FIRST FLOOR, UNIT 6
UNION PARK
PACKET BOAT LANE
UXBRIDGE UB8 2GH

TEL: 01895 430700
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MMP DESIGN

Project No. 4402

Proposed Construction of a Single Level Basement and Alterations at;

65 Goldhurst Terrace, NW6 3HB

Structural Design Calculations

S. R. MASTERS
B.Sc.(Hons).,C.Eng.,M.I.Struct.E.,M.B.Eng.

October 2014

Contents

INTRODUCTION TO MMP DESIGN

MMP Design Limited was formed as a private limited company in 1988 by one of the current Directors. Since then it has developed into it's present form as a firm of consulting engineers with expertise in Structural and Civil Engineering Services.

Within the Company experience has been gained in a range of projects from structural surveys through refurbishment to multi-million pound developments and the Directors have experience in residential, retail, commercial, community care and educational projects. The Company also has commitment to all types of work including Design and Construct projects.

The Company philosophy is to provide the fullest and most cost effective service to Clients. The Directors have a direct involvement with each project taking on the day to day control in order to provide the best possible service and the experience of the principals in the construction processes ensures that the objectives of buildability and cost effectiveness are met.

With regard to the Company's association with retro-fit basements, we have been working within this field since 1999 and during that time have had a direct involvement in the design of more than 670 such schemes.

MMP DESIGN DIRECTORS

Steven R. Masters - BSc(Hons).,C.Eng.,M.I.Struct.E.,M.B.Eng.
Philip Seastram - BSc(Hons).
Andrew J. Stone - BSc(Hons).,C.Eng.,M.I.C.E.,M.I.H.T.,Eur.Ing.

EVIDENCE OF COMPETENCE & RESOURCES

Details of Organisation

Name:	MMP Design
Address:	First Floor Unit 6 Union Park Packet Boat Lane Uxbridge UB8 2GH
Contact:	S. R. Masters

Nature of Organisation

Consulting Civil, Structural and Highway Engineers

Incident/Accident Record

None recorded

Membership of Professional Bodies

S. R. Masters - BSc(Hons).,C.Eng.,M.I.Struct.E.,M.B.Eng.
A. J. Stone - BSc(Hons).,C.Eng.,M.I.C.E.,M.I.H.T.,Eur.Ing.

Professional Indemnity/Liability Insurance

PI is in place to cover our duties under CDM with cover limited to £1,000,000 and the liability period limited to 6 years. Details are available upon request.

Details of Persons to be Employed

S. R. Masters & A. J. Stone – Chartered Engineers & Project Leaders
P. Seastram – Project Leader & Designer
S. Barrow – Technician
N. King & R. Shapland – CAD Operators

Familiarity with Construction Processes

The Directors have extensive experience in underpinning and retro-fit basement construction and have been instrumental in the development of some of the working practices adopted by the leading basement constructors.

Awareness of Relevant Health & Safety and Fire Regulations

Within the Company we have documentation relating to these matters which are regularly updated and circulated among the Directors and members of staff.

Health & Safety Practices

A copy of the Company's Health & Safety Policy is available upon request.

Management Systems

A Project Director is responsible for the design and resourcing of the project. Generally projects are undertaken in house with occasional external draughting only where necessary. Communications are by way of verbal and/or written instructions. All work is checked before leaving the office.

Resources

The Company comprises three working Directors together with full time and part time technical assistance sufficient to meet the design requirements for this project.

Technical Facilities to Support the Designer(s)

SCALE Structural Design suite
Staad/QSE Structural Analysis suite
Members of BSI
Members of TRADA
Members of BRE

Method of Communication Design Decisions

Design decisions are communicated verbally and confirmed in writing or by drawing revisions. All drawings are issued to relevant parties as required by the Lead Consultant and/or the Client.

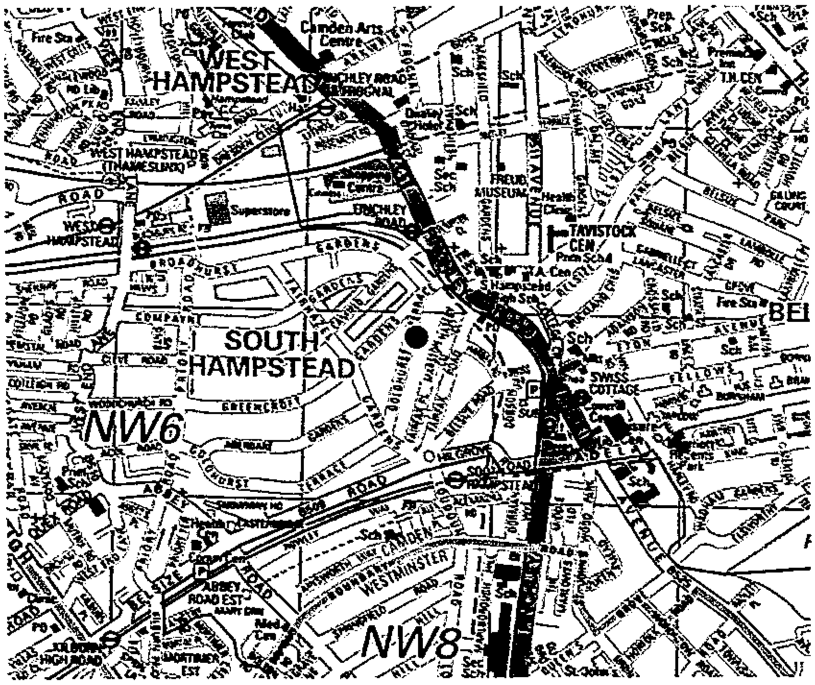
Remaining Risks

Remaining risks will be communicated in writing to the appropriate Authority.

THE SITE AND THE PROPOSED DEVELOPMENT

Goldhurst Terrace is bounded by Finchley Road to the North and Belsize Road to the South and No. 65 occupies a mostly level site sharing party walls with Nos. 63 & 67 which are properties of similar age and design. No. 63 is to the left when viewed from the street.

It is proposed to add a single level basement beneath the full footprint of the existing ground floor and beneath part of the front garden extending to approximately 3.7m below the level of the existing ground floor.



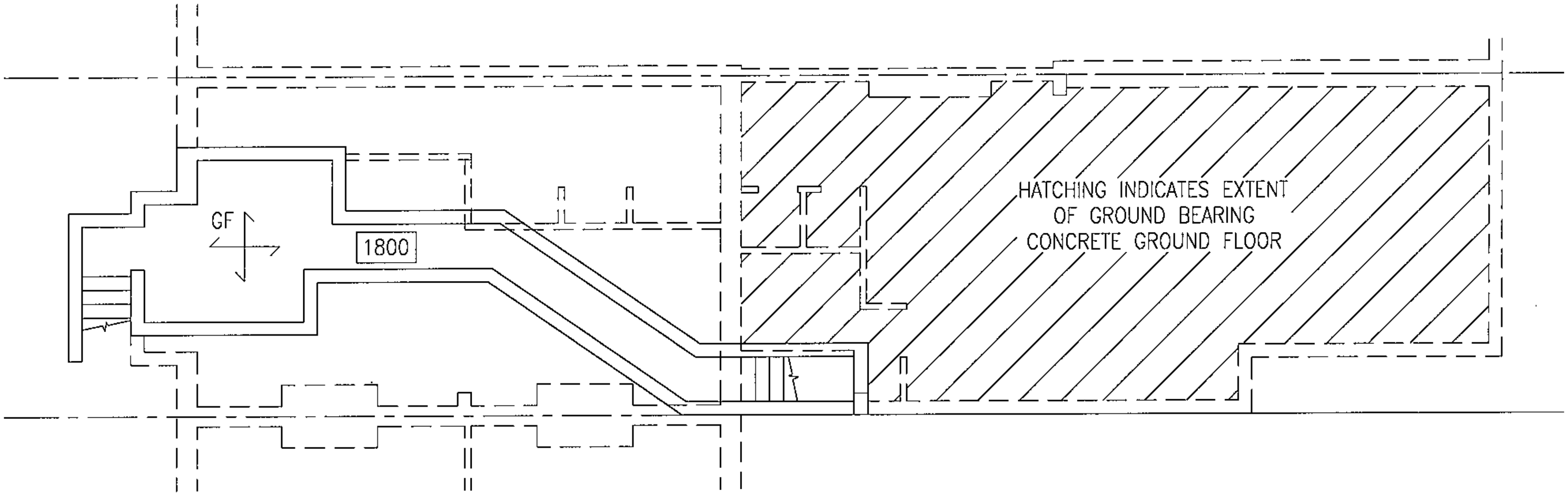
EXISTING STRUCTURE

The existing structure is a late 19th century mid-terrace property originally comprising three storeys at the front beneath a tile covered pitched roof and with a 3 storey annexe to the rear beneath a flat roof. The front roof space has subsequently been converted to form additional habitable space and single storey extensions have been added to the side and rear of the rear annexe. There is a cellar beneath the front section of the property with a clear ceiling height of less 1.8m.

The external and party walls are of solid masonry which likely extend down to a corbelled brick footing; the internal load bearing walls are also of masonry except at the uppermost floor levels where they are of timber studwork.

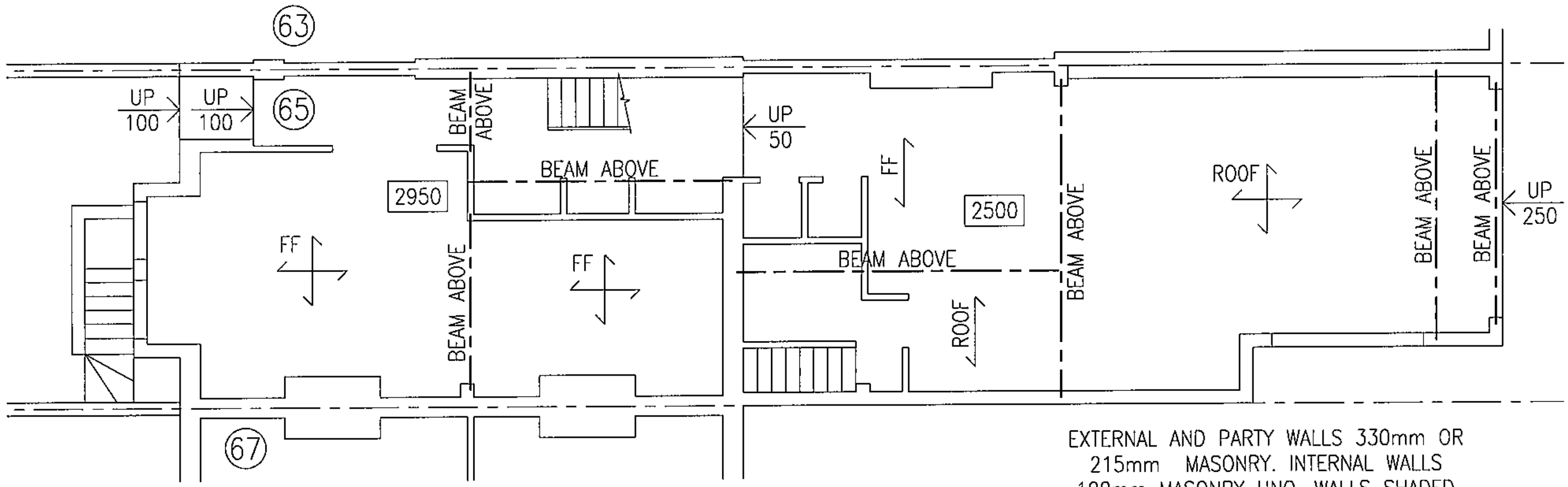
All floors are of suspended timber except for the cellar at the front and the ground floor within the annexe and the extensions which comprise a ground bearing concrete slab.

The property is generally in it's original structural form except where previously described and plans showing the existing structural layout are attached.



EXISTING BASEMENT FLOOR PLAN

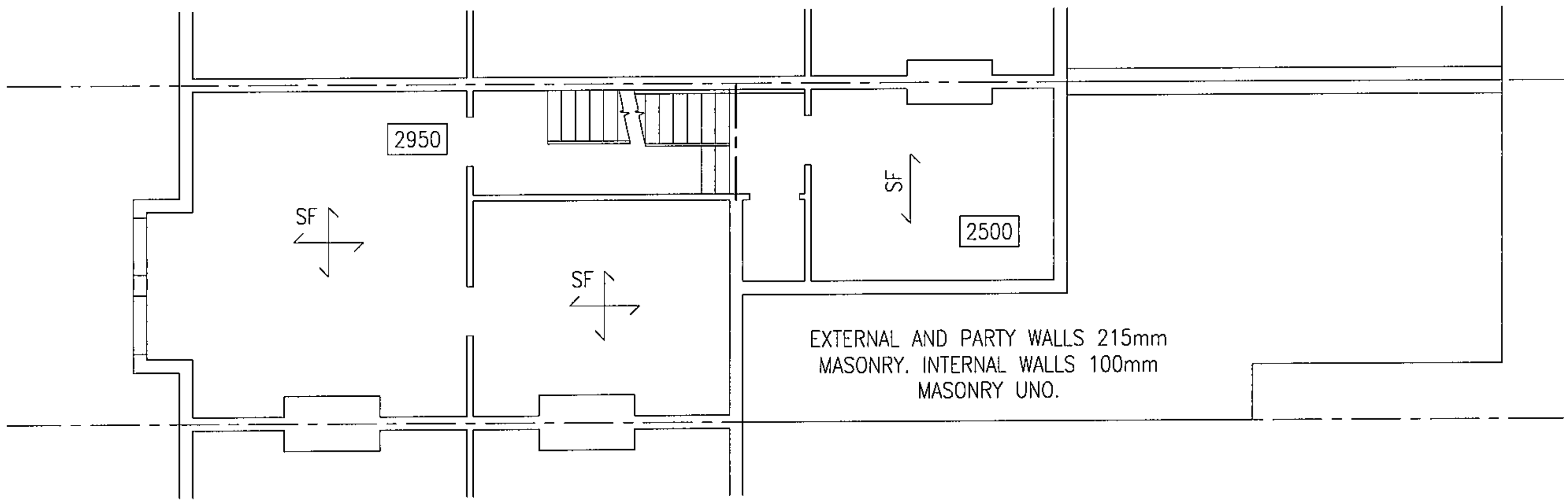
SCALE 1:100



EXTERNAL AND PARTY WALLS 330mm OR 215mm MASONRY. INTERNAL WALLS 100mm MASONRY UNO. WALLS SHADED ARE STUDWORK.

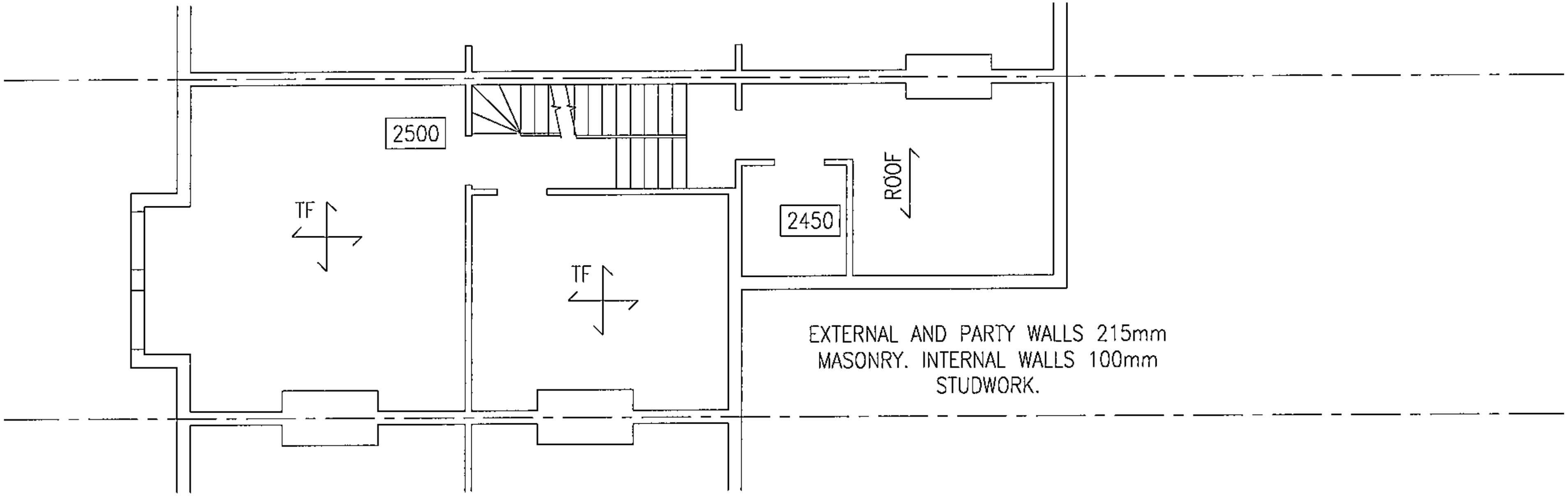
EXISTING GROUND FLOOR PLAN

SCALE 1:100



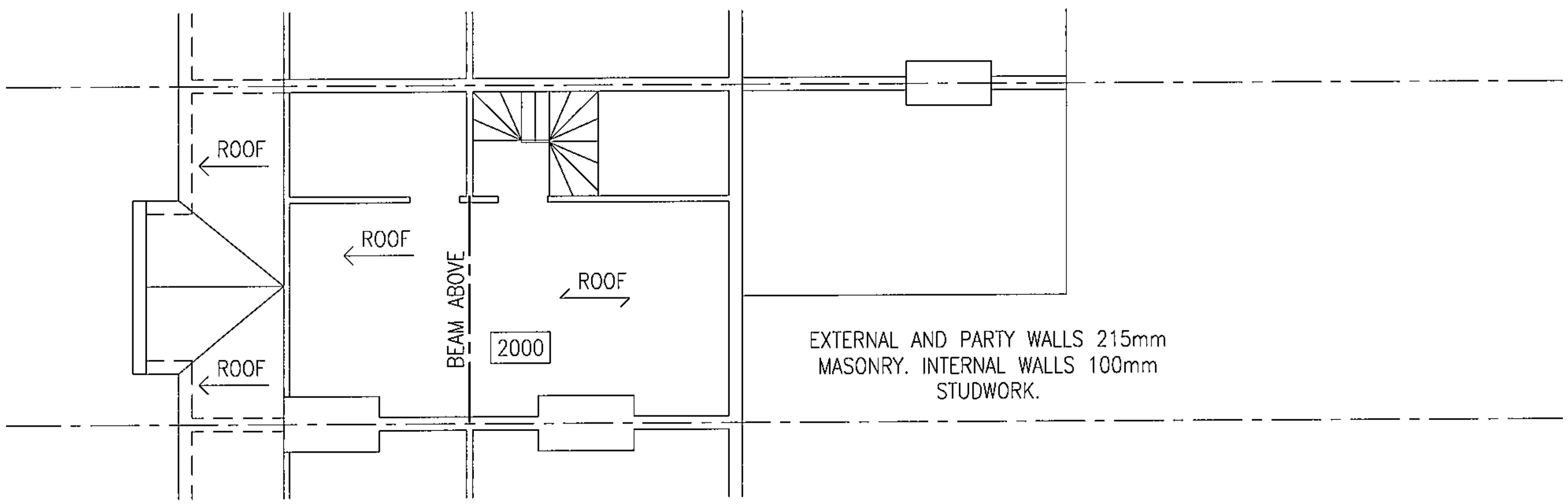
EXISTING FIRST FLOOR PLAN

SCALE 1:100



EXISTING SECOND FLOOR PLAN

SCALE 1:100



EXISTING THIRD FLOOR PLAN

SCALE 1:100

Chelmer Site Investigations

Unit 15, East Hanningfield Industrial Estate
Old Church Road, East Hanningfield, Essex CM3 8AB
Telephone: 01245 400 930 Fax: 01245 400 933
Email: info@siteinvestigations.co.uk Website: www.siteinvestigations.co.uk

Factual Report

Client:	Dig for Victory Ltd
Site:	61 Goldhurst Terrace London NW6
CSI Ref:	FACT/4265
Dated:	12th February 2014

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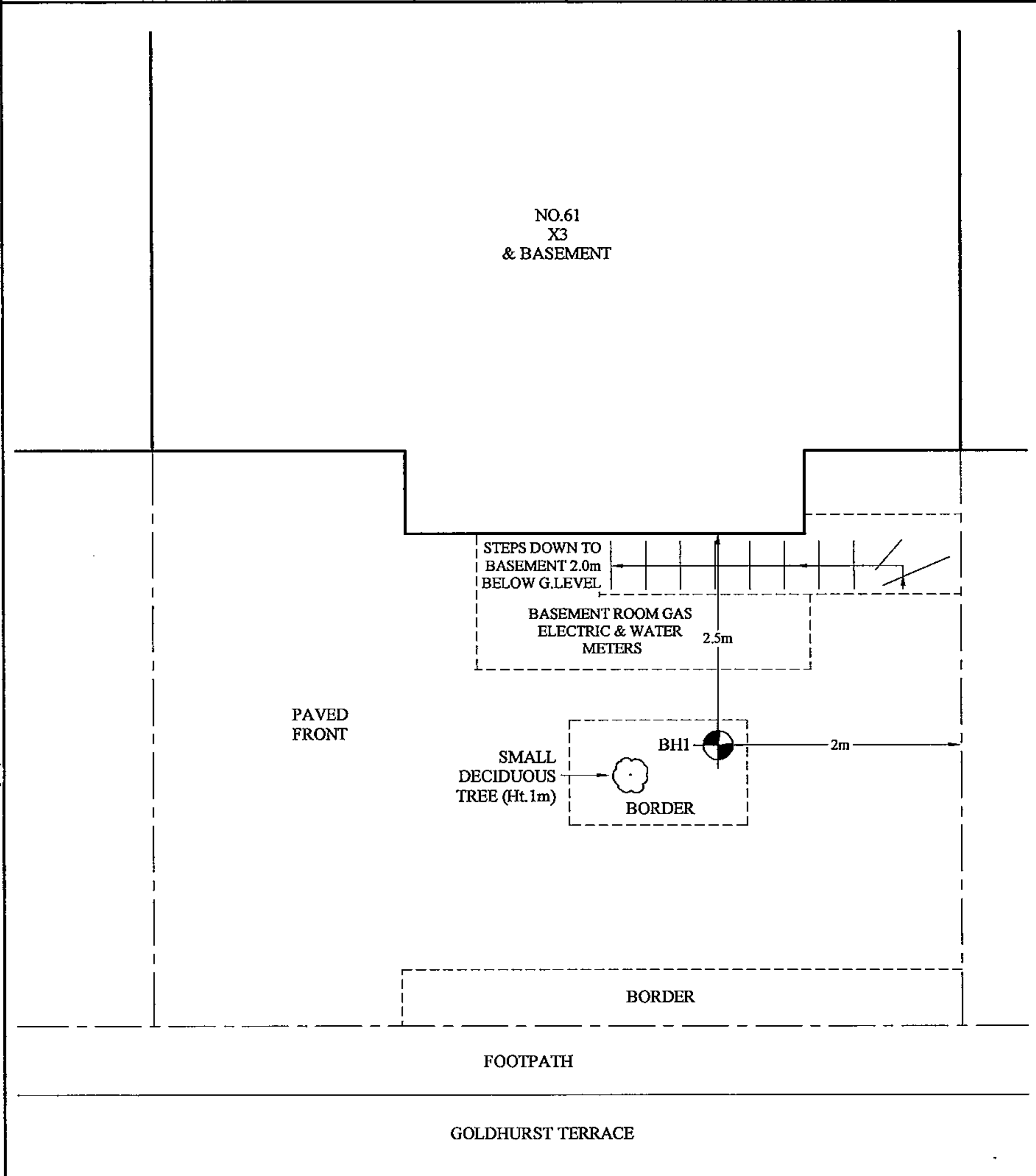
Chelmer Site investigations

Client: Dig for Victory Ltd	**Scale:** N.T.S.	**Sheet:** 1 of 1	**Date:** 12.2.14
Location: 61 Goldhurst Terrace, London NW6	**Job No:** 4265	**Weather:** Overcast	**Drawn by:** JG / **Checked by:** ME



Notes:

On site tree identification for guidance only. Not authenticated.

Key:

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Client:	Dig for Victory Ltd	Scale:	N.T.S.	Sheet No:	1 of 1	Weather:	Overcast	Date:	12.2.14
Site:	61 Goldhurst Terrace, London NW6	Job No:	4265	Borehole No:	1	Boring method:	Hand auger		

Depth Mtrs.	Description of Strata	Thick-ness	Legend	Sample	Test Type	Test Result	Root Information	Depth to Water	Depth Mtrs
G.L. 0.15	TOPSOIL	0.15							
0.9	MADE GROUND: medium compact, dark brown, very silty clay, with gravel and brick fragments.	0.75		D			Hair and fibrous roots to 0.9m.		0.5
	Firm, orange-brown, grey veined, silty CLAY, with partings of orange and brown, silt and fine sand, claystone nodules and selenite crystals.	1.4		D	V	62 66	No roots observed below 0.9m.		1.0
	.....becoming stiff from 1.4m.			D	V	78 82			1.5
2.3				D	V	94 100			2.0
	Very stiff, orange-brown, grey veined, silty CLAY, with partings of orange and brown, silt and fine sand, frequent claystone nodules and selenite crystals.	2.7		D	V	140+ 140+			2.5
				D	V	140+ 140+			3.0
				D	V	140+ 140+			3.5
				D	V	140+ 140+			4.0
				D	V	140+ 140+			4.5
5.0	Borehole ends at 5.0m			D	V	140+ 140+			5.0

Drawn by:	JG	Approved by:	ME

Remarks: Borehole dry and open on completion.

Key: T.D.T.D. Too Dense to Drive
D Small Disturbed Sample J Jar Sample
B Bulk Disturbed Sample V Pilcon Vane (kPa)
U Undisturbed Sample (U100) M Mackintosh Probe
W Water Sample N Standard Penetration Test Blow Count

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REPORT NOTES

Equipment Used

Hand tools, Mechanical Concrete Breaker and Spade, Hand Augers, 100mm/150mm diameter Mechanical Flight Auger Rig, GEO205 Flight Auger Rig, Window Sampling Rig, and Large or Limited Access Shell & Auger Rig upon request and/or access permitting.

On Site Tests

By Pilcon Shear-Vane Tester (Kn/m^2) in clay soils, and/or Mackintosh Probe in granular soils or made ground and/or upon request Continuous Dynamic Probe Testing and Standard Penetration Testing.

Note:

Details reported in trial-pits and boreholes relate to positions investigated only as instructed by the client or engineer on the date shown.

We are therefore unable to accept any responsibility for changes in soil conditions not investigated i.e. variations due to climate, season, vegetation and varying ground water levels.

Full terms and conditions are available upon request.

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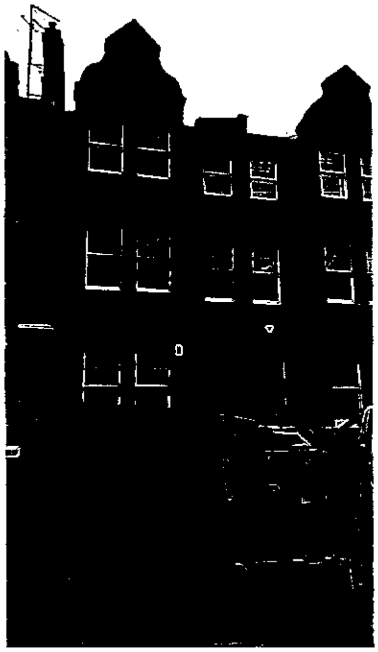

Factual Report

Client:	Dig for Victory Ltd
Site:	63 Goldhurst Terrace London NW6
CSI Ref:	FACT/5126B
Dated:	12th August 2014

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Old Church Road, East Hanningfield, Essex CM3 8AB
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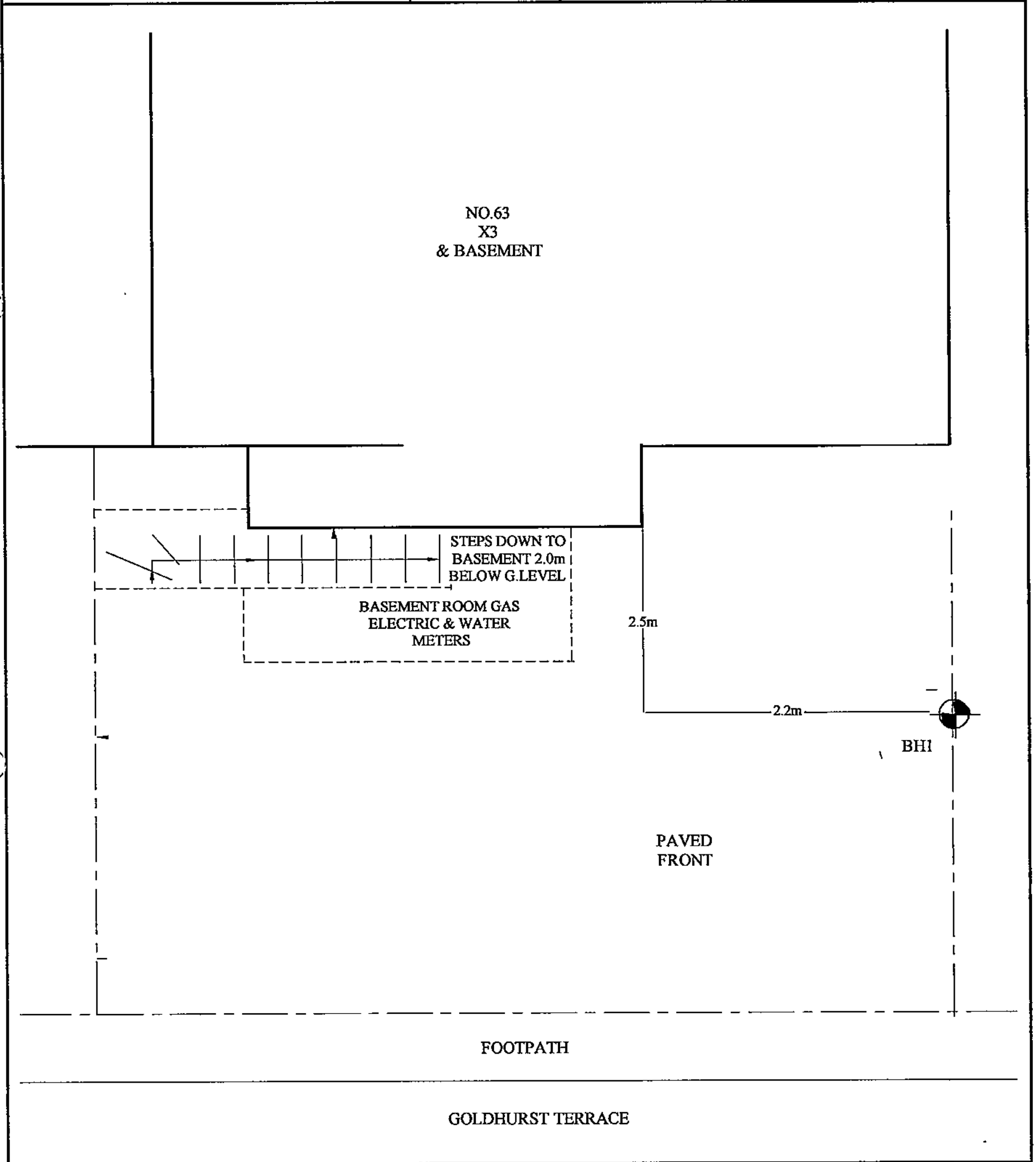
Client: Dig for Victory Ltd	**Scale:** N.T.S.	**Sheet:** 1 of 1	**Date:** 12.8.14	
Location: 63 Goldhurst Terrace, London NW6	**Job No:** 5126B	**Weather:** Overcast	**Drawn by:** JG	**Checked by:** ME



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Client: Dig for Victory Ltd	**Scale:** N.T.S.	**Sheet No:** 1 of 1	**Weather:** Overcast	**Date:** 12.8.14	
Site: 63 Goldhurst Terrace, London NW6	**Job No:** 5126B	**Borehole No:** 1	**Boring method:** Hand auger		

Depth Mtrs.	Description of Strata	Thickness	Legend	Sample	Test Type	Test Result	Root Information	Depth to Water	Depth Mtrs
G.L. 0.15	BRICK PAVING	0.15							
0.9	MADE GROUND: medium compact, dark brown, very silty clay, with gravel and brick fragments.	0.75		D			Hair and fibrous roots to 0.9m.		0.5
	Firm, orange-brown, grey veined, silty CLAY, with partings of orange and brown, silt and fine sand, claystone nodules and selenite crystals. .....becoming stiff from 1.4m.	1.4		D	V	62 66	No roots observed below 0.9m.		1.0
				D	V	78 82			1.5
2.3				D	V	94 100			2.0
	Very stiff, orange-brown, grey veined, silty CLAY, with partings of orange and brown, silt and fine sand, frequent claystone nodules and selenite crystals.	2.7		D	V	140+ 140+			2.5
				D	V	140+ 140+			3.0
				D	V	140+ 140+			3.5
				D	V	140+ 140+			4.0
				D	V	140+ 140+			4.5
5.0	Borehole ends at 5.0m			D	V	140+ 140+			5.0

Drawn by: JG **Approved by:** ME

Remarks: Borehole dry and open on completion.

