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DESIGN TIPS

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Contents

HOUSE RULES

Some general guidelines for you're the first years of engineering:

1. Always try to think of answer before asking a question. Getting the short quick answer without the research/reading you mean miss all the "grey" areas (and avoids you getting BS). Hints: Look at past jobs, take a look in the library for book on the subject.
2. Never be afraid to ask a question.
3. Always ask a question with a note pad or similar such that you can take notes or the senior engineer can sketch details for you.
4. Try to collect questions and do a few at once. Generally good times to ask questions are first thing in the morning, just before/after lunch.
5. If you are unable to think of how to start a problem, start with sketch.
6. Expect your first few budgets to be "blowouts", we do. The budgets are quoted on engineering with experience, not a graduate. My first budget went 5 times over, but the next similar project it was two times. Learning is what is expected of you.
7. Expect to be put pushed to your limit. We will want to know what this is such that we can slot you into your best position.
8. You will be asked question by clients and subcontractors, if the question is complex or about times, record the question and tell them you will get back to them after discussing this with the certifying engineer.
9. Always act professional in front of clients, even if they are your best friend.
10. Record all conversions with client and subcontractors, in written form.
11. If in trouble or struggling, put your hand up, we can't read your mind.
12. Read the contents page of each standard; while we don't expect you to know the standards like the back of your hand, we do expect you to know what's in them. In the appendix is a current set of standards that you will want to be familiar with.

MEETING GUIDELINES

- Thoroughly prepare before every meeting; studying during meetings does not work.
- Treat everyone in the room as your equal; egos are always detrimental to progress.
- Keep your focus on the big picture, not on the details; set policy, not implementation.
- Focus on the core issues of the matter at hand, leaving peripheral issues for others.
- Practice intensive listening; nobody has ever learned anything while talking.
- Organize your thoughts before speaking, stay on topic, and be brief.
- Speak selectively and infrequently; talk only when it is an improvement on silence.
- Disagree without being disagreeable; visible emotion is never appropriate.
- Be willing to compromise when necessary; partial progress is better than no progress.
- Selectively inject humour to alleviate tension.

BOOKS OF INTREST

Some of them are from other countries, remember engineering is a global profession and other countries may be leading the way in certain areas. For example, the DNV-Riso (Denmark) would be considered one of leading authority for wind turbines.

Concrete:
"Concrete structures" Warner et al
"Design of Prestressed Concrete" Gilbert
"Reinforced concrete mechanics" MacGregor,
"PCI design Handbook" PCI,
"Precast concrete Handbook" NPCAA,
"Reinforced concrete elements" Beletich,
"Reinforcement detailing handbook" CIA,
"Standard method of detailing" Istruct,
"Design of tilt-up concrete wall panels" CIA,
"Reinforced concrete"- economic guide R/C, P/T ect.
"Structural engineering pocket book"- Fiona Cobb

Steel:
"Steel designer manual" The steel construction institute,
"Design of Portal Frame Buildings" ASI
"Rigid and simple connection series" ASI
"Design Guide - Portal Frames Steel Sheds and Garages" ASI
"Economical Structural Steelwork" & ASI Journal
"Structural steel semi-rigid connections" Faella,
"semi-rigid" base plate design notes, AISC design guides.

Foundation: "PRINCIPLES OF FOUNDATION ENGINEERING" Das

Materials: Roarke

Finite element: "finite element design of concrete structures" Rombach

Costing: "commercial and industrial building cost guide" Cordell

Vibration: "Design guide 14" AISC, HIVOSS for bridges

Wind: "wind loading of structures" John Holmes

WHAT IS AN ENGINEER

Ever wondered what you got yourself into well here is a brief over view:

According to USA President Herbert Hoover, who was an engineer before he became a politician, said: The great liability of the engineer …compared to men of other professions……is that his works are out in the open where all can see them His acts …..step by step …are in hard substances. He cannot bury his mistakes in the grave like the DOCTORS. He cannot argue them into thin air…..or blame the judge…..like the LAWYERS. He cannot, like the ARCHITECT, cover his figures with trees and vines. He cannot, like the politicians, screen his shortcomings by blaming his opponents….and hope the people will forget. The ENGINEER simply cannot deny he did it. If his works do not work……he is damned.

A design engineer's responsibility should include:

- The application of specialized civil engineering knowledge, training, and experience to evaluate, analyze, design, specify, detail, and observe the construction of force-resisting elements of structures. Such expertise includes consideration of strength, stability, deflection, stiffness, ductility, potential modes of failure, and other characteristics that affect the behavior of a structure.
- Assuring the structural safety of the design, details, checking shop drawing.
- Detailing is as important as design since proper detailing of engineering designs is an essential link in the planning and engineering process as some of the most devastating collapses in history have been caused by defective connections or DETAILING.
- Detailing is necessary not only for the steel structures but also for the RCC members as it is the translation of all the mathematical expressions and equation's results.
- An engineer must account for temperature changes, weather, and many other factors during design, and choose materials that can withstand such elements. He or she must create a structure with just enough deflection and sway to account for natural shifts and expansion without creating danger or discomfort for occupants. Finally, he or she must complete the design and specify materials that fit within the project budget.

ENGINEERING QUOTES

These quotes are provided to inspire you, give you confidence and show you that engineering has no black and white, just shades of grey.

I. Most of serious design stuff ups or budget blow outs have begun with the saying "This is straight forward, no need to think about it, just get to work." sdz (Structural) Jan 09
II. The first 90% takes 10% of the time, the last 10% takes the other 90% of the time TheBlacksmith Aug 02
III. There is more than 1 way to skin a cat. (applies to all situations)
IV. The client will always try to alter your design during construction.
V. "It's not wrong until it can't be made right" ie: many mistakes can be fixed before it's too late. Example changing rebar before placing concrete. Clevegar Dec 02
VI. "You can't push a rope," unless L<3xDia. daniel2 Oct 02 & curvbridger Dec 02
VII. "20% of your time will be spent on the design concept and general arrangement; 80% will be spent on details". krd Jan 03
VIII. "If you don't have time to do it right, when are you going to find time to do it over?" Wrightguy Jan 03
IX. An extra dollar spent on "design" will save ten dollars during construction. whyun Jan 03
X. In all detailing use good line contrast – lighter lines for dimension lines and bolder lines for the object lines.
XI. The elephant in the room is your reputation precedes you.
XII. "a safe structure will be the one whose weakest link is never overloaded by the greatest force to which the structure is subjected" Petroski 1992
XIII. "While computers are indispensable tools, they will never replace the judgment of experienced engineers who have mastered the art of structural engineering." Clifford Schwinger
XIV. "It's no trick to get the answers when you have all the data. The trick is to get the answers when you only have half the data and half that is wrong and you don't know which half." William Thomson,

XV. "to some extent the choice of level of detail in any part of an engineering procedure must to some extent be governed by the crudest part of that procedure"

XVI. "All models are wrong, some are useful." A corollary to this statement is that all models are wrong, some are just more subtly wrong. Elms 1985

XVII. "A reasonable probability is the only certainty." E. W. Howe,

XVIII. "The person who insists on seeing with perfect clearness before deciding, never decides." Henri-Frederic Amiel

XIX. "The only thing that is certain is that nothing is certain." Pliny the Elder,

XX. The square-cubed law. "When an object is scaled up by a certain factor, the volume increases with the cube of the factor, while the cross section and surface areas increase with the square of the factor".

XXI. THE PURPOSE OF THE CODE *"Well, the main function of the Code is to keep people out of trouble, to make structures safe, to make it difficult for somebody to design an unsafe structure."* Chester P. Siess

XXII. REORGANIZATION SEEDS *"Because supporting sciences do not always provide crisp answers to engineering problems, building requires the use of judgment. Judgment has soft boundaries and is influenced strongly by what is considered to be acceptable risk."* Mete Sozen (2006)

XXIII. *"The major part of the college training of civil engineers consists in the absorption of the laws and rules which apply to relatively simple and well-defined materials, such as steel or concrete. This type of education breeds the illusion that everything connected with engineering should and can be computed on the basis of a priori assumptions. As a consequence, engineers imagined that the future science of foundations would consist in carrying out the following program: Drill a hole into the ground. Send the soil samples obtained from the hole through a laboratory with standardized apparatus served by conscientious human automatons. Collect the figures, introduce them into the equations, and compute the result. Since the thinking was already done by the man who derived the equation, the brains are merely required to secure the contract and to invest the money. The last remnants of this period of unwarranted optimism are still found in attempts to prescribe simple formulas for computing the settlement of buildings or of the safety factor of dams against piping. No such formulas can possibly be obtained except by ignoring a considerable number of vital factors."* Karl Terzaghi

XXIV. I don't know much about statistics, but I do know that if something has a 50-50 chance of going wrong, 9 times out of 10 it will. Author unknown

XXV. "There is a difference between knowing you have this up your sleeve and using it in your calculations." Csd72 2009

XXVI. A common mistake that people make when trying to design something completely foolproof is to underestimate the ingenuity of complete fools - *Douglas Adams*

XXVII. Engineering problems are under-defined, there are many solutions, good, bad and indifferent. The art is to arrive at a good solution. This is a creative activity, involving imagination, intuition and deliberate choice - *Ove Arup*

XXVIII. He ... insists that no mathematical formula, however exact it may appear to be, can be of greater accuracy than the assumptions on which it is based, and he draws the conclusion that experience still remains the great teacher and final judge.- *James Kip Finch*

XXIX. How could you do anything so vicious? It was easy my dear, don't forget I spent two years as a building contractor. - *Priscilla Presley & Ricardo Montalban*

XXX. When engineers and quantity surveyors discuss aesthetics and architects study what cranes do we are on the right road.- *Ove Arup*

XXXI. Architecture begins where engineering ends. Walter Gropius

XXXII. *There are always many possible solutions, the search is for the best – but there is no best – just more or less good"*– **Ove Arup**

XXXIII. *"If the structural shape does not correspond to the materials of which it is made there can be no aesthetic satisfaction"* – **Eduardo Torroja**

XXXIV. Hardy Cross once wrote: Strength is essential but otherwise not important.

CONCEPT DESIGN

Decisions taken at concept/Preliminary design stage will influence the extent to which the actual structure approximates to the ideal, but so will decisions taken at detailed design stage. Consideration of each of the ideal characteristics in turn will give some indication of the importance of preliminary building design. Look at the construction process in its entirety, including the contractual arrangements, the procurement route, and the level of rationalisation.

a. Safety.
   The ideal structure must not collapse in use. It must be capable of carrying the loading required of it with the appropriate factor of safety. This is more significant at detailed design stage as generally any sort of preliminary design can be made safe. Pay particular attention to fire requirements however.
b. Serviceability.
   The ideal structure must not suffer from local deterioration/failure, from excessive deflection or vibration. Detailed design cannot correct faults induced by bad preliminary design.
c. Economy.
   The structure must make minimal demands on labour and capital; it must cost as little as possible to build and maintain. At preliminary design stage it means choosing the right types of material for the major elements of the structure, and arranging these in the right form.
d. Appearance.
   The structure must be pleasing to look at. Decisions about form and materials are made at preliminary design stage; the sizes of individual members are finalised at detailed design stage.

Things that are discussed and attended to during the concept design stage:

- *Type of construction*— reinforced concrete, precast concrete, reinforced masonry, structural steel, cold-formed steel, wood, etc.
- *Column locations*—A uniform grid facilitates repetitive member sizes, reducing the cost and increasing the speed of construction. Bay dimensions may also be optimized to minimize material quantities while efficiently accommodating specific space requirements, such as parking garages and partition layouts.
- *Bracing or shear wall locations*—Horizontal forces due to wind, earthquakes, etc. must be transferred down from the superstructure to the foundations. The most efficient means of accomplishing this is usually to provide vertical bracing or shear walls oriented in each principle direction, which must be coordinated with functional and aesthetic requirements for partitions, doors, and windows.
- *Floor and roof penetrations*—Special framing is often required to accommodate stairs, elevators, mechanical chases, exhaust fans, and other openings.
- *Floor-to-floor heights*—Adequate space must be provided for not only the structure itself, but also raised floors, suspended ceilings, ductwork, piping, lights, and cable runs for power, communications, computer networks, etc. This may affect the type of floor system (reinforced concrete beams, joists, or flat plates; structural steel beams or open web steel joists; cold-formed steel or wood joists or trusses) that is selected.
- *Exterior cladding*—The building envelope not only defines the appearance of the facility, but also serves as the barrier between the inside and outside worlds. It must be able to resist wind and other weather effects while permitting people, light, and air to pass through openings such as doors, windows, and louvers.
- *Equipment and utility arrangements*—Large equipment (air handling units, condensers, chillers, boilers, transformers, switchgear, etc.) and suspended utilities (ductwork, piping, light fixtures, conduits, cable trays, etc.) require adequate support, especially in areas subject to seismic activity that can induce significant horizontal forces.
- *Modifications to existing buildings*—Changing the type of roof or roofing material, adding new equipment, and removing load-bearing walls are common examples of renovation measures that require structural input.

PRELIMINARY DESIGN

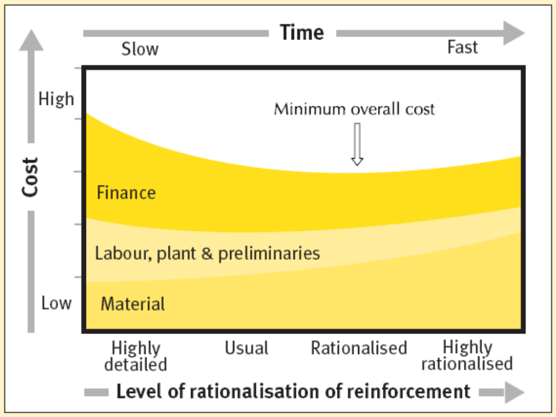


Figure 3: The diagrammatic relationship between rationalisation of reinforcement, time and minimum overall cost.

Study of Architectural Drawings

As the building is to be constructed as per the drawings prepared by the Architect, it necessary for the Designer to correctly visualize the structural arrangement as proposed by the Architect. A design engineer, after studying Architect's plans, can suggest necessary change like additions/deletions and orientations of columns and beams as required from structural point of view.

For this, the designer should have a complete set of prints of original approved architectural drawings of the buildings namely i) Plans at all the floor levels, ii) Elevations, (front, back and sides), iii) Salient cross sections where change in elevation occurs and any other sections that will aid to visualize the structure more easily .The cross sections should show the internal details like locations of windows, doors. Toilets staircases, lift machine room, staircase rooms, and any other special features like gutter at roof level, projections proposed to give special elevation treatment, etc. Always work with drawings of the same scale.

