Preliminary Design of Building Structures

3rd Year

Structural Engineering

2008/9

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Contents

BE Structural Eng – Project III

1. Introduction

1.1 Course Outline

Goals

The goal is that you will:

(a) be able to plan an outline scheme design for building structures, and;

(b) be able to verify, through approximate design, that your scheme is viable.

In other words, you will learn to carry out a complete preliminary design of a structure.

The detailed design of elements is covered in your other modules such as concrete and steel design.

Preliminary Design

Preliminary design is:

"a rapid, approximate, manual method of designing a structure."

– T.J. MacGinley

"In the initial stages of the design of a building structure it is necessary, often at short notice, to produce alterative schemes that can be assessed…"

– ISE Green Book

Therefore, it should be:

- simple;
- quick;
- conservative, and;
- reliable.

Lengthy analytical methods should be avoided.

It is often based upon vague and limited information on matters affecting the structure such as imposed loads, nature of finishes, dimensions.

It is needed to:

- obtain costs estimates;
- compare alternative schemes for architectural and functional suitability;
- obtain initial estimates for computer analysis, and;
- check a completed detailed design.

Each structural scheme should be:

- suitable for its purpose;
- sensibly economical, and;
- not unduly sensitive to likely changes as the design progresses.

We will develop methods of analysing and designing structures that meet the above requirements.

Note that even though detailed design does not feature, it should be clear that knowledge of it is central to preliminary design.

1.2 Syllabus

The topics covered in this subject include the following:

- Relevant Literature
- Economic Structural Scheme Design
- Load Types and Scenarios
- Stability of Structures
- Mechanisms of Load Transfer
- Tributary Lengths, Areas, and Loadwidths
- Analysis of Portal Frame Structures
- Structural Materials
- Movement/Expansion Joints
- Preliminary Loading
- Load Takedown
- Car Park Layout Design
- Preliminary Analysis
- Preliminary Design of Elements
- Example Scheme Designs

1.3 Programme

This subject is examined by continuous assessment. Case Study reports are the basis for marking.

The subject is lectured in Semester 2 only as follows:

- Thursday 10:00 – 13:00
    - lecture covering these notes and other material;
    - group presentations of reports.

1.4 Studying

This subject is different to others. It is simply not possible to “cram” for, due its nature. Here is some advice to help you achieve your best.

This Subject is Different?

- This subject is unique – there are no “right” answers:
    - this is the difference between science & engineering;
    - this is why students find the subject difficult;
    - this also makes it difficult to teach!

- Teaching:
    - Learn from experienced engineers, through discussion and reading;
    - Lectures on course material;
    - Case-Studies and presentation.

Remember, contrary to what your education to-date may lead you to believe:

- Engineering is not an exact science;
- There are no “right” answers.

This will become apparent in the coming weeks!

So How Do I Learn?

The three best pieces of advice (and past students agree) are:

1. **Attend lectures**: simple, but for subjects like this, year-on-year poor results and attendance show a strong correlation.
2. **Take lots of notes**: a lot will be said – try not miss important points.
3. **Ask lots of questions**: both from you lecturers for this subject and from other lecturers or engineers you have access to.

Other important pieces are:

- Submissions:
    - Learn how to sketch – engineers communicate through sketches;
    - Do not try avoid the question by showing irrelevant details/information;
    - Text is not appreciated;
    - Learn how to visualize what you are proposing as a design;
    - Learn to understand the global and local behaviour of structures.

- Personally:
    - Be prepared to present to the class and speak clearly and loudly;
    - Learn to accept criticism in front of the class – in this way, everybody learns from each other's mistakes;
    - Contribute your fair share to the Case Study groups.

1.5 General Report Advice

From previous years' experience there are a number of common problems that occur and are given here.

What is expected

Overall:

- Understand structural behaviour (bending, axial, membrane action etc.)
- Appreciate allowances needed for lateral loads and thermal expansion
- Apply preliminary design rules of thumb (*span*/15, 50*N* etc.)
- Pay due regard to the 'Key Principles'

In particular:

- Lots of quick well-annotated sketches ("picture says a 1000 words")
- Appropriate sections/elevations/details e.g. RC column bar arrangement
- Appreciation of your 'numbers' (2T16 in a 500×500 RC Column?!)
- Use an appropriate top-down approach to your scheme, for example:
    1. stability/expansion joints
    2. column/beam layout
    3. approx sizes of important members
    4. load takedown
    5. prelim design of typical members
- Confidence in your knowledge and an ability to 'customise' design recommendations without compromising on strength requirements

What is not wanted

- Obvious details, e.g. "typical bolted end-plate connection", "cross-section through a masonry wall". There are no marks available for such details – don't waste your time!
- Lots of text – a sketch and a few bullet points will say it better
- 'Magic' numbers or formulae that just appear from nowhere – if it's an unusual (we haven't used in class say) formula briefly reference it.
- Overly detailed calculations, e.g. crack-width calculation for a beam.

General Advice

- Make sure your structure is stable in all 3-dimensions before moving on.
- Carry any expansion joints all the way through the structure down to ground level - remember they are being designed as totally separate structures that are very close to one another.
- Follow the "advice" in the problem, e.g. "minimum structural intrusion is expected" means don't put a column there unless absolutely necessary.
- Try visualising the structure and your solution. Play around with solutions in your head or on the paper before committing to one in particular.

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1.6 Report Requirements

The purpose of these guidelines is to help you avoid wasting time on irrelevant aspects and to prepare you for the workplace in 15 months' time.

Reports that do no conform to the following may or may not be accepted for submission, at the discretion of the lecturer.

General

- Submit one report per group;
- Reports should be covered (clear plastic to front) and bound with slide-on spine binder.
- Clearly identify the group letter on the front cover sheet;
- Each group member is to sign the front cover of the report;
- Each group member should retain a copy of the report for reference.

Content

- The report should consist of annotated sketches;
- In some cases a little text is appropriate: keep it to a minimum;
- In some cases a few calculations are appropriate; again keep to a minimum;
- Answer the question: do not provide irrelevant details!

Format

- Sketches are to be done on lightly squared paper, e.g. calculation pads;
- Any calculations are to be done on lightly squared paper also;
- The use of CAD is not recommended – it is better to improve your sketching;
- Text (excluding annotations) should be typed in the following style: Times New Roman, size 14 font; Justify alignment, and; double line spacing.

Length

- The report should be between 5 and 10 pages in length.

Submission

- Reports are to be submitted at the time stated on the problem;
- Late submissions will not be accepted – clients in the real world do not accept tardiness so start preparing for it now!

Presentation

- The report is to be photocopied onto acetate for presentation;
- The group must be ready to present and defend the report in class.

1.7 Reading Material

Reading about projects and new techniques will be a major part of your engineering career (CPD). By reading about the problems and solutions of various buildings you'll learn what works. Here is a good starting list – most of it will be in the library.

Institution of Structural Engineers:

- Manual for the design of reinforced concrete ("the Green book")
- Manual for the design of structural steelwork ("the Grey book")
- Manual for the design of masonry ("the Red book")
- Design recommendations for multi-storey and underground car parks (3^{rd} edition)
- Stability of buildings
- The Structural Engineer – fortnightly magazine, in the library

Join the IStructE for free at: http://www.istructe.org/students/.

Codes:

British Standards – BS8110, BS5950, BS5268, BS5628, BS6399

Eurocodes – EC1, EC2, EC3, EC4, EC5, EC6

Magazines:

Journal of the Institution of Engineers of Ireland

Institution of Civil Engineers: The New Civil Engineer (NCE) magazine

Company Magazines, e.g. Arup Journal

Books:

Homebond Manual – don't underestimate the complexity of a house

Library – some of the best are:

William Addis –

- *Structural engineering: the nature of theory and design*

- *The art of the structural engineer*

Angus J. Macdonald - *Structure and architecture*

Mario Salvadori – *Why buildings stand up*

Matthys Levy & Mario Salvadori – *Why buildings fall down*

J.E. Gordon –

- *Structures, or why things don't fall down*
- *The new science of strong materials, or why you don't fall through the floor*

Malcolm Millias – *Building Structures, 2nd Edn.*

Peter Rice – *An Engineer Imagines*

Trade Organisations:

Corus, The Concrete Centre, The Brick Development Association (Google them).

Bunf:

Suppliers' manuals (companies are very happy to send out their stuff to you)

- Kingspan/Tegral – industrial buildings
- Breton/Flood Flooring/Concast etc – precast concrete structures, precast slabs
- Bat – for timber connectors

People:

Your work placement company projects – talk to engineers you worked with.

Web:

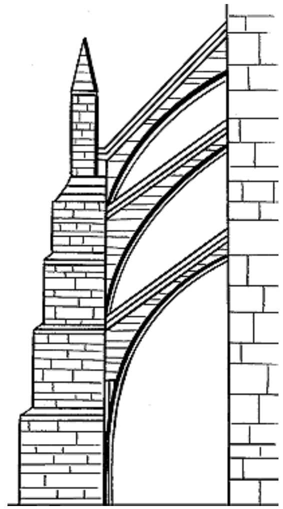
Almost everything to be found.

Talks:

Come to the talks organized by the IStructE, IEI etc. – look out for notices.

1.8 Some Introductory Problems

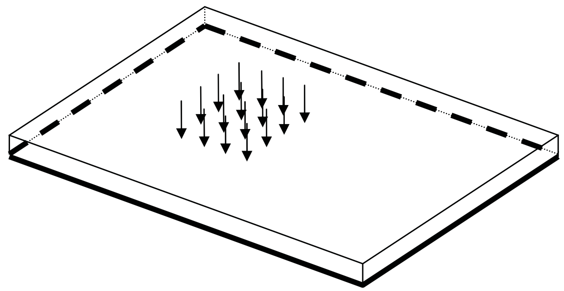
Easy:



1. What is the function of a buttress in a medieval Cathedral?
2. How does it work?

A bit harder:

1. In a rectangular 2-way spanning structure (be it slab/wall/steel plate etc), which is subject to a uniform area load and has simply supported sides, which side receives most load – the short, or the long side.
2. Why?



Tricky:

1. What are braced and unbraced frames?
2. Can a structure only have one type, or both?
3. When can you mix, or why can't you mix, these structures?

Hint: Once, you know what they are; consider the deflected shapes of each type.

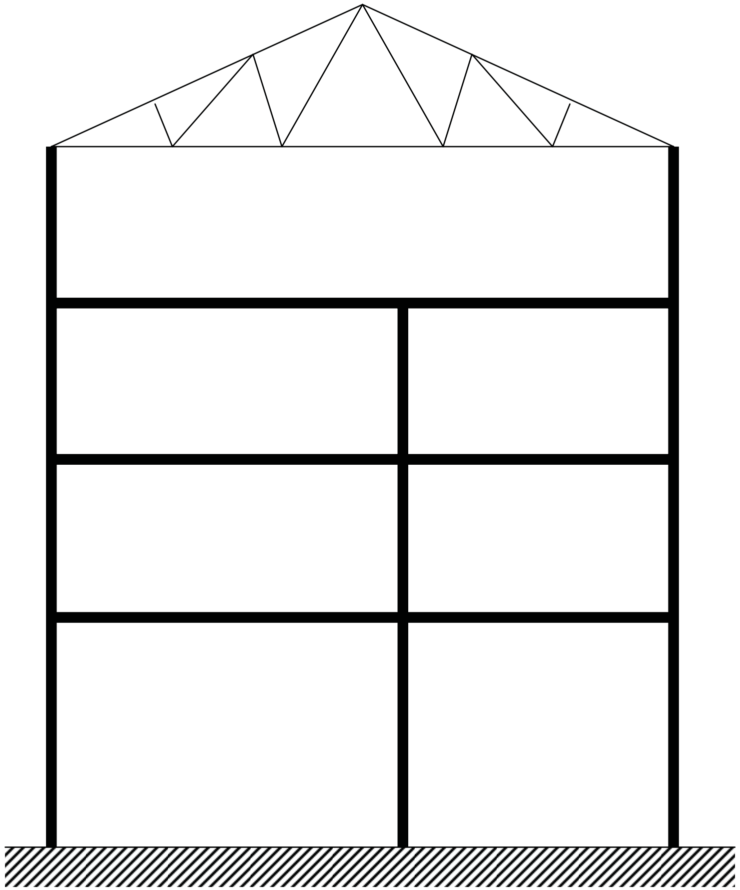
2. Overall Structural Behavior

2.1 Primary function of a structure

Institution of Structural Engineers:

> *"Structures…must safely resist the forces to which they may be subject."*

In the following structures identify the possible loads that need to be designed for.



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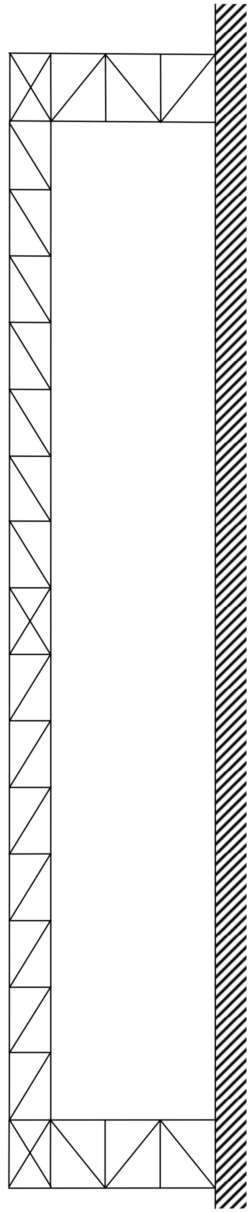
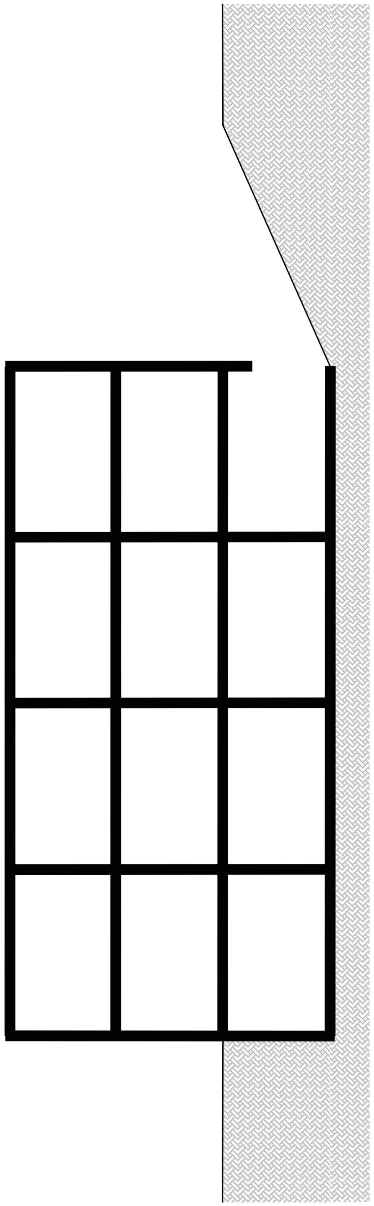
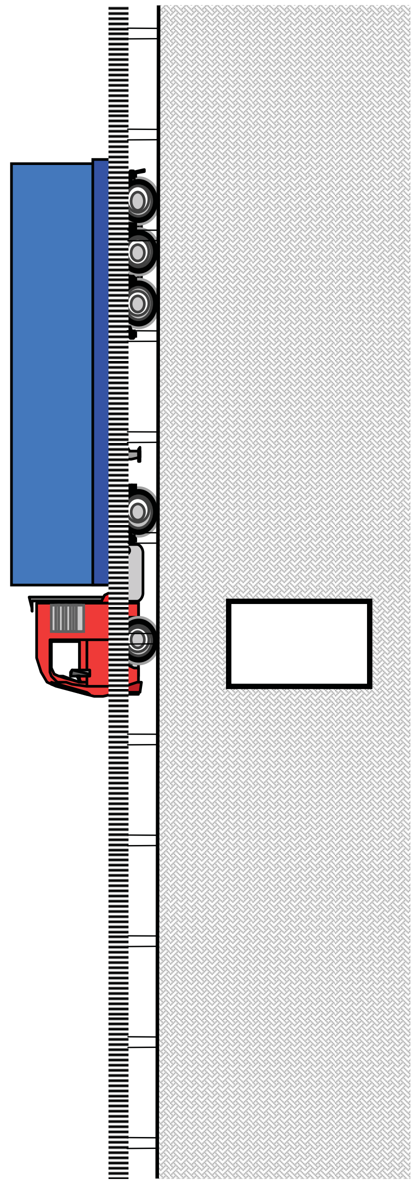
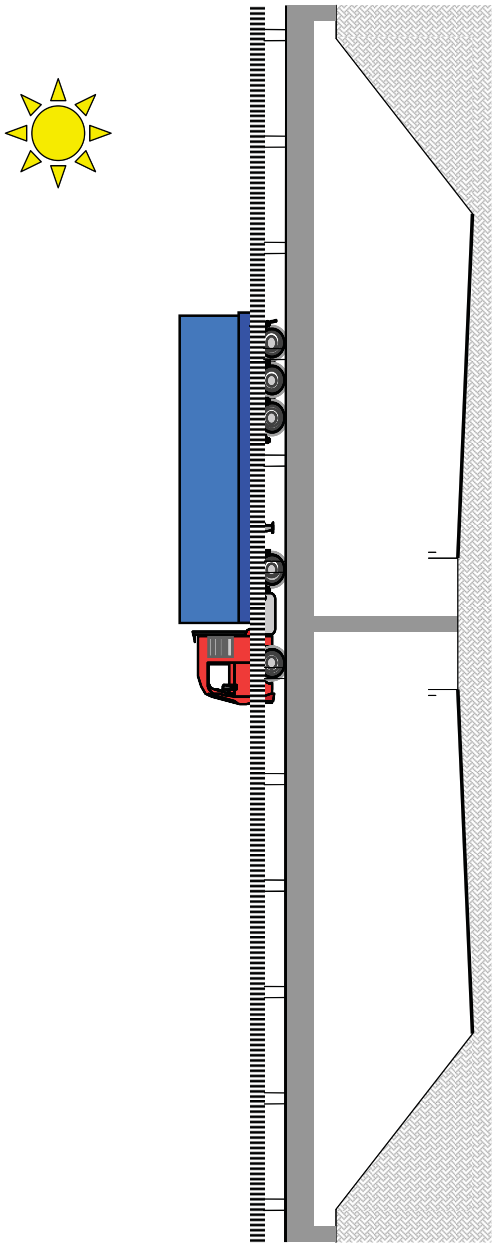






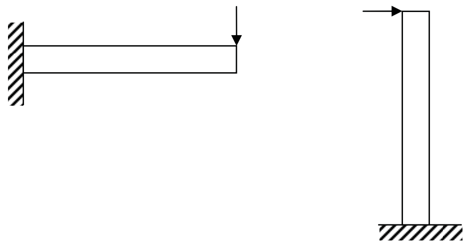


2.2 *Structural Members*

We refer to a range of possible structural materials under the following structural actions.

(i) Cantilever Action

Examples: balconies, canopies over doorways, older bridges

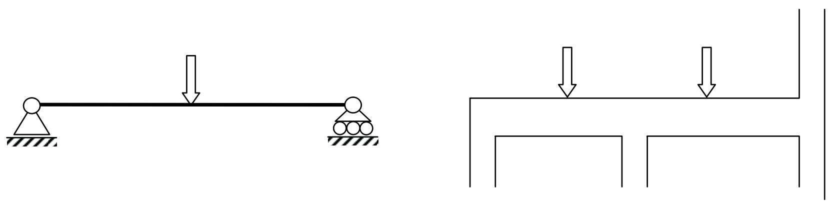


- Carry load by: ______________________________

- Span/depth ratio and/or load is about:______________________

- Materials:______________________________

(ii) Beam Action (bending between 2 or more supports)



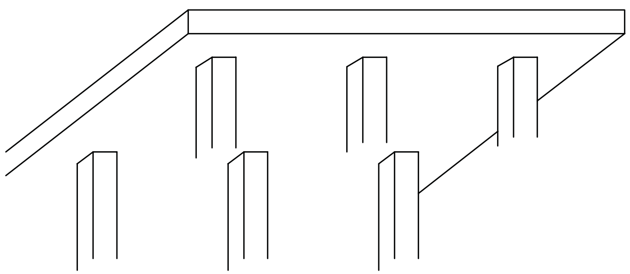
- Carry load by: ______________________________

- 'Normal' size beams are good for spans of: ________________________________

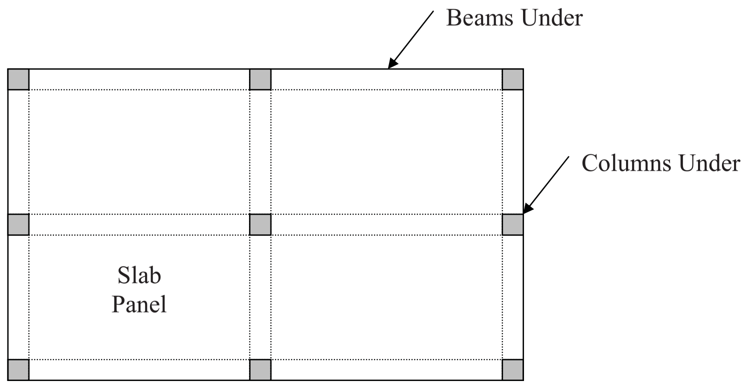
- Materials:____

(iii) Two-way Bending Action

Bending between 2 or more supports in each direction



Flat Slab



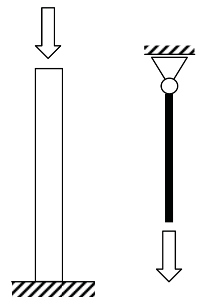
Beam and Slab

- Carry load by: ____

- Materials:____

(iv) One-way Axial Force Action

Examples: columns/struts, ties

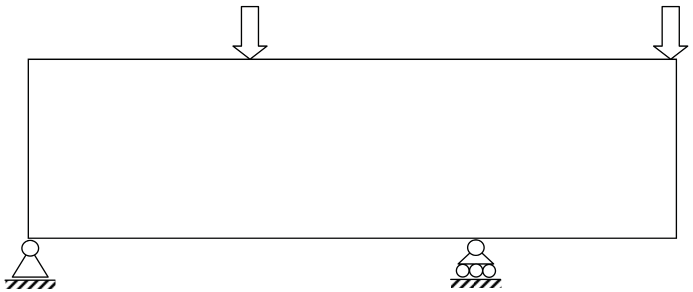


- Carry load by: ____________________________________

- Comment on the suitability of the following materials:

Material	Tension	Compression
Steel		
R.C.		
P.S.C.		
Timber		
Masonry		
Structural Glass		

(v) Diaphragm Action (a.k.a Membrane or Deep Beam Behaviour)



Membrane action

By definition, 'regular beam' action becomes 'deep beam' action when span/depth ratio goes less than about 2.

- For span/depth > 2, ______________________________ dominates
- For span/depth < 2, _______________________________ dominates
- Carry load by: ____
- Suitable materials include: ____
- Because: ____

2.3 *Structural Systems*

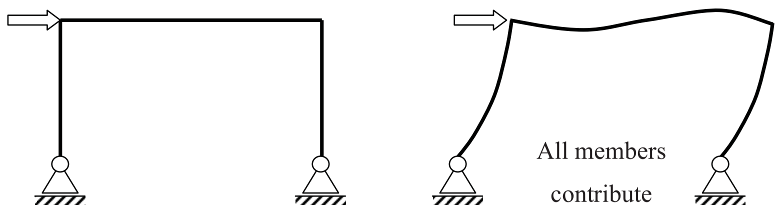
(i) Trusses

- Individual members undergo:____
- But the system overall carries load through:_________________________________
- More economical than beams for:____
- Comment on the suitability of the following materials:

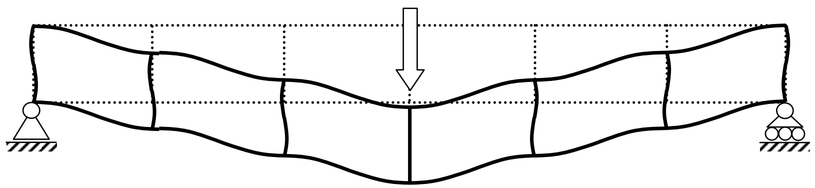
Material	**Comment**
Steel	
R.C.	
P.S.C.	
Timber	
Masonry	
Structural Glass	

(ii) Frames

Frame under lateral load:



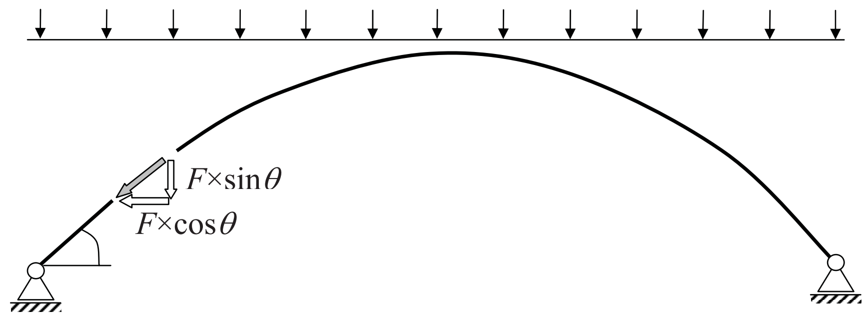
Frame action is what happens in Vierendeel Girders (named after Belgian engineer):



- Consists of groups of beams and/or columns:________________________________ connected together
- Individual members undergo:____
- In lateral loading, it does the job of a:_____________________________________
- In vertical loading, it does the job of a:____________________________________
- Comment on the suitability of the following materials:

Material	Comment
Steel	
R.C.	
P.S.C.	
Timber	
Masonry	
Structural Glass	

(iii) Arches



- Predominant action is:________________ but can also undergo_______________

- Used to span: ___________________________distances

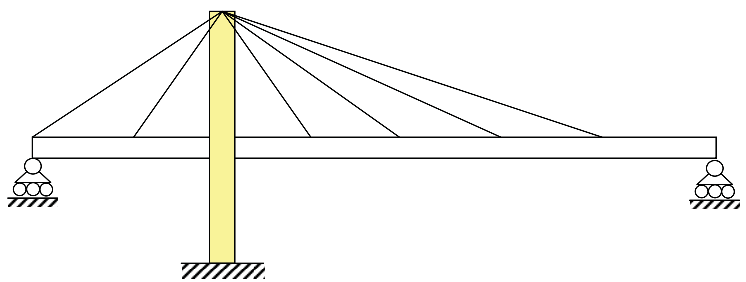
- Generates *substantial* horizontal thrust. This is the dominant limitation to the arch.

Horizontal thrust is equal to the horizontal component of the axial force in the arch at the point where it meets the support.

It is difficult to design foundations to take significant horizontal force unless they are in rock. Possible solutions are:

- provide a tie (tied arch);
- use an elliptical or half-circular arch so that $\theta = 90^o$.

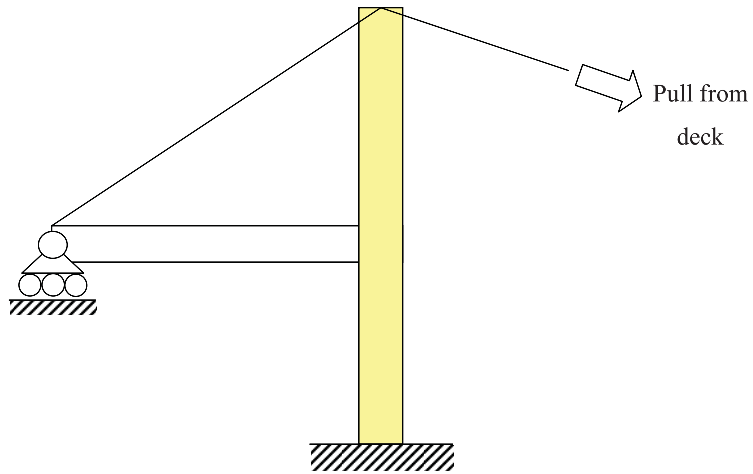
(iv) Cable Supported Structures



Cable-Stayed Bridge (e.g. William Dargan Bridge, Dundrum)

- A vertical tower is needed;
- Weights must be counterbalanced (otherwise, there will be a massive overturning moment on the support);
- The beam acts as a continuous beam with supports provided by the cables;
- In addition, the beam must resist some compression generated by the strut-tie arrangement:

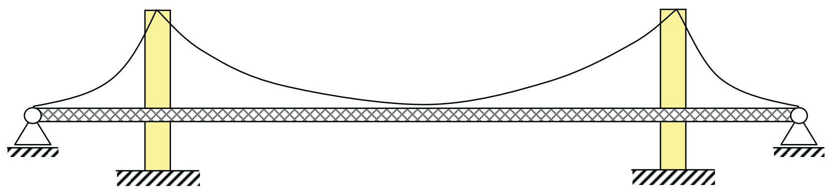
Draw the flow of forces: designate Compression as (⇦ ⇨); conversely for Tension.



- Used for bridges in 200 m to 400 m span range.

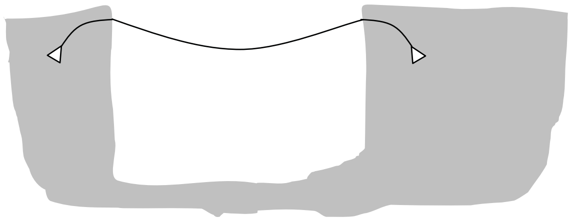
- Also used for roofs where there must be ___________________________________

(v) Catenary Structures



Suspension Bridge

- More difficult to construct than cable-stayed
- Capable of the longest spans
- Flexible - may need stiffening truss
- Stressed ribbon uses catenary action:



Stressed Ribbon Bridge (e.g. Kilmacannogue)

(vi) Fabric Structures

Fabric (cloth) has no flexural strength, i.e., it has no strength in bending. It buckles in compression. Its only strength is in tension.

Fabric is subject to 2-way axial force action, i.e., membrane action. However, it is different from a deep beam in that (a) it takes no compression and (b) the fabric deforms in response to load into a 2D catenary shape. These deformations are large compared with the geometry of the structure (think of a 10 m beam deflecting 2 m!). This change in shape changes the load path further, and hence the shape, and hence the loadpath… This phenomenon is known as geometric non-linearity.

Fabric roofing material is now available commercially.

Advantages/Disadvantages:

- very light self weight;
- great spans with ease (30 m in Berlin zeppelin hanger);

- it stretches;
- it has different strengths parallel and perpendicular to the weave;
- it must be stretched (prestressed);
- snow will tend to drift;
- long-term durability is improving as fabric technology improves.

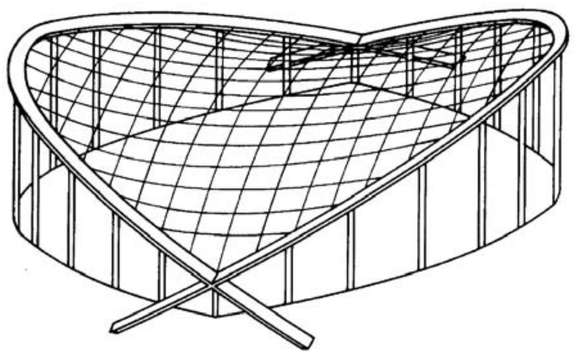
(vii) Air supported structures

It is possible to construct ‘buildings’ from fabric with no compressive members. The idea is based on maintaining a positive pressure inside the building – like a balloon. Air locks are provided at all entrances. Leaks will result in collapse if not repaired.

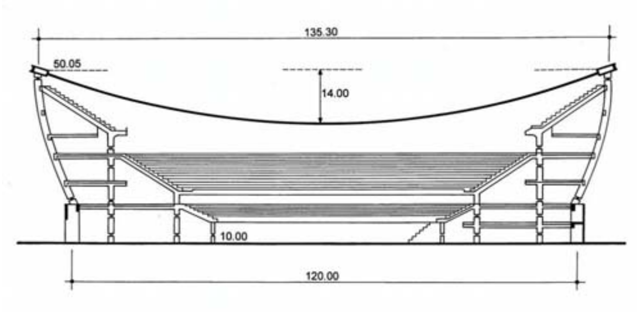
This principle has been used as ‘shuttering’ in buildings made from reinforced concrete!

(viii) Hyperbolic paraboloid

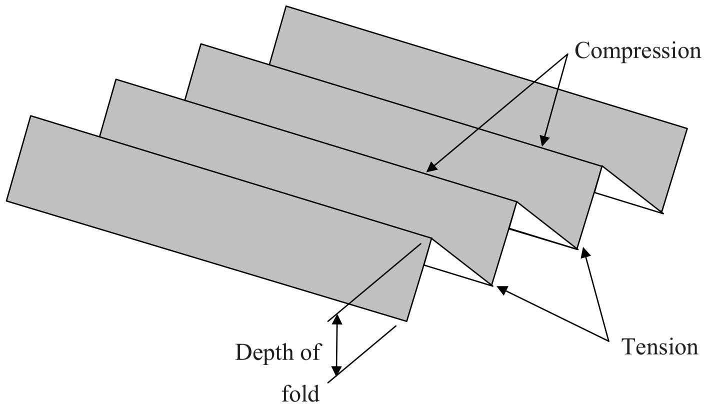
Roofs can have a hyperbolic paraboloid shape, i.e., parabola in one direction and hyperbola in the other (saddle shape). This shape is structurally efficient. The load is carried predominantly by membrane action.



The saddle dome ice hockey stadium in Calgary, Canada is a hyperbolic paraboloid and is made of prestressed concrete sections 600 mm deep.



(ix) Folded Plate



A folded plate is stronger than a flat plate as it acts like a beam/slab, taking tension in the lower members and compression in the upper members (if simply supported). Structurally, the depth of the fold is more important than the depth of the members. To see this in action make a paper model and load it. Note that it only spans one way to any extent.

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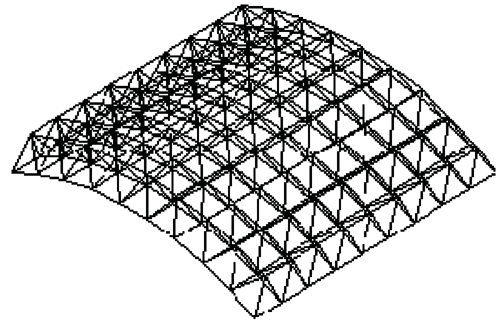
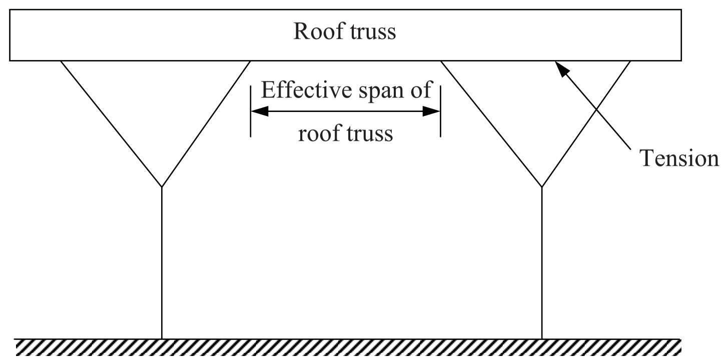
(x) Space Frames *(a misnomer - axial forces only)*

It is possible to have trusses in 2-Dimensions, i.e., 2-way spanning trusses. They come in proprietary forms. As for regular trusses, the systems acts in bending but the members are in compression or tension. They are structurally efficient (high strength-to-weight ratio) but expensive relative to more conventional forms.



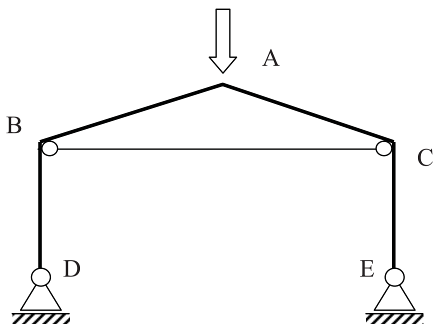
(xi) Tree structures (Stansted airport)

These are in effect a form of 3D truss system – a series of struts and ties. They are not particularly efficient although the inclined members do reduce the effective clear span of the roof truss.



Examples of Structural Members & Systems

Possible Answers: 1 = cantilever, 2 = beam/1-way bending, 3 = 2-way bending, 4 = 1-way axial force action, 5 = membrane/2-way axial force, 6 = truss, 7 = frame or Vierendeel, 8 = arch, 9 = cable supported, 10 = suspension/catenary.

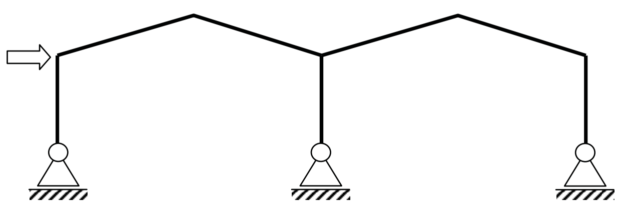


1. What is the structural action in transferring vertical load from A to B & C?

____

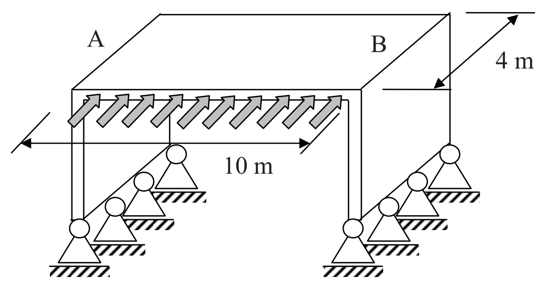
2. What is the action in transferring it from B & C to D & E?

