Section 12

Building Construction and Equipment

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12.1 INDUSTRIAL PLANTS

by Vincent M. Altamuro

REFERENCES: Hodson, ''Maynard's Industrial Engineering Handbook,'' 4th ed., McGraw-Hill. Cedarleaf, ''Plant Layout and Flow Improvement,'' McGraw-Hill. Immer, ''Materials Handling,'' McGraw-Hill. Rosaler, ''Standard Handbook of Plant Engineering,'' 2d ed., McGraw-Hill. Merritt and Ricketts, ''Building Design and Construction Handbook,'' McGraw-Hill.

PURPOSES

Industrial plants serve many **functions**. They can:

1. Protect people, products, and equipment from the weather.
2. Preserve and conserve energy.
3. Condition the inside environment to be suitable for the processes, products, and people engaged therein.
4. Protect the outside environment from any fumes, dust, noise, or other contaminants their processes produce.
5. Provide physical security for their contents.
6. Block access, visual and/or physical, of those not authorized to see inside or enter them.
7. Provide the stable, strong, and smooth platform or surface required for operations.
8. Be the frameworks for the distribution networks of the services needed—electric power, lights, fuels, compressed air, gases, steam, air conditioning, fire protection, water, drainage piping, communications.
9. Support the cranes, hoists, racks, and other lifting and holding equipment attached to their superstructures.
10. Be integral parts of equipment (such as drying ovens) by having one or more walls serve as sides of them, and the like.
11. Be safe, pleasant, and efficient places to work for employees, impress visitors, and reflect positively on the company.
12. Fit the surroundings, blend in, be aesthetically attractive, make architectural statements.

This list of design criteria can be expanded to include functions other than those cited and which may be specific to a given proposed plant.

After a plant's expected present and future activities and contents are defined, it is designed to provide the desired functions, and then built to provide spaces for operating personnel and to house equipment and services. An existing plant considered for purchase or lease must be selected so that it will provide its user a competitive advantage. Generally, the decision to own or rent a plant depends on factors such as the expected life of the project, prudent use of capital, the possible need for early occupancy, the availability of a rentable building that matches the requirements, and the possibility of either a very rapid growth or complete failure of the enterprise. Some **commercial real estate** developers will erect a building to meet the specifications of the future operators of the industrial plant and then lease it to them under a long-term contract, often with an option to buy it at the end of the lease. A new structure to be designed and built must have its specifications clearly established so that it will serve its intended use and fit its surroundings.

The creation of an **industrial plant** involves the commitment of sizable resources for many years. The plant is therefore a valuable asset, both present and future. Its design and construction must be timely and within budget, and once undertaken, the project must, at most, be subject to minor modifications only, and preferably none. Once the structure is in place, it is very difficult or not economically feasible to move or change the plant. Even if not truly irreversible, design decisions can be corrected or changed only with great difficulty and added expense. There is often little difference in the cost of designing a plant and configuring its equipment and services layout one way versus another. The difference arises in the operating expenses and efficiencies, i.e., the wrong way versus the best way.

When designing a plant, one should estimate how long it is expected to last—both economically and physically. When making this determination, one accounts for the expected lives of the products manufactured, the processes and manufacturing methods used, the machines used, and the extant technology, as well as the duration of the market, the supply of labor, and so forth. Plant life and production life should coincide to the maximum extent possible. Major difficulties arise when a plant becomes obsolete and may warrant either abandonment or, at best, major modification for conversion. Often the manufacturing methods employed in production have so far outgrown the original methods that the conversion of an existing plant can not be justified at all.

Obsolescence can be deferred by making the factory versatile, adaptable, and flexible. Possible changes in conditions can be anticipated and the ability to alter the plant can be included in the original design. An industrial plant is said to be *versatile* if it can do more than one thing, if its equipment can switch over and make more than one variation of the product, or if its output can be raised or lowered easily to match demand. It is said to be *adaptable* if it can make other related products with only programming and tooling changes to its equipment, without the need to alter their layout. It has *flexibility* to the degree that it is easy to move its services, machines, and other equipment to change the layout and flow paths, and thereby accommodate different products and changing product mixes. Such a building will have a minimum of permanent walls, barriers, and other fixed features. A plant can be made to be general-purpose or special-purpose, or more one than the other. A general-purpose plant is more generic and can be used for a wide variety of purposes. It is usually easier and less expensive to design, build, and convert to new use and is more salable when no longer needed. A special-purpose plant is designed for a specific task. It is more efficient and can make a lower-cost product, as long as the specifications stay within narrow limits. With modular machines, quick-change tooling, and programmable automation, such as robots, it is possible to gain the advantages of both general- and special-purpose construction in one plant. It can be built with the versatility, adaptability, and flexibility to be configured one way for one set of requirements, then reconfigured when conditions change. This enables the manufacturer to produce the wide variety of products that consumers want, and still make them with the lower per unit costs of a special-purpose plant. Both economy of scale and economy of scope are possible in the same facility.

There are trends in the design of some products which are causing plants to be constructed differently, in addition to the demand for the benefits of versatility, adaptability, and flexibility cited above. Some products are getting smaller: computers, electronic circuits, TV cameras, and the like. For other products, greater manufacturing precision and freedom from processing contamination require the establishment of ultraclean facilities such as clean rooms. These and other trends are causing some plants to be built with the ratio of manufacturing space to administrative space different from that in the past. In some plants with large engineering, design, test, quality control, research and development, documentation, clerical, and other departments, the prior ratios have been reversed, so that manufacturing and warehousing spaces constitute a rather small percent of the total plant.

In addition to the basic building structure and its supporting services, an industrial plant comprises people, raw materials, piece parts, work in process, finished products, warehouse stock, machines and supporting equipment, systems and services, office furniture and files, computers, supplies, and a host of other things. While each is an individual entity, the plant must be designed so that all can work together in an **integrated and balanced system**. For a new facility, the best way to achieve this end is to design it in a progressive manner, going from the general to the specific, with each step based on prior decisions, until a set of detailed specifications is established. The sequence of design decisions is not linear, wherein one is finalized before the next one starts. Rather, it is more like a series of loops, with the output of one feeding back and possibly modifying prior decisions. For example, plant size could influence the site chosen, which then could influence the amount of air conditioning and other services needed, all of which, in turn, require space that could change the prior estimate of plant size. Almost all decisions have an impact on one or more interrelated plant specifications. The design of the product influences the degree of automation, which sets in motion a chain of decisions, starting with the number of employees required, down through the number of toilets, the size of the cafeteria, medical department, parking lot, personnel department, etc., and ultimately even placement of emergency exit doors.

PLANT DESIGN

An industrial plant must fulfill its intended functions efficiently and economically. Its design must consider and account for the basic operating conditions to be served. A detailed discussion along those lines follows.

Prerequisite Data, Decisions, and Documentation

The decision to build or buy an individual plant is made by the company's senior executives. It involves the commitment of large sums of money and is properly based on major considerations: need for additional production capacity; introduction of a new product; entering new markets; availability of new and/or better technology and machinery; building a new plant to replace an existing inefficient one or refurbishing and tailoring the existing plant for new production processes and cycles; relocation to a different area, especially if closer proximity to markets and availability of specialized labor are involved; making products in house which were previously bought for resale. Studies, analyses and projections provide input data to help determine the proper site, size, shape, and specifications of a plant. Economic analyses, technological forecasts, and market surveys are used to justify the investment in a plant and to calculate the approximate amount of money required, break-even point, payback period, and profitability. Detailed product design and engineering documentation includes drawings, parts lists, test points, inspection standards, and specifications. Product variations, models, sizes, and options are defined. Sales forecast data addresses expected unit volumes, seasonality, peaks and valleys, growth projections, and other patterns. In addition to projections, constraints on the plant must also be understood at the outset. These may include location restrictions, budgetary limitations, timing deadlines, degree of mechanization, the type of building and its appearance, limits on its effluents, exhaust fumes, and noise, and other effects on the neighborhood and environment.

Activities and Contents

The determination of the activities and contents of a plant is facilitated by a series of analyses and management decisions. Make/buy decisions determine which items or component piece parts are to be made in the plant and which are to be purchased and stored until needed. Some purchased items are used as received, others need work, (painting, plating, cutting to size). Parts purchased on a just-in-time basis will reduce the amount of in-process storage space required.

Processes are classified as **continuous** (refineries, distilleries, paper mills, chemical and plastic resin production); **repetitive** (automobiles, air conditioners, appliances, computers, telephones, toys); or **intermittent,** custom job shop, or to order (elevators, ships, airliners). Some plants include combinations of these processes when they make several types of products. In such mixed, or **hybrid,** situations, one product and processing method usually dominates, but there are cases where a plant set up for mass production work has a special-order shop to make small quantities of variations of the basic product, such as a vending machine that accepts only foreign coins. The manufacturing methods used in these processes may include casting, shearing, bending, drawing, forming, welding, machining, assembly, etc. (see Sec. 13). **Engineering documentation** used to facilitate manufacturing analysis includes operations analyses, flow process charts, precedence diagrams, ''gozinto'' charts, bills of materials, and exploded views of subassemblies and final assemblies. With computer-aided design (CAD) and other graphics, these documents can often be constructed, updated, and stored in a common database.

These documents help define the operations, their sequence and interrelationships, inspection points, storage points, and the points in the process where materials and parts join others to form subassemblies and the finished product. (See Hodson, ''Maynard's Industrial Engineering Handbook.'') Special characteristics of the operations may be noted on these documents, such as ''Very noisy operation,'' ''Piece parts can be stacked, but after assembly, they cannot,'' and the like. Product movement is determined. People/tools/things must move in an industrial plant. Workers and their tools go to the stationary work in process, as in shipbuilding; the worker and the work go to stationary tools, as in a general machine shop; the work in process goes to the workers and the tools, as in an assembly line. Other combinations of the relative movement of workers, tools, and work in progress are used. (See Cederleaf, ''Plant Layout and Flow Improvement.'')