Key: T.D.T.D. Too Dense to Drive
D Small Disturbed Sample J Jar Sample
B Bulk Disturbed Sample V Pilcon Vane (kPa)
U Undisturbed Sample (U100) M Mackintosh Probe
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Note:

Details reported in trial-pits and boreholes relate to positions investigated only as instructed by the client or engineer on the date shown.

We are therefore unable to accept any responsibility for changes in soil conditions not investigated i.e. variations due to climate, season, vegetation and varying ground water levels.

Full terms and conditions are available upon request.

SOIL CONDITIONS & FOUNDATIONS

Site investigations have been carried out at No. 61 Golhurst Terrace on 12th February 2014 and at No. 63 on 12th August 2014, both by Chelmer Site Investigations and their reports reference 4265 and 5126B respectively are attached. These reports both confirm the presence of stiff silty London Clay becoming very stiff with depth and no significant water presence.

In the absence of any laboratory testing we have looked to BS.8002, BS.8004 and the Reinforced Concrete Designers Handbook (by Charles E. Reynolds and James C. Steedman) for a suggested range of parameters to be adopted for the design. For the soil profile previously described the guidance suggests an Angle of Internal Friction of 20-40° and an allowable Net Bearing Pressure (with no addition for depth of embedment) of 75-150 kN/m^2.

Hence the following parameters will be adopted.

> $\phi = 30°$ (so Ka = 0.333) and $\delta = 18\ kN/m^3$
> Allowable bearing stress at GL = 75 kN/m^2
> Allowable bearing at Basement Level = 75 + soil removed, say = 125 kN/m^2

These parameters have been confirmed by previous testing regimes carried out over a period of almost 15 years and are accepted by the checking authorities of at least 13 London Boroughs. They represent the long term condition which when combined with the design being based on active earth pressures results in a much simplified but rather conservative approach.

It should be noted that the nature of the construction of a basement ensures that the front lightwell excavation is formed first in order to gain access to the working area; in effect a substantial and full depth trial pit is formed before any foundation works are commenced. Should the conditions encountered vary in any way from those described above then the design will be re-visited before any underpinning works are commenced.

WATER

As previously described, the soil type anticipated at this site is London Clay and no significant water presence is anticipated. The Clay has a relatively low permeability to water and in essence presents an almost complete barrier but there can be some permeation albeit extremely slowly and there is also the possibility of some faster flow through fissures or localised zones of more granular material which could cause an occasional build up against the new basement wall. It is for these reasons that water will be assumed with the level being 0.75 x the retained depth or at 1m below GL, whichever is the worst condition.

HEAVE & SETTLEMENT

The underpinning process involves transferring the foundation loads to a lower level and inevitably this leads to some settlement. Some movement will also be caused by the sequential transfer of load between different parts of the structure but the careful control of the underpinning process and sequence will keep such movements to a practicable minimum. Particular care will be taken in the vicinity of the more vulnerable parts of the existing fabric.

The depth to the London Clay and the modest dimensions of the site are such that the heave of the Clay is unlikely to exceed a few millimetres or to have any discernible effect outside the site boundaries. Any movement that does occur will be further mitigated by the necessarily slow rate of the excavation and construction.

At the lower level of the basement floor slab will be used to resist these heave forces and by supporting the slab with the deeper underpinning and the internal column foundations, the resulting upward movement is used to counteract the increased settlements expected due to the increased dig depth.

SLOPE INSTABILITY

The ground is essentially level and therefore we confirm slope instability will not be initiated due to these works.

IMPACT ON DRAINAGE AND SURFACE WATER

We understand that there is no statutory drainage within the area of influence of the proposed basement works and with regard to surface water, the majority of the proposed basement is below the existing extension and concrete paving. We do not foresee significant impact on the surface water courses.

It is commonly accepted that increasing the size of an existing cellar as we are proposing has little or no effect on the flow of local water in relation to adjoining properties. In fact even if mobile water was forced to find an alternative route as a consequence of the basement construction, any increase in the level of that water is likely to be significantly less than the natural variations associated with seasonal changes and rises in levels from extreme rainfall events. We concur with these views.

EFFECTS ON ADJACENT STRUCTURES

Outside of the basement area the change of vertical stresses in the ground may result in limited upward movements but the underpinning of the party walls may also cause some very minor settlements and horizontal movements towards the new basement.

In addition the underpinning operations may cause localised settlements of the party walls only which might result in cracks forming at the junctions of the walls of the adjacent properties where they abut the party walls. It should be stressed however that any anticipated movements are expected to be minimal as they are generally suppressed by the stiffness of the structures above and those adjoining.

It is our experience that the potential for damage will be limited to the party walls but this can be mitigated by appointing a suitably experience Contractor familiar with propping techniques and sequential operations and by the Designer giving the necessary consideration to the risk by specifying measures to ensure that significant damage is avoided. This would typically be in the form of transitional underpins where we consider the structure above to be particularly vulnerable but otherwise by ensuring that the foundation transitions occur at inherently strong intersections of the more robust load bearing walls.

As a result we anticipate that should any damage occur it will be classified as Category 0 in the Category of Damage Chart, CIRIA C580. Category 0 is Negligible; hairline cracks of less than 0.1mm.

However, there will always be some movement as it can never be completely avoided and there are occasions where unforeseen conditions beneath the property which were not or could not be detected by the pre-construction investigations will result in more extensive damage. From our experience of designing almost 600 retro-fit basement the chance of such an occurrence is less than 2% and even then the damage would be classified as Category 1 in the Category of Damage Chart. Category 1 is Very Slight, fine cracks less than 1mm that can be easily treated during normal decoration.

DESIGN PRINCIPLES

Ground Floor Structure

Where the existing internal below ground floor level load bearing structure is to be removed, replacement will be by the use of steel and/or timber beams supported by the existing load bearing walls or new load bearing brick piers and/or steel posts.

To ensure the continued stability of the structure without reliance from the adjoining properties, the existing and any new load bearing basement walls are strapped to the structural ground floor deck using 30mm x 5mm galvanised mild steel straps placed at 2m centres.

New beams are not considered 'restrained' unless there is a mechanical connection to the top flange (or within 75mm of it). Hence timber floor joists do not restrain the compression flange unless they are notched into the web or nailed/screwed to a timber flange plate.

In order to restrict any possible damage to the existing structure, the deflection in the new beams is restricted to 1/360th of the overall span, under the total characteristic load condition.

Timber

The exact structural layout of any existing ground floor joists is often unknown although sometimes the general direction of the span of the joists is. There will almost certainly be a foundation under each load bearing and/or masonry ground floor level wall; it also likely that there are numerous sleeper walls supporting nominal floor joists and experience would suggest that these are likely to be only 50mm x 100mm joists spaced at little more than 400mm centres. The spacing of the sleeper walls is also likely to be little more than 2.0m.

The new ground floor support structure will therefore need to replicate this arrangement. However, since the exact location of the sleeper walls is unknown, the main beam layout will be created first with a beam provided under each load bearing and/or masonry wall. It will then be necessary to provide additional beams to replace each sleeper wall. Hence sleeper wall beams will be designed to span up to various lengths and support at least 2.0m width of floor and ceiling. All main beams will then be designed assuming the worst ground floor loading case.

For DL of (2 x 0.6)+0.5 = 1.70 kN/m and IL of (2 x 1.5) = 3.00 kN/m,

Provide 152x152 UC.23 for spans up to 4.5m,
152x152 UC.30 for spans up to 5.0m,

Concrete

The exact structural detail of any existing concrete ground bearing ground floor is also unknown although the thickness has been assumed as 200mm (plus 50mm finishes) and the non load bearing masonry walls will likely have been built off the slab.

In such cases it will necessary to provide beams to support the slab; these will be spaced at approximately 600mm centres hence several floor support beams will be designed to span up to various lengths and support at least 0.6m width of floor and new ceiling. All main beams will then be designed assuming the worst ground floor loading case.

For DL of (0.60 x 6.00)+0.50 = 4.10 kN/m and IL of (0.60 x 1.50) = 0.90 kN/m,

Provide 152x152 UC.23 for spans up to 4.0m
152x152 UC.30 for spans up to 4.5m

Basement

The remaining load bearing structure will be underpinned in a traditional 'hit and miss' method to achieve the increased headroom required. The underpins comprise a vertical stem which is immediately beneath the existing wall and a base which usually has a toe and a nominal heel. The heel size is determined by ignoring the earth pressure and considering the maximum vertical load on the wall only, using this to find a minimum foundation width based on the soil bearing capacity.

The toe of the base is then determined by considering the minimum vertical dead load on the wall along with the maximum pressure from the retained soil and with the wall assumed to be acting as a cantilever. In calculating the toe size, the maximum allowable bearing pressure is not exceeded and a minimum factor of safety against overturning of 2.5 is achieved.

The toe and/or stem will only be reinforced when the underpin stem is subjected to tensile stresses due to the pressures from the retained material. This usually only occurs where the London Clays are present or where the retained depth of soil is large.

To check the stresses in the underpin stem, the overturning moment taken about the basement slab is used. However, the design of the toe and the overall stability is based on the overturning moment taken about the underside of the underpin base.

We assume the soil/stem interface to be friction free as ultimately this provides the most onerous design.

Lightwell

These are invariably formed within the front garden of the property and are therefore adjacent to the public highway. Consequently surcharge loads are considered and are taken as either of the following, whichever produces the more onerous design conditions.

a... a uniformly distributed load of 2.5 kN/m^2, applied from within the garden and assuming private vehicle parking is possible,

b... a uniformly distributed load of 10 kN/m^2, applied from the highway and/or footpath,

c... a point load of 40 kN (a typical wheel load), applied over an area 0.3m x 0.3m and assumed to act at a point 0.6m from the property boundary, out toward the highway.

The lightwell walls comprise a vertical stem and a base with a toe and occasionally a heel. The reinforced concrete wall provides all of the necessary resistance to the applied overturning forces and is cast against the soil. The size of the base toe is determined by considering only the self-weight of the wall along with the maximum pressure from the retained soil and any surcharge. In calculating the toe size, the maximum allowable bearing pressure is not exceeded and a minimum factor of safety against overturning of 1.5 is achieved. Since the base is usually cast up against the front wall of the basement, the design of the toe and the overall stability is based on the overturning moment taken about the top of the wall base.

Ground Water

No ground water is anticipated during the construction period. If any is found it will be locally removed from the excavations by local pumping from the excavated area to a sump area.

Water and moisture will generally be excluded from the permanent structure by the reinforced concrete walls/slab and the provision of an internal drained cavity system on the inside face of the walls/slab. Any water from the cavity system will drain to a sump in the external lightwell and be pumped into the house surface water drainage system. The concrete walls/slab will prevent the migration of large quantities of water or soil particles and therefore the drained cavity will only need to deal with a limited quantity of ground water.

The upward water pressure on the basement will be resisted by the reinforced concrete basement slab tied into the concrete underpinning to the walls. There is sufficient weight in the loading to the underpinned walls and the basement structure to resist any 'floatation' effects.

DESIGN CRITERIA

General

The detailed structural design of the proposed works will be carried out in accordance with current British Standards, Building Regulations and appropriate Guidance Documents published by CIRIA, ICE, IStructE etc. The design and drawings will be submitted to the local Building Control for approval and the construction inspected by the Building Inspector on site.

Existing Brickwork

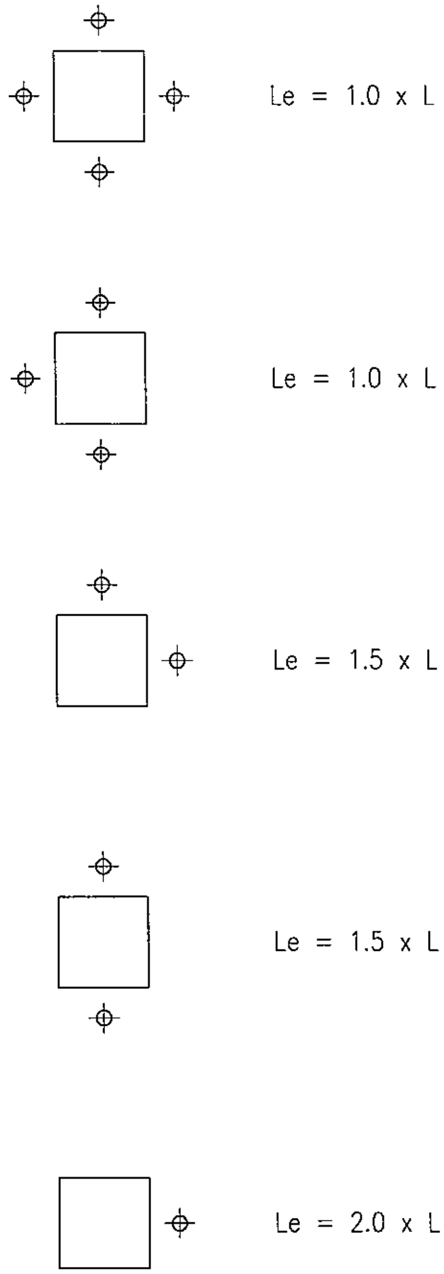
Assuming 7N bricks in lime mortar, from CP.111 the basic compressive strength = 0.49 N/mm^2
Hence under a concentrated load, bearing strength = 1.5 x 0.49, say 0.7 N/mm^2

Typical Underpinning Sequence

6	1	4	7	2	5	8	3	6	1	4	7

Materials

Concrete is grade C35 N/mm^2 using Sulphate Resisting cement unless otherwise directed.
Reinforcement is grade 500 N/mm^2
Mortar is Class (iii).



EFFECTIVE LENGTH OF BASEMENT POSTS

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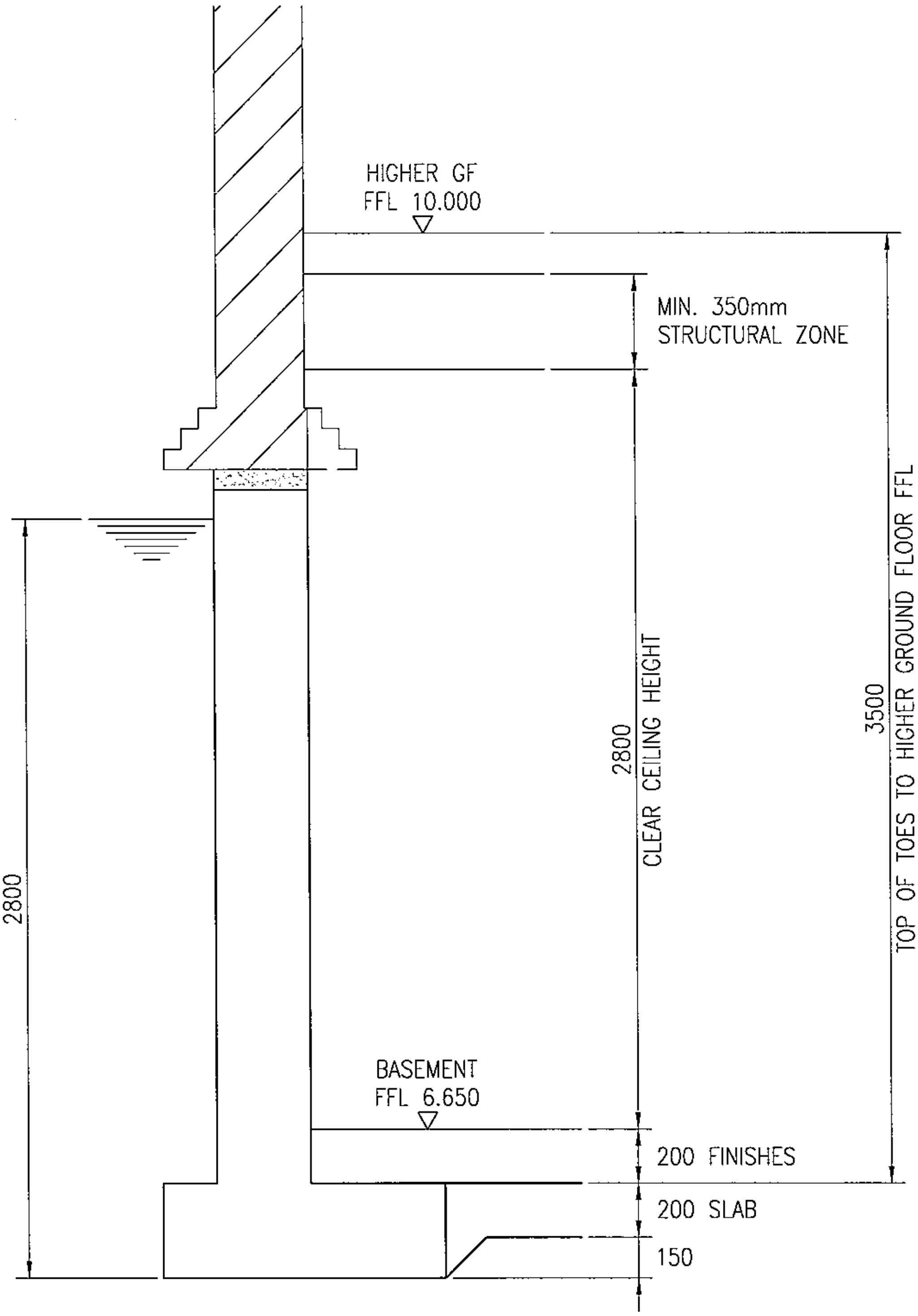
Project: 65 GOLDHURST TERRACE, NW6		Date: OCT/14	Drawn by: AFB
		Scales: 1:25	Checked:
Title: COLUMN EFFECTIVE LENGTHS		Job No. 4402	Drg. No. SK1 / Rev.



TYPICAL FRONT UNDERPINNING DETAIL

SCALE 1:25

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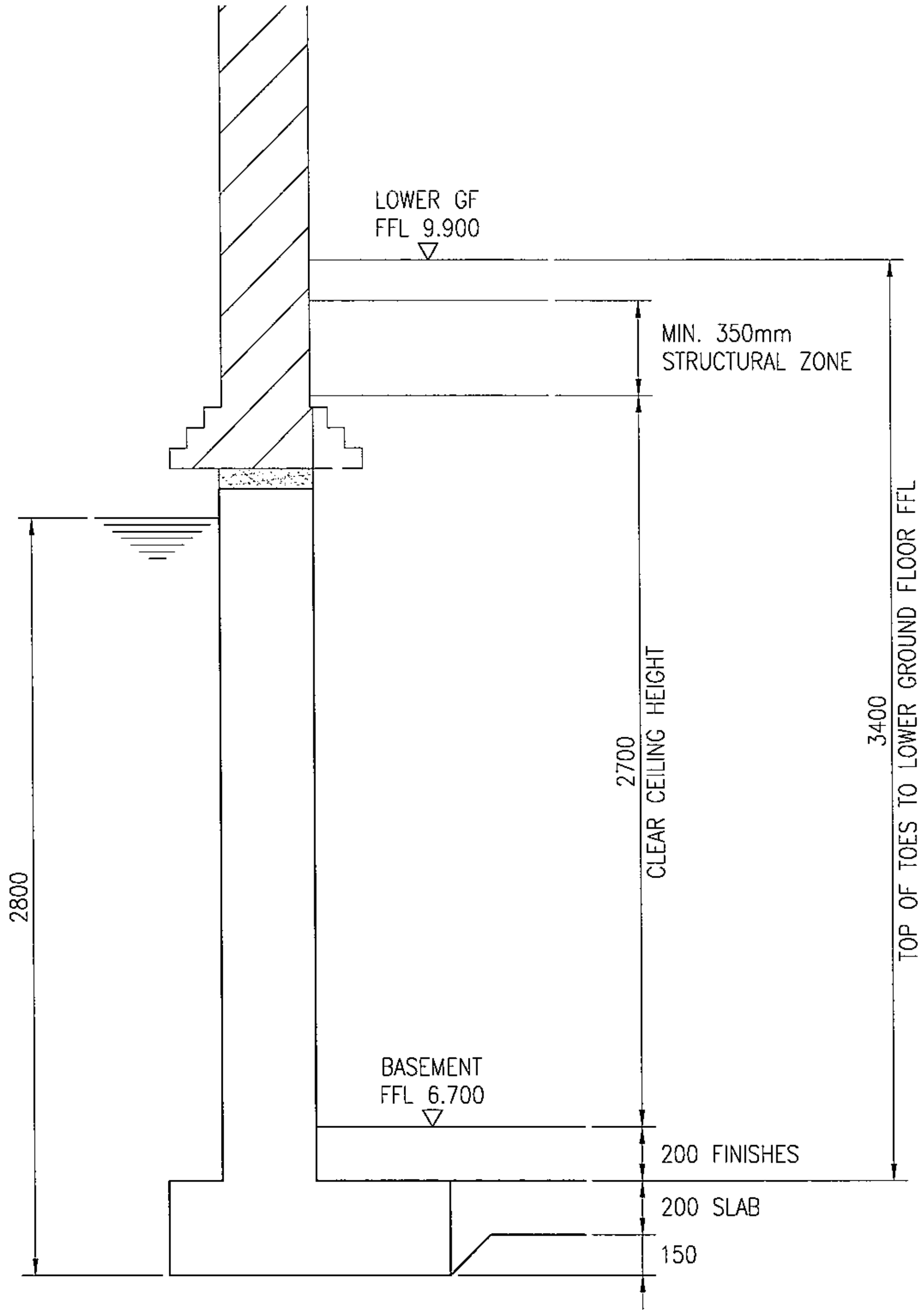
Drg. No.	SK2
Rev.	



TYPICAL REAR UNDERPINNING DETAIL

SCALE 1:25

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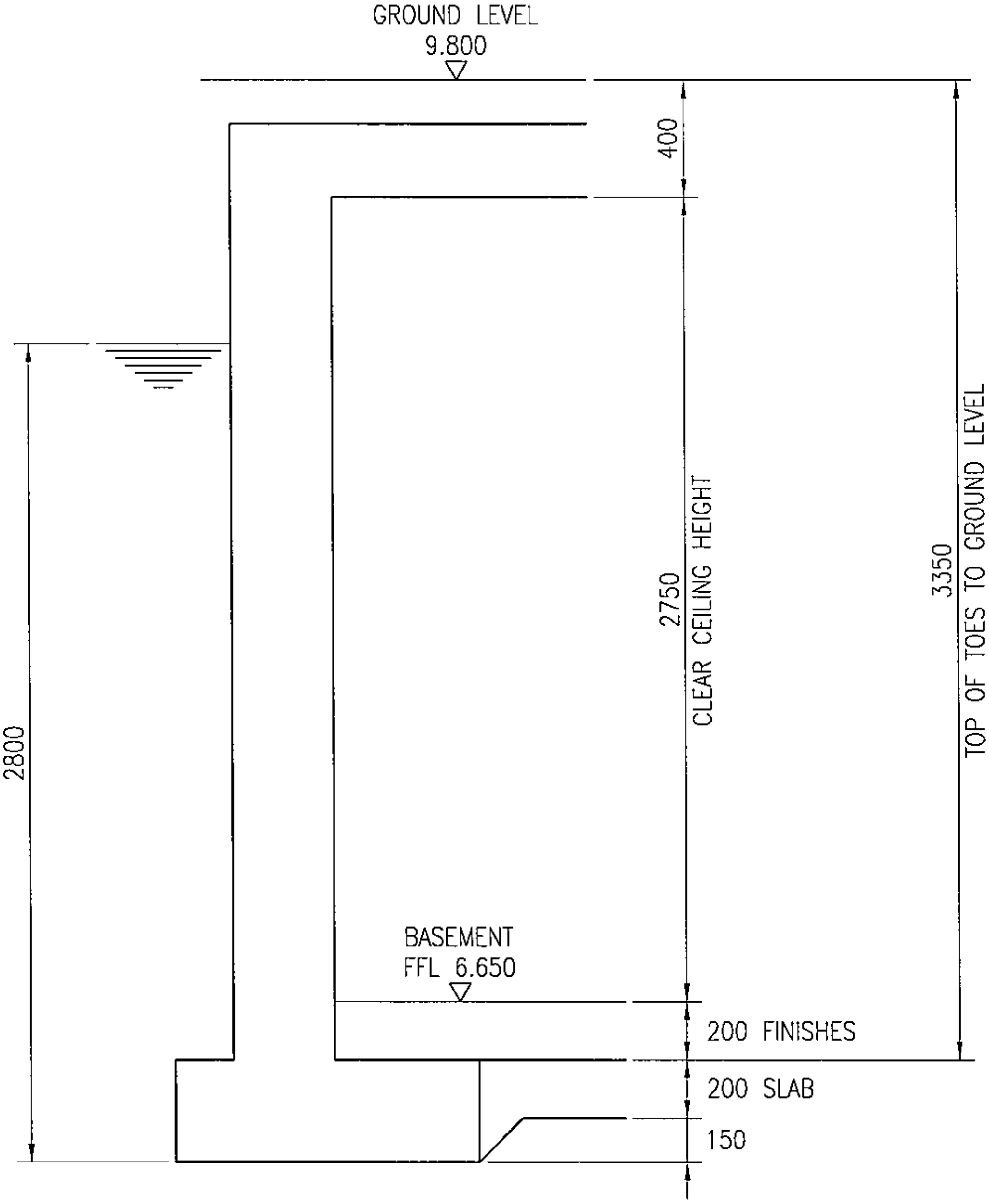
Title: TYPICAL UNDERPIN SECTION	Job No. 4402 — Drg. No. SK3 — Rev.



TYPICAL BELOW GARDEN WALL DETAIL
SCALE 1:25

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Date:	OCT/14
Drawn by:	AFB
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Job No.	4402
Drg. No.	SK4
Rev.	

DESIGNERS RISK ASSESSMENT

Excavations

Care must be taken to prevent sides of excavations from collapsing.

Suspended Floors

The use of suspended insitu reinforced concrete ground slabs is expensive and impractical due to the extent of formwork required and the thickness of slab required.

Precast beam and block floors provide reduced weight and quick installation with holes and cutting for designed services carried out on site at the time of installation. However, during installation, and indeed before the floor is screeded, safety netting or air bags shall be provided to prevent injury due to operatives falling between the joists.

In-situ concrete slabs cast onto a profiled steel permanent shuttering provides a suitable alternative to the beam and block and removes the need for the netting or air bags. However, the manufacturer should always be consulted about temporary span propping that may be required prior to the concrete achieving it's design strength.

Masonry Walls

A 150mm minimum thickness is required for design load resistance and height to thickness ratios. However the blocks tend to be too heavy to manhandle and so load bearing blockwork walls will be specified as 215mm thick and formed from 100mm thick blocks laid on their side.

Steel Beams

Where possible, large span beams will be spliced to minimise manhandling. Other ways of minimising the weight of steel sections is to specify two channels bolted back to back in lieu of a single UB or UC section. However, there will be occasions where neither option will be practical and/or possible and the Contractor will be made aware of such situations.

Hazards & Risks Which Cannot be Designed Out

Potential Hazards	Action Required	Risk Assessment
Falls from Height	Works being carried out - provide hand rails and access scaffolding to all openings.	Medium
Falling Debris	Works carried out above public access - provide toe boards, netting and protection fans.	High
Materials Storage	Existing roofs and floors are not to be used for storage of materials without reference to the Engineer or for supporting access scaffolding.	High

Potential Hazards	Action Required	Risk Assessment
Lifting of Steelwork	Steel sections to be lifted using mechanical means where unable to be manually lifted.	High
Erection of Steelwork	Contractor responsible for providing method statement for erection procedure, including any temporary bracing.	Medium
Lifting of Timber	Timber rafters and joists to be lifted using mechanical means where unable to be manually lifted.	High
Fixing of Timber	Timbers to be fixed in accordance with good building practice.	Medium
Reinstate Existing Roof Finishes	Method statement to allow for temporary waterproofing if required.	Low
Use of Cutting Equipment – Flame or Disc.	Fire risk - use suitable protective methods – remove inflammable materials.	High
Painting	Touch up steelwork with primer – take precautions against vapour inhalation, eye and skin contact and fire. Wear protective clothing.	Low
Excavation	Take precaution against collapse of excavation and hazards of persons falling in.	High
Precast Concrete units	Lift into position using mechanical assistance. Storage at ground level in a safe manner.	Medium
Insitu Concrete Construction	Take precautions to prevent skin/eye contact. Protect public and site staff from falling objects and spillage. Ensure adequate care when fixing reinforcement.	Medium

Potential Hazards	Action Required	Risk Assessment
Formwork/Falsework	Design temporary works in a manner that makes allowances for all loadings, including accidental loads. Ensure adequate vertical and diagonal bracing. Supports not to be removed until period specified.	Medium
Forming new Openings in Walls	Provide temporary works to support wall and loads above opening. Install new support lintel and reinstate prior to removal of temporary supports.	Medium

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CALCULATION SHEET			
Project	65 GOLDHURST TERRACE, NW6	Job No.	4402
Title	BASEMENT	Date	OCT/14
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Sheet No.	LD/1	Rev	

UNIT LOADS in kN/m^2	DEAD	IMPOSED
Pitched Roofs		
Pitched roof with tiles and battens over felt, unlined but including ceiling below	1.35	0.90
Pitched roof with tiles and battens over felt, unlined but excluding ceiling below	1.05	0.65
Pitched roof with tiles and battens over felt, lined and including ceiling below	1.50	0.90
Pitched roof with tiles and battens over felt, lined but excluding ceiling below	**1.20**	**0.65**
Flat Roofs		
Flat roof of three layer felt, access for maintenance only	0.75	0.75
Flat roof of lead, access for maintenance only	**1.00**	**0.75**
Flat roof of three layer felt and full access	0.75	1.50
Flat roof of lead and full access	**1.00**	**1.50**
Suspended floors		
Timber upper floor including ceiling	**0.50**	**1.50**
Timber ground floor including services and suspended ceiling	**1.00**	**1.50**
200mm Concrete in-situ ground floor including services and suspended ceiling	**6.00**	**1.50**
150mm Concrete in-situ ground floor including services and suspended ceiling	4.80	1.50
Concrete precast ground floor including services and suspended ceiling	5.00	1.50
Allowance for lightweight partitions if position not known	0.00	1.00
External walls		
215 mm solid masonry, plastered one side	**4.80**	**0.00**
330 mm solid masonry, plastered one side	**7.20**	**0.00**
440 mm solid masonry, plastered one side	9.50	0.00
Timber studwork, tile hung with plasterboard and skim internally	1.00	0.00
250 mm cavity masonry, plastered one side	4.80	0.00
Internal walls		
100 mm solid masonry, plastered both sides	**2.60**	**0.00**
215 mm solid masonry, plastered both sides	5.00	0.00
330 mm solid masonry, plastered both sides	7.50	0.00
100 mm timber studwork, lathe and plaster both sides	**0.60**	**0.00**
100 mm timber studwork, plasterboard and skim both sides	**0.60**	**0.00**

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CALCULATION SHEET	Project: 65 GOLDHURST TERRACE, NW6	Job No.: 4402	
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<u>NEW RC SLAB - SPAN 2450</u>

SLAB	6.00	
FINISHES	4.00	
CEILING	0.25	
SERVICES	0.25	
IMPOSED		5.00
	10.50	5.00

TOTAL ULT. LOAD = 22.70

∴ MOMENT = 17.03 kN.m/m
SHEAR = 27.81 kN/m.