During the study following points should be noted. The drawings should be examined to find out,

- Whether the plan shows all the required dimensions and levels so that the designer can arrive at the lengths and sizes of different members .Wherever necessary, obligatory member size as required by Architect (on architectural grounds) are given or otherwise .
- Whether the plans and schedules of doors and windows etc. are supplied so as to enable designer to decide beam size at these locations.
- Whether thickness of various walls and their height (in case of partition walls) is given.
- Whether functional requirements and utility of various spaces are specified in the plans. These details will help in deciding the imposed load on these spaces.
- Whether material/ratings for walls are specified.
- Note the false ceiling, lighting arrangement, lift/s along with their individual carrying capacity (either passenger or goods), Air Conditioning ducting, acoustical treatment ,R.C.C. cladding, finishing items, fixtures, service/s' opening proposed by the Architect .
- Note the position/s of expansion joints, future expansion (horizontal and/ vertical) contemplated in the Architect's plan and check up with the present scope of work (indicated in the "Field Data" submitted by the field engineers).The design of the present phase will account for future expansion provision such as loads to be considered for column and footing design (combined /expansion joint footing) resulting if any .
- Whether equipment layout has been given, particularly in the areas where heavy machinery is proposed to be located.

- Special features like sun breakers ,fins, built- in cupboards with their sections so as to enable designer to take their proper cognizance
- Whether the location/s of the over head water tanks specified by the Architect and whether "Field Data" submitted by field engineer furnishes the required capacity of each over head water tank
- What type of water proofing treatment is proposed?
- Cranes?
- Forklift? Size, use ect
- What is the end users intent for the structure?
- Partitions deflections windowns

Stormwater and sewer lines along with a variety of other services such as electrical conduits lift pits and even air conditioning ducts will need to coordinate with our foundations. The optimum structural solution may need to be modified to suit these competing constraints.

Any discrepancy from above scrutiny should be brought to the notice of the Architect in an RFI, these matters should be sorted out before proceeding with any design. Try to get as many RFI's as possible in your first RFI, this will reduce the washing machine effect.

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/reo joints for Glulam;
- 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.

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. It should be noted that only one or two structural materials tend to be used in any given construction project. This is to **minimise the diversity of skills required** in the workforce.

Strength

It should 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 overreinforced 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.

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

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.

Consider needs to be given to the coordination of mechanical, electrical, plumbing, egress, architectural, civil, landscaping, fire-protection, security and more. You need to account for others disciplines requirements of your structure coordination area's are: plumbing and process piping engineering disciplines, including but not limited to various water, waste and drainage systems, process and fuel gasses, medical gasses, vacuum services, special process fluids, as well as associated fixtures, equipment, controls and appurtenances. Service cores should be of a size sufficient size and in vertical alignment.

Examples are

• Beam penetrations are provided for duct work and piping;

• Slab edges are designed and detailed to accept the fascia;

• Floor openings are coordinated with the stairs and elevators;

• Openings are provided for mechanical shafts; and

• Floor to floor height is developed considering building usage, utility and ceiling requirements.

• Roof geometry must suite the projected usage of the facility; considering such constraints as utilities, security, piping and suspended loading.

• Depth of roof must accommodate suspended HVAC units and other process related equipment.

• Maximum shipping depth varies based on shop location and site location, local ordinance, over-the-road clearances, trucking availability, shop capacity or size restrictions.

• Maximum shipping length varies based on trucking availability, local ordinance, shop crane capacity, shop size restrictions, site laydown area, installation crane capacity, and handling and lateral stability requirements.

• Maximum weight of shipping piece varies based on trucking availability, local ordinance, shop crane capacity and installation crane capacity.

• Bracing geometry should suite the usage of the facility, considering openings, and other penetrations and circulation requirements in the final facility.

• Elevations must be coordinated with the final usage of the facility or finished elevation requirements.

• Shoring requirements, special erection needs, design assumptions very helpful additions to the design documents.

• The lateral stability of the structure is a function of the initial design assumptions, the erection sequence and the erector- installed temporary bracing. Regardless of the nature of the structure, the erector is responsible for the lateral stability as it is installed. The erector's temporary bracing must therefore sustain the forces imposed on the structure during the installation process. For the erector to accomplish this, the documentation should identify the lateral-load-resisting system and connecting diaphragm elements that provide for lateral strength and stability in the completed structure; and any special erection conditions or other considerations that are required by the design concept, such as the use of shores, jacks or loads that must be adjusted as erection progresses to set or maintain camber, position within specified tolerances or prestress

Concrete

Design of concrete framed facilities also requires a similar under- standing of the construction process and coordination. Variables such as those listed below are examples of such considerations:

• Shoring and re-shoring requirements.

• Loading and support of concrete.

• Form deflection limits.

• Concrete finish requirements.

• Joint location and details in slabs on grade and walls.

• Precast shipping restrictions or trucking availability.

• Cold or hot weather concreting procedures noted within the design documents.

• Layout of column anchor bolts including the foundation rein-forcing provides the basis for accurate initial construction.

Simple considerations such as the selection of wall thicknesses must be done in concert with structural engineering needs as well as accommodating mechanical and electrical elements within the walls. Allowances for the depth of framing, piping, ductwork, suspended ceilings and similar concealed items must be considered as part of the overall design, especially if any of the systems must be stacked within a space. Mechanical systems including heating, ventilation and air conditioning, gas and water pipes and vents, and fire protection systems where applicable generally present the greatest impact on structural systems. The size of ductwork and piping elements and accommodation for the changes in direction of these systems requires provision for openings, chases and horizontal bulkheads that impact placement of structural framing.
Electrical systems which include electric power, lighting, communications (voice, media and data) and controls generally fit within the structural framing without major problems due the small size and flexibility of wiring and conduits. However, these systems have become increasingly more complex in terms of the amount of wiring. This complexity is of particular concern with advanced systems like Structural Insulated Panels or other closed panel products. In addition, lighting appurtenances such as fixtures, control panels, built-in components and similar features occasionally create problems which must be resolved by altering the framing design, moving framing elements or moving the electrical wiring or devices.

The coverings that make up the typical structural system also interact in direct and indirect ways with the ability to keep moisture from becoming a problem in the building. The selection of certain materials that either attach to the structural system or are part of it can create a situation in some climates where the material acts as an unintended vapor retarder and thus retains moisture from either inside or outside the building. Bulk water movement into the building also can be influenced by the structural system. For example, it often is necessary to cut through the structural sheathing to run plumbing and mechanical vents. To reduce the potential for leaks, the design of the venting systems should be coordinated with the structural system design. Even the slope of the roof's structural members can impact water penetration and should be considered.
In addition to the space limitations relative to mechanical systems discussed previously, there must also be adequate provision in terms of wall, floor, or roof thickness to accommodate insulation.

Foundation's impact on utilities and moisture

The foundation system can interact in at least two significant areas with other systems in the building moisture management and utilities. The elevation of the foundation can be too low for proper placement of utilities, or it can be placed to preclude effective drainage. The first case can lead to bulk water entry into the basement or crawlspace. It can also result in failure of the sanitary sewer if the slope is too low for adequate gravity discharge. In addition, just the very presence of utility openings creates potential routes for water entry. In the second case, the flow of water toward the building is one of the most common reasons for wet basements.
Utilities can also be damaged or destroyed if the foundation is not designed to accommodate them. Piping that runs through a foundation is a good example where the allowance for a sleeve should be part of the structural system design. Otherwise, it is not uncommon for plumbing supply and sewer pipes to be sheared at the point of entry through the foundation. Failures can also occur if copper piping is buried in aggressive soils or directly in contact with concrete. The structural system should consider these types of systems interactions in the design stage.
Finally, the foundation system can interact with the thermal envelope to create moisture problems under the right conditions, especially with crawl space construction. Placement of insulation in the floor of a crawlspace typically requires foundation ventilation. Under the right circumstances, this approach can contribute to the very moisture problems it is intended to prevent. Thus, an economical structural solution may create a negative outcome because of failure to reconcile it with the thermal envelope design.

GENERAL DESIGN STEPS

1. Always start with preliminary sketches, creating your own plans of the structure elements from the architects drawings will remove congestion of ideas from the paper and give you an idea of the building concept. As with any structure, conceptually determine the type of system that best meets the layout, size, budget, fire-protection, loading and expected durability. Butter paper is good for this task.
2. Draw/sketch frames indicating the number of stories (including the vertical expansion proposed for design).and number of columns forming the particular frame
3. With a framing concept in hand, do some preliminary sizing calculations to set structure depths and verify that the concept will adequately support the loads and meet serviceability criteria.
4. Identify slab or beam runs and frames. Name all the frames of the layout, number span and columns members

    - All beams of the same types having approximately equal span (+) or (-) 5% variation), magnitude of loading, support conditions and geometric property are grouped together . the heaviest beam of the group is considered for the design. In the preliminary beam design, value of reaction at both ends are worked out for all the loadings acting on the beam.
    - Beams shall be treated as

        - A rectangular beam if it does not support any slab on either side.
        - As L-beam if it supports a slab on one side an
        - As T-beam if it supports slab on both sides.

5. Write the relevant column and beam numbers involved in the frame.
6. Dimension the storey heights (including plinth to footing level) and spans of beams.
7. Show the type of joint at foundation assumed for analysis (i.e. fixed or hinged at bottom).
8. Show all the loads coming on the beams and nodal vertical and horizontal forces.
9. The preferred solution. pattern loading is not required for dead load, (which eliminates some of discrepancy in flat slab design between the tables of coefficients, and accurate analysis). It is therefore necessary, for flat slabs to consider a few loading cases, which are calculated for working loads and then factored for ultimate load. These are: Dead load on all spans, live load on even spans, and live load on odd spans. To these should be added the analysis for the prestress loading, wind and any other loadings. For both ultimate and serviceability design.
10. Design the reinforcing for these beam runs
11. For columns that are not part of the lateral force resisting system, do the design and detail them.
12. Identify which beams might participate in lateral resistance (for moment frames) and create models to analyze the overall building for wind and seismic
13. Design the reinforcing for the lateral system beams and columns.
14. Design any shear walls that are used.
15. For beams acting as collectors, re-design with added tension or compression forces and supplement the reinforcement accordingly.
16. For unique situations, floor openings, special concentrated loads, floor drops, ramps and stairs, provide design efforts on these special areas and adjust supporting beams and columns as necessary Depending on the form and intended use of the building other effects may need to be considered, for example:
    - Braking and Impact forces in buildings where vehicles are present.
    - Effects of earth pressures & ground water on basement walls
    - Buoyancy effects
    -

17. Vertical or Lateral effects of machinery
18. Design the footings.
19. Get all your documents Signed off on sections and details prior to them going to the CAD department. (A cursory review and signoff of sections and details by the senior engineer is required to catch mistakes before sending sections and details to the CAD department. Such a review saves time and is informative for the engineer whose details are being critiqued.)

Loadings

Before starting the final design it is necessary to obtain approval of the preliminary drawings from the other members of the design team. The drawings may require further amendment, and it may be necessary to repeat this process until approval is given by all parties. When all the comments have been received it is then important to marshal all the information received into a logical format ready for use in the final design. This may be carried out in the following sequence:
-checking of all information
-preparation of a list of design data
-amendment of drawings as a basis for final calculations.

Checking of all information

To ensure that the initial design assumptions are still valid, the comments and any other information received from the client and the members of the design team, and the results of the ground investigation, should be checked.

Stability

Ensure that no amendments have been made to the sizes and to the disposition of the core and shear walls. Check that any openings in these can be accommodated in the final design.

Movement joints

Ensure that no amendments have been made to the disposition of the movement joints.

Loading

Check that the loading assumptions are still correct. This applies to dead and imposed loading such as floor finishes, ceilings, services, partitions and external wall thicknesses, materials and finishes thereto.
Make a final check on the design wind loading and consider whether or not loadings such as earthquake, accidental, constructional or other temporary loadings should be taken into account. In general the load case including permanent, imposed, and wind load will be most onerous for all elements, however it is not normally considered necessary to include wind load for members that do not form part of the direct wind resistance system as the wind load effects will be small and can be neglected. However local effects do need to be checked.

Fire resistance, durability and sound insulation

Establish with other members of the design team the fire resistance required for each part of the structure, the durability classifications that apply to each part and the mass of floors and walls (including finishes) required for sound insulation.

Foundations

Examine the information from the ground investigation and decide on the type of foundation to be used in the final design. Consider especially any existing or future structure adjacent to the perimeter of the structure that may influence not only the location of the foundations but also any possible effect on the superstructure and on adjacent buildings.

Performance criteria

Establish which codes of practice and other design criteria are to be used in the final design.

Materials

Decide on the concrete mixes and grade of reinforcement to be used in the final design for each or all parts of the structure, taking into account the fire-resistance and durability requirements, the availability of the constituents of concrete mixes and any other specific requirements such as water resisting construction for basements.

Hazards

Identify any hazard resulting from development of the scheme design. Explore options to mitigate.

IDENTIFYING THE PRESENCE OF ERRORS

These are provided to help you review your work, generally the engineer in charge will not have the time to check all you calculations, Thus **YOU** need to check these a thoroughly as you can.

i. Is the deflected shape consistent with what was expected? When reviewing displaced shape from analysis software, look for beams that rotate at beam-column connections; evaluate whether you intended for the connections to be rigid or not. Verify that the beams you expected to deflect most actually do. Verify that the frames you expected to deflect most under lateral Do.

ii. If most beams are the same size, why are the others not? Evaluate whether you would expect the different beam to be bigger, smaller or the same.

iii. Are the moment diagrams consistent with what was expected? When reviewing moment diagrams from analysis software, look for columns not part of the lateral load resisting system that have moment at the base; evaluate whether the support should be or not. Look for torsion in girders; evaluate whether you intended for the beam-girder connections to cause torsion or not. Verify that the locations where you expected negative have negative moment. Verify that the locations of points, points of zero moment, are where you expected them.

iv. Is the beam depth consistent with standard rules-of-thumb?

v. For lateral load in any direction, do the connections and bracing provide a continuous load path to the foundation? This is why it is good to Draw cross-sections through the entire structure,

vi. Does the building weigh what you anticipate?

vii. Does total base shear equal total applied lateral load?

viii. Do connection details match the assumptions used in the analysis? Identify locations in the structure where you intended to have a rigid or semi-rigid connection

ix. Are the primary structural member sizes similar to members in similar projects?

x. Do beams deflect more than permitted?

xi. Intuitively and instinctively, would you want to walk under it, live above it, and climb on it?

xii. Pipe penetrations, HVAC supports, electrical manholes, civil elevations and locations and the like. Has the conduit been considered? If you overlay the plans, do things line up. Are there dimensional errors between discipline drawings? The most common error I've found is the locations of building using coordinates. If you check the coordinates on the corners of a building, does the dimensions match the structural drawing?

xiii. Before passing on any work to the next stage ask yourself, are you happy with it?
xiv. A variety of tactics are employed when performing reviews. Those tactics areas follows:
a) Look at the big picture.
b) Verify load paths.
c) Review framing sizes.
d) Look at connection details for constructability.
e) Look for mistakes.
f) Look for subtleties.
g) Look at the drawings for constructability.
h) Review for clarity.
i) Look for omissions.
j) Look for "little" little things.
k) Look for the "big" little things.
l) Verify that the structural drawings match the architectural and MEP drawings.
m) Look for mistakes
n) • Look for the subtleties
o) • Look at the drawings through contractor's eyes.
p) • Look at the drawings through contractor's eyes.
q) • Review for clarity/consistency
r) • Look for omissions
s) • Look for "little" little things
t) • Look for "little" little things
u) • Look for "big" little things
v) • Structural dwgs. match Arch. dwgs.?
w) •Any unrealistic load paths?