____



3. What is the structural action in transferring horizontal load to the ground?

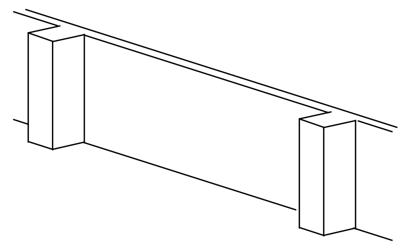
____



4. What is the structural action from the load to A & B?

____

5. and from A & B to the ground? ____



6. Free standing wall: how is the horizontal wind load transferred to the piers?

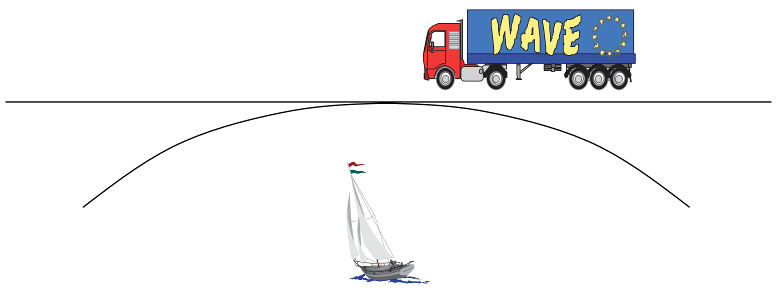
____

7. And how is it transferred from the piers to the ground?

____

8. Would you allow a damp proof course? ________________________________

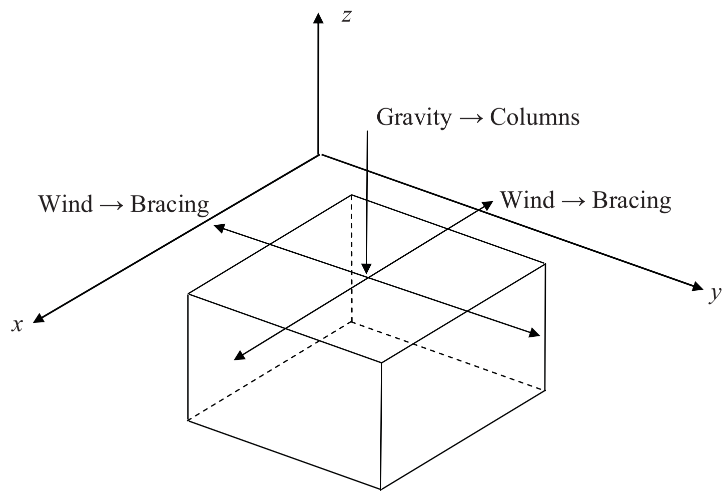
9. Indicate the types of support (fixed, pinned or sliding) you would specify at each support point on the arch bridge:



2.4 Stability of Buildings

Introduction

There are loads in all 3-dimensions of a building:



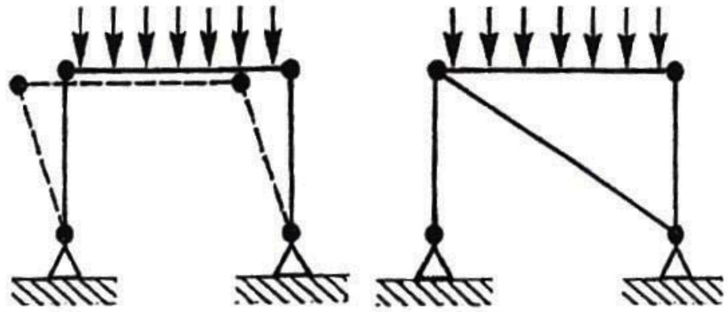
Note the following:

- load can act in either direction in each dimension;
- It's usually safe to ignore uplift of the whole structure, but not in the roof design or when a basement extends below the water table.

So loads in 5 directions must be resisted:

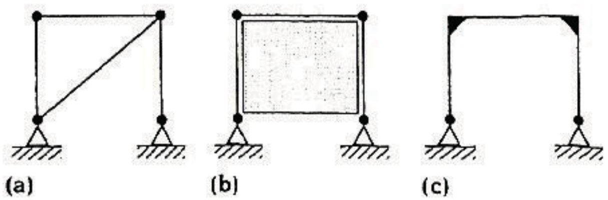
- Columns for vertical;
- Braced/unbraced frames for horizontal.

Even without loads in the lateral directions, bracing is required due to the inaccuracies of actual construction: buildings may not be perfectly plumb.

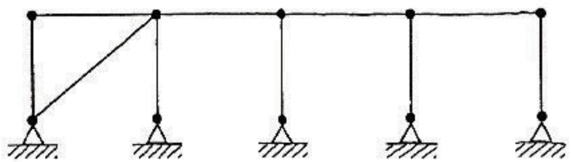


The frame on the left is in equilibrium, but is not stable (try to balance a pen, even on its 'fat' end!). The frame on the right is stable due to the addition of the diagonal member, even thought this additional member does not contribute to the vertical load carrying capacity of the structure.

Some ways to stabilize a single-bay frame are:



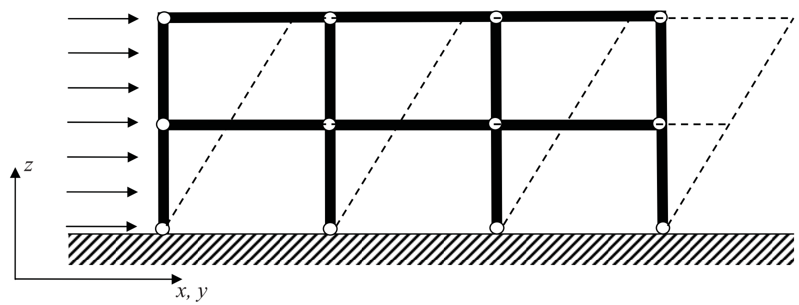
In this, (a) and (b) are termed 'braced' frames whilst (c) is referred to as a 'sway', or unbraced frame. A single braced bay can stabilize a row of bays:



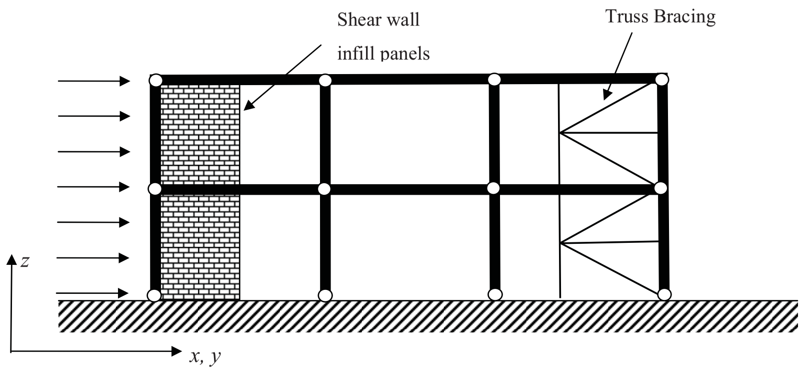
Definitions:

- *Braced Frame*:
    - load resisted through bending of large in-plane elements.
- *Unbraced or Sway Frame*:
    - load resisted through moment connections of framework – generally not used unless absolutely necessary due to the expense of the moment connections and the larger deflections.

Taking a 2-storey frame, unless we provide lateral stability we have:



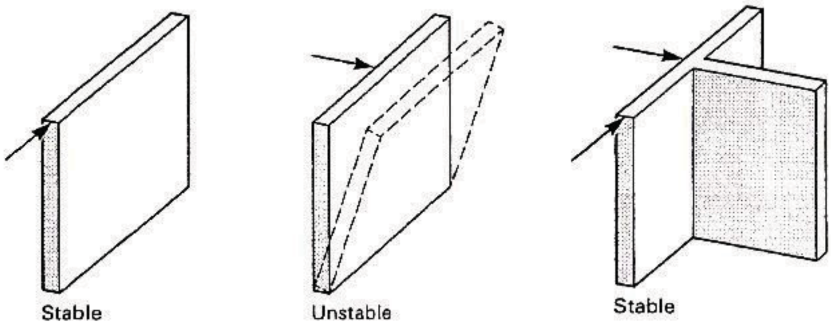
So we provide bracing similar to:



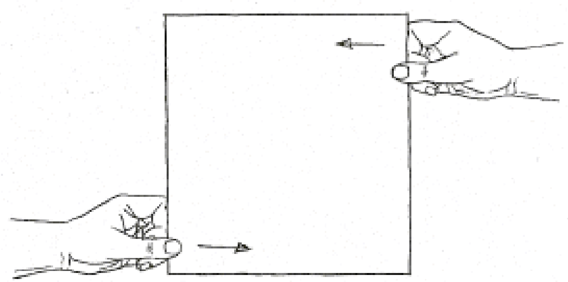
Important points to note:

1. Bracing is required at both ends *unless* the wind load (e.g.) can be transferred through the floors;
2. Bracing at both ends constrains thermal expansion – this may cause problems in a long structure;
3. Bracing is required in both dimensions, *x* and *y*, and must be able to resist load in each direction;
4. If there is bracing at both ends, bracing may be designed for a single load direction.

When considering walls as bracing, remember they can only take in-plane loads:



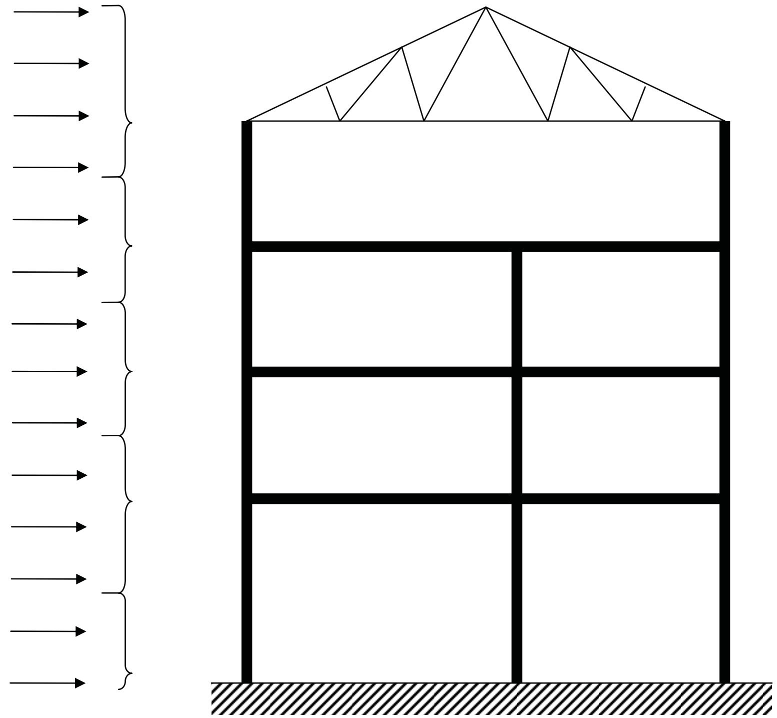
Just like a piece of card:



Lateral Load Paths

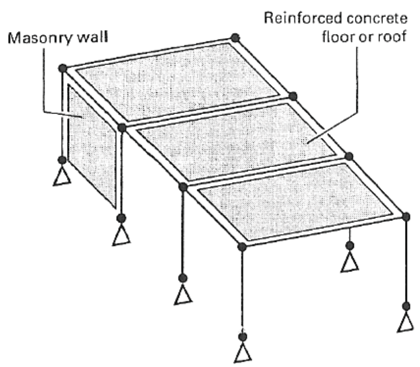
The wind load that needs to be resisted must take the following path:

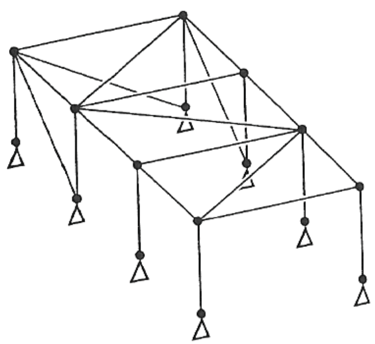
1. Wind hits façade (glazing or brickwork);
2. Façade spans between floors vertically;
3. Floors transfer load to the braced elements through diaphragm action;
4. Bracing takes load from each floor to ground.



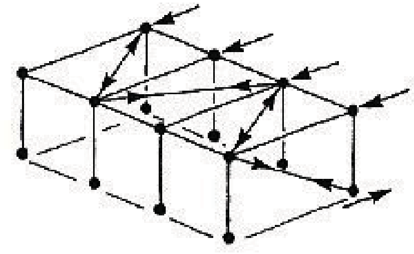
Sketch more of the lateral load path for yourself.

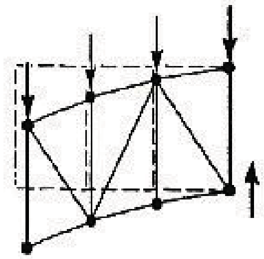
Identify the elements providing the load path in the following structures:

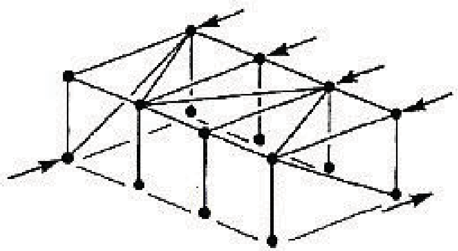


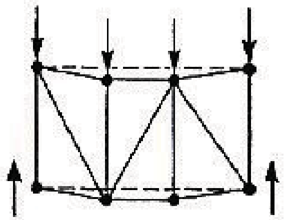


Write notes on the following figures and what they describe.

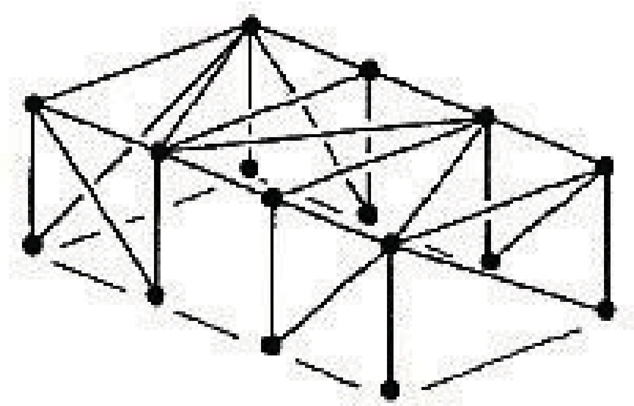


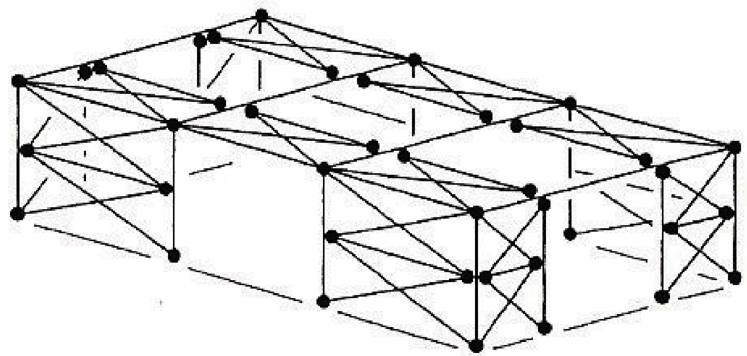






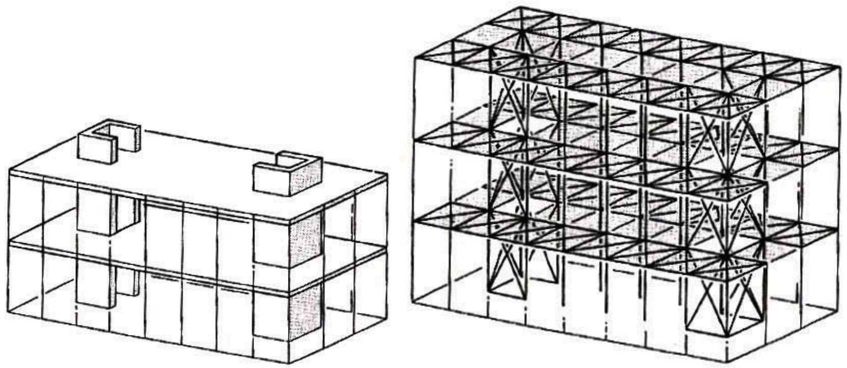
Are the following stable? – sketch the various load paths to check.





In Practice

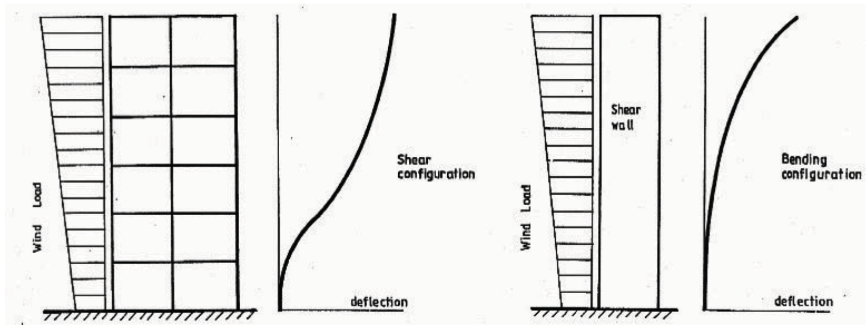
Actual structures have bracing systems that look along the lines of the following:

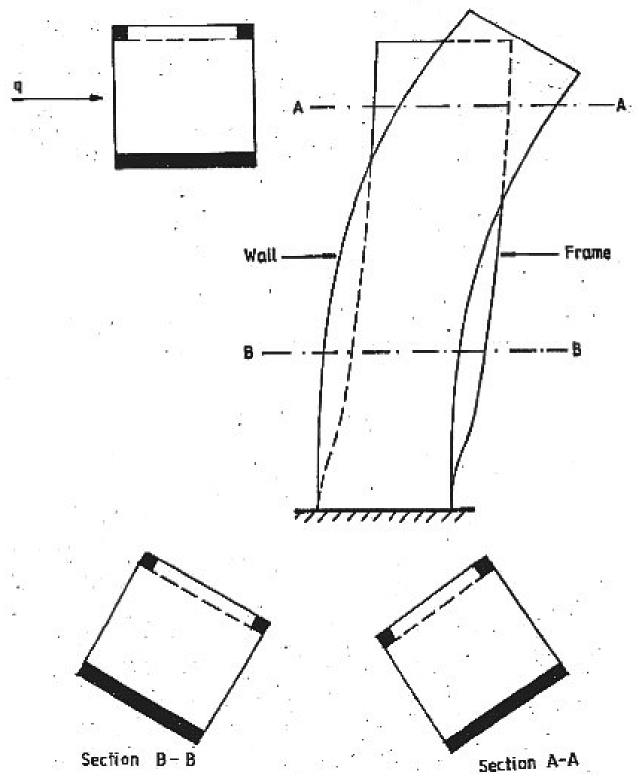


Obviously floor space in buildings is at a premium so structural designers try to use features that must be in the building for the lateral stability of the building. List some:

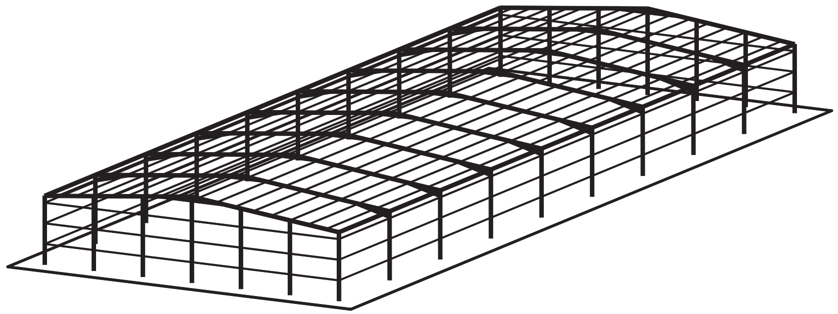
Mixing Bracing Systems in the Same Direction

What are the following diagrams telling us?



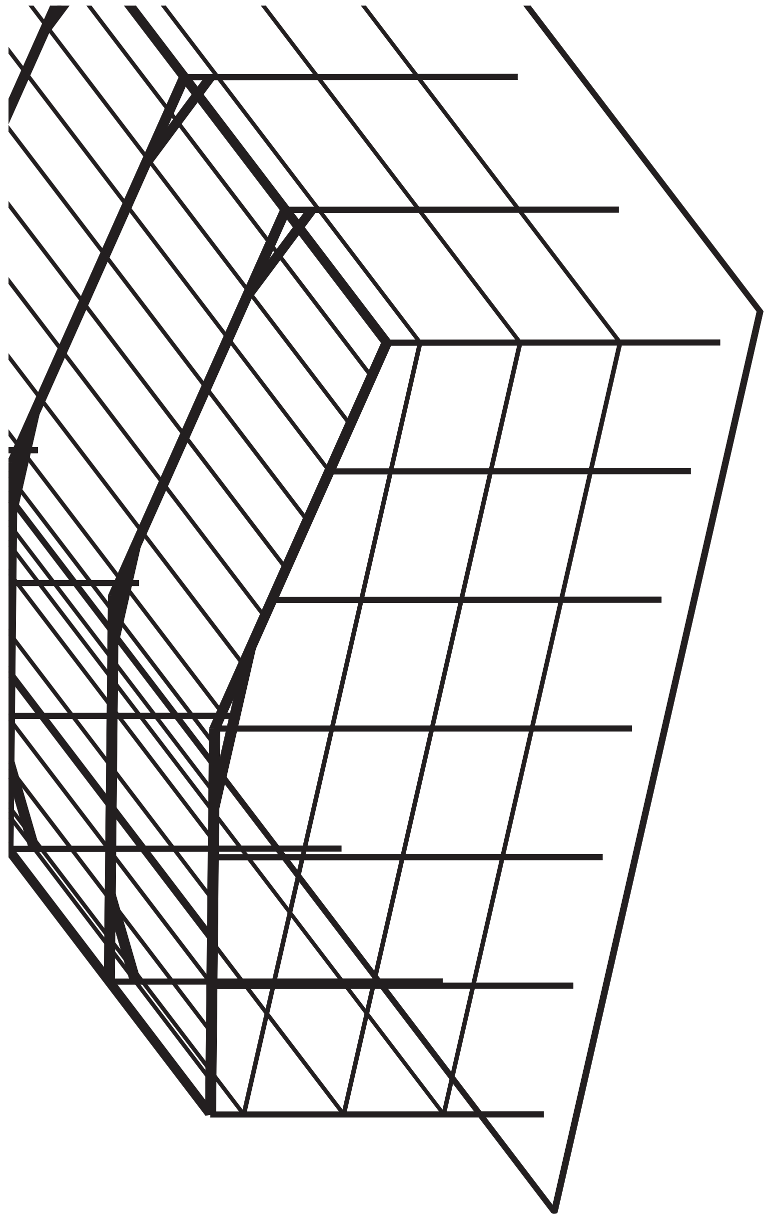


Lateral Stability of Portal Frame Structures



A portal frame structure (usually used in industrial estates etc.) is the most ordinary of buildings, yet is complex for lateral stability.

Write some notes on why this is, and sketch the required stability elements on the attached end gable. The end gable is not a portal frame.



2.5 *Allowing for Movement*

Joints are required to allow two parts of a structure move relatively, due to:

- Temperature;
- Moisture;
- Ground movements.

Movement joints are difficult to waterproof and detail – therefore minimize. Joints need to allow 15-25 mm movement.

Building Control joints:

Required to prevent cracking where a structure:

- or parts of a structure, are large;
- spans different ground conditions;
- changes height considerably;
- has a shape that suggests a point of natural weakness.

Important: Advice on joint spacing can be variable and conflicting, but here goes:

Structure type:	IStructE/Corus	Cobb	Anecdotal
Concrete	50 m 25 m: exposed RC	50 m 25 m: exposed RC	60 – 70 m
Steel – Industrial	125–150 m	100 – 150 m	
Steel – commercial	Simple: 100 m Continuous: 50 m	50 – 100 m	
Masonry		40 – 50 m	

Extracts

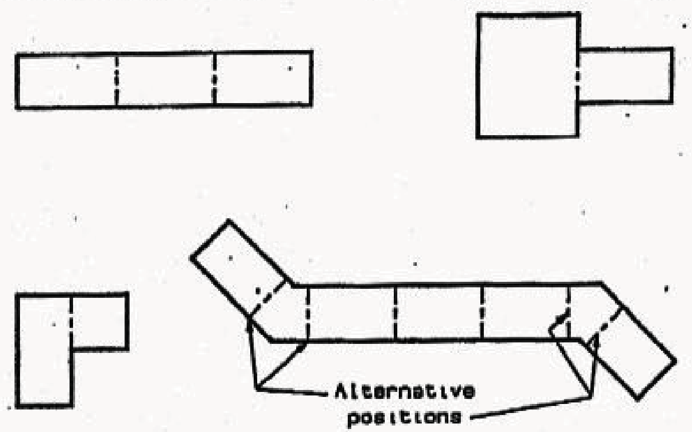
IStructE Green Book (RC)

2.4 Movement joints

Movement joints should be provided to minimize the effects of movements caused by, for example, shrinkage, temperature variations, creep and settlement.

The effectiveness of movement joints depends on their location. Movement joints should divide the structure into a number of individual sections, and should pass through the whole structure above ground level in one plane. The structure should be framed on both sides of the joint.

Some examples of positioning movement joints in plan are given in Fig. 1.



1 Location of movement joints

Movement joints may also be required where there is a significant change in the type of foundation or the height of the structure.

For reinforced concrete frame structures, movement joints at least 25mm wide should normally be provided at approximately 50m centres both longitudinally and transversely. In the top storey and for open buildings and exposed slabs additional joints should normally be provided to give approximately 25m spacing.

Attention should be drawn to the necessity of ensuring that joints are incorporated in the finishes and in the cladding at the movement joint locations.

IStructE Grey Book (Steel)

2.4 Movement joints

Joints should be provided to minimize the effects of movements arising from temperature variations and settlement. The effectiveness of movement joints depends on their location, which should divide the structure into a number of individual sections. The joints should pass through the whole structure above ground level in one plane. The structure should be framed on both sides of the joint, and each section should be structurally independent and designed to be stable and robust without relying on the stability of adjacent sections.

Joints may also be required where there is a significant change in the type of foundation, plan configuration or the height of the structure. Where detailed calculations are not made, joints to permit movement of 15 to 25mm should normally be provided at approximately 50m centres both longitudinally and transversely. For single-storey sheeted buildings it may be appropriate to increase these spacings. Attention should be drawn to the necessity of incorporating joints in the finishes and in the cladding at the movement joint locations.

In addition a gap should generally be provided between steelwork and masonry cladding to allow for the movement of columns under loading.

Corus

3.4 Expansion Joints

Advice on the need for expansion joints in steel framed buildings is covered in many reference documents. Unfortunately much of it is conflicting.

As temperature increases, steel expands. In BS5950:Part 1:1990, steel is quoted as having a coefficient of linear thermal expansion of 12×10^{-6} per degree centigrade. In the UK a temperature range of -5C to +35°C, or a variation from the mean of + or -20°C should be considered. There are two methods of dealing with expansion, the first is to allow expansion to occur and to provide movement joints as necessary to cope with it, dependant upon the size of the building. In this case for a 20°C change, the expansion is $20 \times 12 \times 10^{-6} \times 10^{3} = 0.24$mm per metre length. A building 100m long has a free expansion length of 50m, so each end would move $0.24 \times 50 = 12$mm. The calculated movement in an expansion joint would be double that of one section, that is 24mm for a 100m long building. Current general recommendations have largely been based on acceptable movements of + or -12mm, i.e. expansion joints at 50m spacing. The true situation is somewhat different from the theory. The second method of dealing with expansion is to constrain it, indeed the example quoted above neglects any constraint to expansion which a braced bay at the end of the building may provide. Clearly the only method which would allow full free expansion would be for the building to have a single vertically braced bay placed centrally within its length. This however is never practicable. Erection techniques for industrial buildings favour braced bays at or near the ends thus constraining any expansion, whether we want them to or not. The effect of constrained expansion can be measured as a stress induced in the steel, i.e. the Elastic Modulus of steel, $E = 205$ KN/mm^2, therefore the stress induced in a constrained member for a 20°C temperature change is $20 \times 12 \times 10^{-6} \times 205 \times 10^{3} = 49.2$N/mm^2 or say 50N/mm^2. This value could be used in combination with other loading considerations in the member design. BS5950 suggests a load factor of 1.2 for temperature effects. No further explicit advice is given, but a realistic load case would not combine a temperature rise of 20°C with a full roof snow load, for instance.

In reality a combination of free expansion and constraint will exist in most buildings. If we consider that most connections include bolts in clearance holes, a degree of free expansion capability exists. However with common practice including braced bays at the building extremities, expansion will always be, to a large degree, constrained. In practice buildings over 400m long have been constructed and are performing satisfactorily without expansion joints.

Further information on this topic is available in 'Steelwork Design Guide to BS5950: Volume 4: Essential Data for Designers,' published by the Steel Construction Institute. Table 2 gives a summary of recommendations for the spacing of expansion joints in steel framed buildings.

Table 2: Spacing of Expansion joints in Steel framed buildings

Single Storey Buildings	Generally	Note 1	150m
	Building subject to Higher Internal Temperatures	Note 1	125m
Multi-Storey Buildings	Simple Construction	Note 1	100m
	Continuous Construction	Note 2	50m
Roof Sheeting	Down the slope	Note 3	20m
	Along the slope		No Limit
Masonry Walls	Clay bricks	Note 4	15m
	Calcium Silicate bricks	Note 4	9m
	Concrete Masonry	Note 4	6m

Notes:

1. Where the stress due to the constraint of thermal expansion has been considered in the member design, no limit is necessary in simple construction.
2. Larger spacings are possible if the stresses due to the constraint of thermal expansion are considered in the member design.
3. Longer lengths are possible where provision for expansion is made.
4. For more detail refer to BS5628: Part 3: cl 20 and Appendix A.

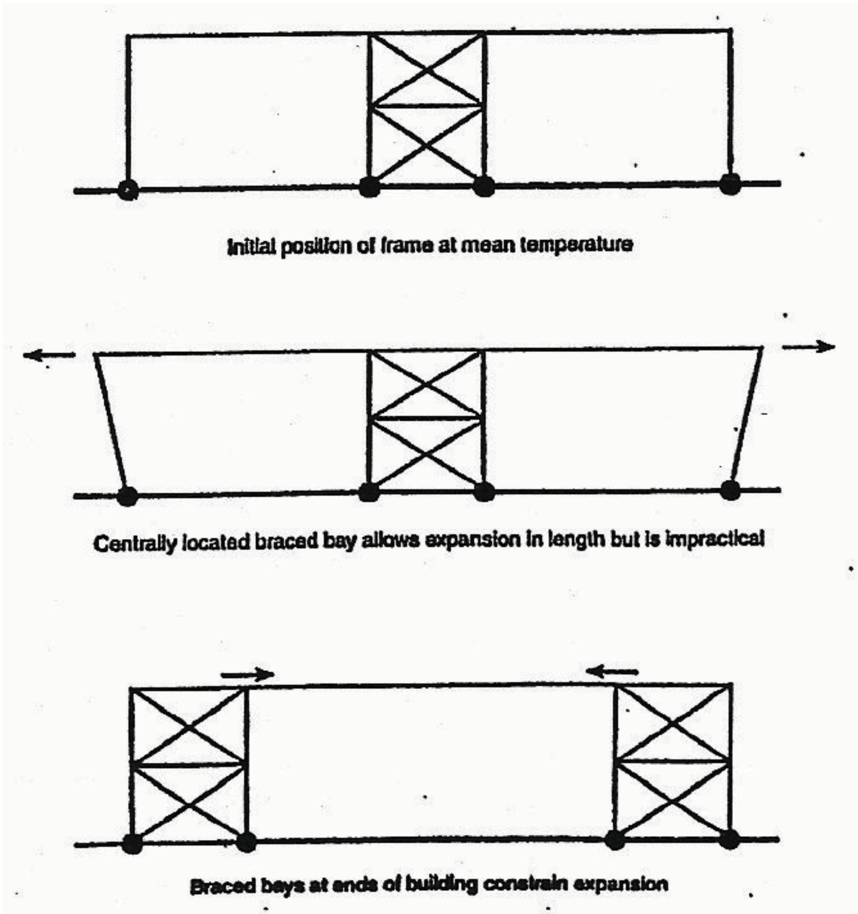


Figure 4: The effect of Braced Bays on Thermal Expansion

Movement joints

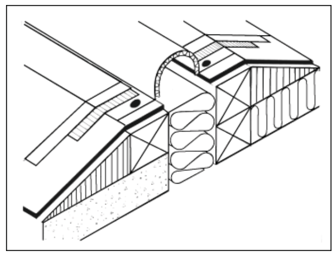
Used to divide structural elements into smaller elements due to local effects of temperature and moisture content.

Material	Spacing
Clay bricks	On plan: up to 12 m c/c (6 m from corners); Vertically: 9 m or every 3 storeys if *h* > (12 m or 4 storeys)
Concrete blocks	3 m – 7 m c/c
Steel roof sheeting	20 m c/c down the slope

Examples

Joint in an RC slab:

Joint in a roof:



Kansai Airport:

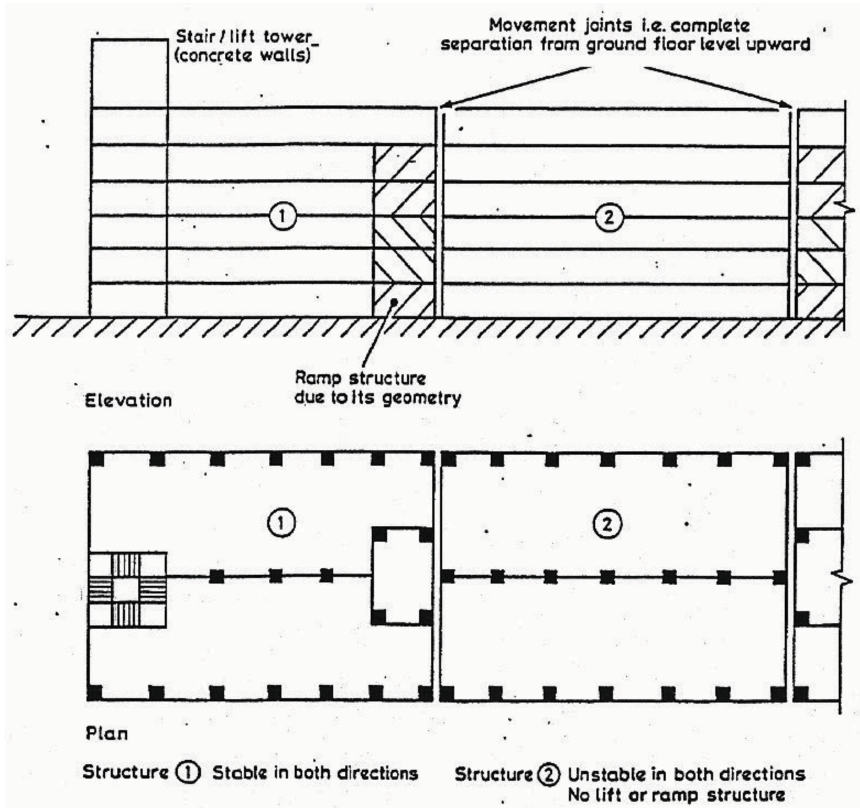
The building moves three-dimensionally in repose to potential stress from temperature shrinkage, earthquakes and uneven setting. In order to make the structure capable of absorbing deformation, expansion joints have been placed in 11 locations, at approximate intervals of 150m along the length of the 1.7km structure. The joints are 450-600mm in width.

Effect on Stability

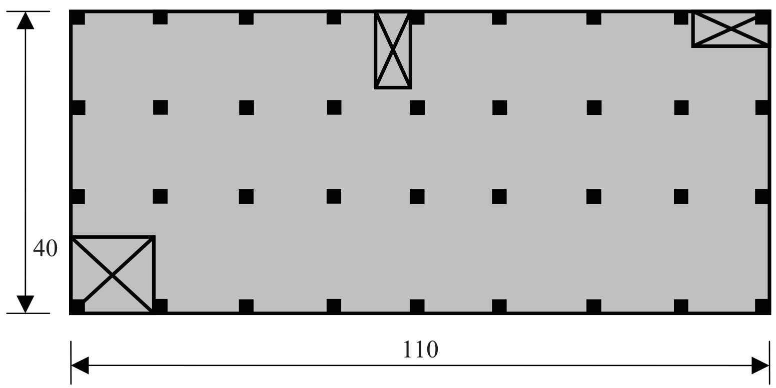
"*The positions of movement joints should be considered for their effect on the overall stability of the structure*" – Cobb

This has important implications:

- Every part of a structure must be stable in its own right;
- Just as columns are required in each portion separated by a movement joint, each portion must be capable of resisting horizontal load on its own.



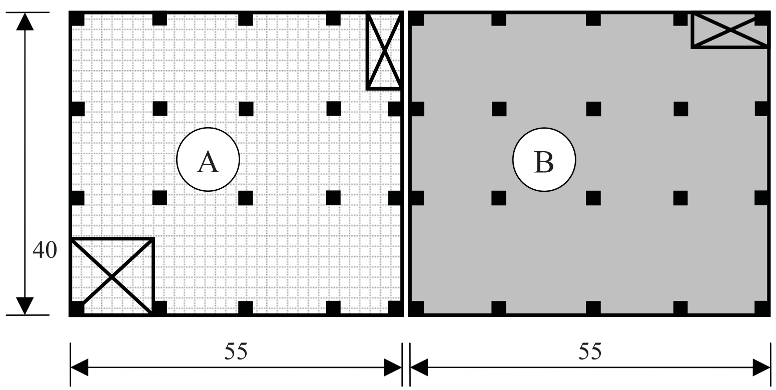
Example



Architect's Plan Showing Stair/Lift Cores

(Note: column layout is figurative only)

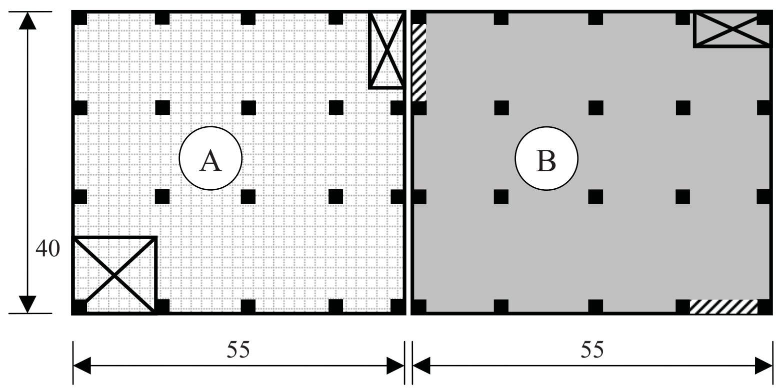
The Project Engineer decides to pursue an in-situ reinforced concrete option: balancing the extra cost and potential problems with each expansion joint, they choose to put one joint in, running N-S, at the mid-line, i.e., 55 m each side. No expansion joint is required in the E-W direction, as 40 < 50 m.



Engineer's Movement Joint Plan

Note:

- Double columns at interface – there are now 2 separate structures built very closely beside each other;
- Stability of each structure – clearly structure *A* is stable, whilst *B* is not.



Final Structural Layout

(Note: ////// indicates shear wall)

- Now structure-*B* is stable in both N-S and E-W directions;
- The shear walls may be RC, or masonry infill panels;
- Shear walls can have limited window opes, if required.

Case Study

Beaumont Hospital car park is an excellent example of lateral stability design, car park design, and building control joints. Similarly to the two walls that are very close together in the building just above, two *X*-braced frames either side of the control joint are clearly visible.

3. Structural Materials and Structural Form

> *"There are always many possible solutions, the search is for the best – but there is no best – just more or less good"*
>
> – **Ove Arup**
>
> *"If the structural shape does not correspond to the materials of which it is made there can be no aesthetic satisfaction"* – **Eduardo Torroja**

Every building and structure is unique in every way. The designers' solution reflects this, and must also be unique – an optimum balance of pros and cons sympathetic to the project. This is so both for the materials to be used, and for the type of construction to be used.

3.1 Choice of Structural Material

The notes that follow are an extract from *Reinforced & Prestressed Concrete Design, The Complete Process*, by E. J. OBrien & A. S. Dixon.

In reading these notes keep in mind:

- Different or extensions of existing materials, for example:
    - Reinforced masonry;
    - Glulam timber;
    - Hollow precast units;
    - Water-filled steel elements.
- What priorities do the different members of the design team assign to each of the criteria given in the notes?
- Industrial disputes also affect choice, most prominently though cost of labour.
- Most importantly, it should be evident that new techniques/methods/systems are always emerging – keep up to date.

IMPORTANT ADDENDUM

As an example of an aspect that has emerged since the writing of Prof. O'Brien's book, there is no mention of the environmental **sustainability** of the materials. In the space of a single generation, this is likely to become the governing factor in the choice of material. It will be up to you to keep abreast of such developments as your career progresses.

Introduction

The principal criteria which influence the choice of structural material are:

(a) strength;
(b) durability (resistance to corrosion);
(c) architectural requirements;
(d) versatility;
(e) safety;
(f) speed of erection;
(g) maintenance;
(h) cost;
(i) craneage.