FROM PAGES CS/2+3, PROV. 200 SLAB WITH H10-200 BOTH WAYS, TOP+BOTTOM.

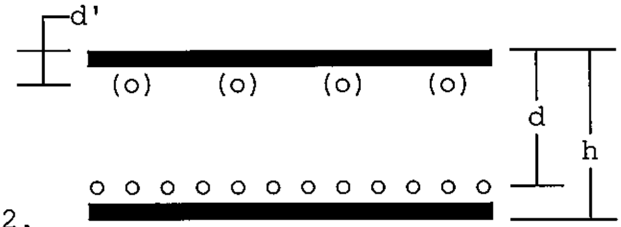
FROM PAGE CS/4, NO ADDITIONAL SHEAR REINF'T REQ'D.

Office: 5831

Location: NEW RC SLAB

Bending in solid slabs (with comp.steel if reqd.), designed per metre width, with checks on minimum steel and span/effective-depth ratio

Calculations are based on formulae in Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to d/2.



Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Moment before redistribution Mbef=17.03 kNm per metre width
Slab containing section being analysed is considered as non-continuous.

Characteristic concrete strength	fcu=35 N/mm^2
Characteristic steel strength	fy=500 N/mm^2
Longitudinal reinforcement is high-yield steel.	
Diameter of tension bars	dia=10 mm
Nominal concrete cover	cover=40 mm
Overall thickness of slab	h=200 mm
Effective depth of section	d=155 mm
Area of tension steel required	As=M*10^6/(z*fy/gammaS) =266 mm2/metre width.
Chosen spacing of tension bars	pch=200 mm
Diameter of distribution bars	diamn=10 mm
Spacing of distribution bars	pchDA=200 mm

TENSION REINFORCEMENT SUMMARY	Characteristic strength	500 N/mm^2
	Diameter of bars	10 mm
	Spacing of bars	200 mm
	Effective depth	155 mm
	Area of steel required	266 mm2/m
	Area of steel provided	392 mm2/m
	Percentage provided	0.196 %
	Weight of steel provided	3.08 kg/m^2

DISTRIBUTION REINFORCEMENT SUMMARY	Characteristic strength	500 N/mm^2
	Diameter of bars	10 mm
	Spacing of bars	200 mm
	Depth to bar centres	145 mm
	Area of steel required	260 mm2/m
	Area of steel provided	392 mm2/m
	Percentage provided	0.196 %
	Weight of steel provided	3.08 kg/m^2

Check on span/effective-depth ratio

Basic ratio for simp.-sup.slab	bs'd=20 (see Table 3.9)
Mod.factor for tension steel	modf1=1.8491
Diameter of compression bars	diac=10 mm
Spacing of comp.bars provided	pchCA=200
Compression steel provided	As'pr=1000/pchCA*PI*diac^2/4 =392.7 mm^2 per m
Percentage of compression steel	per'=100*As'pr/(1000*d)=0.25335 %

From Equation 9 of BS8110, with percentage of comp.steel=0.25335 %,

Mod.factor for compression steel	modf2=1+per'/(3+per')=1.0779
Maximum permissible span/effective-depth ratio	ps'd=bs'd*modf1*modf2=39.862
Effective span of slab	span=2.45 m
True span/effective-depth ratio	as'd=1000*span/d=15.806
As this does not exceed	39.862, this is Acceptable.

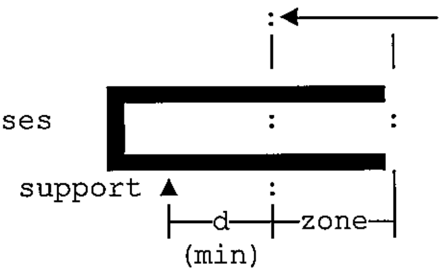
No 81

Location: NEW RC SLAB

Shear in longitudinal zone of solid slab of given width

Since distance from support varies throughout zone considered, enhancement of shear resistance due to proximity to support is not applicable.

The calculations for shear in the zone of solid slab are in accordance with Clauses 3.5.5 of BS8110 throughout.



In the case of a uniform load, the design section to be considered need not be less than a minimum distance from support equal to effective depth.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Effective depth	d=155 mm
Breadth of slab zone	b=1000 mm
Characteristic concrete strength	fcu=35 N/mm^2
Diameter of tension bars	dia=10 mm
Number of tension bars	nbars=5
Shear force due to ult.loads	V=27.81 kN

Area of longitudinal steel As=nbars*PI*dia^2/4=5*3.1416*10^2/4
=392.7 mm^2

Equivalent percentage of steel per=100*As/(b*d)=100*392.7/(1000*155)
=0.25335 %

Design shear stress (equation 21) v=V*1000/(b*d)=27.81*1000/(1000*155)
=0.17942 N/mm^2

Assuming that no link reinforcement at all is to be provided.
As eff.depth does not exceed 1985 mm, depth-factor f00d=(400/d)^.25
=(400/155)^.25
=1.2675

Then from formula in Note 2 below Table 3.8, with pcnt=100*As/(b*d),
Design shear stress in concrete vc=0.79*pcnt^(1/3)*f00d/1.25
=0.79*0.25335^(1/3)*1.2675/1.25
=0.50686 N/mm^2

As characteristic concrete strength exceeds 25 N/mm^2
therefore increase vc according to footnote in Table 3.8.
Modified design shear stress vc=vc*(fcu/25)^(1/3)
=0.50686*(35/25)^(1/3)
=0.56702 N/mm^2

As vc=0.56702 N/mm^2 exceeds v=0.17942 N/mm^2 no shear links are needed.

No 83

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Project	65 GOLDHURST TERRACE, NW6	Job No.	4102
CALCULATION SHEET	Title: BASEMENT	Date	OCT/14
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GROUND FLOOR LEVEL BEAMS -

BEAM B1 - SPAN 1000

		DL	IL
SLAB	0.70 x 10.5 =	7.35	
GRILL	1.00 x 0.50 =	0.50	
IL	1.70 x 5.00 =		8.50
O/W	SAY =	0.50	
		8.35	8.50

BY INSPECTION, PROV. 203UC46

(REACTIONS DL = 4.2, IL = 4.3)

BEAM B2 - SPAN 3000

		DL	IL
SLAB	1.20 x 10.5 =	12.60	
GRILL	0.60 x 0.50 =	0.30	
IL	1.80 x 5.00 =		9.00
WALL	0.50 x 4.80 =	2.40	
O/W	SAY =	0.50	
		15.80	9.00

BY INSPECTION, PROV. 203UC46

(REACTIONS DL = 23.7, IL = 13.5)

BEAM B3 - SPAN 2450

		DL	IL
SLAB	0.50 x 10.50 =	5.25	
	5.00 =		2.50
WALL	0.50 x 4.80 =	2.40	
O/W	SAY =	0.50	
		8.15	2.50

BY INSPECTION, PROV. 203UC46

(REACTIONS DL = 10.0, IL = 3.1)

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CALCULATION SHEET	Title: BASEMENT	Date	OCT/14
By: SM	Checked:	Sheet No. GF/2	Rev

BEAM B4 - SPAN 2600

GF	2.60 x 1.00 =	2.60		
	0.50 =		1.30	
o/w	SAY =	0.50		
		3.10	1.30	

BY INSPECTION, PROV. 152UC23

(REACTIONS, DL = 4.0, IL = 1.7)

BEAM B5 - SPAN 1050

FF	0.50 x 0.50 =	0.25	
	1.50 =		0.75
GF	0.50 x 1.00 =	0.50	
	1.50 =		0.75
WALL	3.20 x 2.60 =	8.32	
o/w	SAY =	0.50	
		9.57	1.50

BY INSPECTION, PROV. 152UC23

(REACTIONS DL = 5.0, IL = 0.8)

BEAM B6 - SPAN 1200

GF	1.00 x 1.00 =	1.00	
	1.50 =		1.50
WALL	3.20 x 1.00 =	3.20	
o/w	SAY =	0.50	
		4.70	1.50

POINT LOAD B5 = 5.0
= 0.8

BY INSPECTION, PROV 152UC23

(REACTIONS, DL = 7.6, IL = 1.7 LH
DL = 3.0, IL = 0.9 RH)

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CALCULATION SHEET	TITLE: BASEMENT	DATE	OCT/14
	BY: SM / CHECKED:	SHEET NO.	GF/3

BEAM B7 - SPAN 4500

FF	1	2.60x0.50	=	1.30	
	1	1.50	=		3.90
GF	1	2.60x1.00	=	2.60	
	1	1.50	=		3.90
WALL	1	3.20x2.60	=	8.32	
o/w	1	SAY	=	0.50	
				12.72	7.80

POINT LOAD B6 = 7.6
= 1.7

FROM GFC/1, PROV. 203UC60

BEAM B8 - SPAN 1350

GF	1	0.50x1.00	=	0.50	
	1	x1.50	=		0.75
GF	1	0.50x10.50	=	5.25	
	1	5.00	=		2.50
o/w		SAY	=	0.50	
				6.25	3.25

POINT LOAD B5 = 5.0 B7 = 34.5
= 0.8 = 18.9

FROM GFC/2, PROV. 203UC46

BEAM B9 - SPAN 4400

GF	1	2.00x1.00	=	2.00	
	1	1.50	=		3.00
o/w	1	SAY	=	0.50	
				2.50	3.00

CHIMNEY SAY 12.4 x 2.6 = 32.24

FROM GFC/3, PROV. 203UC52

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BEAM B10 - SPAN 4200

GF	1.80 x 1.00 =	1.80	
	1.50 =		2.70
O/W	SAY =	0.50	
		2.30	2.70

CHIMNEY AS B9 SAY

FROM GFC/4, PROV. 203UC46

BEAM B11 - SPAN 1050

GF	2.80 x 1.00 =	2.80	
	1.50 =		4.20
O/W	SAY =	0.50	
		3.30	4.20

BY INSPECTION, PROV. 152UC23

(REACTIONS DL = 1.7, IL = 2.2)

BEAM B12 - SPAN 4200

ROOF	1.20 x 1.20 =	1.44	
	0.65 =		0.78
TF, SF, FF	3 x 2.6 x 0.50 =	3.90	
	1.50 =		11.70
GF	2.0 x 1.00 =	2.00	
	1.50 =		3.00
BRICK	3.20 x 2.60 =	8.32	
STUD	8.20 x 0.60 =	4.92	
O/W	SAY =	0.50	
		21.08	15.48

FROM GFC/5, PROV. 203UC71

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	BY: SM	CHECKED	SHEET NO. GF/5 / REV

BEAM B13 - SPAN 2900

ROOF	2.90x1.20	=	3.48	
	0.65	=		1.89
TF,SF,FF	3x2.1x0.5	=	3.15	
	1.5	=		9.45
GF	2.10x1.00	=	2.10	
	1.50	=		3.15
GF	1.30x6.00	=	7.80	
	1.50	=		1.95
WALL	3.20x7.20	=	23.04	
WALL	6.00x4.80	=	28.80	
WALL	2.30x1.00	=	2.30	
O/W	SAY	=	1.00	
			71.67	16.44

POINT LOAD (FF ANNEXE BEAM)

ROOF	2.6x2.60x1.00	=	6.76	
	0.75	=		5.07
SF,F	x2x1.6x0.5	=	4.16	
	1.5	=		12.48
WALL	x5.70x4.80	=	71.14	
O/W	x SAY 1.00	=	2.60	
			84.66	17.55

POINT LOAD B10 = 30.6
= 5.7

FROM GFC/6, PROV. 254UC73

BEAM B14 - SPAN 1150

UDL 1 AS B13

UDL 2 AS B13 GF SAY	=	2.10	3.15
	=	7.80	1.95
O/W	=	1.00	
		10.90	5.10

POINT LOAD B12 = 44.3 + 32.5

FROM GFC/7, PROV. 203UC46

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BEAM B15 - SPAN 4900

GF | 0.90 x 1.00 = 0.90
| 1.50 = 1.35
O/W | SAY = 1.00
1.90 1.35

POINT LOAD B11 = 1.7 B14 = 25.9
= 2.2 = 15.0

FROM GFC/8, PROV. 203UC46

BEAM B16 - SPAN 3100

ROOF | 3.50 x 1.20 = 4.20
| 0.60 = 2.10
TF, SF, FF | 3 x 4.3 x 0.5 = 6.45
| 1.5 = 19.35
GF | 4.30 x 1.00 = 4.30
| 1.50 = 6.45
WALL | 3.20 x 2.60 = 8.32
WALL | 8.20 x 0.60 = 4.92
O/W | SAY = 1.00
29.19 27.90

POINT LOAD B9 = 31.3 B10 = 30.6
= 6.6 = 5.7

FROM GFC/9, PROV. 203UC60

BEAM B17 - SPAN 2200

UDL 1 AS B16 SAY = 29.19
= 27.90
0.51 x B16 (LESS WALLS) = 7.98
= 13.95
37.17 41.85

UDL 2 AS B16 GF = 4.30 6.45
+ O/W = 1.00
5.30 6.45

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PROJECT: 65 GOLDHURST TERRACE, NW6		JOB NO.: 4402	
CALCULATION SHEET	TITLE: BASEMENT	DATE: OCT/14	
	BY: SM	CHECKED:	SHEET NO.: GF/7	REV:

BEAM B17 - CONT.

POINT LOAD B12 = 44.3 / = 32.5 B7 = 30.3 / = 17.9 B15 = 9.6 / = 7.1

FROM GFC/10, PROV. 203UC46

BEAM B18 - SPAN 3900

GF	0.50x6.00	=	3.00	
	1.50	=		0.75
o/w	SAY	=	0.50	
			3.50	0.75

CHIMNEY SAY 9.4x2.6 = 24.44

FROM GFC/11, PROV. 203UC46

BEAM B19 - SPAN 1050

GF	2.20x1.00	=	2.20	
	1.50	=		3.30
GF	0.50x6.00	=	3.00	
	1.50	=		0.75
o/w	SAY	=	0.50	
			5.70	4.05

POINT LOAD B18 = 32.2 / = 1.5

FROM GFC/12, PROV. 203UC46

BEAM B20 - SPAN 4150

GF	2.70x6.00	=	16.20	
	1.50	=		4.05
o/w	SAY	=	1.00	
			17.20	4.05

FROM GFC/13, PROV. 203UC46

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Project: 65 GOLDHURST TERRACE, NW6		Job No.: 4402	
CALCULATION SHEET	Title: BASEMENT	Date: OCT/14	
	By: SM / Checked:	Sheet No.: GF/8	Rev:

BEAM B21 - SPAN 3000

GF	1	2.50 x 6.00	=	15.00	
	1	1.50	=		3.75
o/w	1		=	0.50	
				15.50	3.75

BY INSPECTION, PROV. 203UC46

(REACTIONS DL = 23.25, IL = 5.6)

BEAM B22 - SPAN 3900

GF	1	2.20 x 6.00	=	13.20	
	1	1.50	=		3.30
o/w	1	SAY	=	0.50	
				13.70	3.30

BY INSPECTION, PROV. 203UC46

(REACTIONS DL = 26.7, IL = 6.4)

BEAM B23 - SPAN 3900

ROOF	1	2.30 x 1.00	=	2.30	
	1	0.75	=		1.73
GF	1	1.10 x 6.00	=	6.60	
	1	1.50 x 1.50	=		2.25
WALL	1	3.20 x 4.80	=	15.36	
o/w	1	SAY	=	0.50	
				24.76	3.98

FROM GFC/14, PROV. 203UC52

BEAM B24 - SPAN 4600

GF	1	1.00 x 6.00	=	6.00	
	1	1.50	=		1.50
o/w	1	SAY	=	1.00	
				7.00	1.50

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CALCULATION SHEET	Title: BASEMENT		Date: OCT/14	
	By: SM	Checked:	Sheet No.: GF/9	Rev:

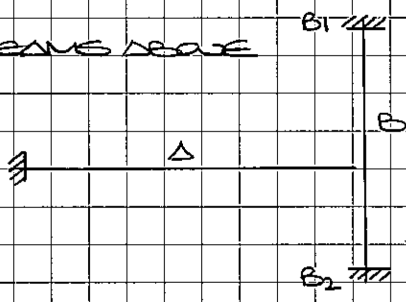
BEAM B24 - CONT.

POINT LOAD B19 = 12.2   B20 = 35.7
= 2.6   = 8.4

FROM GFC/15, PROV. 203UC60

BEAM B25 - SPAN 5100

BEAMS ABOVE



BEAM A REACTION AS PAGE GF/5
B13 POINT LOAD (ANNEXE BEAM)

= 84.66
= 17.55

UDL 1 ON BEAM B AS ABOVE

= 32.56
= 6.75

UDL 2 ON BEAM B SAY ROOF | 1.70x1.00 = 1.70
| 0.75 = 1.28
o/w | SAY = 0.50
2.20   1.28

BY SIMPLE STATICS, REACTIONS B1 = 104.3   B2 = 85.7
= 22.0   = 19.1

UDL GF | 2.00x6.00 = 12.00
| 1.50 = 3.00
o/w | SAY = 1.00
13.00   3.00

POINT LOAD B18 = 29.1   B24 = 33.1
1.5   = 7.4
B21 = 23.3   BEAM OVER = 104.3
= 5.6   = 22.0

FROM GFC/16, PROV. 254UC89

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CALCULATION SHEET	Title: Basement	Date: Oct/14	
	By: SM / Checked:	Sheet No. GF/10	Rev

<u>Beam B26 - Span 800</u>

UDL as B23 say = 24.76
= 3.98

By inspection, Prov. 203UC46

(Reactions DL = 9.9, IL = 1.6)

<u>Beam B27 - Span 4400</u>

GF	1.00 x 6.00 =	6.00	
	1.50 =		1.50
o/w	say =	1.00	
		7.00	1.50

Point load B21 = 23.3   B22 = 26.7
= 5.6   = 6.4

From GFC/17, Prov. 203UC71

BEAM	LHS		RHS	
REF.	DL	IL	DL	IL
B1	4.2	4.3	4.2	4.3
B2	23.7	13.5	23.7	13.5
B3	10.0	3.1	10.0	3.1
B4	4.0	1.7	4.0	1.7
B5	5.0	0.8	5.0	0.8
B6	7.6	1.7	3.0	0.9
B7	34.5	18.9	30.3	17.9
B8	40.4	20.4	7.5	3.7
B9	31.3	6.6	31.3	6.6
B10	30.6	5.7	30.6	5.7
B11	1.7	2.2	1.7	2.2
B12	44.3	32.5	44.3	32.5
B13	162.0	35.3	161.1	35.6
B14	55.2	27.9	25.9	15.0
B15	9.6	7.1	27.3	16.7
B16	101.2	54.4	51.2	44.4
B17	87.6	73.6	38.5	31.7
B18	32.2	1.5	29.1	1.5
B19	12.2	2.6	26.0	3.2

BEAM	LHS		RHS	RHS
REF.	DL	IL	DL	IL
B20	35.7	8.4	35.7	8.4
B21	23.3	5.6	23.3	5.6
B22	26.7	6.4	26.7	6.4
B23	48.3	7.8	48.3	7.8
B24	47.0	10.5	33.1	7.4
B25	54.4	12.4	201.7	39.4
B26	9.9	1.6	9.9	1.6
B27	42.8	9.9	38.0	8.7

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BEAM BEARINGS

Allowable bearing stresses beneath concentrated loads such as beam bearings are;

For existing brickwork, 0.7 N/mm^2 (EXB)

For new 50N brickwork, 3.1 N/mm^2 (NWB)

For new 35N concrete, 5.9 N/mm^2 (CON)

CP = Concrete Padstone; EB = single Engineering Brick; BC = Bearing onto Concrete.

All loads are un-factored.

BEAM	END	LOAD	TYPE	BEARING
B1	LH	8.5	NWB	EB
	RH	8.5	NWB	EB
B2	RH	37.2	NWB	EB
B3	LH	13.1	CON	100 BC
B4	LH	5.7	EXB	EB
	RH	5.7	EXB	EB
B6	RH	3.9	EXB	EB
B8	LH	60.8	NWB	300x100x150 CP
	RH	11.2	EXB	EB
B9	LH	37.9	EXB	450x100x150 CP
B11	RH	3.9	EXB	EB
B19	RH	29.2	EXB	450x100x225 CP
B20	LH	44.1	EXB	203x102 UB x 700mm
B22	RH	33.1	EXB	500x100x225 CP
B23	RH	56.1	CON	100 BC
B26	LH	11.5	EXB	EB
B27	RH	46.7	EXB	203x102 UB x 700mm

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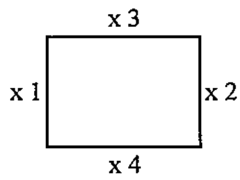
CALCULATION SHEET			
Project: 65 GOLDHURST TERRACE, NW6		Job No. 4402	
Title: BASEMENT		Date: OCT/14	
By: SM	Checked	Sheet No. GF/13	Rev

BENDING MOMENTS IN NEW COLUMN C1

Enter the following:

	Beam Ref;	Char. DL	Char. LL
at x1	B3	10.00	3.10
at x2			
at x3	B2	23.70	13.50
at x4			
		33.70	16.60



Total Load = **50.30** kN
Total Ultimate Load = **73.74** kN

Enter Column Height = **3.10** m

Total Ultimate Sway (x1-x2) say = **0.35** kN (say 2.5% of Dead Load)
Total Ultimate Sway (x3-x4) say = **0.83** kN

Net DL (x1-x2) = **10.00** kN
Net DL (x3-x4) = **23.70** kN

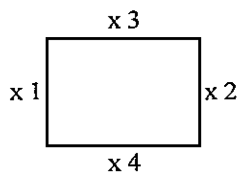
Max. LL (x1-x2) = **3.10** kN
Max. LL (x3-x4) = **13.50** kN

Total Ultimate Moment (x1-x2) say = **2.98** kN.m
Total Ultimate Moment (x3-x4) say = **8.05** kN.m

BENDING MOMENTS IN NEW COLUMN C2

Enter the following:

	Beam Ref;	Char. DL	Char. LL
at x1			
at x2			
at x3	B16	101.20	54.40
at x4			
		101.20	54.40



Total Load = **155.60** kN
Total Ultimate Load = **228.72** kN

Column Height = **3.10** m

Total Ultimate Sway (x1-x2) say = **0.00** kN (say 2.5% of Dead Load)
Total Ultimate Sway (x3-x4) say = **3.54** kN

Net DL (x1-x2) = **0.00** kN
Net DL (x3-x4) = **101.20** kN

Max. LL (x1-x2) = **0.00** kN
Max. LL (x3-x4) = **54.40** kN

Total Ultimate Moment (x1-x2) say = **0.00** kN.m
Total Ultimate Moment (x3-x4) say = **33.85** kN.m

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CALCULATION SHEET	Project: 65 GOLDHURST TERRACE, NW6		Job No.: 4402	
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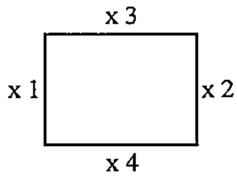
BENDING MOMENTS IN NEW COLUMN C3

Enter the following:

	Beam Ref;	Char. DL	Char. LL
at x1			
at x2			
at x3	B17	87.60	73.60
at x4	B16	51.20	44.40
		138.80	118.00



Total Load = **256.80** kN
Total Ultimate Load = **383.12** kN

Enter Column Height = **3.10** m

Total Ultimate Sway (x1-x2) say = **0.00** kN (say 2.5% of Dead Load)
Total Ultimate Sway (x3-x4) say = **4.86** kN

Net DL (x1-x2) = **0.00** kN
Net DL (x3-x4) = **36.40** kN

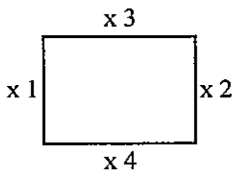
Max. LL (x1-x2) = **0.00** kN
Max. LL (x3-x4) = **73.60** kN

Total Ultimate Moment (x1-x2) say = **0.00** kN.m
Total Ultimate Moment (x3-x4) say = **31.93** kN.m

BENDING MOMENTS IN NEW COLUMN C4

Enter the following:

	Beam Ref;	Char. DL	Char. LL
at x1			
at x2			
at x3			
at x4	B17	38.50	31.70
		38.50	31.70



Total Load = **70.20** kN
Total Ultimate Load = **104.62** kN

Column Height = **3.10** m

Total Ultimate Sway (x1-x2) say = **0.00** kN (say 2.5% of Dead Load)
Total Ultimate Sway (x3-x4) say = **1.35** kN

Net DL (x1-x2) = **0.00** kN
Net DL (x3-x4) = **38.50** kN

Max. LL (x1-x2) = **0.00** kN
Max. LL (x3-x4) = **31.70** kN

Total Ultimate Moment (x1-x2) say = **0.00** kN.m
Total Ultimate Moment (x3-x4) say = **14.64** kN.m

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Project	65 GOLDHURST TERRACE, NW6		Job No.	4402
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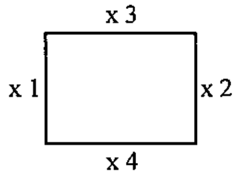
BENDING MOMENTS IN NEW COLUMN C5

Enter the following:

	Beam Ref;	Char. DL	Char. LL
at x1			
at x2			
at x3	B13	162.00	35.30
at x4			
		162.00	35.30



Total Load = **197.30** kN

Total Ultimate Load = **283.28** kN

Enter Column Height = **3.10** m

Total Ultimate Sway (x1-x2) say = **0.00** kN (say 2.5% of Dead Load)

Total Ultimate Sway (x3-x4) say = **5.67** kN

Net DL (x1-x2) = **0.00** kN

Net DL (x3-x4) = **162.00** kN

Max. LL (x1-x2) = **0.00** kN

Max. LL (x3-x4) = **35.30** kN

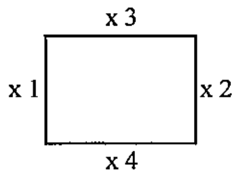
Total Ultimate Moment (x1-x2) say = **0.00** kN.m

Total Ultimate Moment (x3-x4) say = **45.91** kN.m

BENDING MOMENTS IN NEW COLUMN C6

Enter the following:

	Beam Ref;	Char. DL	Char. LL
at x1			
at x2			
at x3	B14	55.20	27.90
at x4	B13	161.10	35.60
		216.30	63.50



Total Load = **279.80** kN

Total Ultimate Load = **404.42** kN

Column Height = **3.10** m

Total Ultimate Sway (x1-x2) say = **0.00** kN (say 2.5% of Dead Load)

Total Ultimate Sway (x3-x4) say = 7.57 kN

Net DL (x1-x2) = **0.00** kN

Net DL (x3-x4) = 105.90 kN

Max. LL (x1-x2) = **0.00** kN

Max. LL (x3-x4) = **35.60** kN

Total Ultimate Moment (x1-x2) say = **0.00** kN.m

Total Ultimate Moment (x3-x4) say = 43.99 kN.m

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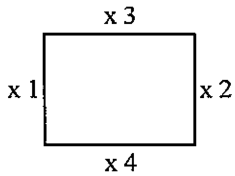
CALCULATION SHEET	Project: 65 GOLDHURST TERRACE, NW6		Job No.: 4402	
	Title: BASEMENT		Date: OCT/14	
	By: SM	Checked:	Sheet No.: GF/16	Rev:

BENDING MOMENTS IN NEW COLUMN C7

Enter the following:

	Beam Ref;	Char. DL	Char. LL
at x1	B15	27.30	16.70
at x2	B24	47.00	10.50
at x3			
at x4			
		74.30	27.20



Total Load = **101.50** kN
Total Ultimate Load = **147.54** kN

Enter Column Height = **3.10** m

Total Ultimate Sway (x1-x2) say = **2.60** kN (say 2.5% of Dead Load)
Total Ultimate Sway (x3-x4) say = **0.00** kN

Net DL (x1-x2) = **19.70** kN
Net DL (x3-x4) = **0.00** kN

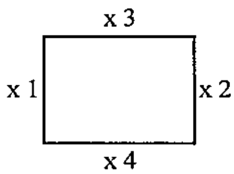
Max. LL (x1-x2) = **16.70** kN
Max. LL (x3-x4) = **0.00** kN

Total Ultimate Moment (x1-x2) say = **13.49** kN.m
Total Ultimate Moment (x3-x4) say = **0.00** kN.m

BENDING MOMENTS IN NEW COLUMN C8

Enter the following:

	Beam Ref;	Char. DL	Char. LL
at x1			
at x2			
at x3	B25	54.40	12.40
at x4			
		54.40	12.40



Total Load = **66.80** kN
Total Ultimate Load = **96.00** kN

Column Height = **3.10** m

Total Ultimate Sway (x1-x2) say = **0.00** kN (say 2.5% of Dead Load)
Total Ultimate Sway (x3-x4) say = **1.90** kN

Net DL (x1-x2) = **0.00** kN
Net DL (x3-x4) = **54.40** kN

Max. LL (x1-x2) = **0.00** kN
Max. LL (x3-x4) = **12.40** kN

Total Ultimate Moment (x1-x2) say = **0.00** kN.m
Total Ultimate Moment (x3-x4) say = **15.50** kN.m

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CALCULATION SHEET				
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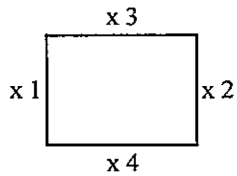
BENDING MOMENTS IN NEW COLUMN C9



Enter the following:

	Beam Ref;	Char. DL	Char. LL
at x1			
at x2			
at x3			
at x4	B25	201.70	39.40
		201.70	39.40

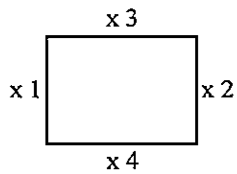
Total Load = **241.10** kN — Enter Column Height = **3.10** m
Total Ultimate Load = **345.42** kN

Total Ultimate Sway (x1-x2) say = **0.00** kN (say 2.5% of Dead Load)
Total Ultimate Sway (x3-x4) say = **7.06** kN

Net DL (x1-x2) = **0.00** kN — Max. LL (x1-x2) = **0.00** kN
Net DL (x3-x4) = **201.70** kN — Max. LL (x3-x4) = **39.40** kN

Total Ultimate Moment (x1-x2) say = **0.00** kN.m
Total Ultimate Moment (x3-x4) say = **56.43** kN.m

BENDING MOMENTS IN NEW COLUMN C10



Enter the following:

	Beam Ref;	Char. DL	Char. LL
at x1			
at x2	B23	48.30	7.80
at x3	B27	42.80	9.90
at x4	B26	9.90	1.60
		101.00	19.30

Total Load = **120.30** kN — Column Height = **3.10** m
Total Ultimate Load = **172.28** kN

Total Ultimate Sway (x1-x2) say = **1.69** kN (say 2.5% of Dead Load)
Total Ultimate Sway (x3-x4) say = **1.84** kN

Net DL (x1-x2) = **48.30** kN — Max. LL (x1-x2) = **7.80** kN
Net DL (x3-x4) = **32.90** kN — Max. LL (x3-x4) = **9.90** kN

Total Ultimate Moment (x1-x2) say = **13.25** kN.m
Total Ultimate Moment (x3-x4) say = **11.91** kN.m

BASEMENT FLOOR LEVEL COLUMN FORCES (ULTIMATE)

COL REF.	AXIAL LOAD	BENDING MOMENTS		SHEAR FORCES		OFFSET YES/NO	TYPE	REMARKS
		MAJOR AXIS	MINOR AXIS	MAJOR AXIS	MINOR AXIS			
C1	74	8.1	3.0	0.8	0.4	N	A	150x150x8 SHS
C2	229	-	33.9	-	3.5	Y	B	200x150x10 RHS
C3	383	-	31.9	-	4.9	N	C	250x150x10 RHS
C4	105	-	14.6	-	1.4	Y	D	150x100x10 RHS
C5	283	-	45.9	-	5.7	Y	B	
C6	404	-	44.0	-	7.6	N	C	
C7	148	13.5	-	2.6	-	N	E	150x100x8 RHS
C8	96	-	15.5	-	1.9	Y	D	
C9	345	-	56.4	-	7.1	Y	C	
C10	172	13.3	11.9	1.7	1.8	N	A	

Project:	65 Goldhurst Terrace London NW6	Page:	GFC/1
		Made by:	SM
Client:	Dig For Victory	Date:	Oct/14
Title:	Basement Extension	Ref No:	4402

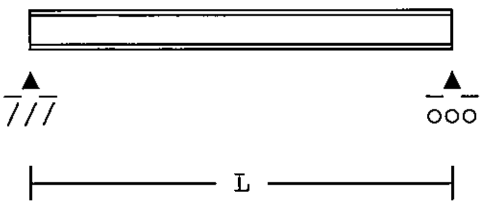
MMP DESIGN

Office: 5831

Location: GROUND FLOOR LEVEL BEAM B7

Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.



