(Comp. stress in reinf.)
Steel stress in tensile face, fst	213	N/mm2	(Tensile stress in reinf.)
from M, As =	398	mm2	
from N, As =	396	mm2	OK but use min. 0.4%,

As req'd = 405mm² T&B:- PROVIDE 4TX40
(ie 2TX40 T&B - 2514mm²T&B) - 2.48% o/a - @330 cc.)
Links : - PROVIDE TX10 @ 300

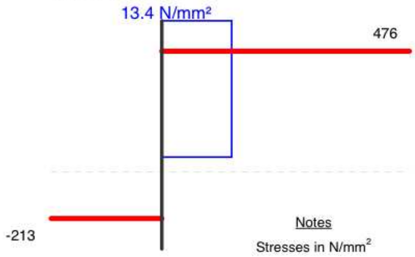
OK

Strain diagram

about Y-Y axis



Stress diagram



Notes
Stresses in N/mm^2
Compression +ve

Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	Column Side	GS	26-Jan-09	1
	COLUMN LOAD TAKE DOWN & DESIGN FOR SYMMETRICALLY REINFORCED RECT. COLUMNS BENT ABOUT TWO AXES TO BS 8110:1997	Checked	Revision	Job No
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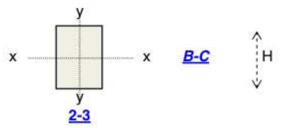
INPUT

Location Column Side

Orientation n/a



concrete density, kN/m^3 24.0

γ_{fgk} 1.40

γ_{fqk} 1.60

Dimensions

			Level	6	5	4	3	2	1
Spans	Cl to Cl	n/a	m	0.00					
		B-C	m	7.50	7.50	7.50	7.50	7.50	7.50
		1-2	m	7.50	7.50	7.50	7.50	7.50	7.50
		2-3	m	7.50	7.50	7.50	7.50	7.50	7.50
Slab	thickness (solid)		mm	250	300	300	300	300	300
	span direction,(ll to) x, y or b			b	b	b	b	b	b
Beams	width	n/a	mm	0					
	depth o/a	n/a	mm	0					
	width	B-C	mm	0					
	depth o/a	B-C	mm	0					
	width	1-2	mm	0					
	depth o/a	1-2	mm	0					
	width	2-3	mm	0					

Column below		(col above)	6	5	4	3	2	1
H (ll to yy)	mm	0	450	450	450	450	450	450
B (ll to xx)	mm	0	450	450	450	450	450	450
Height (fl. to floor.)	m	0.00	3.00	3.00	3.00	3.00	3.00	3.00

Loads (characteristic uno)

				Level	6	5	4	3	2	1
Slab	(inc swt.)	gk		kN/m2	6.43	7.78	7.78	7.78	7.78	7.78
		qk		kN/m2	1.50	3.50	3.50	3.50	3.50	3.50
Beams	(swt.)	gk		kN/m	included	included	included	included	included	included
	line loads (-extra over slab loads and beam self weight)									
	n/a	gk		kN/m	0.0					
		qk		kN/m	0.0					
	B-C	gk		kN/m	0.0					
		qk		kN/m	0.0					
	1-2	gk		kN/m	7.2	7.2	7.2	7.2	7.2	7.2
		qk		kN/m	0.0					
	2-3	gk		kN/m	7.2	7.2	7.2	7.2	7.2	7.2
		qk		kN/m	0.0					

At column position, other applied loads (eg loads from cantilevers)

Gk	kN (char)	0.0
Qk	kN (char)	0.0
Mxx	kNm (ult)	0.0
Myy	kNm (ult)	0.0

Loads per floor			6	5	4	3	2	1
Floor	Gk	kN	235.1	273.0	273.0	273.0	273.0	273.0
Floor	Qk	kN	42.2	98.4	98.4	98.4	98.4	98.4
Column below	Gk	kN	14.6	14.6	14.6	14.6	14.6	14.6

OUTPUT

		Column level	**6 to 5**	**5 to 4**	**4 to 3**	**3 to 2**	**2 to 1**	**Below 1**
Cumulative loads in column.								
	Gk	kN	249.7	537.3	824.8	1112.4	1399.9	1687.5
	Qk	kN	42.2	140.6	239.1	337.5	435.9	534.4
	Qk redn	factor	1.0	0.9	0.8	0.7	0.6	0.6
	Qk red*	kN	42.2	126.6	191.3	236.3	261.6	320.6
	N	kN	**417**	**955**	**1461**	**1935**	**2378**	**2875**
Moments in column								
about x-x Mxx	top	kNm	54.7	45.3	45.3	45.3	45.3	45.3
about y-y Myy	top	kNm	191.3	145.5	145.5	145.5	145.5	145.5
Mxx	bottom	kNm	45.3	45.3	45.3	45.3	45.3	
Myy	bottom	kNm	145.5	145.5	145.5	145.5	145.5	

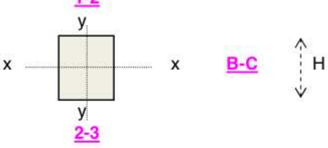
Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client	0	Made by	Date	Page
Location	Column Side	GS	26-Jan-09	2
	COLUMN LOAD TAKE DOWN & DESIGN FOR SYMMETRICALLY REINFORCED RECT. COLUMNS BENT ABOUT TWO AXES	Checked	Revision	Job No
	Originated from RCC51.xls on CD © 1999 BCA for RCC	GS	-	-

INPUT

Location **Column Side**

Level considered: **Bottom (Max N)**

Orientation 1-2



Dimensions		about x-x	about y-y
h (ll to yy)	mm	450	-
b (ll to xx)	mm	-	450
lo, clear height	mm	2750	2750
ß	value	0.85	0.85
Braced or Unbraced?	B or U	B	B

Loads

Axial	N	kN	2875	-
Moments	top	kNm	45.3	145.5
	bottom	kNm	0.0	0.0

Column properties

fcu		30
fy		500
cover to link	mm	30
Max sized main bar	mm	40
Probable percentage As	%	2.50%
link diameter	mm	10