The properties of reinforced and prestressed concrete are compared below with the properties of structural steel, timber and masonry under each of these nine headings. Typically, only one or two structural materials tend to be used in any given construction project to **minimise the diversity of skills required** in the workforce.

Strength

The relative strengths of the six main structural materials have already been discussed above. However, it should also be noted that the **ability of a material to sustain external loads is dependent on the mechanisms by which the loads are carried in a member**. For example, members which are in pure compression or tension will carry their loads more efficiently than members in bending since the stress is evenly distributed across the section (this will be seen in the following section). For this reason, the available strength of a structural material depends as much on the method of load transfer as its characteristic strength. Nevertheless, it can in general be stated that reinforced and prestressed concrete and structural steel are strong materials. Relative to these, timber and masonry are generally rather weak and are more suitable for short spans and/or light loads.

Durability

The durability of a material can be defined as its ability to resist deterioration under the action of the environment for the period of its design life. Of the four raw materials used in construction, steel has by far the least resistance to such corrosion (or rusting as it is more commonly known), particularly in aggressive humid environments. Hence, the durability of a structural material which is wholly or partly made from steel will largely be governed by how well the steel is protected.

A significant advantage of reinforced and prestressed concrete over other structural materials **is their superior durability**. The durability of the concrete itself is related to the proportions of its constituents, the methods of curing and the level of workmanship in the mixing and placing of the wet concrete. The composition of a concrete mix can be adjusted so that its durability specifically suits the particular environment. The protection of the steel in reinforced and prestressed concrete against the external environment is also dependent on the concrete properties, especially the porosity. However, its resistance to corrosion is also proportional to the amount of surrounding concrete, known as the cover, and the widths to which cracks open under day-to-day service loads.

Structural steel, like concrete, is considered to be very durable against the agents of wear and physical weathering (such as abrasion). However, one of its greatest drawbacks is its **lack of resistance to corrosion**. Severe rusting of steel members will result in a loss in strength and, eventually, to collapse. The detrimental effect of rusting is found to be negligible when the relative humidity of the atmosphere is less than approximately 70 per cent and therefore **protection** is only required in unheated temperate environments. Where corrosion is likely to be a problem, it can often be prevented by protective paints. Although protective paints are very effective in preventing corrosion, they do **add significantly to the maintenance costs** (unlike concrete for which maintenance costs ire minimal).

For **timber** to be sufficiently durable in most environments it must be able to resist the natural elements, insect infestation, fungal attack (wet and dry rot) and extremes in temperature. Some timbers, such as cedar and oak, possess natural resistance against deterioration owing to their density and the presence of natural oils and resins. However, for the types of timber most commonly used in construction, namely softwoods, some form of **preservative** is required to increase their durability. **When suitably treated, timber exhibits excellent properties of durability.**

Masonry, like concrete, can also be adapted to suit specific environments by selecting more resistant types of blocks/bricks for harsh environments. Unreinforced **masonry is particularly durable** and can last well beyond the typical 50 year design life.

Architectural requirements

The appearance of a completed structure is the most significant architectural feature pertinent to material choice since the aesthetic quality of a completed structure is largely determined by the finish on the external faces.

For concrete, this final appearance is dependent on the **standards of placement and compaction** and the quality of the formwork. Badly finished concrete faces, with little or no variation in colour or texture over large areas, can form the most unsightly views. Concrete is a versatile material, however, and when properly placed, it is possible to produce structures with a wide variety of visually appealing finishes In the case of precast concrete, an excellent finished appearance can usually be assured since manufacture is carried out in a controlled environment.

Exposed structural steel in buildings is displeasing to the eye in many settings and must be covered in cladding in order to provide an acceptable finish. An exception to this is the **use of brightly painted closed, hollow, circular or rectangular sections**.

Timber and masonry structures will generally have an **excellent finished appearance**, providing a high quality of workmanship is achieved. Masonry also offers a sense of scale and is available in a wide variety of colours, textures and shapes. In addition to their aesthetic fatalities, concrete and masonry structures also have the advantage of possessing good sound and thermal insulation properties.

Versatility

The versatility of a material is based as its ability (a) to be **fabricated in diverse forms** and shapes and (b) to undergo substantial **last-minute alterations** on site without detriment to the overall design. Steel can easily be worked into many efficient shapes on fabrication but is only readily available from suppliers in standard sections. **Concrete** is far more versatile in this respect as it can readily be **formed by moulds into very complex shapes**. Timber is the most limited as it is only available from suppliers in a limited number of standard sides. **Laminated timber, on the other hand can be profiled and bent into complex shapes.** Masonry can be quite versatile since the dimensions of walls and columns can readily be changed at any time up to construction. The disadvantage of steel, timber and precast concrete construction is their lack of versatility on site compared with in situ reinforced concrete and masonry to which substantial last-minute changes can be made. In situ prestressed concrete is not very versatile as changes can require substantial rechecking of stresses.

Safety

The raw material of concrete is very brittle and failure at its ultimate strength can often occur with little or no warning. **Steel**, being a very ductile material, will undergo **large plastic deformations** before collapse, thus giving **adequate warning** of failure. The safety of reinforced concrete structures can be increased by providing **'under-reinforced' concrete members** (the concepts of under-reinforced and over-reinforced concrete are discussed in Chapter 7). In such members, the ductile steel reinforcement effectively fails in tension before the concrete fails in compression, and there is considerable deformation of the member before complete failure. Although timber is a purely elastic material, it has a very low stiffness (approximately 1/20th that of steel) and hence, like steel, it will generally undergo considerable defection before collapse.

An equally important aspect of safety is the **resistance of structures to fire**. Steel loses its strength rapidly as its temperature increases and so steel members must be protected from fire to prevent collapse before the occupants of the structure have time to escape. For structural steel, protection in the form of intumescent paints, spray-applied cement-binded fibres or encasing systems, is expensive and can often be unsightly. Concrete and masonry possess fire-resisting properties far superior to most materials. In reinforced and prestressed concrete members, the concrete acts as a protective barrier to the reinforcement, provided there is sufficient cover. Hence, concrete members can retain their strength in a fire for sufficient time to allow the occupants to escape safely from a building. **Timber**, although combustible, does not ignite spontaneously below a temperature of approximately 500 °C. At lower temperatures, timber is only charred by direct contact with flames. The **charcoal layer** which builds up on the surface of timber during a fire protects the underlying wood from further deterioration and the structural properties of this 'residual' timber remain unchanged.

Speed of erection

In many projects, the speed at which the structure can be erected is often of paramount importance due to restrictions on access to the site or completion deadlines. In such circumstances, the **preparation and fabrication of units offsite will significantly reduce the erection time**. Thus, where precast concrete (reinforced and/or prestressed) and structural steel are used regularly, the construction tends to be very fast Complex timber units, such as laminated members and roof trusses, can also be fabricated offsite and quickly erected.

The construction of in situ concrete structures requires the fixing of reinforcement the erection of shuttering, and the castings, compaction and curing of the concrete. The shutters can only be removed or 'struck' when the concrete has achieved sufficient strength to sustain its self-weight. During the period before the shutters can be struck, which can be several days, very little other construction work can take place (on that part of the structure) and hence the overall erection time of the complete structure tends to be slow. **Masonry construction**, though labour intensive, **can be erected very rapidly** and the structure can often be built on after as little as a day.

Maintenance

Less durable structural materials such as structural steel and timber require treatment to **prevent deterioration**. The fact that the treatment must be repeated at intervals during the life of the structure means that there is a **maintenance requirement** associated with these materials. In fact, for some of the very large exposed steel structures, protective paints must be applied on a continuous basis. Most concrete and masonry structures require virtually no maintenance.

An exception to this is structures in particularly harsh environments, such as coastal regions and areas where do-icing salts are used (bridges supporting roads). In such cases, regular inspections of reinforced and prestressed concrete members are now becoming a standard part of many maintenance programmes.

Cost

The cost of structural material is of primary interest when choosing a suitable material for construction. The relative cost per unit volume of the main construction materials will vary between countries. However, the overall cost of a construction project is not solely a function of the unit cost of the material.

For example, although concrete is cheaper per unit volume than structural steel, reinforced concrete members generally require a greater volume than their equivalent structural steel members because of the lower strength of concrete.

As a consequence, reinforced concrete can become the more expensive structural material. If reinforced concrete members are cast in situ, constructions costs tend to be greater than for the steel structure because of the longer erection time and the intensive labour requirements. However, the high cost of structural steel and its protection from corrosion and are counteract any initial saving with the result that either material can be more cost effective. In general, it is only by comparing the complete cost of a project that the most favourable material can be determined. As a general guide, however, it can be said that reinforced concrete and structural steel will incur approximately the same costs, masonry will often prove cheaper than both where it is feasible while the cost of timber is very variable.

Craneage

In certain circumstances, the choice of structural material and construction method may be determined by the **availability of craneage**. For example, in a **small project**, it may be possible to **avoid the need for cranes by the use of load-bearing masonry walls and timber floors**. Depending on their weight and size, structural steel and precast concrete units may require substantial craneage and it is often the limit on available craneage that dictates the size of such units.

In general, in situ concrete requires little craneage although cranes, when available, can be used for moving large shutters.

The table below serves as a summary of the relative advantages and disadvantages of the four types of structural material under the categories discussed above. At this stage, it should be appreciated that the choice of any structural material is heavily dependent on the particular structure and the conditions under which it is constructed.

Sustainability

Add in here some notes on the class discussion:

Comparison of the structural properties of concrete (reinforced and prestressed), structural steel, timber and masonry.

	Concrete (RC and PC)	**Structural Steel**	**Timber**	**Masonry**
Strength	Excellent	Excellent	Fair	Good, except in tension
Durability	Excellent	Poor against corrosion*	Poor*	Excellent
Appearance	Fair	Fair	Excellent	Excellent
Safety	Excellent	Poor fire resistance*	Good	Excellent
Speed of erection	Slow for in situ	Very fast	Very fast	Very fast but labour intensive
Versatility	Excellent for in situ, poor otherwise	Poor	Fair	Very good
Sustainability				

*unless protected

Order of priority given to the material evaluation criteria by different design team members

Priority	Client	Contractor	Architect	Structural Engineer	Building Services Engineer	Quantity Surveyor
1						
2						
3						
4						
5						
6						

3.2 *Choice of Structural Form*

Key Principles in Choosing Structural Form

All of the Case Studies, though on different topics, try to show that there are a number of factors that contribute, in different measures, to the structural scheme adopted. Also, it will be clear that there is no perfect answer – simply a weighted balance of the pros and cons of any given solution. Factors include:

1. **Technical Requirements**
   - Structure Scale:
     - Stability in all directions – Vertical and Orthogonal Horizontals
     - Accommodation of movement – either by joints or stress design
     - Global load paths are identified
   - Element Scale:
     - Proportional sizes, e.g. span/d ratios or N/20 etc.
     - Global actions are allowed for in the element scheme

2. **Economic Requirements**
   - Materials (Refer to the handout):
     - Raw cost – can it be locally sourced?
     - Placement cost – e.g. block layers are expensive currently
     - Transport of fabricated elements – special requirements?
   - Constructability
     - Is the structure repeatable as possible
     - Minimum number of trades on site
     - Transport/craneage appropriate for the material considered?

3. Functional Requirements

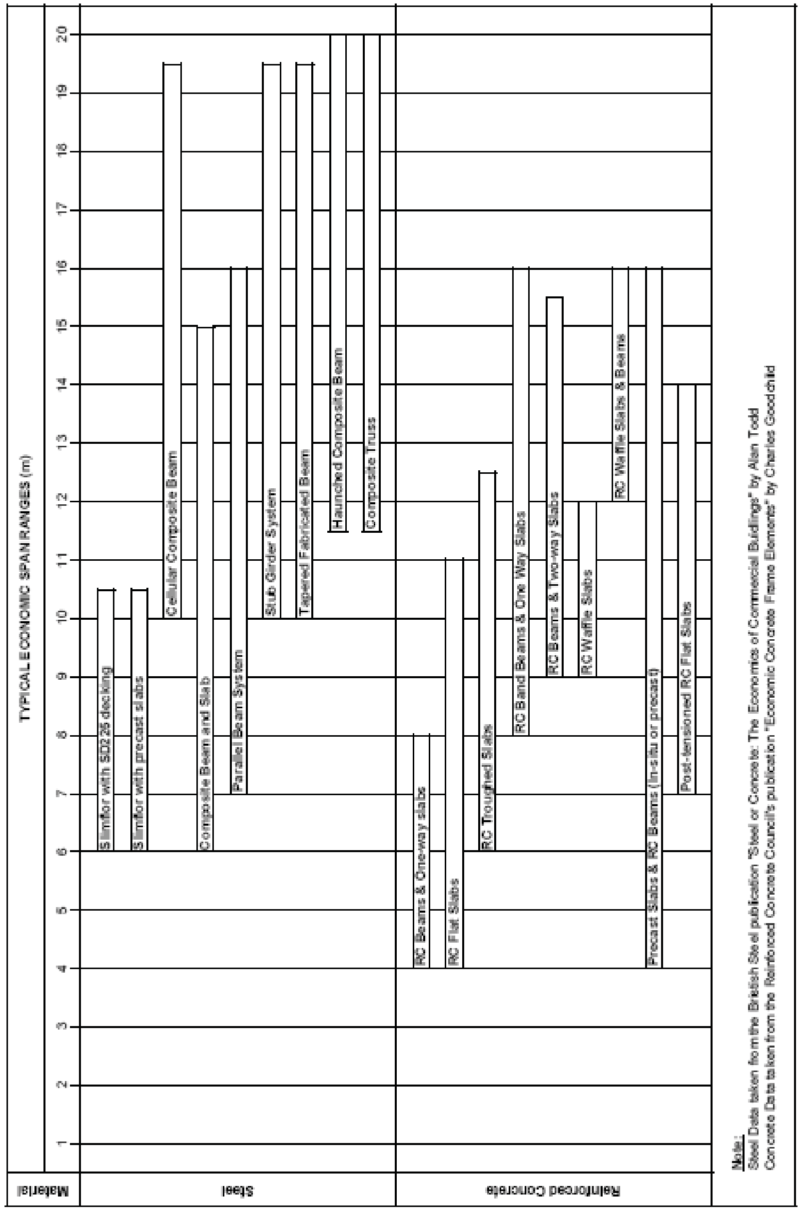
- Building Service Integration:
  - Expect holes in beams – allow for it early on
  - Flat soffits are beneficial in heavily serviced buildings
- Client's focus:
  - Speculative commercial will require clear spans for example
  - Landmark headquarters will possibly mean a dramatic structure
- Architecture:
  - Complement the architecture if possible
  - Get involved as early as possible in the design
- Planning:
  - Minimise structural depths if required
  - Drainage schemes to be appropriate to site and local drainage
  - Environmental considerations

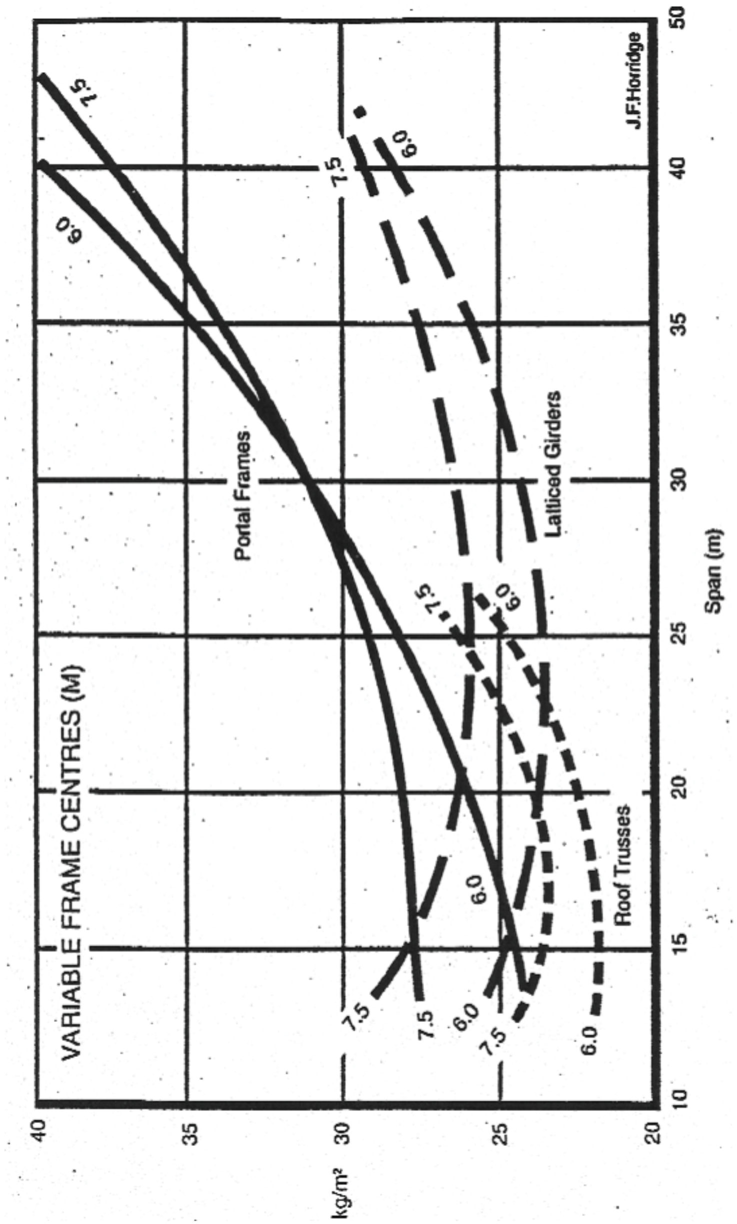
Again, where does **sustainability** fit into the above decision-making process?

Choice of Form

The span of the structure is the main consideration. For the two usual forms of construction, the first of the following charts advises what forms of construction are appropriate for what spans for steel and concrete.

The second chart gives a comparison of the weights of structure required for various spans and types of construction for single-storey steel buildings. These buildings tend to be extremely well engineering economically.



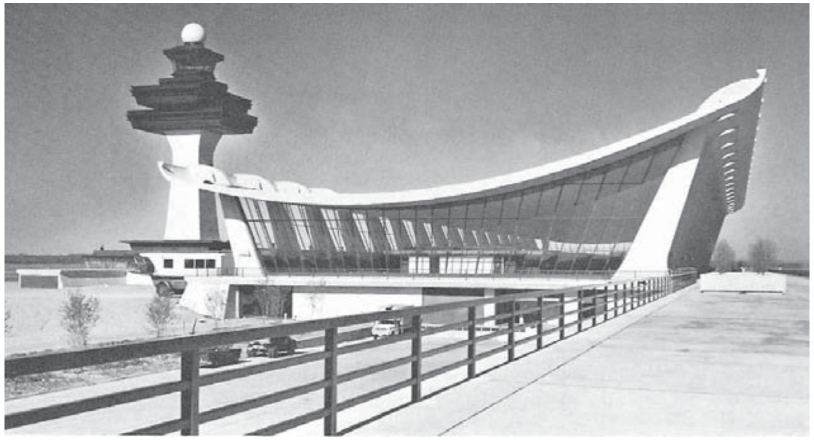


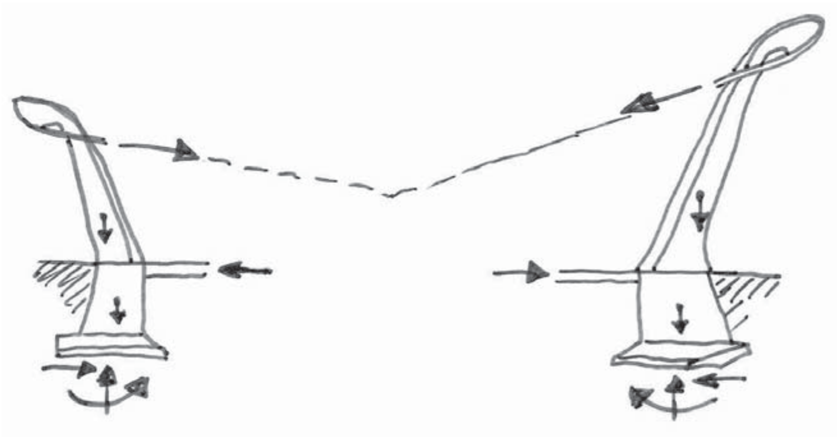
4. Precedence Studies

4.1 Introduction

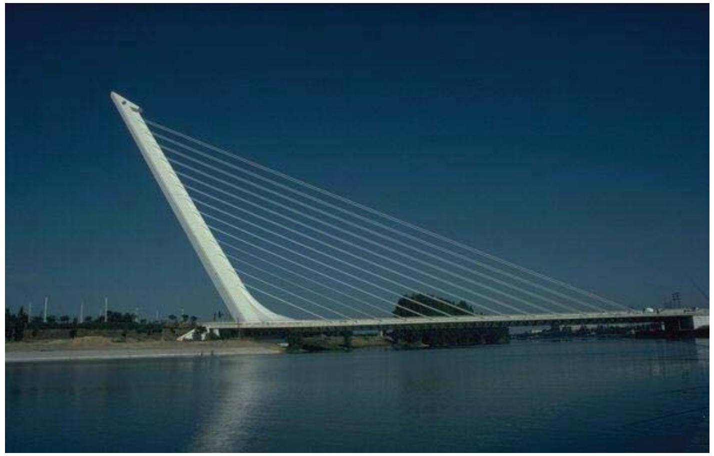
In this section we have a look at some interesting structures of the past and see how they integrate the preceding ideas of stability, material and form to achieve economic aesthetic and stunning solutions to the design brief.

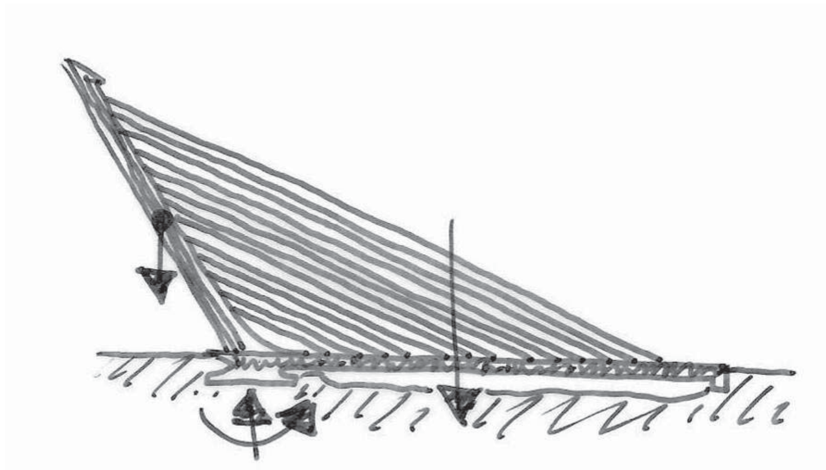
4.2 Dulles Airport



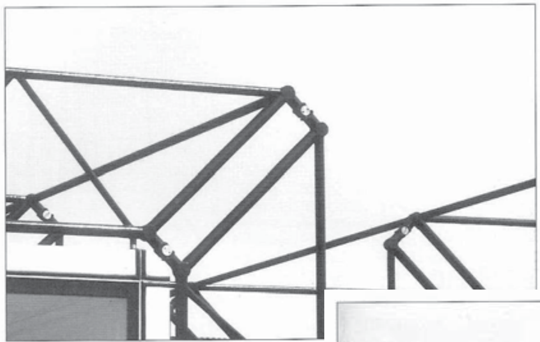


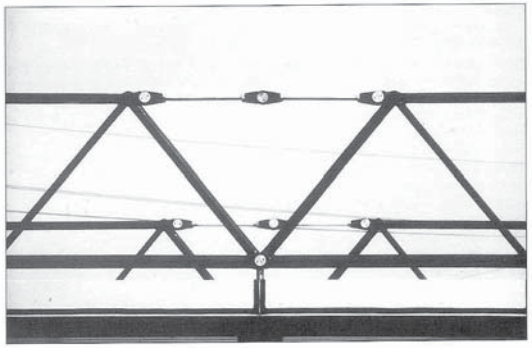
4.3 Alamillo Bridge

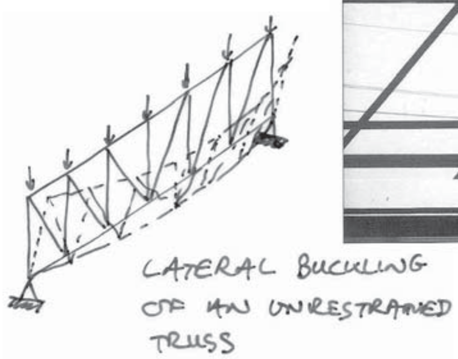


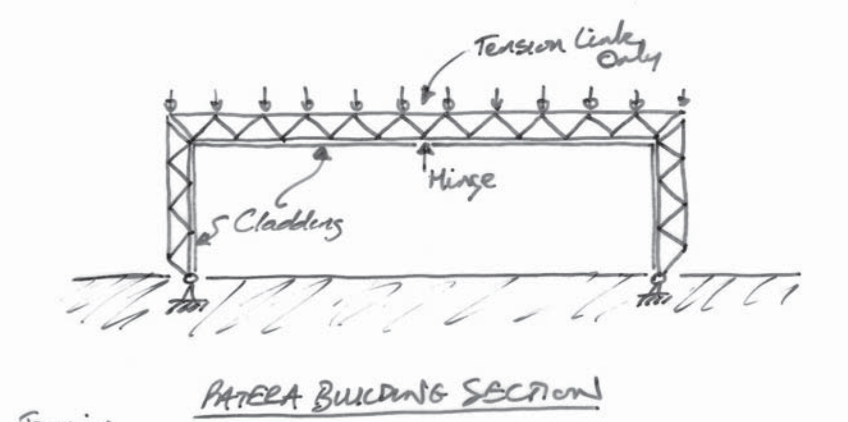


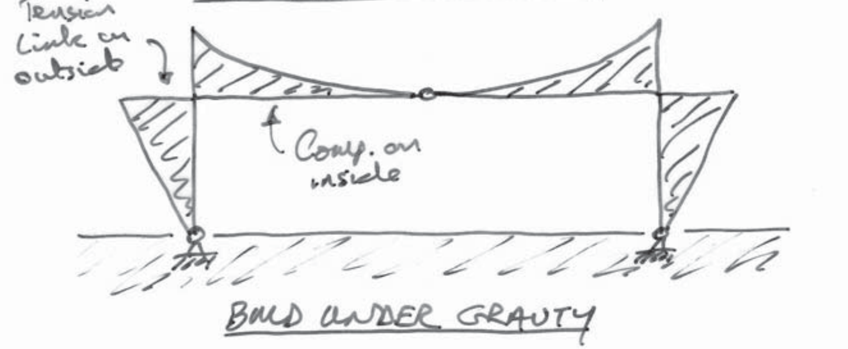
4.4 Patera Building System

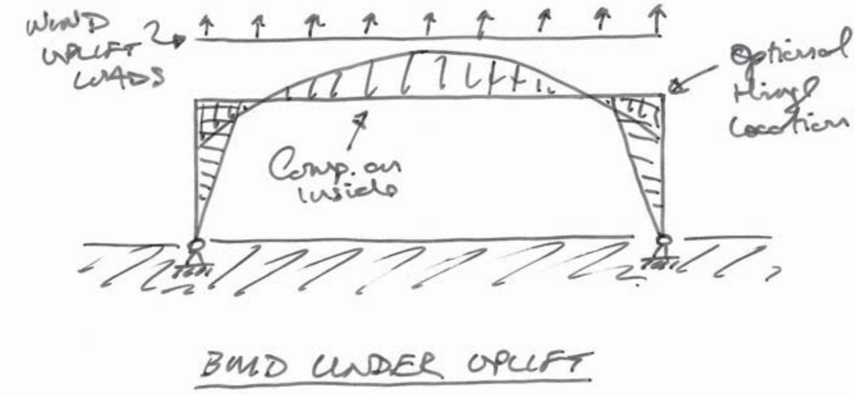




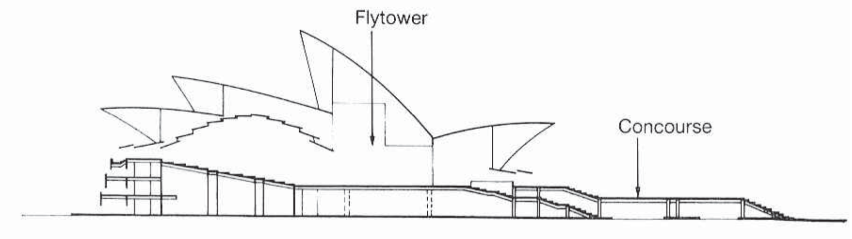




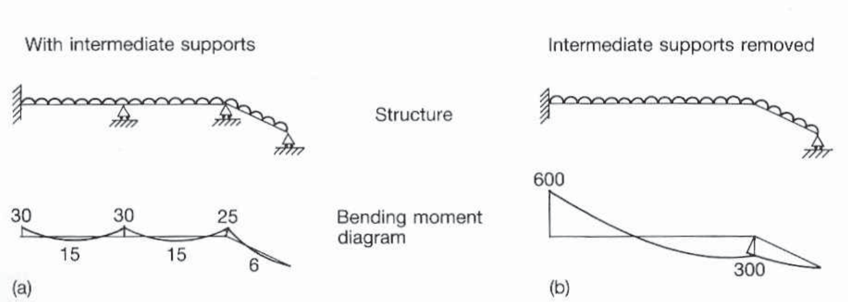




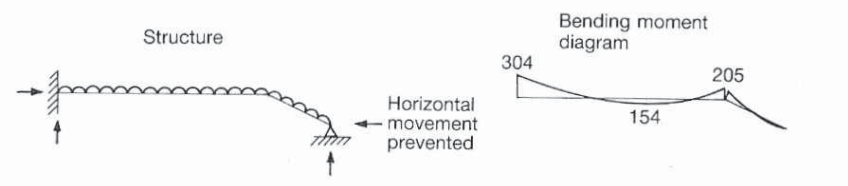
4.5 Sydney Opera House – Podium



2.4 Sydney Opera House; longitudinal cross-section (competition entry) showing how the shells envelope the fly-tower and acoustic ceiling.



2.5 A simplified mathematical model of the concourse beams of the Sydney Opera House: (a) bending moments with supports as originally proposed; (b) increase in bending moments due to removal of interior supports.



2.6 A model of the concourse beams of the Sydney Opera House showing a reduction in bending moments due to horizontal propping force at right-hand support.

Utzon wanted to remove the columns and asked the engineers if it would be possible. The engineers said yes, but it would be more expensive. But why is it necessary to remove them: the area is for deliveries only, and the columns to not interfere with that function?

4.6 Sydney Opera House – General

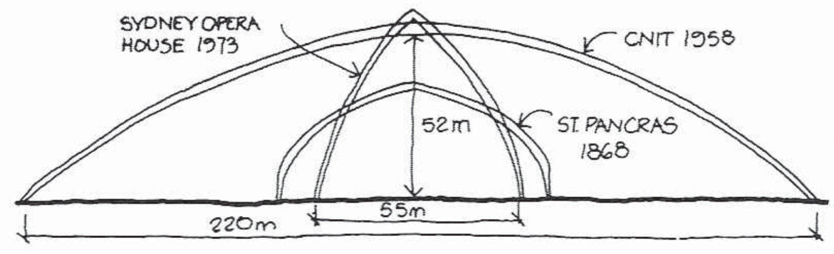


Fig. 11.54 Span comparisons

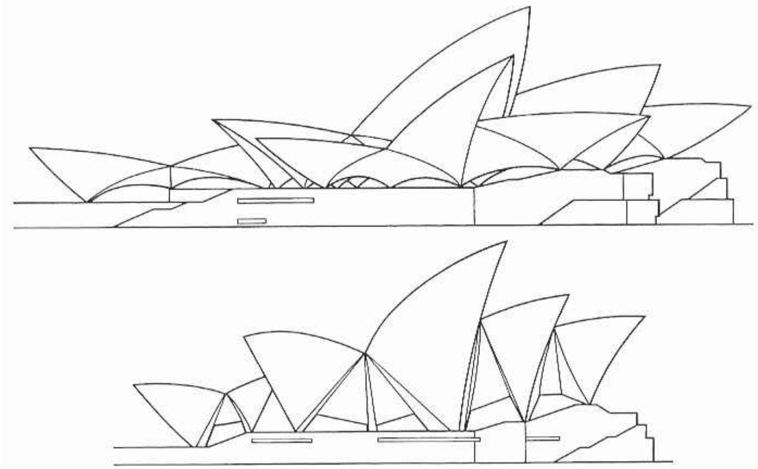
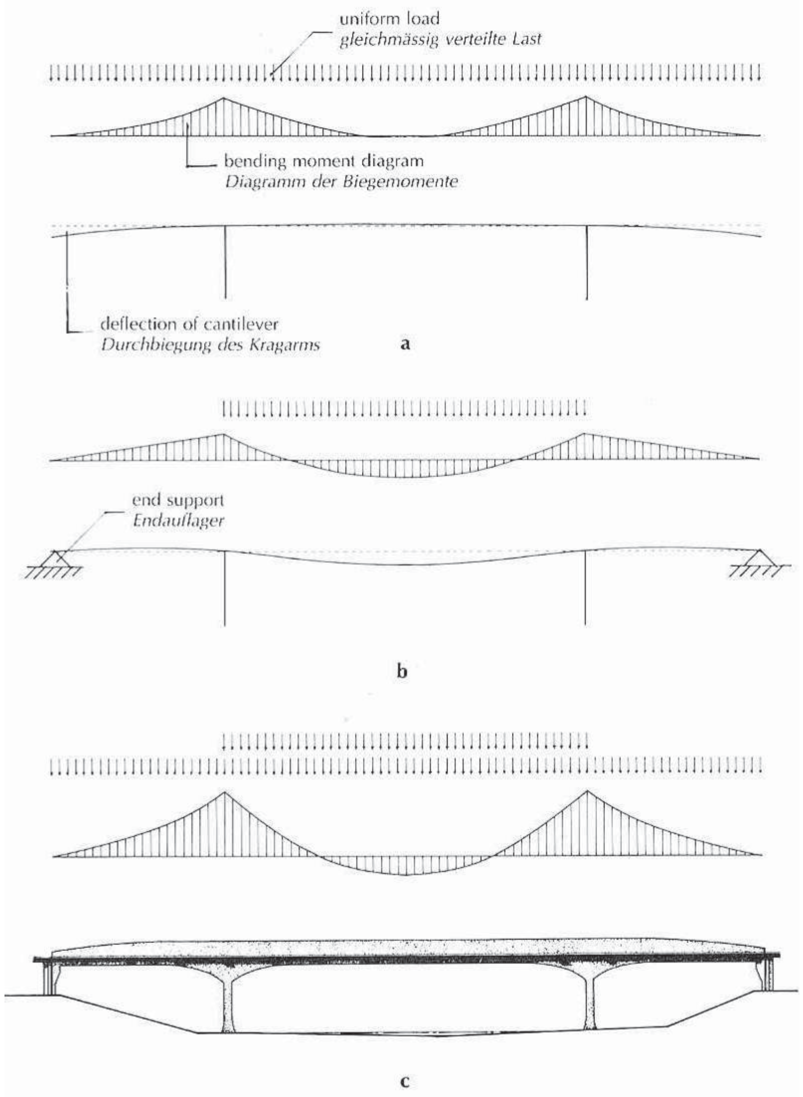


Fig. 7.46 Opera House, Sydney, Australia, 1957–65; Jorn Utzon, architect; Ove Arup & Partners, structural engineers. The upper drawing here shows the original competition-winning proposal for the building which proved impossible to build. The final scheme, though technically ingenious, is considered by many to be much less satisfactory visually. The significant difference between this and the buildings in Figs 7.41 to 7.45 is one of scale.

Even Candela was brought in to aid the design of the shells but to no avail. Would it all have been easier if Utzon had an engineer on board from day one?