Beam span	L=4.5 m
203 x 203 x 60 UC.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Distance from left support	Lc(1)=1.0 m
Dead load (unfactored)	Gkc(1)=7.6 kN
Imposed load (unfactored)	Qkc(1)=1.7 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.5 m
Dead load (unfactored)	Gku(1)=12.72 kN/m
Imposed load (unfactored)	Qku(1)=7.8 kN/m
Maximum span bending moment	83.489 kNm
Design shear force	Fv=78.539 kN
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =182.93 N/mm^2

UNIVERSAL COLUMN DESIGN SUMMARY

203 x 203 x 60 UC Grade S 275

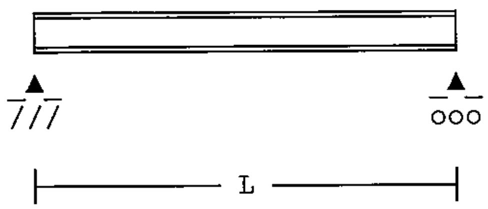
	Maximum shear force	78.539 kN
	Shear capacity	325.09 kN
	Max. applied moment	83.489 kNm
	Moment capacity	180.4 kNm
	Buckling resistance	120 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	120 kNm
	Unfactored DL defln	6.1196 mm
	Unfactored LL defln	3.4741 mm
	Limiting deflection	12.5 mm
Unfactored end shears	DL shear at LHE	34.531 kN
	LL shear at LHE	18.872 kN
	DL shear at RHE	30.309 kN
	LL shear at RHE	17.928 kN

No408

Location: GROUND FLOOR LEVEL BEAM B8

Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.



Beam span	L=1.35 m
203 x 203 x 46 UC.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Distance from left support	Lc(1)=0.1 m
Dead load (unfactored)	Gkc(1)=34.5 kN
Imposed load (unfactored)	Qkc(1)=18.9 kN
Distance from left support	Lc(2)=0.2 m
Dead load (unfactored)	Gkc(2)=5.0 kN
Imposed load (unfactored)	Qkc(2)=0.8 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=1.35 m
Dead load (unfactored)	Gku(1)=6.25 kN/m
Imposed load (unfactored)	Qku(1)=3.25 kN/m
Maximum span bending moment	9.7043 kNm
Design shear force	Fv=89.192 kN
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =272.75 N/mm^2

UNIVERSAL COLUMN DESIGN SUMMARY

203 x 203 x 46 UC Grade S 275

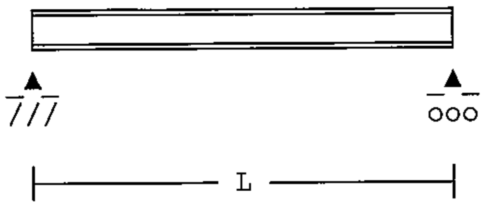
	Maximum shear force	89.192 kN
	Shear capacity	241.4 kN
	Max. applied moment	9.7043 kNm
	Moment capacity	136.68 kNm
	Buckling resistance	135.56 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	135.56 kNm
	Unfactored DL defln	0.082295 mm
	Unfactored LL defln	0.039703 mm
	Limiting deflection	3.75 mm
Unfactored end shears	DL shear at LHE	40.422 kN
	LL shear at LHE	20.375 kN
	DL shear at RHE	7.515 kN
	LL shear at RHE	3.7123 kN

No408

Location: GROUND FLOOR LEVEL BEAM B9

Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.



Beam span	L=4.4 m
203 x 203 x 52 UC.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.4 m
Dead load (unfactored)	Gku(1)=2.5 kN/m
Imposed load (unfactored)	Qku(1)=3.0 kN/m
Dist. from left support to start	Lau(2)=1.4 m
Distance from left support to end	Lbu(2)=3.0 m
Dead load (unfactored)	Gku(2)=32.24 kN/m
Imposed load (unfactored)	Qku(2)=0 kN/m
Maximum span bending moment	85.082 kNm
Design shear force	Fv=54.369 kN
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =176.29 N/mm^2

UNIVERSAL COLUMN DESIGN SUMMARY

203 x 203 x 52 UC Grade S 275

	Maximum shear force	54.369 kN
	Shear capacity	268.78 kN
	Max. applied moment	85.082 kNm
	Moment capacity	155.93 kNm
	Buckling resistance	99.957 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	99.957 kNm
	Unfactored DL defln	9.1109 mm
	Unfactored LL defln	1.3578 mm
	Limiting deflection	12.222 mm
Unfactored end shears	DL shear at LHE	31.292 kN
	LL shear at LHE	6.6 kN
	DL shear at RHE	31.292 kN
	LL shear at RHE	6.6 kN

No408

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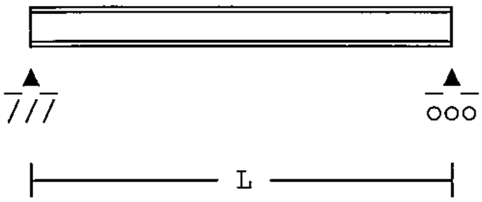
Location: GROUND FLOOR LEVEL BEAM B10



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=4.2 m
203 x 203 x 46 UC.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.2 m
Dead load (unfactored)	Gku(1)=2.3 kN/m
Imposed load (unfactored)	Qku(1)=2.7 kN/m
Dist. from left support to start	Lau(2)=1.3 m
Distance from left support to end	Lbu(2)=2.9 m
Dead load (unfactored)	Gku(2)=32.24 kN/m
Imposed load (unfactored)	Qku(2)=0 kN/m
Maximum span bending moment	78.011 kNm
Design shear force	Fv=51.943 kN
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =173.36 N/mm^2

UNIVERSAL COLUMN DESIGN SUMMARY

203 x 203 x 46 UC Grade S 275

	Maximum shear force	51.943 kN
	Shear capacity	241.4 kN
	Max. applied moment	78.011 kNm
	Moment capacity	136.68 kNm
	Buckling resistance	86.162 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	86.162 kNm
	Unfactored DL defln	8.9354 mm
	Unfactored LL defln	1.1677 mm
	Limiting deflection	11.667 mm
Unfactored end shears	DL shear at LHE	30.622 kN
	LL shear at LHE	5.67 kN
	DL shear at RHE	30.622 kN
	LL shear at RHE	5.67 kN

No408

Project: 65 Goldhurst Terrace
London NW6
Client: Dig For Victory
Title: Basement Extension

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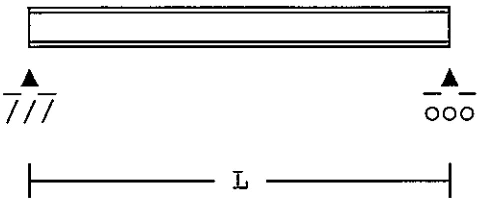
Location: GROUND FLOOR LEVEL BEAM B12

Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.



Beam span	L=4.2 m
203 x 203 x 71 UC.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.2 m
Dead load (unfactored)	Gku(1)=21.08 kN/m
Imposed load (unfactored)	Qku(1)=15.48 kN/m
Maximum span bending moment	119.69 kNm
Design shear force	Fv=113.99 kN
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =195.69 N/mm^2

UNIVERSAL COLUMN DESIGN SUMMARY

203 x 203 x 71 UC Grade S 275

	Maximum shear force	113.99 kN
	Shear capacity	343.12 kN
	Max. applied moment	119.69 kNm
	Moment capacity	211.74 kNm
	Buckling resistance	156.36 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	156.36 kNm
	Unfactored DL defln	5.4676 mm
	Unfactored LL defln	4.0151 mm
	Limiting deflection	11.667 mm
Unfactored end shears	DL shear at LHE	44.268 kN
	LL shear at LHE	32.508 kN
	DL shear at RHE	44.268 kN
	LL shear at RHE	32.508 kN

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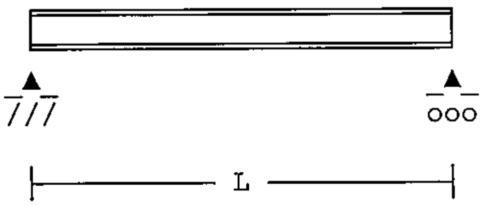
Location: GROUND FLOOR LEVEL BEAM B13



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=2.9 m
254 x 254 x 73 UC.	
Young's Modulus	E=205 kN/mm2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Distance from left support	Lc(1)=0.3 m
Dead load (unfactored)	Gkc(1)=30.6 kN
Imposed load (unfactored)	Qkc(1)=5.7 kN
Distance from left support	Lc(2)=1.85 m
Dead load (unfactored)	Gkc(2)=84.66 kN
Imposed load (unfactored)	Qkc(2)=17.55 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=2.9 m
Dead load (unfactored)	Gku(1)=71.67 kN/m
Imposed load (unfactored)	Qku(1)=16.44 kN/m
Maximum span bending moment	226.87 kNm
Design shear force	Fv=283.3 kN
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =234.55 N/mm2

UNIVERSAL COLUMN DESIGN SUMMARY

254 x 254 x 73 UC Grade S 275

	Maximum shear force	283.3 kN
	Shear capacity	360.57 kN
	Max. applied moment	226.87 kNm
	Moment capacity	260.34 kNm
	Buckling resistance	232.67 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	232.67 kNm
	Unfactored DL defln	4.6777 mm
	Unfactored LL defln	1.0278 mm
	Limiting deflection	8.0556 mm
Unfactored end shears	DL shear at LHE	162.01 kN
	LL shear at LHE	35.303 kN
	DL shear at RHE	161.09 kN
	LL shear at RHE	35.623 kN

No408

Project: 65 Goldhurst Terrace
London NW6
Client: Dig For Victory
Title: Basement Extension

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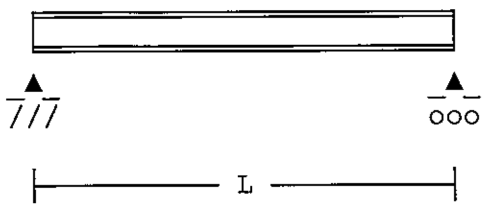
Location: GROUND FLOOR LEVEL BEAM B14



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=1.15 m
203 x 203 x 46 UC.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Distance from left support	Lc(1)=0.4 m
Dead load (unfactored)	Gkc(1)=44.3 kN
Imposed load (unfactored)	Qkc(1)=32.5 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=0.4 m
Dead load (unfactored)	Gku(1)=71.67 kN/m
Imposed load (unfactored)	Qku(1)=16.44 kN/m
Dist. from left support to start	Lau(2)=0.4 m
Distance from left support to end	Lbu(2)=1.15 m
Dead load (unfactored)	Gku(2)=10.9 kN/m
Imposed load (unfactored)	Qku(2)=5.1 kN/m
Maximum span bending moment	38.536 kNm
Design shear force	Fv=121.94 kN

UNIVERSAL COLUMN
DESIGN SUMMARY

203 x 203 x 46 UC Grade S 275

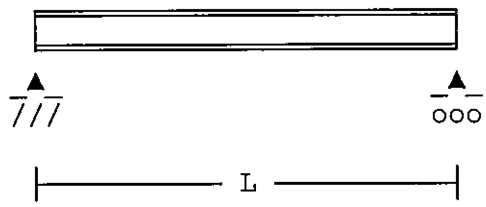
	Maximum shear force	121.94 kN
	Shear capacity	241.4 kN
	Max. applied moment	38.536 kNm
	Moment capacity	136.68 kNm
	Buckling resistance	136.68 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	136.68 kNm
	Unfactored DL defln	0.19705 mm
	Unfactored LL defln	0.11595 mm
	Limiting deflection	3.1944 mm
Unfactored end shears	DL shear at LHE	55.239 kN
	LL shear at LHE	27.875 kN
	DL shear at RHE	25.904 kN
	LL shear at RHE	15.026 kN

No408

Location: GROUND FLOOR LEVEL BEAM B15

Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.



Beam span	L=4.9 m
203 x 203 x 46 UC.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Distance from left support	Lc(1)=1.25 m
Dead load (unfactored)	Gkc(1)=1.7 kN
Imposed load (unfactored)	Qkc(1)=2.2 kN
Distance from left support	Lc(2)=4.2 m
Dead load (unfactored)	Gkc(2)=25.9 kN
Imposed load (unfactored)	Qkc(2)=15.0 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.9 m
Dead load (unfactored)	Gku(1)=1.9 kN/m
Imposed load (unfactored)	Qku(1)=1.35 kN/m
Maximum span bending moment	44.479 kNm
Design shear force	Fv=64.966 kN
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =156.82 N/mm^2

UNIVERSAL COLUMN DESIGN SUMMARY

203 x 203 x 46 UC Grade S 275

	Maximum shear force	64.966 kN
	Shear capacity	241.4 kN
	Max. applied moment	44.479 kNm
	Moment capacity	136.68 kNm
	Buckling resistance	77.941 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	77.941 kNm
	Unfactored DL defln	4.6582 mm
	Unfactored LL defln	3.12 mm
	Limiting deflection	13.611 mm
Unfactored end shears	DL shear at LHE	9.6213 kN
	LL shear at LHE	7.0891 kN
	DL shear at RHE	27.289 kN
	LL shear at RHE	16.726 kN

No408

Project: 65 Goldhurst Terrace
London NW6
Client: Dig For Victory
Title: Basement Extension

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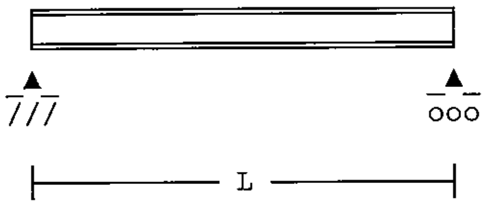
Location: GROUND FLOOR LEVEL BEAM B16



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=3.1 m
203 x 203 x 60 UC.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Distance from left support	Lc(1)=0.3 m
Dead load (unfactored)	Gkc(1)=31.3+30.6=61.9 kN
Imposed load (unfactored)	Qkc(1)=6.6+5.7=12.3 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=3.1 m
Dead load (unfactored)	Gku(1)=29.19 kN/m
Imposed load (unfactored)	Qku(1)=27.9 kN/m
Maximum span bending moment	119.23 kNm
Design shear force	Fv=228.58 kN
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =217.66 N/mm^2

UNIVERSAL COLUMN DESIGN SUMMARY

203 x 203 x 60 UC Grade S 275

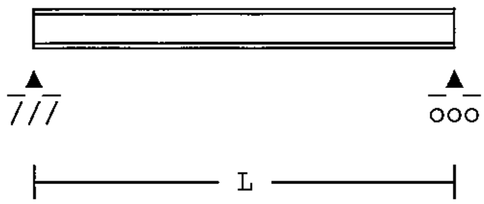
	Maximum shear force	228.58 kN
	Shear capacity	325.09 kN
	Max. applied moment	119.23 kNm
	Moment capacity	175.71 kNm
	Buckling resistance	142.78 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	142.78 kNm
	Unfactored DL defln	3.6697 mm
	Unfactored LL defln	2.8439 mm
	Limiting deflection	8.6111 mm
Unfactored end shears	DL shear at LHE	101.15 kN
	LL shear at LHE	54.355 kN
	DL shear at RHE	51.235 kN
	LL shear at RHE	44.435 kN

No408

Location: GROUND FLOOR LEVEL BEAM B17

Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.



Beam span	L=2.2 m
203 x 203 x 46 UC.	
Young's Modulus	E=205 kN/mm2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Distance from left support	Lc(1)=0.4 m
Dead load (unfactored)	Gkc(1)=44.3 kN
Imposed load (unfactored)	Qkc(1)=32.5 kN
Distance from left support	Lc(2)=0.95 m
Dead load (unfactored)	Gkc(2)=30.3 kN
Imposed load (unfactored)	Qkc(2)=17.9 kN
Distance from left support	Lc(3)=1.15 m
Dead load (unfactored)	Gkc(3)=9.6 kN
Imposed load (unfactored)	Qkc(3)=7.1 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=0.95 m
Dead load (unfactored)	Gku(1)=37.17 kN/m
Imposed load (unfactored)	Qku(1)=41.85 kN/m
Dist. from left support to start	Lau(2)=0.95 m
Distance from left support to end	Lbu(2)=2.2 m
Dead load (unfactored)	Gku(2)=5.3 kN/m
Imposed load (unfactored)	Qku(2)=6.45 kN/m
Maximum span bending moment	111.8 kNm
Design shear force	Fv=240.44 kN
Length of beam between restraints	LT=1.25 m

UNIVERSAL COLUMN DESIGN SUMMARY

203 x 203 x 46 UC Grade S 275

	Maximum shear force	240.44 kN
	Shear capacity	241.4 kN
	Max. applied moment	111.8 kNm
	Moment capacity	116.56 kNm
	Buckling resistance	136.68 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	136.68 kNm
	Unfactored DL defln	2.0511 mm
	Unfactored LL defln	1.6431 mm
	Limiting deflection	6.1111 mm
Unfactored end shears	DL shear at LHE	87.613 kN
	LL shear at LHE	73.614 kN
	DL shear at RHE	38.524 kN
	LL shear at RHE	31.706 kN

Project: 65 Goldhurst Terrace
London NW6
Client: Dig For Victory
Title: Basement Extension

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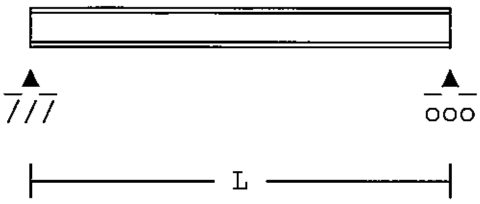
Location: GROUND FLOOR LEVEL BEAM B18



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=3.9 m
203 x 203 x 46 UC.	
Young's Modulus	E=205 kN/mm2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=3.9 m
Dead load (unfactored)	Gku(1)=3.5 kN/m
Imposed load (unfactored)	Qku(1)=0.75 kN/m
Dist. from left support to start	Lau(2)=0.85 m
Distance from left support to end	Lbu(2)=2.8 m
Dead load (unfactored)	Gku(2)=24.44 kN/m
Imposed load (unfactored)	Qku(2)=0 kN/m
Maximum span bending moment	60.137 kNm
Design shear force	Fv=47.394 kN
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =181.36 N/mm2

UNIVERSAL COLUMN DESIGN SUMMARY

203 x 203 x 46 UC Grade S 275

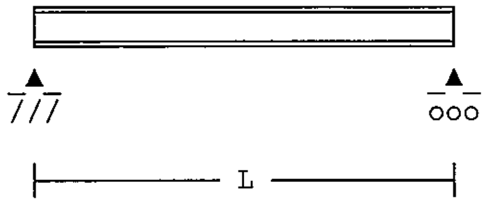
	Maximum shear force	47.394 kN
	Shear capacity	241.4 kN
	Max. applied moment	60.137 kNm
	Moment capacity	136.68 kNm
	Buckling resistance	90.138 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	90.138 kNm
	Unfactored DL defln	6.6954 mm
	Unfactored LL defln	0.24115 mm
	Limiting deflection	10.833 mm
Unfactored end shears	DL shear at LHE	32.182 kN
	LL shear at LHE	1.4625 kN
	DL shear at RHE	29.127 kN
	LL shear at RHE	1.4625 kN

No408

Location: GROUND FLOOR LEVEL BEAM B19

Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.



Beam span	L=1.05 m
203 x 203 x 46 UC.	
Young's Modulus	E=205 kN/mm2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Distance from left support	Lc(1)=0.75 m
Dead load (unfactored)	Gkc(1)=32.2 kN
Imposed load (unfactored)	Qkc(1)=1.5 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=1.05 m
Dead load (unfactored)	Gku(1)=5.7 kN/m
Imposed load (unfactored)	Qku(1)=4.05 kN/m
Maximum span bending moment	11.645 kNm
Design shear force	Fv=41.506 kN

UNIVERSAL COLUMN DESIGN SUMMARY

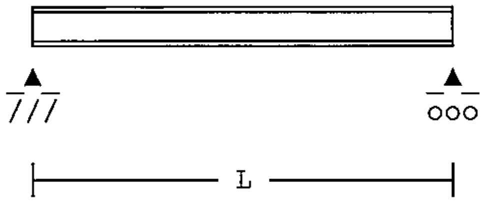
203 x 203 x 46 UC Grade S 275

	Maximum shear force	41.506 kN
	Shear capacity	241.4 kN
	Max. applied moment	11.645 kNm
	Moment capacity	136.68 kNm
	Buckling resistance	136.68 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	136.68 kNm
	Unfactored DL defln	0.072946 mm
	Unfactored LL defln	0.0097915 mm
	Limiting deflection	2.9167 mm
Unfactored end shears	DL shear at LHE	12.193 kN
	LL shear at LHE	2.5548 kN
	DL shear at RHE	25.992 kN
	LL shear at RHE	3.1977 kN

No408

Office: 5831

Location: GROUND FLOOR LEVEL BEAM B20



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=4.15 m
203 x 203 x 46 UC.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.15 m
Dead load (unfactored)	Gku(1)=17.2 kN/m
Imposed load (unfactored)	Qku(1)=4.05 kN/m
Maximum span bending moment	65.79 kNm
Design shear force	Fv=63.412 kN
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =191.23 N/mm^2

UNIVERSAL COLUMN DESIGN SUMMARY

203 x 203 x 46 UC Grade S 275

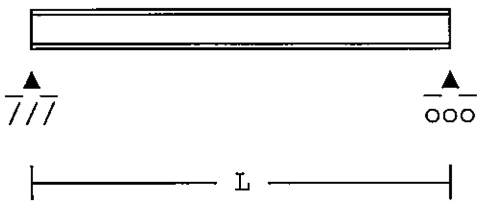
	Maximum shear force	63.412 kN
	Shear capacity	241.4 kN
	Max. applied moment	65.79 kNm
	Moment capacity	136.68 kNm
	Buckling resistance	95.041 kNm
	Moment factor (mLT)	0.925
	Resistance (Mb/mLT)	102.75 kNm
	Unfactored DL defln	7.0907 mm
	Unfactored LL defln	1.6696 mm
	Limiting deflection	11.528 mm
Unfactored end shears	DL shear at LHE	35.69 kN
	LL shear at LHE	8.4038 kN
	DL shear at RHE	35.69 kN
	LL shear at RHE	8.4038 kN

No408

Location: GROUND FLOOR LEVEL BEAM B23

Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.



Beam span	L=3.9 m
203 x 203 x 46 UC.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=3.9 m
Dead load (unfactored)	Gku(1)=24.76 kN/m
Imposed load (unfactored)	Qku(1)=3.98 kN/m
Maximum span bending moment	78.012 kNm
Design shear force	Fv=80.012 kN
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =197.79 N/mm^2

UNIVERSAL COLUMN DESIGN SUMMARY

203 x 203 x 46 UC Grade S 275

	Maximum shear force	80.012 kN
	Shear capacity	241.4 kN
	Max. applied moment	78.012 kNm
	Moment capacity	136.68 kNm
	Buckling resistance	98.304 kNm
	Moment factor (mLT)	0.925
	Resistance (Mb/mLT)	106.27 kNm
	Unfactored DL defln	7.9612 mm
	Unfactored LL defln	1.2797 mm
	Limiting deflection	10.833 mm
Unfactored end shears	DL shear at LHE	48.282 kN
	LL shear at LHE	7.761 kN
	DL shear at RHE	48.282 kN
	LL shear at RHE	7.761 kN

No408

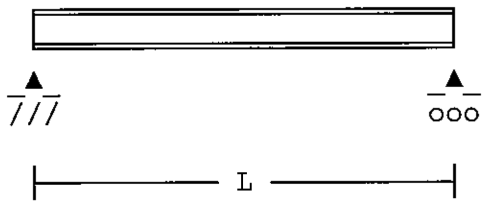
Project: 65 Goldhurst Terrace
London NW6
Client: Dig For Victory
Title: Basement Extension

MMP DESIGN

Page: GFC/15
Made by: SM
Date: Oct/14
Ref No: 4402

Office: 5831

Location: GROUND FLOOR LEVEL BEAM B24



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=4.6 m
203 x 203 x 60 UC.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Distance from left support	Lc(1)=0.7 m
Dead load (unfactored)	Gkc(1)=12.2 kN
Imposed load (unfactored)	Qkc(1)=2.6 kN
Distance from left support	Lc(2)=1.95 m
Dead load (unfactored)	Gkc(2)=35.7 kN
Imposed load (unfactored)	Qkc(2)=8.4 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.6 m
Dead load (unfactored)	Gku(1)=7.0 kN/m
Imposed load (unfactored)	Qku(1)=1.5 kN/m
Maximum span bending moment	111.2 kNm
Design shear force	Fv=82.603 kN
Length of beam between restraints	LT=2.65 m
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =241.6 N/mm^2

UNIVERSAL COLUMN DESIGN SUMMARY

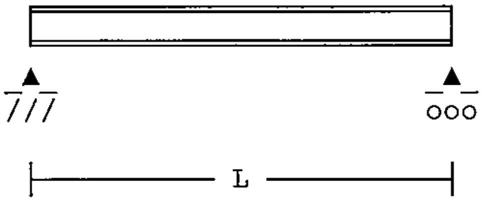
203 x 203 x 60 UC Grade S 275

	Maximum shear force	82.603 kN
	Shear capacity	325.09 kN
	Max. applied moment	111.2 kNm
	Moment capacity	180.4 kNm
	Buckling resistance	158.49 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	158.49 kNm
	Unfactored DL defln	9.6894 mm
	Unfactored LL defln	2.1923 mm
	Limiting deflection	12.778 mm
Unfactored end shears	DL shear at LHE	47.01 kN
	LL shear at LHE	10.493 kN
	DL shear at RHE	33.09 kN
	LL shear at RHE	7.4065 kN

No408

Office: 5831

Location: GROUND FLOOR LEVEL BEAM B25



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=5.1 m
254 x 254 x 89 UC.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Distance from left support	Lc(1)=2.5 m
Dead load (unfactored)	Gkc(1)=23.3 kN
Imposed load (unfactored)	Qkc(1)=5.6 kN
Distance from left support	Lc(2)=4.15 m
Dead load (unfactored)	Gkc(2)=33.1 kN
Imposed load (unfactored)	Qkc(2)=7.4 kN
Distance from left support	Lc(3)=4.9 m
Dead load (unfactored)	Gkc(3)=29.1 kN
Imposed load (unfactored)	Qkc(3)=1.5 kN
Distance from left support	Lc(4)=5.0 m
Dead load (unfactored)	Gkc(4)=104.3 kN
Imposed load (unfactored)	Qkc(4)=22.0 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=5.1 m
Dead load (unfactored)	Gku(1)=13.0 kN/m
Imposed load (unfactored)	Qku(1)=3.0 kN/m
Maximum span bending moment	167.76 kNm
Design shear force	Fv=345.49 kN
Length of beam between restraints	LT=2.45 m
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =255.27 N/mm^2

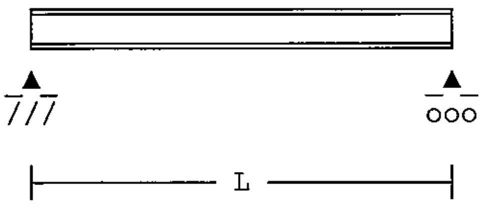
UNIVERSAL COLUMN DESIGN SUMMARY

254 x 254 x 89 UC Grade S 275

	Maximum shear force	345.49 kN
	Shear capacity	426.29 kN
	Max. applied moment	167.76 kNm
	Moment capacity	305.48 kNm
	Buckling resistance	311.43 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	311.43 kNm
	Unfactored DL defln	8.6649 mm
	Unfactored LL defln	1.9394 mm
	Limiting deflection	14.167 mm
Unfactored end shears	DL shear at LHE	54.38 kN
Unfactored end shears	LL shear at LHE	12.374 kN
Unfactored end shears	DL shear at RHE	201.72 kN
Unfactored end shears	LL shear at RHE	39.426 kN

Office: 5831

Location: GROUND FLOOR LEVEL BEAM B27



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=4.4 m
203 x 203 x 71 UC.	
Young's Modulus	E=205 kN/mm2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Distance from left support	Lc(1)=1.75 m
Dead load (unfactored)	Gkc(1)=23.3 kN
Imposed load (unfactored)	Qkc(1)=5.6 kN
Distance from left support	Lc(2)=2.2 m
Dead load (unfactored)	Gkc(2)=26.7 kN
Imposed load (unfactored)	Qkc(2)=6.4 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.4 m
Dead load (unfactored)	Gku(1)=7.0 kN/m
Imposed load (unfactored)	Qku(1)=1.5 kN/m
Maximum span bending moment	118.29 kNm
Design shear force	Fv=75.693 kN
Length of beam between restraints	LT=2.2 m
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =252.2 N/mm2

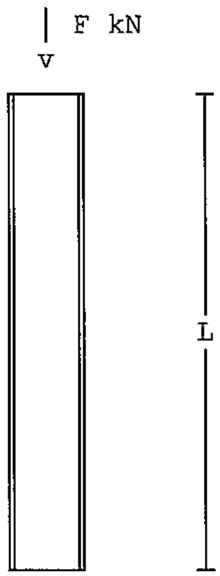
UNIVERSAL COLUMN DESIGN SUMMARY

203 x 203 x 71 UC Grade S 275

	Maximum shear force	75.693 kN
	Shear capacity	343.12 kN
	Max. applied moment	118.29 kNm
	Moment capacity	211.74 kNm
	Buckling resistance	201.51 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	201.51 kNm
	Unfactored DL defln	7.7125 mm
	Unfactored LL defln	1.7947 mm
	Limiting deflection	12.222 mm
Unfactored end shears	DL shear at LHE	42.783 kN
	LL shear at LHE	9.8727 kN
	DL shear at RHE	38.017 kN
	LL shear at RHE	8.7273 kN

No408

Location: COLUMNS C1 & C10



SHS column in 'simple' construction

Calculations are in accordance with BS5950 and 'SHS Design Examples to BS5950' published by British Steel General Steels.