OUTPUT

Design criteria

N	kNm	**2875**
M	kN	**165.0**
about		**Y-Y**

PROOF

Slenderness

le	mm	2338	2337.5
Slenderness		5.19	5.19
Limit for short column		15.0	15.0
Design column as		Short	
Column is		Short	Short
		about x-x	about y-y

Design moments

Min eccentricity, 0.05 h	kNm	57.5	
Madd	n/a - short column		
d	mm	390	390
Nuz	kN	5138.4	5138.4
Nbal	kN	1316.3	1316.3
K		0.592	0.592
b' or, if slender, h? - b'	mm	390	390
ßa		0.018	0.018
au	mm	4.8	4.8
Madd	kNm	0.0	0.0
Eqns 32-35 ok to use?		ok	
Braced columns M1	kNm	0.0	0.0
Mi	kNm	27.2	87.3
Mi, (Mi=0 if Le/h>20)	kNm	27.2	87.3
Mi, (Mi=0 if b/h>3)		27.2	87.3

Design moments for braced columns

M2	kNm	45.3	145.5
Mi+Madd	kNm	27.2	87.3
M1+Madd/2	kNm	0.0	0.0
eminN	kNm	n/a	57.5
Maximum	kNm	45.3	145.5

Design moments (cont)

Design moments for unbraced columns

		about x-x	about y-y
M2+100% Madd	kNm	n/a	n/a
eminN	kNm	n/a	n/a
Maximum	kNm	n/a	n/a

Design moments

	about x-x	about y-y
kNm	**45.3**	**145.5**
	Short	
	Braced	Braced

Biaxial bending

Mx/h'	0.116	
My/b		0.373
Critical direction	Y-Y	
N/bhfcu	0.47	
ß	0.43	

Maximum design moment

= 145.5+0.43*390/390*45.3

= kNm - **165.0**

Project RC Structure		REINFORCED CONCRETE COUNCIL			
Client 0		Made by GS	Date 26-Jan-09	Page 3	
Location Column Side					
COLUMN LOAD TAKE DOWN & DESIGN FOR SYMMETRICALLY REINFORCED RECT. COLUMNS BENT ABOUT TWO AXES		Checked GS	Revision -	Job No -	
Originated from RCC51.xls on CD	© 1999 BCA for RCC				

INPUT

Level designed: **Bottom (Max N)**

Axial load, N	2875	kN	fcu	30	N/mm²
Moment, M	165.0	kNm	fy	500	N/mm²
about	Y-Y	axis	fyv	500	N/mm²
Height, h (ll to yy, L'r to xx)	450	mm	γm	1.05	steel
Breadth, b (ll to xx)	450	mm	γm	1.5	concrete
Max bar diameter	40	mm	Link Ø	10	mm
cover (to link)	30	mm			

CALCULATIONS

from M As = {M - 0.67fcu.b.dc(h/2 - dc/2)}/[(h/2-d').(fsc+fst).gm]

from N As = (N - 0.67fcu.b.dc/gm) / (fsc - fst) As = Ast = Asc: dc=min(h,0.9x)

d' =	60	mm	.67fcu/gm =	13.4	N/mm²
d =	390	mm	fy/gm =	476.2	N/mm²
critical about Y-Y axis:…... h=	450	mm			
b=	340	mm			

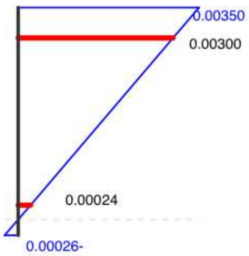
from iteration, neutral axis depth, x, =	418.4	mm	
dc	376.5	mm	
0.67.fcu.b.dc/gm	2270.6	kN	
Steel comp strain	0.00300		
Steel tens strain	-0.00024		
Steel stress in comp. face, fsc	476	N/mm2	(Comp. stress in reinf.)
Steel stress in tensile face, fst	-47	N/mm2	(Tensile stress in reinf.)
from M, As =	1154	mm2	
from N, As =	1154	mm2	OK

As req'd = 1154mm² T&B:- PROVIDE 4TX40
(ie 2TX40 T&B - 2514mm²T&B) - 2.48% o/a - @330 cc.)
Links : - PROVIDE TX10 @ 300

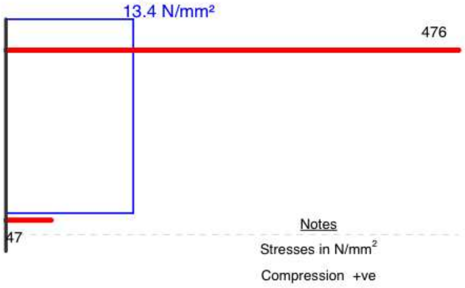
OK

Strain diagram

about Y-Y axis



Stress diagram

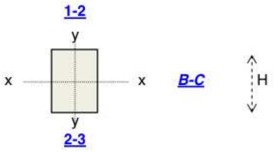


Project	RC Structure	REINFORCED CONCRETE COUNCIL		
Client		Made by	Date	Page
Location	Column Internal	GS	26-Jan-09	1
	COLUMN LOAD TAKE DOWN & DESIGN FOR SYMMETRICALLY REINFORCED RECT. COLUMNS BENT ABOUT TWO AXES TO BS 8110:1997	Checked	Revision	Job No
	Originated from RCC51.xls on CD © 1999 BCA for RCC	GS	-	-

INPUT

Location Column Internal

Orientation



concrete density, kN/m^3	24.0
γ_{fgk}	1.40
γ_{fqk}	1.60

				Level	6	5	4	3	2	1
Dimensions										
Spans	Cl to Cl	A-B	m		7.50	7.50	7.50	7.50	7.50	7.50
		B-C	m		7.50	7.50	7.50	7.50	7.50	7.50
		1-2	m		7.50	7.50	7.50	7.50	7.50	7.50
		2-3	m		7.50	7.50	7.50	7.50	7.50	7.50
Slab	thickness (solid)		mm		250	300	300	300	300	300
	span direction,(ll to) x, y or b				b	b	b	b	b	b
Beams	width	A-B	mm		0					
	depth o/a	A-B	mm		0					
	width	B-C	mm		0					
	depth o/a	B-C	mm		0					
	width	1-2	mm		0					
	depth o/a	1-2	mm		0					
	width	2-3	mm		0					
Column below				(col above)						
	H (ll to yy)		mm	0	450	450	450	450	450	450
	B (ll to xx)		mm	0	450	450	450	450	450	450
Height (fl. to floor.)			m	0.00	3.00	3.00	3.00	3.00	3.00	3.00