An industrial plant should be designed to facilitate, not hamper, the **smooth movement of people, material, and information**. The efficiency of flow is also influenced by the layout of the equipment and other contents of the building. A multilevel building implies movement between the levels, which usually takes more time, energy, and expense than having the same activities on one level. Some operations are performed better in high structures, where raw material is elevated and then gravity-fed down through the lower levels as it evolves into a finished product. A plant built into the side of a hill receives material directly into an upper level without the need to elevate it. In most cases, a single-level plant affords the opportunity for the most efficient flow of materials and product. Even a single-level plant should have all its floor at the same elevation so that material moving from one area to another need not go up or down steps, ramps, or inclines. Features of the building such as toilets and utilities should be located where they will not interfere with the most efficient plant layout, and will not have to be moved in relayouts or expansions.

To the maximum extent possible, the **material flow** through an industrial plant should be smooth, straight, unidirectional, and coupled (the output of one machine should be the input to another, without a large cushion of inventory between them); require the least rehandling; be at constant speed over the shortest direct route, with the least energy and expense; and be always directed toward the shipping dock. Raw materials and piece parts should move continuously through the plant as they are converted progressively by a combination of people and equipment into finished products of the desired quantity and quality. Materials must flow with few interruptions, side excursions, or stops, so that they are in the plant for the shortest time possible and thereby facilitate expeditious product completion. Where possible, movements should be combined with operations, such as having a mobile robot or an automatic guided vehicle (AGV) work on (inspect, sort, mark, pack) the item while transporting it. An analysis of the plant's activities, including a list of its contents and an analysis of their relative movements, results in a rational determination of how they must flow through it. This must be done for materials (raw materials, purchased parts, work in process, finished goods, scrap, and supplies), people, data, and services.

Space and Size Calculations

Annual sales forecasts plotted by the month sometimes show peaks and valleys due to **seasonal variations** in demand. A plant with the capacity to produce the highest month's demand has much of its capacity idle or underutilized in the other months. The ideal plant would have a constant output every month, with long production runs of each item to minimize tool changes and setup times. This is not always practicable, for either very high inventory buildups or shortages could result. The compromise is the calculation of a production schedule that is more level than the sales forecast, builds as little inventory as possible without risking shortages, and allows for rejects, equipment breakdowns, vacations, bad weather, and other interruptions and inefficiencies. This will result in a plant with less production equipment and space but with more inventory space (and associated material handling equipment) to accumulate finished product to meet shipping demands in response to sales. Obviously, the warehouse and other storage areas must accommodate the maximum amount of inventory expected to be stored.

The proper size of a plant is determined by **calculating the total space required** (immediate and future) by all its contents: material at all stages of production, people (employees and visitors), equipment (production, materials handling, support, and services) and the ancilliary spaces required (working room, aisles, lobbies). At first, only the **general types of equipment are specified** (spot welders, overhead chain conveyors, forklift trucks, etc.). Later, **specific items are selected** and listed by manufacturer, model number, capacity, speed, size, weight, power, and other requirements. (See Sec. 10.) Finally, one model may be substituted for another to gain a particular advantage. A simplified typical equipment selection calculation follows:

1. Define the operation: Joining.
2. Decide the method: Welding.
3. Note personnel available: Semiskilled.
4. Determine general machine type: Spot welder.
5. Calculate required production output:

Spot Welds Required

	Product			
	A	B	C	Total
Number of spots	120	60	80	
Units per year	10,000	40,000	20,000	
Spots per year, millions	1.2	2.4	1.6	5.2
Spots per day 250 days/year)				20,800
Spots per hour (8 h/day)				2,600
Spots per minute (60 min/h)				44

6. Select a particular candidate machine, considering type of metal, thickness of metal, diameter of spot. From catalog, choose manufacturer (Hobart) and model (series 1500 rocker arm SW-V).
7. Calculate capacity of one machine in minutes per spot.

Operating time	0.03
Material handling time	0.06
Setup time allocated	0.01
Minutes per spot	0.10

8. Calculate the number of machines needed.

$$N = \frac{TP}{60\ HU}$$

where N = number of machines required; T = standard time to perform the operation, minutes; P = production needed per day, operations; H = standard working hours per day; U = use factor—up-time of the machine, its utilization (percent of time it is producing), or its efficiency. Example:

$$N = \frac{0.10 \times 20{,}800}{60 \times 8 \times 0.80} = 5.4 \text{ machines}$$

9. Round off to next higher number: 5.4 becomes 6 machines.
10. Alternatively, return to catalog to see of there is a faster model. If so, 5.4 could become 5 of a different model machine.
11. Record specifications of selected machine on an equipment card or in a computer file. Appropriate notations should be made therein, such as:

Crane must go over this machine.
Must be near outside wall for venting.
Will require supplemental lighting.
Requires a special foundation.
Requires a drain for water.
Allow room on side for gear changes.
Allow room for largest-size sheet-metal parts.

12. Use data for all machines needed to help establish specifications for equippage of the plant:

Sum of space required (including space for workers, material, aisles, services) becomes size of that department or function

Sum of utilities and supplies (electricity, water, steam, compressed air, oxygen, nitrogen, acetylene, coolants, lubricants, air conditioning) addresses requirements for them.

Sum of weights affects floor loading capacities and specifications. For some parameters, the extreme value is important in establishing the building specifications; i.e., the height of the tallest machine may set the required clear height inside the plant, locally or throughout. Equipment mounted on columns or roof beams will influence their size. Specific equipment may require special foundations or subfloor access pits.

Sums of the prices of the machines and the required tooling, the number of workers, their required skills, and their wages and benefits provide information regarding the investment and operating expenses for the plant.

The same procedure is used to determine the **material handling equipment,** its required space, and its influence on the plant's specifications. (See Immer, ''Materials Handling.'') Equipment may be capable of moving materials horizontally, vertically, or in both directions; some have a fixed path of travel, while the paths of others can be varied. The general types of equipment that move materials horizontally in a fixed path include conveyors, monorails, and carts pulled by trucks, dragged by chains in floor troughs, or that follow buried wires. Conveyors are overhead or floor-level type. Overhead conveyors clear the floors of some congestion but add to the loads carried by the building's columns and beams. Other types of conveyors are installed at floor level or at working height, and include belt conveyors and powered or unpowered roller conveyors. The general types of material handling equipment that travel in variable horizontal paths include hand trucks, powered trucks, pallet trucks and automatic guided vehicles. Those that travel in fixed vertical paths include elevators, skip hoists, chutes, and lift tables. Those that travel in both horizontal and vertical fixed paths include traveling bridge cranes, gantry cranes, jib cranes, and pneumatic tubes. Those that travel in both horizontal and vertical variable paths include robots and forklift trucks. Forklift trucks may be powered by either battery, gasoline, or liquefied petroleum (LP) gas. Those powered by batteries will require the installation of charging facilities within the plant, and those with internal combustion engines will require safe fuel-storage space.

The installation of each of the above-listed material handling equipment types will influence the plant's specifications in some way. In computing loads on the building's structural members, all static and dynamic loads arising from material handling equipment must be factored into their design, as applicable.

With respect to **raw material,** its form, weight, size, temperature, ruggedness, and other qualities are considered, as are the expected distances to be moved, speed of transit, number of trips, and amount carried per trip. For **warehouses,** the important considerations are cubic space and density of use for various package sizes, weights, and stacking patterns.

The same basic approach is used for **support functions** and **plant services**. The space required for people can be estimated by listing them by functions, jobs, and categories (male versus female, plant versus office, department, location, shift), then allocating space to each. For example, office floor space allocation in ft^2 (m^2) per person may be: plant manager, from 150 to 300 (13.9 to 27.9) depending on size and type of plant; assistant plant manager, 125 to 250 (11.6 to 23.2); department heads, 100 to 200 (9.3 to 18.6); section heads, engineers, specialists, 100 to 150 (9.3 to 13.9); general personnel, 50 to 100 (4.6 to 9.3). When accounting for all employees in an industrial plant, the space per person can range from about 200 to 4,000 ft^2 (18.6 to 370 m^2), depending on the type of industry and the degree of automation employed. An estimate of the male/female ratio often must be made early in the planning stage, for many localities will issue a building permit only if adequate sanitary and rest facilities are included for each gender.

The space required for each department and function is compiled and added to show the approximate total plant size. See Fig. 12.1.1, which also shows the percent of the total plant floor space allotted to functional subtotals. The size of the plant should not be deemed final without including the manner in which to allow for anticipated growth and/or plant expansion. Anticipated growth, if realistic, must enter into the decisions reached as to the initial size of the plant to be built. When dealing with the initial plant size vis-à-vis any anticipated future expansion, some questions usually posed are: Should it be large enough to accommodate expected growth, knowing that it will be too big initially? Should it meet only present needs, then be expanded as and if required? Should a midsize compromise be made? A plant too big for present needs will cost more to build and operate and may place the firm in an uncompetitive position. Expenses for taxes, insurance, heating, air conditioning, etc. for a partially vacant plant are almost the same as for one fully occupied. On the other hand, a plant built just to satisfy present requirements of a growing business can quickly become cramped and inefficient and may lead to rental of external storage and warehouse space, with all the added expense and loss of control entailed thereby. Any future expansion of the original plant will not only be disruptive to operations, but also will cost significantly more than building it bigger in the first place. The correct choice is made by analyzing all options and comparing expected costs and profits and the realistic probability of growth.

Department or function	Area ft²	Area m²
Main receiving	5,700	530
Incoming inspection	900	84
Main stockroom	9,600	892
Large-component storage	6,000	557
Coil steel storage	1,840	171
Steel coil line	2,360	219
Sheet metal fabrication	18,000	1,672
Welding	3,000	279
Sheet-steel storage	13,800	1,282
Copper tubing storage	600	56
Tubing fabrication	1,200	111
Cleaning and painting	9,600	892
Insulating	1,200	111
Electrical subassembly	1,200	111
Final assembly	33,600	3,121
Finished goods storage	40,200	3,735
Wood storage and fabrication	2,400	222
Shipping	9,600	892
Maintenance and tool room	600	56
Print shop	600	56
Plant offices	1,200	111
Engineering laboratory	12,000	1,115
Reproduction room	420	39
Canteen/eating area	3,600	334
Medical/first aid	330	31
Offices and lobby	21,600	2,007
Toilets	1,650	154
Total	202,800	18,840

Function	% of total plant	ft²	m²
All receiving, incoming inspection, storage, and shipping	44.2	89,440	8,309
All production	35.3	71,360	6,629
Engineering lab and repro	6.2	12,420	1,154
Support areas: Maintenance and tool, print, first aid, canteen, toilets, plant offices	4.0	7,980	741
General offices and lobby	10.3	21,600	2,007
Total	100.0	202,800	18,840

Fig. 12.1.1 Space required by department, subtotals by function, and for the entire facility for a plant designed and built to manufacture roof-mounted commercial air conditions. *(Source: VMA, Inc.)*

If a building is to be expanded or upgraded easily and economically after it is built, that capability must be included at the outset. This is done by deciding in advance the direction and degree of possible future expansion, knowing that if the business grows, not all sections of the plant may have to be expanded to the same degree. After the basic form of the building shape and envelope have been addressed, attention is then directed to other matters: location of the building on the property; materials of construction for internal walls that may have to be rearranged; location of spaces and equipment that will be difficult or impossible to move in the future, such as toilets, shipping/receiving docks, transformers, heavy machinery, and bridge cranes.