The column is part of simple constr-uction and in accordance with 4.7.7 it is not necessary to consider the effect of pattern loading. All beams supported by the column are assumed to be fully loaded.

It is assumed that all elements of the column remain in compression.

Factored axial compressive load	F=172 kN
Factored BM about major axis x-x	Mx=13.3 kNm
Factored BM about minor axis y-y	My=11.9 kNm
Length between restraints	L=3100 mm

150 x 150 x 8 SHS - Hot finished.
Properties (cm): A=44.8 rx=5.77 Zx=199 Sx=237 Ix=1490 J=2350 C=291
Young's Modulus E=205 kN/mm^2

Effective length factor ef=1.5
Compressive strength pc=pe*py/(phi+(phi^2-pe*py)^0.5)
=202.05 N/mm^2

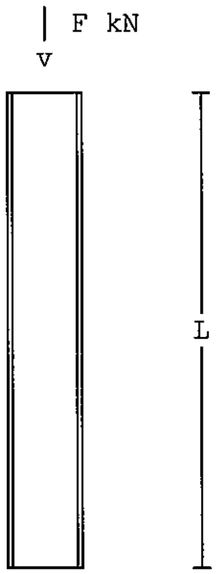
HOT FINISHED SQUARE HOLLOW SECTION — In accordance with EN 10210
150 x 150 x 8 SHS Grade S 275

SECTION SUMMARY — Section is satisfactory for axial load, buckling resistance and overall buckling check.

DESIGN SUMMARY

Axial compressive load	172 kN
Compressive resistance	905.2 kN
Moment about major axis	13.3 kNm
Buckling resistance	65.175 kNm
Moment about minor axis	11.9 kNm
Minor axis resistance	54.725 kNm
Overall buckling check	0.61153 < 1

Location: COLUMNS C2 & C5



SHS column in 'simple' construction

Calculations are in accordance with BS5950 and 'SHS Design Examples to BS5950' published by British Steel General Steels.

The column is part of simple constr-uction and in accordance with 4.7.7 it is not necessary to consider the effect of pattern loading. All beams supported by the column are assumed to be fully loaded.

It is assumed that all elements of the column remain in compression.

Factored axial compressive load	F=283 kN
Factored BM about major axis x-x	Mx=0 kNm
Factored BM about minor axis y-y	My=45.9 kNm
Length between restraints	L=3100 mm

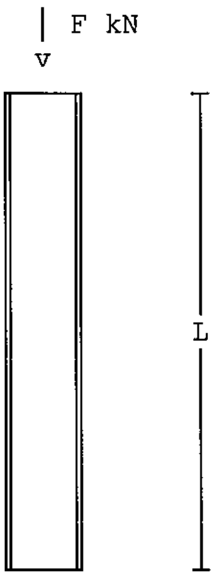
200 x 150 x 10 RHS - Hot finished.
Properties (cm): A=64.9 rx=7.41 Zx=357 Sx=436 Ix=3570
J=4410 C=475 Zy=302 Sy=356 Iy=2260 ry=5.91
Young's Modulus E=205 kN/mm^2

Effective length factor ef=2
Compressive strength pc=pe*py/(phi+(phi^2-pe*py)^0.5)
=146.86 N/mm^2

HOT FINISHED	In accordance with EN 10210
RECTANGULAR HOLLOW SECTION	200 x 150 x 10 RHS Grade S 275
SECTION SUMMARY	Section is satisfactory for axial load, buckling resistance and overall buckling check.

DESIGN SUMMARY	Axial compressive load	283 kN
	Compressive resistance	953.14 kN
	Moment about minor axis	45.9 kNm
	Minor axis resistance	83.05 kNm
	Overall buckling check	0.84959 < 1

Location: COLUMNS C3, C6 & C9



SHS column in 'simple' construction

Calculations are in accordance with BS5950 and 'SHS Design Examples to BS5950' published by British Steel General Steels.

The column is part of simple constr -uction and in accordance with 4.7.7 it is not necessary to consider the effect of pattern loading. All beams supported by the column are assumed to be fully loaded.

It is assumed that all elements of the column remain in compression.

Factored axial compressive load	F=404 kN
Factored BM about major axis x-x	Mx=0 kNm
Factored BM about minor axis y-y	My=56.4 kNm
Length between restraints	L=3100 mm

250 x 150 x 10 RHS - Hot finished.
Properties (cm): A=74.9 rx=9.08 Zx=494 Sx=611 Ix=6170
J=6090 C=605 Zy=367 Sy=426 Iy=2760 ry=6.06

Young's Modulus E=205 kN/mm^2

Effective length factor ef=2
Compressive strength pc=pe*py/(phi+(phi^2-pe*py)^0.5)
=152.38 N/mm^2

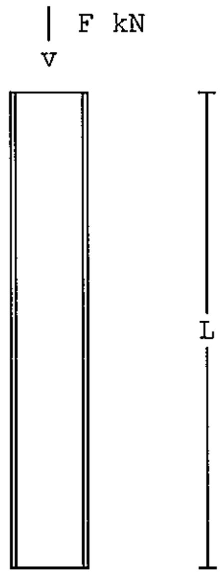
HOT FINISHED RECTANGULAR HOLLOW SECTION — In accordance with EN 10210
250 x 150 x 10 RHS Grade S 275

SECTION SUMMARY — Section is satisfactory for axial load, buckling resistance and overall buckling check.

DESIGN SUMMARY

Axial compressive load	404 kN
Compressive resistance	1141.3 kN
Moment about minor axis	56.4 kNm
Minor axis resistance	100.93 kNm
Overall buckling check	0.9128 < 1

Location: COLUMNS C4 & C8



SHS column in 'simple' construction

Calculations are in accordance with BS5950 and 'SHS Design Examples to BS5950' published by British Steel General Steels.

The column is part of simple constr -uction and in accordance with 4.7.7 it is not necessary to consider the effect of pattern loading. All beams supported by the column are assumed to be fully loaded.

It is assumed that all elements of the column remain in compression.

Factored axial compressive load	F=105 kN
Factored BM about major axis x-x	Mx=0 kNm
Factored BM about minor axis y-y	My=15.5 kNm
Length between restraints	L=3100 mm

150 x 100 x 10 RHS - Hot finished.
Properties (cm): A=44.9 rx=5.34 Zx=171 Sx=216 Ix=1280
J=1430 C=214 Zy=133 Sy=161 Iy=665 ry=3.85
Young's Modulus E=205 kN/mm^2

Effective length factor ef=2
Compressive strength pc=pe*py/(phi+(phi^2-pe*py)^0.5)
=70.306 N/mm^2

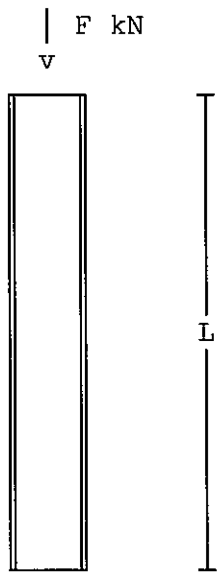
HOT FINISHED
RECTANGULAR HOLLOW SECTION
In accordance with EN 10210
150 x 100 x 10 RHS Grade S 275

SECTION
SUMMARY
Section is satisfactory for axial load, buckling resistance and overall buckling check.

DESIGN
SUMMARY

Axial compressive load	105 kN
Compressive resistance	315.67 kN
Moment about minor axis	15.5 kNm
Minor axis resistance	36.575 kNm
Overall buckling check	0.75641 < 1

Location: COLUMN C7



SHS column in 'simple' construction

Calculations are in accordance with BS5950 and 'SHS Design Examples to BS5950' published by British Steel General Steels.

The column is part of simple constr-uction and in accordance with 4.7.7 it is not necessary to consider the effect of pattern loading. All beams supported by the column are assumed to be fully loaded.

It is assumed that all elements of the column remain in compression.

Factored axial compressive load	F=148 kN
Factored BM about major axis x-x	Mx=13.5 kNm
Factored BM about minor axis y-y	My=0 kNm
Length between restraints	L=3100 mm

150 x 100 x 8 RHS - Hot finished.
Properties (cm): A=36.8 rx=5.44 Zx=145 Sx=180 Ix=1090
J=1200 C=183 Zy=114 Sy=135 Iy=569 ry=3.94

Young's Modulus	E=205 kN/mm^2
Effective length factor	ef=2
Compressive strength	pc=pe*py/(phi+(phi^2-pe*py)^0.5) =73.37 N/mm^2

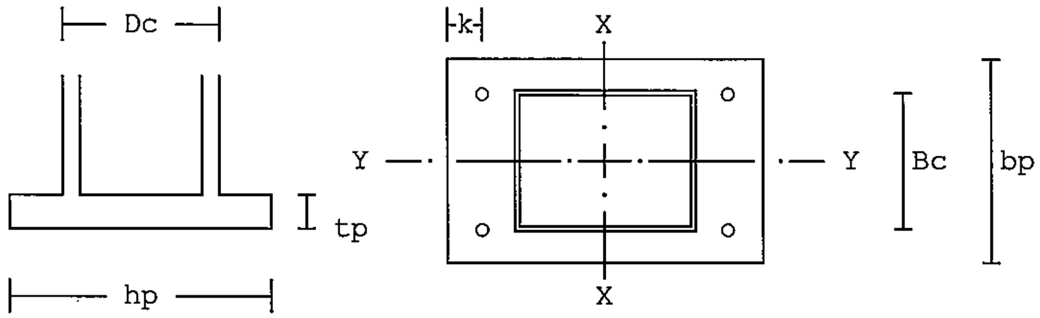
HOT FINISHED RECTANGULAR HOLLOW SECTION	In accordance with EN 10210 150 x 100 x 8 RHS Grade S 275
SECTION SUMMARY	Section is satisfactory for axial load, buckling resistance and overall buckling check.

DESIGN SUMMARY

Axial compressive load	148 kN
Compressive resistance	270 kN
Moment about major axis	13.5 kNm
Buckling resistance	49.5 kNm
Overall buckling check	0.82087 < 1

Location: BASEPLATE TO COLUMNS C1 & C10

Calculations are in accordance with 'Joints in Steel Construction Moment Connections' published by The Steel Construction Institute.



ELEVATION PLAN

Axial load (+ve compression) N=172 kN
Moment about X-X axis M=25.2 kNm
Shear on the base in Y direction Fy=3.5 kN
150 x 150 x 8 SHS - Hot finished.
Properties (cm): A=44.8 rx=5.77 Zx=199 Sx=237 Ix=1490 J=2350 C=291
Length of baseplate hp=350 mm
Breadth of baseplate bp=350 mm
Edge distance to bolt centre line k=40 mm
Assumed fillet weld size sw=8 mm

Strength of concrete fcu=35 N/mm2
Special control must be applied over the placing of the high strength bedding material.
Assumed weld size sw=8 mm
Selected baseplate thickness tp=22 mm

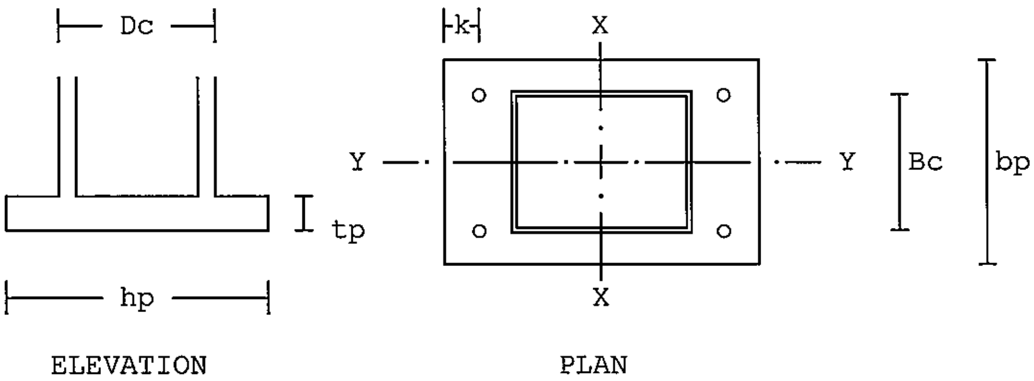
Number of bolts to be used n=4
Bolt diameter bd=16 mm
Selected fillet weld size sw=8 mm

SUMMARY

BASEPLATE	Size 350 mm x 350 mm x 22 mm	
REQUIREMENTS	Grade S 275 steel	
	Edge distance	40 mm
	Number of H.D. bolts	4
	Diameter of bolts	M 16
	Grade 4.6	
	Concrete/grout (fcu)	35 N/mm^2
WELDS	Fillet weld (all round)	8 mm
	Contact areas on the baseplate and column are machined to give a tight bearing contact.	

Location: BASEPLATE TO COLUMNS C2 & C5

Calculations are in accordance with 'Joints in Steel Construction Moment Connections' published by The Steel Construction Institute.



Axial load (+ve compression) N=283 kN
Moment about X-X axis M=45.9 kNm
Shear on the base in Y direction Fy=5.7 kN
200 x 150 x 10 RHS - Hot finished.
Properties (cm): A=64.9 rx=7.41 Zx=357 Sx=436 Ix=3570
J=4410 C=475 Zy=302 Sy=356 Iy=2260 ry=5.91
Length of baseplate hp=400 mm
Breadth of baseplate bp=350 mm
Edge distance to bolt centre line k=40 mm
Assumed fillet weld size sw=8 mm

Strength of concrete fcu=35 N/mm2
Special control must be applied over the placing of the high strength bedding material.
Assumed weld size sw=8 mm
Selected baseplate thickness tp=28 mm

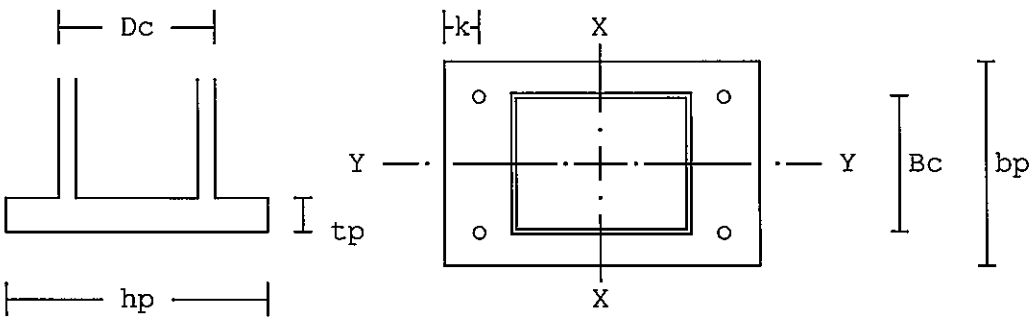
Number of bolts to be used n=4
Bolt diameter bd=16 mm
Selected fillet weld size sw=8 mm

SUMMARY

BASEPLATE REQUIREMENTS	Size 400 mm x 350 mm x 28 mm	
	Grade S 275 steel	
	Edge distance	40 mm
	Number of H.D. bolts	4
	Diameter of bolts	M 16
	Grade 4.6	
	Concrete/grout (fcu)	35 N/mm^2
WELDS	Fillet weld (all round)	8 mm
	Contact areas on the baseplate and column are machined to give a tight bearing contact.	

Location: BASEPLATE TO COLUMNS C3, C6 & C9

Calculations are in accordance with 'Joints in Steel Construction Moment Connections' published by The Steel Construction Institute.



ELEVATION PLAN

Axial load (+ve compression)	N=404 kN
Moment about X-X axis	M=56.4 kNm
Shear on the base in Y direction	Fy=7.6 kN

250 x 150 x 10 RHS - Hot finished.
Properties (cm): A=74.9 rx=9.08 Zx=494 Sx=611 Ix=6170
J=6090 C=605 Zy=367 Sy=426 Iy=2760 ry=6.06

Length of baseplate	hp=450 mm
Breadth of baseplate	bp=350 mm
Edge distance to bolt centre line	k=40 mm
Assumed fillet weld size	sw=8 mm

Strength of concrete fcu=35 N/mm2
Special control must be applied over the placing of the high strength bedding material.

Assumed weld size	sw=8 mm
Selected baseplate thickness	tp=22 mm

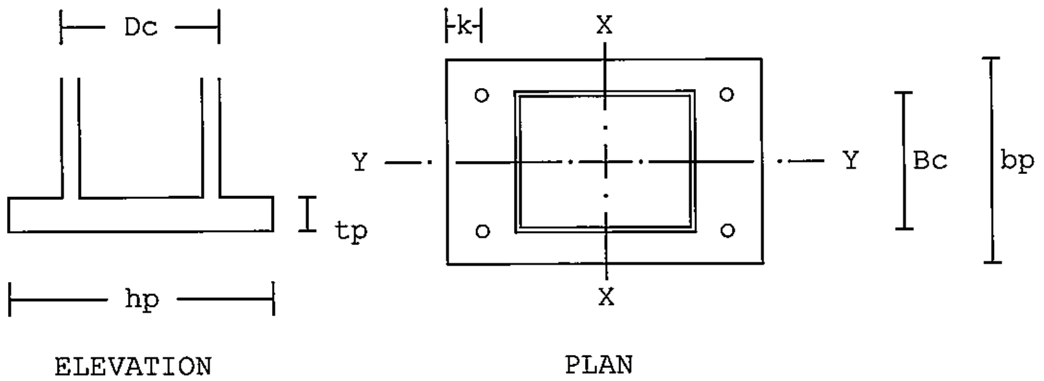
Number of bolts to be used	n=4
Bolt diameter	bd=16 mm
Selected fillet weld size	sw=8 mm

SUMMARY

BASEPLATE	Size 450 mm x 350 mm x 22 mm	
REQUIREMENTS	Grade S 275 steel	
	Edge distance	40 mm
	Number of H.D. bolts	4
	Diameter of bolts	M 16
	Grade 4.6	
	Concrete/grout (fcu)	35 N/mm^2
WELDS	Fillet weld (all round)	8 mm
	Contact areas on the baseplate and column are machined to give a tight bearing contact.	

Location: BASEPLATE TO COLUMNS C4 & C8

Calculations are in accordance with 'Joints in Steel Construction Moment Connections' published by The Steel Construction Institute.



Axial load (+ve compression)	N=105 kN
Moment about X-X axis	M=15.5 kNm
Shear on the base in Y direction	Fy=1.9 kN

150 x 100 x 10 RHS - Hot finished.
Properties (cm): A=44.9 rx=5.34 Zx=171 Sx=216 Ix=1280
J=1430 C=214 Zy=133 Sy=161 Iy=665 ry=3.85

Length of baseplate	hp=350 mm
Breadth of baseplate	bp=300 mm
Edge distance to bolt centre line	k=40 mm
Assumed fillet weld size	sw=8 mm

Strength of concrete fcu=35 N/mm2
Special control must be applied over the placing of the high strength bedding material.

Assumed weld size	sw=8 mm
Selected baseplate thickness	tp=20 mm

Number of bolts to be used	n=4
Bolt diameter	bd=16 mm
Selected fillet weld size	sw=8 mm

SUMMARY
BASEPLATE
REQUIREMENTS

Size 350 mm x 300 mm x 20 mm
Grade S 275 steel

Edge distance	40 mm
Number of H.D. bolts	4
Diameter of bolts	M 16

Grade 4.6

Concrete/grout (fcu)	35 N/mm^2

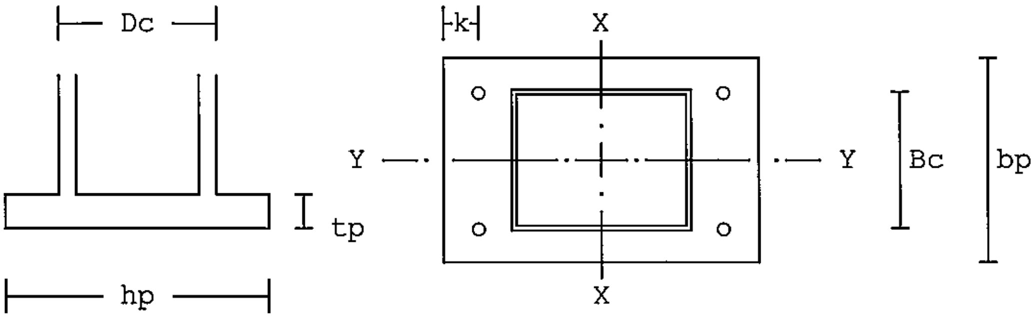
WELDS

Fillet weld (all round)	8 mm

Contact areas on the baseplate and column are machined to give a tight bearing contact.

Location: BASEPLATE TO COLUMN C7

Calculations are in accordance with 'Joints in Steel Construction Moment Connections' published by The Steel Construction Institute.



ELEVATION PLAN

Axial load (+ve compression)	N=148 kN
Moment about X-X axis	M=13.5 kNm
Shear on the base in Y direction	Fy=2.6 kN

150 x 100 x 8 RHS - Hot finished.
Properties (cm): A=36.8 rx=5.44 Zx=145 Sx=180 Ix=1090
J=1200 C=183 Zy=114 Sy=135 Iy=569 ry=3.94

Length of baseplate	hp=350 mm
Breadth of baseplate	bp=300 mm
Edge distance to bolt centre line	k=40 mm
Assumed fillet weld size	sw=8 mm

Strength of concrete fcu=35 N/mm^2
Special control must be applied over the placing of the high strength bedding material.

Assumed weld size	sw=8 mm
Selected baseplate thickness	tp=14 mm

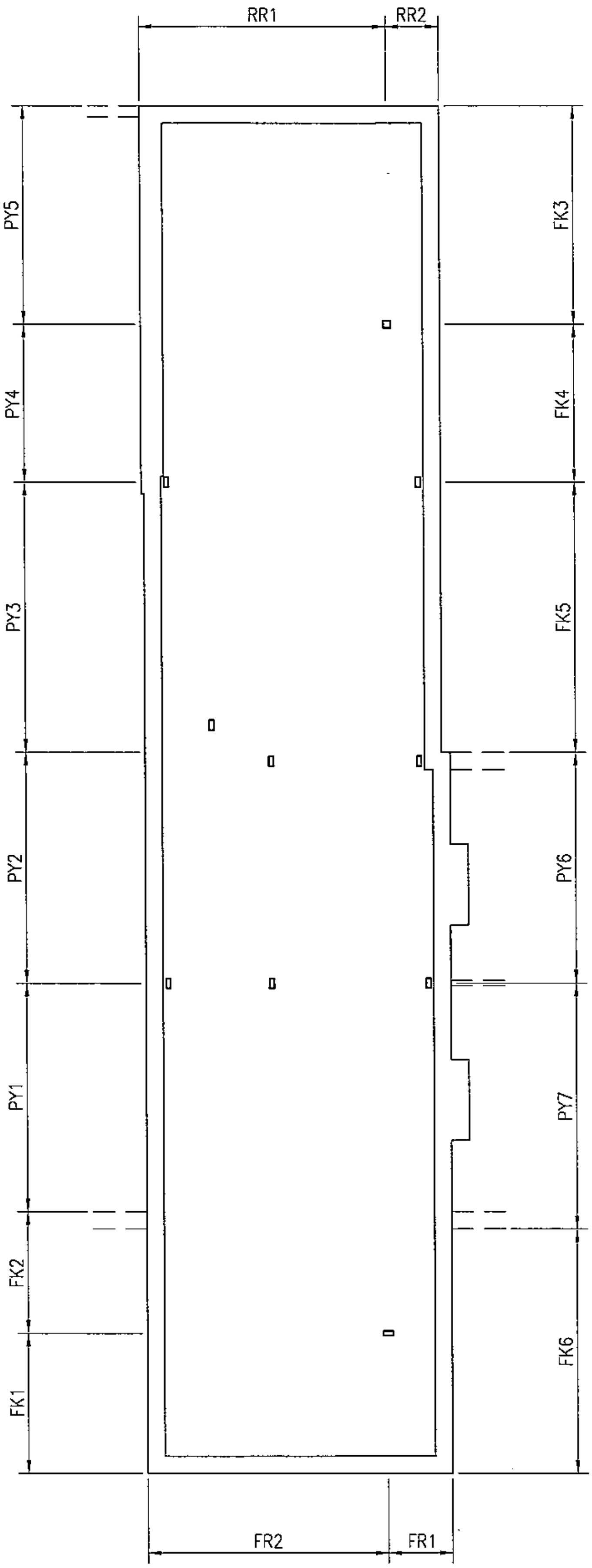
Number of bolts to be used	n=4
Bolt diameter	bd=16 mm
Selected fillet weld size	sw=8 mm

SUMMARY

BASEPLATE	Size 350 mm x 300 mm x 14 mm	
REQUIREMENTS	Grade S 275 steel	
	Edge distance	40 mm
	Number of H.D. bolts	4
	Diameter of bolts	M 16
	Grade 4.6	
	Concrete/grout (fcu)	35 N/mm^2
WELDS	Fillet weld (all round)	8 mm
	Contact areas on the baseplate and column are machined to give a tight bearing contact.	

65 GOLDHURST TERRACE, NW6 JOB. 4402

BASEMENT PAGE. BS/1



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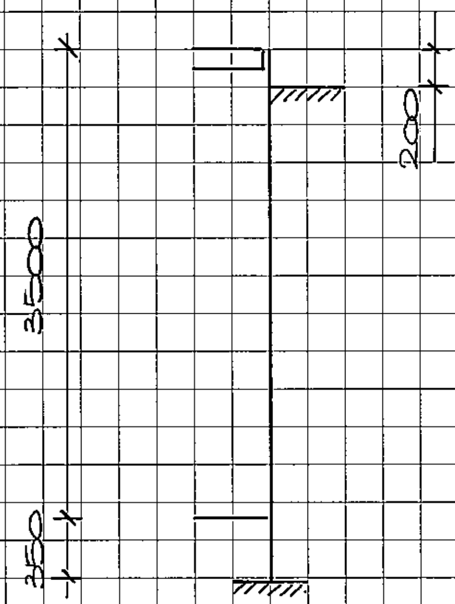
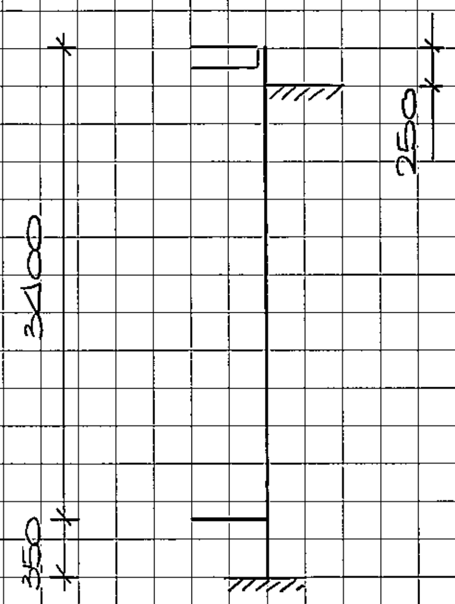
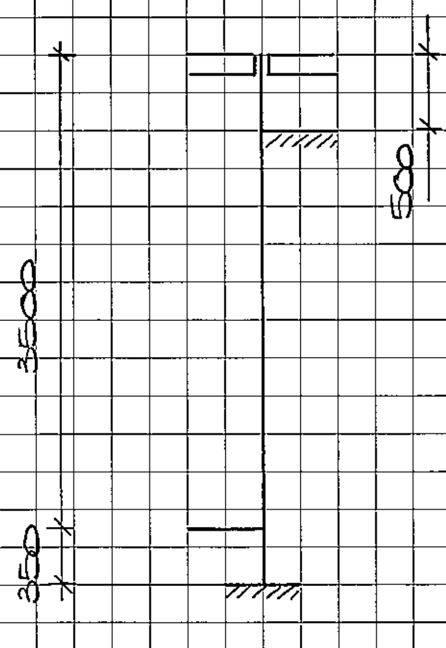
Project: 65 GOLDHURST TERRACE, NW6		Job No.: 1102	
CALCULATION SHEET	Title: BASEMENT	Date: OCT/14	
	By: SM / Checked:	Sheet No.: 85/2	Rev



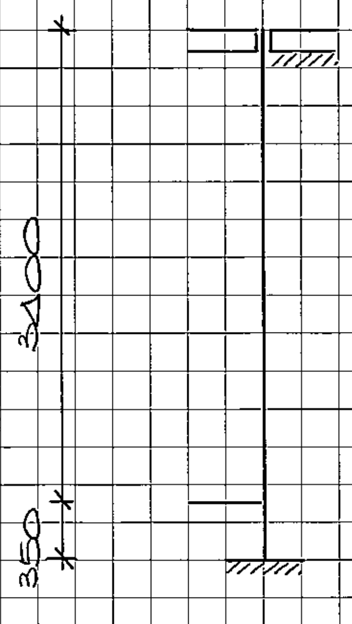
FLANK 1,2+6 AND FRONT WALLS



FLANK 3-5 AND REAR WALLS



PARTY 1,2,6+7 WALLS



PARTY 3-5 WALLS

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CALCULATION SHEET				
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FOUNDATION LINE LOADS OF EXISTING WALLS TO BE UNDERPINNED