4.7 Robert Maillart – Beam Bridge

The evolution of structural form:



4.8 Pompidou – Lateral Stability

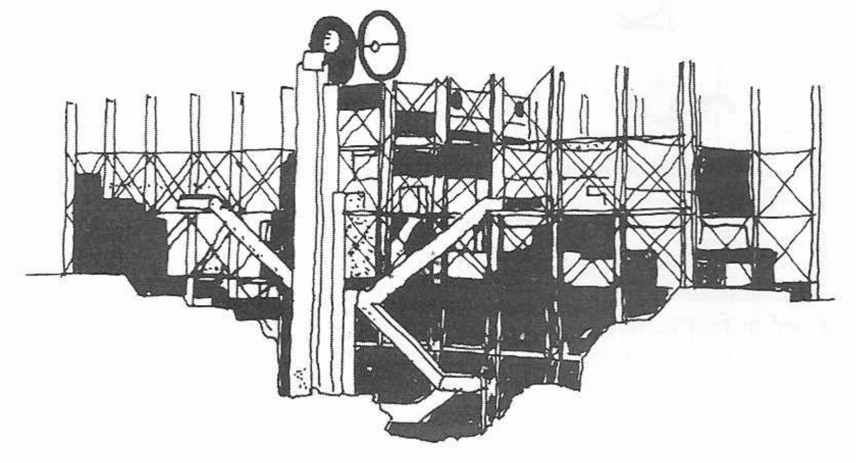


Fig. 11.55 The competition entry

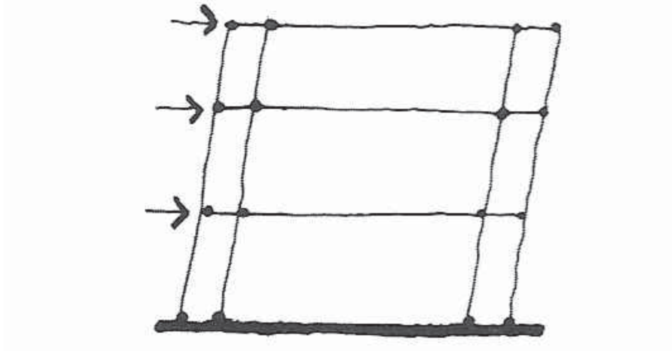


Fig. 11.56 Sway mechanism of the competition entry

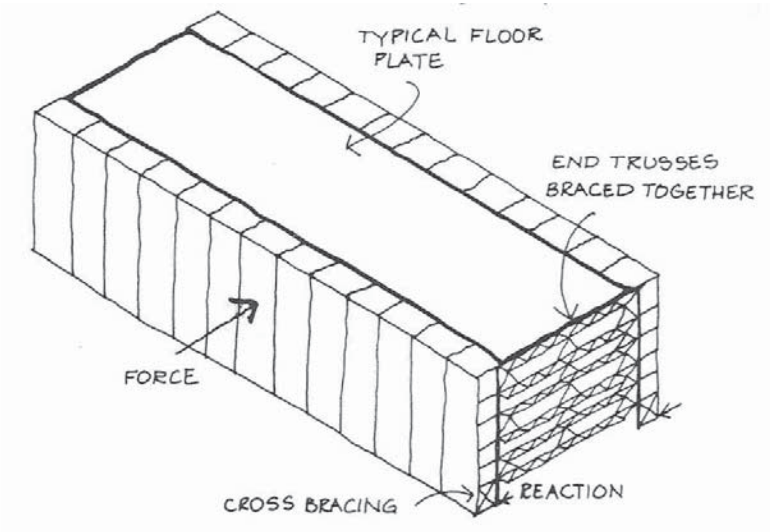


Fig. 11.60 Lateral stability

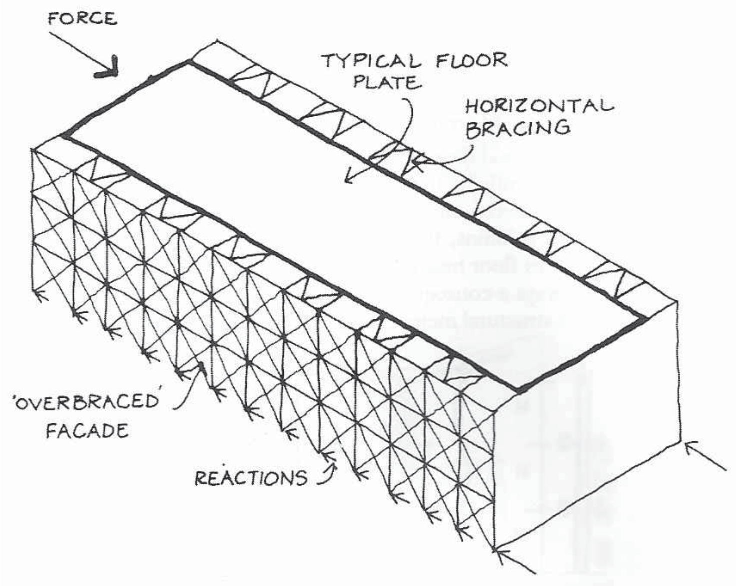
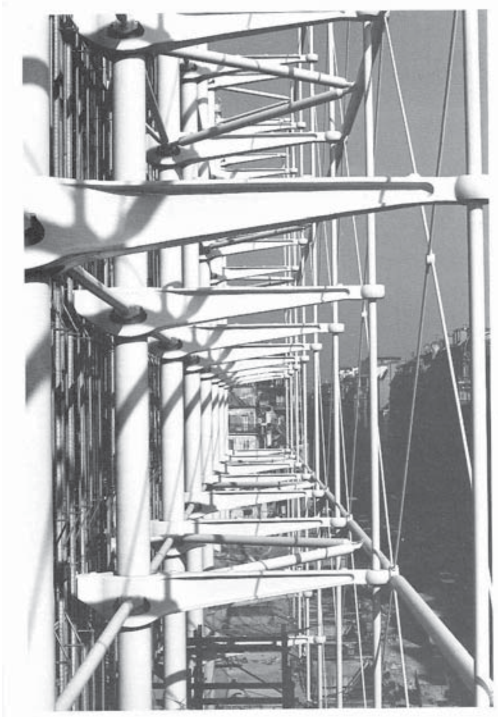
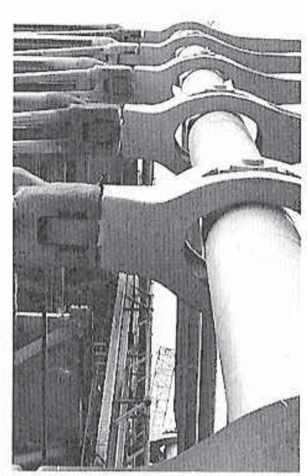


Fig. 11.59 Longitudinal stability

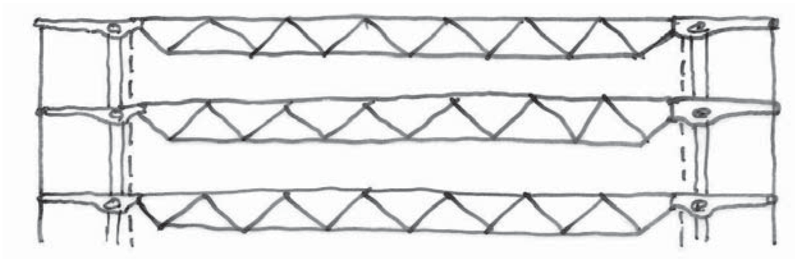
4.9 Pompidou – General



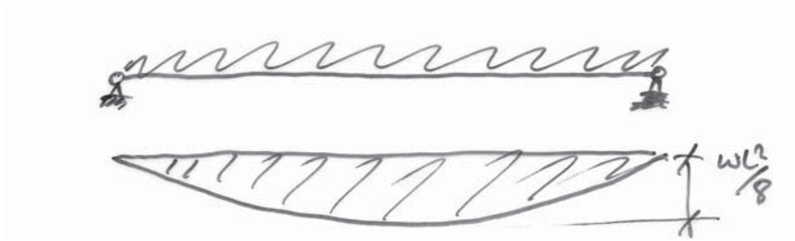


Gerberettes punctuate column. Space between gerberette and column lightens and articulates connection.

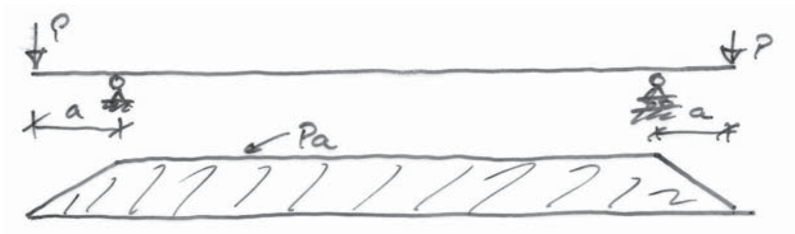
Is the form of this building derived from structural or architectural principles?



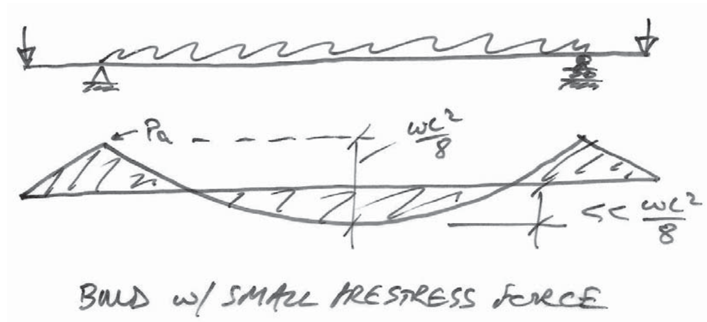
Rough structural section showing columns & Gerberettes

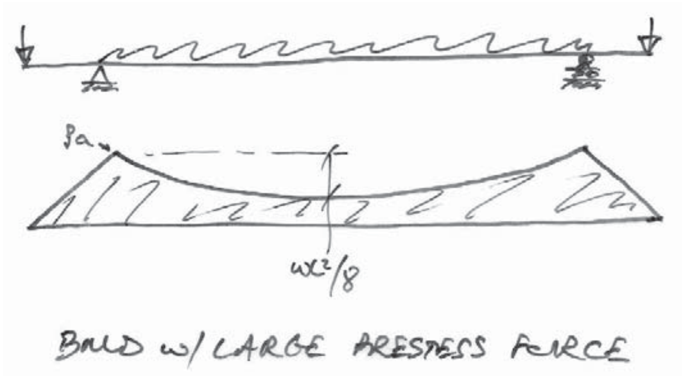


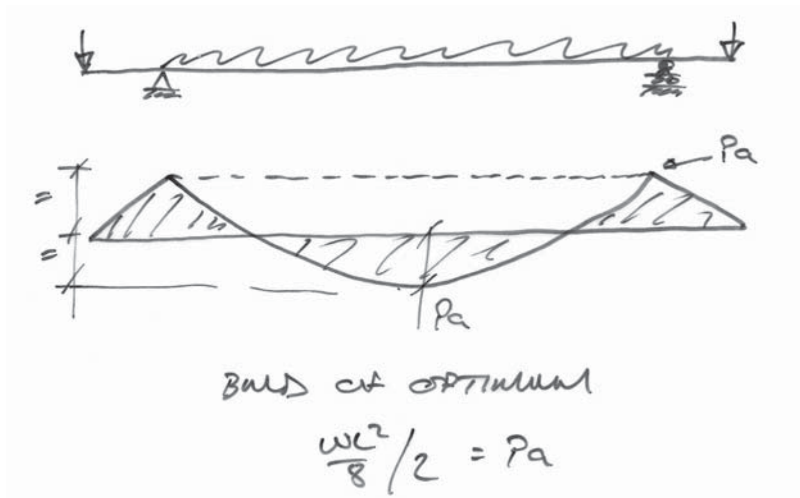
BMD for Simply-Supported Span



BMD for forces applied to cantilevered ends







4.10 Nervi – Some Examples

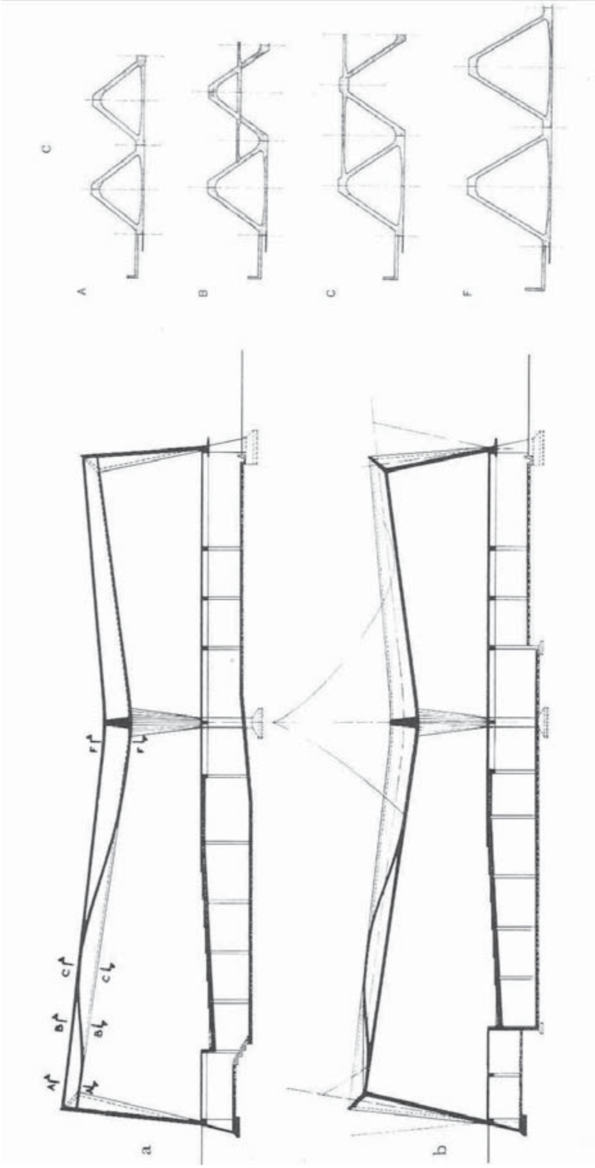


Fig. 34. UNESCO General Assembly building: (a) and (b) longitudinal sections and (c) details of the roof structure. Figure 34b shows the moment diagram which reveals a maximum positive moment at section CC (Fig. 34a) and a maximum negative moment at the intermediate support, section FF. At the sections of maximum stress the resistance provided by the upper or lower part of the folded slab were insufficient. A second element was therefore added: a reinforced concrete slab that would pass from the upper edge to the lower one following the statical needs. The slab is clearly visible in the two longitudinal sections and is further clarified by Fig. 34c. Naturally the definition of the variation, within the framework of the static guide, is influenced by factors of architectural and aesthetic nature.

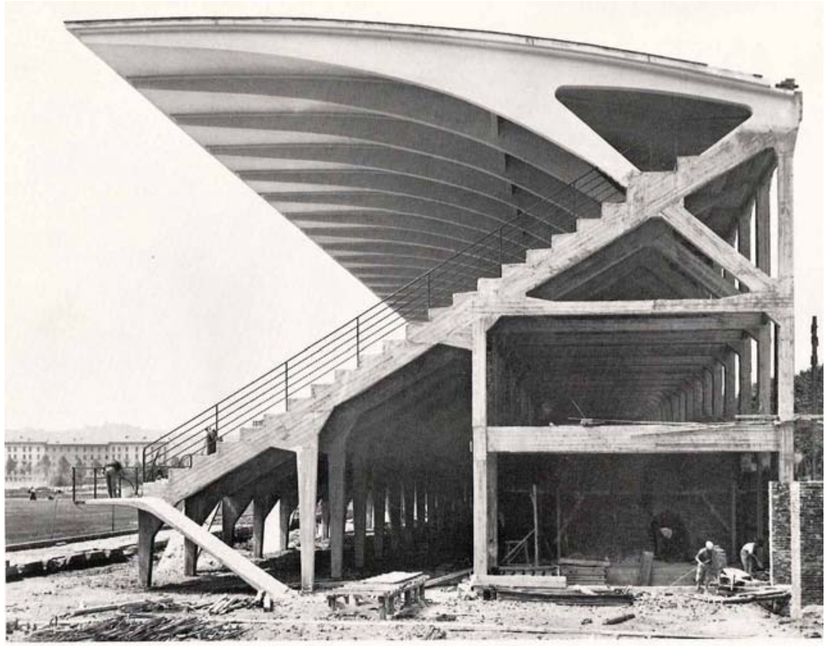


Fig. 18. The Municipal Stadium: covered grandstand during construction. The structural system demonstrated by Fig. 17 is plainly evident.

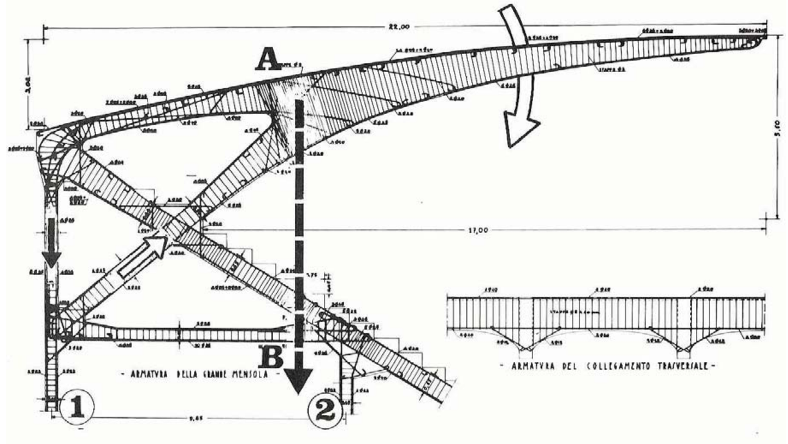
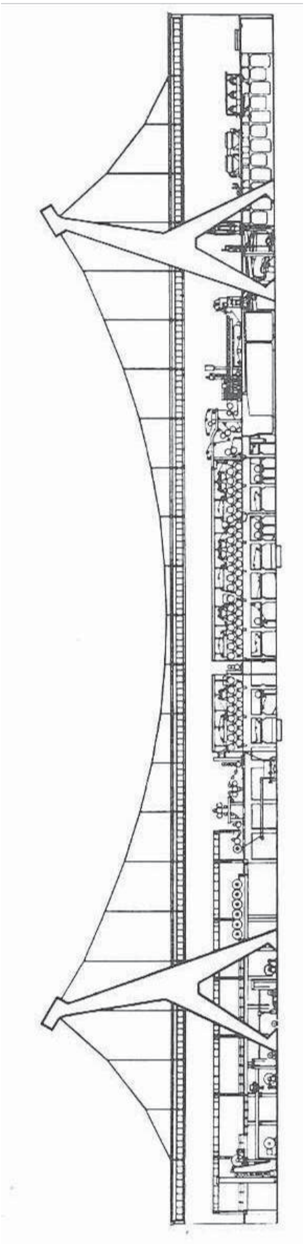
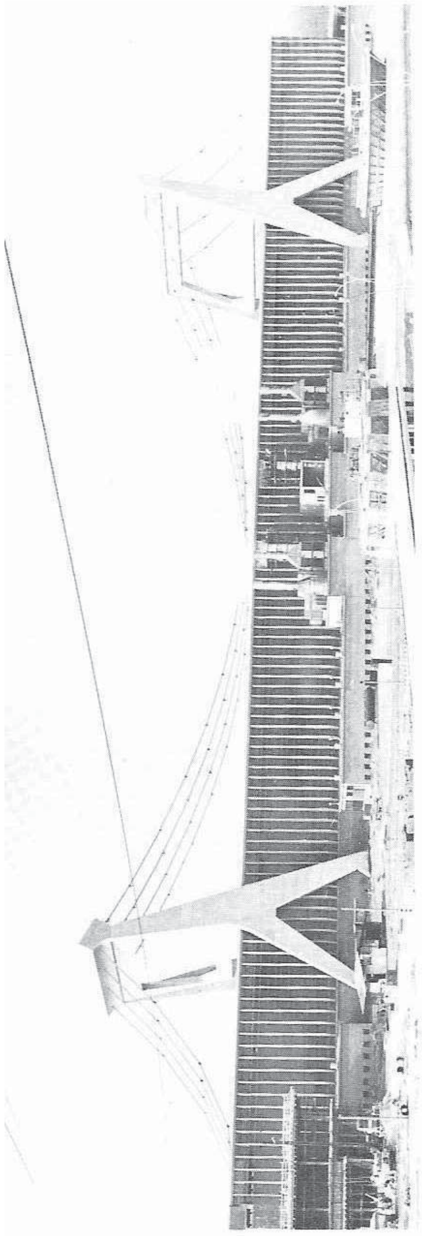
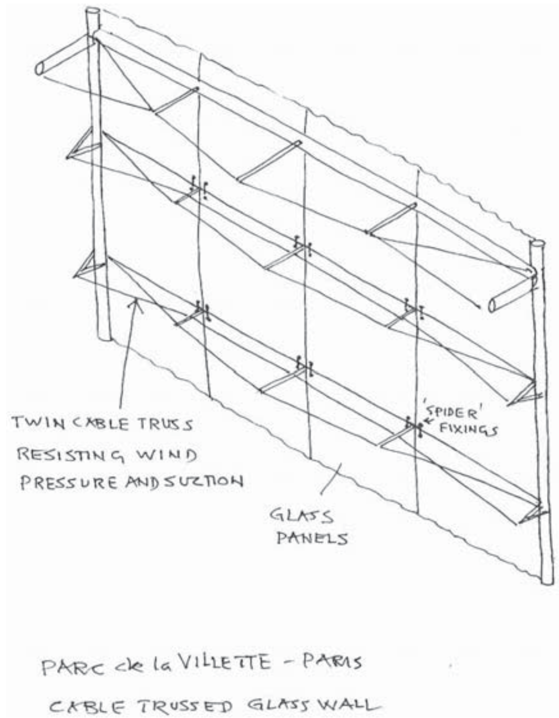


Fig. 17. The Municipal Stadium, Florence: schematic drawing of the covered grandstand. The dotted line A-B represents the resultant of all the loads coming down on columns 1 and 2. The fact that this resultant falls within the two columns eliminates the necessity of costly foundation anchorages.



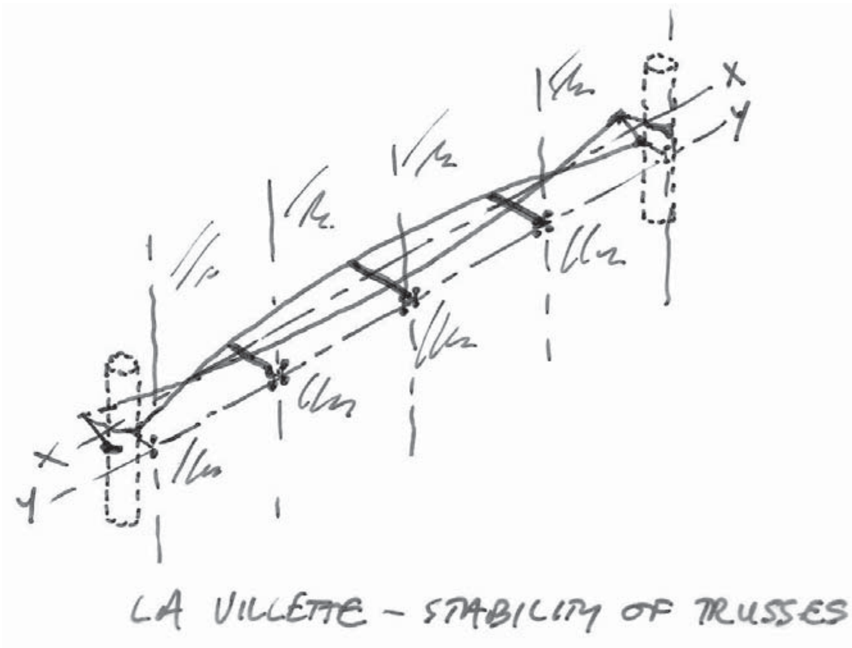


4.11 Peter Rice - La Villette

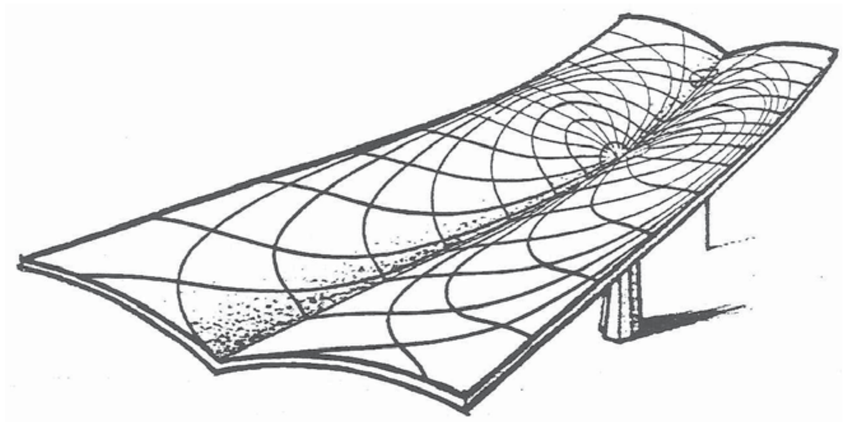


Two bow-string trusses are provided; one each for positive and negative (suction) pressures on the glazed elevation.

Of significance though, is that there are no vertical cable elements. By causing the truss and the glass to roate about different axes (X-X and Y-Y), the glass dead weight stabilizes the whole structure.

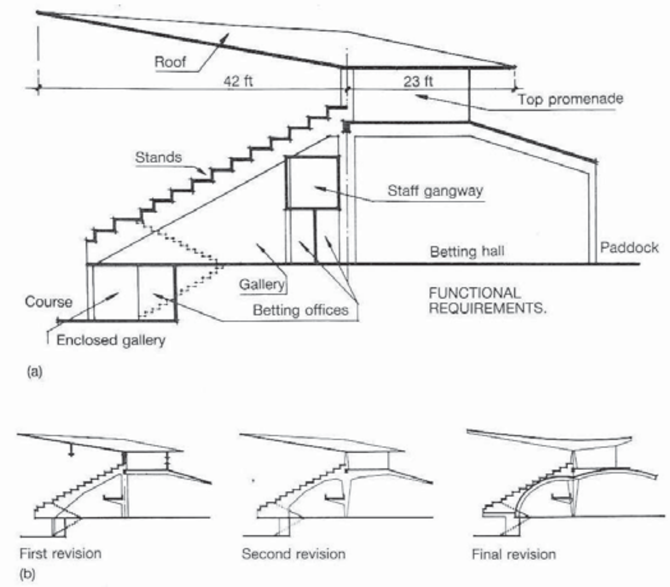


4.12 Torroja – Madrid Racecourse



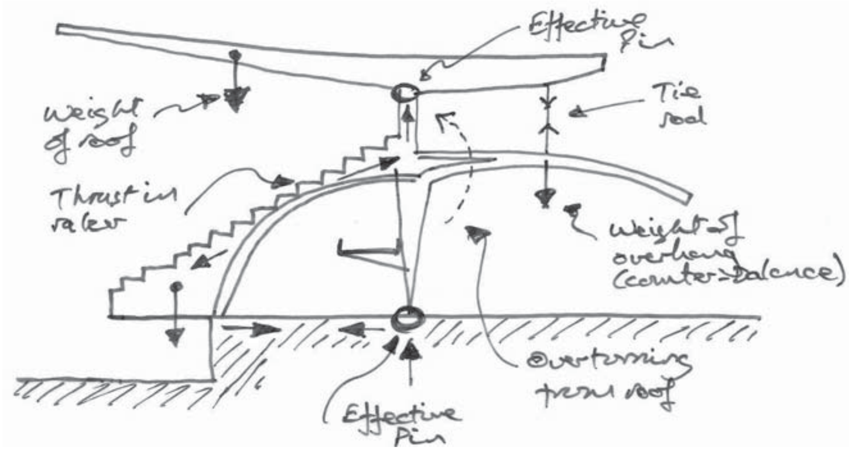
DISTRIBUTION OF STRESSES

Evolution of Structural Concept.

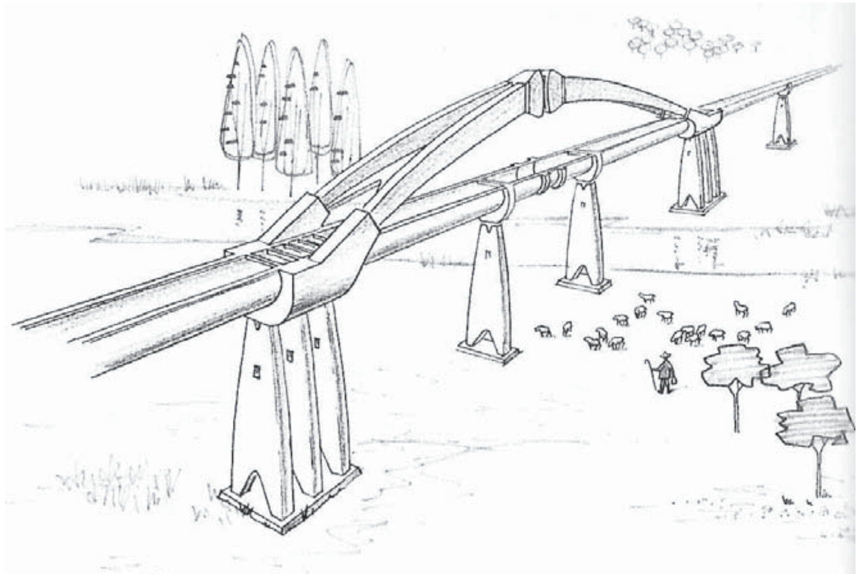


17.2 The design process as illustrated by Eduardo Torroja for his Zarzuela Grandstand, Madrid, 1935: (a) spaces and surfaces required to fulfil function; (b) progressive refinement of form.

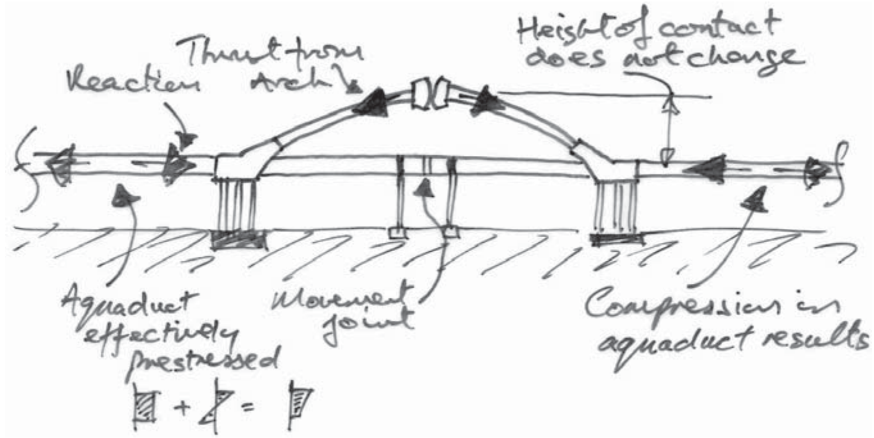
Here is an example of the Engineer's Aesthetic – why was Torroja not satisfied with event he second scheme which is structurally indistinct from the final scheme?



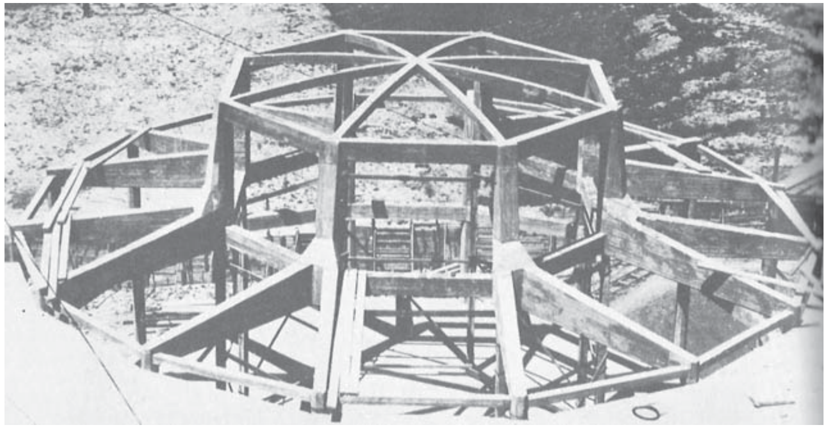
4.13 Torroja – Half-Mile Viaduct, Unbuilt, 1956

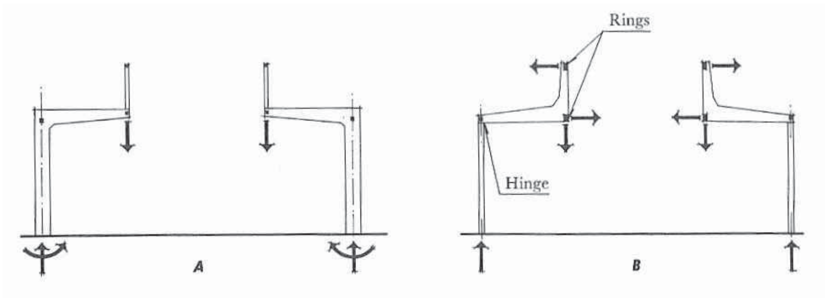


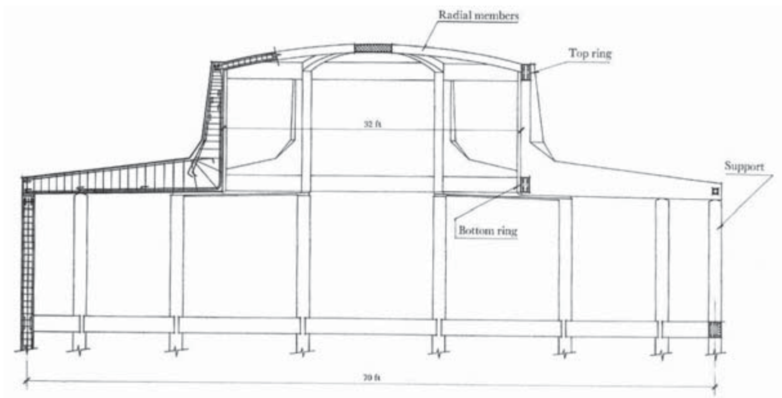
The structural concept:



4.14 Torroja – Operating Theatre, Madrid University

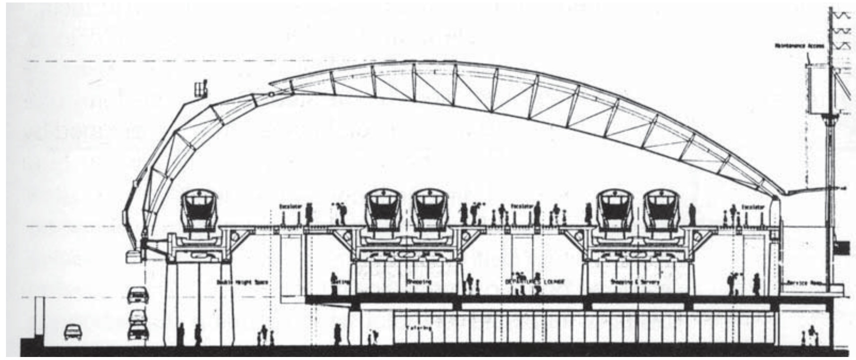




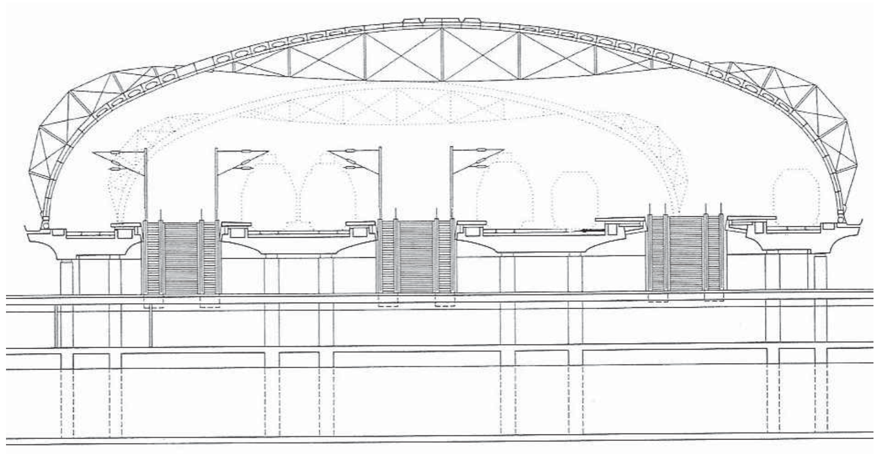


4.15 Two Recent Train Stations

Waterloo International



Lehrter Main Station in Berlin



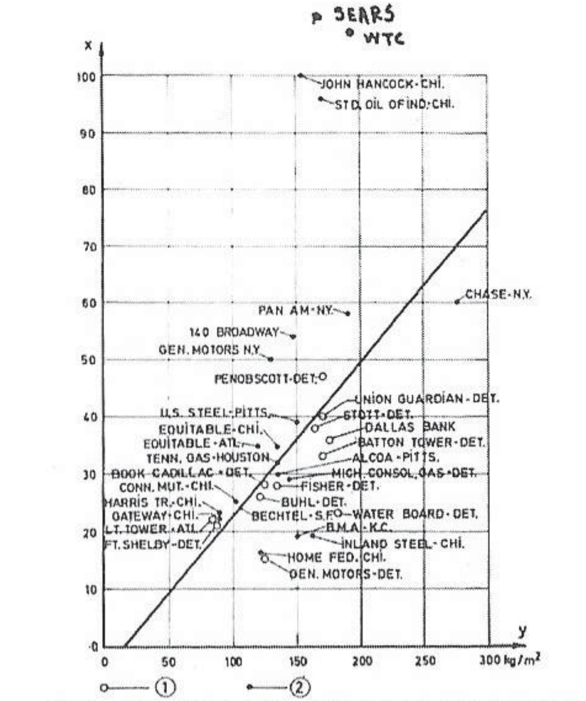
What are the structural links between these two structures?

What that in mind, is this form functional, structural or architectural?

4.16 Case Study: Fazlur Kahn and the Evolution of Tall Buildings

At the start of the 1960s, shear wall or moment frames were used for lateral stability. These are costly, and it seemed though 40 storeys was about as high as was economically possible.

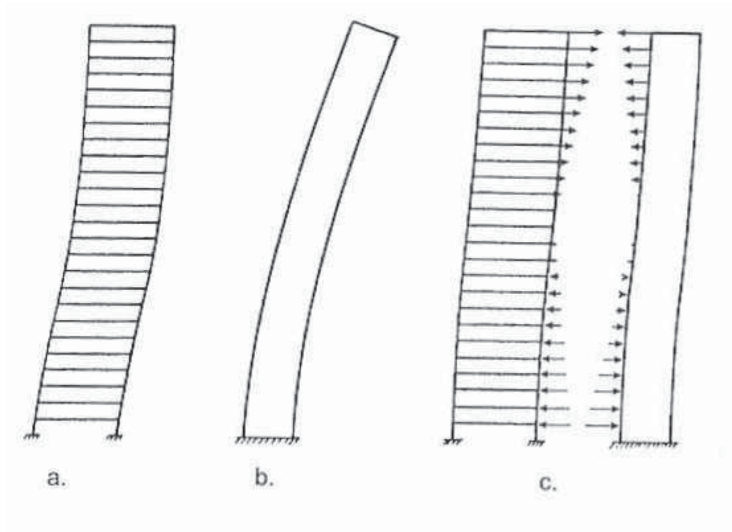
By the mid 1970s a number of buildings had broken the trend:



Number of stories/weight of steel ratio for various buildings in the U.S.A. 1. Construction before 1945. 2. Construction after 1945.

How is this possible, and who is responsible?

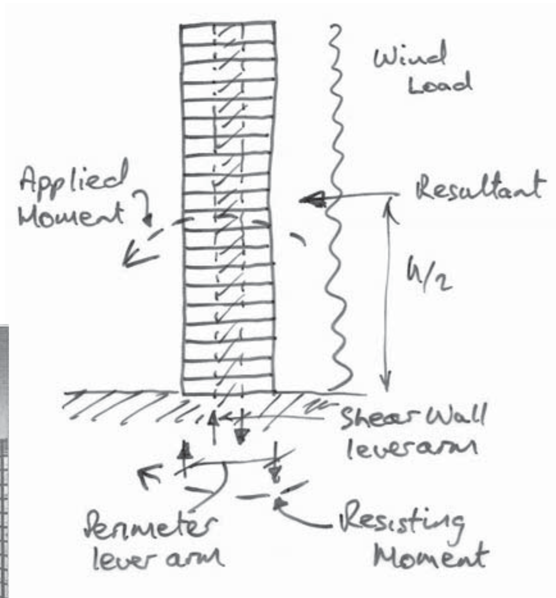
Are the resulting buildings the work of an architect, or an engineer, or both?



Brunswick Building, completed in 1965. The Chicago Civic Center Plaza is seen in the foreground. (Photograph by Hedrich-Blessing, courtesy of Skidmore, Owings & Merrill LLP.)

Typical deflected profiles under lateral loading: (a) shear-type deformation of a sway frame; (b) bending-type deformation of the shear wall as a vertical cantilever; (c) connected together in a structure, the two forms restrain each other.

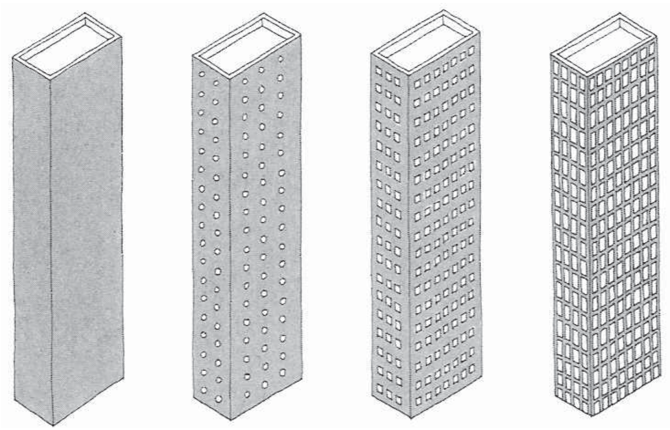
Khan now realised that the most efficient form is to have the structure around the perimeter of the building – maximizing the lever arm to resist the overturning moment.