•Loads jumping in/out of shear walls / braced frames?
•Any unrealistic "rigid" diaphragms?
•Any loads on the structure not in the computer model?

Typical framing to verify model
- Typical framing to verify model
- Major load carrying members
- Wind and seismic loads
- Unique framing not in computer model

Items requiring special attention:
Elevators
Escalators
Folding partitions
Special hang points
Facades
Davits
Special hang points
Rooftop MEP loads
Heavy hung piping
Stairs
Monumental stairs
Hangers
Special loads on joists
Horizontal loads from rigging
Hangers
Theater rigging
Catwalks

Expansion joints

Check connections:
- Critical connections
- Unusual connections
- Connections w/ complex geometry
- Connections w/ large reactions

. Look for mistakes:
- Wrong reactions
- Members too small
- Improper framing configurations
- Not enough reinforcing steel
- Punching shear problems
- Missing structural integrity reinforcing steel
- Missing sections and details
- Mistakes in sections and details
- Column splices at inappropriate locations (mid-height of 80' unbraced height).
- Columns mistakenly assumed to be braced by mezzanines.
- Floors diaphragms w/ insufficient strength/stiffness to brace columns

Look at drawings through the eyes of a contractor:
- Everything shown that will allow contractor to build structure without having to guess or issue RFI's?
- Every linear foot of building perimeter covered by a section?
- Everything clearly indicated?
- Can drawings be interpreted by someone who's not an engineer?

. Review for clarity & consistency:
- Look for conflicts between framing plans and sections/details.
- Inconsistencies with framing
- Inconsistencies with framing

Group similar beams
Consistency = simplicity = economy
- Drafting inconsistencies

Look for omissions:
QA reviews
- Missing things often hardest to find
- Missing:

Section and details
Dimensions and elevations
Sizes
Reinforcing steel

Type of Error	Characteristics	Dangers
Bounded Rationality	Oversimplifies complex issues	Disregards information
Imperfect Rationality	Relies only on past experience	Does not apply basic principles
Reluctant Rationality	Jumps to conclusions	Fails to explore all possibilities

DESIGN CONSIDERATIONS

1. Beams need to have shear effects considered specifically (increased deflection, possibly higher reinforcement requirements in concrete, etc) when SPAN/DEPTH <=10
2. Beam design is normally deflection governed when SPAN/DEPTH >=25.
3. When checking drawings, looking at Section Modulus is a good gauge of a section's sizing, even if using Limit States Design. Knowing the extreme fibre stress is a good "feel" for the beam size.
4. Always consider a minimum accidental eccentricity of 100mm in your construction. Increase this to 150mm in residential work
5. Internal and External Spans, Because of the reduction of continuity at the outer column, moments in outer spans tend to be greater, and so, of course, are the deflections. It is therefore desirable that the external spans should be shorter than the internal ones. (Preferably about 10% to 20% shorter). If this is not possible, the slab depth may be increased in the outer spans and if a uniform slab depth is desired, in the internal spans as well. Alternatively a greater proportion of load can be balanced in the external span.
6. Design shelf angles for the load at the very tip for strength (ULS), centre of bearing for serviceability (SLS). This ensures that any rotation of the beam at the support does not lead to overstress in the fixing; Particularly for stiffened angles.
7. When in doubt, add confinement to concrete.
8. Curtailment of reinforcing should occur at a distance of 130% development length past the point where strength is last required, or Ld+d from support, whichever is greater.
9. To minimize the risk of timber floors (and all high frequency floors; Applies to Cold Formed Steel as well), check that the deflection is no greater than 1 to 2mm under a 1kN point load at centre. Do not consider T-Beam stiffening effect for this check unless the plywood is glued and screwed; slip and fastener loosening may not permit adequate composite action otherwise.
10. For steel and concrete beams, check the estimated natural frequency, equal to 18/SQRT(Total Deflection in mm), result in hertz (HZ). Use anticipated actual loads in this check (thus typically 0.25kPa to 0.35 kPa) rather than full SLS loads. A result of 15Hz or higher should be double checked with the point load check, a result between 8HZ and 15 HZ is likely okay, with likihood of difficulty increasing as the result decreases, and anything between 5HZ and 8HZ should be subject to a full accelerative methodology vibration check. Picking the loading is very important, and entirely subjective; A good guide is to consider 30% of your floor load as the likely "routine" load. That way you are basing the load used on the code's anticipated exposure loads for the floor type. Remember that vibration problems normally happen under light loading.
11. Use Preferred dimensions:

    - Offices & retail 6.0, 7.2, 9.0, 10.5, 12, 15m grids
    - Some retail outlets 5.5m or 11m grids (to suit shop units)
    - Car parks (6,7.5 or 7.2) (8.4m by 10.2m or 8.4m by 8.4m)

12. Use load and resistance calculation techniques that have stood the test of time, but update as necessary possibly due to failures
13. Use characteristic values e.g., 5% exclusion values
14. Use prototypes where possible reduces the impact of contingency (prototype are not limited to cars ect an example would be, Pile-always try and get the first pile tested)
15. Check designs and inspect construction quality control reduces human error see the checking questions below;
16. Make appropriately conservative assumptions in analysis in complex analyses, this technique can sometimes be difficult, e.g., leaving out non-structural elements is not always conservative.
17. Check complex analyses with more simple methods where possible reduces model uncertainty and human error;
18. Develop and Use your own experience and heuristics

WHAT IS HEURISTIC?

Recognize that heuristics are used everywhere in design and think about their limits.
Koen _2003_ has defined a heuristic as "anything that provides a plausible aid or direction in the solution of a problem but is in the final analysis unjustified, incapable of justification, and potentially fallible." Heuristics are techniques we as structural engineers use to help us solve problems and perform designs that would otherwise be intractable or too expensive. According to Koen, all parts of the design process are heuristics. At the limit, his thesis is likely to be true, but let's consider some heuristics that are a bit more obvious. As will become evident, most tools and ideas you use to design structures are heuristics. Consider a few common heuristics.

2. The yield stress for high-strength steel is the 0.2% offset stress. This is a heuristic; it helps us solve problems using high strength steel.
3. The dynamics of wind loads can be ignored in the design of most buildings. If you are designing a low-rise building, you use equivalent static wind loads. You do not include the dynamic effects of the wind directly.
4. Occupancy live load can be modelled as a uniformly distributed static load. Look around you, the live load is not uniformly distributed and is not usually static
5. As a final example, consider the determination of the effective flange width of reinforced concrete T-beams. The procedure used in codes, have been in use for over 90 years Chen et al. 2007. It is a crude but effective way to account for shear lag, a heuristic.

Heuristics are absolutely vital to our ability to design structures. We use them every day without thinking about them, and that is okay as long as we recognize the limits of our heuristics. When the Tacoma Narrows bridge was designed, the heuristic that was used was that wind load only needed to be examined to see how it deflected the bridge laterally Petrosky 1994. That heuristic had reached its limit of applicability for that bridge.
The belief, based on the heuristic that if a structure is still standing then it must be safe is a human error never fall into this trap

CONCRETE PROPERTIES

1. Temperature and shrinkage causes tensile forces in concrete, due to the interaction of reinforcement and concrete; Cracking levels depend on,
   a) Tensile strength of concrete.
   b) The cover thickness.
   c) The diameter of rebar
   d) Rate of corrosion.
   e) Modulus of elasticity of concrete and reinforcement
   f) Spacing of reinforcement
   f) Cement content (or factor) of concrete
   g) Water-cement ratio of concrete
   h) Curing method and length of curing
   i) Aggregate gradation and type (high absorption coarse aggregate increases shrinkage)
   j) Coefficient of expansion for aggregate, least for lime stone highest for granite
2. Poisson's ratio: A value of about 0.15-0.2 is usually considered for design.
3. Shear strength: The strength of concrete in PURE SHEAR has been reported to be in the range of 10 to 20% of its compressive strength.
4. Factors influencing creep:
   Creep increases when,
   a) Cement content is high,
   b) w/c ratio is high,
   c) Aggregate content is low,
   d) Air entertainment is high,
   e) Relative humidity is low,
   f) Temperature (causing moisture loss) is high,
   g) Size / thickness of the member is small,
   h) Loading occurs at an early age &
   i) Loading is sustained over a long period.
   j) Coarse aggregate type and gradation
   k) Magnitude of loading
5. Effect of creep:
   Detrimental results in RC structures due to creep:
   a) Increased deflection of beams and slabs.
   b) Increased deflection of slender columns (possibly leading to buckling)
   c) Gradual transfer of load from concrete to reinforcing steel in compression members.
   d) Loss of prestress in prestressed concrete.
6. Symmetrical arrangements of reinforcement will aid to avoid the differential restraint.
7. Reduction of moments on account of moment redistribution is generally **NOT APPLIED TO COLUMNS.**
8. Reinforcement availability:
   Standard diameter sizes (mm): 6, 8, 10, 12, 16, 20, 24, 32, 40
   Standard lengths: > 12mm diameter: 12 metres
   < 12mm diameter: from a coil

   If you're not going to inspect everything keep the difference in bars sizes greater than 3mm (1/8 inch).
9. These values are approximate and should be used only as a check on the total estimated quantity:

   Slabs - 80 - 110 kg/m3 (50-70lb/ft3) (flat slab120-220kg/m3 (75-140lb/ft3))
   Columns - 200 - 450 kg/m3 (125-280lb/ft3)
   Walls - 60 - 100 kg/m3 (25-65lb/ft3)
   R/C footings 70-90 kg/m3 (45-60lb/ft3)
   Pile caps - 110 - 150 kg/m3 (70-95lb/ft3)
   Rafts - 60 - 70 kg/m3 (40-45lb/ft3)
   Beams - 150 - 220 kg/m3 (95-140lb/ft3) (edge 180kg/m3)
   Transfer slabs 150kg/m3 (95lb/ft3)
   Retaining walls-110kg/m3 (70lb/ft3)

Stairs – 135kg/m3 (85lb/ft3)

Note: The actual reinforcement quantity in the element will vary according to detailing practice and efficiency of the concrete element. For jobs where a provisional quantity of reinforcement is part of the contract documents, the rates should be determined by measurement of the typical elements plus allowance for non-typical and laps.

Reinforcement estimates

In order for the cost of the structure to be estimated it is necessary for the quantities of the materials, including those of the reinforcement, to be available. Fairly accurate quantities of the oncrete and brickwork can be calculated from the layout drawings. If working drawings and schedules for the reinforcement are not available it is necessary to provide an estimate of the anticipated quantities.
In the case of reinforcement quantities the basic requirements are, briefly: for bar reinforcement to be described separately by: steel type, diameter and weight and divided up according to:
a) element of structure, e.g. foundations, slabs, walls, columns, etc.
b) bar 'shape', e.g. straight, bent or hooked; curved; links, stirrups and spacers. for fabric (mesh) reinforcement to be described separately by: steel type, fabric type and area, divided up according to a) and b) above. There are different methods for estimating the quantities of reinforcement; three methods of varying accuracy are given below.

Method 1

The simplest method is based on the type of structure and the volume of the reinforced concrete elements. Typical values are, for example:
warehouses and similarly loaded and proportioned structures: 1 tonne of reinforcement per 10m3
offices, shops, hotels: 1 tonne per 13.5m3
residential, schools: 1 tonne per l5m3.
However, while this method is a useful check on the total estimated quantity it is the least accurate, and it requires considerable experience to break the tonnage down.

Method 2

Another method is to use factors that convert the steel areas obtained from the initial design calculations to weights, e.g. kg/m2 or kg/m as appropriate to the element.
If the weights are divided into practical bar diameters and shapes, this method can give a reasonably accurate assessment. The factors, however, do assume a degree of standardization both of structural form and detailing.
This method is likely to be the most flexible and relatively precise in practice, as it is based on reinforcement requirements indicated by the initial design calculations. Reference should be made to standard tables and spreadsheets available from suitable organisations (e.g. The Concrete Centre).

Method 3

For this method sketches are made for the 'typical' cases of elements and then weighted. This method has the advantages that: the sketches are representative of the actual structure the sketches include the intended form of detailing and distribution of main and secondary reinforcement
an allowance of additional steel for variations and holes may be made by inspection.
This method can also be used to calibrate or check the factors described in method 2 as it takes account of individual detailing methods.
When preparing the reinforcement estimate, the following items should be considered:
Laps and starter bars – A reasonable allowance should be made for normal laps in both main and distribution bars, and for starter bars. This should be checked if special lapping arrangements are used.
Architectural features – The drawings should be looked at and sufficient allowance made for the reinforcement required for such 'non-structural' features.
Contingency – A contingency of between 10% and 15% should be added to cater for some changes and for possible omissions.