WALL	Quantity	Unit load		Line Load		Total
		Dead	Imposed	Dead	Imposed	Load
FRONT 1, REAR 2 AND FLANK 3 & 6 WALLS - WALL SELF WEIGHT ONLY						
FRONT 2 AND FLANK 1 & 2 WALLS						
Roof slab	1.40	10.50	5.00	14.70	7.00	21.70
		TOTALS (kN/m)		**14.70**	**7.00**	**21.70**
PARTY 1 & 2 WALLS						
Pitched roof	3.60	1.20	0.65	4.32	2.34	6.66
Third floor	2.00	0.50	1.50	1.00	3.00	4.00
Second floor	2.00	0.50	1.50	1.00	3.00	4.00
First floor	2.00	0.50	1.50	1.00	3.00	4.00
Ground floor	2.00	1.00	1.50	2.00	3.00	5.00
First - eaves/roof wall	8.20	4.80	0.00	39.36	0.00	39.36
Foundation - first wall	3.70	7.20	0.00	26.64	0.00	26.64
		TOTALS (kN/m)		**75.32**	**14.34**	**89.66**
		WALL ONLY (kN/m)		**66.00**		
PARTY 3 WALL						
Flat roof	3.20	1.00	0.75	3.20	2.40	5.60
Second floor	3.10	0.50	1.50	1.55	4.65	6.20
First floor	3.10	0.50	1.50	1.55	4.65	6.20
Ground floor	2.10	6.00	1.50	12.60	3.15	15.75
Ground - eaves/roof wall	8.30	4.80	0.00	39.84	0.00	39.84
Foundation - ground wall	0.75	7.20	0.00	5.40	0.00	5.40
		TOTALS (kN/m)		**64.14**	**14.85**	**78.99**
		WALL ONLY (kN/m)		**45.24**		
PARTY 4 & FLANK 4 WALLS						
Flat roof	2.60	1.00	0.75	2.60	1.95	4.55
Ground floor	1.30	6.00	1.50	7.80	1.95	9.75
Ground - eaves/roof wall	3.00	4.80	0.00	14.40	0.00	14.40
Foundation - ground wall	0.75	7.20	0.00	5.40	0.00	5.40
		TOTALS (kN/m)		**30.20**	**3.90**	**34.10**
		WALL ONLY (kN/m)		**19.80**		
PARTY 5 WALL						
Flat roof	2.20	1.00	0.75	2.20	1.65	3.85
Ground floor	1.10	6.00	1.50	6.60	1.65	8.25
Ground - eaves/roof wall	3.00	4.80	0.00	14.40	0.00	14.40
Foundation - ground wall	0.75	7.20	0.00	5.40	0.00	5.40
		TOTALS (kN/m)		**28.60**	**3.30**	**31.90**
		WALL ONLY (kN/m)		**19.80**		

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CALCULATION SHEET			
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Title: BASEMENT		Date: OCT/14	
By: SM	Checked:	Sheet No.: BS/4	Rev:

FOUNDATION LINE LOADS OF EXISTING WALLS TO BE UNDERPINNED

WALL	Quantity	Unit load		Line Load		Total
		Dead	Imposed	Dead	Imposed	Load
REAR 1 WALL						
Flat roof	0.50	1.00	0.75	0.50	0.38	0.88
Ground floor	1.00	6.00	1.50	6.00	1.50	7.50
Ground - eaves/roof wall say	2.00	4.80	0.00	9.60	0.00	9.60
Foundation - ground wall	0.75	7.20	0.00	5.40	0.00	5.40
		TOTALS (kN/m)		**21.50**	**1.88**	**23.38**
		WALL ONLY (kN/m)		**15.00**		
FLANK 5 WALL						
Flat roof	1.00	1.00	0.75	1.00	0.75	1.75
Ground floor	1.50	6.00	1.50	9.00	2.25	11.25
Ground - eaves/roof wall	3.00	4.80	0.00	14.40	0.00	14.40
Foundation - ground wall	0.75	7.20	0.00	5.40	0.00	5.40
		TOTALS (kN/m)		**29.80**	**3.00**	**32.80**
		WALL ONLY (kN/m)		**19.80**		
PARTY 6 & 7 WALLS						
Pitched roof	3.60	1.20	0.65	4.32	2.34	6.66
Third floor	3.60	0.50	1.50	1.80	5.40	7.20
Second floor	3.60	0.50	1.50	1.80	5.40	7.20
First floor	3.60	0.50	1.50	1.80	5.40	7.20
Ground floor	3.60	1.00	1.50	3.60	5.40	9.00
First - eaves/roof wall	8.20	4.80	0.00	39.36	0.00	39.36
Foundation - first wall	3.70	7.20	0.00	26.64	0.00	26.64
		TOTALS (kN/m)		**79.32**	**23.94**	**103.26**
		WALL ONLY (kN/m)		**66.00**		

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MOMENT DUE TO RETAINED SOIL AND WATER - FRONT AND FLANK 1, 2 & 6 WALLS

London Clay density = 18kN/m^3 and angle of internal friction = 30 °

Hence Ka =	0.333	m	and Dd = dry density
Retained depth (Hr) =	3.65	m	Ds = saturated density
Depth of water (Hw) =	2.80	m	Dw = density of water
Surcharge (W) =	5.00	kN/m^2	

Now calculate the maximum pressures from the retained material;

	At u/s base			At top of base
Pressure due to dry soil, P1 =	5.10	kN/m^2	= Ka x Dd x (Hr - Hw)	5.10
Pressure due to dry soil surcharge, P2 =	5.10	kN/m^2	= Ka x Dd x (Hr - Hw)	5.10
Pressure due to submerged soil, P3 =	7.64	kN/m^2	= Ka x Ds x Hw	6.69
Pressure due to water, P4 =	27.47	kN/m^2	= Dw x Hw	24.03
Pressure due to surcharge, P5 =	1.67	kN/m^2	= Ka x W	1.67

Hence the forces acting on the wall due to the retained pressures are;

Force due to dry soil, F1 =	2.17	kN	= P1 x (Hr - Hw) x 0.5	2.17
Force due to dry soil surcharge, F2 =	14.28	kN	= P2 x Hw	12.50
Force due to submerged soil, F3 =	10.70	kN	= P3 x Hw x 0.5	8.19
Force due to water, F4 =	38.46	kN	= P4 x Hw x 0.5	29.44
Force due to surcharge, F5 =	6.08	kN	= P5 x Hr	5.50

and the overturning moments due to the forces acting on the wall are;

OTM due to dry soil, M1 =	6.68	kN.m	= F1 x (Hw + (Hr - Hw)/3)	5.92
OTM due to dry soil surcharge, M2 =	19.99	kN.m	= F2 x Hw x 0.5	15.31
OTM due to submerged soil, M3 =	9.99	kN.m	= F3 x Hw / 3	6.69
OTM due to water, M4 =	35.89	kN.m	= F4 x Hw / 3	24.04
OTM due to surcharge, M5 =	11.10	kN.m	= F5 x Hr x 0.5	9.08

Therefore, total force due to retained soil and water =	71.69	kN	57.80	kN
and total overturning moment due to retained soil and water =	83.66	kN.m	61.04	kN.m

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MOMENT DUE TO RETAINED SOIL AND WATER - PARTY 1, 2, 6 & 7 WALLS

London Clay density = 18kN/m^3 and angle of internal friction = 30 °

Hence Ka =	0.333	m	and Dd = dry density
Retained depth (Hr) =	3.35	m	Ds = saturated density
Depth of water (Hw) =	2.80	m	Dw = density of water
Surcharge (W) =	1.50	kN/m^2	

Now calculate the maximum pressures from the retained material;

	At u/s base			At top of base
Pressure due to dry soil, P1 =	3.30	kN/m^2	= Ka x Dd x (Hr - Hw)	3.30
Pressure due to dry soil surcharge, P2 =	3.30	kN/m^2	= Ka x Dd x (Hr - Hw)	3.30
Pressure due to submerged soil, P3 =	7.64	kN/m^2	= Ka x Ds x Hw	6.69
Pressure due to water, P4 =	27.47	kN/m^2	= Dw x Hw	24.03
Pressure due to surcharge, P5 =	0.50	kN/m^2	= Ka x W	0.50

Hence the forces acting on the wall due to the retained pressures are;

Force due to dry soil, F1 =	0.91	kN	= P1 x (Hr - Hw) x 0.5	0.91
Force due to dry soil surcharge, F2 =	9.24	kN	= P2 x Hw	8.09
Force due to submerged soil, F3 =	10.70	kN	= P3 x Hw x 0.5	8.19
Force due to water, F4 =	38.46	kN	= P4 x Hw x 0.5	29.44
Force due to surcharge, F5 =	1.68	kN	= P5 x Hr	1.50

and the overturning moments due to the forces acting on the wall are;

OTM due to dry soil, M1 =	2.71	kN.m	= F1 x (Hw + (Hr - Hw)/3)	2.39
OTM due to dry soil surcharge, M2 =	12.94	kN.m	= F2 x Hw x 0.5	9.90
OTM due to submerged soil, M3 =	9.99	kN.m	= F3 x Hw / 3	6.69
OTM due to water, M4 =	35.89	kN.m	= F4 x Hw / 3	24.04
OTM due to surcharge, M5 =	2.81	kN.m	= F5 x Hr x 0.5	2.25

Therefore, total force due to retained soil and water =	60.98	kN	48.13	kN
and total overturning moment due to retained soil and water =	64.33	kN.m	45.28	kN.m

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MOMENT DUE TO RETAINED SOIL AND WATER - PARTY 3-5 WALLS

London Clay density = 18kN/m^3 and angle of internal friction = 30 °

Hence Ka =	0.333	m	and Dd = dry density
Retained depth (Hr) =	3.75	m	Ds = saturated density
Depth of water (Hw) =	2.80	m	Dw = density of water
Surcharge (W) =	1.50	kN/m^2	

Now calculate the maximum pressures from the retained material;

	At u/s base			At top of base
Pressure due to dry soil, P1 =	5.70	kN/m^2	= Ka x Dd x (Hr - Hw)	5.70
Pressure due to dry soil surcharge, P2 =	5.70	kN/m^2	= Ka x Dd x (Hr - Hw)	5.70
Pressure due to submerged soil, P3 =	7.64	kN/m^2	= Ka x Ds x Hw	6.69
Pressure due to water, P4 =	27.47	kN/m^2	= Dw x Hw	24.03
Pressure due to surcharge, P5 =	0.50	kN/m^2	= Ka x W	0.50

Hence the forces acting on the wall due to the retained pressures are;

Force due to dry soil, F1 =	2.71	kN	= P1 x (Hr - Hw) x 0.5	2.71
Force due to dry soil surcharge, F2 =	15.96	kN	= P2 x Hw	13.97
Force due to submerged soil, F3 =	10.70	kN	= P3 x Hw x 0.5	8.19
Force due to water, F4 =	38.46	kN	= P4 x Hw x 0.5	29.44
Force due to surcharge, F5 =	1.88	kN	= P5 x Hr	1.70

and the overturning moments due to the forces acting on the wall are;

OTM due to dry soil, M1 =	8.44	kN.m	= F1 x (Hw + (Hr - Hw)/3)	7.49
OTM due to dry soil surcharge, M2 =	22.34	kN.m	= F2 x Hw x 0.5	17.11
OTM due to submerged soil, M3 =	9.99	kN.m	= F3 x Hw / 3	6.69
OTM due to water, M4 =	35.89	kN.m	= F4 x Hw / 3	24.04
OTM due to surcharge, M5 =	3.52	kN.m	= F5 x Hr x 0.5	2.89

Therefore, total force due to retained soil and water =	69.70	kN	56.01	kN
and total overturning moment due to retained soil and water =	80.18	kN.m	58.22	kN.m

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MOMENT DUE TO RETAINED SOIL AND WATER - REAR AND FLANK 3-5 WALLS

London Clay density = 18kN/m^3 and angle of internal friction = 30 °

Hence Ka =	0.333	m	and Dd = dry density
Retained depth (Hr) =	3.50	m	Ds = saturated density
Depth of water (Hw) =	2.80	m	Dw = density of water
Surcharge (W) =	1.50	kN/m^2	

Now calculate the maximum pressures from the retained material;

	At u/s base			At top of base
Pressure due to dry soil, P1 =	4.20	kN/m^2	= Ka x Dd x (Hr - Hw)	4.20
Pressure due to dry soil surcharge, P2 =	4.20	kN/m^2	= Ka x Dd x (Hr - Hw)	4.20
Pressure due to submerged soil, P3 =	7.64	kN/m^2	= Ka x Ds x Hw	6.69
Pressure due to water, P4 =	27.47	kN/m^2	= Dw x Hw	24.03
Pressure due to surcharge, P5 =	0.50	kN/m^2	= Ka x W	0.50

Hence the forces acting on the wall due to the retained pressures are;

Force due to dry soil, F1 =	1.47	kN	= P1 x (Hr - Hw) x 0.5	1.47
Force due to dry soil surcharge, F2 =	11.76	kN	= P2 x Hw	10.29
Force due to submerged soil, F3 =	10.70	kN	= P3 x Hw x 0.5	8.19
Force due to water, F4 =	38.46	kN	= P4 x Hw x 0.5	29.44
Force due to surcharge, F5 =	1.75	kN	= P5 x Hr	1.58

and the overturning moments due to the forces acting on the wall are;

OTM due to dry soil, M1 =	4.46	kN.m	= F1 x (Hw + (Hr - Hw)/3)	3.94
OTM due to dry soil surcharge, M2 =	16.46	kN.m	= F2 x Hw x 0.5	12.61
OTM due to submerged soil, M3 =	9.99	kN.m	= F3 x Hw / 3	6.69
OTM due to water, M4 =	35.89	kN.m	= F4 x Hw / 3	24.04
OTM due to surcharge, M5 =	3.06	kN.m	= F5 x Hr x 0.5	2.48

Therefore, total force due to retained soil and water =	64.14	kN	50.97	kN
and total overturning moment due to retained soil and water =	69.87	kN.m	49.77	kN.m

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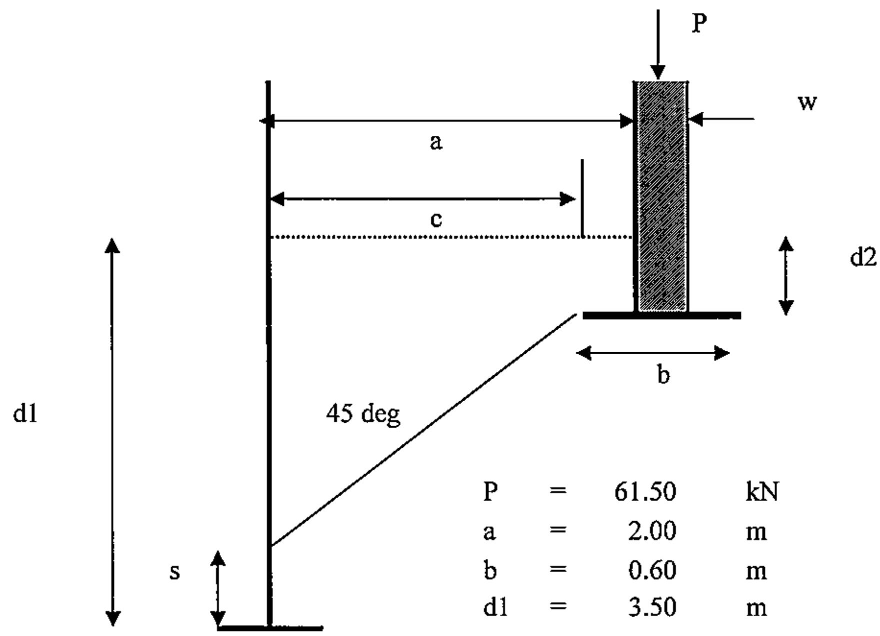
	Title: BASEMENT		Date OCT/14
	By SM	Checked	Sheet No. BS/9 Rev

OVERTURNING MOMENTS FROM ADJACENT WALL

FLANK 5 WALL



P	=	61.50	kN
a	=	2.00	m
b	=	0.60	m
d1	=	3.50	m
d2	=	0.60	m
w	=	0.33	m
k	=	0.333	
Soil density	=	18.00	kN/m3

	U/S OF BASE		TOP OF BASE	
c = edge of adjacent footing to wall =	1.87	m	1.87	m
Width of load spread at strike level =	4.33	m	4.33	m
s = Height of strike above base of wall =	1.04	m	0.69	m
Vertical surcharge pressure at strike level =	14.20	kN/m2	14.20	kN/m2
Horizontal surcharge pressure at strike level =	4.73	kN/m2	4.73	kN/m2
Horizontal force =	4.90	kN	3.24	kN
Lever arm =	0.52	m	0.34	m
OTM =	2.54	kNm/m	1.11	kNm/m

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CALCULATION SHEET				
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By	SM	Checked	Sheet No.	BS/10	Rev

MOMENT DUE TO HIGHWAY LOADS - FRONT WALLS

Surcharge loads from an adjoining highway are either 10 kN/m^2 or a 40 kN wheel load. The wheel load is assumed to be applied 0.6m from the property boundary while the highway UDL is assumed to apply up to the boundary.

Enter >	Distance from back of retaining wall to pavement boundary =	**2.5**	m
	Depth of retaining wall below ground level =	**3.65**	m
	Angle of internal friction =	**30**	°
	Point Load from road =	**40**	kN
	UDL from road =	**10**	kN/m^2

Highway PL strikes wall at 0.70 m above base of wall
Highway UDL strikes wall at 1.15 m above base of wall

Highway PL reduces to 1.04 kN/m^2 at 0.70 m above base of wall
and reduces further to 0.76 kN/m^2 at base of wall

Ka = 0.333   Soil density = 18 kN/m^3

HIGHWAY POINT LOAD

Pressure on wall at 0.70 m above base = 0.35 kN/m^2
and at base of wall = 0.25 kN/m^2
Force on wall = 0.21 kN

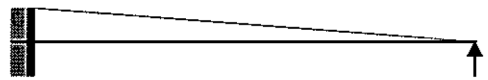
HENCE OTM ON WALL = 0.02 kN.m ***(Top of Toe)***
HENCE OTM ON WALL = 0.08 kN.m ***(U/S of Toe)***

HIGHWAY UDL

Pressure on wall at 1.15 m above base = 3.33 kN/m^2
Force on wall = 3.83 kN

HENCE OTM ON WALL = 1.07 kN.m ***(Top of Toe)***
HENCE OTM ON WALL = 2.20 kN.m ***(U/S of Toe)***

REACTIONS & MOMENTS FOR A PROPPED CANTILEVER WITH A TRIANGULAR LOAD

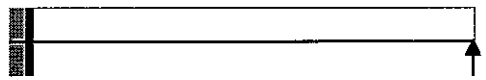


ENTER	W =	65.61	kN (Total load)
	L =	3.65	m (Span)
	Hence X =	1.63	m (Location of maximum moment from pinned support)

SUPPORT A IS THE FIXED SUPPORT, SUPPORT B IS THE PIN

MA =	**-31.93**	**kN.m**	**RA =**	**52.49**	**kN**
M Span =	**14.27**	**kN.m**	**RB =**	**13.12**	**kN**

REACTIONS & MOMENTS FOR A PROPPED CANTILEVER WITH A UDL



ENTER	W =	8.28	kN (Total load)
	L =	3.65	m (Span)
	Hence X =	1.37	m (Location of maximum moment from pinned support)

SUPPORT A IS THE FIXED SUPPORT, SUPPORT B IS THE PIN

MA =	**-3.78**	**kN.m**	**RA =**	**5.18**	**kN**
M Span =	**2.12**	**kN.m**	**RB =**	**3.11**	**kN**

FRONT 2 AND FLANK 1 & 2 WALLS

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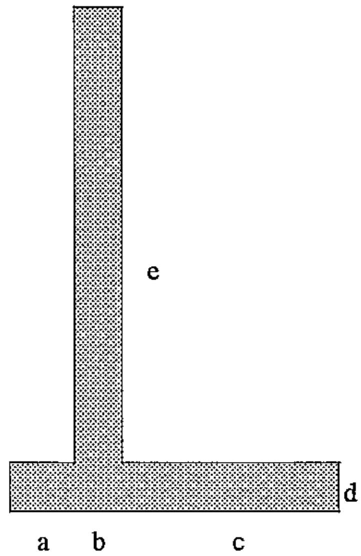
CALCULATION SHEET	Project: 65 GOLDHURST TERRACE, NW6		Job No.: 4402	
	Title: BASEMENT		Date: OCT/14	
	By: SM	Checked	Sheet No.: BS/12	Rev

ECCENTRIC BASE DESIGN - FRONT 1 & FLANK 6 WALLS

Enter the following:-			
Dim. a =	0.000	m	
Dim b =	0.350	m	
Dim c1 =	0.000	m	
Dim c2 =	4.300	m	
Dim c3 =	4.300	m	
Dim d =	0.350	m	
Dim e =	3.300	m	
OTM =	85.90	kN.m	
Load 1 =	0.00	kN/m - maximum vertical load	
Load 2 =	0.00	kN/m - self weight of wall	

Note
Case 1 = maximum load from above, no OTM
Case 2 = Case 1 with OTM added
Case 3 = self weight of wall above with OTM

Take moments about the toe	Case 1	Case 2	Case 3
Retaining wall, stem weight =	27.72	27.72	27.72
Retaining wall, base weight =	2.94	39.06	39.06
Lever arm stem =	0.175	4.475	4.475
Lever arm base =	0.175	2.325	2.325
Lever arm vertical load =	0.175	4.475	4.475
Restoring moment =	5.37	214.86	214.86
Applied OTM =	0.00	-85.90	-85.90
Total vertical load =	**30.66**	**66.78**	**66.78**
Net total moment =	**5.37**	**128.96**	**128.96**



	Case 1	Case 2	Case 3	
Distance to load centroid =	0.175	1.931	1.931	m
Hence, eccentricity =	0.000	0.394	0.394	m
W/A =	87.60	14.36	14.36	kN/m^2
M/Z =	0.00	7.30	7.30	kN/m^2
Hence, max. pressure =	**87.60**	**21.66**	**21.66**	kN/m^2
and min. pressure =	87.60	7.06	7.06	kN/m^2
FoS v overturning =	**N/A**	**2.5**	**2.5**	

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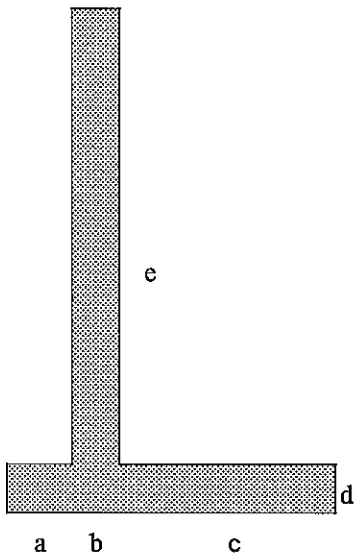
CALCULATION SHEET	Project: 65 GOLDHURST TERRACE, NW6		Job No. 4402	
	Title: BASEMENT		Date: OCT/14	
	By: SM	Checked	Sheet No. BS/13	Rev

ECCENTRIC BASE DESIGN - FRONT 2 AND FLANK 1 & 2 WALLS

Enter the following:-			
	Dim. a =	**0.100**	m
	Dim b =	**0.350**	m
	Dim c1 =	**0.100**	m
	Dim c2 =	**1.500**	m
	Dim c3 =	**1.900**	m
	Dim d =	**0.350**	m
	Dim e =	**2.750**	m
	OTM =	**35.70**	kN.m
	Load 1 =	**21.70**	kN/m - maximum vertical load
	Load 2 =	**8.40**	kN/m - self weight of wall

Note
Case 1 = maximum load from above, no OTM
Case 2 = Case 1 with OTM added
Case 3 = self weight of wall above with OTM

Take moments about the toe	Case 1	Case 2	Case 3
Retaining wall, stem weight =	23.10	23.10	23.10
Retaining wall, base weight =	4.62	16.38	19.74
Lever arm stem =	0.275	1.675	2.075
Lever arm base =	0.275	0.975	1.175
Lever arm vertical load =	0.275	1.675	2.075
Restoring moment =	13.59	91.01	88.56
Applied OTM =	0.00	-35.70	-35.70
Total vertical load =	**49.42**	**61.18**	**51.24**
Net total moment =	**13.59**	**55.31**	**52.86**



	Case 1	Case 2	Case 3	
Distance to load centroid =	0.275	0.904	1.032	m
Hence, eccentricity =	0.000	0.071	0.143	m
W/A =	89.85	31.37	21.80	kN/m^2
M/Z =	0.00	6.85	7.99	kN/m^2
Hence, max. pressure =	**89.85**	**38.22**	**29.79**	kN/m^2
and min. pressure =	89.85	24.53	13.82	kN/m^2
FoS v overturning =	**N/A**	**2.5**	**2.5**	

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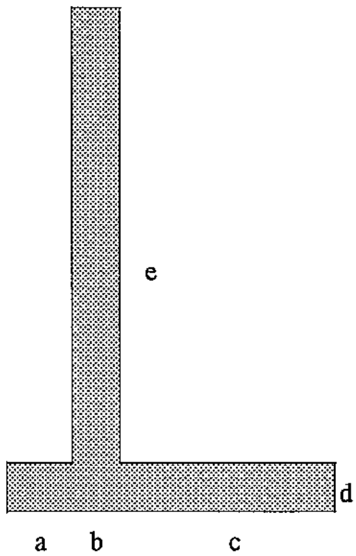
CALCULATION SHEET				
Project: 65 GOLDHURST TERRACE, NW6		Job No. 4402		
Title: BASEMENT		Date: OCT/14		
By: SM	Checked:	Sheet No. BS/14	Rev:	

ECCENTRIC BASE DESIGN - PARTY 2, 3, 6 & 7 WALLS

Enter the following:-			
	Dim. a =	0.350	m
	Dim b =	0.350	m
	Dim c1 =	0.350	m
	Dim c2 =	1.050	m
	Dim c3 =	1.400	m
	Dim d =	0.350	m
	Dim e =	2.750	m
	OTM =	64.30	kN.m
	Load 1 =	96.50	kN/m - maximum vertical load
	Load 2 =	66.00	kN/m - self weight of wall

Note
Case 1 = maximum load from above, no OTM
Case 2 = Case 1 with OTM added
Case 3 = self weight of wall above with OTM

Take moments about the toe	Case 1	Case 2	Case 3
Retaining wall, stem weight =	23.10	23.10	23.10
Retaining wall, base weight =	8.82	14.70	17.64
Lever arm stem =	0.525	1.225	1.575
Lever arm base =	0.525	0.875	1.050
Lever arm vertical load =	0.525	1.225	1.575
Restoring moment =	67.42	159.37	158.85
Applied OTM =	0.00	-64.30	-64.30
Total vertical load =	**128.42**	**134.30**	**106.74**
Net total moment =	**67.42**	**95.07**	**94.55**



	Case 1	Case 2	Case 3	
Distance to load centroid =	0.525	0.708	0.886	m
Hence, eccentricity =	0.000	0.167	0.164	m
W/A =	122.30	76.74	50.83	kN/m^2
M/Z =	0.00	43.96	23.84	kN/m^2
Hence, max. pressure =	**122.30**	**120.71**	**74.67**	**kN/m^2**
and min. pressure =	122.30	32.78	26.99	kN/m^2
FoS v overturning =	**N/A**	**2.5**	**2.5**	

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CALCULATION SHEET	Project: 65 GOLDHURST TERRACE, NW6		Job No. 4402	
	Title: BASEMENT		Date: OCT/14	
	By: SM	Checked	Sheet No. BS/15	Rev

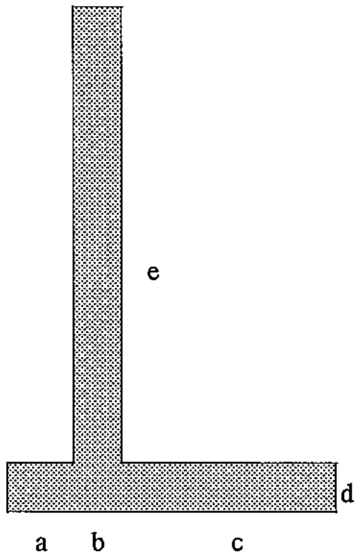
ECCENTRIC BASE DESIGN - PARTY 3 WALL

Enter the following:-

Dim. a =	**0.300**	m
Dim b =	**0.350**	m
Dim c1 =	**0.300**	m
Dim c2 =	**1.600**	m
Dim c3 =	**2.250**	m
Dim d =	**0.350**	m
Dim e =	**2.750**	m
OTM =	**80.20**	kN.m
Load 1 =	**79.00**	kN/m - maximum vertical load
Load 2 =	**45.20**	kN/m - self weight of wall

Note
Case 1 = maximum load from above, no OTM
Case 2 = Case 1 with OTM added
Case 3 = self weight of wall above with OTM

Take moments about the toe	Case 1	Case 2	Case 3
Retaining wall, stem weight =	23.10	23.10	23.10
Retaining wall, base weight =	7.98	18.90	24.36
Lever arm stem =	0.475	1.775	2.425
Lever arm base =	0.475	1.125	1.450
Lever arm vertical load =	0.475	1.775	2.425
Restoring moment =	52.29	202.49	200.95
Applied OTM =	0.00	-80.20	-80.20
Total vertical load =	**110.08**	**121.00**	**92.66**
Net total moment =	**52.29**	**122.29**	**120.75**



	Case 1	Case 2	Case 3	
Distance to load centroid =	0.475	1.011	1.303	m
Hence, eccentricity =	0.000	0.114	0.147	m
W/A =	115.87	53.78	31.95	kN/m^2
M/Z =	0.00	16.40	9.71	kN/m^2
Hence, max. pressure =	**115.87**	**70.17**	**41.66**	**kN/m^2**
and min. pressure =	115.87	37.38	22.24	kN/m^2
FoS v overturning =	**N/A**	**2.5**	**2.5**	

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	Title: BASEMENT		Date: OCT/14	
	By: SM	Checked:	Sheet No. BS/16	Rev:

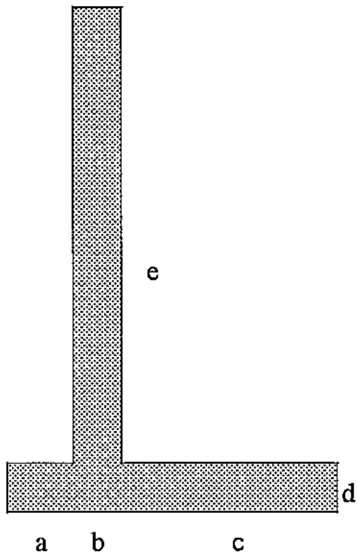
ECCENTRIC BASE DESIGN - PARTY 4 & 5 WALLS

Enter the following:-

Dim. a =	0.100	m
Dim b =	0.350	m
Dim c1 =	0.100	m
Dim c2 =	2.650	m
Dim c3 =	3.200	m
Dim d =	0.350	m
Dim e =	2.750	m
OTM =	80.20	kN.m
Load 1 =	34.10	kN/m - maximum vertical load
Load 2 =	19.80	kN/m - self weight of wall

Note
Case 1 = maximum load from above, no OTM
Case 2 = Case 1 with OTM added
Case 3 = self weight of wall above with OTM

Take moments about the toe	Case 1	Case 2	Case 3
Retaining wall, stem weight =	23.10	23.10	23.10
Retaining wall, base weight =	4.62	26.04	30.66
Lever arm stem =	0.275	2.825	3.375
Lever arm base =	0.275	1.550	1.825
Lever arm vertical load =	0.275	2.825	3.375
Restoring moment =	17.00	201.95	200.74
Applied OTM =	0.00	-80.20	-80.20
Total vertical load =	**61.82**	**83.24**	**73.56**
Net total moment =	**17.00**	**121.75**	**120.54**