Superstructure members, pipes, monorails, ducts, and so on can be terminated at the proposed expansion side of the plant, with suitable end caps or terminations which may be removed easily and activated quickly when required. Installations of major utilities are prudently sized to handle future expansion. If the initial design strategy includes anticipation of vertical expansion via mezzanines, balconies, or additional stories, foundations, footings, superstructure, and other structural elements must be designed with that expansion in mind. To accommodate that requirement at some later date will prove to be prohibitively expensive and will disrupt ongoing operations to a degree unimaginable. By the same token, if future operations require a contraction of active space in the proposed plant, space would be available, at worst, for sublease to others. Regardless of whether possible expansion or contraction is contemplated, the initial plant design should consider both possibilities and seek to have either occur with minimum disruption of production.

Plant Emplacement and Site Selection

The emplacement of a plant is defined by its geographical location, site, position, and orientation, in that order. Geographical location is determined by the country, region, area, state, county, city, and municipality in which the plant is situated. The site is that particular plot of land which can be identified by street address or block and lot numbers in that geographical location. The position and orientation speak to the exact placement and compass heading of the building on the site.

Typical considerations entering each stage of the placement decision, from broad to specific, follow.

Location: Political and economic environment. Stability of currency and government. Market potential. Raw materials supply. Availability and cost of labor. Construction or rental costs. Environmental regulations. Climate. Applicable laws within the governing jurisdiction. Port, river, rail, highway, airport quality. Utilities and their relative costs. Taxes. Incentives offered. Local commercial services. Housing, schools, hospitals, shops, libraries, and other community attractions. Crime rate, police and fire protection. General ambiance of the locality.

Site: Availability. Price. Incentives offered. Building and zoning codes. Near highway entrance/exit, on railroad spur, waterway, and major road. Public transportation for employees. Availability of water, sewer, and other utilities. Topography, elevation, soil properties, subsurface conditions, drainage, flood risk, earthquake faults. Neighbors. Visibility.

Position: Placement for possible expansion with sides of building that may be extended facing an open area or parking lot and the sides not to be extended close to the property line. Proximity to railroad tracks, road, utilities or other fixed features. Positions of buildings on neighboring property.

Orientation: Turned toward or away from winter's wind and summer's sun, as desired, considering the frequently open large shipping and receiving doors. Needs for heating, air conditioning, natural light, color matching, and the like that would be affected by compass heading.

Proximity to competitors may be desirable if the locale has developed into a well-recognized center for that industry, where are found skilled people, and a wide variety of suppliers and supporting services (testing laboratories, local centers of higher learning with faculty available for consultation on an ad hoc basis). Without these conditions, and especially if the product is heavy and expensive to ship, it is desirable to locate close to market areas and distant from competitors.

The amount of land required depends on plant size, the number of employees and their need for parking space, the probability of storing some raw materials and finished goods outdoors, the need to turn large trucks around on the property, the possibility of future expansion of the plant, zoning, and the possible desire to create an open park-like ambiance. Total land area of from 4 to 8 times the plant size is usually adequate; the lower end of the range applies in built-up areas, and the higher end applies in suburban areas. In some locations the land can cost more than the plant.

Before placement of a plant becomes final, all conditions and extremes should be simulated, including winter storms, rainy seasons, floods, several consecutive days of 100°F heat, power outages, labor disruptions, and the like. It is unlikely that the perfect site will be found, thus sought-after attributes must be ranked in order of importance. As an incentive to get new industry, some governments will build roads and schools and reroute buses if necessary. Test borings are often taken at a candidate site before a commitment is made to buy or lease it to ensure that it is suitable. A plot plan (see Fig. 12.1.2) can be made with an approximation of the planned plant on the candidate site indicating land size, topography, drainage, position on the plot, anticipated expansion, compass orientation, location of power lines, railroad tracks, roads, rivers, open fields, neighbors, etc.

Configuration

The configuration of a plant is determined ideally by the optimum layout of its contents. Compromises are usually made, however, often resulting in a conventionally shaped building. A building with several extended branches is very expensive. For a given floor area, a square building requires less total wall length than other quadrilateral shapes, but rectangular buildings are the most common compromise between cost and efficiency. For the lease or purchase of an existing building, the best approach is to design the best layout and then seek an existing building within the desired area in which that layout may be accommodated. For a new building, the layout is set down before the location is selected and then adjusted for the particular features of the site. A layout prepared after the site has been selected can be tailored to fit the site, all the while maintaining the desired features of that layout.

Before the configuration and layout become final, certain broad and tentative choices about the type of building should be made. These include general-purpose versus special-purpose, multilevel versus single level, use of mezzanines and balconies, with windows or windowless, etc. Setting aside considerations of material flow, a multilevel building enjoys the advantages, on a unit floor area basis, of using less land and costing less to build. The columns, however, will generally be spaced closer together, thereby presenting an impediment to desired flow paths and a reduction in options for overall layout. A single-level building, on the other hand, can more easily support heavier floor loads by virtue of its concrete slab on grade floor construction.

The matter of shipping/receiving floor level vis-à-vis truck bed level must be resolved satisfactorily, keeping in mind that loading and unloading ought to be effected with small hoists or forklifts for maximum efficiency. To that end, loading floor level can be built to be flush with

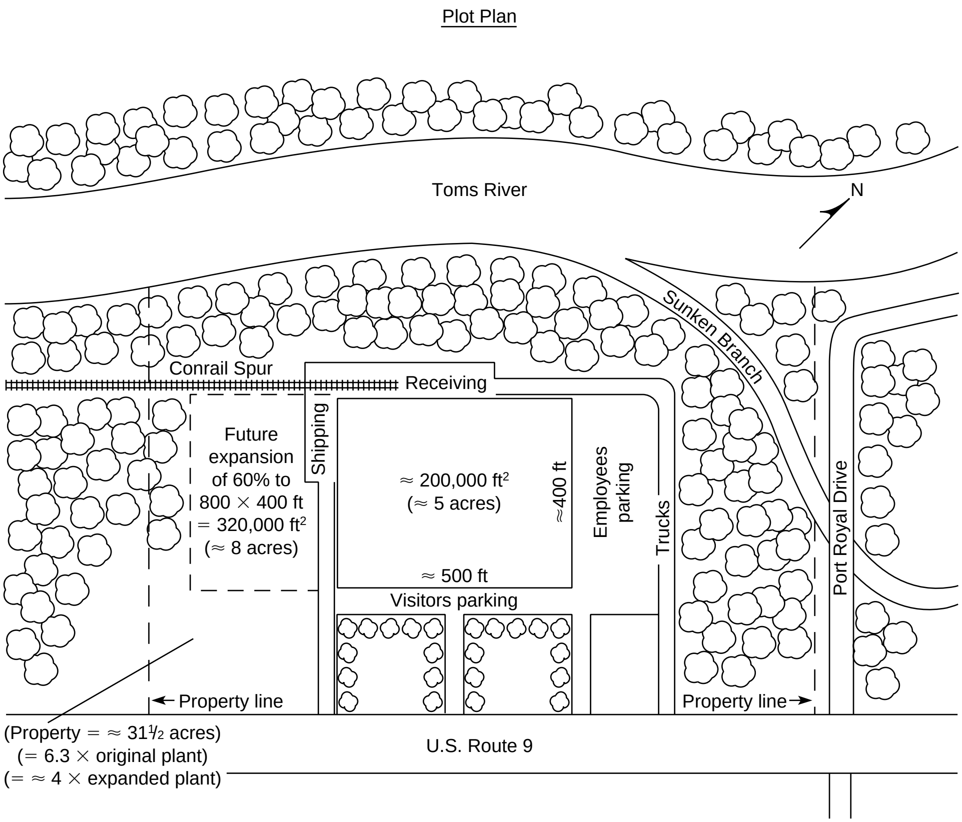


Fig. 12.1.2 Plot plan of an industrial plant. *(Source: VMA, Inc.)*

the bed height of most of the anticipated truck traffic, or the truck parking apron can be inclined downward to align the two. If those cases when trucks with other bed heights have to be accommodated, small demountable ramps can be employed, the same being built sufficiently rigid to permit forklift traverse.

Balconies and mezzanines may be added to gain more storage space, to locate offices with a view of the production areas below, for raw materials or subassembly work that can be gravity-fed and drop-delivered to the operations below, and for the placement of service equipment (hot-water tanks, compressors, air conditioners, and the like). Basements may be included for the placement of heavy equipment, pumps, compressors, furnaces, the lower portion of very tall machines, supplies, and employee parking.

Windows in an industrial plant lower lighting bills, permit truer color matching in natural light, aid cooling and ventilation when opened, may provide a means of egress in case of fire, and may lower fire insurance rates. The advantages of a windowless plant are easy control of the intensity and direction of light (elimination of glare, contrasts, shadows, diffusion, and changing direction); (sometimes) less expensive construction; and less heat transfer, dust and dirt infiltration, maintenance expense to wash and repair glass, and worker distraction; better security; lower theft insurance rates. It is generally easier and more economical to design a windowless building. The absence of fenestration allows more flexibility in interior layouts by virtue of uninterrupted wall space. The absence of low windows, in particular, obviates the need for blinds or drapes to keep the interior private from the passing viewer. Elimination of an enticing target to vandals is not to be dismissed lightly; often damage to machinery and equipment, as well as injury to personnel, results from flying missiles launched at and through windows.