10. In normal circumstances and where N-grade (normal) concrete is used, forms may generally be removed after the expiry of the following periods:

Type of Form Work (Location)		Min period before striking
a) Form Work Vertical formwork to columns, Walls, beam		16 - 24 hrs
b) Soffit formwork to slabs (props to be re-fixed immediately after removal of formwork)		3 days
c) Soffit formwork to beams (props to be re-fixed immediately after removal of formwork)		7 days
d) Props to slabs:	Spanning up to 4.5m (16 ft)	7 days
	Spanning Over 4.5m (16ft)	14 days
e) Props to beams & arches:	Spanning up to 6m (55ft)	14 days
	Spanning Over 6m (55ft)	21 days

11. CONCRETE MIX RULES OF THUMB

- ADDING 3L OF WATER TO ONE CUBIC METER OF FRESHLY MIXED CONCRETE WILL:
  a. Increase slump about 25mm (1 inch)
  b. Decrease compressive strength about 1 to 2 mPa (200 to 300 psi)
  c. Increase shrinkage potential about 10%
  d. Waste as much as 1/4 bag of cement
- IF FRESHLY MIXED CONCRETE TEMPERATURE INCREASES 10 DEGREES:
  a. About 3L (1 gallon) OF WATER TO ONE CUBIC METER (per cubic yard maintains) maintains equal slump
  b. Air content decreases about 1%
  c. Compressive strength decreases about 0.5 to 1.2 mPa (150 to 200 psi)
- IF THE AIR CONTENT OF FRESHLY MIXED CONCRETE:
  a. Increases 1%, then compressive strength decreases about 5%
  b. Decreases 1%, then slump decreases about 10mm 1/2 inch
  c. Decreases 1%, then durability decreases about 10%

12. The main components of cast-in-place concrete floor systems are concrete, reinforcement (normal and/or post-tensioned), and formwork. The cost of the concrete, including placing and finishing, usually accounts for about 30% to 35% of the overall cost of the floor system.

Table 1: Load distribution by backpropping for serviceabilit

LOCATION	LOAD	NO BACK-PROPS	ONE LEVEL OF BACKPROPS		TWO LEVELS OF BACKPROPS	
		On slab	On slab	In props	On slab	In props
New slab being cast	total	100%	100%		100%	
Falsework/formwork	W_p	100%		100%		100%
On supporting slab(1)		100% W_p	70% W_p		65% W_p	
In backprops	W_b1			30% W_p		35% W_p
On lower slab (2)			30% W_p		23% W_p	
In backprops	W_b2					12% W_p
On lower slab (3)					12% W_p	

4.

Notes
1. Assumes lower and supporting floors have been struck, have taken up their deflected shape and are carrying their self-weight
2. Floor loading from imposed loads and self-weight is not considered
3. The strength of particular slabs to carry applied loads will have to be considered separately
4. All floors are suspended floors

13. Where post-tensioned reinforcement is used, a concrete compressive strength of at least 40mPa (5,000) psi is usually specified to attain, among other things, more cost-effective anchorages and higher resistance in tension and shear.
14. Having the greatest influence on the overall cost of the floor system is the formwork, which is about 45% to 55% of the total cost.
Three basic principles govern formwork economy for site-cast concrete structures:

- Specify readily available standard form sizes. This is essential to achieve economical formwork. Most projects do not have the budget to accommodate custom forms, unless they are required in a quantity that allows mass production.
- Repeat sizes and shapes of the concrete members wherever possible. Repetition allows forms to be reused from bay to bay and from floor to floor, resulting in maximum overall savings.
- Strive for simple formwork. There are countless variables that must be evaluated and then integrated into the design of a building. Economy has traditionally meant a time-consuming search for ways to reduce the quantities of materials. For example, it may seem appropriate to vary the depth of beams with the loading and span variations, providing shallower beams where the loads or spans are smaller. This approach would result in moderate savings in materials, but would create additional costs in formwork, resulting in a substantially more expensive structure—quite the opposite effect of that intended. Providing a constant beam depth while varying the amounts of reinforcement along the span length is the simplest and most cost-effective solution.

15. ABRASIVE RESISTANCE of concrete increases with compressive strength and use of aggregate shall have low abrasion loss under standardized testing or high abrasion resistance
16. For steel bars to lose one mm diameter due to corrosion, it takes about 12.5 years. For 6mm dia to corrode completely it takes about 75 years in good conditions. But due to practical reasons the number of years reduces due to hostile corrosive environment. In coastal applications, poor consolidation of concrete, insufficient cover, use of chloride admixtures, 6mm rebar corrode completely in 5 years or less.
17. The most significant impact that admixtures have on concrete is usually a positive increase in the ability to consolidate the concrete. Air entraining admixtures, low and high range water reducing admixtures, and set retarding admixtures all contribute to better consolidation, which provides better durability and more consistent properties of the concrete throughout its cross section.
18. Tests have shown that the cracked torsional stiffness can be as low as 1/10 of the un-cracked torsional stiffness (AS5100 limits this to 20%)
19. concepts:

    1. Increase Temperature change through a wall doesn't cause tensile or compression loads. A wall with perfectly pinned supports will only bow (with small temperature differences say up 40deg (even thought this is very unlikely to happen since concrete is a heat sink). Note in below 1 deg concrete changes it behavior and you can have other problems with regard to freezing ect, but that is a topic for another day.

    2. Restraints cause environmental loadings (will be referred to as service loadings from now on). Ie if you have a piece of wire and you lay it on the ground and heat it up, it will expand but no compressive force can develop due to no restraints. Now if we get this same piece of wire and restraint it at both ends and cool it down it will induce a tensile load in the wire.

    3. Concrete is normally designed for two different states, 1. limit, 2. service limit. in limit state it is conservative to assume that the wall is pin pin restraints and worst bending moment from the wall loads is found. for service limits ie temperature and shrinkage, restraints are all that matter, the more restraint the worse the case for these service loads, Just think of a slab and restraint due to walls and shrinkage.

    4. In really high temperature situations you can get spalling cause by expansion of the concrete; this is not in accordance with basic first principles and develops from gas pockets and internal bonds.

    The area's you need to look at are: top, bottom and between walls corners restraints. These will cause restraint moments, note however if you are tanking the structure these items are not as critical.

MODELLING

To model concrete to any real effect you must understand a few parameters first. Effect sections Ieff, concrete sections crack during bending hence you do not get full Ig for preliminary analysis the following Ieff's are suggested, these do not include the effects of creep as mentioned above.

Suggested Effective Member Properties for Analysis
Member Effective moment of inertia for analysis

Structural Members	/ Service	/ Ultimate
Beams	/ 0.5EcIg	/ 0.35EcIg
Flat plates	/ 0.35EcIg	/ 0.25EcIg.

Flat plates (equivalent slab-beams consideration)/1.0EcIe / 0.70EcIe
This is basically according 10.11.1 and R10.11.1 x1.43 for service as per ACI 318.
Note: Ig is the gross uncracked moment of inertia. Use gross areas for input of cross-sectional areas.

For Columns	$0.80\ I_g$	when	$N^* / fc'\ A_g > 0.5$
	$0.60\ I_g$		$N^* / fc'\ A_g = 0.2$
	$0.40\ I_g$		$N^* / fc'\ A_g = -0.05$
For Structural Walls	$0.45\ I_g$	when	$N^* / fc'\ A_g = 0.2$
	$0.25\ I_g$		$N^* / fc'\ A_g = 0$
	$0.50\ I_g$		$N^* / fc'\ A_g = -0.1$

COMPATIBILITY TORSION:

Please do extra research on this as you will need to know it limitations such as box beams, curved beams ect.

AS3600- 8.3.2 Torsion redistribution
Where torsional strength is not required for the equilibrium of the structure and the torsion in a member is induced solely by the angular rotation of adjoining members, it shall be permissible to disregard the torsional stiffness in the analysis and torsion in the member, if the torsion reinforcement requirements of Clauses 8.3.7 and the detailing requirements of Clause 8.3.8 are satisfied. C8.3.2 Torsion redistribution The concept behind this Clause has been derived from compatibility torsion and incorporated in the ACI 318 Code. In a statically indeterminate structure, where alternative load paths exist and the torsional strength of a member is not required for equilibrium (i.e. compatibility torsion), the torsional stiffness of the members may be disregarded in analysis and torsion may be ignored in design. However, minimum torsional reinforcement in accordance with Clause 8.3.7 must still be provided to avoid serviceability problems.

ACI: Two conditions determine whether it is necessary to consider torsional stiffness in the analysis of a given structure: (1) the relative magnitude of the torsional and flexural stiffness's, and (2) whether torsion is required for equilibrium of the structure (equilibrium torsion) or is due to members twisting to maintain deformation compatibility (compatibility torsion). In the case of compatibility torsion, the torsional stiffness may be neglected.

Within that code the ACI explains that if you have secondary beams framing into a primary edge beam, as long as you design the secondary beams for pinned ends, you can then design the primary beam for a minimum torsion loading instead of a full analysis which would assume fixed ends and calculated torsion.

They even have a couple of 3D sketch-views of two structures showing the difference between a structure which doesn't need the torsional resistance for stability and one that does need the torsional resistance for stability. For a typical exterior bay of a building, where the interior joists or beams are designed with assumed pinned exterior ends, then the exterior beam's torsional resistance is not theoretically needed for the structural stability of the floor.

For a beam that has a single cantilevered slab hanging off the edge of it, the torsional strength of that beam is essential for the cantilevered slab to remain cantilevered...thus for that sort of case you must include the torsional aspects of the design.

PRELIMINARY DESIGN RULES OF THUMB

1. The cost of reinforce concrete (in place) is usually somewhere between \$100/m3 and \$800/m3. This illustrates the fact that for a "rule of thumb" to be any good, the background for its development needs to be known. That in turn means that most of the "rules of thumb" are most applicable by the engineer that came up with them; and that everybody else better be careful in using them.
2. If placing concrete directly from a truck or concrete pump, place concrete vertically into the face of concrete already in place. Never allow the concrete to fall more than 1 to 1.5 metres.
3. The possible spans, and associated depths, depend on the loading to which the beam is subjected. The figures given assume 'normal' commercial building loads. They do not apply to more heavily loaded situations (e.g. plant rooms) or to unconventional loading scenarios.
4. This information is given without prejudice and is for guidance purposes only. It is suitable for possibly initial sizing of structural elements for architectural scheme or costing purposes for actual building projects the size of structural elements must be verified through detailed design by a qualified structural engineer.
5. Prepare drawings properly & accurately if possible label each bar and show its shape for clarity
6. Indicate proper cover-clear cover, nominal cover or effective cover to reinforcement.
7. Decide detailed location of opening/hole and supply adequate details for reinforcements around the openings.
8. Use commonly available size of bars and spirals. For a single structural member the number of different sizes of bars shall be kept to a minimum.
9. Show enlarged details at corners, intersections of walls, beams and column joint and at similar situations.
10. Congestion of bars should be avoided at points where members intersect and make certain that all rein. Can be properly placed.
11. In the case of bundled bars, lapped splice of bundled bars shall be made by splicing one bar at a time; such individual splices within the bundle shall be staggered.
12. Make sure that hooked and bent up bars can be placed and have adequate concrete protection.
13. Indicate all expansion, construction and contraction joints on plans and provide details for such joints.
14. The location of construction joints shall be at the point of minimum shear approximately at mid or near the mid points. It shall be formed vertically and not in a sloped manner.

<u>DO NOT'S-GENERAL:</u>

15. Bonded reinforcement shall not extend across an expansion joint and the break between the sections shall be complete.
16. Flexural reinforcement preferably shall not be terminated in a tension zone.
17. Bars larger than 36mm dia. Shall not be bundled.
18. Lap splices shall be not be used for bars larger than 36mm dia except where welded.
19. Where dowels are provided, their diameter shall not exceed the diameter of the column bars by more than 3mm.
20. Primary movement joints are required to prevent cracking where buildings (or parts of buildings) are large, where a building spans different ground conditions, changes height considerably or where the shape suggests a point of natural weakness. Without detailed calculation, joints should be detailed to permit 15–25 mm movement unless seismic pounding is an issue then this should be increased to min 200mm. Advice on joint spacing for different building types can be variable and conflicting. While rules of thumb are provided be sure to seek guidance of an experienced engineer. Expansion joint is a movement (functional) joint which is installed to accommodate volume change due to temperature changes, shrinkage, and change in moisture content.. The other members of this family of joints are:
    - Control (contraction) joints

- Shrinkage strips.

According to MARK FINTEL the use of Expansion joints in a building is a controversial issue. There is a great divergence of opinion concerning the importance of expansion joints in concrete construction. Some experts recommend joint spacing's as low as 30 ft while others consider expansion joints entirely unnecessary.

Joint spacing's of roughly 100 to 200 ft for concrete structures seem to be typical ranges recommended by various authorities. For steel structures a spacing of 200 ft is normal.

However, the spacing is most of the times also dictated by the following factors which determine the location of such joints:

- New building adjoining existing building
- Long low building abutting higher building
- Wings adjoining main structure
- Long low connecting wings between buildings
- Intersections at wings of 'L', 'T', or 'U' shaped buildings.
- Soil strata in length of structure vary in nature -- better to have joints (such situation is very rare)
- Different type of foundations in structure (i.e. building founded piles and raft)

21. Actual concrete deflections are influenced by many factors which cannot be fully taken into account.

- Tensile strength of concrete a change in strength from 2.7 to 2.1 can increase deflections by 50%
- Modulus of concrete +/- 20%
- Early construction loading
- Shrinkage wrapping

Always remember load can only be estimated and even dead loads cannot usually be calculated to within 5% accuracy. With this in mind not calculation need have more than 2 significant figures.

22. For high-risks facilities such as public and commercial tall buildings, design considerations against extreme events (bomb blast, high velocity impact) is very important. It is recommended that guidelines on abnormal load cases and provisions on progressive collapse prevention should be included in the current Building Regulations and Design Standards. Requirements on ductility levels also help improve the building performance under severe load conditions.

INITIAL ESTIMATIONS

"To design even a simply supported beam, the designer needs to guess the beam size before he can include its self-weight in the analysis."

BEAMS:

OVERALL DEPTH OF BEAMS:

MEMBER	SPAN/OVERALL DEPTH RATIO reinforced	Max recommended span	SPAN/OVERALL DEPTH RATIO Pre-stressed/post-tensioned	Max recommended span
Rectangular BEAM width >250mm or span/15 but less than 5D.	10 TO 14 (con't 20-26)	8m (con't12m)	13-20	12m (con't 15m)
Flanged beams	12-18 (con't 18-21)			
cantilever	2-6	5m		
Band beams (*b*≈3*D*) or span/5	18-20	8-12	25-30	14 (18m con't)
For non band beams -Concrete beams: 2 to 1 depth to width ratio the width (b) of a rectangular beam should be between 1/3 and 2/3 of the effective length (d). The larger fraction is used for relatively larger design moments. Band width - L/4 to L/3 max at slab soffit (L for average transverse span) I would normally taper the sides of the band with a 1 to 1.5' taper for the L/25 or a 2' taper for the L/33 to save weight but that depends on formwork costs) Band depth - L/25 to L/30 (L for longitudinal span)				

Notes Note:

1. Beams need more depth to fit sufficient reinforcing in section so check detailing early
2. The maximum spans listed here are not absolute limits. Longer spans are possible with every type, but may not be economical. As a rule of thumb for estimates of thickness above the span on deflection ratio's should be multiplied by "maximum recommended span"/"actual span")
3. The higher number are given for light loadings (about 1.5 kpa) and the lower numbers for heavy loadings (about 10kpa)

Common band beam widths are 1200,1800 and 2400

b For flanged sections with the ratio of the flange to the rib width greater than 3, the Table value for beams should be multiplied by 0.8.
c For members, other than flat slab panels, which support partitions liable to be damaged by excessive deflection of the member, and where the span exceeds 7m, the Table value should be multiplied by 7/span.

d For flat slabs where the greater span exceeds 8.5m, the Table value should be multiplied by 8.5/span.

e The values may not be appropriate when the formwork is struck at an early age or when the construction loads exceed the design load. In these cases the deflection may need to be calculated using advice in specialist literature.