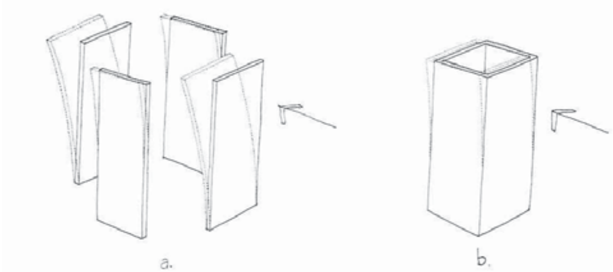
Constructed in 1964, Chestnut-DeWitt Apartments was the first tall building to employ the framed tube structural system. (Photograph by Hedrich-Blessing, courtesy of Skidmore, Owings & Merrill LLP.)

The Chestnut-DeWitt Apartments (1964) uses the framed tube system.

Evolution of the Framed Tube concept.



Four walls subjected to wind: (a) when separate, the two walls bending about their weak axis offer minimal resistance, but (b) when joined to act as a tubular structure, the weak walls are forced to participate in resisting wind. By invoking a three-dimensional structural response, the total effectiveness of the four walls is increased significantly. (Drawing by David Fung.)

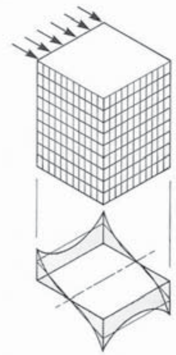


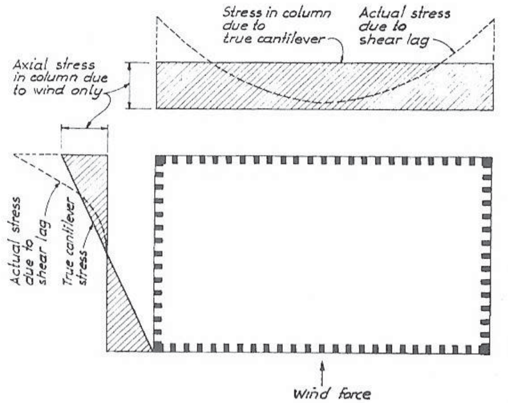
The basic structural action tells us that the second moment of area is increasingly hugely by connecting the four shear walls, rendering a much stiffer structure.

Note that this requires closely spaced exterior columns and large perimeter beams.

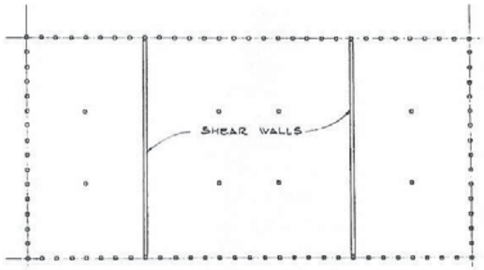
There are some problems with the idea though; the columns near the centre of the building do not carry as much load as simple beam theory tells us:

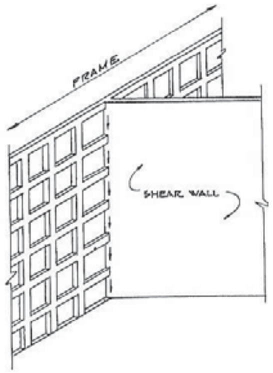
This is called shear lag.





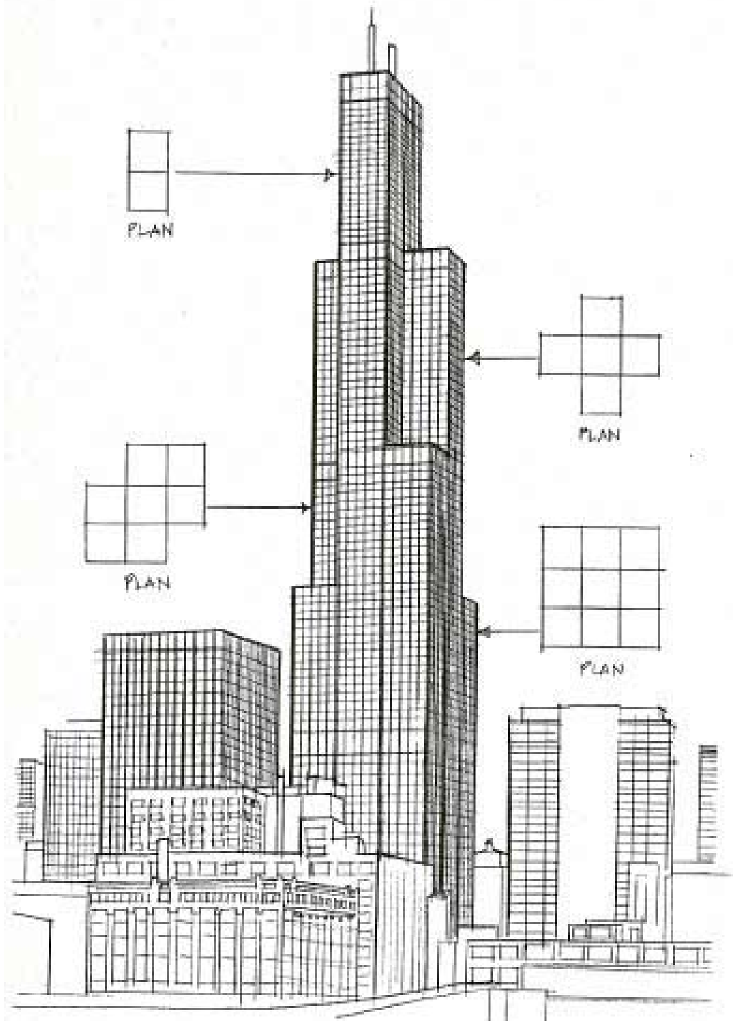
Khan realized that the solution to this was to stiffen the perimeter walls by brining internal shear walls out to the perimeter:

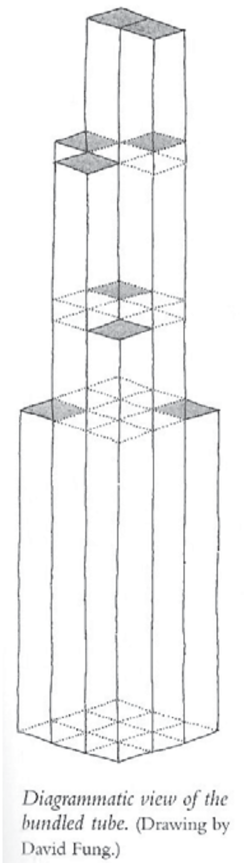




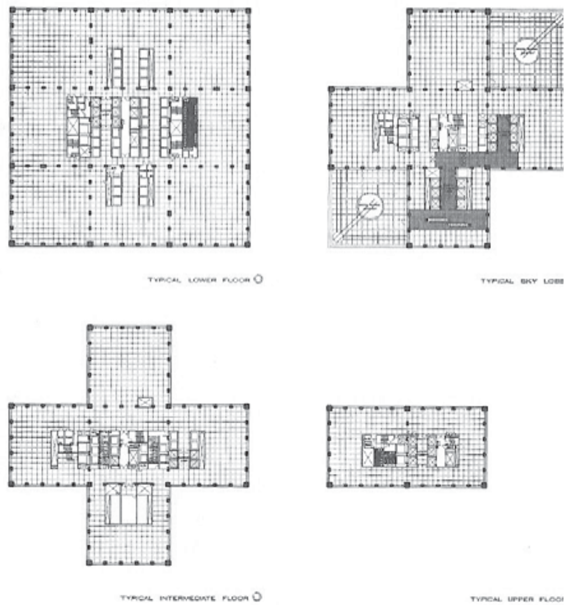
This is not ideal for the function of the building. But Kahn and Graham (SOM architect) realized that this full resistance was not required all the way up the structure, and that these interior walls might be exterior at different levels.

Thus the concept of the bundled tube was born, leading to…



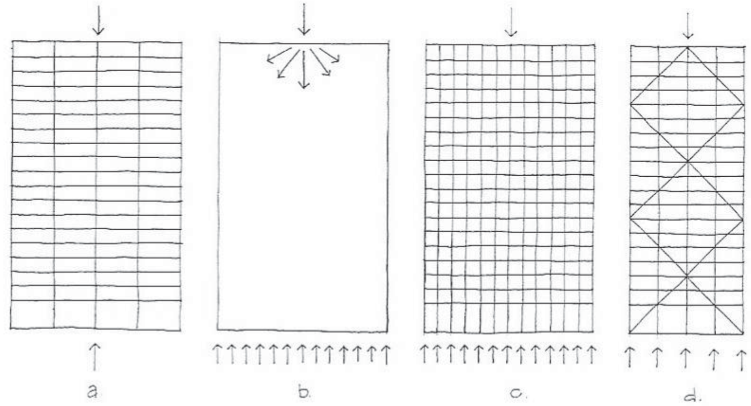


Diagrammatic view of the bundled tube. (Drawing by David Fung.)

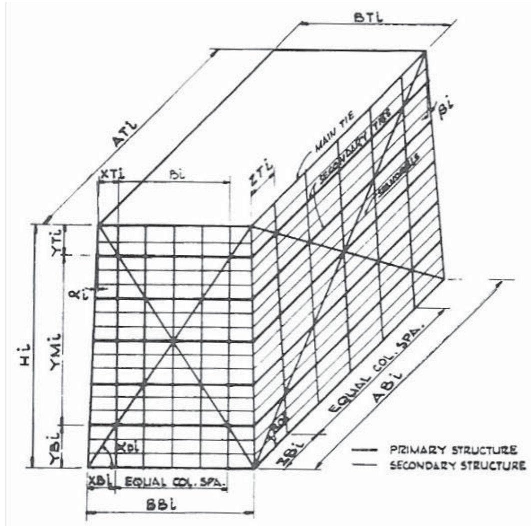


Varied floor plans were created by "dropping off" modular cells. Two corner tube-units extend no further than the 50th floor, another two end at the 66th floor, and another three at the 90th floor. Floor areas range from 52, 670 square feet at the base to 12,283 square feet at the top of the tower. (Courtesy of Skidmore, Owings & Merrill LLP.)

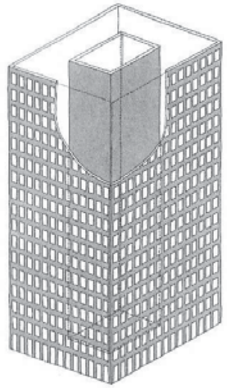
Other forms Kahn developed are the trussed tube:



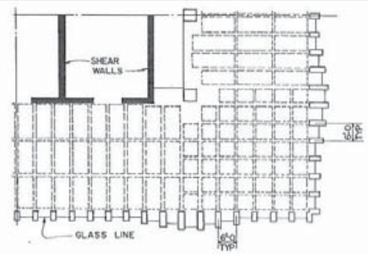
Usual beam/frame structures do not share load well (a). The ideal distribution of vertical load in (b) can be simulated by closely spaced columns (c), or by the integration of beams columns and diagonal elements (d).



And the tube-in-a-tube idea, where beams link the two stability stuctures:



Note that in this structure, the corners of the cores are heavily loaded. This is expressed on the exterior of One Shell Plaza as undulating column sizes.

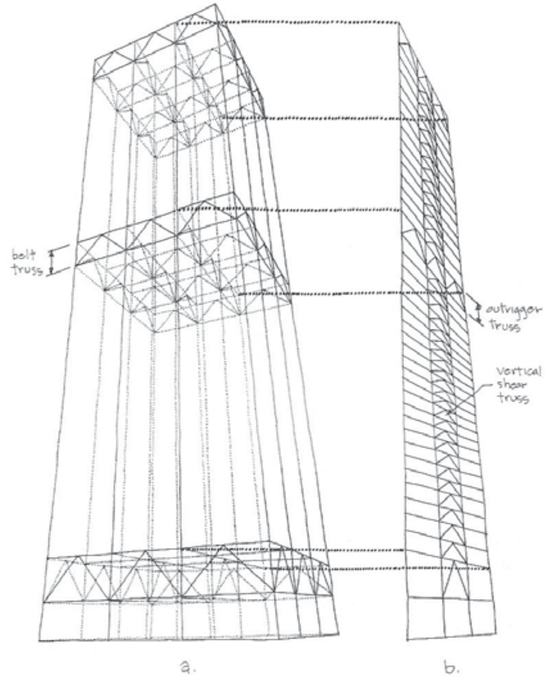


TYPICAL CORNER AREA FLOOR PLAN

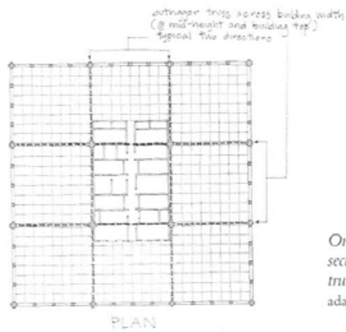
Schematic floor plan at corner shows the joist- and waffle-slab floor framing systems. (Courtesy of Skidmore, Owings & Merrill LLP.)

Variation of column depth on the perimeter, from 2 feet to 4 feet, created visual interest while serving a structural purpose. (Unidentified photographer, courtesy of Skidmore, Owings & Merrill LLP.)

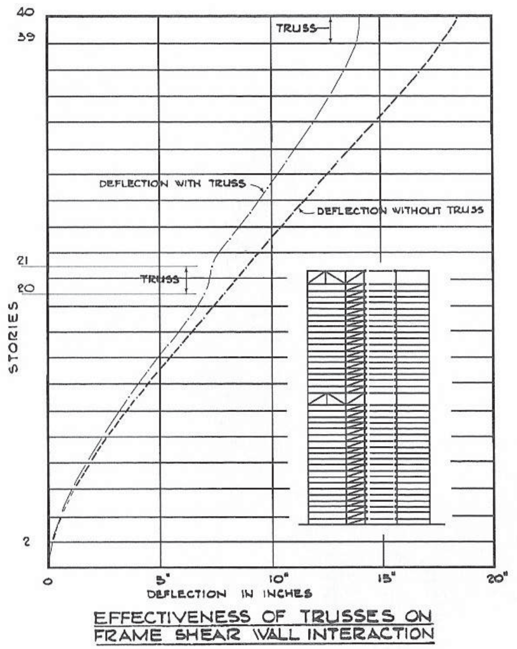
And the outrigger truss system – useful in seismic areas:

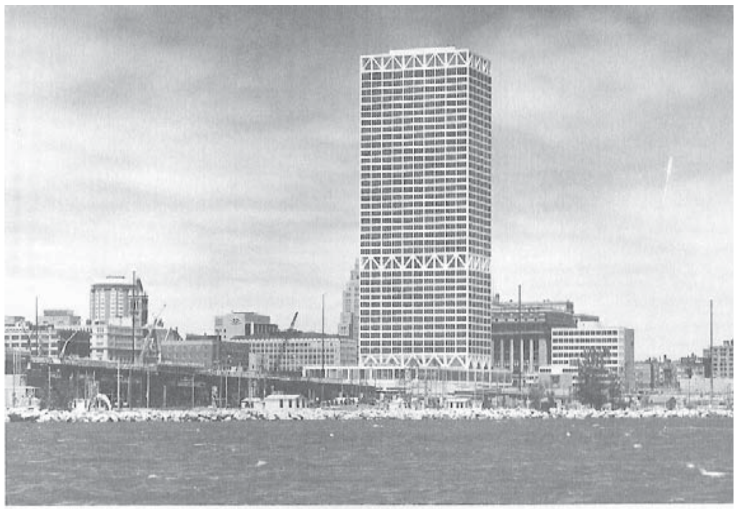


Three types of truss system were combined in the belt truss interaction system: (a) the belt truss and the outrigger truss systems are shown in a perspective view of the structure, and (b) the vertical shear truss and the outrigger truss systems are shown in a typical (four locations at core) interior wall plane. (Drawing by David Fung.)



On the BHP House design, outrigger trusses trisected the floor plan to connect with the vertical shear truss systems at the core. (Drawing by David Fung, adapted from Yuncken Freeman Architects, *BHP House*.)

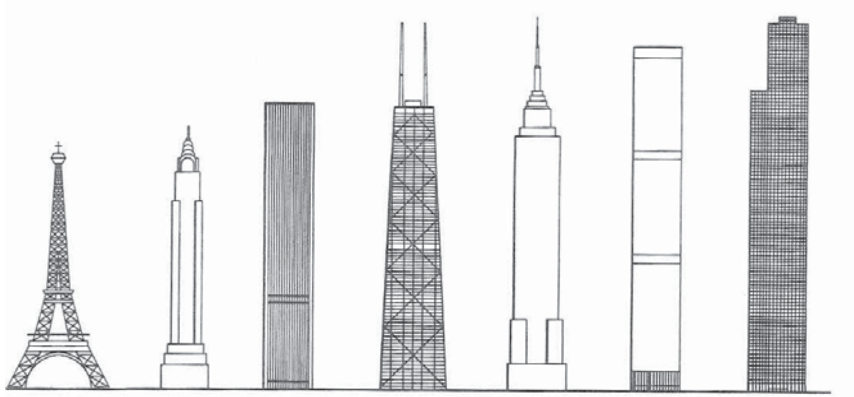


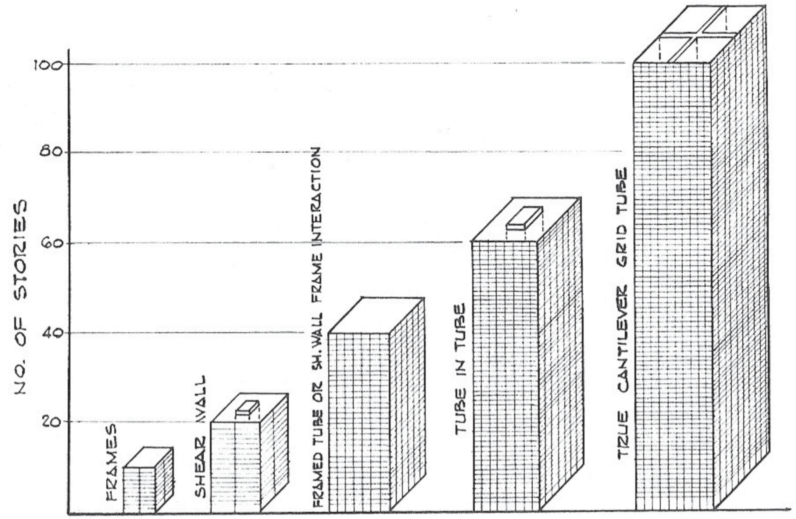


Graham and Khan considered the First Wisconsin Center's belt truss system worthy of bold expression. The 42-story building was completed in 1974. (Ezra Stoller © Esto.)

In summary, Khan helped develop the concepts used in the world's tallest structures.

It is clear that structural considerations play the prime role in final form of the building.





At a tall building symposium in 1966 (proceedings published in Tall Buildings, *1967), Khan demonstrated the dramatic increase in height that could be achieved economically by using a "true cantilever grid tube" structural system.*

4.17 Structures Photographs Presentation

A presentation of photographs depicting various aspects of structural form and behaviour will be given as the basis for discussion. The presentation will be made available online at www.colincaprani.com in due course. Relate notes here to the slide number:

5. Preliminary Analysis

5.1 *Preliminary Loading*

General philosophy: keep it conservative, yet realistic, at preliminary design stage.

Imposed Loading

- Based on BS 6399: Parts 1 & 2;
- Choose the highest where options exist;
- No live load reduction factors should be allowed for.

Some common loads should be remembered:

Use	Load (kN/m^2)
Commercial speculative offices	5
Light office	2.5
Residential	1.5
Car park	2.5
Plant rooms	7.5

Extract from BS6399: Part 1: 1996:

Table 1. Minimum imposed floor loads

Type of activity/occupancy for part of the building or structure	Examples of specific use		Uniformity distributed load kN/m^2	Concentrated load kN
A Domestic and residential activities (Also see category C)	All usages within self-contained dwelling units Communal areas (including kitchens) in blocks of flats with limited use (See note 1) (For communal areas in other blocks of flats, see C3 and below)		1.5	1.4
	Bedrooms and dormitories except those in hotels and motels		1.5	1.8
	Bedrooms in hotels and motels Hospital wards Toilet areas		2.0	1.8
	Billiard rooms		2.0	2.7
	Communal kitchens except in flats covered by note 1		3.0	4.5
	Balconies	Single dwelling units and communal areas in blocks of flats with limited use (See note 1)	1.5	1.4
		Guest houses, residential clubs and communal areas in blocks of flats except as covered by note 1	Same as rooms to which they give access but with a minimum of 3.0	1.5/m run concentrated at the outer edge
		Hotels and motels	Same as rooms to which they give access but with a minimum of 4.0	1.5/m run concentrated at the outer edge
B Offices and work areas not covered elsewhere	Operating theatres, X-ray rooms, utility rooms		2.0	4.5
	Work rooms (light industrial) without storage		2.5	1.8
	Offices for general use		2.5	2.7
	Banking halls		3.0	2.7
	Kitchens, laundries, laboratories		3.0	4.5
	Rooms with mainframe computers or similar equipment		3.5	4.5
	Machinery halls, circulation spaces therein		4.0	4.5
	Projection rooms		5.0	To be determined for specific use
	Factories, workshops and similar buildings (general industrial)		5.0	4.5
	Foundries		20.0	To be determined for specific use
	Catwalks		—	1.0 at 1 m centres
	Balconies		Same as rooms to which they give access but with a minimum of 4.0	1.5/m run concentrated at the outer edge
	Fly galleries		4.5 kN/m run distributed uniformly over width	—
	Ladders		—	1.5 rung load

Table 1. Minimum imposed floor loads *(continued)*

Type of activity/occupancy for part of the building or structure	Examples of specific use		Uniformity distributed load kN/m²	Concentrated load kN
C Areas where people may congregate C1 Areas with tables	Public, institutional and communal dining rooms and lounges, cafes and restaurants (See note 2)		2.0	2.7
	Reading rooms with no book storage		2.5	4.5
	Classrooms		3.0	2.7
C2 Areas with fixed seats	Assembly areas with fixed seating (See note 3)		4.0	3.6
	Places of worship		3.0	2.7
C3 Areas without obstacles for moving people	Corridors, hallways, aisles, stairs, landings etc. in institutional type buildings (not subject to crowds or wheeled vehicles), hostels, guest houses, residential clubs, and communal areas in blocks of flats not covered by note 1. (For communal areas in blocks of flats covered by note 1, see A)	Corridors, hallways, aisles etc. (foot traffic only)	3.0	4.5
		Stairs and landings (foot traffic only)	3.0	4.0
	Corridors, hallways, aisles, stairs, landings, etc. in all other buildings including hotels and motels and institutional buildings	Corridors, hallways, aisles, etc. (foot traffic only)	4.0	4.5
		Corridors, hallways, aisles, etc., subject to wheeled vehicles, trolleys etc.	5.0	4.5
		Stairs and landings (foot traffic only)	4.0	4.0
	Industrial walkways (light duty)		3.0	4.5
	Industrial walkways (general duty)		5.0	4.5
	Industrial walkways (heavy duty)		7.5	4.5
	Museum floors and art galleries for exhibition purposes		4.0	4.5
	Balconies (except as specified in A)		Same as rooms to which they give access but with a minimum of 4.0	1.5/m run concentrated at the outer edge
	Fly galleries		4.5 kN/m run distributed uniformly over width	—
C4 Areas with possible physical activities (See clause 9)	Dance halls and studios, gymnasia, stages		5.0	3.6
	Drill halls and drill rooms		5.0	9.0
C5 Areas susceptible to overcrowding (See clause 9)	Assembly areas without fixed seating, concert halls, bars, places of worship and grandstands		5.0	3.6
	Stages in public assembly areas		7.5	4.5
D Shopping areas	Shop floors for the sale and display of merchandise		4.0	3.6

Table 1. Minimum imposed floor loads *(continued)*

Type of activity/occupancy for part of the building or structure	Examples of specific use	Uniformity distributed load kN/m²	Concentrated load kN
E Warehousing and storage areas. Areas subject to accumulation of goods. Areas for equipment and plant.	General areas for static equipment not specified elsewhere (institutional and public buildings)	2.0	1.8
	Reading rooms with book storage, e.g. libraries	4.0	4.5
	General storage other than those specified	2.4 for each metre of storage height	7.0
	File rooms, filing and storage space (offices)	5.0	4.5
	Stack rooms (books)	2.4 for each metre in storage height but with a minimum of 6.5	7.0
	Paper storage for printing plants and stationery stores	4.0 for each metre of storage height	9.0
	Dense mobile stacking (books) on mobile trolleys, in public and institutional buildings	4.8 for each metre of storage height but with a minimum of 9.6	7.0
	Dense mobile stacking (books) on mobile trucks, in warehouses	4.8 for each metre of storage height but with a minimum of 15.0	7.0
	Cold storage	5.0 for each metre of storage height but with a minimum of 15.0	9.0
	Plant rooms, boiler rooms, fan rooms, etc., including weight of machinery	7.5	4.5
	Ladders	—	1.5 rung load
F	Parking for cars, light vans, etc. not exceeding 2500 kg gross mass, including garages, driveways and ramps	2.5	9.0
G	Vehicles exceeding 2500 kg. Driveways, ramps, repair workshops, footpaths with vehicle access, and car parking	To be determined for specific use	

NOTE 1. Communal areas in blocks of flats with limited use refers to blocks of flats not more than three storeys in height and with not more than four self-contained dwelling units per floor accessible from one staircase.
NOTE 2. Where these same areas may be subjected to loads due to physical activities or overcrowding, e.g. a hotel dining room used as a dance floor, imposed loads should be based on occupancy C4 or C5 as appropriate. Reference should also be made to clause 9.
NOTE 3. Fixed seating is seating where its removal and the use of the space for other purposes is improbable.

All floors should be designed to carry the uniformly distributed or concentrated load, whichever produces the greatest stresses (or where critical, deflection) in the part of the floor under consideration.

The categories adopted for types of activity/occupancy are:

A Domestic and residential activities
B Office and work areas not covered elsewhere
C Areas where people may congregate
D Shopping areas
E Areas susceptible to the accumulation of goods
F/G Vehicle and traffic areas

5.1.2 *Uniformly distributed loads*

The uniformly distributed loads given in table 1 are the uniformly distributed static loads per square metre of plan area and provide for the effects of normal use.

5.1.3 *Concentrated loads*

Concentrated loads should be assumed to act at points on the member to give the greatest moment, shear (or where critical, deflection). Concentrated loads should be applied to individual members and assumed to act on them unless there is evidence that adequate interaction exists to ensure that the load can be shared or spread.

When used for the calculation of local effects such as crushing and punching, the concentrated loads should be assumed to act at a position and over an area of application appropriate to their cause. Where this cannot be foreseen, a square contact area with a 50 mm side should be assumed.

Dead Load

Dead loads are derived from the densities of materials to be used. However, usually the dimensions of the elements are not known prior to preliminary sizing.

The ultimate reference is:

- Eurocode 1: *Actions on Structures*: Part 2: Annex 2: *Densities of Building Materials and Stored Materials*

Some common densities are:

Material	**Density** (kN/m^3)
Reinforced concrete	24
Structural steel	77
Timber – softwood	4 – 6
Timber – hardwood	6 – 10
Blocks – solid	21
Blocks – hollow	12
Bricks	22

Designers usually build up a list of the dead loads for common build-ups – two sets of build-ups are in the following pages.

In deriving dead load, be conservative at preliminary design stage.

After calculation of dead and imposed load, determine the composite gamma factor. This provides insight into the governing type of load (dead or live) and is also very useful after the full load takedown when only service loads are to be designed for in the foundation dedsign.

Remember that: $\gamma_G = 1.4; \gamma_Q = 1.6$, and,

$$w_{ser} = G_k + Q_k$$
$$w_{ult} = \gamma_G \cdot G_k + \gamma_Q \cdot Q_k$$

So define the composite factor of safety:

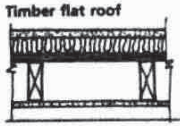
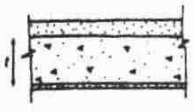
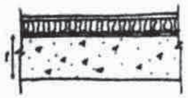
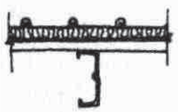
$$\gamma_{Comp} = \frac{w_{ult}}{w_{ser}}$$

Values of γ_{Comp} nearer 1.4 indicate dead load is governing; those nearer 1.6 indicate live load is governing. To reduce loads overall if there is a problem, try change the governing load first.

The Structural Engineer's Handbook gives the following:

Typical unit floor and roof loadings

Permanent partitions shown on the floor plans should be considered as dead load. Flexible partitions which may be movable should be allowed for in imposed loads, with a minimum of 1 kN/m^2.

Timber floor 	Live loading: domestic/office	1.5/2.5 kN/m^2
	(Office partitions)	(1.0)
	Timber boards/plywood	0.15
	Timber joists	0.2
	Ceiling and services	0.15
	Domestic/ office totals	**2.0/4.0 kN/m^2**
Timber flat roof 	Snow and access	0.75 kN/m^2
	Asphalt waterproofing	0.45
	Timber joists and insulation	0.2
	Ceiling and services	0.15
	Total	**1.55 kN/m^2**
Timber pitched roof 	Snow	0.6 kN/m^2
	Slates, timber battens and felt	0.55
	Timber rafters and insulation	0.2
	Ceiling and services	0.15
	Total	**1.5 kN/m^2**
Internal RC slab 	Live loading: office/ classroom/corridors, etc.	2.5/3.0/4.0 kN/m^2
	Partitions	1.0 (minimum)
	50 screed/75 screed/raised floor	1.2/1.8/0.4
	Solid reinforced concrete slab	24t
	Ceiling and services	0.15
	Total	**– kN/m^2**
External RC slab 	Live loading: snow and access/office/bar	0.75/2.5/5.0 kN/m^2
	Slabs/paving	0.95
	Asphalt waterproofing and insulation	0.45
	50 screed	1.2
	Solid reinforced concrete slab	24t
	Ceiling and services	0.15
	Total	**– kN/m^2**
Metal deck roofing 	Live loading: snow/wind uplift	0.6/–1.0 kN/m^2
	Outer covering, insulation and metal deck liner	0.4
	Purlins – 150 deep at 1.5 m c/c	0.3
	Services	0.1
	Primary steelwork: light beams/trusses	0.5–0.8/0.7–2.4
	Total	**– kN/m^2**

One more relevant to Irish construction is:

Standard Dead Loads

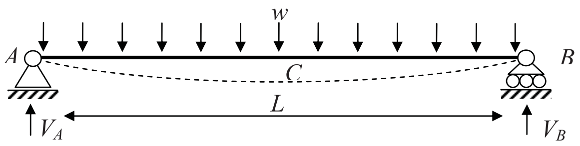
	Build-up	kN/m^2	Build-up	kN/m^2
Floor	**Timber Floor**		**Concrete Floor**	
	Floorboards/plywood deck	0.15	Precast H'core Unit 150 Dp.	2.25
	Floor Joists	0.1-	200 Dp.	2.56
	Plasterboard Ceiling …or…	0.15	250 Dp.	3.22
	…Lath and Plaster	0.5	300 Dp.	3.55
	Insulation	0.05	400 Dp.	4.3
	Pugging (plaster/mortar)	0.4	R.C Slab 150 Dp.	3.6
	Carpet/finish	0.2	200 Dp.	4.8
	Typical Floor Wt (L&P,	1.4	250 Dp.	6
	Typical Floor Wt (Plasterbd, Ins.)	0.75	300 Dp.	7.2
			Screed 50 mm	1.2
			70 mm	1.68
Walls	**Masonry Walls**		**Studded Partition**	
	100mm Blockwork	2.2	Timber/Plasterboard	0.5
	215mm Solid Blockwork	4.54	Timber/Lath & Plaster	1.1
	215mm Hollow Blockwork	2.5		
	100mm Brickwork	2.25		
	Plaster (Gypsum)	0.25		
	Rendering (pebble dash)	0.5		
Roof	**Industrial**		**Timber**	
	Insulated Profiled Decking	0.12	Slates (Natural)	0.35
	Trocal type, built up roof		Slates (Asbestos)	0.2
	Glazing	0.5	Tiles (Concrete)	0.5
	Pressed Steel Purlins	0.1	Battens	0.03
	Steel Frame	0.2	Felt	0.05
	Services	0.1	Insulation	0.05
	Suspended Ceiling	0.05	Timber Truss	0.2
			Plasterboard Ceiling …or…	0.15
			…Lath and Plaster	0.5

5.2 *Preliminary Structural Analysis*

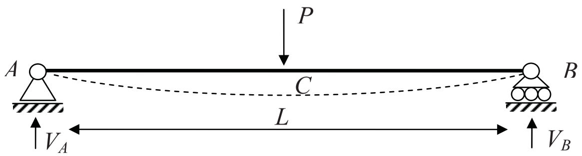
For preliminary design quick, reasonably accurate, appropriate forms of analysis are needed to determine load effects that the structure must be capable of resisting.

Statically Determinate Beams

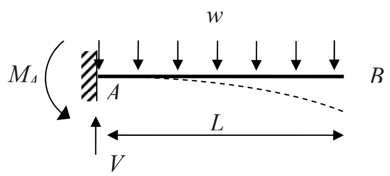
These are the basis of more complicated analyses: the usual cases need to be known.



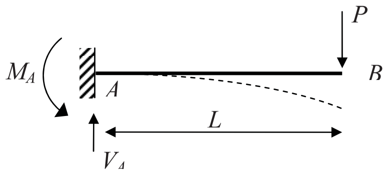
$$V_A = V_B = \frac{wL}{2},\; M_C = \frac{wL^2}{8},\; \delta_C = \frac{5wL^4}{384EI}$$



$$V_A = V_B = \frac{P}{2},\; M_C = \frac{PL}{4},\; \delta_C = \frac{PL^3}{48EI}$$

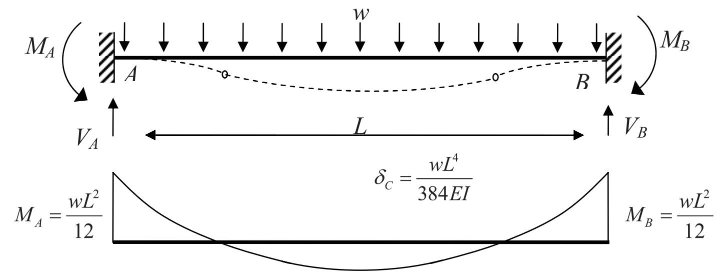


$$V_A = wL\,,\; M_A = \frac{wL^2}{2},\; \delta_B = \frac{wL^4}{8EI}$$

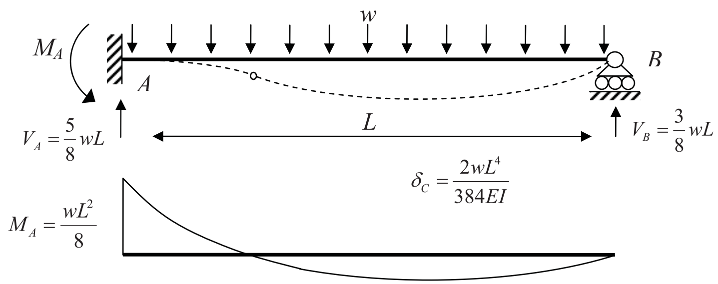


$$V_A = P\,,\; M_A = PL\,,\; \delta_B = \frac{PL^3}{3EI}$$

Statically Indeterminate Beams



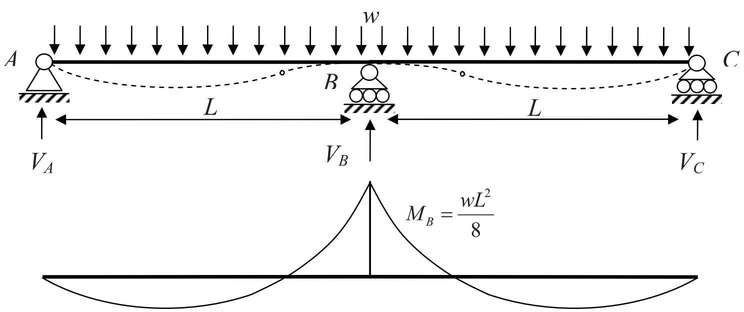
What is the moment at the centre of the beam?



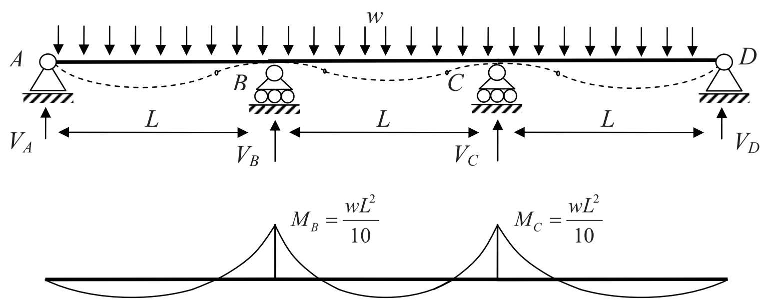
Again, what is the moment at the centre of the beam?

Continuous Beams

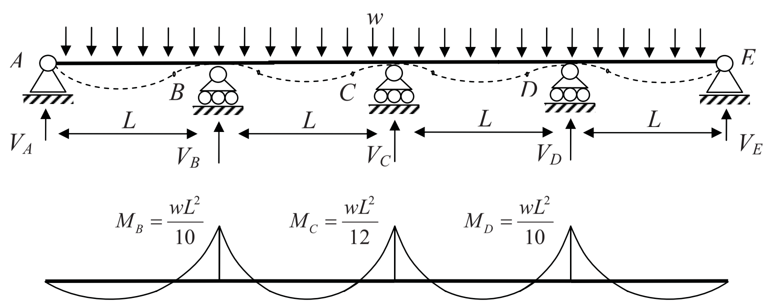
The reactions have been considered previously, so only moments are done here.



What is the moment at mid-span?



What are the mid-span moments of the beam?



What are the mid-span moments of the beam?

For more spans, the moments over the first interior supports are as shown, and the moments over other internal supports are taken as $\frac{wL^2}{12}$.

Determine the mid-span moments for the above beams.

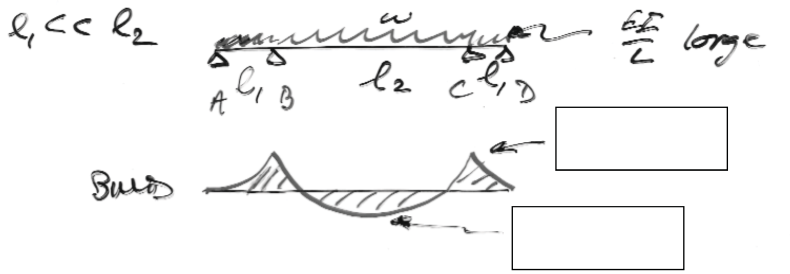
The IStructE Green Book – adapted – gives the following:

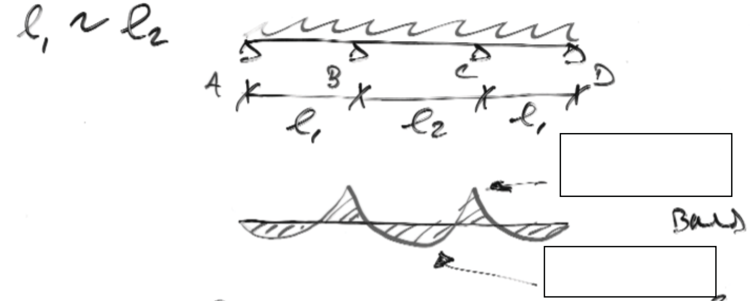
	At outer support	Near middle of end span	At first interior support	At middle of interior spans	At interior supports
Moment	0	$\frac{wL^2}{11.11}$	$\frac{wL^2}{9.09}$	$\frac{wL^2}{14.3}$	$\frac{wL^2}{12.5}$
Shear	$\frac{wL}{2.22}$	-	$\frac{wL}{1.66}$	-	$\frac{wL}{1.81}$

These figures allow for moment redistribution.