The relative positions and detailed layouts of each department's machines, support equipment, services, and offices are based on analyses of their functions, contents, activities, operations, flow, relationships, and frequency of contacts. The layouts may be product- or process-oriented, or a combination of the two. In a product-oriented layout, machines are arranged in the sequence that the production process requires. This permits the product to advance in a direct path, such as on an assembly line, and with smooth material flow. In a process-oriented layout, the machines are grouped by type, such as a welding or drilling. This requires that the products requiring those operations be brought to that area. It is used where products vary and the output of each is low, such as in a job shop, and allows production to continue even when a machine breaks down.

Another early decision involves the preferred placement of the receiving and shipping departments and the related basic pattern of material flow through the plant. If it is preferred to have the material enter at one end of the plant and exit at the other (Fig. 12.1.3*a* and *b*), then separate receiving and shipping facilities, equipment, and personnel will be required. Capital investment and operating economies ensue from combining these functions, with shared personnel, equipment, supervision, and space, but then the basic material flow will loop back to allow the finished product to exit from the same location where raw materials entered (Fig. 12.1.3*c*).

The drawing showing the size, shape and position of each department or area of an industrial plant is called a **block diagram**. It is developed by constructing a series of **analysis documents**. These include the frequency of relationship chart, proximity preference matrix, relative position block arrangement, sized block arrangement, initial block diagram, refined block diagram, and final block diagram (or simply block diagram). The **frequency of relationship chart** (also called a **from/to chart**) is constructed from an analysis of the engineering and manufacturing documentation and the activities of the plant. It shows the frequency and magnitude of movement, flow, and contacts between the entries. It may be weighted to include the importance of high-priority factors. The **proximity preference matrix** (also called a **relationship chart**) is constructed the same way. It shows (Fig. 12.1.4) which functions, departments, equipment and people should be close to each other, how close, and why, and which should be away from which other and why. It is constructed with a diamond-shaped box at the intersection of every two

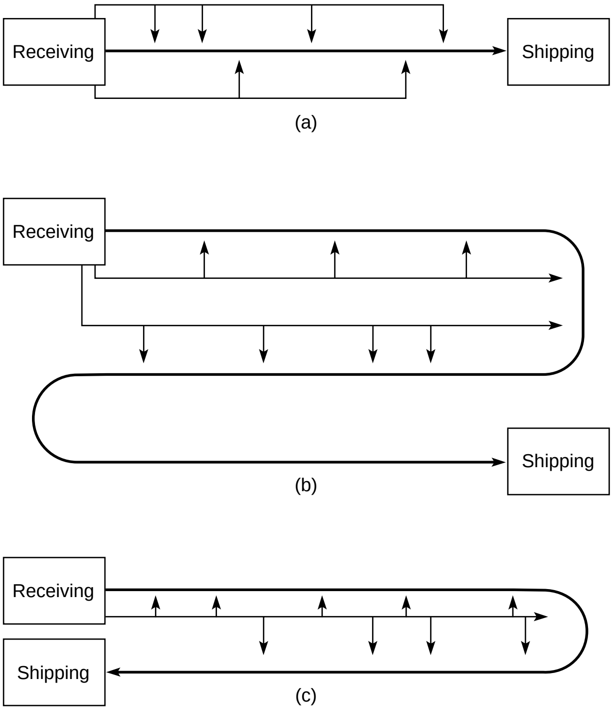
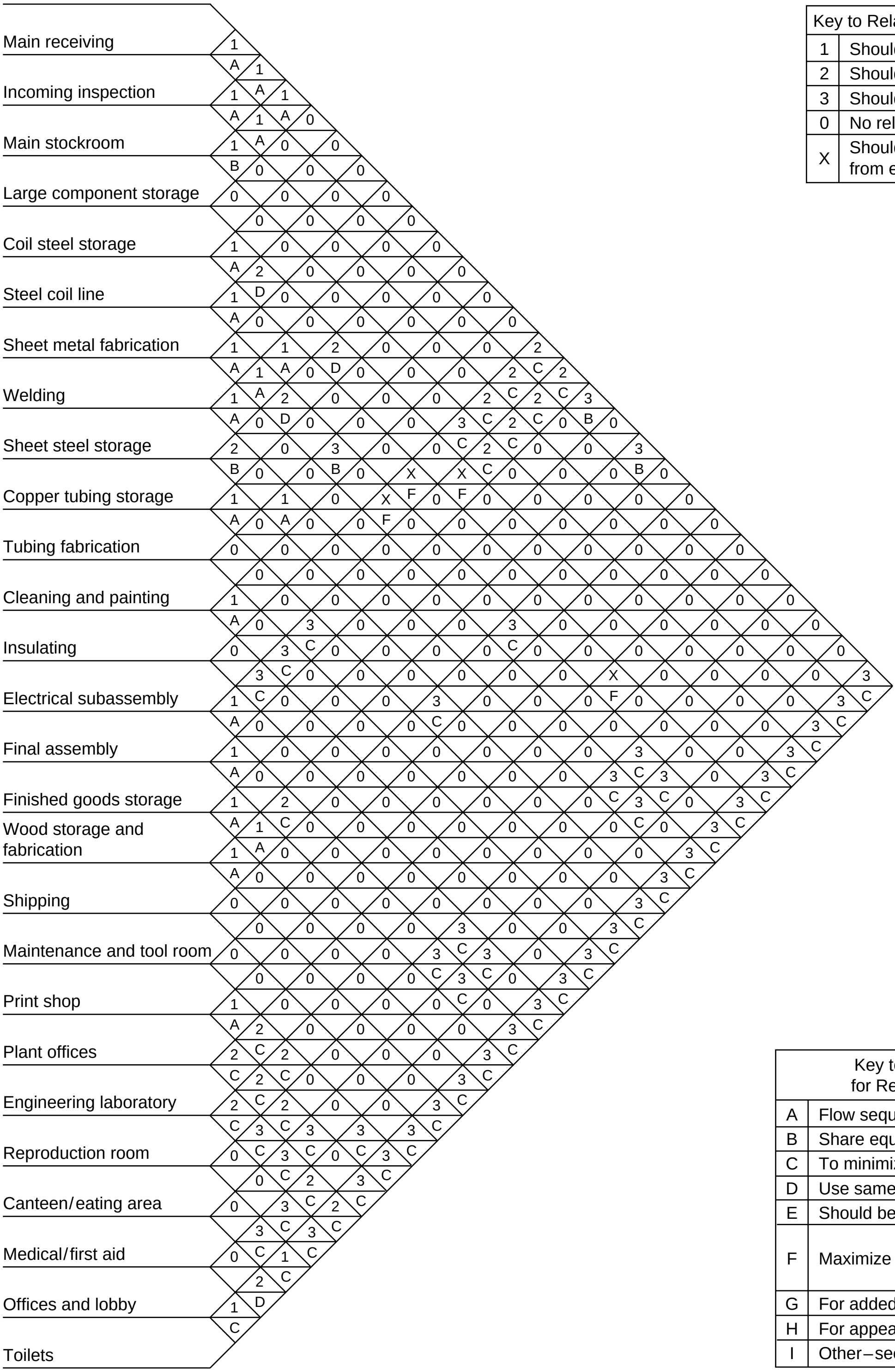


Fig. 12.1.3 Three different relative positions of a plant's receiving and shipping areas. *(Source: VMA, Inc.)*

entries. The notations made in each box show the importance of their need to be either close or separated by a number or code. The reasons can also be shown by entering a code in the lower half of the box, as shown in Fig. 12.1.4. The matrix may be made at the function or department level and, later, at the individual machine or person level. The objective of the matrix and chart is to lay out the plant so that things are as close to (or as far from) other things to satisfy the criteria established, and ensure that those with the highest number of contacts are so located to minimize the time, distance, and energy required.

When this is done, a **relative position block arrangement** (Fig. 12.1.5) may be made; this shows the various arrangements possible by shifting around pieces representing the departments. The arrangement selected is the one that best satisfies the relationships (in decreasing order of importance) and degrees of contacts, as previously determined. The size of each model (or computer graphic representation) is the same because only the relative positions of the departments or areas is of interest at this stage of the design. Assigning a different color or background pattern to each adds to clarity and, when carried through to final and detailed drawings, helps visualize quickly the totals of separated areas; e.g., the total of inventory storage areas spread throughout the plant can be understood quickly if all are shaded the same color on the drawings. The **sized block arrangement diagram** (Fig. 12.1.6) is the relative position block arrangement with the size of each area scaled (but still square) to represent its square footage as determined by its expected contents and room for expansion (unless the expansion is to be handled by extending the building, in which case the department should be placed where expansion is proposed. The **initial block diagram** converts the square representation of each department into a shape that is more suitable for its contents and activities, but of the same square footage. The **refined block diagram** (Fig. 12.1.7) adjusts the initial shapes to effect a compromise between the advantages of having them be the best operational shape and those of fitting them in an economical rectangular building. L-, T-, U-, H-, and E-shaped buildings are often the result of such configuration compromises.