	Case 1	Case 2	Case 3	
Distance to load centroid =	0.275	1.463	1.639	m
Hence, eccentricity =	0.000	0.087	0.186	m
W/A =	112.40	26.85	20.15	kN/m^2
M/Z =	0.00	4.54	6.17	kN/m^2
Hence, max. pressure =	**112.40**	**31.39**	**26.33**	**kN/m^2**
and min. pressure =	112.40	22.31	13.98	kN/m^2
FoS v overturning =	**N/A**	**2.5**	**2.5**	

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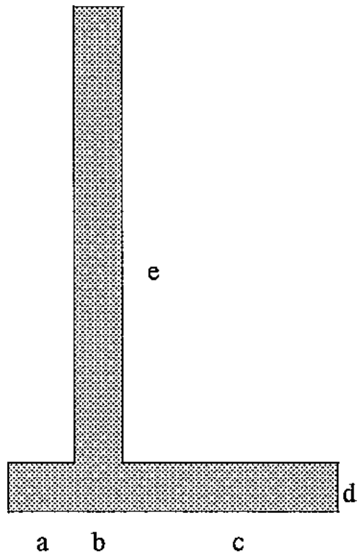
Project	65 GOLDHURST TERRACE, NW6		Job No.	4402
Title	BASEMENT		Date	OCT/14
By	SM	Checked	Sheet No.	BS/17	Rev

ECCENTRIC BASE DESIGN - REAR 1 AND FLANK 4 & 5 WALLS

Enter the following:-			
Dim. a =	**0.100**	m	
Dim b =	**0.350**	m	
Dim c1 =	**0.100**	m	
Dim c2 =	**2.400**	m	
Dim c3 =	**3.200**	m	
Dim d =	**0.350**	m	
Dim e =	**2.750**	m	
OTM =	**72.40**	kN.m	
Load 1 =	**34.10**	kN/m - maximum vertical load	
Load 2 =	**15.00**	kN/m - self weight of wall	

Note
Case 1 = maximum load from above, no OTM
Case 2 = Case 1 with OTM added
Case 3 = self weight of wall above with OTM

Take moments about the toe	**Case 1**	**Case 2**	**Case 3**
Retaining wall, stem weight =	23.10	23.10	23.10
Retaining wall, base weight =	4.62	23.94	30.66
Lever arm stem =	0.275	2.575	3.375
Lever arm base =	0.275	1.425	1.825
Lever arm vertical load =	0.275	2.575	3.375
Restoring moment =	17.00	181.40	184.54
Applied OTM =	0.00	-72.40	-72.40
Total vertical load =	**61.82**	**81.14**	**68.76**
Net total moment =	**17.00**	**109.00**	**112.14**



	Case 1	**Case 2**	**Case 3**	
Distance to load centroid =	0.275	1.343	1.631	m
Hence, eccentricity =	0.000	0.082	0.194	m
W/A =	112.40	28.47	18.84	kN/m^2
M/Z =	0.00	4.89	6.01	kN/m^2
Hence, max. pressure =	**112.40**	**33.36**	**24.85**	kN/m^2
and min. pressure =	112.40	23.58	12.83	kN/m^2
FoS v overturning =	**N/A**	**2.5**	**2.5**	

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CALCULATION SHEET

Project: 65 GOLDHURST TERRACE, NW6		Job No.: 4402	
Title: BASEMENT		Date: OCT/14	
By: SM	Checked:	Sheet No.: BS/18	Rev:

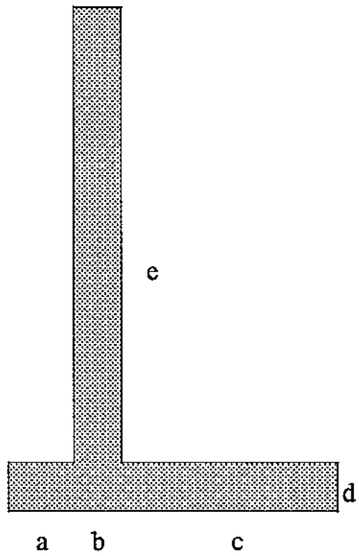
ECCENTRIC BASE DESIGN - REAR 2 & FLANK 3 WALLS

Enter the following:-

Dim. a =	**0.000**	m
Dim b =	**0.350**	m
Dim c1 =	**0.000**	m
Dim c2 =	**3.750**	m
Dim c3 =	**3.750**	m
Dim d =	**0.350**	m
Dim e =	**3.150**	m
OTM =	**69.90**	kN.m
Load 1 =	**0.00**	kN/m - maximum vertical load
Load 2 =	**0.00**	kN/m - self weight of wall

Note
Case 1 = maximum load from above, no OTM
Case 2 = Case 1 with OTM added
Case 3 = self weight of wall above with OTM

Take moments about the toe	Case 1	Case 2	Case 3
Retaining wall, stem weight =	26.46	26.46	26.46
Retaining wall, base weight =	2.94	34.44	34.44
Lever arm stem =	0.175	3.925	3.925
Lever arm base =	0.175	2.050	2.050
Lever arm vertical load =	0.175	3.925	3.925
Restoring moment =	5.15	174.46	174.46
Applied OTM =	0.00	-69.90	-69.90
Total vertical load =	**29.40**	**60.90**	**60.90**
Net total moment =	**5.15**	**104.56**	**104.56**



	Case 1	Case 2	Case 3	
Distance to load centroid =	0.175	1.717	1.717	m
Hence, eccentricity =	0.000	0.333	0.333	m
W/A =	84.00	14.85	14.85	kN/m^2
M/Z =	0.00	7.24	7.24	kN/m^2
Hence, max. pressure =	**84.00**	**22.09**	**22.09**	kN/m^2
and min. pressure =	84.00	7.61	7.61	kN/m^2
FoS v overturning =	**N/A**	**2.5**	**2.5**	

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CALCULATION SHEET				
	Project: 65 GOLDHURST TERRACE, NW6		Job No.: 4402	
	Title: BASEMENT		Date: OCT/14	
	By: SM	Checked:	Sheet No.: BS/19	Rev:

ECCENTRIC BASE DESIGN WITH COLUMN C9 - PARTY 3 & 4 WALLS

Enter the following:-		
Dim. a =	**0.550**	m
Dim b =	**0.350**	m
Dim c1 =	**1.200**	m
Dim c2 =	**1.700**	m
Dim c3 =	**1.700**	m
Dim d =	**0.350**	m
Dim e =	**2.750**	m
Dim f =	**0.265**	m
OTM =	**80.20**	kN.m
Load 1 =	**39.50**	kN/m - maximum vertical load
Load 2 =	**22.60**	kN/m - self weight of wall

Note

Case 1 = maximum load from above, no OTM

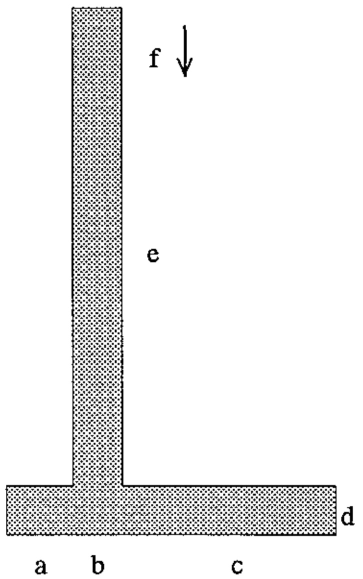
Case 2 = Case 1 with OTM added

Case 3 = self weight of wall above with OTM

Now enter load from pier/post = **309.30** kN

and enter o/a length of base = **1.7** m

Take moments about the toe	Case 1	Case 2	Case 3
Retaining wall, stem weight =	39.27	39.27	39.27
Retaining wall, base weight =	29.99	37.13	37.13
Lever arm stem =	1.375	1.875	1.875
Lever arm base =	1.050	1.300	1.300
Lever arm vertical load =	1.375	1.875	1.875
Lever arm post/pier load =	0.935	1.435	1.435
Restoring moment =	467.01	691.65	637.78
Applied OTM =	0.00	-136.34	-136.34
Total vertical load =	**445.71**	**452.85**	**424.12**
Net total moment =	**467.01**	**555.31**	**501.44**



	Case 1	Case 2	Case 3	
Distance to load centroid =	1.048	1.226	1.182	m
Hence, eccentricity =	0.002	0.074	0.118	m
W/A =	124.85	102.45	95.95	kN/m^2
M/Z =	0.79	17.43	26.06	kN/m^2
Hence, max. pressure =	**125.63**	**119.89**	**122.01**	kN/m^2
and min. pressure =	124.06	85.02	69.89	kN/m^2
FoS v overturning =	**N/A**	**5.1**	**4.7**	

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CALCULATION SHEET	Project: 65 GOLDHURST TERRACE, NW6		Job No.: 4402	
	Title: BASEMENT		Date: OCT/14	
	By: SM	Checked:	Sheet No.: BS/20	Rev:

ECCENTRIC BASE DESIGN WITH COLUMN C10 - FLANK 4 & 5 WALLS

Enter the following:-			
	Dim. a =	**0.250**	m
	Dim b =	**0.350**	m
	Dim c1 =	**0.550**	m
	Dim c2 =	**1.250**	m
	Dim c3 =	**1.400**	m
	Dim d =	**0.350**	m
	Dim e =	**2.750**	m
	Dim f =	**0.100**	m
	OTM =	**71.00**	kN.m
	Load 1 =	**33.45**	kN/m - maximum vertical load
	Load 2 =	**19.80**	kN/m - self weight of wall

Note

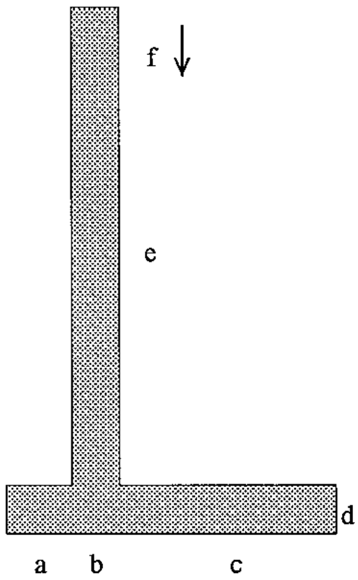
Case 1 = maximum load from above, no OTM

Case 2 = Case 1 with OTM added

Case 3 = self weight of wall above with OTM

Now enter load from pier/post =	**120.30**	kN
and enter o/a length of base =	**1.7**	m

Take moments about the toe	Case 1	Case 2	Case 3
Retaining wall, stem weight =	39.27	39.27	39.27
Retaining wall, base weight =	16.42	26.42	28.56
Lever arm stem =	0.725	1.425	1.575
Lever arm base =	0.575	0.925	1.000
Lever arm vertical load =	0.725	1.425	1.575
Lever arm post/pier load =	0.450	1.150	1.300
Restoring moment =	133.28	299.77	299.81
Applied OTM =	0.00	-120.70	-120.70
Total vertical load =	**232.86**	**242.85**	**221.79**
Net total moment =	**133.28**	**179.07**	**179.11**



	Case 1	Case 2	Case 3	
Distance to load centroid =	0.572	0.737	0.808	m
Hence, eccentricity =	0.003	0.188	0.192	m
W/A =	119.11	77.22	65.23	kN/m^2
M/Z =	1.65	46.99	37.65	kN/m^2
Hence, max. pressure =	**120.76**	**124.21**	**102.89**	kN/m^2
and min. pressure =	117.46	30.23	27.58	kN/m^2
FoS v overturning =	**N/A**	**2.5**	**2.5**	

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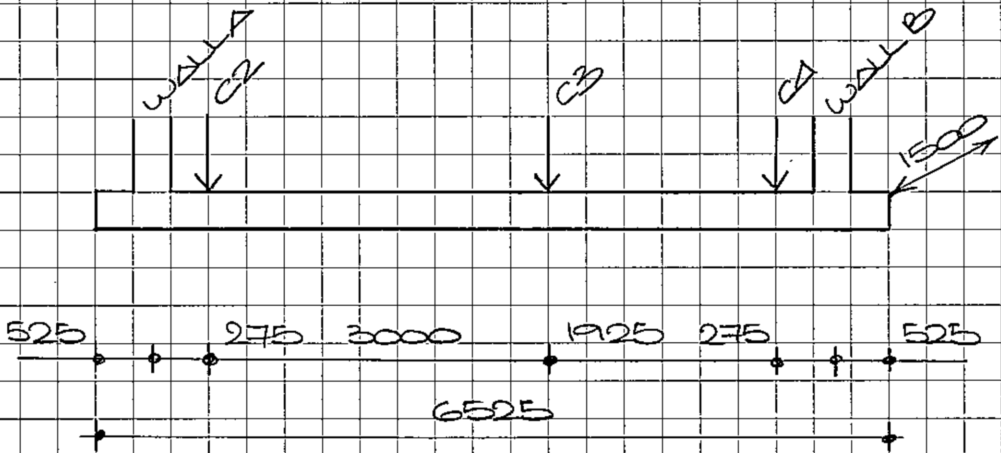
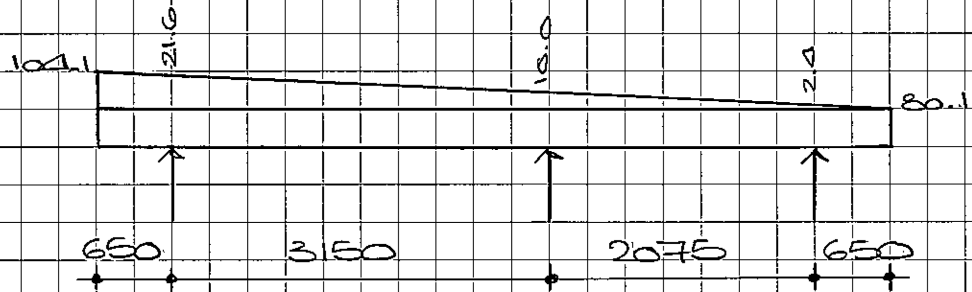
Consulting civil and Structural Engineers

Project: 65 GOLDHURST TERRACE, NW6		Job No. 4402
CALCULATION SHEET	Title: BASEMENT	Date: OCT/14
	By: SM — Checked:	Sheet No. BS/21 — Rev:

COMBINED FOUNDATION TYPE X -



WA = 103.3 KN/M C2 = 155.6 C4 = 70.2
WB = 89.7 KN/M C3 = 256.8



FROM ANALYSIS PAGES BS/25-28

MAX. SHEAR = 165.8 KN. (232 ULT.)
SPAN MOMENT = 70.6 KN.M (99 ULT)
SUPP. MOMENT = 77.9 KN.M. (109 ULT.)

FROM PAGES BS/31-35, PROV. 6H16 TOP/M
6H16 BOTT/M
H10-300 LINKS
(4 LEGS/M).

LOCATION OF LOAD CENTROID

FOUNDATION TYPE X

Enter Foundation Dimensions Length = 6.525 Width = 1.50 Depth = 0.55

DESCRIPTION	Load	LA x-x	Moment x	Load	LA y-y	Moment y
Wall A	154.95	0.525	81.349	154.95	0.750	116.213
Column C2	155.60	0.800	124.480	155.60	0.750	116.700
Column C3	256.80	3.800	975.840	256.80	0.750	192.600
Column C4	70.20	5.725	401.895	70.20	0.750	52.650
Wall B	134.55	6.000	807.300	134.55	0.750	100.913
Foundation self weight	129.20	3.263	421.499	129.20	0.750	96.896
	0.00	0.000	0.000	0.00	0.000	0.000
	0.00	0.000	0.000	0.00	0.000	0.000
	0.00	0.000	0.000	0.00	0.000	0.000
	0.00	0.000	0.000	0.00	0.000	0.000
	0.00	0.000	0.000	0.00	0.000	0.000
	0.00	0.000	0.000	0.00	0.000	0.000
	901.30		**2812.362**	**901.30**		**675.971**

Origin to centroid x-x = 3.120 m
Origin to centroid y-y = 0.750 m

Hence Eccentricity x-x = 0.142 m **and Ey-y =** 0.000 m

Now Moment about x-x = 128.11 kN.m **and My-y =** 0.00 kN.m

Basic Pressures (kN/m²) **W/A =** 92.09 **Mx-x =** 12.04 **My-y =** 0.00

Max. Pressure under Base = 104.12 kN/m²

Min. Pressure under Base = 80.05 kN/m²

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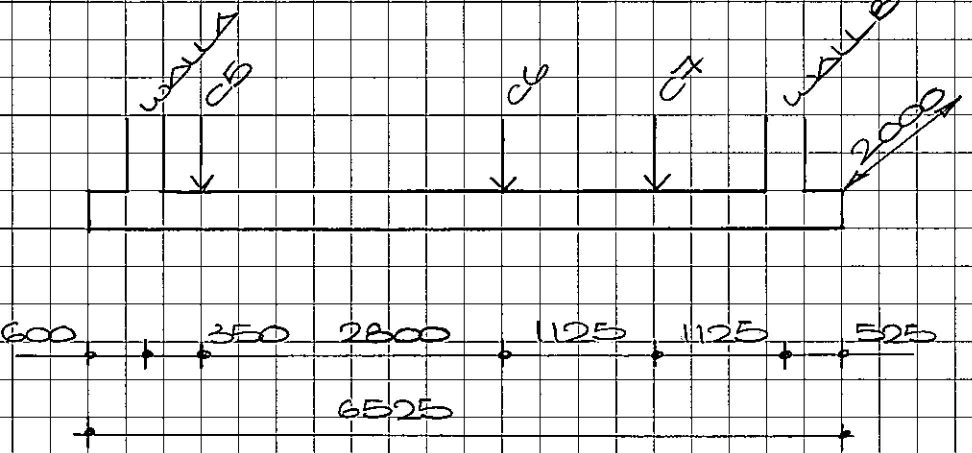
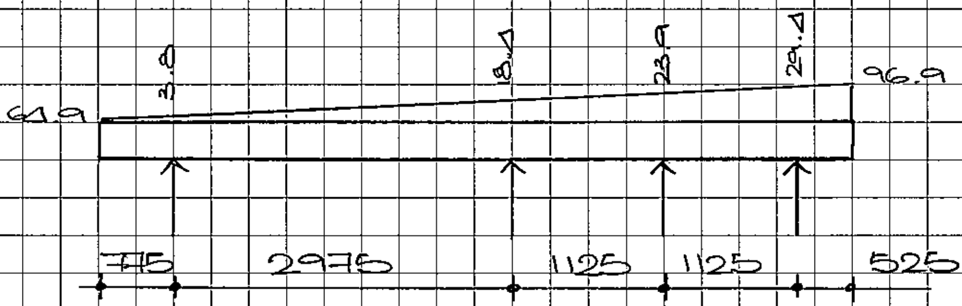
PROJECT: 65 GOLDHURST TERRACE, NW6		JOB NO. 4102	
CALCULATION SHEET	TITLE: BASEMENT	DATE: OCT/14	
	BY: SM / CHECKED:	SHEET NO. BS/23	REV

COMBINED FOUNDATION TYPE Y -



WA = 68.1 KN/M
WB = 84.4 KN/M
C5 = 197.3 KN.
C6 = 279.8 KN.
C7 = 101.5 KN.



FROM ANALYSIS PAGES BS/29-33

MAX SHEAR = 128.3 KN. (180 ULT.)
MAX SPAN MOMENT = 47.6 KN.M (67 ULT.)
SUPP. MOMENT = 54.4 KN.M (76 ULT.)

FROM PAGES BS/34-35, REBAR AS TYPE X

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LOCATION OF LOAD CENTROID

FOUNDATION TYPE Y

Enter Foundation Dimensions Length = **6.525** Width = **2.00** Depth = **0.55**

DESCRIPTION	Load	LA x-x	Moment x	Load	LA y-y	Moment y
Wall A	136.20	0.525	71.505	136.20	1.000	136.200
Column C5	197.30	0.800	157.840	197.30	1.000	197.300
Column C6	279.80	3.800	1063.240	279.80	1.000	279.800
Column C7	101.50	5.725	581.088	101.50	0.325	32.988
Wall B	168.80	6.000	1012.800	168.80	1.000	168.800
Foundation self weight	172.26	3.263	561.998	172.26	1.000	172.260
	0.00	0.000	0.000	0.00	0.000	0.000
	0.00	0.000	0.000	0.00	0.000	0.000
	0.00	0.000	0.000	0.00	0.000	0.000
	0.00	0.000	0.000	0.00	0.000	0.000
	0.00	0.000	0.000	0.00	0.000	0.000
	0.00	0.000	0.000	0.00	0.000	0.000
	1055.86		**3448.471**	**1055.86**		**987.348**

Origin to centroid x-x = 3.266 m
Origin to centroid y-y = 0.935 m

Hence Eccentricity x-x = 0.004 m **and Ey-y =** 0.065 m

Now Moment about x-x = 3.73 kN.m **and My-y =** 68.51 kN.m

Basic Pressures (kN/m²) W/A = 80.91 Mx-x = 0.26 My-y = 15.75

Max. Pressure under Base = 96.92 kN/m²

Min. Pressure under Base = 64.90 kN/m²

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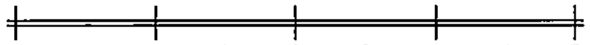
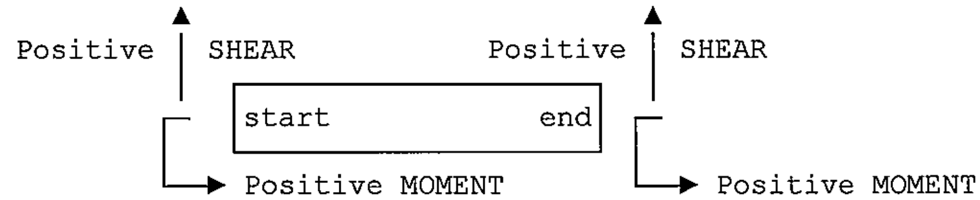
* LOCATION: COMBINED FOUNDATION TYPE X
*
* 4 SPAN CONTINUOUS BEAM
*
*
* All units are kN & m and combinations thereof.
* Left hand end span is a cantilever.
* Right hand end span is a cantilever.
* Member numbers are in brackets, other numbers are joint numbers.
*
* 
* 1 (1) 2 (2) 3 (3) 4 (4) 5
*
* The effects of positive results for forces acting on the ends of
* spans are depicted below. THINK OF THE JOINTS AS APPLYING FORCES
* TO THE BEAM SPAN ENDS.



TABULATE DISPLACEMENTS, FORCES, REACTIONS
PRINT DATA, RESULTS FROM BS/25
TYPE PLANE FRAME
METHOD ELASTIC
NUMBER OF JOINTS 5
NUMBER OF MEMBERS 4
NUMBER OF SUPPORTS 3
NUMBER OF SEGMENTS 15 TRACE
NUMBER OF LOADINGS 1
JOINT COORDINATES
1 0 0
2 0.65 0 SUPPORT
3 3.8 0 SUPPORT
4 5.875 0 SUPPORT
5 6.525 0
JOINT RELEASES
2 THRU 4 MOMENT Z
3 THRU 4 FORCE X
MEMBER INCIDENCES
1 THRU 4 RANGE 1 2 4 5
CONSTANTS E 28E6 ALL G 11.2E6 ALL
MEMBER PROPERTIES
1 RECTANGLE DY 0.55 DZ 1
2 RECTANGLE DY 0.55 DZ 1
3 RECTANGLE DY 0.55 DZ 1
4 RECTANGLE DY 0.55 DZ 1
LOADING CASE 1: UNFACTORED DEAD & LIVE ON ALL SPANS
MEMBER LOADS
1 FORCE Y UNIFORM W -80.1
1 FORCE Y LINEAR WA -24 WB -21.6 LA 0 LB 0.65
2 FORCE Y UNIFORM W -80.1
2 FORCE Y LINEAR WA -21.6 WB -10 LA 0 LB 3.15
3 FORCE Y UNIFORM W -80.1
3 FORCE Y LINEAR WA -10 WB -2.4 LA 0 LB 2.075
4 FORCE Y UNIFORM W -80.1

4 FORCE Y LINEAR WA -2.4 WB 0 LA 0 LB 0.65
SOLVE

LOADING CASE 1: UNFACTORED DEAD & LIVE ON ALL SPANS
JOINT DISPLACEMENTS

JOINT	X DISPLACEMENT	Y DISPLACEMENT	Z ROTATION
1	0.000000000	0.000095172	-0.000149904
2	0.000000000	0.000000000	-0.000162107
3	0.000000000	0.000000000	0.000076735
4	0.000000000	0.000000000	-0.000011628
5	0.000000000	-0.000015528	-0.000021143
6	0.000000000	0.000088695	-0.000149907
7	0.000000000	0.000082255	-0.000149933
8	0.000000000	0.000075852	-0.000150002
9	0.000000000	0.000069482	-0.000150136
10	0.000000000	0.000063143	-0.000150358
11	0.000000000	0.000056829	-0.000150687
12	0.000000000	0.000050536	-0.000151148
13	0.000000000	0.000044258	-0.000151760
14	0.000000000	0.000037987	-0.000152546
15	0.000000000	0.000031715	-0.000153526
16	0.000000000	0.000025434	-0.000154724
17	0.000000000	0.000019133	-0.000156159
18	0.000000000	0.000012801	-0.000157854
19	0.000000000	0.000006428	-0.000159829
20	0.000000000	-0.000039901	-0.000166574
21	0.000000000	-0.000078458	-0.000157975
22	0.000000000	-0.000113188	-0.000138708
23	0.000000000	-0.000142106	-0.000111153
24	0.000000000	-0.000163727	-0.000077672
25	0.000000000	-0.000177060	-0.000040608
26	0.000000000	-0.000181602	-0.000002285
27	0.000000000	-0.000177339	0.000034990
28	0.000000000	-0.000164738	0.000068930
29	0.000000000	-0.000144745	0.000097265
30	0.000000000	-0.000118780	0.000117745
31	0.000000000	-0.000088734	0.000128136
32	0.000000000	-0.000056965	0.000126226
33	0.000000000	-0.000026293	0.000109818
34	0.000000000	0.000005758	0.000051835
35	0.000000000	0.000008779	0.000032245
36	0.000000000	0.000009755	0.000017356
37	0.000000000	0.000009291	0.000006564
38	0.000000000	0.000007912	-0.000000736
39	0.000000000	0.000006056	-0.000005140
40	0.000000000	0.000004082	-0.000007246
41	0.000000000	0.000002263	-0.000007645
42	0.000000000	0.000000795	-0.000006924
43	0.000000000	-0.000000212	-0.000005670
44	0.000000000	-0.000000724	-0.000004464
45	0.000000000	-0.000000792	-0.000003884
46	0.000000000	-0.000000542	-0.000004504
47	0.000000000	-0.000000183	-0.000006896
48	0.000000000	-0.000000974	-0.000013411

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JOINT	X DISPLACEMENT	Y DISPLACEMENT	Z ROTATION
49	0.000000000	-0.000001991	-0.000014955
50	0.000000000	-0.000003039	-0.000016278
51	0.000000000	-0.000004110	-0.000017398
52	0.000000000	-0.000005196	-0.000018331
53	0.000000000	-0.000006288	-0.000019094
54	0.000000000	-0.000007381	-0.000019704
55	0.000000000	-0.000008467	-0.000020180
56	0.000000000	-0.000009541	-0.000020537
57	0.000000000	-0.000010599	-0.000020792
58	0.000000000	-0.000011636	-0.000020963
59	0.000000000	-0.000012650	-0.000021067
60	0.000000000	-0.000013638	-0.000021120
61	0.000000000	-0.000014597	-0.000021140

LOADING CASE 1: UNFACTORED DEAD & LIVE ON ALL SPANS
MEMBER FORCES AT START OF FIRST SEGMENT AND ENDS OF ALL SEGMENTS

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	BENDING MOMENT
1	1	0.0000	0.0000	0.0000
	6	0.0000	4.5075	-0.0977
	7	0.0000	9.0081	-0.3906
	8	0.0000	13.5018	-0.8783
	9	0.0000	17.9885	-1.5606
	10	0.0000	22.4683	-2.4372
	11	0.0000	26.9412	-3.5078
	12	0.0000	31.4071	-4.7720
	13	0.0000	35.8661	-6.2296
	14	0.0000	40.3182	-7.8803
	15	0.0000	44.7633	-9.7238
	16	0.0000	49.2015	-11.7597
	17	0.0000	53.6328	-13.9878
	18	0.0000	58.0571	-16.4078
	19	0.0000	62.4745	-19.0193
	2	0.0000	66.8850	-21.8221
2	2	0.0000	136.2842	21.8221
	20	0.0000	-115.0084	4.5608
	21	0.0000	-93.8950	26.4928
	22	0.0000	-72.9440	44.0080
	23	0.0000	-52.1554	57.1406
	24	0.0000	-31.5292	65.9247
	25	0.0000	-11.0654	70.3943
	26	0.0000	9.2360	70.5835
	27	0.0000	29.3750	66.5265
	28	0.0000	49.3516	58.2574
	29	0.0000	69.1658	45.8102
	30	0.0000	88.8176	29.2191
	31	0.0000	108.3070	8.5182
	32	0.0000	127.6340	-16.2584
	33	0.0000	146.7986	-45.0767
	3	0.0000	165.8008	-77.9025
3	3	0.0000	120.1576	77.9025
	34	0.0000	-107.7288	-62.1411
	35	0.0000	-95.3701	-48.0943
	36	0.0000	-83.0815	-35.7522
	37	0.0000	-70.8629	-25.1052
	38	0.0000	-58.7145	-16.1435
	39	0.0000	-46.6362	-8.8576

	40	0.0000	-34.6279	-3.2376
	41	0.0000	-22.6897	0.7260
	42	0.0000	-10.8217	3.0431
	43	0.0000	0.9763	3.7233
	44	0.0000	12.7042	2.7762
	45	0.0000	24.3620	0.2117
	46	0.0000	35.9498	-3.9607
	47	0.0000	47.4674	-9.7312
	4	0.0000	58.9149	-17.0901
4	4	0.0000	52.8450	17.0901
	48	0.0000	-49.2735	-14.8776
	49	0.0000	-45.7089	-12.8197
	50	0.0000	-42.1512	-10.9160
	51	0.0000	-38.6005	-9.1665
	52	0.0000	-35.0567	-7.5706
	53	0.0000	-31.5198	-6.1281
	54	0.0000	-27.9899	-4.8388
	55	0.0000	-24.4669	-3.7022
	56	0.0000	-20.9508	-2.7182
	57	0.0000	-17.4417	-1.8864
	58	0.0000	-13.9395	-1.2065
	59	0.0000	-10.4442	-0.6782
	60	0.0000	-6.9559	-0.3012
	61	0.0000	-3.4745	-0.0753
	5	0.0000	0.0000	0.0000

LOADING CASE 1: UNFACTORED DEAD & LIVE ON ALL SPANS
SUPPORT REACTIONS

JOINT	X FORCE	Y FORCE	Z MOMENT
2	0.0000	203.1692	0.0000
3	0.0000	285.9583	0.0000
4	0.0000	111.7599	0.0000

EQUILIBRIUM CHECK	SUM OF FORCES	REACTION
FORCES IN DIRECTION X	0.0000	0.0000
FORCES IN DIRECTION Y	-600.8875	600.8875
MOMENTS ABOUT AXIS Z	-1875.2913	1875.2913

```
3 FORCE Y LINEAR WA -18.4 WB -23.9 LA 0 LB 1.125
4 FORCE Y UNIFORM W -64.9
4 FORCE Y LINEAR WA -23.9 WB -29.4 LA 0 LB 1.125
5 FORCE Y UNIFORM W -64.9
5 FORCE Y LINEAR WA -29.4 WB -32 LA 0 LB 0.525
SOLVE
```