				Level	6	5	4	3	2	1
Loads	(characteristic uno)									
Slab	(inc swt.)	gk	kN/m2		6.43	7.78	7.78	7.78	7.78	7.78
		qk	kN/m2		1.50	3.50	3.50	3.50	3.50	3.50
Beams	(swt.)	gk	kN/m		included	included	included	included	included	included
	line loads (-extra over slab loads and beam self weight)									
	A-B	gk	kN/m		0.0					
		qk	kN/m		0.0					
	B-C	gk	kN/m		0.0					
		qk	kN/m		0.0					
	1-2	gk	kN/m		0.0					
		qk	kN/m		0.0					
	2-3	gk	kN/m		0.0					
		qk	kN/m		0.0					
At column position, other applied loads (eg loads from cantilevers)										
		Gk	kN (char)		0.0					
		Qk	kN (char)		0.0					
		Mxx	kNm (ult)		0.0					
		Myy	kNm (ult)		0.0					

Loads per floor								
Floor	Gk	kN	361.7	437.3	437.3	437.3	437.3	437.3
Floor	Qk	kN	84.4	196.9	196.9	196.9	196.9	196.9
Column below	Gk	kN	14.6	14.6	14.6	14.6	14.6	14.6

OUTPUT

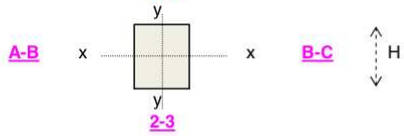
		Column level		**6 to 5**	**5 to 4**	**4 to 3**	**3 to 2**	**2 to 1**	**Below 1**
Cumulative loads in column.									
		Gk	kN	376.3	828.2	1280.1	1732.0	2184.0	2635.9
		Qk	kN	84.4	281.3	478.1	675.0	871.9	1068.8
		Qk redn	factor	1.0	0.9	0.8	0.7	0.6	0.6
		Qk red*	kN	84.4	253.1	382.5	472.5	523.1	641.3
		N	kN	**662**	**1564**	**2404**	**3181**	**3895**	**4716**
Moments in column									
about x-x	Mxx	top	kNm	61.2	57.8	57.8	57.8	57.8	57.8
about y-y	Myy	top	kNm	61.2	57.8	57.8	57.8	57.8	57.8
	Mxx	bottom	kNm	57.8	57.8	57.8	57.8	57.8	
	Myy	bottom	kNm	57.8	57.8	57.8	57.8	57.8	

Project	RC Structure	REINFORCED CONCRETE COUNCIL	
Client	0	Made by	Date	Page
Location	Column Internal	GS	26-Jan-09	2
	COLUMN LOAD TAKE DOWN & DESIGN FOR SYMMETRICALLY REINFORCED RECT. COLUMNS BENT ABOUT TWO AXES	Checked	Revision	Job No
	Originated from RCC51.xls on CD © 1999 BCA for RCC	GS	-	-

INPUT

Location **Column Internal**
Level considered: **Bottom (Max N)**

Orientation 1-2



Dimensions		about x-x	about y-y
h (ll to yy)	mm	450	-
b (ll to xx)	mm	-	450
lo, clear height	mm	2750	2750
ß	value	0.85	0.85
Braced or Unbraced?	B or U	B	B
Loads			
Axial N	kN	4716	-
Moments top	kNm	57.8	57.8
bottom	kNm	0.0	0.0

Column properties		
fcu		30
fy		500
cover to link	mm	30
Max sized main bar	mm	40
Probable percentage As	%	2.50%
link diameter	mm	10

OUTPUT

Design criteria

N	kNm	**4716**
M	kN	**111.7**
about		**Y-Y**

PROOF

Slenderness

le	mm	2338	2337.5
Slenderness		5.19	5.19
Limit for short column		15.0	15.0
Design column as		Short	
Column is		Short	Short
		about x-x	about y-y

Design moments

Min eccentricity, 0.05 h	kNm	94.3	
Madd		n/a - short column	
d	mm	390	390
Nuz	kN	5138.4	5138.4
Nbal	kN	1316.3	1316.3
K		0.111	0.111
b' or, if slender, h? - b'	mm	390	390
ßa		0.018	0.018
au	mm	0.9	0.9
Madd	kNm	0.0	0.0
Eqns 32-35 ok to use?		ok	
Braced columns M1	kNm	0.0	0.0
Mi	kNm	34.7	34.7
Mi, (Mi=0 if Le/h>20)	kNm	34.7	34.7
Mi, (Mi=0 if b/h>3)		34.7	34.7

Design moments for braced columns

M2	kNm	57.8	57.8
Mi+Madd	kNm	34.7	34.7
M1+Madd/2	kNm	0.0	0.0
eminN	kNm	n/a	94.3
Maximum	kNm	57.8	94.3

Design moments (cont)

Design moments for unbraced columns

		about x-x	about y-y
M2+100% Madd	kNm	n/a	n/a
eminN	kNm	n/a	n/a
Maximum	kNm	n/a	n/a

Design moments

kNm	**57.8**	**94.3**
	about x-x	about y-y
	Short	
	Braced	Braced

Biaxial bending

Mx/h'	0.148	
My/b		0.242
Critical direction	Y-Y	
N/bhfcu	0.78	
ß	0.30	

Maximum design moment

= 94.3+0.30*390/390*57.8

= kNm - **111.7**

Project RC Structure		REINFORCED CONCRETE COUNCIL			
Client 0		Made by GS	Date 26-Jan-09	Page 3	
Location Column Internal					
COLUMN LOAD TAKE DOWN & DESIGN FOR SYMMETRICALLY REINFORCED RECT. COLUMNS BENT ABOUT TWO AXES		Checked GS	Revision -	Job No -	
Originated from RCC51.xls on CD	© 1999 BCA for RCC				