After the main aisles are drawn, as straight as possible, to serve as



Key to Relationship Rating	
1	Should be contiguous
2	Should be very close
3	Should be close
0	No relationship
X	Should be far away from each other

Key to Reasons for Relationships	
A	Flow sequence
B	Share equipment/personnel
C	To minimize travel distance
D	Use same services/utilities
E	Should be on outside wall
F	Maximize distance – noise – vibration – fumes
G	For added safety/security
H	For appearance/image
I	Other–see notes

Fig. 12.1.4 A proximity preference matrix listing the departments, functions, and areas, and ranking how near or far each should be relative to the others, and why. *(Source: VMA, Inc.)*

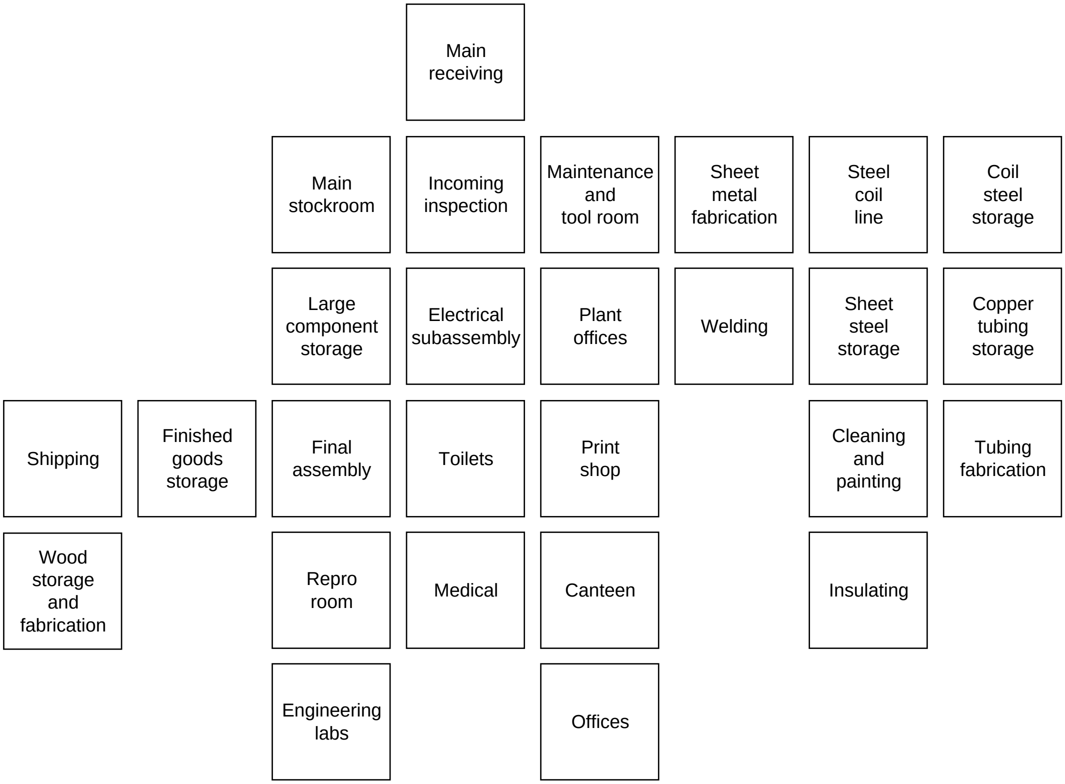


Fig. 12.1.5 A relative position block arrangement that attempts to satisfy and optimize the dictates of the prior proximity preference matrix. *(Source: VMA, Inc.)*

dividers between departments, the size and shape of each area can be fine-tuned, and a **final block diagram,** or simply **block diagram,** is drawn, keeping the same color code scheme used throughout. Detailed layouts are constructed for each department and function to fit within the spaces allocated in the block diagram. When assembled onto one drawing, the block diagrams become the detailed layout for the entire plant.

There are several aids to constructing both block diagrams and detailed layouts. These range from scaled templates to three-dimensional models of machines and equipment. They are available as plain blocks and as highly detailed plastic or cast-metal models. Computer-based optimization programs are also available. Some computer-aided design packages contain libraries of plant components and equipment that can be selected, positioned, rotated, and moved on the screen until a satisfactory layout is achieved. See Figs. 12.1.8 and 12.1.9 for two- and three-dimensional, respectively, computer-generated detailed layout drawings. Arrows are added to these diagrams to show flow paths. Copies of these drawings can be annotated with dimensions and machinery and furniture descriptors and given to vendors and contractors for them to submit bids to supply and install the items. They are also kept on file for use in future maintenance and/or revisions.

Features not expected to be moved in the future or that will become permanent parts of the building should be located first, e.g., stairways, doors, toilets, transformers, steam boilers, fuel tanks, pumps, compressors, piping, and permanent walls. Accurately dimensioned definitive drawings must be made to guide installers of equipment and services.

In addition to the layouts of the production areas, the support functions must also be planned. Support groups assist the production departments; they include research and development, testing laboratories, engineering, production planning and control, quality control, machine shop, sales and advertising, technical literature, purchasing, data processing, accounting and finance, files storage, personnel, medical, administration and management, general office, supply storage, reception lobby, conference rooms, training rooms, library, mail room, copying and reproduction, cafeteria, vending machines, water fountains, washrooms, toilets, lockers, custodial and maintenance, security, and so on. Adequate space must be provided for the people working in these areas and their workstations must be designed with ergonomics, lighting, comfort, and safety in mind.

Many industrial plants have the supervisors' offices situated on the factory floor for better contact with and control of their people. Likewise, offices for quality control inspectors, industrial engineers, and manufacturing or process engineers often are located in the production areas. Such offices may be built with the building or may be purchased and installed as preengineered, prefabricated units. General and administrative offices usually are distant from the plant's machinery and equipment, whose noise and vibrations would otherwise affect the performance of the occupants therein and their ancillary equipment (computers, for example).

Office furniture may be arranged in military style (orderly straight rows), the open plan method (fewer and movable partitions), landscaped (free form with plants and curved dividers), individual offices (glass, wallboard, steel, or masonry walls), or any combination of these. In those arrangements wherein the partitions and dividers are classified as furniture instead of parts of the building, they may be depreciated over a much shorter period of time than the permanent structure. Space is allocated on the basis of position in the organization chart, with the senior executives getting windows (if any) and the most senior getting a large corner office. Floor coverings, ranging from tile to carpeting, again depends on position, as do the amount and type of furniture.

The plant's reception lobby should measure approximately 160 ft^2 (14.9 m^2) if seating for four persons is required, and at least 200 ft^2 (18.6 m^2) for 10 visitors. Add 60 to 100 ft^2 (5.6 to 9.3 m^2) if a recep-

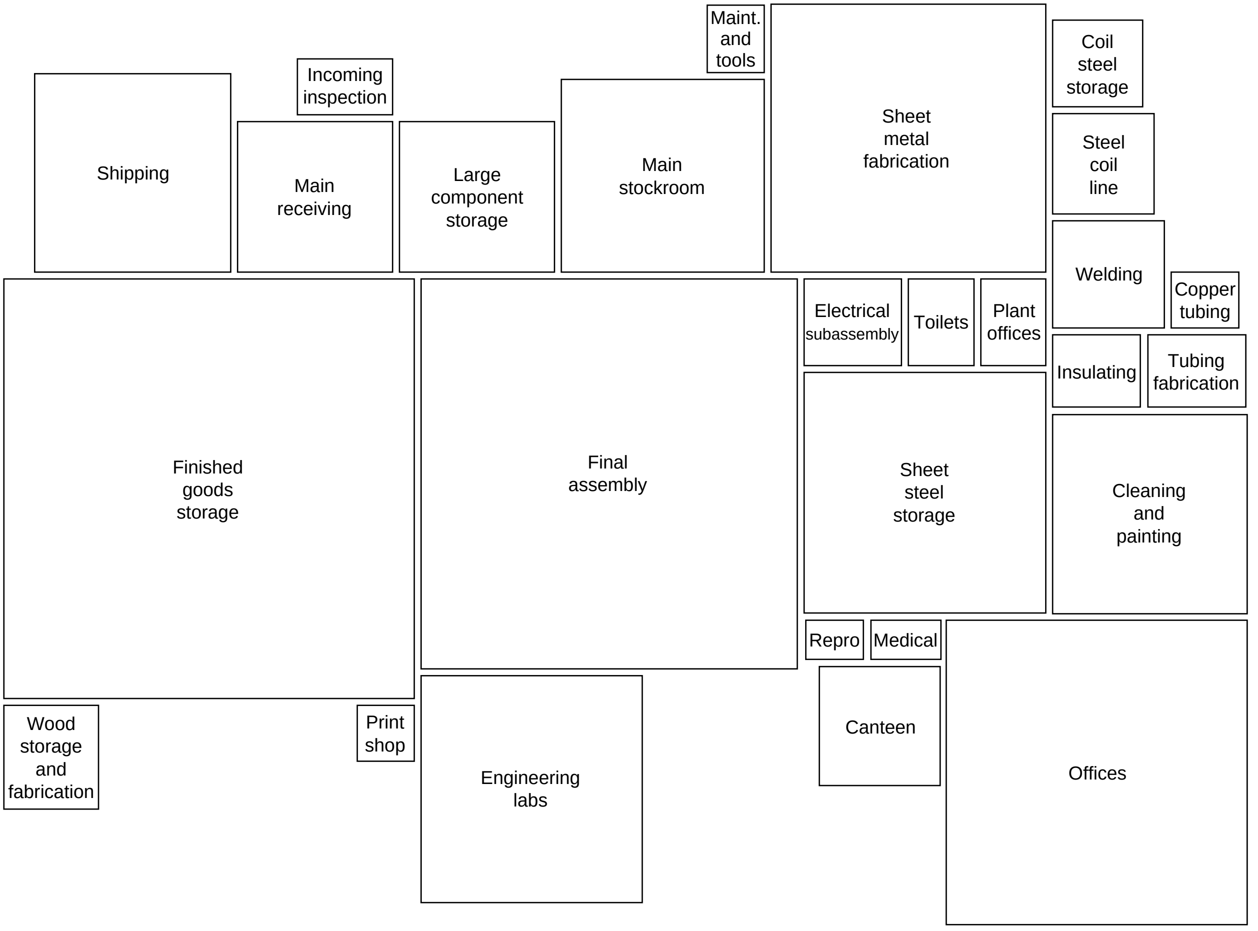


Fig. 12.1.6 A sized block arrangement that converts the prior relative position block arrangement into one that shows the required size of each department, function, and area, while maintaining the preferred relative position of each. *(Source: VMA, Inc.)*

tionist is to be seated there. Cloak rooms require 6 ft^2 (0.6 m^2) per 10 garments.