Sizing:
For non-cantilevers: d (mm) = span (mm)/26 + 300, round the result to nearest 25mm.
For cantilevers: d (mm) = span (mm)/7 + 300, round the result to nearest 25mm.
For non-cantilevers:
If span < 6000mm, b (mm) = 300
If 6000 < span < 9000, b = 350
If 9000 < span < 12000, b = 400
For cantilevers, b (mm) = 300

Earthquake Loading:
The total earthquake load on a building is called the Base Shear, V. Estimate this loading V as,
V = 0.1W, where W is the total weight of the building.

1. Beam sections should be designed for:
    - Moment values at the column face & (not the value at centre line as per analysis)
    - Shear values at distance of d from the column face. (not the value at centre line as per analysis)
    - Moment redistribution is allowed for static loads only.
2. Use higher grade of concrete in most of the beams that are doubly reinforced.
3. Whenever possible try to use T-beam or L-beam concept so as to avoid compression reinforcement.
4. Use a min. of 0.2% for compression reinforcement to aid in controlling the deflection, creep and other long term deflections.
5. Length of curtailment shall be checked with the required development length.
6. Keep the higher diameter bars away from the N.A (i.e. layer nearest to the tension face) so that max. Lever arm will be available.
7. The maximum area of either the tension or compression reinforcement in a horizontal element is 4% of the gross cross-sectional area of the concrete.
8. Shear where shear stress (*v*) is considered to be critical, it can be calculated as follows:
23. $v = V/bd$ for beams ideally it should be less than 2 N/mm2 to avoid congestion, but this may not be possible for transfer beams where shear is critical.
24. Where splices are provided in bars, they shall be, as far as possible, away from the sections of maximum stresses and shall be staggered.
25. Were the depth of beams exceeds 750mm in case of beams without torsion and 450mm with torsion provide face rein.
26. Deflection in slabs/beams may be reduced by providing compression reinforcement.
27. Only closed stirrups shall be used for transverse rein. For members subjected to torsion/”compatibility torsion” and for members likely to be subjected to reversal of stresses as in Seismic forces.
28. To accommodate bottom bars, it is good practice to make secondary beams shallower than main beams, at least by 50mm.

SLABS:

OVERALL SLAB DEPTH:

SLAB	Reinforced SPAN/ DEPTH	Max recommended span	Pre-stressed span/depth	Max recommended span
One- way simply supported slab	20-30	6m	25-33	8m
One-way continuous slabs	28-34	7m	28-42	9m
Two-way simply supported slabs	28-34 for L/B=<1.5 24-30 for L/B>1.5	6.5m		
Two-way continuous slabs	36-40 for L/B=<1.5 30-34 for L/B>1.5	7.5m		
Cantilever slab	6-11	3m		
Flat plate	23-28	7m	30-45	9m
Slab with drop panels (min L/6 each side of column, 1.3 time the thickness)	28-36		35-50	12m
Band-beams ($b \approx 3D$)			35-45	11m
Profiled steel decking/concrete composite	35–40	8m (shored)		

Note:
- The maximum spans listed here are not absolute limits. Longer spans are possible with every type, but may not be economical.
- for flat plates and flat slabs with drop panels, the longer of the two orthogonal spans is used in the determination of the span-to-depth ratio, while for edge-supported slabs, the shorter span is used.
- minimum fire resistance normally require a depth of at least 125mm, 150 is best for fitting the reo in.
- The higher span to depth ratios are for light loadings (about 1.5 kPa) and the lower span to depth ratio's for heavy loadings (about 10kpa). The spans assume roughly 1.50 kPa for superimposed dead loading (SDL).

Notes
- For two-way spanning slabs (supported on beams), the check on the ratio of span/effective depth should be carried out on the shorter span. For flat slabs, the longer span should be taken.
- For flanged sections with the ratio of the flange to the rib width greater than 3, the Table value for beams should be multiplied by 0.8.
- For members, other than flat slab panels, which support partitions liable to be damaged by excessive deflection of the member, and where the span exceeds 7m, the Table value should be multiplied by 7/span.
- For flat slabs where the greater span exceeds 8.5m, the Table value should be multiplied by 8.5/span.
- The values may not be appropriate when the formwork is struck at an early age or when the construction loads exceed the design load. In these cases the deflection may need to be calculated using advice in specialist literature.

1. Provide a max spacing of 250mm(8") for main reinforcement in order to control the crack width and spacing.
2. A min. of 0.24% shall be used for the roof slabs since it is subjected to higher temperature. Variations than the floor slabs. This is required to take care of temp. Differences.
3. Spans are defined as being from centreline of support to centreline of support. Although square bays are to be preferred on grounds of economy, architectural requirements will usually dictate the arrangement of floor layouts and the positioning of supporting walls and columns. Pinned supports are assumed.
4. Particular attention is drawn to the need to resolve lateral stability, and the layout of stair and service cores, which can have a dramatic effect on the position of vertical supports. Service core floors tend to have large holes, greater loads but smaller spans than the main area of floor slab.
5. Eliminating drops results in simpler false work and formwork arrangements, enables rapid floor construction and giving maximum flexibility to the occupier.
6. The benefits of using in-situ concrete flat slab construction should be investigated at the conceptual design stage. Consider not only the benefits in terms of potential design efficiencies but also the major advantages for the overall construction process, notably in simplifying the installation of services and the savings in construction time.
7. To optimise the slab thickness, consider all factors such as the method of design, the presence or absence of holes, the importance of deflections, and previous experience.
8. Deflections will generally be greatest at the centre of each panel. However, as partitions may be placed along column lines, it is usual to check deflections here also. The possible effect of deflections on cladding should also be considered carefully. Edge thickenings, up stand and down stand beams should be avoided, as they disrupt the construction process.
9. There is evidence that early striking and early loading through rapid floor construction has some impact on long-term deflections.
10. Thin flat slab construction will almost certainly require punching shear reinforcement at columns.
11. Minimum recommended thickness for slabs for fire is 120mm
12. Drape slab tendons to high points at the faces of the bands at the slab soffit and run flat over band width at minimum top cover, except at end columns where they are draped to the centerline of the column and centroid of the slab (Dslab/2 from the top surface).
13. When openings in floors or roofs are required such openings should be trimmed where necessary by special beams or reinforcement so that the designed strength of the surrounding floor is not unduly impaired by the opening. Due regard should be paid to the possibility of diagonal cracks developing at the corners of openings. The area of reinforcement interrupted by such openings should be replaced by an equivalent amount, half of which should be placed along each edge of the opening. For flat slabs, openings in the column strips should be avoided.
14. When is it an advantage to use fabric mesh in suspended concrete slabs?

Designing slabs with mesh reinforcement is a proposition that can produce substantial cost savings when consideration is given to the following points.

a) Mesh should be sufficient for shrinkage control without additional reinforcement.

b) The structural system should be predominantly a "one way slab" system, with extra bars at the maximum moment.

c) Lapping of mesh should be minimal.

d) Lapping locations should be clearly documented so as to eliminate any possibility of top and bottom laps being coincident and so as to maximise usage and minimise cutting.

e) Lapping should be achieved using bar splices so that each mesh remains in the same plane.

f) Mesh lengths should be factor of the sheet length preferably using full sheets to minimise wastage.

PRESTRESS:

1. **Advantages of using prestressed concrete**

a) Increased clear spans
b) Thinner slabs
c) Lighter structures
d) Reduced cracking and deflections
e) Reduced storey height
f) Rapid construction
g) Water tightness
h) Note: use of prestressed concrete does not significantly affect the ultimate limit state (except by virtue of the use of a higher grade of steel).

2. **Maximum length of slab**
   50m, bonded or unbounded, stressed from both ends.
   25m, bonded, stressed from one end only.
3. **Mean prestress**
   Typically P/A: Slabs: 0.7 – 2.5 mPa
   Beams: 1.0 – 3.0 mPa
   1.4mPa economic design
   3.5 mPa is max before shortening ect will become problems

4. **Cover**
   Take minimum cover to be 25mm.
5. Allow sufficient cover for (at least) nominal bending reinforcement over the columns, in both directions (typically T16 bars in each direction).
6. **Effect of restraint to floor shortening**
   All concrete elements shrink due to drying and early thermal effects but, in addition, prestressing causes elastic shortening and ongoing shrinkage due to creep. Stiff vertical members such as stability walls restrain the floor slab from shrinking, which prevents the prestress from developing and thus reducing the strength of the floor. This should be considered in the design of the stability system or allowed for in the method of construction
7. Pour strips are usually 1200 to 2000 wide, reinforced as required for strength, normally located in the centre third of the span, and left as long as possible before casting.
8. The actual shortening caused by the prestress is only about 10% of the total shortening. The other 90% will be caused by creep, shrinkage and temperature change.
9. For these sort of slab dimensions, shortening should not crack the columns excessively as long as the axial prestress is relatively low, 1.5 - 2.2 MPa (200-300psi).
10. 12.7dia -100mm2 = fp 1840mPa
11. 15.2 dia = 143mm2=1750mPa
12. Concrete genrally 32mPaand 22 mPa at transfer at 4 days after placement with a low stress added at 1 dy for cracking control
13. Immediate loss = 6mm total =15-25% of prestressing

occupancy	Partition and other super imposed loads kpa	Design live load kpa	Load balance kpa
carpark	-	3	(.7-.9)sw
Shopping centre	0-2	5	(.9-1.1)SW
residential	2-4	2	Sw+50%of partition loads
Office buildings	.5-1.0	3	(0.8-1.0)SW
Storage (check transfer)	-	2.4 /m hieght	SW+20-30%ll

Do stiff elements (restraining elements) existing in the structure? If only one still element then possibly ok, ensure this is close to the shrinkage point ect. If more than on stressing being transmitted into the structure instead of the slab, try to change structure thus that this isn't a problem

Closely examine anchorage zones and anti-burst steel configurations and clash points.

Attention needs to be paid to:
Access for jacking min 600mm
Restraint forces from columns and walls, for multi storey building the first prestress is the most critical.
Change in centroid at drop panels
Detailing of anchorage
Detailing of reinforcement

Prestressing rate
Office: 5 kg.m2 + 30kg/m2 normal
Retial 6 kg/m2 + 35 kg/m2 normal
Roofs 7.5 k/m2 +55 kg/m2 normal

COLUMNS:

1. Use higher grade of concrete when the axial load is predominant.
2. Go for higher section properties when the moment is predominant.
9. Restrict the maximum % of reinforcement to about 3%. In an in-situ column the absolute maximum reinforcement is 6% or 10% at laps.
3. Approximate method for allowing for moments: multiply the axial load from the floor Immediately above the column being considered) by:
   1.25-interior columns (allows for pattern loading)
   1.50-edge columns
   2.00-corner columns
   But try keep the columns to constant size for the top two storeys.
4. Preliminary sizing- best to aim for coulmns with 1 to 2 % reinfcoing

   - Column- H/10-20,
   - edge columns H/7-9,
   - corner column H/6-8
   - Ac can be estimated for stocky columns by Ac= N/15 (1% reo), Ac=18 (2% reo) or N/20 (3% reo) for N32 concrete. (N in newtons)

5. A column should have minimum section 200-250 sq, if it is not an obligatory size column.
6. In addition, all columns shall be designed for minimum eccentricity equal to [(unsupported length of column /500)+(lateral dimension /30] subject to minimum eccentricity of 20mm.
7. A reinforced column shall have at least six bars of longitudinal reinforcement for using in transverse helical reinforcement.-for CIRCULAR sections.
8. A min four bars one at each corner of the column in the case of rectangular sections.
9. Keep outer dimensions of column constant, as far as possible, for reuse of forms.
10. When does a column change to wall? generally this considered at about 4 times the thickness, however for fire purposes if the fire can get to all four sides it should be considered as a column.
11. For service load keep the total stress about 0.3fc' for gravity load keep this about 0.15f'c.
12.

REINFORCED CONCRETE COLUMNS:
Sizing:
For preliminary design use square columns.
If the building height is 3 stories or less:
If beam span < 6000mm, h (mm) = 300
If 6000 < beam span < 9000, h = 350
If 9000 < beam span < 12000, h = 400
If the building height is 4 to 9 stories:
If beam span < 6000mm, h (mm) = 400
If 6000 < beam span < 9000, h = 500
If 9000 < beam span < 12000, h = 600

REINFORCED CONCRETE WALLS:

1. The minimum reinforcement for the RCC wall subject to Bending moment shall be as follows:
   A. Vertical reinforcement:
      a) Cross sectional area for deformed bars not larger than 16mm in diameter and with characteristic strength 500 mPa N/mm^2 or greater.

      Maximum horizontal spacing for the vertical reinforcement shall neither exceed three times the wall thickness nor 450mm.
   B. Horizontal reinforcement.
      a) cross sectional area for deformed bars not larger than 16mm in diameter and with characteristic strength 500 mPa N/mm^2 or greater.

      Maximum vertical l spacing for the vertical reinforcement shall neither exceed three times the wall thickness nor 450mm.

      NOTE: The minimum reinforcement may not always be sufficient to provide adequate resistance to effects of shrinkage and temperature.
2. Walls H/30-45
   a) 0.75 H, if the rotations are restrained at the ends by floors where h is the height of the wall.
   b) 1.0h .
3. generally H/50 is a good starting point for tilt panels
4. Expansion joint spacing's

- Concrete large thermal differences 25 m (e.g. for roofs) otherwise refer below

   - Movement joints should be provided to minimise the effects of movement caused by shrinkage, temperature variations, creep and settlement. The effectiveness of a movement joint depends on there location, movement joints should divide the building into a number of individual sections. Movement joints should pass through the whole structure above ground in one plane. The structure should be framed both sides of the joint.