LOADING CASE 1: UNFACTORED DEAD & LIVE ON ALL SPANS
JOINT DISPLACEMENTS

JOINT	X DISPLACEMENT	Y DISPLACEMENT	Z ROTATION
1	0.000000000	0.000062463	-0.000082311
2	0.000000000	0.000000000	-0.000095471
3	0.000000000	0.000000000	0.000049648
4	0.000000000	0.000000000	-0.000006764
5	0.000000000	0.000000000	-0.000001900
6	0.000000000	-0.000005929	-0.000007879
7	0.000000000	0.000058227	-0.000082315
8	0.000000000	0.000054024	-0.000082342
9	0.000000000	0.000049853	-0.000082415
10	0.000000000	0.000045710	-0.000082558
11	0.000000000	0.000041592	-0.000082794
12	0.000000000	0.000037494	-0.000083146
13	0.000000000	0.000033408	-0.000083638
14	0.000000000	0.000029327	-0.000084294
15	0.000000000	0.000025243	-0.000085137
16	0.000000000	0.000021144	-0.000086192
17	0.000000000	0.000017021	-0.000087481
18	0.000000000	0.000012859	-0.000089030
19	0.000000000	0.000008645	-0.000090861
20	0.000000000	0.000004364	-0.000093000
21	0.000000000	-0.000023141	-0.000100897
22	0.000000000	-0.000045956	-0.000097802
23	0.000000000	-0.000066888	-0.000087595
24	0.000000000	-0.000084658	-0.000071706
25	0.000000000	-0.000098278	-0.000051584
26	0.000000000	-0.000107044	-0.000028699
27	0.000000000	-0.000110549	-0.000004537
28	0.000000000	-0.000108681	0.000019393
29	0.000000000	-0.000101630	0.000041563
30	0.000000000	-0.000089890	0.000060428
31	0.000000000	-0.000074265	0.000074420
32	0.000000000	-0.000055871	0.000081953
33	0.000000000	-0.000036139	0.000081423
34	0.000000000	-0.000016823	0.000071203
35	0.000000000	0.000002022	0.000039810
36	0.000000000	0.000003446	0.000031240
37	0.000000000	0.000004363	0.000023847
38	0.000000000	0.000004860	0.000017539
39	0.000000000	0.000005015	0.000012226
40	0.000000000	0.000004899	0.000007814
41	0.000000000	0.000004577	0.000004210
42	0.000000000	0.000004106	0.000001322
43	0.000000000	0.000003536	-0.000000944
44	0.000000000	0.000002912	-0.000002682

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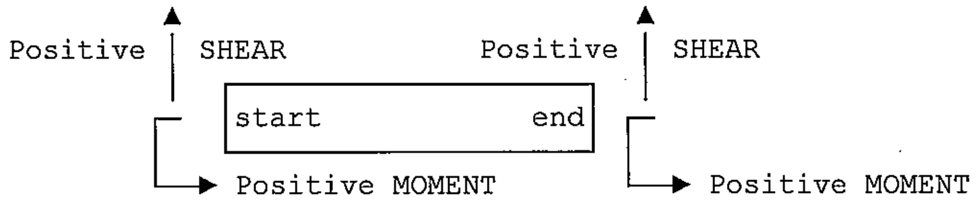
```
* LOCATION: COMBINED FOUNDATION TYPE Y
*
* 5 SPAN CONTINUOUS BEAM
* ______________________
*
* All units are kN & m and combinations thereof.
* Left hand end span is a cantilever.
* Right hand end span is a cantilever.
* Member numbers are in brackets, other numbers are joint numbers.
*
* +-------+-------+-------+-------+-------+
* 1  (1)  2  (2)  3  (3)  4  (4)  5  (5)  6
*
* The effects of positive results for forces acting on the ends of
* spans are depicted below.  THINK OF THE JOINTS AS APPLYING FORCES
* TO THE BEAM SPAN ENDS.
*
*
*
*
*
*
```




```
TABULATE DISPLACEMENTS, FORCES, REACTIONS
PRINT DATA, RESULTS FROM BS/29
TYPE PLANE FRAME
METHOD ELASTIC
NUMBER OF JOINTS 6
NUMBER OF MEMBERS 5
NUMBER OF SUPPORTS 4
NUMBER OF SEGMENTS 15 TRACE
NUMBER OF LOADINGS 1
JOINT COORDINATES
1 0 0
2 0.775 0 SUPPORT
3 3.75 0 SUPPORT
4 4.875 0 SUPPORT
5 6 0 SUPPORT
6 6.525 0
JOINT RELEASES
2 THRU 5 MOMENT Z
3 THRU 5 FORCE X
MEMBER INCIDENCES
1 THRU 5 RANGE 1 2 5 6
CONSTANTS E 28E6 ALL G 11.2E6 ALL
MEMBER PROPERTIES
1  RECTANGLE DY 0.55 DZ 1
2  RECTANGLE DY 0.55 DZ 1
3  RECTANGLE DY 0.55 DZ 1
4  RECTANGLE DY 0.55 DZ 1
5  RECTANGLE DY 0.55 DZ 1
LOADING CASE 1: UNFACTORED DEAD & LIVE ON ALL SPANS
MEMBER LOADS
1 FORCE Y UNIFORM W -64.9
1 FORCE Y LINEAR WA 0 WB -3.8 LA 0 LB 0.775
2 FORCE Y UNIFORM W -64.9
2 FORCE Y LINEAR WA -3.8 WB -18.4 LA 0 LB 2.975
3 FORCE Y UNIFORM W -64.9
```

JOINT	X DISPLACEMENT	Y DISPLACEMENT	Z ROTATION
45	0.000000000	0.000002270	-0.000003986
46	0.000000000	0.000001639	-0.000004951
47	0.000000000	0.000001041	-0.000005672
48	0.000000000	0.000000492	-0.000006244
49	0.000000000	-0.000001079	-0.000007004
50	0.000000000	-0.000002059	-0.000006737
51	0.000000000	-0.000002905	-0.000006061
52	0.000000000	-0.000003590	-0.000005073
53	0.000000000	-0.000004093	-0.000003871
54	0.000000000	-0.000004401	-0.000002553
55	0.000000000	-0.000004509	-0.000001218
56	0.000000000	-0.000004420	0.000000034
57	0.000000000	-0.000004143	0.000001106
58	0.000000000	-0.000003694	0.000001896
59	0.000000000	-0.000003098	0.000002304
60	0.000000000	-0.000002387	0.000002230
61	0.000000000	-0.000001602	0.000001572
62	0.000000000	-0.000000787	0.000000229
63	0.000000000	-0.000000417	-0.000003016
64	0.000000000	-0.000000849	-0.000003984
65	0.000000000	-0.000001289	-0.000004814
66	0.000000000	-0.000001733	-0.000005517
67	0.000000000	-0.000002177	-0.000006104
68	0.000000000	-0.000002618	-0.000006584
69	0.000000000	-0.000003050	-0.000006970
70	0.000000000	-0.000003472	-0.000007270
71	0.000000000	-0.000003880	-0.000007495
72	0.000000000	-0.000004272	-0.000007657
73	0.000000000	-0.000004645	-0.000007766
74	0.000000000	-0.000004999	-0.000007831
75	0.000000000	-0.000005332	-0.000007865
76	0.000000000	-0.000005642	-0.000007878

LOADING CASE 1: UNFACTORED DEAD & LIVE ON ALL SPANS
MEMBER FORCES AT START OF FIRST SEGMENT AND ENDS OF ALL SEGMENTS

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	BENDING MOMENT
1	1	0.0000	0.0000	0.0000
	7	0.0000	3.3597	-0.0867
	8	0.0000	6.7325	-0.3474
	9	0.0000	10.1184	-0.7827
	10	0.0000	13.5174	-1.3932
	11	0.0000	16.9294	-2.1797
	12	0.0000	20.3546	-3.1428
	13	0.0000	23.7928	-4.2832
	14	0.0000	27.2442	-5.6016
	15	0.0000	30.7086	-7.0987
	16	0.0000	34.1861	-8.7751
	17	0.0000	37.6767	-10.6315
	18	0.0000	41.1804	-12.6685
	19	0.0000	44.6972	-14.8870
	20	0.0000	48.2270	-17.2875
	2	0.0000	51.7700	-19.8707
2	2	0.0000	97.8361	19.8707
	21	0.0000	-84.1140	-1.8241
	22	0.0000	-70.1990	13.4818
	23	0.0000	-56.0909	26.0087

	24	0.0000	-41.7897	35.7184
	25	0.0000	-27.2955	42.5725
	26	0.0000	-12.6083	46.5329
	27	0.0000	2.2720	47.5611
	28	0.0000	17.3454	45.6189
	29	0.0000	32.6117	40.6680
	30	0.0000	48.0712	32.6701
	31	0.0000	63.7236	21.5870
	32	0.0000	79.5691	7.3803
	33	0.0000	95.6077	-9.9882
	34	0.0000	111.8393	-30.5569
	3	0.0000	128.2639	-54.3639
3	3	0.0000	93.7932	54.3639
	35	0.0000	-87.5319	-47.5640
	36	0.0000	-81.2432	-41.2348
	37	0.0000	-74.9269	-35.3783
	38	0.0000	-68.5832	-29.9965
	39	0.0000	-62.2119	-25.0915
	40	0.0000	-55.8132	-20.6654
	41	0.0000	-49.3869	-16.7202
	42	0.0000	-42.9332	-13.2580
	43	0.0000	-36.4519	-10.2809
	44	0.0000	-29.9432	-7.7909
	45	0.0000	-23.4069	-5.7901
	46	0.0000	-16.8432	-4.2806
	47	0.0000	-10.2519	-3.2643
	48	0.0000	-3.6332	-2.7435
	4	0.0000	3.0131	-2.7200
4	4	0.0000	41.6350	2.7200
	49	0.0000	-34.9612	0.1525
	50	0.0000	-28.2600	2.5235
	51	0.0000	-21.5312	4.3908
	52	0.0000	-14.7750	5.7524
	53	0.0000	-7.9912	6.6064
	54	0.0000	-1.1800	6.9504
	55	0.0000	5.6588	6.7827
	56	0.0000	12.5250	6.1009
	57	0.0000	19.4188	4.9032
	58	0.0000	26.3400	3.1874
	59	0.0000	33.2888	0.9515
	60	0.0000	40.2650	-1.8066
	61	0.0000	47.2688	-5.0889
	62	0.0000	54.3000	-8.8976
	5	0.0000	61.3588	-13.2346
5	5	0.0000	50.1900	13.2346
	63	0.0000	-46.8865	-11.5357
	64	0.0000	-43.5769	-9.9526
	65	0.0000	-40.2612	-8.4854
	66	0.0000	-36.9395	-7.1344
	67	0.0000	-33.6117	-5.8997
	68	0.0000	-30.2778	-4.7817
	69	0.0000	-26.9379	-3.7804
	70	0.0000	-23.5919	-2.8961
	71	0.0000	-20.2398	-2.1290
	72	0.0000	-16.8817	-1.4794
	73	0.0000	-13.5175	-0.9474
	74	0.0000	-10.1472	-0.5332

75	0.0000	-6.7709	-0.2371
76	0.0000	-3.3885	-0.0593
6	0.0000	0.0000	0.0000

LOADING CASE 1: UNFACTORED DEAD & LIVE ON ALL SPANS
SUPPORT REACTIONS

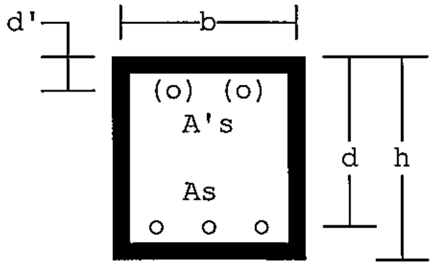
JOINT	X FORCE	Y FORCE	Z MOMENT
2	0.0000	149.6061	0.0000
3	0.0000	222.0571	0.0000
4	0.0000	44.6481	0.0000
5	0.0000	111.5488	0.0000

EQUILIBRIUM CHECK	SUM OF FORCES	REACTION
FORCES IN DIRECTION X	0.0000	0.0000
FORCES IN DIRECTION Y	-527.8600	527.8600
MOMENTS ABOUT AXIS Z	-1835.6108	1835.6108

Location: COMBINED FOUNDATION TYPE X (TYPE Y SIMILAR)

Bending in rectangular beams with optional calculations for shear, lap lengths, bar curtailment and limiting span/effective-depth ratio

Calculations are based on formulae in Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to 0.5*d.



Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Moment before redistribution	Mbef=109 kNm
Beam being analysed is considered as non-continuous.	
Characteristic concrete strength	fcu=35 N/mm^2
Max.aggregate size (for bar spc)	hagg=15 mm
Char.strength of long'l bars	fy=500 N/mm^2
Longitudinal reinforcement is high-yield steel.	
Diameter of tension bars	dia=16 mm
Diameter of link legs	dial=10 mm
Char.strength of link steel	fyv=500 N/mm^2
High-yield steel shear reinforcement.	
Nominal concrete cover	cover=75 mm
Overall depth of section	h=550 mm
Effective depth of section	d=457 mm
Breadth of section	b=1000 mm

Longitudinal reinforcement

Tension steel provided per=100*Aspr/(b*h)
=0.21717 % of gross section.

TENSION REINFORCEMENT SUMMARY

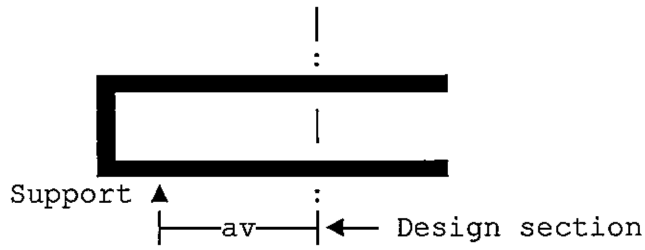
Characteristic strength	500 N/mm^2
Diameter of bars	16 mm
Number of bars	6
arranged in a single layer	
Cover to all steel	75 mm
Area of steel required	722.15 mm^2
Area of steel provided	1206.4 mm^2
Percentage provided	0.21717 %
Weight of steel provided	9.47 kg/m
Max.permissible spacing	235 mm
Link size assumed	10 mm

Check on span/effective-depth ratio

Basic ratio for simp.-sup.beam   bs'd=20 see Table 3.9
As applied-moment factor   M'bd2=M*1000*1000/(b*d^2)
=0.51674 N/mm^2
Mod.factor for tension steel from
equation 7 (Table 3.10) modf1=0.55+(477-fs)/(120*(.9+M'bd2))=2.182
but this cannot exceed 2, so   modf1=2
Number of comp.bars provided   nbarc=6
Diameter of compression bars   diac=12 mm
Area of comp.steel provided   As'pr=678.58 mm^2
Percentage of compression steel   per'n=100*As'pr/(b*d)=0.14702 %
From Equation 9 of BS8110, with percentage of comp.steel=0.14702 %,
Mod.factor for compression steel   modf2=1+per'n/(3+per'n)=1.0467
Maximum permissible
span/effective-depth ratio   ps'd=bs'd*modf1*modf2=41.869
Span of beam (see Cls.3.4.1.2-4)   span=6.525 m
Actual span/effective-depth ratio as'd=1000*span/d=14.278
As this does not exceed   41.869, this is Acceptable.

Shear reinforcement

Shear calculations are in accordance with Clauses 3.4.5 of Code



Location for shear calculation: AT SUPPORT
Effective breadth for shear   bv=1000 mm
Shear force due to ultimate load   V=232 kN
Distance from support   av=229 mm
No.of tension bars effective at section nbars=6
Design shear stress in concrete   vc=0.79*pcnt^(1/3)*f00d^.25/1.25
=0.40542 N/mm^2
As bv exceeds 350 mm note the conditions in Clause 3.4.5.5:
i) that no longitudinal bar should be more than 150 mm from a vertical leg, and
ii) (because bv exceeds d), that the transverse spacing of the legs must not exceed the effective depth d (i.e. 457 mm).
Number of legs to be provided   nlegs=4
Chosen link spacing   sv'=300 mm

Use 10 mm links (4 legs), spaced at 300 mm ctrs.along beam.

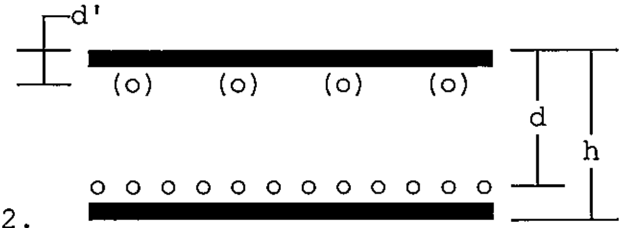
When detailing steel, watch carefully the requirements of Cl.3.4.5.5.

SHEAR REINFORCEMENT SUMMARY		
	Characteristic strength	500 N/mm^2
	Diameter of links	10 mm
	Number of legs	4
	Spacing	300 mm
	Approx.weight of links	9.5558 kg/m

Location: UNDERPINNING - CHARACTERISTIC MOMENT < 62.1 kN.m

Bending in solid slabs (with comp.steel if reqd.), designed per metre width, with checks on minimum steel and span/effective-depth ratio

Calculations are based on formulae in Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to d/2.



Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Moment before redistribution   Mbef=1.4*62.1
=86.94 kNm per metre width
Slab containing section being analysed is considered as non-continuous.
Characteristic concrete strength   fcu=35 N/mm^2
Characteristic steel strength   fy=500 N/mm^2
Longitudinal reinforcement is high-yield steel.
Diameter of tension bars   dia=16 mm
Nominal concrete cover   cover=75 mm
Overall thickness of slab   h=350 mm
Effective depth of section   d=267 mm
Area of tension steel required   As=M*10^6/(z*fy/gammaS)
=788.34 mm2/metre width.

Chosen spacing of tension bars   pch=150 mm
Diameter of distribution bars   diamn=10 mm
Spacing of distribution bars   pchDA=150 mm

TENSION REINFORCEMENT SUMMARY		
	Characteristic strength	500 N/mm^2
	Diameter of bars	16 mm
	Spacing of bars	150 mm
	Effective depth	267 mm
	Area of steel required	788.34 mm2/m
	Area of steel provided	1340 mm2/m
	Percentage provided	0.38286 %
	Weight of steel provided	10.52 kg/m^2

DISTRIBUTION REINFORCEMENT SUMMARY		
	Characteristic strength	500 N/mm^2
	Diameter of bars	10 mm
	Spacing of bars	150 mm
	Depth to bar centres	254 mm
	Area of steel required	455 mm2/m
	Area of steel provided	523 mm2/m
	Percentage provided	0.14943 %
	Weight of steel provided	4.11 kg/m^2

Check on span/effective-depth ratio

Basic ratio for cantilever slab	bs'd=7 (see Table 3.9)
Mod.factor for tension steel	modf1=1.6544
Diameter of compression bars	diac=16 mm
Spacing of comp.bars provided	pchCA=150
Compression steel provided	As'pr=1000/pchCA*PI*diac^2/4
	=1340.4 mm² per m
Percentage of compression steel	per'=100*As'pr/(1000*d)=0.50203 %

From Equation 9 of BS8110, with percentage of comp.steel=0.50203 %,

Mod.factor for compression steel	modf2=1+per'/(3+per')=1.1434
Maximum permissible span/effective-depth ratio	ps'd=bs'd*modf1*modf2=13.241
Effective span of slab	span=3.3 m
True span/effective-depth ratio	as'd=1000*span/d=12.36
As this does not exceed	13.241, this is Acceptable.

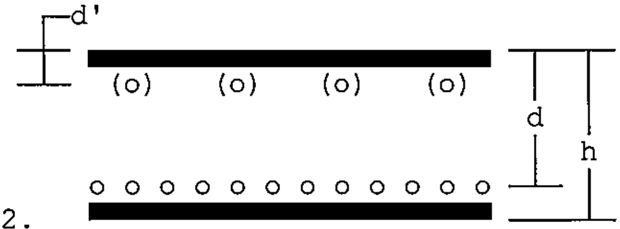
No 81

Office: 5831

Location: UNDERPINNING - CHARACTERISTIC MOMENT < 49.8 kN.m

Bending in solid slabs (with comp.steel if reqd.), designed per metre width, with checks on minimum steel and span/effective-depth ratio

Calculations are based on formulae in Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to d/2.



Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Moment before redistribution Mbef=1.4*49.8
=69.72 kNm per metre width
Slab containing section being analysed is considered as non-continuous.
Characteristic concrete strength fcu=35 N/mm^2
Characteristic steel strength fy=500 N/mm^2
Longitudinal reinforcement is high-yield steel.
Diameter of tension bars dia=12 mm
Nominal concrete cover cover=75 mm
Overall thickness of slab h=350 mm
Effective depth of section d=269 mm
Area of tension steel required As=M*10^6/(z*fy/gammaS)
=627.49 mm2/metre width.

Chosen spacing of tension bars pch=100 mm
Diameter of distribution bars diamn=10 mm
Spacing of distribution bars pchDA=150 mm

TENSION REINFORCEMENT SUMMARY	Characteristic strength	500 N/mm^2
	Diameter of bars	12 mm
	Spacing of bars	100 mm
	Effective depth	269 mm
	Area of steel required	627.49 mm2/m
	Area of steel provided	1130 mm2/m
	Percentage provided	0.32286 %
	Weight of steel provided	8.87 kg/m^2

DISTRIBUTION REINFORCEMENT SUMMARY	Characteristic strength	500 N/mm^2
	Diameter of bars	10 mm
	Spacing of bars	150 mm
	Depth to bar centres	258 mm
	Area of steel required	455 mm2/m
	Area of steel provided	523 mm2/m
	Percentage provided	0.14943 %
	Weight of steel provided	4.11 kg/m^2

Check on span/effective-depth ratio

Basic ratio for cantilever slab	bs'd=7 (see Table 3.9)
Mod.factor for tension steel	modf1=1.8553
Diameter of compression bars	diac=12 mm
Spacing of comp.bars provided	pchCA=100
Compression steel provided	As'pr=1000/pchCA*PI*diac^2/4 =1131 mm^2 per m
Percentage of compression steel	per'=100*As'pr/(1000*d)=0.42044 %

From Equation 9 of BS8110, with percentage of comp.steel=0.42044 %,

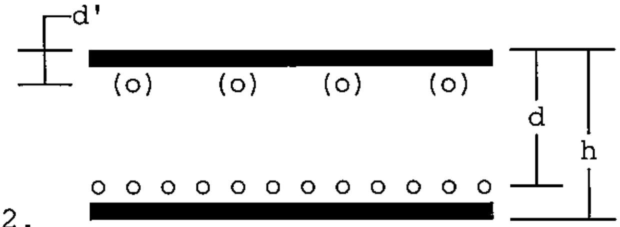
Mod.factor for compression steel	modf2=1+per'/(3+per')=1.1229
Maximum permissible span/effective-depth ratio	ps'd=bs'd*modf1*modf2=14.584
Effective span of slab	span=3.15 m
True span/effective-depth ratio	as'd=1000*span/d=11.71

As this does not exceed 14.584, this is Acceptable.

Location: UNDERPINNING - CHARACTERISTIC MOMENT < 58.2 kN.m

Bending in solid slabs (with comp.steel if reqd.), designed per metre width, with checks on minimum steel and span/effective-depth ratio

Calculations are based on formulae in Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to d/2.



Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Moment before redistribution	Mbef=1.4*58.2 =81.48 kNm per metre width

Slab containing section being analysed is considered as non-continuous.

Characteristic concrete strength	fcu=35 N/mm^2
Characteristic steel strength	fy=500 N/mm^2

Longitudinal reinforcement is high-yield steel.

Diameter of tension bars	dia=12 mm
Nominal concrete cover	cover=75 mm
Overall thickness of slab	h=350 mm
Effective depth of section	d=269 mm
Area of tension steel required	As=M*10^6/(z*fy/gammaS) =733.34 mm2/metre width.
Chosen spacing of tension bars	pch=100 mm
Diameter of distribution bars	diamn=8 mm
Spacing of distribution bars	pchDA=100 mm

TENSION REINFORCEMENT SUMMARY	Characteristic strength	500 N/mm^2
	Diameter of bars	12 mm
	Spacing of bars	100 mm
	Effective depth	269 mm
	Area of steel required	733.34 mm2/m
	Area of steel provided	1130 mm2/m
	Percentage provided	0.32286 %
	Weight of steel provided	8.87 kg/m^2

DISTRIBUTION REINFORCEMENT SUMMARY	Characteristic strength	500 N/mm^2
	Diameter of bars	8 mm
	Spacing of bars	100 mm
	Depth to bar centres	259 mm
	Area of steel required	455 mm2/m
	Area of steel provided	502 mm2/m
	Percentage provided	0.14343 %
	Weight of steel provided	3.94 kg/m^2

Check on span/effective-depth ratio

Basic ratio for cantilever slab	bs'd=7 (see Table 3.9)
Mod.factor for tension steel	modf1=1.6222
Diameter of compression bars	diac=12 mm
Spacing of comp.bars provided	pchCA=100
Compression steel provided	As'pr=1000/pchCA*PI*diac^2/4 =1131 mm^2 per m
Percentage of compression steel	per'=100*As'pr/(1000*d)=0.42044 %

From Equation 9 of BS8110, with percentage of comp.steel=0.42044 %,

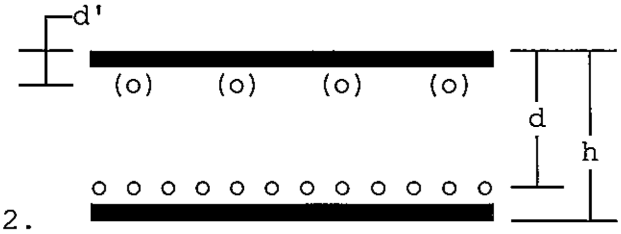
Mod.factor for compression steel	modf2=1+per'/(3+per')=1.1229
Maximum permissible span/effective-depth ratio	ps'd=bs'd*modf1*modf2=12.751
Effective span of slab	span=3.4 m
True span/effective-depth ratio	as'd=1000*span/d=12.639
As this does not exceed	12.751, this is Acceptable.

No 81

Location: UNDERPINNING - CHARACTERISTIC MOMENT < 45.3 kN.m

Bending in solid slabs (with comp.steel if reqd.), designed per metre width, with checks on minimum steel and span/effective-depth ratio

Calculations are based on formulae in Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to d/2.



Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Moment before redistribution   Mbef=1.4*45.3
    =63.42 kNm per metre width
Slab containing section being analysed is considered as non-continuous.
Characteristic concrete strength   fcu=35 N/mm^2
Characteristic steel strength   fy=500 N/mm^2
Longitudinal reinforcement is high-yield steel.
Diameter of tension bars   dia=10 mm
Nominal concrete cover   cover=75 mm
Overall thickness of slab   h=350 mm
Effective depth of section   d=270 mm
Area of tension steel required   As=M*10^6/(z*fy/gammaS)
    =568.68 mm2/metre width.

Chosen spacing of tension bars   pch=100 mm
Diameter of distribution bars   diamn=10 mm
Spacing of distribution bars   pchDA=100 mm

TENSION REINFORCEMENT SUMMARY		
	Characteristic strength	500 N/mm^2
	Diameter of bars	10 mm
	Spacing of bars	100 mm
	Effective depth	270 mm
	Area of steel required	568.68 mm2/m
	Area of steel provided	785 mm2/m
	Percentage provided	0.22429 %
	Weight of steel provided	6.16 kg/m^2

DISTRIBUTION REINFORCEMENT SUMMARY		
	Characteristic strength	500 N/mm^2
	Diameter of bars	10 mm
	Spacing of bars	100 mm
	Depth to bar centres	260 mm
	Area of steel required	455 mm2/m
	Area of steel provided	785 mm2/m
	Percentage provided	0.22429 %
	Weight of steel provided	6.16 kg/m^2

Check on span/effective-depth ratio

Basic ratio for cantilever slab	bs'd=7 (see Table 3.9)
Mod.factor for tension steel	modf1=1.6589
Diameter of compression bars	diac=10 mm
Spacing of comp.bars provided	pchCA=100
Compression steel provided	As'pr=1000/pchCA*PI*diac^2/4 =785.4 mm^2 per m
Percentage of compression steel	per'=100*As'pr/(1000*d)=0.29089 %

From Equation 9 of BS8110, with percentage of comp.steel=0.29089 %,

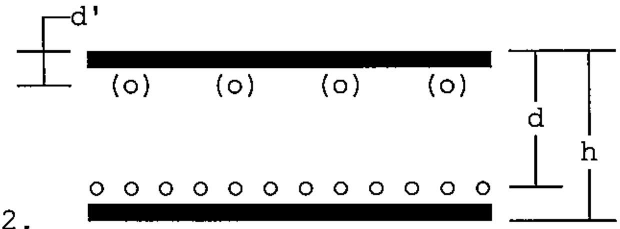
Mod.factor for compression steel	modf2=1+per'/(3+per')=1.0884
Maximum permissible span/effective-depth ratio	ps'd=bs'd*modf1*modf2=12.639
Effective span of slab	span=3.15 m
True span/effective-depth ratio	as'd=1000*span/d=11.667
As this does not exceed	12.639, this is Acceptable.

No 81

Location: UNDERPINNING - CHARACTERISTIC MOMENT < 35.7 kN.m

Bending in solid slabs (with comp.steel if reqd.), designed per metre width, with checks on minimum steel and span/effective-depth ratio

Calculations are based on formulae in Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to d/2.



Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Moment before redistribution	Mbef=1.4*35.7 =49.98 kNm per metre width

Slab containing section being analysed is considered as non-continuous.

Characteristic concrete strength	fcu=35 N/mm^2
Characteristic steel strength	fy=500 N/mm^2

Longitudinal reinforcement is high-yield steel.

Diameter of tension bars	dia=10 mm
Nominal concrete cover	cover=75 mm
Overall thickness of slab	h=350 mm
Effective depth of section	d=270 mm
Area of tension steel required	As=M*10^6/(z*fy/gammaS) =448.16 mm2/metre width.
Chosen spacing of tension bars	pch=150 mm

TENSION (AND DISTRIBUTION) REINFORCEMENT SUMMARY

Characteristic strength	500 N/mm^2
Diameter of bars	10 mm
Spacing of bars	150 mm
Effective depth	270 mm
Area of steel required	455 mm2/m
Area of steel provided	523 mm2/m
Percentage provided	0.14943 %
Weight of steel provided	4.11 kg/m^2

Check on span/effective-depth ratio

Basic ratio for simp.-sup.slab	bs'd=20 (see Table 3.9)
Mod.factor for tension steel	modf1=1.5328
Diameter of compression bars	diac=10 mm
Spacing of comp.bars provided	pchCA=150
Compression steel provided	As'pr=1000/pchCA*PI*diac^2/4
	=523.6 mm^2 per m
Percentage of compression steel	per'=100*As'pr/(1000*d)=0.19393 %

From Equation 9 of BS8110, with percentage of comp.steel=0.19393 %,

Mod.factor for compression steel	modf2=1+per'/(3+per')=1.0607
Maximum permissible span/effective-depth ratio	ps'd=bs'd*modf1*modf2=32.518
Effective span of slab	span=3.15 m
True span/effective-depth ratio	as'd=1000*span/d=11.667
As this does not exceed	32.518, this is Acceptable.

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