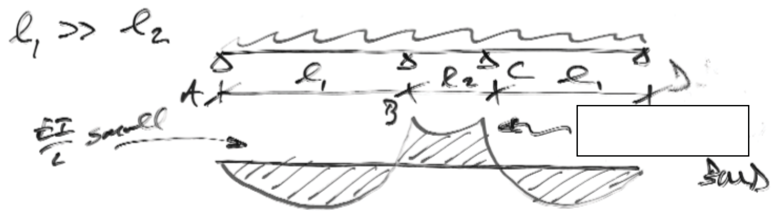
Unequal-Span Continuous Beams

Fill in the areas shown:

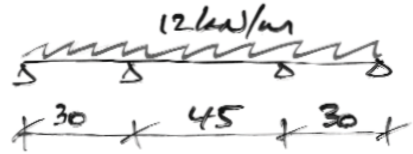




$l_1 = ? \times l_2$ for M_{BA} to equal M_{BC} ?



Estimate M_B and M_C for:



Workings:

Analysis for Preliminary Design

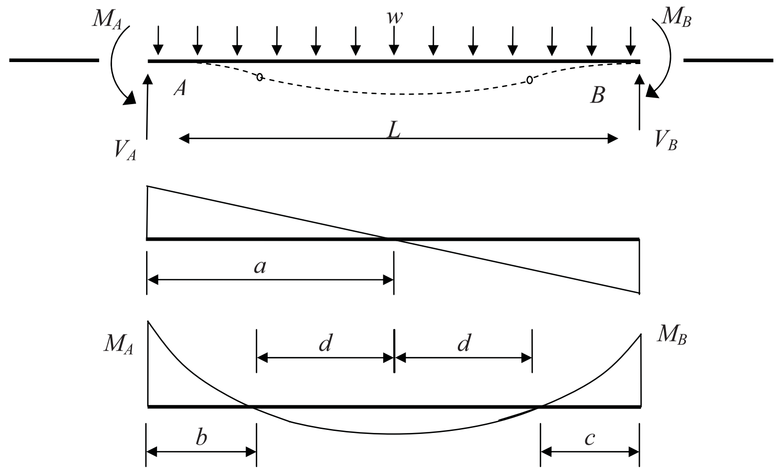
To get a rough idea of the rebar required for your scheme, it is usual to take the largest moment in your section:

1. What is your configuration – propped cantilever, 3-span beam etc…
2. Take the maximum value of moment (i.e. smallest value of denominator).

So for a 3-span beam take $\frac{wL^2}{10}$, for a fixed-fixed beam take $\frac{wL^2}{12}$ etc.

Typical internal span

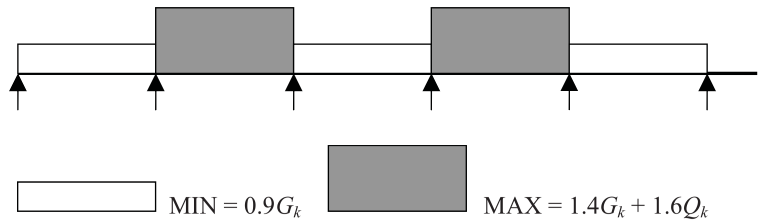
For more detailed design, or to find the positions of the points of contraflexure, the following is helpful:



Assuming only M_A and M_B are known, take moments about B to give: $V_A = \frac{wL}{2} - \frac{(M_A - M_B)}{L}$; sum the vertical forces to get $V_B = wL - V_A$. Therefore, $a = \frac{V_A}{w}$ and so $M_{max} = \frac{V_A^2}{2w} - M_A$. Defining $d = \frac{\sqrt{V_A^2 + 2wM_A}}{w}$, then the distances to the points of contraflexure are, b $b = a - d$ and $c = L - a + d$.

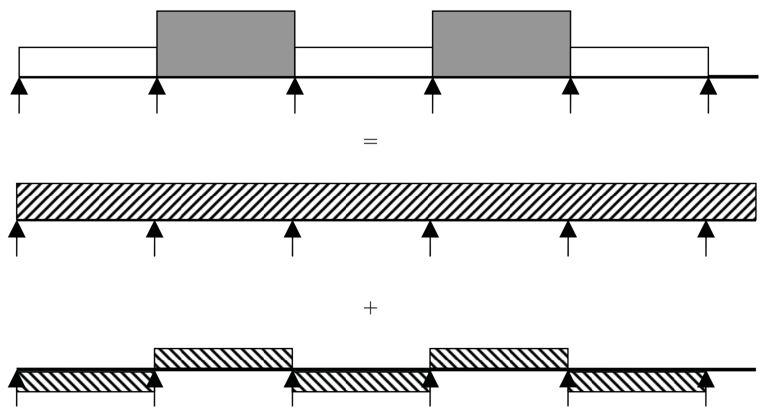
Load Patterning

For design of any continuous structure, it is necessary to consider load patterning to determine the design envelope for shear and moment etc:



MIN = $0.9G_k$ MAX = $1.4G_k + 1.6Q_k$

This presents problems with our moment formulae previously. A way around this is to do the following:



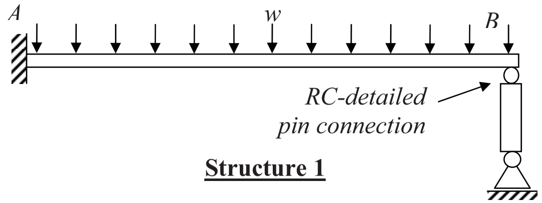
MIN + (MAX-MIN)/2; i.e. $1.15G_k + 0.8Q_k$

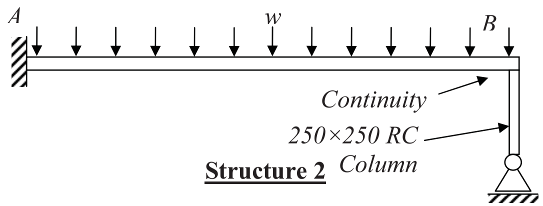
(MAX-MIN)/2; i.e. $0.25G_k + 0.8Q_k$

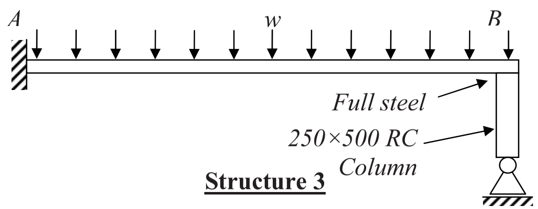
Why is this better?

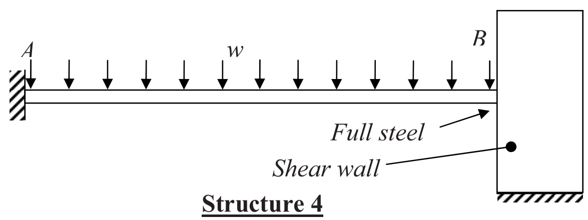
Qualitative Assessment for Moments

Consider the beam shown in the figures, should the RC beam (250W×300Dp.) be designed for moments at support B? And if so, what value would you take? Remember that large amounts of cracking are not desirable.







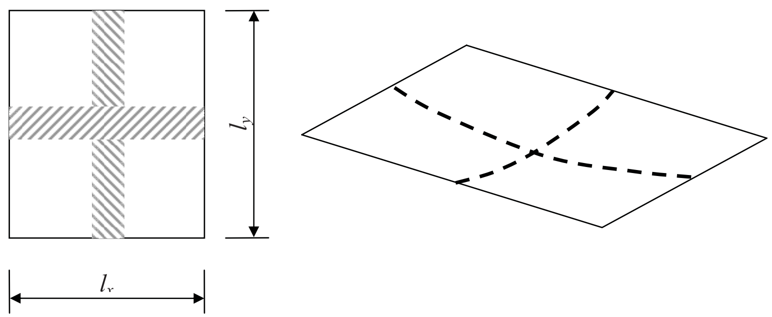


Moment

Simply Supported Rectangular Plate

The exact analysis of plates is considerably difficult. Some simplifying assumptions lead to easier methods of analysis that are reasonably accurate.

Take a rectangular plate, simply supported on all sides, loaded with a uniformly distributed load, w, and consider two central unit-width strips:



The load on the strip in the x-direction is w_x; likewise, w_y. Also, $w = w_x + w_y$. The deflection of each strip must be identical at the centre point: $\frac{5w_x l_x^4}{384EI_x} = \frac{5w_y l_y^4}{384EI_y}$. Hence, assuming $I_x = I_y$, and letting $r = \frac{l_y}{l_x}$, then $w_x = w_y r^4$ and as $r > 1$ the load taken in the x-direction is greater than that in the y-direction. Further, $w_y = w \cdot \frac{1}{1+r^4}$ and $w_x = w \cdot \frac{r^4}{1+r^4}$.

The moments taken in each direction are then:

$$M_x = \frac{w_x l_x^2}{8} = wl_x^2\left(\frac{1}{8}\cdot\frac{r^4}{1+r^4}\right) = \alpha_x w l_x^2 \quad M_y = \frac{w_y l_x^2}{8} = wl_y^2\left(\frac{1}{8}\cdot\frac{1}{1+r^4}\right) = \alpha_y w l_y^2$$

where α_x and α_y correspond to those of BS 8110, Pt. 1: 1997, Tb. 3.13.

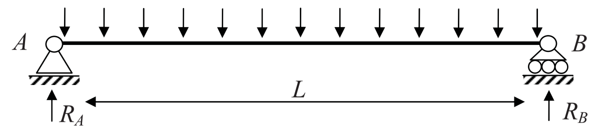
Other support conditions can be used, and a similar approach using compatibility of displacement can be used.

5.3 *Tributary Areas*

- Building loads usually originate as uniformly distributed over some area.
- Each structural element supports some of this load
- Therefore, each structural element has an associated area from which its load originates – its tributary area, tributary length and load width.
- This process is essentially that of tracing the load path through a structure.

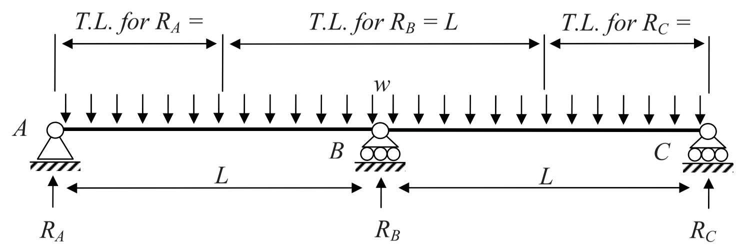
Tributary Length (TL)

Consider a simply supported beam:

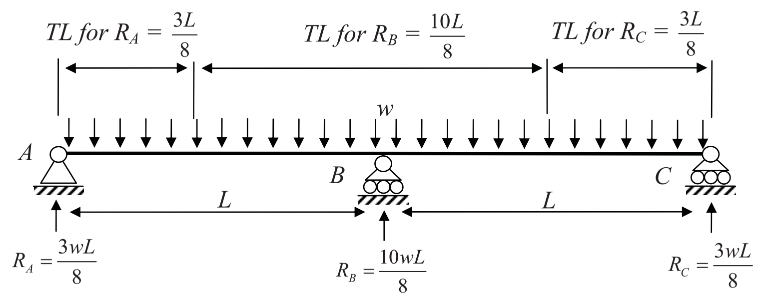


As the reactions $R_A = R_B = \frac{wL}{2}$, they have an TL of $L/2$.

By extension, for multiple simply supported spans:

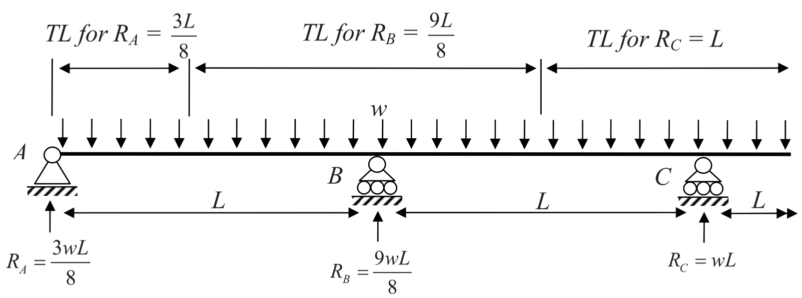


The TLs are different for continuous spans:



Therefore, the TL depends on the form of the structure.

For more than two spans, the intermediate spans have a TL of L. The end support has a TL of $\frac{3L}{8}$, whilst the second internal support has a TL of $\frac{5L}{8}+\frac{L}{2}=\frac{9L}{8}$:

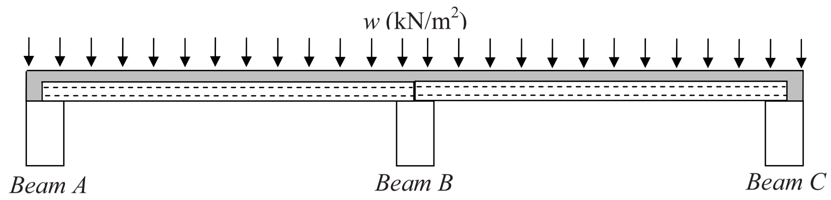


For spans of uneven length, in preliminary design, it is usual to interpolate based on the principles above.

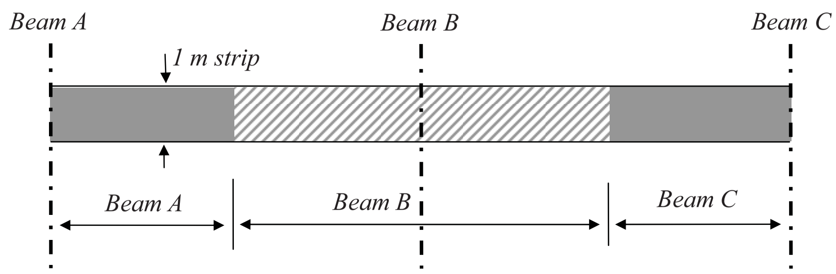
Load Width

For beams, the UDL arises from the loads applied to the flooring system.

The simple case is a one-way spanning simply-supported flooring system (say precast units):



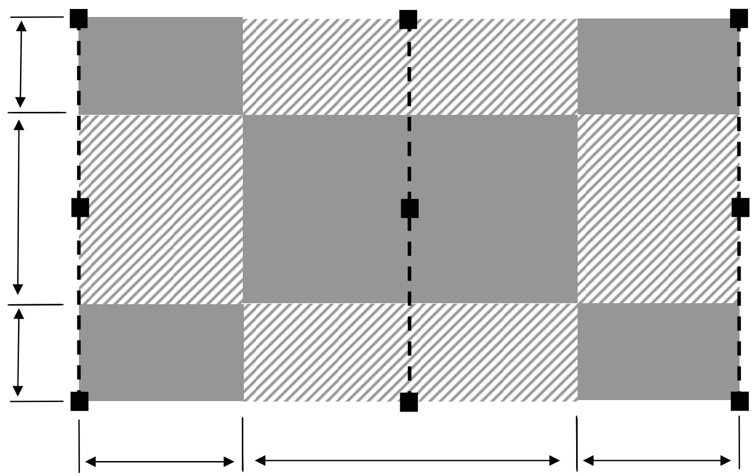
The load taken by each beam is derived from its load-width:



The load-width is the same as the TL for the "beam" of the floor system.

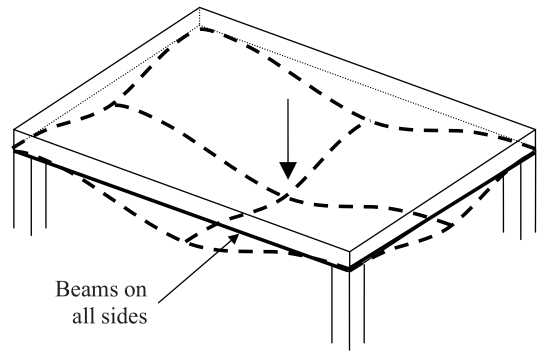
Tributary Area

The combination of the loadwidth (transverse to a beam) and the tributary length (longitudinal to the beam) result in the tributary area for a beam support. Fill in the lengths for following floor plate layout:

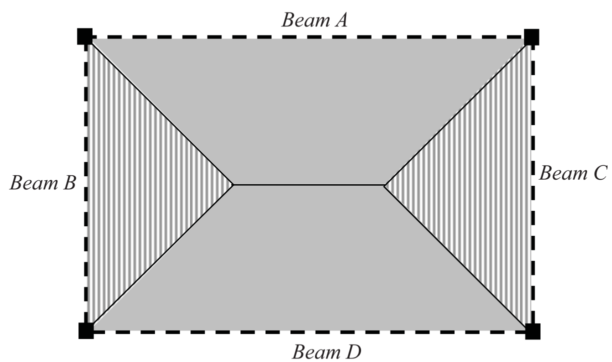


Note that the lengths indicated depend on the type of spans – continuous or simply supported, and result from application of the loadwidths and tributary lengths.

For two-way spans, the load is shared between the supports on all sides:



The tributary areas become more complex as a result:



But, for the internal columns, the tributary areas remain rectangular.

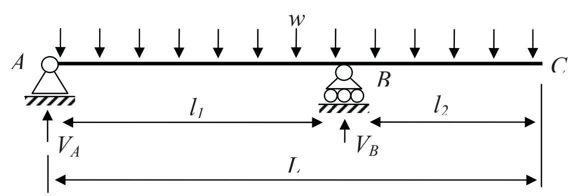
Sketch the loading applied to Beams A and D; and Beams B and C:

5.4 Preliminary Analysis of a Grandstand

The main structural problem with these types of structures is the roof cantilever.

Effect of a cantilever on support reactions

The general case is:



Taking $\sum \text{Moments about } A: \frac{wL^2}{2} = V_B l_1$ and $\sum F_Y = 0: V_A = V_B - wL$.

So, $V_B = \frac{wL^2}{2l_1} = wL\left(\frac{l_1 + l_2}{2l_1}\right)$. Let $r = \frac{l_2}{l_1}$; hence $V_B = wL\left(\frac{r+1}{2}\right)$ and $V_A = wL\left(\frac{r-1}{2}\right)$

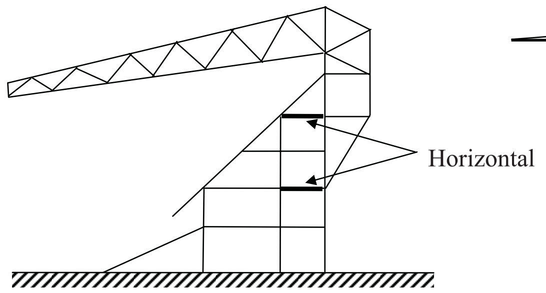
The term in brackets indicates the multiplier to be applied to the total load on the span which is wL.

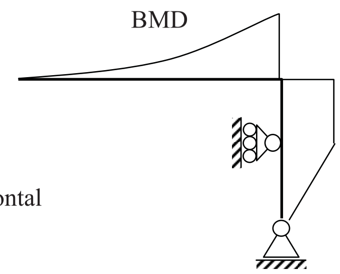
Note that when $l_1 = l_2$, $r = 1$, so $V_B = wL$ and $V_A = 0$. So for $r > 1$ the reaction at A is downwards.

For example, when $r = 2$, $V_B = 1.5wL$ and $V_A = -0.5wL$. So a 50% increase of the total load on the span occurs at the prop reaction, due simply to the geometry.

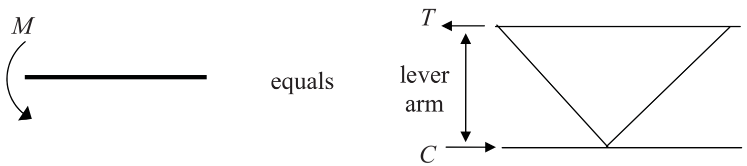
Case Study: Manchester United North Stand

A rough section through this grandstand and its overall structural idealization is:

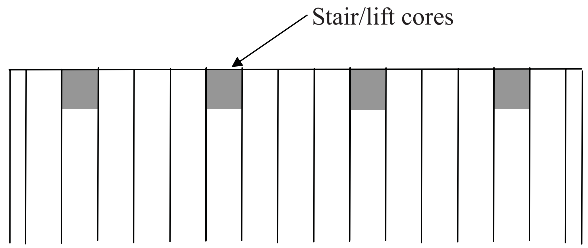




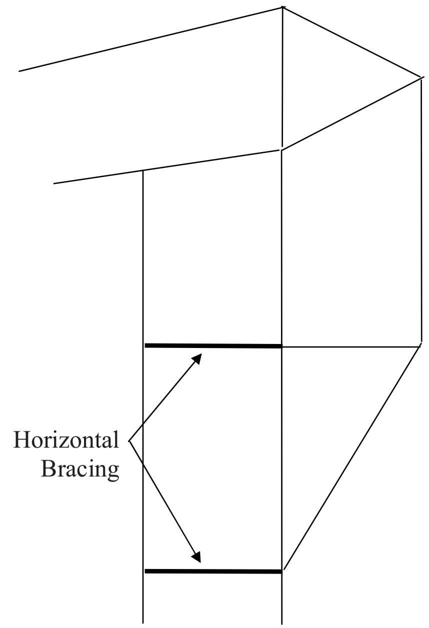
The 'bending moments' in the frame are resisted by the tension and compression of the top and bottom chord of the roof truss:



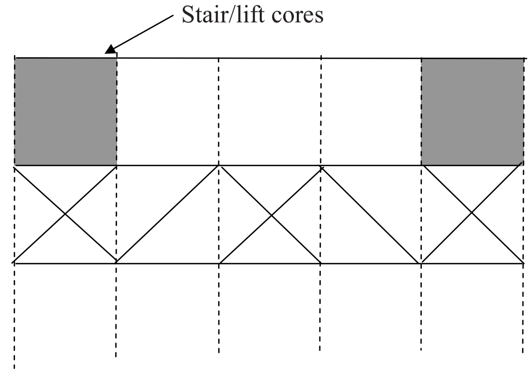
A key plan helps to explain the overall structure:



The members that contribute to the roof cantilever are shown below. What are the structural actions on the members?



What importance does the horizontal bracing have?



6. Preliminary Design

Structural design and analysis is often an iterative process. Section sizes may be needed before an analysis can be carried out to determine the load effects that sections must be designed for (which in turn dictates the size of the section).

- Preliminary sizes are obtained from experience and rules of thumb (other people's experience). These are used in the preliminary analysis.
- The results of the analysis are used to obtain more refined approximate designs.

In preliminary design, these steps are only carried out once. In detailed design the process can take many iterations.

For the usual materials and forms of construction we considered both of these stages.

6.1 Reinforced Concrete

Preliminary sizing

Bending members

Sized through span-effective depth ratios:

Construction		One-way			Two-way		Flat slab
	Imposed load (kN/m^2)	Simply-supported	Contin.	Canti.	Simply-supported	Contin.	
Slab	5	27	31	11	30	40	36
	10	24	28	10	28	39	33
Beam - Rectang.	any	10	12	6			
Beam – flanged	any	12	15	6			

The breadth of a beam is around *d*/3 but not always. Fire resistance requirements stipulate minimum widths, but for ease of construction, 200 mm is a rough minimum, giving about 2 hours fire resistance.

As a rough design check, limit the shear stress to 2 N/mm^2:

$$b \geq \frac{1000V}{2d} \text{ for } f_{cu} \geq 30\ \mathrm{N/mm^2}$$

where V is the maximum ultimate shear.

Columns

These rules only apply for stocky braced columns for which the minimum horizontal dimension is clear height/17.5.

Very roughly: $A_{col} = N \times 50$ (in mm^2) where N is the ultimate axial load in kN. Or assume an average stress across the column of about 25 N/mm^2.

A bit better: for f_{cu} = 35 N/mm^2 (where N is now in Newtons):

1% steel: $A_{col} = N/15$

2% steel: $A_{col} = N/18$

3% steel: $A_{col} = N/21$

To allow for moments in the columns, multiply the load from the **floor immediately above** the column (this allows for patterned loading) by:

1.25 for interior columns;

1.5 for edge columns;

2.0 for corner columns.

Loads from other floors may be considered fully axial; sketch these requirements:

Reinforced Concrete – Approximate Design

Bending members:

Percentage area of steel for a singly reinforced section:

$$\rho_s \approx \left(\frac{M}{bd^2}\right) \Big/ \pi$$

The π is to make it look fancy! Any number from 3.1 (for a loose design) up to about 4.3 (for a tight design) can be used. Note that $A_s = \frac{\rho_s}{100} bd$. Combining these two expressions leads to a very quick estimate of:

$$A_s = \frac{M}{300d}$$

The 2 N/mm^2 limiting shear stress is a sufficient preliminary shear check.

Columns:

Given an area of column, its resistance can be got by considering a mean 'resistance stress' as:

$$0.35 f_{cu} + \frac{\rho}{100}\left(0.67 f_y - 0.35 f_{cu}\right)$$

Alternatively, roughly: $\rho \approx \dfrac{N/A_{col} - 14}{3}$

Derive this:

Punching shear:

The column reaction, V_t, is modified as follows to take account of moment transfer:

- Internal Columns: $V_{eff} = 1.15V_t$;
- Edge/Corner Columns: $V_{eff} = 1.4V_t$.

1. Check maximum shear at column face:

$$v_{max} = \frac{V_{eff}}{u_0 d} \le 0.8\sqrt{f_{cu}} \text{ or } 5 \text{ N/mm}^2$$

where u_0 is the perimeter of the column.

2. Shear stress at the critical section, $1.5d$ from the face of the column:

$$v = \frac{V}{ud}$$

$$u = 2a + 2b + 8\mu d$$

where a and b are the plan dimensions of a rectangular column and μ is the perimeter multiplier of d: in this case, $\mu = 1.5$. If:

$v \le v_c$: No shear reinforcement required.

$v \le 2v_c$: Link reinforcement may be used.

$v > 2v_c$: Alternative proven system to be used.

For preliminary design, it is sufficient to pass Step 1 and to know that $v \le 2v_c$ at the critical perimeter.

A quick v_c is obtained by simplifying the BS 8110 and BS 5400 Pt 4 expressions:

$$v_c = 0.7(\rho_s)^{1/3} \text{ N/mm}^2$$

Even quicker, but less accurate, use $v_c = 0.60$ N/mm^2.

6.2 Prestressed Concrete

Generic preliminary sizing may be based on a span depth ratio of about 36. Usually, the manufacturers' data sheets are used instead. These normally have load versus span charts.

Sample information from Breton Roecrete data is given:

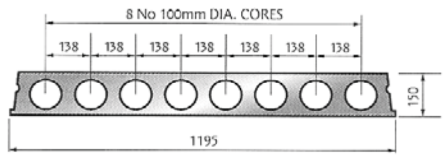
- Hollowcore slabs only:

Safe Superimposed (Service) Loads - Kn/m²														
Unit	Mu	Vco	Effective Span - M											
Depth	(Knm)	(Kn)	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
150	72.4	96.1	17.0	10.1	6.5	4.3								
200	142.5	93.3			11.3	9.1	6.4	4.6						
250	247.8	101.5					13.0	9.8	7.3	5.2				
300	338.0	140.8						13.6	10.6	8.2	6.4	5.0	3.8	
340	412.9	151.6							12.0	10.0	7.7	6.0	4.6	3.5

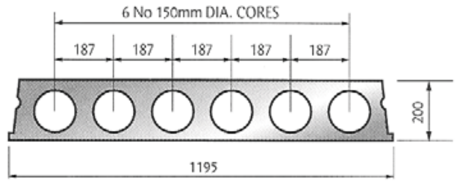
- Composite hollowcore slabs:

Safe Superimposed (Service) Loads - Kn/m²												
Overall Structural Depth	Unit Depth (mm)	Self Wt (KN/m²)	Mu ult (KNM)	Effective Span - M								
				7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
200	150	3.5	102.4	4.6	2.2							
250	200	3.8	188.3	11.5	7.8	5.2	3.3					
300	250	4.3	322.4		12.2	10.4	8.6	6.5	4.8	3.1		
350	300	5.0	412.5			14.1	12.1	9.1	6.8	5.1	3.7	
390	340	5.9	468.7					9.6	7.8	5.8	4.0	2.5

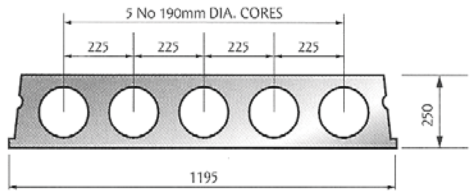
Hollowcore Slabs



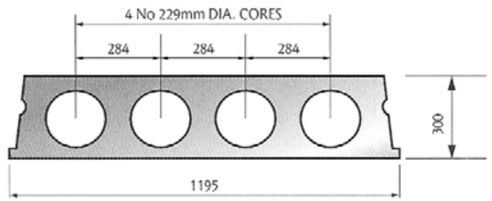
150mm Deep Prestressed Hollowcore Unit
Self WT: 2.3 Kn/m²



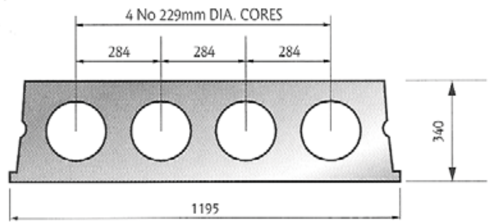
200mm Deep Prestressed Hollowcore Unit
Self WT: 2.6 Kn/m²



250mm Deep Prestressed Hollowcore Unit
Self WT: 3.1 Kn/m²

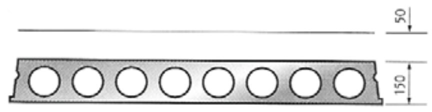


300mm Deep Prestressed Hollowcore Unit
Self WT: 3.8 Kn/m²

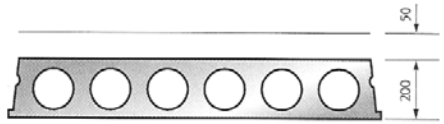


340mm Deep Prestressed Hollowcore Unit
Self WT: 4.7 Kn/m²

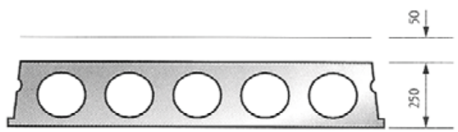
Composite Hollowcore



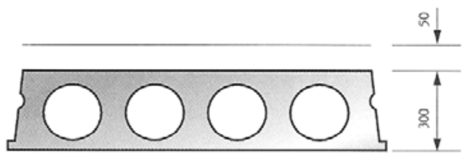
200mm Deep Composite Unit
Composite WT: 3.5 Kn/m²



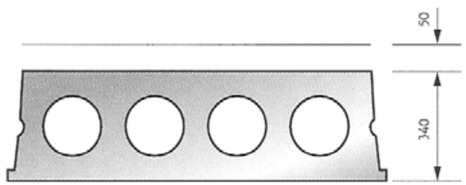
250mm Deep Composite Unit
Composite WT: 3.8 Kn/m²



300mm Deep Composite Unit
Composite WT: 4.3 Kn/m²



350mm Deep Composite Unit
Composite WT: 5.0 Kn/m²



390mm Deep Composite Unit
Composite WT: 5.9 Kn/m²

Example – Design of Hollow Core Slab

Design a PSC hollowcore slab to span 7 m in an office building.

BS6399: Imposed loading, offices = 2.5 kN/m^2
Partitions (assume masonry) = 2.5 kN/m^2
Total Imposed = 5.0 kN/m^2

Table for *Composite Hollowcore* floors tells us that a 200mm deep slab with a 50 mm structural screed will carry an imposed (SLS) load of 11.5 kN/m^2. Therefore, try a 200 mm deep hollowcore slab.

Self wt including screed = 3.8 kN/m^2 (from table)
Ceilings & Services = 0.5
Total DL = 4.3 kN/m^2

ULS load,

$$w_u = 1.4(4.3) + 1.6(5) = 14.0 \text{ kN/m}^2$$

ULS moment at centre is

$$wl^2/8 = 1.2\times14.0(7)^2/8 = 85.8 \text{ kNm/m.}$$

The 1.2 m is the width of the precast unit; hence the line load on the unit is 1.2×14 kN/m. The ultimate moment capacity of the composite floor is 188.3 kNm/m hence the floor has ample ultimate capacity.

6.3 Steel – Non-composite

Typical span/depth ratios for different forms of construction and elements:

Element	Typical span (m)	Span/depth ratio
Floor UBs	4 – 12	15 – 18
Slimfloor	6 – 9	25 – 28
Castellated beams	4 – 12	14 – 17
Transfer beams	6 – 30	10
Trusses supporting floors	6 – 30	10
Plate girders	10 – 30	10 – 12
Parallel chord roof truss	10 – 100	12 – 20
Pitched roof truss	8 – 20	5 – 10
Light roof beams	6 – 60	18 – 30
Lattice roof truss	5 – 20	12 – 15
Space frame (w/ pre-camber)	10 – 100	15 – 30
Columns: UC: single storey	2 – 8	20 – 25
: multi storey	2 – 4	7 – 18
Columns: hollow sections: single storey	2 – 8	20 – 35
: multi storey	2 – 4	7 – 28

Columns:

UC Section	No. of storeys
203 UC	3
254 UC	5
305 UC	8
356 UC	12

6.4 Composite Construction

Layout:

For maximum efficiency, the secondary beams should be longer than the primary beams. The optimum ratio is 4/3. Sketch this layout:

Slab:

- RC: 125-150 mm thick
- Metal deck: 115-175 mm, spanning 2.5 to 3.6 m.
- Precast units: 75-100 mm with 50-200 mm topping can span 3 to 8 m.

A 150 mm deep overall slab with 60 mm decking spans about 2.8-3.5 m depending on mesh and concrete density.

Beam:

Initially size as 80% of non-composite.

For a better check, size beam with Z = (non-composite Z)×(1.6 to 2).

7. Car Park Layout Design

7.1 Introduction

Car park layout design usually falls to the structural engineer. This is because the structural layout and car-park design are integral to one another. Simply put, you can't put a column in a driving lane. Balancing the car park and structural layout is important for the floors overhead. Often two different structural layouts are used for the car park and for (say) overhead offices. A transfer structure is needed in between to link these two layouts.

Defintions

Bay: the parking space for a single vehicle.

Aisle: the driving lane adjacent to the parking bays.

Bin: A 'unit' comprising bays on both sides of an aisle.

Usage

The layout of a car park depends critically on its foreseen use:

- Short or long stay;
- Regular or irregular users;
- Small or large scale parking.

Some typical examples are:

Apartments and Offices:

Long stays — regular users — small scale parking.

Therefore:

- Allow some delays;
- Allow narrower geometries.

Hospital and Airport visitor car parks:

Short stays — irregular users — large car parks.

Therefore:

- Generous geometries;
- Shorter delays per user required;
- High turnover and less congestion required;

Hence, the flow of traffic and the location of exits must be carefully thought through.

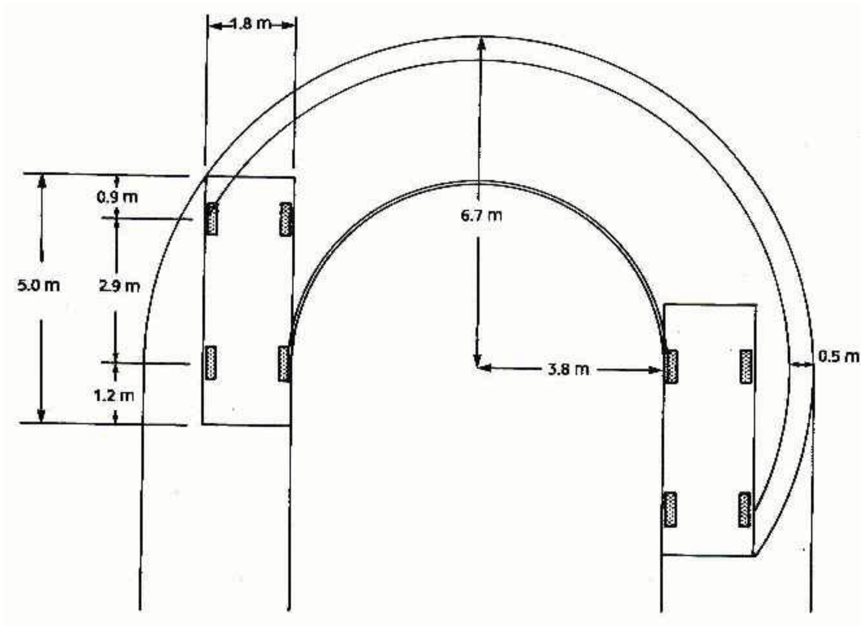
Notes:

7.2 Car Sizes and Swept Path

Car sizes are given in the IStructE's *Design Recommendations*(…) as:

Car Type	*L* (m)	*W* (m)	*H* (m)
Small	3.95	1.75	1.75
Standard	4.75	2.06	1.85
Large	5.40	2.24	2.05
MPV	5.10	2.20	1.90
4×4	5.05	2.25	2.05
Note: Width includes wing mirrors, Height excludes roof bars/boxes etc. Taken from a 1999/2000 UK Review.			

The actual design criteria (given later) are governed by the swept path of a large car. In the case of special design, outside the limits of the recommendations, the figure below must be used to verify the design's adequacy. Using CAD software, the figure below can be superimposed on any part of a proposed layout to verify that there are no clashes. This is not necessary for usual standard designs, however.



Notes:

7.3 Bay, Aisle and Bin Dimensions

Based on the car sizes and stay requirements, the bay sizes are:

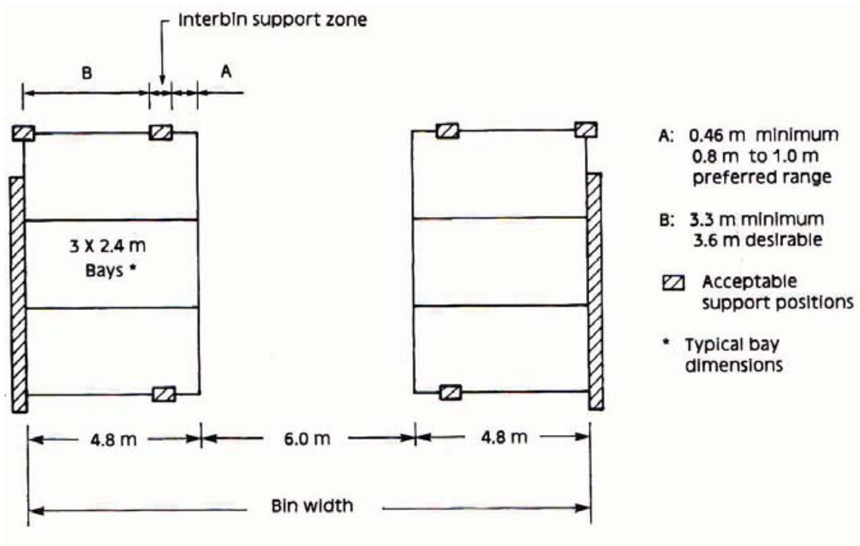
Type	***L*** **(m)**	***W*** **(m)**
Short Stay (≤ 2 hrs.)	4.8	2.5
Long Stay	4.8	2.3
Disabled	4.8	3.6

These dimensions are to be clear of any projections (for example, columns). Typically we design for 2.4 × 4.8 m to make the structural grid regular, as will be seen.