If a cafeteria is included, 20 ft^2 (1.9 m^2) per person of expected occupancy should be provided if it is equipped with tables and chairs, plus space required by any vending machines. Conference and meeting rooms should provide about 20 ft^2 (1.9 m^2) per person of expected attendance, and training rooms with theater-type seating should be 400 ft^2 (37.2 m^2) for groups of 20, 600, (55.8) for 40, and 1000 (92.9) for 80. A plant library will range from 400 to 1000 ft^2 (37.2 to 92.9 m^2), depending on its contents. Record storage requires 6 ft^2 (0.6 m^2) per file cabinet. Storage space must be provided for stationery and supplies. Slop sink and mop closets should be 12 to 15 ft^2 (1.1 to 1.4 m^2). Facilities should be placed as close as possible to those who will use them; those for universal use must be located conveniently. In very large plants, spaces and facilities for universal use must be supplied in multiples, and include toilets, clothes closets, lockers, time clocks, emergency exit doors, copy machines, vending machines, and drinking fountains. Aisle locations and widths are critical elements in the management of internal traffic, and are based on: use only for pedestrians or by material handling vehicles as well, in which case load widths must also be included as a design parameter; whether traffic is one-way or two-way, with loaded vehicles passing each other; traversing vehicular traffic only or with dropoff and/or pickup points along the route; and provision of turnaround space for vehicles (forklifts and the like) or restrictions on maneuvering within aisles. For efficient traffic flow, the configuration that will work best most often is one with one main aisle and a number of smaller feeder aisles, with straight, well-marked aisles intersecting at right angles. Aisles located at exterior walls will result in loss of storage space, and generally will be remote from the central area meant to be served. While necessary and functional, aisle space can occupy from 15 to 30 percent of the plant's total floor space. Planning for aisle space must defer to any applicable labor laws or local ordinances.

Services

An industrial plant's services are those utilities that power and supply the production and support functions. They include electric power and backup emergency power; heating, ventilating, and air conditioning (HVAC); water for processes, drinking, toilets, and cleaning; smoke, fume, and fire detection and fire fighting; natural gas and/or fuel oil, process liquids and gases, compressed air, steam; battery chargers for electric forklift trucks, AGVs, and mobile robots; communication networks and links for telephone, facsimile, computers, and other database nodes; surveillance and security systems; and waste, scrap, and effluent disposal drains and piping. The design of these requires not only the specification of the equipment, but also the layout of the distribution networks, with drops to where needed and points of interface to the apparatus served.

Flexibility is increased and unsightly and dangerous wires are elimi-

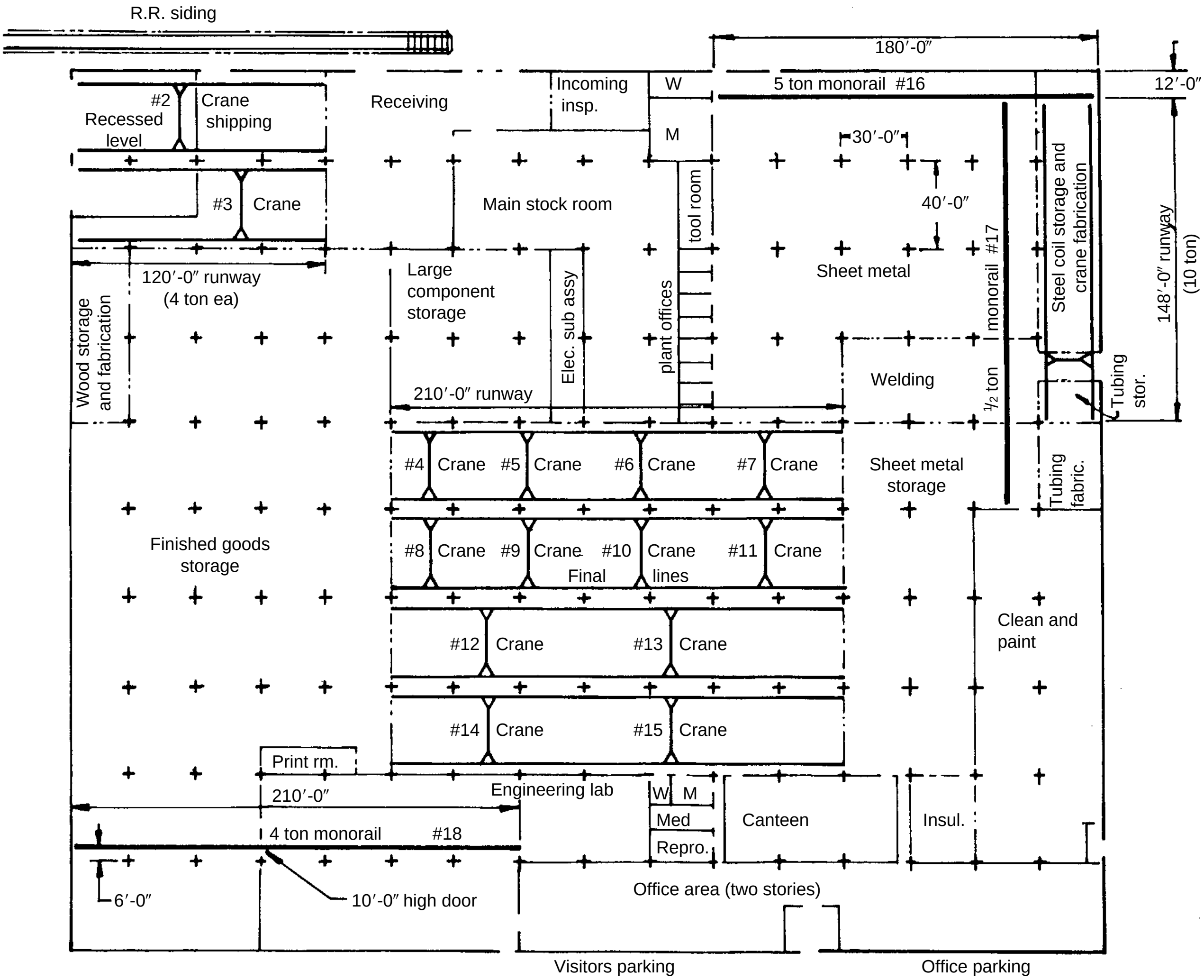


Fig. 12.1.7 A refined block diagram that converts the prior sized block arrangement onto one that makes the shape of each department, function, and area be such that, together, they fit into a building of a more conventional shape, while maintaining the size and relative position of each. *(Source: VMA, Inc.)*

nated by installing buried electric raceways in the floor before concrete is poured. Wires channeled thus are connected to floor-mounted equipment through access caps; changes in machine layout or additions to the complement of machinery are facilitated via connections into the raceways. Definitive, current records of buried raceways document exact locations of raceways and any modifications made thereto over periods of time, and while they are archival in nature, they serve to prevent confusion and guesswork. Other raceways and wire conduits are dispersed through the plant to accommodate electric convenience outlets, communication equipment, and similar services.

Electric power is usually transmitted over the utility's lines at between 22,000 and 115,000 V. The plant usually includes transformers to reduce voltage to 2,300–13,000 V. Most building codes require that these transformers be installed outside the building. The next level of voltage reduction, down to 120–480 V, is provided by transformers usually located inside the building and dispersed to strategic locations to provide balanced service with minimum-length runs of service connections. Alternatively, the plant may arrange for the local utility to supply electric power already stepped down to the levels needed; 240/120-V single phase, three-wire; 208/120-V three-phase, four-wire; 480/277-V three-phase, four wire. The last is more economical for motors and industrial lighting.

Natural light entering the plant through windows and skylights is erratic and difficult to control. All areas of the plant must be illuminated adequately for the activities conducted therein. Section 12.5 lists typical illumination levels. Density of light, illuminance, is designated in footcandles, fc (lumens per square foot) or lux, lx (lumens per square meter). One fc = 10.76 lux. In most cases, tasks requiring illuminance more than 100 to 150 fc (1080 to 1600 lux) also require directed supplemental illumination. No area should be illuminated at a level less than 20 percent of nor more than 5 times that of adjacent areas because eyes have trouble adjusting rapidly to drastic differences in illuminance. Proper illumination is also a function of the type and form of the lighting fixtures. Luminaires are selected to shed direct, indirect, or diffused light. Lamp types include incandescent, fluorescent, mercury vapor, metal halide (multivapor), and high-pressure sodium vapor. The number and type of lamps per luminaire, height, spacing, and percent reflectance of the floor, walls, and ceiling are contributing factors. The plant should have a portion of its lights attached to an emergency power source that switches on when the regular power fails. (See Rosaler,