INPUT

Level designed: **Bottom (Max N)**

Axial load, N	4716	kN	fcu	30	N/mm²
Moment, M	111.7	kNm	fy	500	N/mm²
about	Y-Y	axis	fyv	500	N/mm²
Height, h (ll to yy, L'r to xx)	450	mm	γm	1.05	steel
Breadth, b (ll to xx)	450	mm	γm	1.5	concrete
Max bar diameter	40	mm	Link Ø	10	mm
cover (to link)	30	mm			

CALCULATIONS

from M As = {M - 0.67fcu.b.dc(h/2 - dc/2)}/[(h/2-d').(fsc+fst).gm]

from N As = (N - 0.67fcu.b.dc/gm) / (fsc - fst) As = Ast = Asc: dc=min(h,0.9x)

d' =	60	mm	.67fcu/gm =	13.4	N/mm²
d =	390	mm	fy/gm =	476.2	N/mm²
critical about Y-Y axis:...... h=	450	mm			
b=	340	mm			

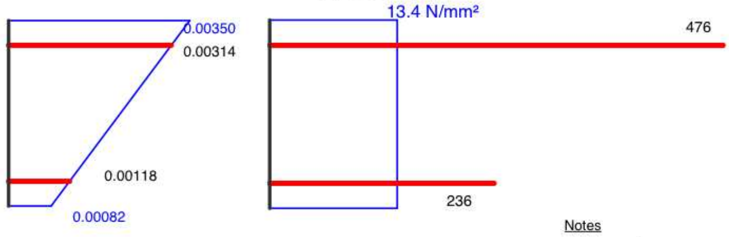
from iteration, neutral axis depth, x, =	587.9	mm	
dc	450.0	mm	
0.67.fcu.b.dc/gm	2713.5	kN	
Steel comp strain	0.00314		
Steel tens strain	-0.00118		
Steel stress in comp. face, fsc	476	N/mm2	(Comp. stress in reinf.)
Steel stress in tensile face, fst	-236	N/mm2	(Tensile stress in reinf.)
from M, As =	2813	mm2	
from N, As =	2813	mm2	OK

As req'd = 2813mm² T&B:- PROVIDE 8TX40
(ie 3TX40 T&B - 3770mm²T&B) - 4.97% o/a - @165 cc.)
Links : - PROVIDE TX10 @ 300

OK

Strain diagram   Stress diagram

about Y-Y axis



Notes
Stresses in N/mm²
Compression +ve

APPENDIX D STRUCTURAL FOUNDATION DESIGN

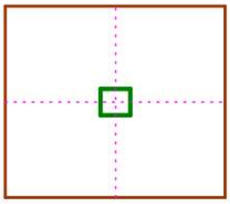
Project	Pad Foundation		REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	Corner	Single column base	GS	26-Jan-09	1
	PAD FOUNDATION DESIGN to BS 8110:1997		Checked	Revision	Job No
	Originated from RCC81.xls on CD	© 1999 BCA for RCC	GS	-	

MATERIALS fcu 30 N/mm² h agg 20 mm γ_c 1.5 concrete
fy 500 N/mm² cover 50 mm γ_s 1.05 steel
Densities - Concrete 24 kN/m³ Soil 20 kN/m³
Bearing pressure 400 kN/m² (net allowable increase)

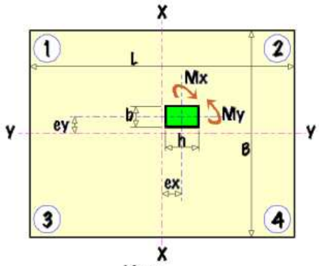
DIMENSIONS mm

BASE
L = 3300
B = 3300
depth H = 500
ex = 0

COLUMN
h = 450
b = 450
ey = 0



Plot (to scale)

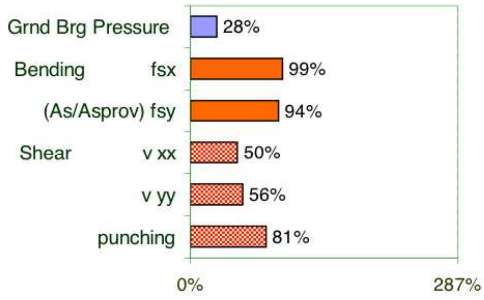


Key

COLUMN REACTIONS kN, kNm *characteristic*

	DEAD	IMPOSED	WIND
Axial (kN)	1050.0	160.0	
Mx (kNm)			
My (kNm)			
Hx (kN)			
Hy (kN)			

STATUS VALID DESIGN



BEARING PRESSURES kN/m² *characteristic*

CORNER	1	2	3	4
no wind	113.1	113.1	113.1	113.1
with wind	113.1	113.1	113.1	113.1

REINFORCEMENT Detail to 3.11.3.2

Mxx = 531.0 kNm
b = 3300 mm
d = 440 mm
As = 2668 mm²
PROVIDE 9 T20 @ 275 & 500 B1
As prov = 2827 mm²

Detail to 3.11.3.2

Myy = 531.0 kNm
b = 3300 mm
d = 420 mm
As = 2795 mm²
PROVIDE 9 T20 @ 275 & 525 B2
As prov = 2827 mm²

BEAM SHEAR

Vxx = 515.2 kN at d from col face
v = 0.355 N/mm²
or Vxx = 285.1 kN at 2d from col face
v = 0.196 N/mm²
vc = 0.389 N/mm²

Vyy = 525.6 kN at d from col face
v = 0.379 N/mm²
or Vyy = 306.0 kN at 2d from col face
v = 0.221 N/mm²
vc = 0.395 N/mm²

PUNCHING SHEAR

d ave = 430 mm
As prov = 0.199 %
v = 0.318 N/mm²

u crit = 6600 mm
v max = 2.420 N/mm² at col face
vc = 0.392 N/mm²

Project	Pad Foundation		REINFORCED CONCRETE COUNCIL		
Client			Made by	Date	Page
Location	Edge	Single column base	GS	26-Jan-09	1
	PAD FOUNDATION DESIGN to BS 8110:1997		Checked	Revision	Job No
	Originated from RCC81.xls on CD	© 1999 BCA for RCC	GS	-	