For driving lanes we allow for the largest of vehicles. However, the individual parking bays can be designed for a more reasonable vehicle size. Lanes must also be designed to allow cars back out of a space. Therefore, 1-way and 2-way lanes are of similar width. Applying the swept path configuration, the recommendations for parking dimensions are given as:

Parking angle	**Aisle Width (m)**	**Bay Width (m)**	**Bin width (m) (4.8 m length)**
90° 1-way	6.00	2.4	16.55
90° 2-way	6.95	2.4	15.60
60°	4.20	2.4	14.95
45°	3.60	2.4	13.80

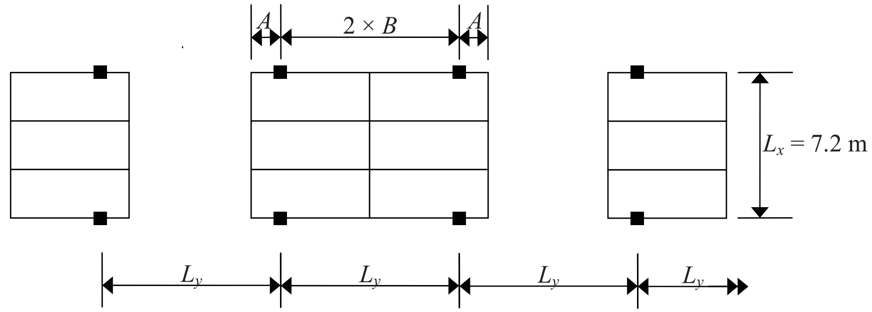
The next figure illustrates the implications for the bin layout. For 3 spaces, the 2.4 m bay width allows for 300 mm extra (over the minimum width of 2.3 m) in which the column can be placed.



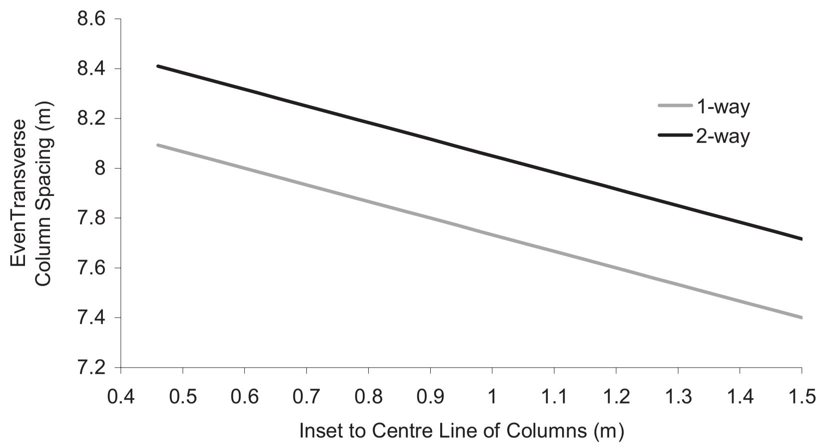
In the figure above, the dimension A is usually about 0.9 m to the column centre line.

Reinforced concrete structures:

This car park structure is normally used in a mixed-use development, e.g. office block, apartments etc. To keep a regular grid of columns, the transverse dimension must balance the aisle width and dimension A. The longitudinal dimension is clearly 7.2 m. The layout shown in the following figure is usual.



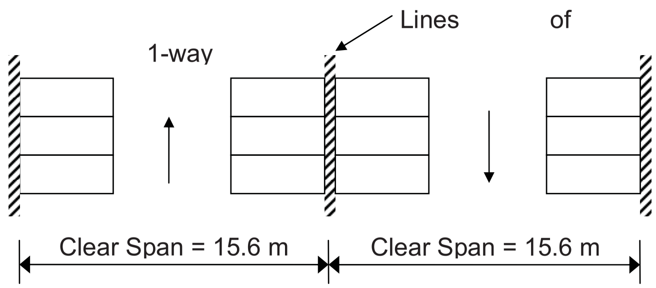
By applying the requirements for A and B, we can relate the inset to the spacing L_y:



Of course it is not absolutely necessary to have equally spaced columns in the transverse direction, but it makes the analysis, design, and construction easier.

Precast Concrete or Steel Structures:

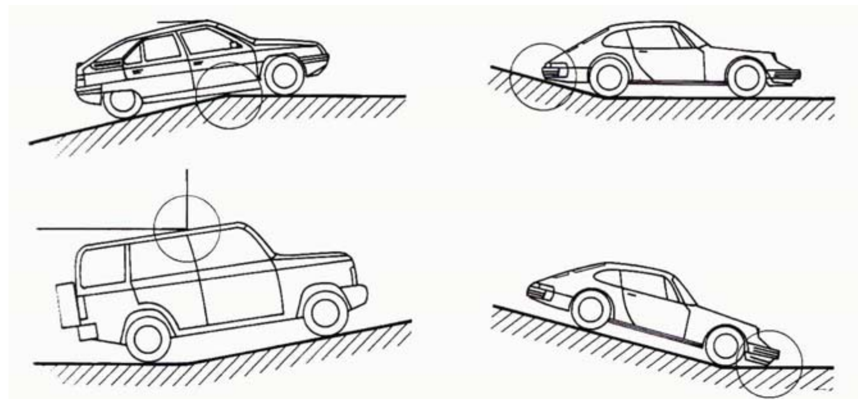
When the structure's main purpose is as a car park, a more efficient structure is to use long span concrete or steel beams, in conjunction with precast concrete slabs. The beams and slabs can span in either direction. This is possible due to the relatively light live load of a car park (2.5 kN/m^2). The layout is:



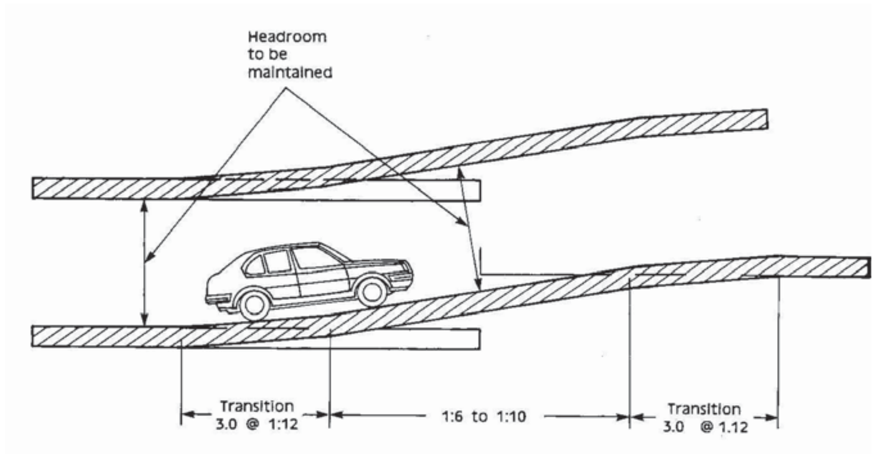
This layout also works for post-tensioned RC floor-slabs (e.g. Dublin Airport).

7.4 Headroom and Ramps

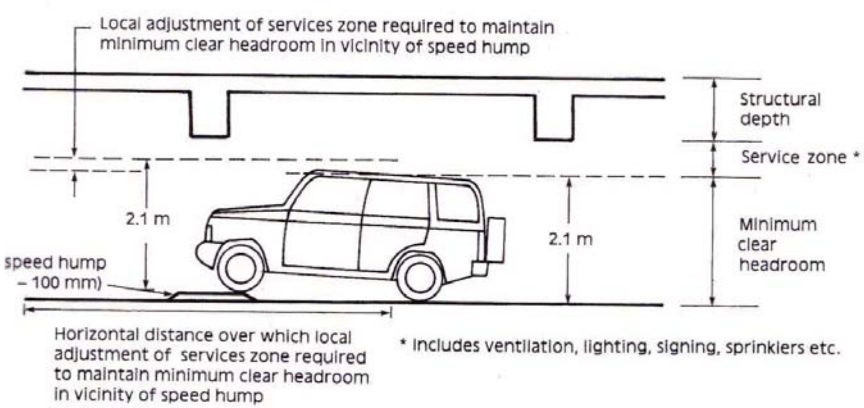
At changes in level, pinch points occur as shown:



In such cases, transition ramps are used at the start and end of each ramp:



The storey height is related to the headroom required as shown in the next figure. The minimum headroom is 2.10 m and this will allow for all MPVs and 4×4s. Where provision is required for high-top converted vehicles for disabled people, the minimum headroom is 2.60 m.



7.5 Ventilation

Car parks must be adequately ventilated due to noxious fumes. To save on mechanical ventilation, natural ventilation is used as much as possible. For this, openings should have an aggregate area of 2.5% of the area of the parking space at that level and be distributed so as to provide effective cross ventilation.

This requirement often results in 'planters' around single-storey basement car parks. Sketch an example:

7.6 *Miscellaneous*

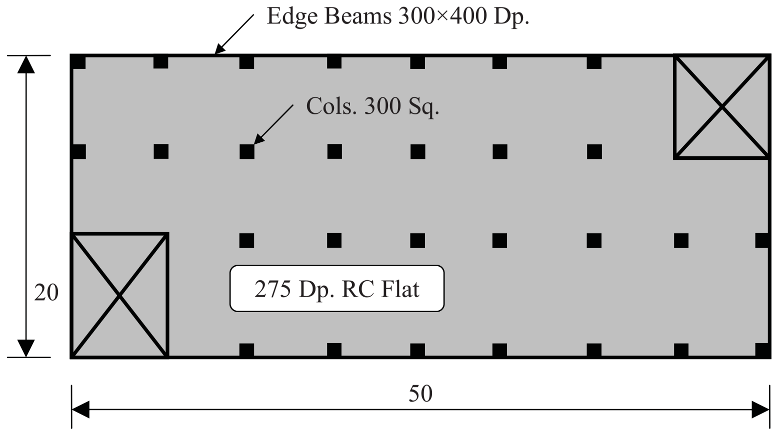
- Durability requirements for car parks are usually more onerous than other structures as chlorides can be brought in by the vehicles.
- Fire protection of structures needs to be considered, especially for structural steel elements, though uncased steel elements are usual in car parks.
- Traffic management can be difficult for large car parks; adequate design is essential.
- The car park surface is usually sloped for drainage: the minimum fall is 1:60, the maximum, 1:20.
- Expansion joints need to be detailed to avoid water ingress.

8. Examples

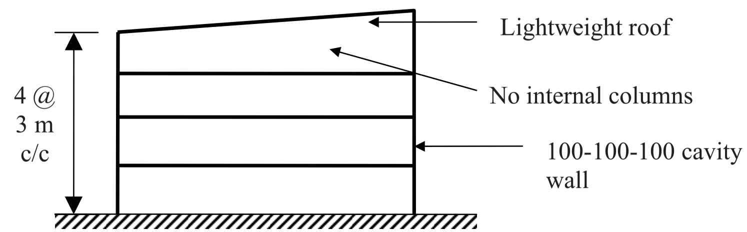
8.1 *Load Takedown*

Problem

Do a load takedown for the following structure and determine the ultimate and service loads for the pad footings under the columns.



Plan



Section

Use: Speculative offices.

Columns are evenly spaced.

Plant rooms located over lift/stair cores.

Ignore takedown for lift/stair cores – only do it for the columns.

Solution

The problem is vague; some assumptions are needed:

- The roof structure is a one-way spanning roof truss system with some glazing;
- The ground floor walls are supported by strip footings;
- The ground floor slab is ground-bearing;
- Only vertical loads need be considered as they are the critical case for th pad foundations.

Evenly spaced columns means that:

- E-W spacing is: 50/8 = 6.25 m;
- N-S spacing is: 20/3 ≈ 6.7 m.

Structural Actions

A flat slab is a two-way spanning reinforced concrete slab. Therefore the tributary areas are derived from those of the ‘continuous’ form of tributary lengths studied previously. The cladding is non-structural: the masonry is supported on the edge-beam at each storey level. The edge beams are supported at each level by the perimeter columns. The roof trusses span N-S onto each perimeter column.

Exercise: you should be able to sketch the actions/load paths just described.

Notes:

Element Loads

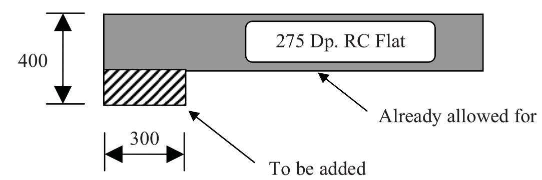
The loads upon each element are required:

Floor	G_k (kN/m^2)	Q_k (kN/m^2)
RC Slab (0.275 × 24) Live load (inc. services partitions etc.)	6.6	5.0
Σ =	6.6	5.0
$w_{ser} = 6.6 + 5.0 = 11.6$ kN/m^2	$w_{ult} = 1.4 \times 6.6 + 1.6 \times 5.0 = 17.24$ kN/m^2	

Composite Load Factor: 17.24/11.6 = 1.486, so dead load governs.

Perimeter Load	G_k (kN/m^2)	Q_k (kN/m^2)
100 Block 100 Brick	2.2 2.25	
Σ =	5.35	0
$w_{ser} = 4.45$ kN/m^2	$w_{ult} = 1.4 \times 4.45 = 6.23$ kN/m^2	

Beam downstand (24 × 0.125 × 0.3) = 0.9 kN/m service, 1.26 kN/m ult.



Composite Load Factor: 1.4. The perimeter line load caused by the wall is:

w_{ult} = 0.6 (proportion of wall) × 3 m (storey height) × 6.23 kN/m^2 + 1.26 = 12.5 kN/m

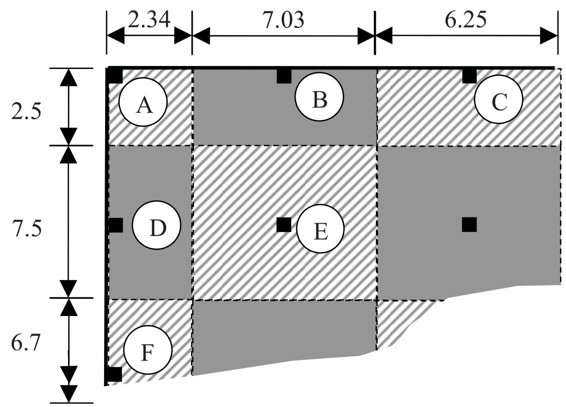
w_{ser} = 12.5/1.4 = 9 kN/m

Roof	G_k (kN/m^2)	Q_k (kN/m^2)
Truss	0.2	
Purlins	0.1	
Glazing	0.5	
Services	0.1	
Imposed (BS 6399: Pt. 3: 1988)		0.6
Σ =	0.9	0.6
$w_{ser} = 0.9 + 0.6 = 1.5$ kN/m^2	$w_{ult} = 1.4 \times 0.9 + 1.6 \times 0.6 = 2.22$ kN/m^2	

Composite Load Factor: 2.22/1.5 = 1.48, so dead load is slightly more important.

Tributary Areas

Tributary areas are worked on the basis of the $\frac{3}{8}L$, $\frac{9}{8}L$ and L formulae for continuous spans. As a result, the columns take many different tributary areas:



Column	Trib. Area (m^2)
A	5.9
B	17.7
C	15.7
D	17.6
E	53.0
F	15.7
Typ. Int.	41.9

It is best to rationalize all of these areas into:

- Worst internal – column E;
- Worst perimeter – column B; because the tributary area is similar, and although the perimeter length is longer (greater cladding load), Column D only takes nominal load from the roof, whereas B directly supports it.
- Corner column, A.

Critical Internal – Column E	P_{ult} (kN)	P_{ser} (kN)
Column self weight		
= 24 × 0.3 × 0.3 × 3 stories × 3 m per storey	19.4 × 1.4	19.4
Load per floor:		
Ult. = 53 m^2 × 17.24 kN/m^2	914	663
Ser. = 914/1.486	(× 3 stories)	(× 3 stories)
Σ =	**2770**	**2009**

Perimeter – Column B	P_{ult} (kN)	P_{ser} (kN)
Column self weight (extra storey)	25.9 × 1.4	25.9
= 4/3 × 19.4		
Load per floor: (floor and wall)		
Ult. = 17.7 × 17.24 + 7.03 × 12.5	393	
Ser. = 17.7 × 11.6 + 7.03 × 9	(× 3 stories)	269
Roof load:		(× 3 stories)
Ult. = 10 × 6.25 × 2.22	139	
Ser. = 139/1.48		94
Σ =	**1355**	**927**

Corner – Column A	P_{ult} (kN)	P_{ser} (kN)
Column self weight (extra storey)	25.9 × 1.4	25.9
= 4/3 × 19.4		
Load per floor: (floor and wall)		
Ult. = 5.9 × 17.24 + 4.85 × 12.5	163	
Ser. = 5.9 × 11.6 + 4.85 × 9	(× 3 stories)	113
Roof load: (half load width)		(× 3 stories)
Ult. = (5/2) × 6.25 × 2.22	70	
Ser. = 139/1.48		47
Σ =	**596**	**412**

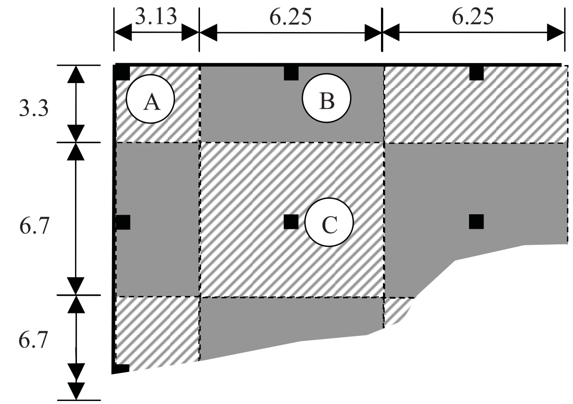
Note: Perimeter line load length: 2.34 + 2.51 = 4.85 m

Notes:

Quicker Solution

- Ignore edge-beam downstand, column self weight and all serviceability calculations, and just use the composite load factor (1.486) at the end;
- Compensate for these inaccuracies by rounding the ultimate load up to, say, 17.5 kN/m^2;

Use equal load widths, i.e.:



Note that the calculations that remain unchanged are the ultimate loads for the floor, perimeter, and roof loadings. Thus:

Column C

$P_{ult} = 3 \times 6.25 \times 6.7 \times 17.5 = 2200$ kN; $P_{ser} = 2200/1.486 = 1480$ kN

Column B

$P_{ult} = 2200/2 + 6.25 \times 11.2 + 10 \times 6.25 \times 2.22$

$= 1100 + 70 + 139 = 1310$ kN; $P_{ser} = 1310/1.486 = 881$ kN

Column A

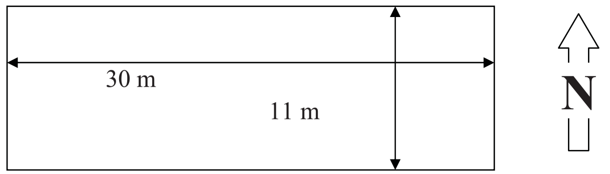
$P_{ult} = 2200/4 + 70 + 139/2 = 690$ kN; $P_{ser} = 690/1.486 = 464$ kN

Compare to the previous results, and examine where the inaccuracies come from. The results are approximate, but this is good enough for many preliminary purposes.

8.2 *Beams, Slab and Column Example*

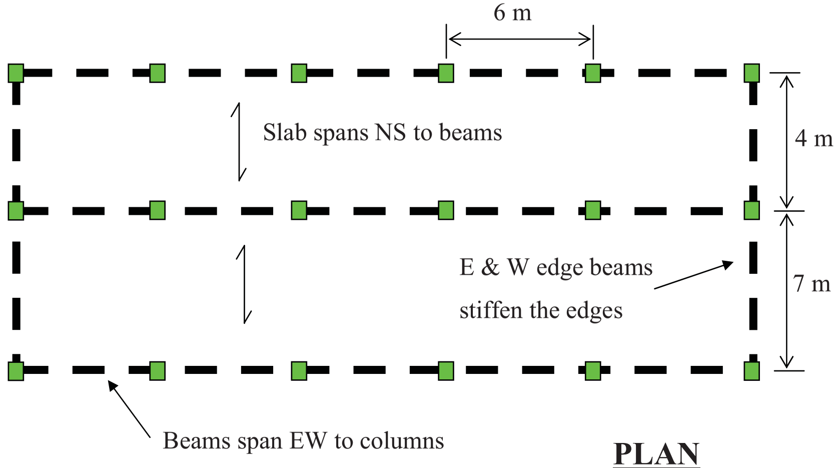
Problem

Do a complete preliminary design for slabs, beams and columns of this 4-storey office building. There is no basement car-parking.



PLAN

The building is stabilised with shear walls. Assume an in-situ slab spanning onto beams and (continuous) beams spanning onto columns.



PLAN

Note:

1. Central line of columns offset to allow for a 3m corridor
2. Take cover to reinforcement to be 20 mm for 1-hour fire protection.
3. Assume in-situ slab, one-way spanning.

Solution

The general procedure is:

1. Determine approximate member sizes;
2. Calculate the loading, both dead (from previous step) and live (from tables);
3. Analyse the structure for bending moments/shear forces and axial loads;
4. Design each of the elements for the bending moments etc.

In the following each element is considered in turn. For each element identify the four steps just described.

Preliminary Design of Slab

Get slab depth from span/depth ratios:

End span of 1-way continuous → span/depth = 27

$\Rightarrow d \ = 7000/27 = 259$ mm

$\Rightarrow h \ = 259 + 20$ cover $+ 16/2$ main bar

$= 287$ mm

In which 16 mm is the assumed maximum bar diameter to be used in a slab. Round off to nearest 25 mm:

$\Rightarrow$ 300 mm, say.

Hence:

$d = 300 - 20 - 16/2 = 272$ mm

Dead load:

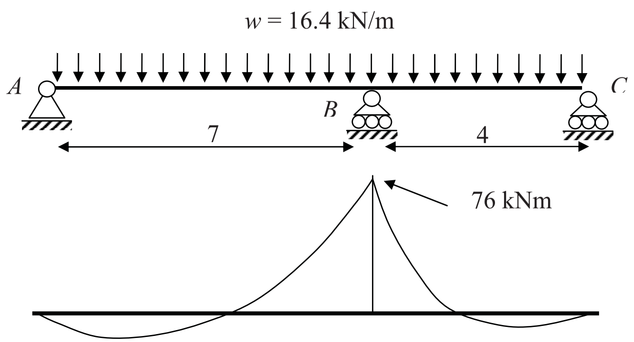
24×0.3 (slab s.w.) + 0.5 (ceilings + services)

$= 7.7$ kN/m^2

Imposed load:

2.5 (occupancy) + 1 (partitions) = 3.5 kN/m^2

Hence, ULS = 1.4(7.7) + 1.6(3.5) = 16.4 kN/m^2



The spans are uneven and so we cannot assume that the moment at B is $wL^2/8$ because we don't know which L to take. Conservatively, we could take the large L and design for it:

$$(M_B)_{AB} = \frac{wL^2}{8} = \frac{16.4 \times 7^2}{8} = 101 \text{ kNm}$$

which is much bigger than the same moment for the span BC:

$$(M_B)_{BC} = \frac{wL^2}{8} = \frac{16.4 \times 4^2}{8} = 33 \text{ kNm}$$

We could split the difference (101-33 = 68) evenly:

$$M_B = \frac{(M_B)_{AB} + (M_B)_{BC}}{2} = 67 \text{ kNm}$$

The smart way is to split the difference in inverse proportion to the lengths (**why?**):

$$(M_B)_{AB} = 101 - 68 \times \frac{1/7}{1/7 + 1/4} = 76 \text{ kNm}$$
$$(M_B)_{BC} = 33 + 68 \times \frac{1/4}{1/7 + 1/4} = 76 \text{ kNm}$$

As the answers are the same it means the joint is balanced ($\sum \text{Moments about } B = 0$). This is also the exact answer from a 'fancy' analysis.

For the reinforcement we use the quick formula:

$$\rho_s \approx \left(\frac{M}{bd^2}\right) \Big/ \pi$$

$$\rho_s \approx \left(\frac{76 \times 10^6}{1000 \times 272^2}\right) \Big/ \pi = 0.327$$

$$\begin{aligned} A_s &= \frac{\rho_s}{100} bd \\ &= \frac{0.327}{100} \times 1000 \times 272 \\ &= 890 \text{ mm}^2 \end{aligned}$$

Compare this to the quicker formula:

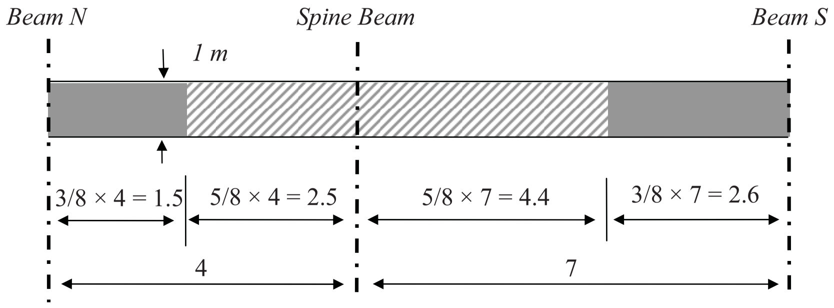
$$A_s = \frac{M}{300d} = \frac{76 \times 10^6}{300 \times 272} = 932 \text{ mm}^2$$

T16 bars at 200 mm c/c provide 1005 mm^2 per metre. Hence, choose this as our design is approximate anyway. The 'proper' design method gives us $A_s = 838 \text{ mm}^2$ so our design is conservative, yet approximate.

Preliminary Design of E-W Beams

We will only look at the central 'spine' beam as this will be critical. As we saw in the Load Takedown example, the downstand adds very little weight so for our approximate design we will ignore it. Hence we can determine the moments etc, first (which is our preferred route).

Using the load widths we have:



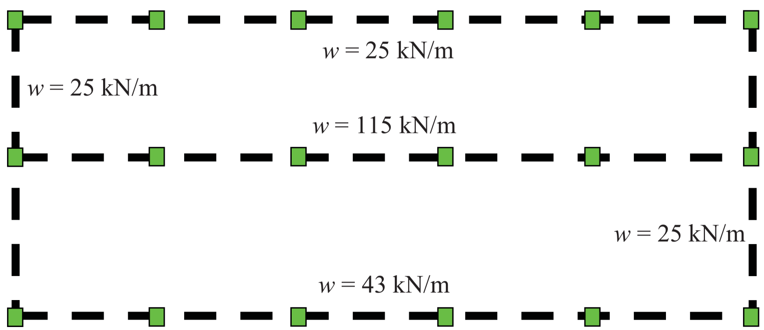
PLAN

Hence the loadwidth on the spine beam is 2.5 + 4.4 = 6.9 m and the load per meter is:

$$w_u = 16.4 \times 7 = 115 \text{ kN/m}$$

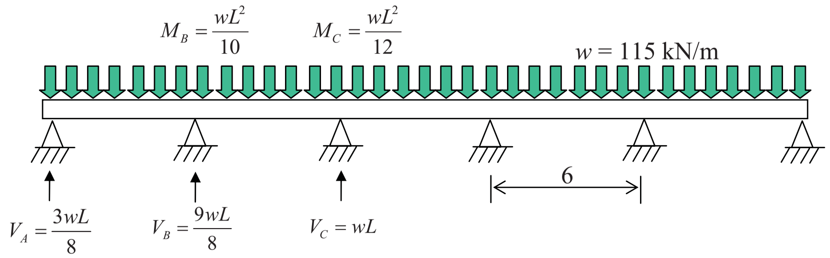
Similarly we can calculate the loads on the other beams.

The load to the N-S beams is appears to be zero as they span in the same direction as the slab. However they do attract load as they must deflect the same as the adjacent slab. So we take maybe 0.5 × loadwidth of a 2-span bay. Hence a 45° load-spread gives a loadwidth of 3 m (for the 6 m column spacing); hence use a 1.5 m loadwidth giving $w = 25$ kN/m.

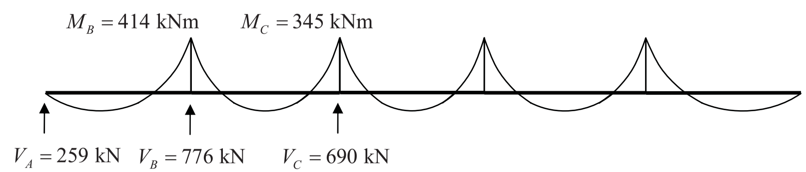


The actual values from a full analysis are 14 / 119 / 47 kN/m. The differences are caused by the uneven spans. Even still though, our values are good enough.

The spans in the spine beam are all even and so the approximate formulae for moments and shears apply:



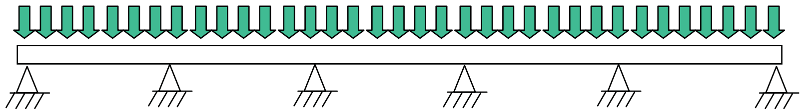
And the numbers are:



If we had carried on the more exact results, allowed for the downstand, and carried out a full 'proper' analysis the moments would be 474 / 354 kNm. Hence our approximate design is still 'ballpark'.

Note that to check the worst shear force we do not choose the highest support reaction value.

Why? Draw the Shear Force Diagram:



The worst shear force is $5wL/8 = 431$ kN.

The span/depth ratio for a flanged continuous beam is 15

$\Rightarrow$ $d = 6000/15 = 400$ mm

$\Rightarrow$ $h = 400 + 20$ (cover) + 12 (shear link) + 25/2 (main bar) = 435 mm

Rounding to nearest 25 mm gives $h = 450$ mm

$\Rightarrow$ $d = 450 - 20 - 12 - 25/2 = 405$ mm

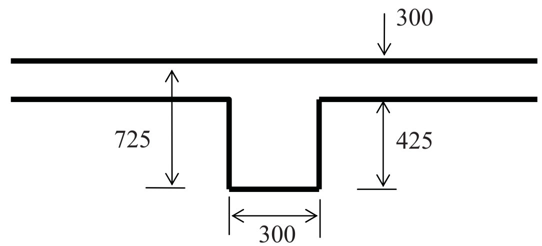
To determine the breadth we will examine the maximum shear stress and limit it to 2.0 N/mm^2:

$$\frac{V}{b_w d} = 2.0 \qquad \Rightarrow \qquad b_w = \frac{V}{2.0d} = \frac{431\times10^3}{2.0\times405} = 532 \text{ mm}$$

This is very wide. We can reduce it by increasing d. A breadth of 300 would fit in nicely with the preliminary column dimension, hence:

$$\frac{V}{b_w d} = 2.0 \qquad \Rightarrow \qquad d = \frac{V}{2.0b_w} = \frac{431\times10^3}{2.0\times300} = 716 \text{ mm}$$

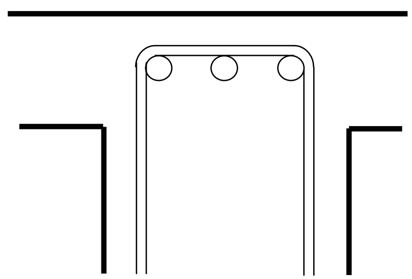
Therefore say $h = 725$ mm and so $d = 725 - 20 - 12 - 25/2 = 681$ mm – detailed design of the shear reinforcement means that the difference in d won't be too important. Also, though this sounds quite deep (and it is), remember that 300 mm of it is in the slab:



This is drawn to scale (more or less!): the important point is that it looks in proportion and this is usually as good a guide as the numbers.
For the main tension steel we use the quick formula:

$$A_s = \frac{M}{300d} = \frac{414\times10^6}{300\times681} = 2026 \text{ mm}^2$$

3 T32 bars provide 2413 mm^2. Choose this to give that extra little bit of 'room' in our design.

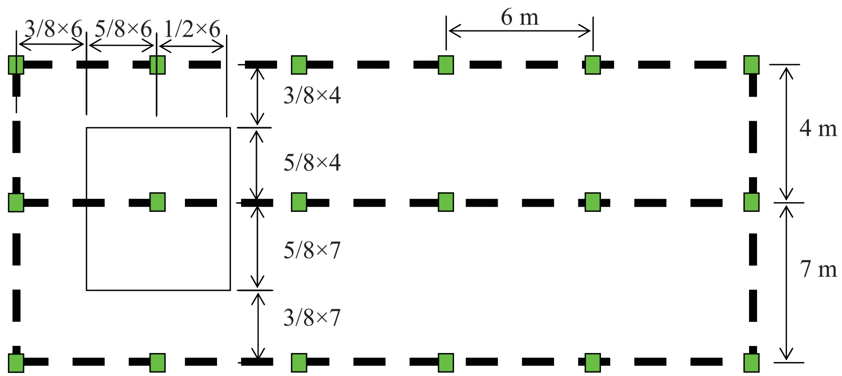


Beam N and Beam S will be the same dimensions as this beam but will have different steel (and probably each have the same) to this beam for ease of construction.

Also note that having the width of the beam the same as that of our column eases the formwork at the beam/column junctions.

Preliminary Column Design

Using the tributary area notion, the load on the column from each floor is:



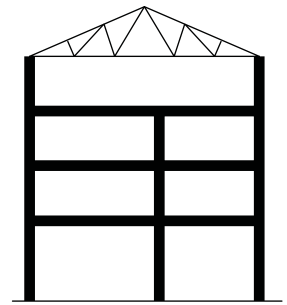
PLAN

Tributary area = (5/8×6 + 6/2)(5/8×7 + 5/8×4) = 46.4 m^2

Load on column from a typical floor = (16.4 kN/m^2)(46.4 m^2)

= 761 kN

For the roof load assume steel roof trusses spanning the full 11 m. Hence, use the same roof loading as per the Load Takedown example:



Hence, roof loading is 2.22 kN/m^2.

However, this only applies to exterior (façade) columns if the trusses span the full width. Hence, total loading on ground floor interior column is:

P_u = 761 × 3.25 = 2473 kN. **Why?**

The façade columns will not be critical in this case (**Why? Check one**).

The roughest design check is:

$$A_{col} = 50P = 50 \times 2473 = 123650 \text{ mm}^2$$

Thus a square column is: $h = \sqrt{123650} = 352 \text{ mm}$, which is significantly greater than the 300 square columns. The next level of detail is:

$$\rho \approx \frac{N/A_{col} - 14}{3}$$

$$\rho \approx \frac{2473 \times 10^3 / 300^2 - 14}{3} = 4.5\%$$

This is still quite high for preliminary design. Try the next level of detail, using 3% steel and a 350 square column:

$$\left[0.35 f_{cu} + \frac{\rho}{100}\left(0.67 f_y - 0.35 f_{cu}\right)\right] A_{col} > P$$

$$\left[0.35 \times 40 + \frac{3}{100}\left(0.67 \times 460 - 0.35 \times 40\right)\right] 350^2 > 2473 \times 10^3$$

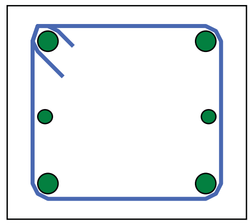
$$[22.8]\, 350^2 > 2473 \times 10^3$$

$$\therefore 2796 > 2473$$

So the columns are acceptable at this level of design. The area of steel required is:

$$A_{sc} = \frac{3}{100} \times 350^2 = 3676 \text{ mm}^2$$

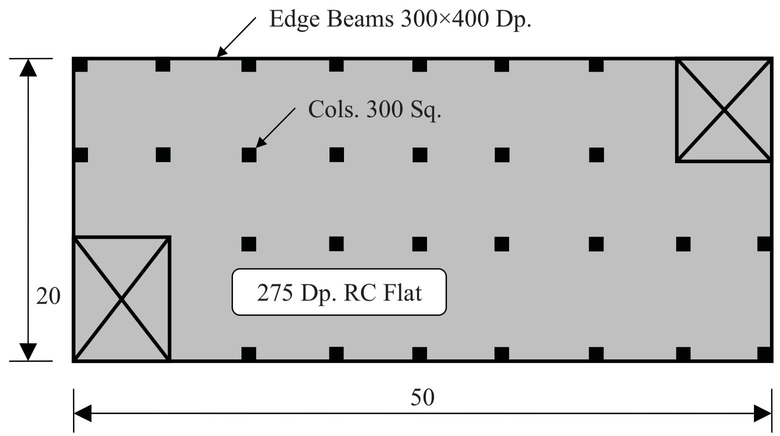
4T32s + 2T25s provides 4199 mm^2 which should be adequate.



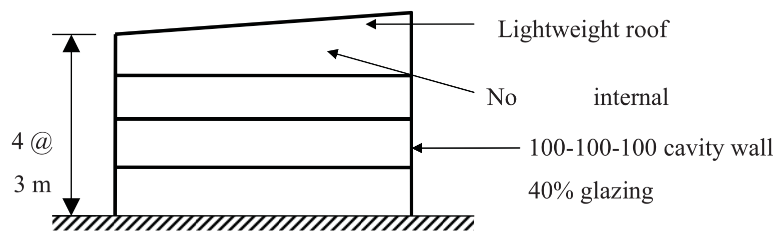
8.3 *Flat Slab Example*

Problem

Using the building example from the load takedown Design Exercise:



Plan



Section

Do the following:

1. Check the slab is adequate, without detailed analysis;
2. Check that punching shear is adequate for the slab and columns shown;
3. Check that the columns as shown are adequate.

In all cases propose appropriate design changes as required.

Solution

Assemble the pertinent information first: from the load takedown solution:

- E-W spacing is: 50/8 = 6.25 m;
- N-S spacing is: 20/3 ≈ 6.7 m.
- $w_{ult} = 17.24$ kN/m^2;
- $P_{ult} = 2770$ kN.

<u>Check the slab is adequate, without detailed analysis</u>

It is adequate to check the span/d ratio as the punching shear requirement will be checked in the second part of the question:

$$d = 275 - 30 \text{ (say)} - 20/2 \text{ (say)} = 237 \text{ mm}$$

$$\frac{l_y}{d} = \frac{6700}{237} = 24.5 \leq 36 \therefore OK$$

As there is lots of 'room' in this aspect of the design, the rebar should be fine, as should the deflection check, which often governs for flat slabs.

<u>Check that punching shear is adequate for the slab and columns shown</u>

- Internal Column:

$$V_t = 6.25 \times 6.7 \times 17.24 = 722 \text{ kN}$$

$$\therefore V_{eff} = 1.15 V_t = 830 \text{ kN}$$

Maximum shear at face of column:

$$u_0 = 2a + 2b = 4 \times 300 = 1200 \text{ mm}$$

$$v_{\max} = \frac{830 \times 10^3}{1200 \times 237} = 2.92 \text{ N/mm}^2$$

$$v_{\max} \leq 0.8\sqrt{40} \text{ or } 5 \text{ N/mm}^2$$
$$\leq 5.06 \text{ or } 5 \leq 5 \therefore OK$$

Shear at critical perimeter, 1.5d from column face:

$$u_{1.5d} = 2a + 2b + 8\mu d = 4 \times 300 + 8 \times 1.5 \times 237 = 4044 \text{ mm}$$

$$v_{1.5d} = \frac{830\times10^3}{4044\times237} = 0.87\text{ N/mm}^2$$

If we conservatively take $v_c = 0.5\text{ N/mm}^2$, then $v_c \le v_{1.5d} \le 2v_c$ and shear reinforcement is to be provided.