Author Spacing:

- Lewerenz (1907) 75 ft (23 m) for walls.
- Hunter (1953) 80 ft (25 m) for walls and insulated roofs, 30 to 40 ft (9to 12 m) for uninsulated roofs.
- Billig (1960) 100 ft (30 m) maximum building length without joints. Recommends joint placement at abrupt changes in plan and at changes in building height to account for potential stress concentrations.
- Wood (1981) 100 to 120 ft (30 to 35 m) for walls.
- Indian Standards Institution (1964) 45m (≈148ft) maximum building length between joints.
- PCA (1982) 200 ft (60 m) maximum building length without joints.
- ACI 350R-83 120 ft (36 m) in sanitary structures partially filled with liquid (closer spacing's required when no liquid present).

5. Shear walls are essentially vertical cantilevers, and may be sized as such; therefore a span-to-depth ratio of 7 is reasonable for a shear wall. However, at this aspect ratio it is highly likely that tension will be developed at the base and this requires justification in the design (see Figure 2.15). Pad foundations should be designed to resist overturning and piles may be required to resist tension. The wall should be checked to ensure that it is 'short', the minimum practical thickness is 200 mm. The wall should be 'braced', i.e. there should be another shear wall in the orthogonal direction. For a building to be classified as shear walled the stiffness of the shear walls must be 6 time the columns, such to ensure they take most / all the load.
6. Shear wall thickness, for concrete 150 for single layer of reo, 220 for double layer
7. Tilt panel propping 2/3 minimum height up panel

FORMWORK

According to the Concrete Reinforcing Steel Institute (CRSI), “formwork and its associated labor is the largest single cost segment of the concrete structural frame—generally more than 50%.”

• maintaining constant depth of horizontal construction
• maintaining constant spacing of beams and joists
• maintaining constant column dimensions from floor to floor
• maintaining constant story heights

Standard Forms

Since most projects do not have the budget to accommodate custom forms, basing the design on readily available standard form sizes is essential to achieve economical formwork. Also, designing for acctual dimensions of standard nominal lumber will significantly cut costs. A simplified approach to formwork carpentry means less sawing, less piecing together, less waste, and les time. this results in reduced labor and mated costs and fewer opportunities for error by construction workers.

Repetition

Whenever possible, the sizes and shapes of the concrete members should be repeated in the structure. By doing this, the forms can be reused from bay to bay and from floor to floor, resulting in maximum overall saving. The relationship between cost and changes in depth of horizontal construction is a major design consideration. By standardizing the size or, if that is not possible, by varying the width and not the depth of beams, most requirements can be met at a lowered cost, since the forms can be reused for all floors. To accommodate load and span variations, only the amount of reinforcement needs to be adjusted. Also, experience has shown that changing the depth of the concrete joist system from floor to floor because of differences in superimposed loads actually results in higher costs. Selecting different joist depths and beam sizes for each floor may result in minor savings in materials, but specifying the same depth for all floors will achieve major savings in forming costs.

Simplicity

In general, there are countless variables that must be evaluated, and then integrated into the design of a building. Traditionally, economy has meant a time-consuming search for ways to cut back on quantity of materials. As noted previously, this approach often creates additional costs-quite the opposite effect of that intended. An important principle in formwork design is simplicity. In light of this principle, the following questions should be considered in the preliminary design stage of any project
1) Will custom forms be cost-effective? Usually, when standard forms are used, both labour and material costs decrease. However, custom forms can be as cost-effective as standard forms if they are required in a quantity that allows mass production. Class A1 concrete finish

It is firstly imperative that you understand that this type of formwork is normally only specified for monumental surfaces of relatively small area. For just what to expect and when it should be specified we suggest reading AS3610 and commentary. We can fax you some extracts if need be. The Cement and Concrete Association also has some useful publications on the subject.

Some information that you won’t find in the above literature but that you may find useful is given below.

Rebates

It is important to specify what materials shall be used to form rebates as standard rebates used to be ripped from maple but are now often ripped from miranti or other cheaper timber. The trouble with some of these is that the surface left after ripping is often not smooth and this has a deleterious effect on the final product. Consider specifying clear strips of radiata pine but also check and approve all rebate timber prior to being used in the forms.

Plastic rebates leave a beautifully smooth finish but can cause serious problems where you have high daily temperature variations. The thermal coefficient of plastic can be as much as 20 times that of timber.

Nails are generally used to hold the rebates in place and the specification may call for these to be punched filled and sanded smooth. Covering nail holes is also a little more difficult with plastic rebates.

Curing

If you are looking for colour control A, we do not recommend wetting intermittently or covering with plastic or leaving the formwork on too long as all of these will tend to lead to a colour variation. Consider the use of a water based acrylic curing compound complying with AS3799 such as Masterkure 404 but remember that some of these do leave a milky film on the surface that can take a while to vanish especially if it is rolled on rather than sprayed.

If your concrete is coloured forget these water based acrylic and go for a solvent based alternative such as Masterkure 402 and make sure you spay it. It does not quite meet the Australian Standard for water retention but it will give you a superior finish.

If you are doing coloured precast work we normally recommend using CCS same day sealer and curing out of the sun and wind.

Bar Chairs

Give serious thought as to how you can keep the reinforcement in position without the need for bar chairs and take extreme care during the placement and vibration operations. Make the contactor aware that you have a cover meter and will reject any elements found with reinforcement cover outside the specification tolerances.

Formwork

For this class of finish the formwork needs to be very stiff and often has a reduced allowance for form tie holes or a detailed setout of the requirements. If you need to have the formwork specifically designed or require an structural engineer's certification
2) Are deep beams cost-effective? As a rule, changing the beam depth to accommodate a difference in load will result in materials savings, but can add considerably to forming costs due to field crew disruptions and increased potential for field error. Wide, flat beams are more cost-effective than deep narrow beams.
3) Should beam and joist spacing be uniform or vary with load? Once again, a large number of different spacing's (closer together for heavy loads, farther apart for light) can result in material savings. However, the disruption in work and the added labour costs required to form the variations may far exceed savings in materials.
4) Are formed surface tolerances reasonable? The suggested tolerances for formed castin-place surfaces are shown in, The following simplified guidelines for specifying the class of formed surface will usually minimize costs:
a) Class I finish should be specified for surfaces prominently exposed to public view,
b) Class II finish should be specified for surfaces less prominently exposed to public view,

c) Class III finish should be specified for all noncritical or unexposed surfaces, and
d) Class IIII finish should be specified for concealed surfaces or for surfaces where roughness is not objectionable. If a more stringent class of surface is specified than is necessary for a particular formed surface, the increase in cost may become disproportionate to the increase in quality

Floors and the required forming are usually the largest cost component of a concrete building structure. The first step towards achieving maximum economy is selecting the most economical floor system for a given plan layout and a given set of loads. This will be discussed in more detail below. The second step is to define a regular, orderly progression of systematic shoring and re-shoring. Timing the removal of the forms and requiring a minimum amount of re-shoring are two factors that must be seriously considered since they can have a significant impact on the final cost.

Figures 1-5 and 1-6 show the relative costs of various floor systems as a function of bay size and superimposed load. Both figures are based on a concrete strength f: = 4000 psi. For a given set of loads, the slab system that is optimal for short spans is not necessarily optimal for longer spaas. Also, for a given spaa, the slab system that is optimal for lighter superimposed loads is not necessarily optimal for heavier loads. Reference 9.3 provides material and cost estimating data for various floor systems, It is also very important to consider the fire resistance of the floor system in the preliminary design stage (see Chapter 10). Required fire resistance ratings can dictate the type of floor system to specify in a particular situation.
The relationship between span length, floor system, and cost may indicate one or more systems to be economical for a given project. If the system choices are equally cost-effective, then other considerations (architectural, aesthetic, etc.) may become the determining factor. Beyond selection of the most economical system for load and span conditions, there are general techniques that facilitate the most economical use of the chosen system.

Slab Systems

Whenever possible, avoid offsets and irregrrlaxities that cause a “stop and start” disruption of labour and require additional cutting (and waste) of materials (for example, breaks in soffit elevation). Depressions for terrazzo, tile, etc. should be accomplished by adding concrete to the top surface of the slab rather than maintaining a constant slab thickness and forming offsets in the bottom of the slab.

Cross section (a) in Fig. 9-2 is less costly to form than cross section (b).

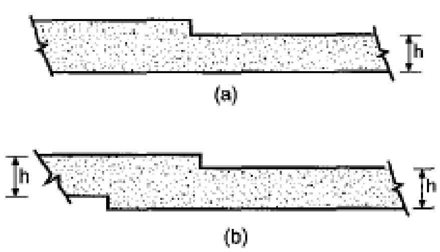
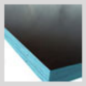
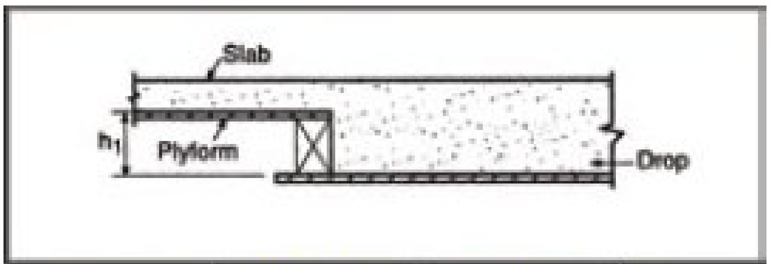


Figure 9-2 Depressions in Slabs

When drop panels are used in two-way systems, the total depth of the drop h should be set equal to the actual nominal lumber dimension plus ¼ in. for ply form (see Fig. 9-3). Table 9-2 lists values for the depth hl based on common nominal lumber sizes. As noted above, designs which depart from standard lumber dimensions are expensive“ Keep drop dimensions constant

Formply (Hardfaced/Formrite)		Standard: AS/NZS 2269			
Appearance	Description	Grade	Thickness	Length x Width (mm)	Applications
	Formply, other-wise known as Hardface, is a plywood made with a bond glue, radiata core and hardwood veneer faces with a high density overlay (HDO) resin impregnated finish that is smooth, perfect for use in concrete formwork and can be used several times.	HDO face HDO back F11/F14	12mm 17mm 17mm 18mm	2400 x 1200 1800 x 1200 2400 x 1200 2400 x 1200	Formwork in walls, floors, roofs, frames, bridges, dams, etc.



Timber size Actual size	Plyform thickness	H1
35	19	55 (54)
47	19	65 (66)
65	19	85 (84)
95	19	115 (114)
130	19	149 (150)
150	19	169 (170)

Figure 9-3 Formwork for Drop Panels

Whenever possible, a minimum 16ft (plus 6 in. minimum clearance) spacing between drop panel edges should be used (see Fig. 9-3). Again, this permits the use of 16ftlong standard lumber without costly cutting of material. For maximum economy, the plan dimensions of the drop panel should remain constant throughout the entire project.

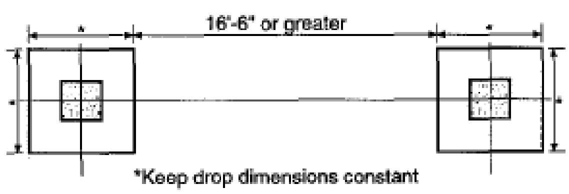
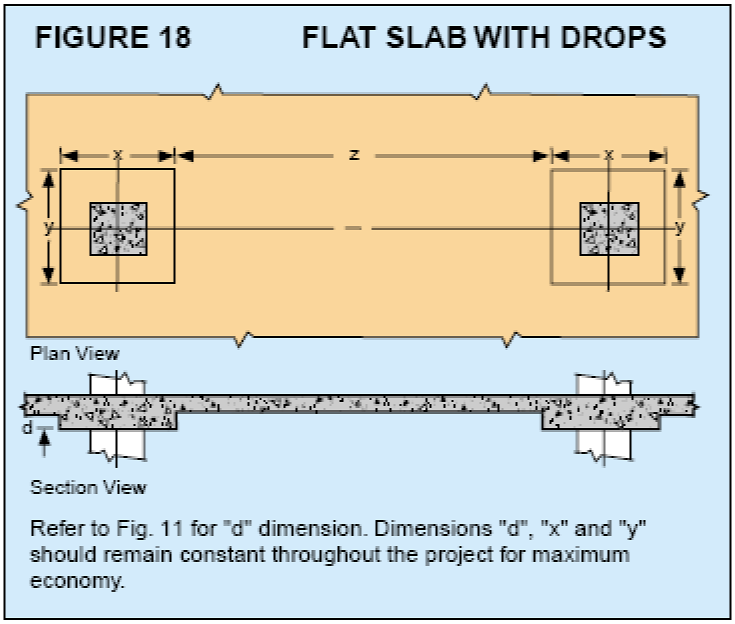


Figure 9-3 Formwork for Drop Panels



Beam-Supported Slab Systems

The most economical use of this relatively expensive system relies upon the principles of standardization and repetition, optimal importance is consistency in-depth, and of second~ importance is consistency in width.
These two concepts will mean a simplified design, less time spent interpreting plans and more time for field crew to produce.

ECONOMICAL ASPECTS OF VERTICAL FRAMING

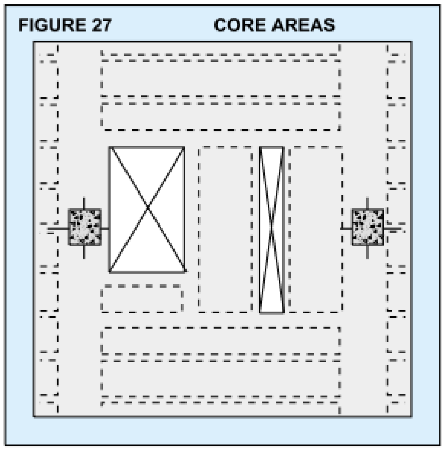
Walls

Walls provide an excellent opportunity to combine multiple functions in a single element; by doing this, a more economical design is achieved. Whh creative layout and design, the same wall can be a fire enclosure for stair or elevator shafts, a member for vertical support, and bracing for lateral loads. Walls with rectangular cross-sections are less costly than non-rectrmgular walls.
9.4.2 Core Area
Core areas for elevators, stairs, and utility shafts are required in many projects. In extreme cases, the core may require more labour than the rest of the floor. Standardizing the size and location of floor openings within the core will reduce costs. Repeating the core framing pattern on as many floors as possible will also help to minimize the overall costs.

Core Areas

Core areas for elevators and stairs are notoriously cost-intensive if formwork economies are neglected.
In extreme cases, the core alone may require more labor than the rest of the floor, on a per-foot basis. Formwork economy here is achieved through a simplification strategy: eliminate as much complexity from the core configuration as possible. The core will cost less to build, if the design follows the principles listed below and illustrated in Figure 27:

- The shape is symmetrical, rectilinear, without acute angles.
- The number of floor openings is minimized.
- Floor and wall openings are constant in size and location within the core.
- The core framing pattern for walls and floors is repeated on as many floors as possible.