MATERIALS fcu 30 N/mm² h agg 20 mm γ_c 1.5 concrete

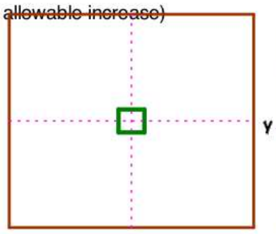
fy 500 N/mm² cover 50 mm γ_s 1.05 steel

Densities - Concrete 24 kN/m³ Soil 20 kN/m³

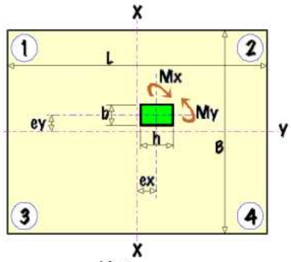
Bearing pressure 400 kN/m² (net allowable increase)

DIMENSIONS mm

BASE	COLUMN
L = 4200	h = 450
B = 4200	b = 450
depth H = 650	
ex = 0	ey = 0



Plot (to scale)

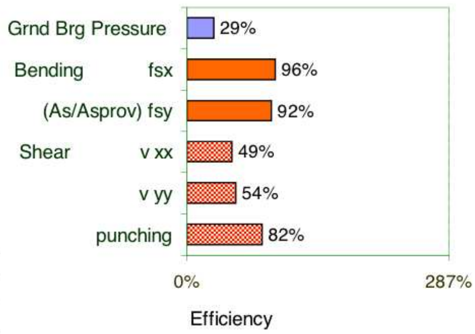


Key

COLUMN REACTIONS kN, kNm *characteristic*

	DEAD	IMPOSED	WIND
Axial (kN)	1688.0	321.0	
Mx (kNm)			
My (kNm)			
Hx (kN)			
Hy (kN)			

STATUS VALID DESIGN



BEARING PRESSURES kN/m² *characteristic*

CORNER	1	2	3	4
no wind	116.5	116.5	116.5	116.5
with wind	116.5	116.5	116.5	116.5

REINFORCEMENT

Detail to 3.11.3.2			Detail to 3.11.3.2		
Mxx =	1204.0	kNm	Myy =	1204.0	kNm
b =	4200	mm	b =	4200	mm
d =	587.5	mm	d =	562.5	mm
As =	4530	mm²	As =	4732	mm²
PROVIDE 10 T25 @ 325 & 575 B1			PROVIDE 10 T25 @ 300 & 600 B2		
As prov =	4909	mm²	As prov =	4909	mm²

BEAM SHEAR

Vxx =	880.2	kN at d from col face	Vyy =	893.9	kN at d from col face
v =	0.357	N/mm²	v =	0.378	N/mm²
or Vxx =	476.0	kN at 2d from col face	or Vyy =	503.4	kN at 2d from col face
v =	0.193	N/mm²	v =	0.213	N/mm²
vc =	0.392	N/mm²	vc =	0.398	N/mm²

PUNCHING SHEAR

d ave =	575	mm	u crit =	8400	mm
As prov =	0.203	%	v max =	3.116	N/mm² at col face
v =	0.326	N/mm²	vc =	0.395	N/mm²

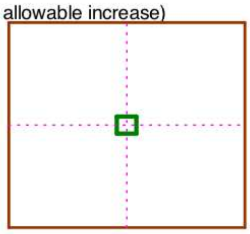
Project	Pad Foundation	**REINFORCED CONCRETE COUNCIL**		
Client		Made by	Date	Page
Location	Internal — Single column base	GS	26-Jan-09	1
	PAD FOUNDATION DESIGN to BS 8110:1997	Checked	Revision	Job No
	Originated from RCC81.xls on CD © 1999 BCA for RCC	GS	-	

MATERIALS

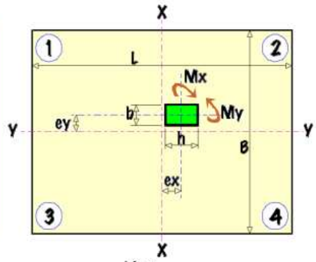
fcu	30	N/mm²	h agg	20	mm	γ_c	1.5	concrete
fy	500	N/mm²	cover	50	mm	γ_s	1.05	steel
Densities - Concrete	24	kN/m³	Soil	20	kN/m³			
Bearing pressure	400	kN/m² (net allowable increase)						

DIMENSIONS mm

BASE		COLUMN	
L =	5400	h =	450
B =	5400	b =	450
depth H =	850		
ex =	0	ey =	0



Plot (to scale)

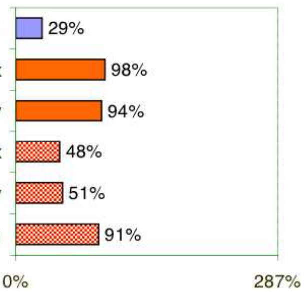


Key

COLUMN REACTIONS kN, kNm *characteristic*

	DEAD	IMPOSED	WIND
Axial (kN)	2636.0	641.0	
Mx (kNm)			
My (kNm)			
Hx (kN)			
Hy (kN)			

STATUS VALID DESIGN



BEARING PRESSURES kN/m² *characteristic*

CORNER	1	2	3	4
no wind	115.8	115.8	115.8	115.8
with wind	115.8	115.8	115.8	115.8

REINFORCEMENT

Detail to 3.11.3.2			Detail to 3.11.3.2		
Mxx =	2674.9	kNm	Myy =	2674.9	kNm
b =	5400	mm	b =	5400	mm
d =	784	mm	d =	752	mm
As =	7542	mm²	As =	7863	mm²
PROVIDE 10 T32 @ 400 & 775 B1			PROVIDE 10 T32 @ 400 & 800 B2		
As prov =	8042	mm²	As prov =	8042	mm²