Check next perimeter, $1.5d + 0.75d = 2.25d$ from column face:

$$u_{2.25d} = u_{1.5d} + 8\times0.75d = 5466\text{ mm}$$

$$v_{1.5d} = \frac{830\times10^3}{5466\times237} = 0.64\text{ N/mm}^2$$

Again, $v_c \le v_{1.5d} \le 2v_c$ and a second perimeter of shear reinforcement is to be provided. It is not necessary to check the next perimeter as it is clear it will be below the conservative value $v_c = 0.6\text{ N/mm}^2$, or the actual value, which should be around $v_c = 0.65\text{ N/mm}^2$.

<u>Result: expect 2 perimeters of shear reinforcement.</u>

- Perimeter Column:

$$V_{eff} = \frac{722}{2}\times1.4 = 505\text{ kN}$$

Maximum shear at face of column:

$$u_0 = 3\times300 = 900\text{ mm}$$

$$v_{\max} = \frac{505\times10^3}{900\times237} = 2.37\text{ N/mm}^2$$

$$\begin{aligned} v_{\max} &\le 0.8\sqrt{40} \text{ or } 5\text{ N/mm}^2 \\ &\le 5.06 \text{ or } 5 \le 5 \therefore OK \end{aligned}$$

Note that this completely ignores the downstand edge beam – therefore there is much more capacity in this design.

Also, roughly, we can expect 1 or 2 perimeters of shear reinforcement. This is based on a caparison of the previous design with its $v_{\max}$.

<u>Result: expect 2 perimeters of shear reinforcement.</u>

<u>Check that the columns as shown are adequate</u>

Check on minimum column dimension: $h > 3000/17.5 = 171 \therefore OK$

The multiplier for the floor loads immediately above is ignored, and a ‘comfortable’ design is therefore required. The roughest design check is:

$$A_{col} = 50P = 50\times2770 = 138500\text{ mm}^2$$

Thus a square column is: $h = \sqrt{138500} = 372\text{ mm}$, which is significantly greater than the 300 square columns.

Consider the next level of detail:

$$\rho \approx \frac{N/A_{col} - 14}{3}$$

$$\rho \approx \frac{2770\times10^3/300^2 - 14}{3} = 5.6\%$$

This is too close to the maximum permissible, 6%, for preliminary design. Try the next level of detailed calculation:

$$\left[0.35 f_{cu} + \frac{\rho}{100}\left(0.67 f_y - 0.35 f_{cu}\right)\right] A_{col} > P$$

Using the maximum possible percentage of rebar:

$$\left[0.35\times40 + \frac{6}{100}\left(0.67\times460 - 0.35\times40\right)\right] 300^2 > 2770\times10^3$$

$$\therefore 2848 > 2770$$

So again the columns are just acceptable at this level of detailed design. This is too tight for preliminary design, therefore increase column size. Note that this does not adversely affect the punching shear calculations. The shear perimeter is now longer, reducing the shear stresses on all perimeters.

<u>Result: increase column size to 350 square, at least for internal ground floor.</u>

8.4 Sample Scheme Problem 1

Problem

An architect has sent you preliminary sketches of a prestigious 5-storey office building; as shown in Figure Q.2. The atrium, double-height entrance lobby and lightweight roof structure are important elements of the scheme – minimum structural intrusion is expected on these features.

The architect informs you that two WC/fire-escape-staircore blocks are required on each floor, as well as two other fire-escape staircores (as shown in the figure); you are required to integrate these elements into your scheme and to advise the architect as to their location. In doing so, you should pay due regard to lateral stability, any expansion joints and travel distance in deciding your layout.

(a) Propose a structural solution for the building, showing:

i. the provision of lateral stability for the building;
ii. expansion joints, if deemed required;
iii. the layout of the vertical load transfer structure;
iv. the support structure of the proposed glazed roof;
v. the support structure of the double-storey entrance.

(50%)

(b) Assuming a reinforced concrete solution, size the principal members (beam, slab and column) for a typical floor, choosing the most probable critical element in each case. For each element, indicate the approximate areas of reinforcement required.

(50%)

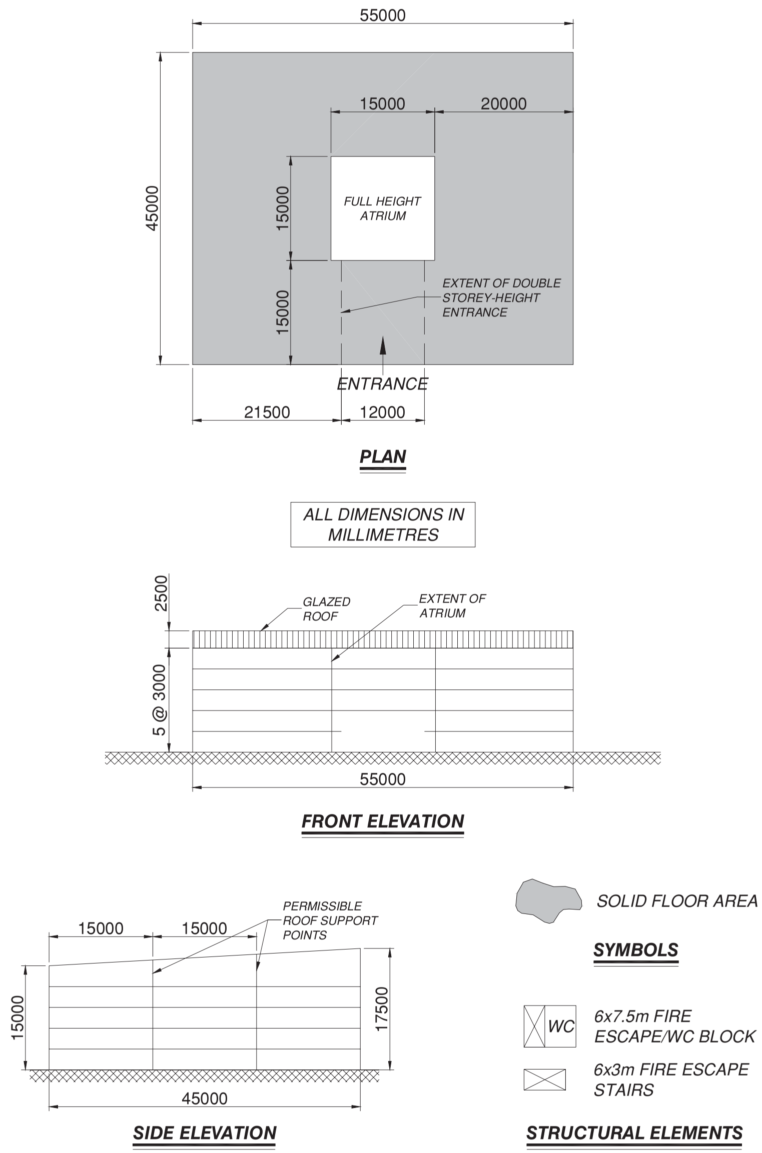


FIGURE Q.2

Solution

Instead of giving answers, this is left to you to try. What is given, however, is the feedback given to a group of students after reading their answers at the problem. They made the mistakes so you don't have to!

General

Overall few got the right balance of text & sketches for Part (a).

Make sure put all of your work in the answer book – even the doodles.

There were a few cases of "magic" numbers.

Lift cores should have been mentioned in the question.

Scheme

Consider expansion joints carefully – make your choice & stick to it.

Having looked at alternative grids, choose one & draw it properly.

There were some very large spans – study the "Economic Span Ranges" carefully.

There was no need for shear walls.

Use rough judgment to size some of Pt. (a) e.g. roof truss member sizes.

Preliminary Design

Errors with basic bending moments, e.g. $wL^2/8$ and PL/4.

Know the design shortcuts, e.g. tributary lengths/areas for beams/columns.

Errors in the use of "quick" formulae (e.g. $wL^2/10$) – make sure you understand where they come from & their limitations.

Do not confuse "critical" & "typical" – the question asked for "typical" beams/cols/slab – not the very difficult unusual ones.

Drawings

Sketches and plans were very poor.

The Plan needs to fill as much of a page as possible & drawn to scale (graph paper).

Draw gridlines on your plan.

Show cols as quick square dots (i.e. with a solid hatching).

Roof trusses were very poor –not drawn to scale properly & wrong configuration

There is not need to "assume sizes" for the plan.

8.5 Sample Scheme Problem 2

Problem

An architect has sent you preliminary sketches of a prestigious 5-storey office building; as shown in Figure Q.2. The atrium, double-height entrance lobby, and glazed elevators are important elements of the scheme – minimum structural intrusion is expected on these features.

The architect informs you that two WC/fire-escape-staircore blocks are required on each floor, as well as two other fire-escape staircores (as shown in the figure); you are required to integrate these elements into your scheme and to advise the architect as to their location. In doing so, you should pay due regard to lateral stability, any expansion joints and travel distance in deciding your layout.

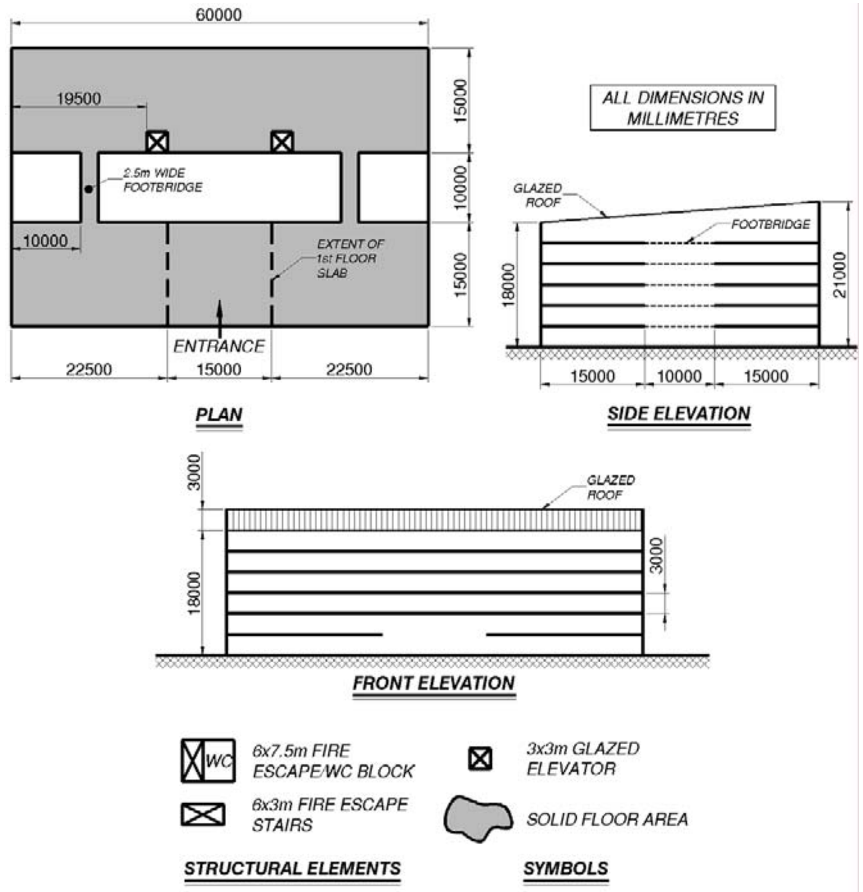
(c) Propose a structural solution for the building, showing:

i. the provision of lateral stability for the building;
ii. expansion joints, if deemed required;
iii. the layout of the vertical load transfer structure;
iv. the support structure of the proposed glazed roof;
v. the footbridge structure.

(50%)

(d) Assuming a reinforced concrete solution, size the principal members (beam, slab and column) for a typical floor, choosing the most probable critical element in each case. For each element, indicate the approximate areas of reinforcement required.

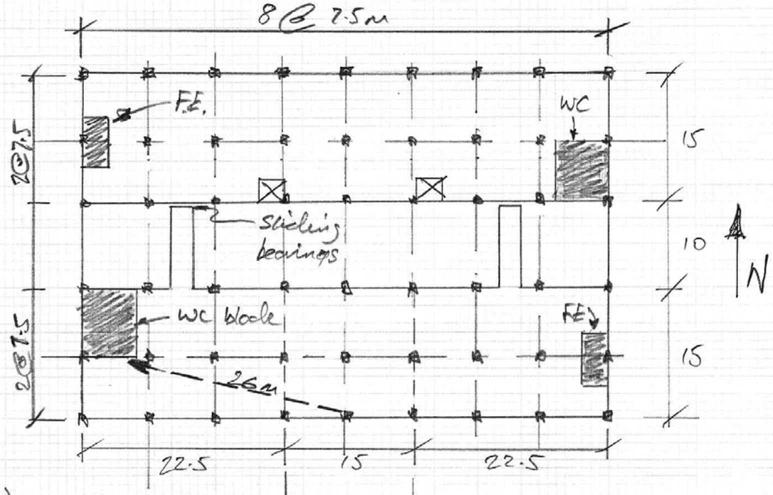
(50%)



Solution

1/

(a)



(i) – Stability provided by the shaded elements
↳ each side of atrium independant
∴ reflect this w/ sliding bearings for any element, (bridge / glazing support) that connect each half.

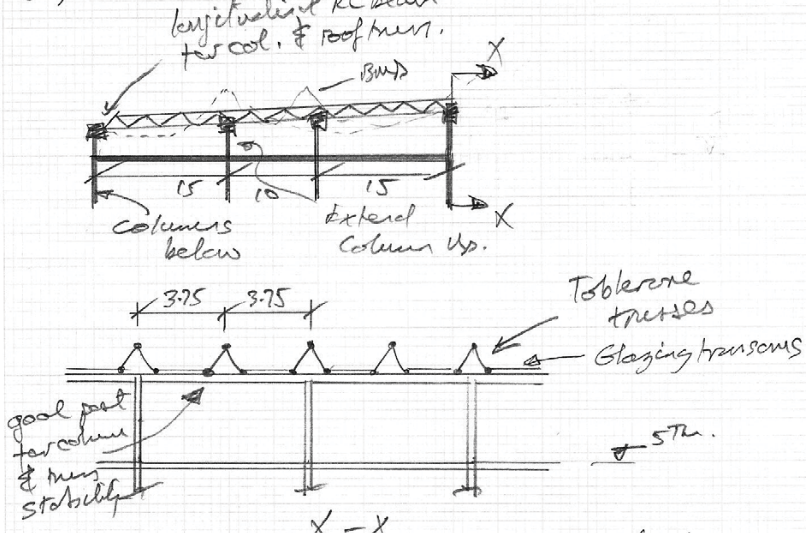
– Furthest travel distance, as shown, ≯ 26m ok.

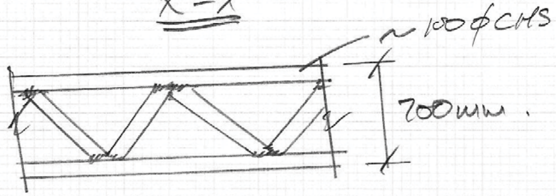
(ii) – No expansion joints running // to North.

(iii) – columns as indicated
↳ assuming an RC solution.
→ choose flat slab, as indicated in Economic span sheet.

2/

(iv)





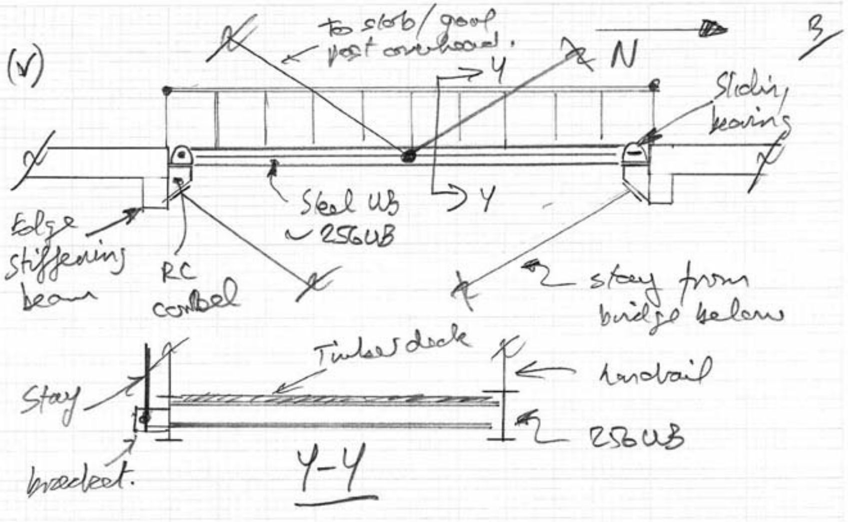
Depth – say span / 15 for single pratt.
– continuous,
∴ reduce "span" by 20% between points of contraflexure
– Toblerone ∴ 2 planes
∴ 2× stiffness
– Small spacing

Depth governed by deflection of glazings

$\therefore$ say $\frac{\text{Span}}{15} \times \frac{4}{5} \times \frac{4}{5}$ say $\sim \frac{\text{span}}{21.6}$

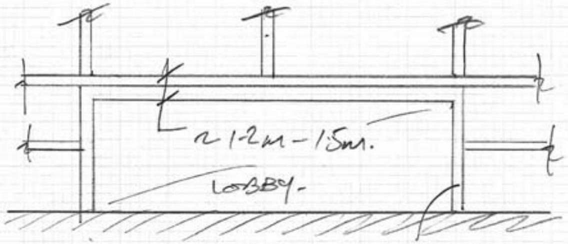
$\therefore$ Depth = 700mm.

(v)



Lobby area:

– Provide 3 No. RC "simply" supported beams to support columns over:



cols not req'd. ~600sq.

4/

(b)

(i) Slab: IStE Green book

∴ Office loading ~ 5.0 kN/m² inc. partitions etc

→ span/d = 36 (Tb 3)

⇒ d = 7500/36 = 208 mm.

+ ϕ/2 + 35 → 2 hr fire rating
↳ say T16

∴ h = 208 + 35 + 8 ~ 250 mm.

Loads: $24 \times 0.25 = G_k = 6\ kN/m^2$

$Q_k = 5$ say

$\therefore w_u = 1.4 \times 6 + 1.6 \times 5 = 16.4\ kN/m^2$

→ say $16.5\ kN/m^2$

Average moment, $M_{ave} = \frac{w l_x l_y}{24}$ (IStE)

$= 16.5 \times 7.5^2 / 24$

$\sim 40\ kNm/m$ width

$\rho \sim \left(\frac{M}{bd^2}\right) / \pi$

$\sim \frac{40 \times 10^6}{1000 \times 207^2} / \pi = 0.3\% > 0.13$ min A_s.

$\Rightarrow A_s \sim \frac{0.3}{100} \times 1000 \times 207 = 621\ mm^2/m$

$\Rightarrow \frac{75\%}{50\%}$ ← col strip $\times 621 \sim 931\ mm^2/m$

⇒ say T16's – 200 (1005)

5/

Shear: Panel size = 7.5^2

load = 16.5

$\pi = 928$ kN

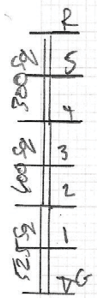
– say 1000kN

- col. min dims

$\hookrightarrow (3000-250)/17.7 = 155$

- Approx area reqd: $A = \frac{1000 \times 10^3 \times (5.5)}{20}$ floors.

$\therefore h = \sqrt{A} = 525$



$\therefore$ say 500 x 500 col. @ ground floor

$\therefore$ ~ about ($2 \times \frac{525}{500} = 2.1\%$ rebar)

Use for 2 storeys, so load is:

$A: \frac{1000 \times 10^3 \times 3.5}{20} \Rightarrow h = \sqrt{A} \sim 400 \times 400$ mm.

Top storey, say 300 x 300.

Thus 300 Sq is critical for slab

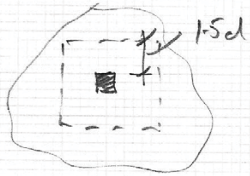
$V = \frac{1250 \times 16.5\,(7.5 \times 7.5)}{(300 \times 4 + 9 \times 250)\,207} = 1.62\ \text{N/mm}^2 \leq 0.6$??

Check $\frac{1250 \times 16.5\,(7.5 \times 7.5)}{(300 \times 4)\,207} = 4.67 < 5$ V ok.

$\rightarrow$ Using 40N concrete.

$l_p = 8(1.5d) + 4(h_c)$

Area $= (l_p/4)^2 = l_p^2/16$



6/

$\therefore v = \frac{16.5 \times (7.5^2 - l_p^2/16) \times 10^3}{l_p d}$

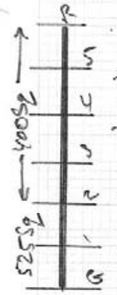
$l_p = 8 \times 1.5 \times 207 + 4 \times 300 = 3684$ mm

$\therefore v = 1.2\ \text{N/mm}^2 > 0.9 \quad \therefore$ change d/h

For a 400 x 400 col, $l_p = 4084$

$\therefore v = 1.08\ \text{N/mm}^2$

$\therefore$ inc. slab depth to $\frac{1.08}{0.9} \times d = 248$



$\therefore$ slab = 300 deep., $d = 300 - 8 - 35 = 257$ mm

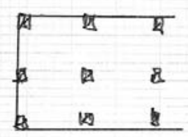
$\therefore G_R = 0.3 \times 24 = 7.2\ \text{kN/m}^2$

$\therefore w_u = 18.1\ \text{kN/m}^2$

$\therefore l_p = 4684$ mm $\therefore v = 0.83\ \text{N/mm}^2 < 0.9$

$\therefore$ Design ok $\rightarrow$ use proprietary shear stud system.

Thus:



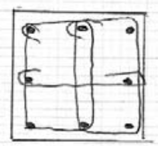
Allowing for edge moments, keep same size

$\therefore G \rightarrow 2 \rightarrow 525$ Sq.

$2 \rightarrow 5 \rightarrow 400$ Sq.

$A_s \sim \frac{2.1}{100} \times 400^2 = 3360\ \text{mm}^2$

$\therefore$ say 8T25 (3927)



For 525:

$A_s \sim \frac{2.1}{100} \times 525^2 \sim 5788\ \text{mm}^2$

$\Rightarrow$ 8T32 (6434)

8.6 *Sample Scheme Problem 3*

Problem

An architect has sent you preliminary sketches of a 4-storey apartment building over an open-air car park; as shown in Figure Q.2. You are required to advise the architect whether or not expansion joints are needed. In addition you need to assess whether lateral stability is achieved with the proposed layout.

(a) Propose a structural solution for the building, showing:

i. the provision of lateral stability for the building;

ii. expansion joints, if deemed required;

iii. the layout of the vertical load transfer structure;

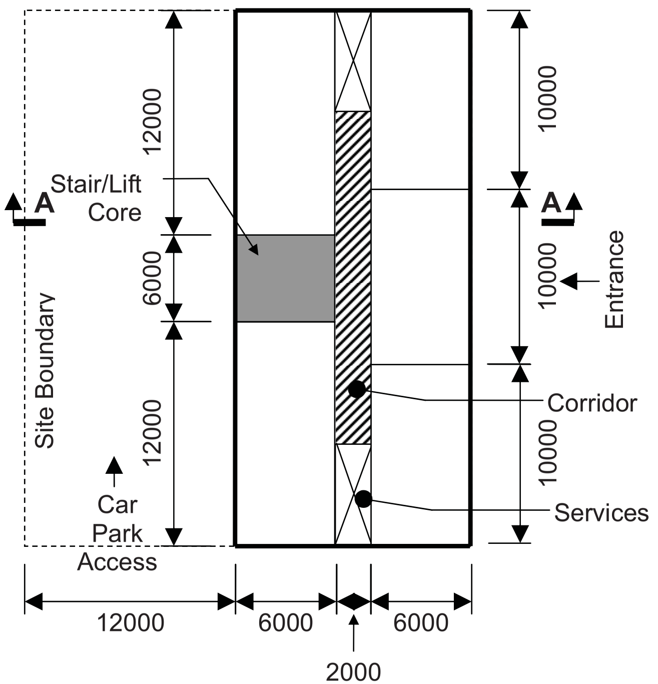
iv. the car park layout;

v. the integration of the car park layout and the vertical load transfer structure.

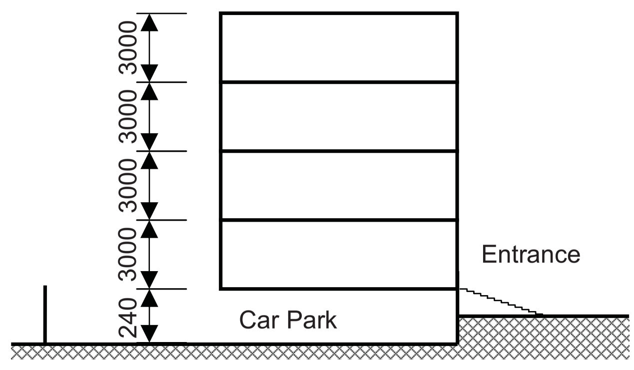
(50%)

(b) Assuming a reinforced and/or precast concrete solution, size the principal members for a typical floor, and a column at car park level – choosing the critical element in each case. For each element, indicate the approximate areas of reinforcement required.

(50%)



PLAN
(All dimensions in mm)



SECTION A-A
(All dimensions in mm)

Solution

As before, you just get the feedback.

General

Overall few got the right balance of text & sketches for Part (a).

Those who used graph paper seemed to provide better sketches.

The term "transfer beam" has a particular meaning.

Scheme

Shear wall overkill – only a few thought of the use of the building.

No need to "assume" preliminary sizes – fill in after prelim design.

Do the service cores go into the car park?

The car park layout should have been first – the "top down approach".

Parking dimensions not adhered to – narrow driving aisles with columns in them!

Drainage info *usually* not needed in a structural scheme design.

Preliminary Design

The numbers generally seemed good.

Inappropriate formulae used: e.g. $wL^2/10$ for a simply supported span!

Rebar for slabs is specified at a spacing, not a number of bars, e.g. T16-200 not 4T16.

Drawings

Sketches and plans were very poor.

Every line has a meaning: not enough information on the sketches.

The Plan needs to fill as much of a page as possible & drawn to scale (graph paper).

Draw gridlines on your plan.

Show cols as quick square dots (i.e. with a solid hatching).

Isometric drawings are not needed.

8.7 Sample Scheme Problem 4

Problem

An architect has sent you preliminary sketches of a 4-storey (3.5 m floor to floor) development as shown in Figure Q.2. The client wants flexibility in the use of the building:

- Initially the building is to be used as apartments with the layout shown in Figure Q.2. Sound and fire isolation of dwellings is important. The building will be masonry clad with 25% glazing.
- The use may change to open-plan offices with fully glazed elevations.

The client understands that there are cost implications for this and that works would be necessary to change the use. Also, the Architect understands that some structural elements may be necessary in the larger apartments.

Part (a)

Propose a structural scheme for the building, giving sufficient information on:

1. The provision of lateral stability when the building is to be used as apartments, taking any expansion joints into account, if deemed necessary.
2. The layout of the vertical load transfer structure; the floor plate; beams, and; structural walls, as applicable to your scheme. This should reflect the possible change of use.
3. The works necessary to achieve lateral stability if in the future the use is to change to open-plan offices.

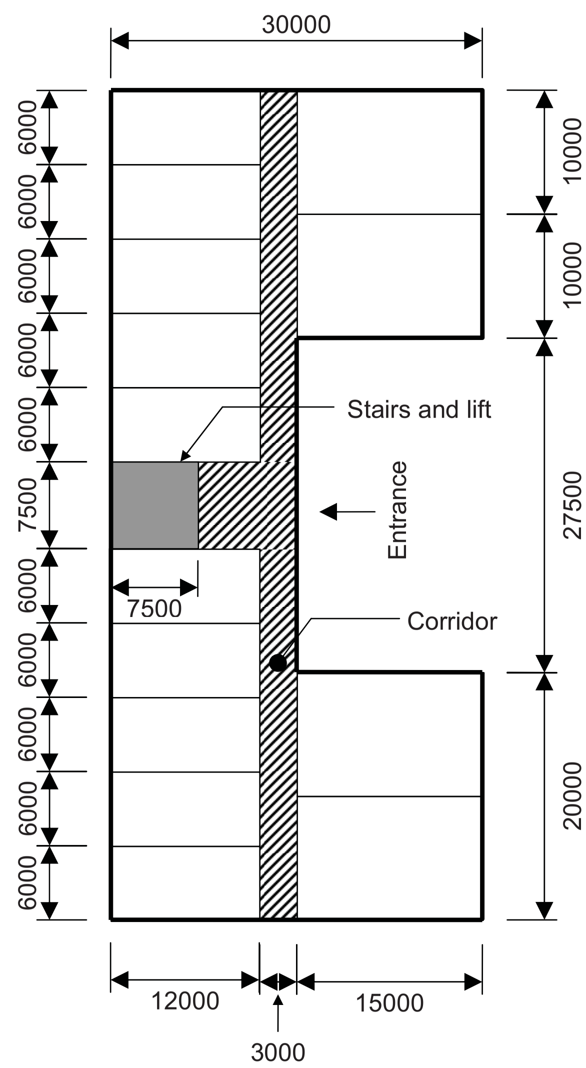
(50%)

Part (b)

Important: In Part (b), only consider the loading appropriate when the building is used as apartments.

Assuming a reinforced and/or precast concrete solution, size the principal members (beam, floor slab, and column) for a typical floor, choosing the critical element in each case. For each element, indicate the approximate areas of reinforcement required.

(50%)

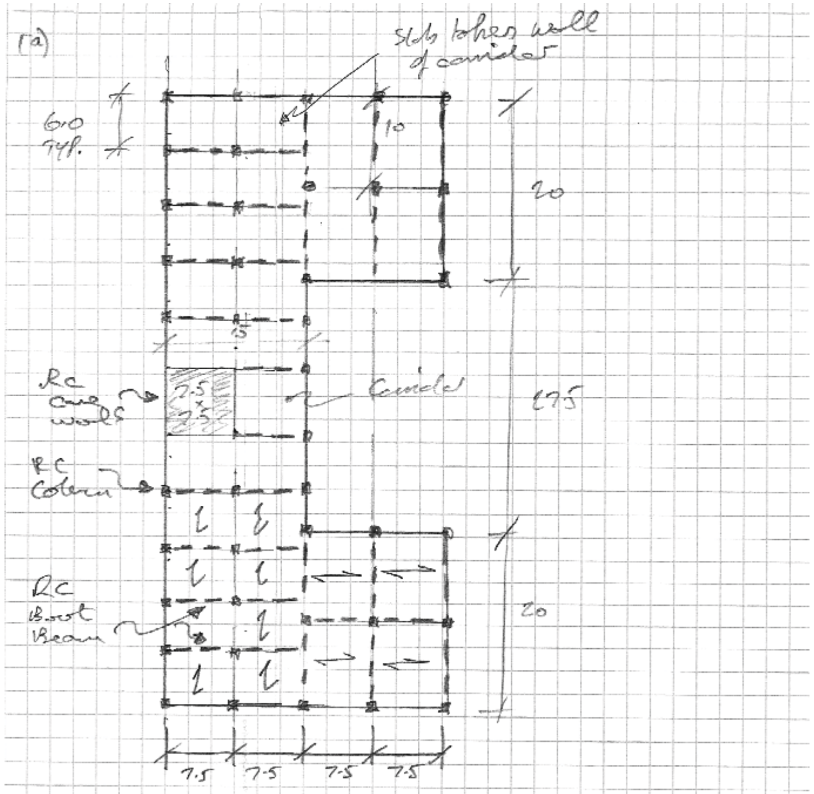


PLAN
(All dimension in mm)

FIGURE Q.2

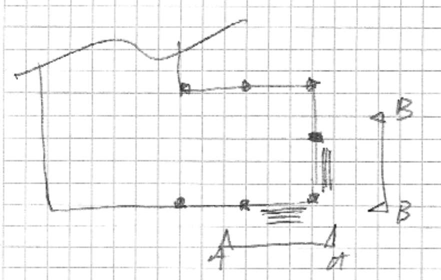
Solution

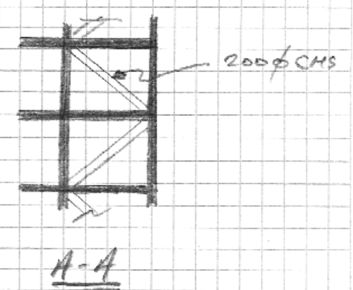
(a)



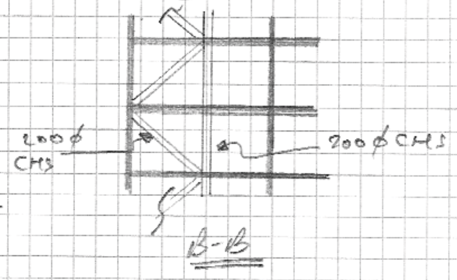
- Use an RC frame w/ masonry party walls
- Provide edge beams to take masonry facade
- Provide internal beams to take the masonry wall loads @ each floor.
- Use 250 Dp. precast units to span the 7.5m largest span
- No expansion joint; lateral stability provided by core & walls.

For change of use, introduce steel bracing into the corners:





Note.
Shear walls are not permissible w/ the fully glazed facade.



(b)

250 Dp Composite slab

$\hookrightarrow G_k = 3.8\ kN/m^2$
Service load $= 9.7 kN/m^2$
$M_u = 188 kNm$

} Bretan catalogue.

$Q_k = 1.5 + 1.0$ for partitions $= 2.5 kN/m^2$
+ ceiling & serv $= \underline{\underline{3.0 kN/m^2}}$

This should allow the design to be used for light offices also.

Slab

Max span = 7.5m

$G_k = 3.8 + 24 \times 0.075 = 5.6\ kN/m^2$ ← screed

$Q_k = 3.0$

$\therefore w_u = 12.64\ kN/m^2$

$\therefore$ 1.2m width; $\Rightarrow 1.2 \times 12.64 = 15.2\ kN/m$

$M_{max} = 15.2 \times 7.5^2/8 = 107\ kNm$

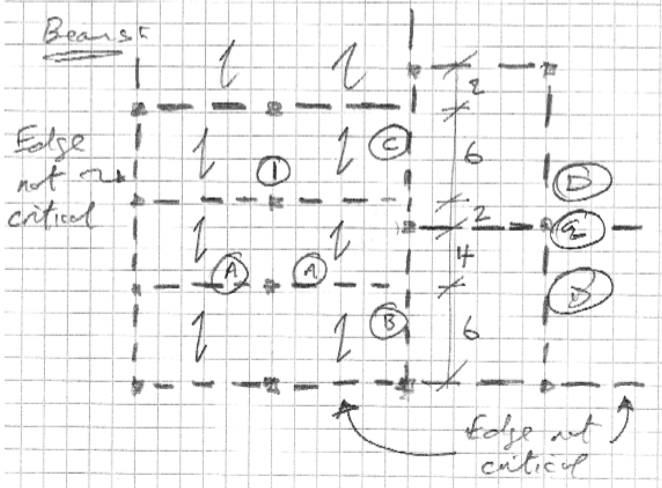
$< 188 \quad \therefore$ OK.

Party Walls

$w_g = 4.54 \times (3.5 - 0.325) = 14.4\ kN/m$ ← 215 solid ← slab

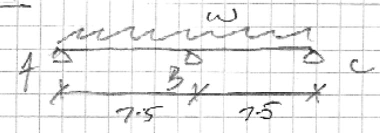
Beams

Edge not critical



Edge not critical

Beam A

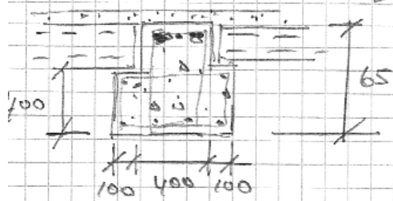


$w = 6 \times 12.64 + 1.4 \times 14.4 = 96\ kN/m$

$M_{max} = wl^2/8 = 96 \times 7.5^2/8 = 675\ kNm$

$s/d = 12$ say $\quad \therefore d = \frac{7500}{12} = 625$

$\therefore$ say $h = 650$; $d = 610\ mm$



$V_{max} = \frac{5}{8} wl$

$= 450\ kN$

$b = \frac{450 \times 10^3}{2 \times 610} = 369$

say $b = 400\ mm$

$V_c = \frac{3}{8} \times wl = \underline{270\ kN}$

$V_B = 2 \times V_{max} = 900\ kN$ to column ①

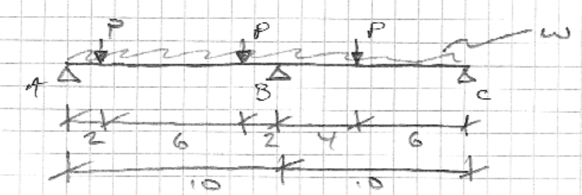
$\rho_s = \left(M/bd^2\right)/\pi = \frac{675 \times 10^6}{400 \times 610^2 \times \pi} = 1.44$

$\therefore A_s = \frac{1.44}{100} \times bd = 3522\ mm^2$

$\therefore$ Say 5T32 (4021 mm²)

Note: This is a singly-reinforced solution.

Beam B & C



$P = 270\text{kN}$ from Beam A

$w = \left(\frac{7.5}{2} + 1\right) 12.64 = 60\text{kN/m}$

Using Trib Areas,

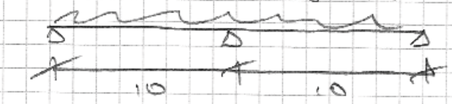
$V_c = 12.64\,(7.5 \times 10/2) = 474\text{kN}$

$\therefore M_B = -270 \times 4 - 60 \times \frac{10^2}{2} + 474 \times 10 = 660\text{kNm}$

$\therefore$ Use design for beam A

Beam D

$w = 7.5 \times 12.64 = 95\text{kN/m}$



$M_{max} = 95 \times 10^2/8 = 1187.5\text{ kNm}$.

$\therefore \rho_s = \left(M/bd^2\right)/\pi = \frac{1187.5 \times 10^6}{400 \times 610^2 \pi} = 2.54$

$\therefore A_s = \frac{2.54}{100} \times 400 \times 610 = 6200\text{mm}^2$

i.e. ~ 8T32 if singly reinforced.

$\therefore$ Say 850 Beam & make B & C ~~the~~ same.

$\therefore$ Will work as doubly reinforced,

Column ①

$P = 900\,(3 + 0.75 + 0.25) = 3600\text{kN}$

floors, roof, for wt.

$\therefore A_{col} = 50 \times 3600 \Rightarrow 424\text{ sq}$

$\therefore$ say 450 sq col.

$\rho_s = \frac{N/A_c - 10}{3} = \left(\frac{3600 \times 10^3}{450^2} - 10\right)/3 = 2.6\%$

$\therefore A_s = \frac{2.6}{100} \times 450^2 = 5250\text{mm}^2$

$\therefore$ Say 6T32 (4825) + 2T20 (628)

Column ②

$P = 12.64 \times (7.5 \times 10) + 7.5 \times 14.4 \times 1.4 = 1100\text{kN}$

$\therefore P_u = 4 \times 1100 = 4400\text{kN}$

$\therefore A_c =$ say 450 sq

$\therefore \rho_s = \left\{\left(\frac{4400 \times 10^3}{450^2}\right) - 10\right\}/3 = 3.9\%$

$\therefore A_s = \frac{3.9}{100} \times 450^2 = 7920\text{mm}^2$

$\therefore$ Say 10T32 (8042)

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