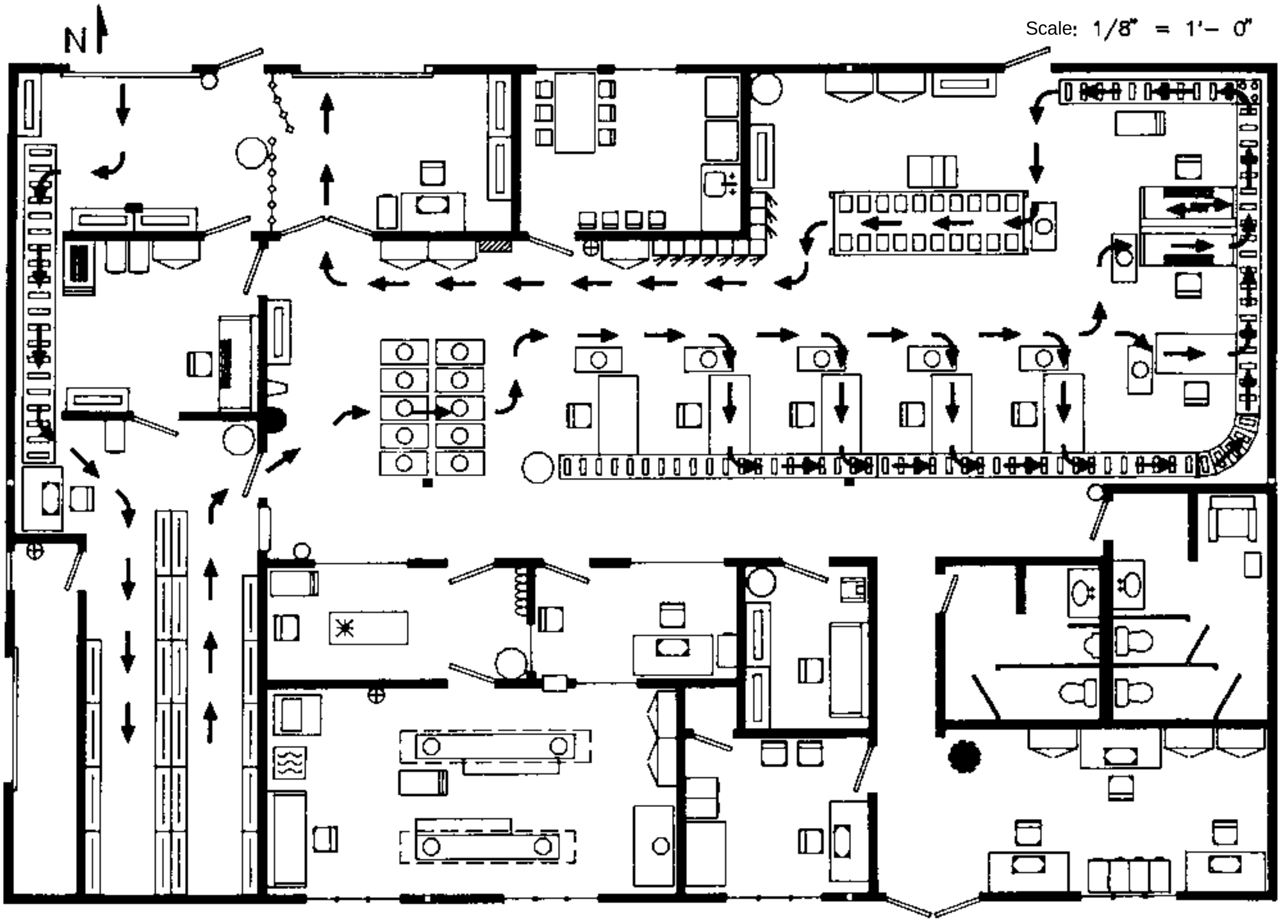


Fig. 12.1.8 A two-dimensional computer-generated detailed layout of one area of an industrial plant, showing furniture, fixtures, and flow path. *(Source: Cederleaf, ''Plant Layout and Flow Improvement,'' McGraw-Hill.)*

''Standard Handbook of Plant Engineering.'') A stationary diesel-, gas-, or gasoline-powered electric generator is usually installed to provide emergency electric power. The available fault current at all points in the electric distribution system should be determined so that protective devices can be installed to interrupt it. Selected circuits should have uninterruptible power supplies (UPSs), isolators, and regulators to protect against outages, voltage surges, sags, spikes, frequency drifts, and electrical noise. Exit signs and a clear path to the exits should be capable of being energized by emergency power, from either a standby generator or batteries.

Water, water distribution, and fixtures are essential for many plant processes, including paint booth ''waterfalls''; for cooling machines, welders, and the like; for adding water as an ingredient in some products; in toilets, showers, cafeterias, and drinking fountains; for sprinkler systems and fire fighting; and for custodial work and landscaping maintenance. If large amounts of hot and/or cold water are required by the process, boilers and/or chillers are provided, along with associated piping and pumps for fuel and water distribution. The number of toilet fixtures as required by building codes is based on the number and sex of expected building occupants. Dispensers that chill and/or heat water are useful to prepare beverages and soups. Water consumption per person per 8-hour shift in personnel facilities ranges from 30 to 80 gal (114 to 303 L).

Sprinklers are installed according to the recommendations of the National Fire Protection Association (NFPA). Automatic sprinklers can be the wet type, wherein water is always in the pipes up to each head, or the dry type, wherein the pipes are filled with air under pressure to restrain water until the fusible link in the head melts and releases the air pressure, allowing water to flow.

The dry type is used outdoors and in unheated areas where water could freeze. Sprinkler heads are either standard or deluge type. Standard heads have a fusible element in each head that is melted by the heat of a fire, thereby opening the head and releasing water. Deluge heads do not have fusible elements, but a deluge valve which is opened in response to a signal from any of several heat sensors situated in the protected area. In the standard type, only the heads whose elements are melted release water, whereas deluge heads act simultaneously. A deluge system is more likely to extinguish sparks at the periphery of a fire, but it also may ruin materials that are doused unnecessarily with water. A preaction system includes sensors and an alarm which gives plant personnel a chance to deal with the fire before sprinklers actuate and douse valuable merchandise. The alarm may also be wired to signal the local fire department. The sprinkler heads must be spaced in accordance with the prevailing code: generally one head per 200 ft^2 (18.6 m^2) for low-hazard areas, 90 ft^2 (8.4 m^2) for high-hazard areas, and about 120 ft^2 (11.2 m^2) for general manufacturing; about 8 to 12 ft (2.44 to 3.66 m) between heads and height of the highest head not over 15 ft (4.57 m) are typical. (See also Sec. 18.3). Drains should be installed to carry away sprinkler water. The supply of water for all of the above listed needs must be adequate and reliable. For fire fighting, the greatest fire hazard and the size of the water supply required to deal with it must be estimated. The expected flow from all hoses and sprinklers, the static

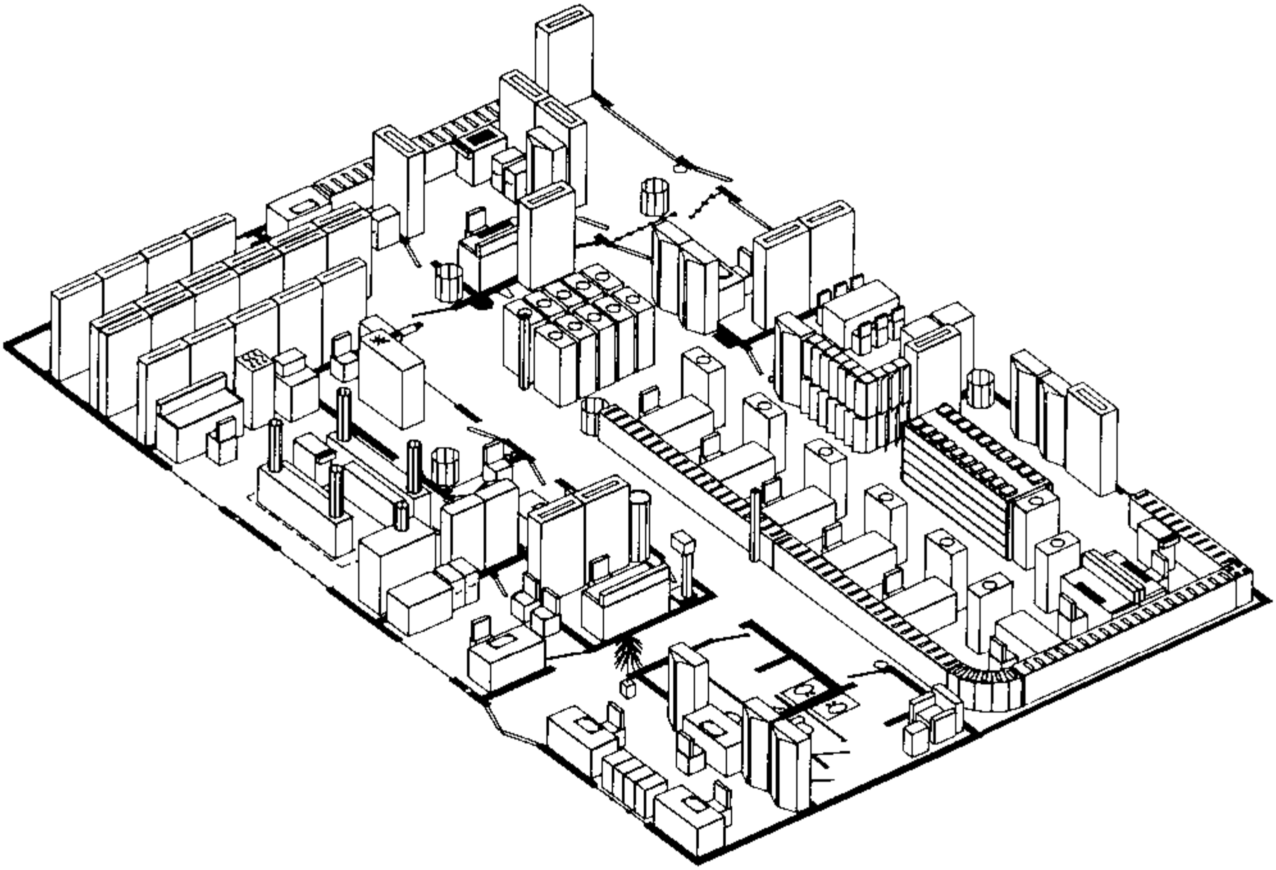


Fig. 12.1.9 A three-dimensional computer-generated detailed layout of the same area shown in Fig. 12.1.8. *(Source: Cederleaf, ''Plant Layout & Flow Improvement,'' McGraw-Hill.)*

pressure, and the minimum flow available at a given residual pressure must be considered. Again, codes and insurance company requirements control. Sources of water include city mains, gravity tanks on towers, reservoirs on roofs or in decorative ponds that are part of the landscaping, wells, nearby lakes, and rivers.

The amount of **heat** required for personnel comfort depends on the plant's location, which in turn, influences the types and capacities of heaters selected. Generally, factory areas can be kept a little cooler, about 65°F than the 72°F recommended for office areas. Heating systems include warm air, hot water, steam, electric, and radiant. Warm air requires a furnace to heat the air and ducts and blowers to distribute it. Circulating hot water and steam heat also require boilers and furnaces and a network of distribution pipes and radiators. Electric space heaters require fixed wiring and local outlets. Unit radiant heats may be fueled by gas, steam, hot water, or electricity and are placed above doors and work areas, oriented to direct heat where it is required. Piping can be embedded in the concrete slab on grade, and in other floors, walls, and ceilings to provide radiant heat from circulating hot water.

Ventilation, the introduction of fresh outside air, the exhaust of stale inside air, or merely the movement of otherwise still inside air, can be effected naturally or mechanically. Natural ventilation requires windows, skylights, louvers, or other openings. Mechanical ventilation requires fans and blowers (and sometimes ducts) to draw air in, circulate it, and exhaust it. The number of cubic feet per minute (cubic metres per minute) of air per person required by people working in an office is about 10 (0.3), and between 25 and 50 (0.7 to 1.4) for those working on the factory floor. The total amount of air needed and the number of air changes per hour determine the number and sizes of fans, motors, and ducts. One change per hour is too little, with no discernible difference in air quality; 50 to 60 changes of air per hour are too many, and the resulting high-velocity drafts cause sensations of chilling and accompanying discomfort. Minimally, 5 changes of air per hour are required.