BEAM SHEAR

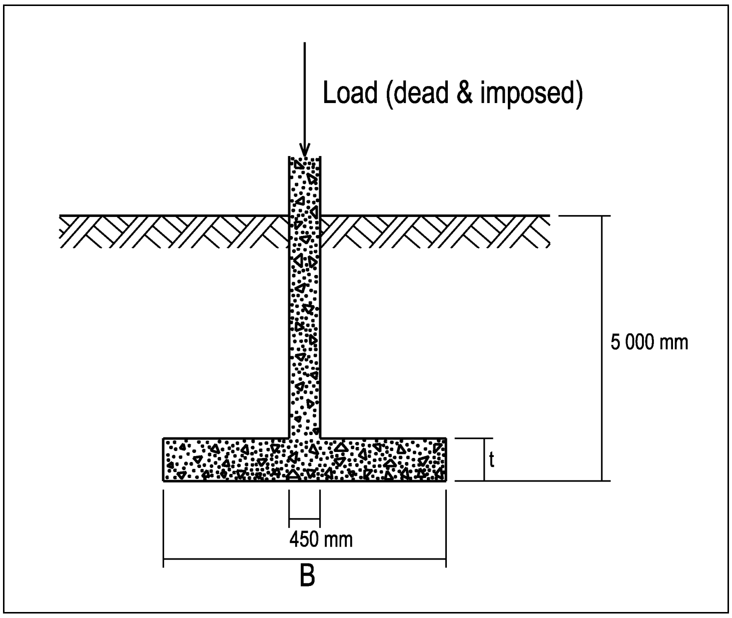
Vxx =	1471.6	kN at d from col face	Vyy =	1489.0	kN at d from col face
v =	0.348	N/mm²	v =	0.367	N/mm²
or Vxx =	781.6	kN at 2d from col face	or Vyy =	816.6	kN at 2d from col face
v =	0.185	N/mm²	v =	0.201	N/mm²
vc =	0.386	N/mm²	vc =	0.391	N/mm²

PUNCHING SHEAR

d ave =	768	mm	u crit =	10800	mm
As prov =	0.194	%	v max =	3.986	N/mm² at col face
v =	0.328	N/mm²	vc =	0.389	N/mm²

APPENDIX E GEOTECHNICAL FOUNDATION DESIGN

1 Corner pad foundation design



B	foundation width	3.3 m
t	foundation thickness	0.5 m
F_D	characteristic dead load	1050.0 kN
F_L	characteristic imposed load	160.0 kN
γ_s	characteristic weight density of soil	20 kN/m³
γ_c	characteristic weight density of concrete	24 kN/m³
c_u	characteristic value of undrained shear strength	90 kPa

Bearing resistance

Required

$V_d \leq R_d$ — 6.5.2.1(1) *Equation 6.1*

V_d – Design value of vertical load
R_d – Design value of resistance to vertical load

Permanent vertical characteristic load — *2.4.2(4) & 6.5.2.1(3)*

Imposed vertical load on column	1050.0 kN
Weight of foundation:	
Weight of rising column	21.9 kN
Weight of foundation pad	130.7 kN
Weight of backfill	961.9 kN
Total foundation weight	1114.4 kN
Total characteristic permanent load (V_k)	2164.4 kN

Variable vertical characteristic load

Total characteristic variable load (V_L) 160 kN

Design approach 1 *2.4.7.3.4.2*

Analytical method *6.5.2.2*
Annex D3

Undrained conditions

$R/A = (\pi + 2)\, c_u s_c + q$ *Equation D1*

Shape factor

Square footing, $s_c = 1.2$

Area of footing

Pad 3.3 m x 3.3 m, $A = 10.89\ m^2$

Bearing resistance

$R = 10.89\, [(\pi + 2)1.2c_u + q]$

Combination 1: A1, M1 and R1 *2.4.7.3.4.2*

Design load (A1)

$V_{d1} = \gamma_G \times V_k + \gamma_Q \times V_L$ *Table A3*
$V_{d1} = 1.35 \times 2164.4 + 1.50 \times 160$
$V_{d1} = 3162.0$ kN

Design strength (M1) *Table A4*

$C_{ud1} = c_{uk} / \gamma_{cu}$
$C_{ud1} = 90 / 1.0$
$C_{ud1} = 90$ kPa

Soil surcharge, design value adjacent to footing (A1) *Table A3*

$q_{d1} = q_k \times \gamma_G$
$q_{d1} = (5 \times 20) \times 1.0$
$q_{d1} = 100$ kPa

Design bearing resistance (R1) *Table A5*

$R_{d1} = R_k / \gamma_{R;v}$
$R_{d1} = R_k / 1.0$
$R_{d1} = R_k$
$R_{d1} = 10.89\, [(\pi + 2)1.2 \times 90 + 100]$
$R_{d1} = 7136.1$ kN

Check

V_{d1} should be $\leq R_{d1}$
3162.0 kN < 7136.1 kN

Footing acceptable for design approach 1, combination 1

Combination 2: A2, M2 and R1 *2.4.7.3.4.2*

Design load (A2)

$V_{d2} = \gamma_G \times V_k + \gamma_Q \times V_L$ *Table A3*

$V_{d2} = 1.0 \times 2164.4 + 1.3 \times 160$

$V_{d2} = 2372.4$ kN

Design strength (M2) *Table A4*

$C_{ud2} = c_{uk} / \gamma_{cu}$

$C_{ud2} = 90 / 1.4$

$C_{ud2} = 64.3$ kPa

Soil surcharge, design value adjacent to footing (A2) *Table A3*

$q_{d2} = q_k \times \gamma_G$

$q_{d2} = (5 \times 20) \times 1.0$

$q_{d2} = 100$ kPa

Design bearing resistance (R1) *Table A5*

$R_{d2} = R_k / \gamma_{R;v}$

$R_{d2} = R_k / 1.0$

$R_{d2} = R_k$

$R_{d2} = 10.89\ [(\pi + 2)1.2 \times 64.3 + 100]$

$R_{d2} = 5408.4$ kN

Check

V_{d2} should be $\leq R_{d2}$

2372.4 kN < 5408.4 kN

Footing acceptable for design approach 1, combination 2

Settlement

Settlement using $V_k \leq R_k / 3$

$V_k + V_L = 2324.4$ kN

Bearing capacity R_{d1} unfactored

$\gamma_{R;v}$ = 1.0 (bearing resistance)