Columns

Although the greatest costs in the structural frame are in the floor system, the cost of column formwork should not be overlooked. Whenever possible, use the same column dimensions for the entire height of the building. Also, use a uniform symmetrical column pattern with all of the columns having the same orientation. Planning along these general lines can yield maximum column economy as well as greater floor framing economy because of the resulting uniformity in bay sizes.

Use the same shape as often as possible throughout the entire building. Square or round columns are the most economical; use other shapes only when architectural requirements so dictate. Columns must be sized not only for adequate strength but also for constructability. For proper concrete placement and consolidation, the engineer must select column sizes and reinforcement to ensure that the reinforcement is not congested. Bar laps splices and locations of bars in beams and slabs framing into the column must be considered. Columns designed with smaller number of larger bras usually improves constructability.

Concrete is more cost effective than reinforcing for carrying compressive axial loads; thus it is more economical to use larger columns sizes with lesser amounts of steel.

Reuse of column forms from story to story results in significant savings. It is economically sound to use the same size column or the entire building and to vary only the logintuduical reinforcement and concrete strength.

Walls

Wall Thickness Trade-offs must be evaluated when designing wall thickness. Reasons to maintain constant wall thickness include repetitive use of standard forms, tie lengths and hardware. Reasons to change wall thickness include accumulating load. When wall thicknesses are changed, incremental steps of 2" or 4" are most efficient. Further, steps should be designed only on the wall face that intersects the horizontal framing. (Figure 31) It is more efficient to step-in formwork toward an opening or building edge than to step formwork away from these conditions.

Use the same wall thickness throughout a project if possible; this facilitates the reuse of equipment, ties, and hardware. In addition, this minimizes the possibilities of error in the field. In all cases, maintain sufficient wall thickness to permit proper placing and vibrating of concrete. Wall openings should be kept to a minimum number since they can be costly and time-consuming.

A few larger openings are more cost-effective than marry smaller openings. Size and location should be constant for maximum reuse of formwork.

. Brick ledges should be kept at a constant height with a minimum number of steps. Thickness as well as height should be in dimensional units of lumber, approximating as closely as possible those of the masonry to be placed. Brick ledge locations and dimensions should be detailed on the structural drawings.

. Footing elevations should be kept constant along any given wall if possible. This facilitates the use of wall gang forms from footing to footing. If footing steps are required, use the minimum number possible.
. For buildings of moderate height, pilasters can be used to transfer column loads into the foundation walls. Gang forms can be used more easily if the pilaster sides are splayed as shown in Fig. 9-9.

Guidelines for member sizing
-for a continuous beam keep beams size constant and vary the reinforcement from span to span.
-wide flat beams are easier to form than deep beams

-spandral beams are most cost intensive than interior beams due to their location at the edge fo the floor slab or at a slab opening.

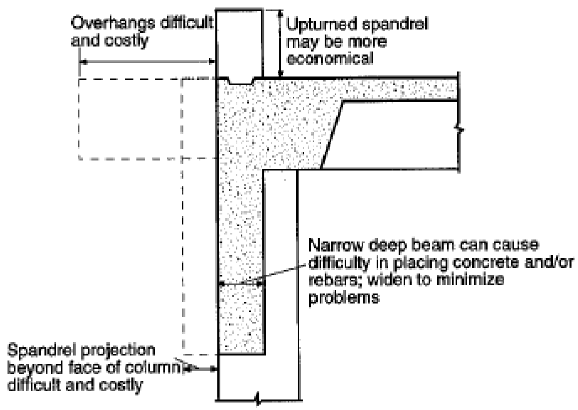


Figure 9-5 Spandrel Beams

Beams should be as wide or wider than the column into which they frame, in addition to formwork economy this also alliavtes some of the reinforcement congestion.

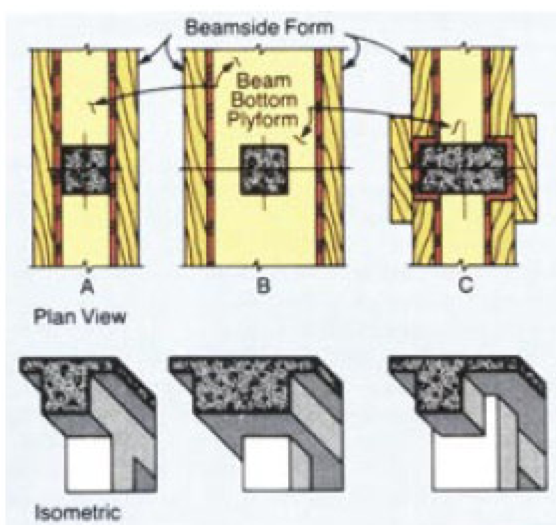


Fig. 5: The extra steps it takes to form out the column with a narrower beam[5]

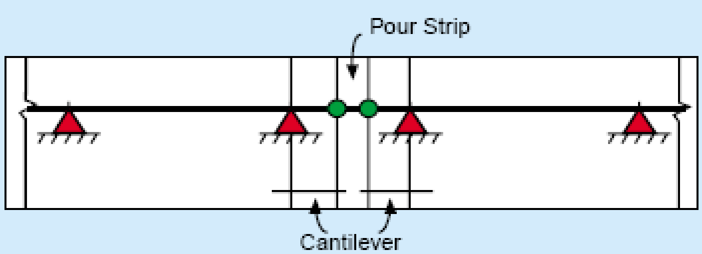
OVERALLSTRUCTURALECONOMY

While it has been the primary purpose of this chapter to focus on those considerations that will significantly impact the costs of the structural system relative to formwork requirements, the 10-step process below should be followed during the preliminary and final design phases of the contraction project as this will lead to overall structure economy:

1. Study the structure as a whole.
2. Prepare freehand alternative sketches comparing all likely structural framing systems.
3. Establish column locations as uniformly as possible, keeping orientation and size constant wherever possible.
4. Determine preliminary member sizes from available design aids
5. Evaluate the sketches and make rough cost comparisons. Consider consulting a formwork office about economic variables relating to formwork, which in turn may influence the basic structural system.
6. Select the framing scheme which best seems to balance structural and aesthetic objectives with economic constraints.
7. Distribute prints of the selected framing scheme to all design and building team members to solicit suggestions that may reduce future changes.
8. Refine the design, placing emphasis on aspects with the greatest economic impact on structural frame cost.
9. Visualize the construction process and the resultant impact on cost.
10. Establish specifications that minimize construction cost and time by including items such as early stripping time and acceptable finish tolerance.

Where pour-strips are used (time-delayed pours to allow for shrinkage in long or posttensioned structures) the backshoring condition may be avoided by designing the slabs adjacent to the pour strips as cantilevers. The pour-strip is designed as simple span, as in Figure 39.

FIGURE 39 POUR STRIP



Designing a pour strip as a simple span supported by cantilevers, avoids costly backshoring condition.

Construction Joint Location

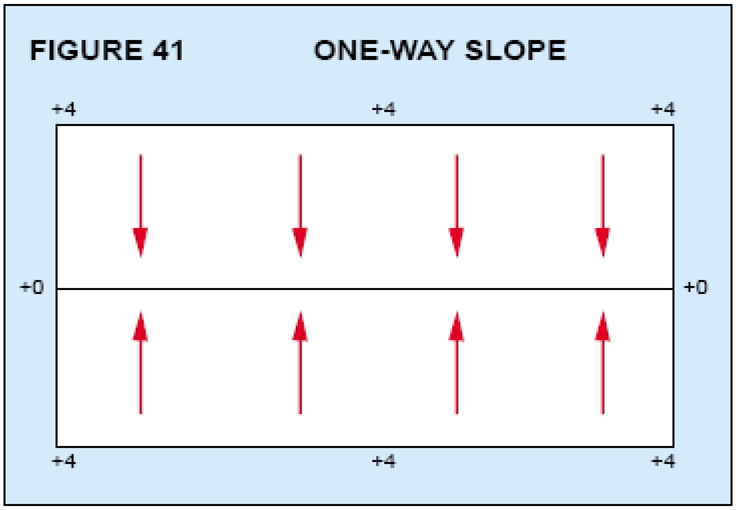
A concrete structure normally is built in progressive stages. (Figure 44)However, to facilitate high-production recycling of equipment and manpower, some latitude in the precise location of construction joints (Figure 45) is desirable. The permissible locations for construction joints should be indicated on the construction drawings, to save time on the job and help ensure a quality structure. The contractor may then select the most efficient sequencing for the construction method to be used. The designer should approve all construction joint locations prior to commencement of the work. Once established, these locations should be communicated to all parties involved in formwork, concrete and reinforcement.

Permanent Slopes

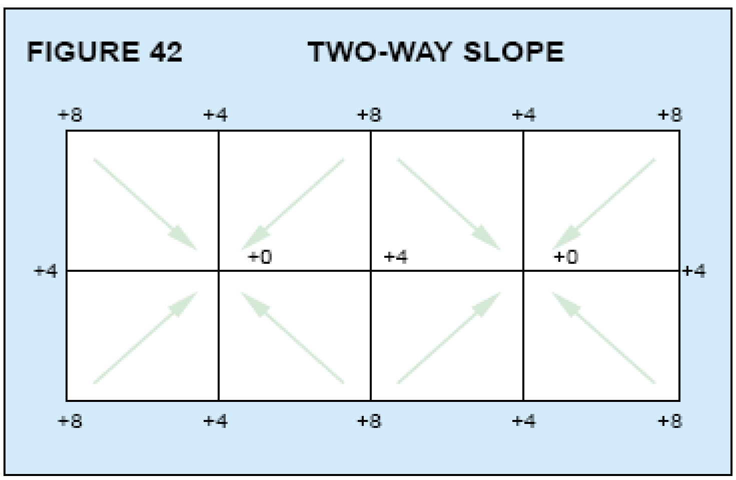
For Drainage Four methods are available to design sloped surfaces (typically for drainage).

a. Top-surface slope — Much preferred due to its considerably lower cost, this method maintains a constant soffit elevation and consequently, is faster to form. It is achieved either by varying slab thickness or with fills. This slope method and method (b) below may require a higher-quality roof membrane than other roof designs. But even with its added cost, the total cost of these methods is much less than methods (c) and (d) below.

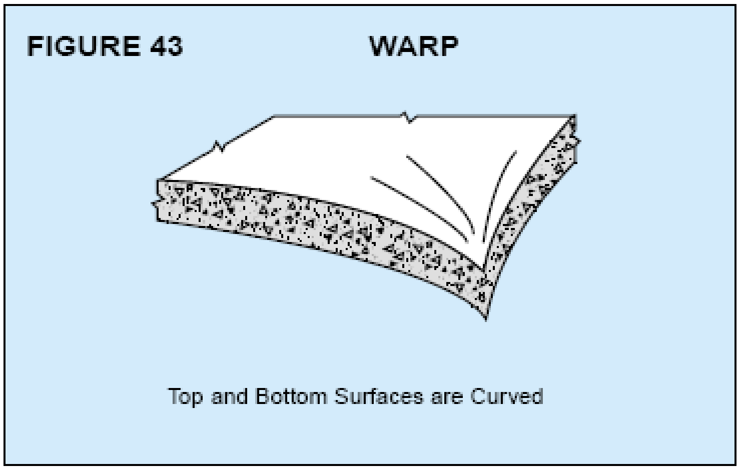
b. One-way slope — top and bottom surfaces fig 41— To reduce deadload and save permanent materials, bottoms of slabs may be sloped to parallel the top. This is more costly than method (a). Positioning the deck at varying elevations is labor-intensive. (Beams should also be sloped to parallel the slab, to avoid variable beam depth.)



Two-way slope—top and bottom surfaces (Figure 42)—This design is an extreme-cost option and almost always can be avoided. With ridges and valleys running in two directions, two-way sloping impedes formwork productivity, with stop-start disruption at each change of slope direction.



Warps (Figure 43)—Of all slope designs, warps are the most extreme impediment to formwork productivity. Forming the curved surfaces requires intricate, expensive carpentry and precision installation. If at all possible, alternative designs should be considered instead.



Camber to Offset Floor Deflection

Typically, cambered slabs are not structural necessities, sufficient stiffness can be designed into floor framing systems to keep deflection within tolerances. This also avoids forming costs associated with camber. If camber is a design imperative, it may be specified much like the sloped surfaces previously discussed: as oneway, two-way, or warped. Again like slopes, costs are progressively higher as complexity increases, with warps at the extreme.

Stripping the formwork and falsework from a concrete Deck

Stripping the formwork under slabs is an issue close to every builder's heart because it directly affects when other trades can get onto the floor that is blocked up with falsework and start hanging the various services Considerations that may influence the timing of the strip are:

Are there any unusual heavy loads such as materials stacking or erection cranes that may require support in future construction works? It is sometimes more economical to leave formwork and false-work in place a little longer and strip out in one hit?
Will floors over need to be carried by the false-work and is there a set stripping and re-shoring procedure? If this is the case it is the responsibility of the design engineer to provide sketches showing the minimum amount of shoring required on each floor and the timing of the stripping operation. This information is normally requested by the managing contractor and is influenced by his site management needs. If the projects structural engineer wants to charge you for the advice we suggest you point them to AS3610 section 2.3 and ask when they stopped complying with code requirements.
Will any scheduled building activities be physically in the way of the stripping operation (blocking access etc)? We have seen money wasted on high strength concrete where the stripping could not be completed due to blocked access ways.
Will cranege and manpower be available at the time to take advantage of an early strip time? Systems such as Table-forms will need a crane booked and ready.
Is the formwork needed elsewhere on site or at another site? Sometimes very competitive formwork rates are contingent upon a specified cycle of form reuse.
Is it cost effective to spend the money on higher strength concrete to allow early stripping? This question may be a function of project programming, staff salaries, early finish incentives and/or liquidated damages.
Is the slab stressed? Stressed slabs can often be stripped earlier as concrete strengths form the only criteria if the future loads are within the floors capacity. We have allowed complete stripping of single storey stressed car park decks in four days using high early strength concrete. On a job we did in Asquith in October 2002 we allowed stripping of each floor of a multistorey warehouse 5 days after each floor was poured. The trick is ensuring that the site cured cylinder strengths are truly representative of the concrete in the deck (but that is another storey).
How stiff is the reinforced concrete slab? If the slab is conventionally reinforced it would normally need to be supported after the concrete has reached its design strength because if the props are taken away too early the slab will deflect beyond recommended limits. This additional deflection is known as creep deflection and is more pronounced when the concrete is relatively "green". Where normal class early age strength concrete is used with reasonable stiffness parameters, guidance can be taken from AS3600 (2001) Tables 19.6.2.4 and Table 19.6.2.5.