γ_G = 1.0 (surcharge)

γ_{cu} = 1.0 shear strength)

$R_k = R_{d1}$

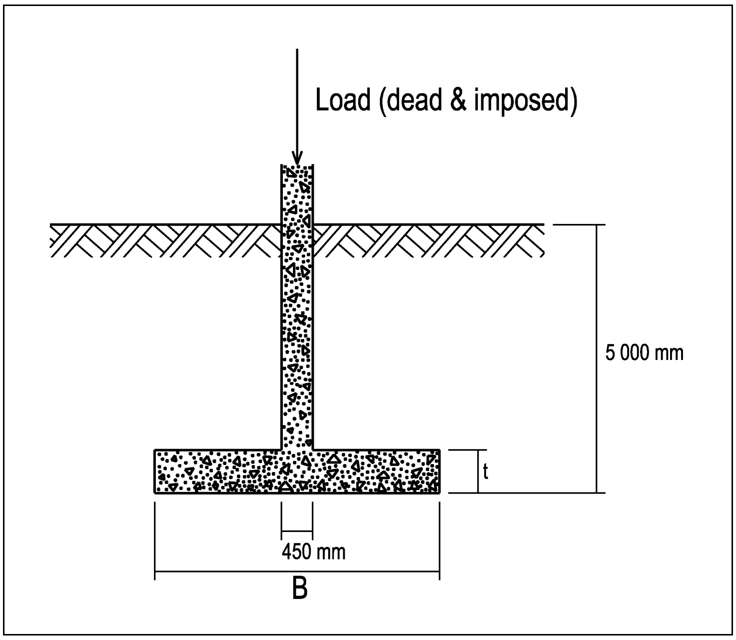
$R_k = 7136.1$ kN

Check

$R_k / (V_k + V_L) = 7136.1/2324.4 = 3.07$ (>3)

Footing acceptable for settlement

2 Edge pad foundation design



B	foundation width	4.2 m
t	foundation thickness	0.65 m
F_D	characteristic dead load	1688.0 kN
F_L	characteristic imposed load	321.0 kN
γ_s	characteristic weight density of soil	20 kN/m³
γ_c	characteristic weight density of concrete	24 kN/m³
c_u	characteristic value of undrained shear strength	90 kPa

Bearing resistance

Required

$V_d \leq R_d$ 6.5.2.1(1) *Equation 6.1*

V_d – Design value of vertical load
R_d – Design value of resistance to vertical load

Permanent vertical characteristic load *2.4.2(4) & 6.5.2.1(3)*

Imposed vertical load on column	1688.0 kN
Weight of foundation:	
Weight of rising column	21.1 kN
Weight of foundation pad	275.2 kN
Weight of backfill	1517.1 kN
Total foundation weight	1813.4 kN
Total characteristic permanent load (V_k)	3501.4 kN

Variable vertical characteristic load

Total characteristic variable load (V_L) 321.0 kN

Design approach 1 *2.4.7.3.4.2*

Analytical method *6.5.2.2 Annex D3*

Undrained conditions

$R/A = (\pi + 2)\, c_u s_c + q$ *Equation D1*

Shape factor

Square footing, $s_c = 1.2$

Area of footing

Pad 4.2 m x 4.2 m, A = 17.64 m²

Bearing resistance

$R = 17.64\, [(\pi + 2)1.2c_u + q]$

Combination 1: A1, M1 and R1 *2.4.7.3.4.2*

Design load (A1)

$V_{d1} = \gamma_G \times V_k + \gamma_Q \times V_L$ *Table A3*
$V_{d1} = 1.35 \times 3501.4 + 1.50 \times 321$
$V_{d1} = 5208.4$ kN

Design strength (M1) *Table A4*

$C_{ud1} = c_{uk} / \gamma_{cu}$
$C_{ud1} = 90 / 1.0$
$C_{ud1} = 90$ kPa

Soil surcharge, design value adjacent to footing (A1) *Table A3*

$q_{d1} = q_k \times \gamma_G$
$q_{d1} = (5 \times 20) \times 1.0$
$q_{d1} = 100$ kPa

Design bearing resistance (R1) *Table A5*

$R_{d1} = R_k / \gamma_{R;v}$
$R_{d1} = R_k / 1.0$
$R_{d1} = R_k$
$R_{d1} = 17.64\, [(\pi + 2)1.2 \times 90 + 100]$
$R_{d1} = 11559.4$ kN

Check

V_{d1} should be $\leq R_{d1}$
5208.4 kN < 11559.4 kN
Footing acceptable for design approach 1, combination 1
Combination 2: A2, M2 and R1 *2.4.7.3.4.2*

Design load (A2)

$V_{d2} = \gamma_G \times V_k + \gamma_Q \times V_L$ *Table A3*

$V_{d2} = 1.0 \times 3501.4 + 1.3 \times 321$

$V_{d2} = 3918.7$ kN

Design strength (M2) *Table A4*

$C_{ud2} = c_{uk} / \gamma_{cu}$

$C_{ud2} = 90 / 1.4$

$C_{ud2} = 64.3$ kPa

Soil surcharge, design value adjacent to footing (A2) *Table A3*

$q_{d2} = q_k \times \gamma_G$

$q_{d2} = (5 \times 20) \times 1.0$

$q_{d2} = 100$ kPa

Design bearing resistance (R1) *Table A5*

$R_{d2} = R_k / \gamma_{R;v}$

$R_{d2} = R_k / 1.0$

$R_{d2} = R_k$

$R_{d2} = 17.64\ [(\pi + 2)1.2 \times 64.3 + 100]$

$R_{d2} = 8760.7$ kN

Check

V_{d2} should be $\leq R_{d2}$

3918.7 kN < 8760.7 kN

Footing acceptable for design approach 1, combination 2

Settlement

Settlement using $V_k \leq R_k / 3$

$V_k + V_L = 38224$ kN

Bearing capacity R_{d1} unfactored

$\gamma_{R;v}$ = 1.0 (bearing resistance)

γ_G = 1.0 (surcharge)

γ_{cu} = 1.0 shear strength)