RETAINING WALLS:

1. Approximate thickness h/10-14
2. For cantilever sheet pile retaining walls, the penetration below the bottom should equal approximately the unsupported height above.
3. T-shaped with horiz. fill: Footing length
    - 0.46*height with 1/3 in front of exposed face.
4. L-shaped with horiz. fill:
    - Footing length ~ 0.65*height with all of footing at toe.
    - Footing length ~ 0.55*height with all of footing at heel.
5. Retaining walls should be attempted with "traditional" dimensions first, and make every effort to correctly size and balance the heel and toe. There are good reasons why these shapes (toe to heel from 0.45 of height to 0.55, etc) are so commonly found. Stability, sliding, etc are easy to satisfy with an oversized heel or toe, but the strength of these members will be very difficult to achieve.
6. Refer masonry design for joint spacing

SLAB ON GRADE

Typical application	Rating of subgrade	Minimum thickness of pavement
Domestic	Medium good poor	100mm (4") 125mm (5")
Commercial/Institutional/Barns up to 5kPa	Medium to good poor	130mm (5") 150mm(6")
Industrial/gas stations/garages up to 20kpa	Medium to good poor	180mm (7") 200mm (8")

1. With the following reinforcement:
- 100mm-one layer SL72 WWF 6x6-W1.4xW1.4
- 125-150mm- one layer SL82 WWF 6x6-W2.9xW2.9
- 180mm- one layer SL92 WWF 6x6-W2.9xW2.9
- 200mm - two layers (SL72) WWF 6x6-W1.4xW1.4
2. Joints spacing:
- Rule of thumb is 24 to 36 times the thickness (gives the spacing in mm/inches).
- Keep the thickness variation to -5mm (-1/4") / +10mm (3/8"), and the subgrade as flat as possible with no abrupt change greater than 15mm (1/2") in about 1.2m (4'). This keeps the "sudden" restraint potential down.
3. For slabs 180mm greater a crack initiator is suggested.
4. No greater than 21m (70') between Dowel sawn joints
5. No greater than 8m (27') between sawn joints
6. Make your joint pattern as square as practicable and with no more than a 2 to 1 ratio of length to width.
7. Always check with client as to preferred Joints system
8. For small-medium projects
   Max area in pour: 600m^2 (60000 sq ft)
   Max joints spacing for external expansion joints 40-50m, internal stop pour slab joints: 60m (200').
9. For large project
   Max area in pour: 800m^2 aim for 600m^2 (60000sq ft)
   Max joints spacing for external expansion joints 60m (200'), internal stop pour slab joints: 80m (250')
10. Joints that transfer 75% or more of the design load can have the slan surround treated as interior.

11. Slabs should be design based on there 90 day flexural strength and specified this way.
12. For uplift loads a rule of thumb we used to use 10 times the thickness of the slab in each direction for load contributing to hold down.

- Design is very dependent on types of forklift, get spec if possible.

Typical forklift

Capacity/type	axle
1 ton forklift	2.75xcapcity
2-3 ton forklift	2.5x capacity
4 ton forklift	2.3 x capacity
1-1.5 stockpicker	1.3 capacity
2-3 ton pallet truck	.7 x capacity

usage	CBR	Sub base thickness
Internal Commercial/industrial	1.5to3 3 or more	100mm 50m cracker dust (100mm H sites with rolled smooth stone finish)
External LL<5kpa LL >5kpa	All 1.5to3 3 or more	100mm 150mm 100mm

STEEL BUILDINGS (NON-COMPOSITE)

Choice of beam system

Element Typical Span/depth Typical Span (m)	Span/depth	Maximum span
Purlins	32	12m
Roof beams (light dead loading)	18-30	6-30m
Floor I beams and joists	18	18m
Floor Plate girder	10-12	25m (80')
Floor Joist (steel only)	17	6-9m (20-30')
Castellated UB's*	14-17	12-20m(40-70')
Transfer beams	10	6-30m
Trusses supporting floors	10	6-30m
Roof trusses (pitch>20)	14-15	17m (55')
Space Frames	15-30	100m (300')
Primary beams (supported by columns)	10-15	12m (40')
Secondary beams (supported by other beams)	15-25	10m (33')
Portal frame leg	35-40	60m (200')
Simple span rafter	24	30m (100')
Simple span roof beam	15	25m (80')
Con't beam or joist	.85 * simple span value	
Column SHS	H/D 20-35	2-8m

* Avoid if high point loads; increase Ireq by 1.3

As a rule of thumb, the defection of a castellated beam is about 25% greater than the deflection of an equivalent beam with the same depth but without web openings.

Columns:

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

Expansion joints

- Steel industrial buildings 100 m typical–150 m maximum c /c.
- Steel commercial buildings 50 m typical–100 m maximum c /c
- Steel/Tilt-up building 50m typical
- Steel roof sheeting 20 m c/c down the slope, no limit along the slope.

1. Deeper is cheaper. All other things being equal, a deeper wide-flange beam will be more economical than a shallower one. There are exceptions to this rule, but it is generally correct.
2. One common misconception is that all steel members have an actual depth equal to their nominal depth.
3. The braces may be designed for a capacity of 2% of the force resisted by the compression portion of the beam.
4. Cantilever "one-to-three".

- Cantilevered steel beams are commonly used to support architectural features, giving the illusion of an unsupported or "flying" edge. How far is too far for a cantilever to stick out? Generally speaking, if a cantilever exceeds 1/3 of the total back span, economy is lost and may lead to design difficulties. So if your beam has a 9m (30’) back span, try to keep an adjacent cantilever to less than 3m (10’) long.

5. TRUSS:

An optimal depth/span ratio for a planar truss is approximately 1/10. Although forces in the CHORDS decrease with increasing depth, forces in the WEB are practically UNCHANGED and increasing the depth increases the lengths of these members. Approximately half the web members are in COMPRESSION and increasing their lengths reduces their efficiency due to the increased susceptibility to BUCKLING.

6. Modeling:

It is normally ok to assume 10%-20% column stiffness for base plate in calculations for deflection.

7. Struts and ties

Slenderness limits:

- Members resisting load other than wind: 8#180
- Members resisting self weight and wind only: 8#250
- Members normally acting as a tie but subject to load reversal due to wind: 8#350
- Minimum strut size is 89CHS or 75 SHS.

8. Portal Frames-UB and WB

- Haunch length = span / 10-15
- Haunch depth = rafter depth (same section)
- Minimum rafter slope = 2.5O
- Rafter depth = span / 60

RIGID FRAME ANALYSIS APPROXIMATIONS; The following "Rules of Thumb" are useful in determining preliminary sizes for Rigid Moment Frames resisting Lateral loads. They are based on the traditional "Portal Frame" approach modified from the authors' experiences with "real" frames.

Interior Columns at Roof

Interior Columns Not at Roof

The moments in beams framing into exterior columns are half of the above

Values

$$M_{col} \approx \frac{1.2H}{2} \bullet \frac{V_{story}}{n_{col}}$$

$$M_{beam} \approx M_{col}/2$$

$$M_{beam} \approx M_{col}$$

9. ***Tall buildings***

STORIES	LATERAL LOAD RESISTING SYSTEM
<30	Rigid frame
30 to 40	Frame - shear truss
41 to 60	Belt truss
61 to 80	Framed tube
81 to 100	Truss - tube w/ interior columns
101 to 110	Bundled tube
111 to 140	Truss - tube without interior columns

End rotation of a simple beam = 0.2 radians

Deflection of simple span beam (reduction due to connections) = 80% of calculated

Roof Framing Systems

For Cantilevered or continuous roof

10. Avoid over-welding:
    - A weld never needs to exceed the connected part strength.
    - Excessive welding can cause serious problems (distortion, cracking, etc.). This can lead to expensive repairs or even rejections.
    - Select fillet welds over full-penetration groove welds when possible.
    - Small and long fillet welds are more economical than large and short welds.
    - Keep weld sizes at 6mm. or less for fillet welds (accommodates a single pass); increase length if needed.

11. Bolts work in most connections.
    - Specify snug tight bolted joints, rather than bearing joints.
    - Don't just fill up beam webs with bolt rows. Use the appropriate number of rows for strength requirements.
    - Provide for tolerances. Use oversized, short-slotted, and long-slotted holes in bolted connections if permitted for connections to concrete, and leave extra space for welded connections.

12. Use single-sided connections, shear tabs, or single angles wherever possible.
13. Orient columns in moment frames so that moment connections are to the column flanges whenever possible.
14. Show connection concepts in sufficient realistic detail to accurately depict what the finished connection may look like. Make embedded plates a minimum 6 in. to 8 in. larger than required for connections as a rule of thumb. Field fixes for embedded plates that are mis-located are time-consuming and expensive.

15. Rule of thumb: The more pieces there are in a connection detail, the more expensive it is to fabricate and erect.
16. Do not over-economize connections. If the overall connection configuration is virtually the same, reducing the amount of weld or bolt count in a single non-repetitive connection, by even a large percentage(e.g., in excess of 25% to 30%), will probably increase the overall time and expense of the project. Repeating connections will reduce connection design, detailing, layout, fabrication, and erection costs due to the reduced learning curve.

17. When showing stiffeners or other plate material, use popular flat bar sizes
    - Select member sizes with sufficient depth to provide reasonable connections.
    - Use heavier columns to eliminate stiffener plates and/or web doubler plates at moment connections if possible.
    - Standardize member sizes as much as possible. Steel may often be purchased at lower costs in bulk quantities. If a mill order is required, there may be a minimum order.
    - Do not reinforce beam web penetrations if not absolutely necessary.
18. Review member sizes for connection economy:
    - Preferably, a supporting beam should have at least the same depth as the supported beam.

19. SUMMARY FOR ECONOMIC FABRICATION

The rate of erection of steel in a structure is controlled by five main factors:

(a) Connection simplicity
(b) Number of members
(c) Number of bolts and/or amount of field welding
(d) Size and efficiency of erection crew, and the equipment at their Disposal
(e) Timely supply of steel.

20. The use of the transportation length may have to be curtailed to avoid damage during transportation try to keep member lengths less than 15m.
21. One rule of thumb for fillet welds on both faces opposite each other is to make the gusset thickness twice the weld size.

22. Simple connections - use grade 8.8, 20mm diameter bolts fin plates} t = 8mm for UB's < 457mm deep partial depth end plates} t = 10mm for UB's > 457mm deep web cleats}
23. Moment connections -use grade 8.8, 20mm or 24mm diameter. Assume end plate thickness equal to bolt diameter (25 thick with M24)
24. Holding down bolts - assume grade 4.6 where possible.
    Standard sizes: M16 x 300
    M20 x 450, 600
    M24 x 450, 600
    M30 x 450, 600
    M36 x 450, 600, 750

25. Welded

Use 6mm fillet where possible.
Relative costs: 6mm fillet in down hand position 1.0
6mm fillet in vertical position 2.0
6mm fillet in overhead position 3.0
For each additional run multiply above by 1.75.
Note: 6mm weld 1 run
Single butt weld in 10mm plate 6.0
For each 5mm of plate thickness multiply above by 4.0.

One pair of stiffeners with fillet welds = 200 lbs of steel
One pair of stiffeners with full penetration welds = 400 lbs of steel
One pair of Web Doubler Plates = 550 lbs of steel
One web doubler plate = 280 lbs of steel
One column splice = 500 lbs of steel

Maximum and minimum induced camber

	Lentgh of section				
Section nominal depth	30 to 41.9	42 to 51.9	52 to 64.9	64 to 84.9	85 to 100
Wshapes 24 and over	1 to 2	1 to 3	2 to 4	3 to 5	3 to 6
Wshapes 14 to 21 and s shapes 12in and over	¾ to 2 1/2	1 to 3	2 to 4	2.5 to 5	?

COMPOSITE BEAM SYSTEMS

scheme	Span/depth	Max span
Simple construction with Rolled sections	20 -28	10.5m primary 18 secondary
Fabricated sections	15-25	12m
Haunched	25 support 32 mid-span	15m
castellated	17-20	16m
Composite truss	12-16	15m

Always camber yur beams ot account for extra dead weight of wet concrete.
Composite:
Your camber is controlled mostly by the pre-composite deflection (self weight of beam + construction load + wet weight of concrete). The idea is to camber the beam enough such that it does not deflect (sag) due to the wet weight of concrete. It is important because if the beam sags under the wet weight of concrete, then you will need to add more concrete to keep the floor level, and it is a vicious cycle. Typically, beams are cambered to around L/360 before bumping up the member size.

Cambering a 40' beam to equate the dead load deflection does not void the L/240 requirement. It only means that you don't have to include the dead load deflection (which will now be ZERO) in computing total deflection. The total deflection will now be only due to the live and superimposed loads.

For maximum structural efficiency:
L secondary/L primary = 4 / 3
Rule of Thumb:

Fabricated beams
Castellated cellular beams (not very good for point loads):
Span/depth < 20
Castellated beams, hole = 0.67D
centres = 0.72D
Cellular beams holes diameter = 0.6D - 0.8D
centres = 1.1 -1.5 diameter
Wher D is the depth of beam
Haunched beams (use as part of frame action):
D = midspan depth
Span/depth ≤ 35 (span/depth including slab = 26- 28
Maximum overall depth at haunch = 2D
Haunch length typically 7 - 10% of span
Tapered beams:
D = midspan depth
Span/depth 15 - 25
Depth at support = 0.5D or maximum taper of 6 degrees
• Openings in beams (non-seismic applications)
Geometrical constraints :
- Limit unstiffened openings to 0.6D depth by 1.5D length
- Limit stiffened openings to 0.7D depth by 2D length
- Space > D apart
- Ideally positioned between L/5 and L/3 from support for beams with UDL
- Position > D from any point load
- Position > 2D (or L/10) from support

- Openings should ideally be located mid-height. If not, the depths. Fabricated beams
Castellated cellular beams (not very good for point loads): hole = 0.67D centres = 0.72D Cellular beams holes diameter = 0.6D - 0.8D centres = 1.1 -1.5 diameter
Haunched beams (use as part of frame action): D = \ Maximum overall depth at haunch = 2D Haunch length typically 7 - 10% of span

• **Openings in beams (non-seismic applications)**
Geometrical constraints :
- Limit unstiffened openings to 0.6D depth by 1.5D length
- Limit stiffened openings to 0.7D depth by 2D length
- Space > D apart
- Ideally positioned between L/5 and L/3 from support for beams with UDL
- Position > D from any point load
- Position > 2D (or L/10) from support
- Openings should ideally be located mid-height. If not, the depths of the upper and lower sections of web should not differ by more than a factor of 2.