$R_k = R_{d1}$

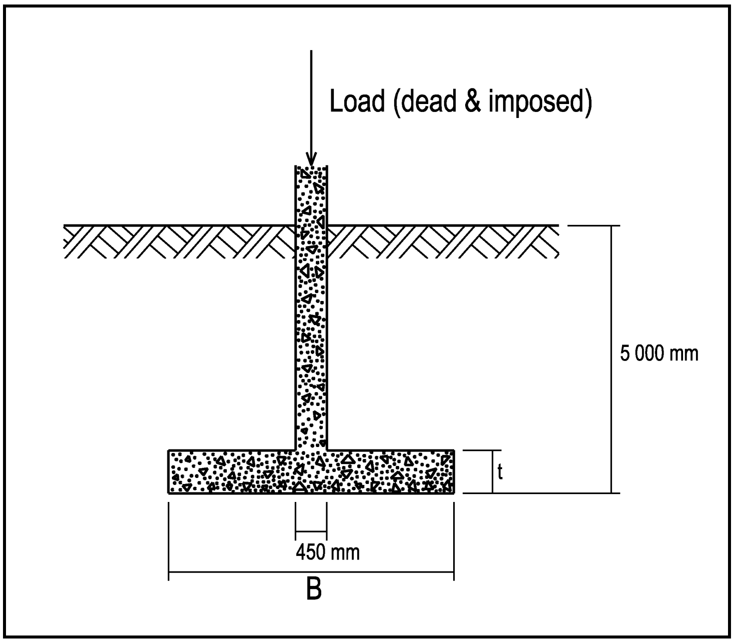
$R_k = 11559.4$ kN

Check

$R_k / (V_k + V_L)$= 3822.4/11559.4 = 3.02 (>3)

Footing acceptable for settlement

3 Internal pad foundation design



B	foundation width	5.4 m
t	foundation thickness	0.85 m
F_D	characteristic dead load	2636.0 kN
F_L	characteristic imposed load	641.0 kN
γ_s	characteristic weight density of soil	20 kN/m³
γ_c	characteristic weight density of concrete	24 kN/m³
c_u	characteristic value of undrained shear strength	90 kPa

Bearing resistance

Required

$V_d \leq R_d$ — 6.5.2.1(1) *Equation 6.1*

V_d – Design value of vertical load
R_d – Design value of resistance to vertical load

Permanent vertical characteristic load — *2.4.2(4) & 6.5.2.1(3)*

Imposed vertical load on column	2636.0 kN
Weight of foundation:	
Weight of rising column	20.2 kN
Weight of foundation pad	594.9 kN
Weight of backfill	2403.5 kN
Total foundation weight	3018.5 kN
Total characteristic permanent load (V_k)	5654.5 kN

Variable vertical characteristic load

Total characteristic variable load (V_L) 641.0 kN

Design approach 1 *2.4.7.3.4.2*

Analytical method *6.5.2.2 Annex D3*

Undrained conditions

$R/A = (\pi + 2)\ c_u s_c + q$ *Equation D1*

Shape factor

Square footing, $s_c = 1.2$

Area of footing

Pad 5.4 m x 5.4 m, A = 29.16 m²

Bearing resistance

$R = 29.16\ [(\pi + 2)1.2c_u + q]$

Combination 1: A1, M1 and R1 *2.4.7.3.4.2*

Design load (A1)

$V_{d1} = \gamma_G \times V_k + \gamma_Q \times V_L$ *Table A3*
$V_{d1} = 1.35 \times 5654.5 + 1.50 \times 641.0$
$V_{d1} = 8595.1$ kN

Design strength (M1) *Table A4*

$C_{ud1} = c_{uk} / \gamma_{cu}$
$C_{ud1} = 90 / 1.0$
$C_{ud1} = 90$ kPa

Soil surcharge, design value adjacent to footing (A1) *Table A3*

$q_{d1} = q_k \times \gamma_G$
$q_{d1} = (5 \times 20) \times 1.0$
$q_{d1} = 100$ kPa

Design bearing resistance (R1) *Table A5*

$R_{d1} = R_k / \gamma_{R;v}$
$R_{d1} = R_k / 1.0$
$R_{d1} = R_k$
$R_{d1} = 29.16\ [(\pi + 2)1.2 \times 90 + 100]$
$R_{d1} = 19108.3$ kN

Check

V_{d1} should be $\leq R_{d1}$
8595.1 kN < 19108.3 kN

Footing acceptable for design approach 1, combination 1

Combination 2: A2, M2 and R1 *2.4.7.3.4.2*

Design load (A2)

$V_{d2} = \gamma_G \times V_k + \gamma_Q \times V_L$ *Table A3*

$V_{d2} = 1.0 \times 5654.5 + 1.3 \times 641$

$V_{d2} = 6487.8$ kN

Design strength (M2) *Table A4*

$C_{ud2} = c_{uk} / \gamma_{cu}$

$C_{ud2} = 90 / 1.4$

$C_{ud2} = 64.3$ kPa

Soil surcharge, design value adjacent to footing (A2) *Table A3*

$q_{d2} = q_k \times \gamma_G$

$q_{d2} = (5 \times 20) \times 1.0$

$q_{d2} = 100$ kPa

Design bearing resistance (R1) *Table A5*

$R_{d2} = R_k / \gamma_{R;v}$

$R_{d2} = R_k / 1.0$

$R_{d2} = R_k$

$R_{d2} = 29.16\,[(\pi + 2)1.2 \times 64.3 + 100]$

$R_{d2} = 14481.9$ kN

Check

V_{d2} should be $\leq R_{d2}$

6487.8 kN < 14481.9 kN

Footing acceptable for design approach 1, combination 2

Settlement

Settlement using $V_k \leq R_k / 3$

$V_k + V_L = 6295.5$ kN

Bearing capacity R_{d1} unfactored

$\gamma_{R;v}$ = 1.0 (bearing resistance)

γ_G = 1.0 (surcharge)

γ_{cu} = 1.0 shear strength)

$R_k = R_{d1}$

R_k = 19108.3 kN

Check

$R_k / (V_k + V_L)$= 6295.5/19108.3 = 3.04 (>3)

Footing acceptable for settlement