Air conditioning adds to employee comfort, increases their productivity, and is essential for the manufacture of certain products. The calculation of the expected heat gains required to specify an **air-conditioning** installation is similar to the procedure for calculating heat loss in the design of a heating system. For air conditioning, the required cooling load is calculated in Btu per hour and converted to required tons of refrigeration by dividing by 12,000. (See Sec. 12.4.) The tonnage required would be one basis for the selection of the type of water-cooled or evaporative condensers, compressors, air handling units, motors, pumps, ducts, registers, circuitry, panels, and controls to be installed. Packaged air-conditioning units mounted on window sills, floors, ceilings, or roofs are often used. (See Merritt and Ricketts, ''Building Design and Construction Handbook.'')

The design and installation of a **compressed-air system** includes the summation of the volume and pressure of air required at all plant locations. The pipe diameters and total pressure drops between compressor and the most remote point of delivery must be determined. Pressure drop is a function of pipe and hose friction and the requirements of air-powered devices. Piping may be fixed, by and large, but the number and types of air-powered devices will change from time to time as production processes are changed or rearranged. Thus, there is some variability to be expected in the calculations to determine compressor discharge volume and pressure. Suitable allowances for unknown quantities are usually factored into the final design selection.

Internal and external communications systems must be installed. With autofacturing (see Sec. 17), the design, manufacture, production control, inventory control, storekeeping, warehousing, sales, and shipping records are all integrated and tied to the same information database. An internal local-area network (LAN), with computers, terminals, displays, and printers distributed throughout the plant and offices, will be required to plan, analyze, order, receive and accept material, record, feed back data, update files, and control and correct operating conditions. AGVs and mobile robots can be programmed to navigate off beacons installed throughout the plant.

Parking for employees and visitors must include spaces for handicapped persons. Parking lots are usually paved with 3 in (7.6 cm) of asphaltic cover over 6 in (15.2 cm) of gravel base. Lines are painted to designate spaces, which may be ''straight in'' (at a 90° angle to the curb), or at a smaller angle, typically 45° to 60°. The angle used influences the width of the aisles, depth (distance from the edge of the aisle to the curb) of each space, and the amount of curb length required for each car. For example, parking at a 45° angle requires an aisle width of about 13 to 15 ft (4.0 to 4.6 m), a space depth of about 20 to 21 ft (6.1 to 6.4 m) and uses about 13 ft (4.0 m) of curb per space (because of the overlaps); a 90° layout requires an aisle width of about 24 to 27 ft (7.3 to 8.2 m) and a space depth of about 19 to 20 ft (5.8 to 6.1 m) and uses about 9 to 10 ft (2.7 to 3.1 m) of curb per space. The width of each space (except those for the handicapped, which are wider) ranges from 9 to 10 ft (2.7 to 3.1 m) and their lengths from 19 to 20 ft (5.8 to 6.1 m). The number of parking spaces to be provided depends on the number of people expected and the extent to which public transportation is available and used. It is expected that there will be more than one employee per vehicle. Factors ranging from 1.2 to 2.5 persons per car can be used, depending on the firm's best estimate of the practices of its employees. For multiple-shift operations, it must be recognized that those working the second shift will arrive and need parking spaces before those on the first shift leave and vacate them. For those using public transportation, a shelter against the weather may be erected; often it is provided by the bus company.

In addition to parking space for automobiles, **space must be provided for trucks** that bring material to the plant and carry products away. Sizes of trucks, truck tractors, and their trailers vary widely, as do their turning radii. Many trucks will drive in frontward, maneuver to turn around, and back into the loading docks and platforms. Space must be provided for them to do this, even when there are other trucks present. The minimum size apron (measured by the distance from the outermost obstruction, whether it be a part of the building, the front of another truck already in the loading dock, or anything else that is in the way), required to maneuver a tractor-trailer into or out of a loading position, in one maneuver and with no driver error, varies with the length of the tractor-trailer expected and the width of the position to be provided. For example:

Tractor-trailer length	Position width	Minimum apron size
35 ft (10.7 m)	10 ft (3.1 m)	46 ft (14.0 m)
	12 ft (3.7 m)	43 ft (13.1 m)
	14 ft (4.3 m)	39 ft (11.9 m)
40 ft (12.2 m)	10 ft (3.1 m)	48 ft (14.7 m)
	12 ft (3.7 m)	44 ft (13.4 m)
	14 ft (4.3 m)	42 ft (12.8 m)
45 ft (13.7 m)	10 ft (3.1 m)	57 ft (17.4 m)
	12 ft (3.7 m)	49 ft (15.0 m)
	14 ft (4.3 m)	48 ft (14.6 m)

Truck heights range from about 8 ft (2.4 m) for a panel truck to around 13.5 ft (4.1 m) for double-axle semis and others. Doorways must be sized accordingly. Seals and pads may be added around the doors to fill the gap between the back of trucks and the building. Besides conventional rollup doors, plastic strips and air curtains may be used to provide a partial barrier between the plant's environment and the weather.

The **plant's security** measures should include structural protection against wind, rain, flood, lightning, earthquakes, and fire. Security against intruders, breakins and vandalism can include fencing, gates, perimeter lights, surveillance cameras, and sensors. Security guard booths may be purchased as prefabricated units, complete with lights, heat, and air conditioners.

Emergency **egress** should provide for more than one route to safety, minimal distances to doors, doors that are locked from the outside but can be opened with a push from the inside (by panic bars), all to be accessible and usable by the handicapped. When designing a plant for ease of use by handicapped employees and visitors, all physical barriers must be eliminated and aids installed, such as ramps, braille signs, wheelchair-width doors and toilet booths, wheelchair-height sinks and drinking fountains, and the like. Applicable local building codes and federal regulations will guide and control the final designs.

Most of the above services and systems are built into the structure or influence its design and, therefore, must be determined before the specifications of the building become final.

Building Structure

The specifications for an industrial plant's **structure** depend on its contents, intended use, general location, and specific site. In the usual order in which they are constructed are foundation and footings, columns (and bay sizes), beams and roof trusses, exterior walls, floor slab, roof decking and covering, exterior doors and fenestration, interior partitions, walls, doorways, services (electric power, compressed air, etc.), interior decor, and other special features.

Firm foundations and footings are required to support the columns, peripheral and some interior walls, machinery, and other equipment. Especially heavy machines, or those subject to large dynamic loads (often repetitive), may require special design that incorporates measures to isolate vibrations. Depth, size, and foundation reinforcement depends on the subsurface conditions. Installation of piles may be dictated by unsuitable load-bearing soil. Almost all foundations consist of cast-in-place concrete; in rare instances, load-bearing concrete blocks are set onto a previously poured concrete base. Site topography and location of the building may require the construction of retaining walls; these may be cast concrete or assembled with precast concrete sections which may serve also to provide a decorative treatment. The subgrade on which the concrete floor slabs will be poured must be well-compacted and made level. Often, the floor slab is not cast until heavy equipment (used to erect superstructure) is removed from the site.

Steel superstructure members are designed to provide the clear spans delineated by the selected **bay sizes**. They also support intermediate floors (if any); roof-mounted equipment; material handling equipment such as monorails; and lighting fixtures, piping, ductwork, and other utilities. The design must account for all dead and live loads, usually in accordance with applicable local building codes and other accepted structural codes. (See Secs. 5, 6, and 12.)

The sizes and locations of **columns and beams** establish the bay sizes. The longer the span, the larger the bay size, but this reflects back into construction with heavier columns, deeper beams and trusses, and concomitant higher costs. Inasmuch as larger bay sizes permit greater freedom in plant layouts and ease material handling, the added expense of large bay sizes is often worth it. Accordingly, the trend in plant design is to incorporate large bays. While any bay size is feasible, a cost/benefit tradeoff may set reasonable limits. Typical bay sizes are 30×40 ft (9×12 m), 40×60 (12×18) and 40×80 (12×24). There are warehouses whose bay size is dictated by the requirement to accommodate stacks of standard pallets with minimum waste space. Aircraft manufacturing plants and commercial hangars enclose enormous column-free cavernous spaces; lengths of 300 to 400 ft (91 to 122 m) are the norm in those applications.

When columns must support heavy traveling bridge cranes, a second row of columns is incorporated into the main columns to provide support for rails and to effect a stiffer structural configuration.

Beams are set at the top of columns and establish the clear height of the building interior; 15, 20, and 25 ft (4.5, 6.1, and 7.6 m) are typical in manufacturing areas, and 30 to 40 ft and more (9.1 to 12.2 m) are employed in warehouse space. When a two-story office is part of a single-level plant (Fig. 12.1.7) the overall height of the building is often set to the second-story height to permit a simple flat roof. Factory clear heights should be at least the height of the highest machine, with a generous margin added; for large products, the clear height is often designed to be twice the height of the largest product. Anything suspended from the beams or girders reduces the clear height. Deeper open roof trusses permit some piping, wiring, and other services to be woven through them and light fixtures placed between them, thereby not reducing clear working space. Roof decking and roofing is affixed to roof trusses or beams.

Exterior walls can be load bearing (support one end of a beam) or nonbearing. If nonbearing, all beam loads are transferred to columns spaced at intervals along the building perimeter. Walls are constructed of masonry [brick or concrete block masonry units (CMUs), sometimes incorporating glass block]; metal panels, usually integrally stiffened; wood for special applications (storage of highway deicing salts); natural stone or stone veneer/precast-concrete panels; or stone veneer on masonry backup walls. Where conditions allow, poured concrete walls are cast on the flat (at grade), then tilted up and secured into place; this technique is attractive especially for warehouses with repetitive wall construction devoid of windows and other openings. Metal panels are usually galvanized steel, with or without paint, or painted aluminum.

Factory floors may be required to sustain live loads from less than 100 pounds per square foot (psf) (0.5 kPa) to over 2,000 psf (96 kPa). A general-purpose light-assembly floor ordinarily will have no less than 100 psf (0.5 kPa) floor load capacity. A special purpose plant floor may be built with floor load capacities which vary from place to place in accordance with the requirements for different load-carrying capacity. Certainly, in the extreme, to build in a maximum floor loading capacity of, say, 2,000 psf (96 kPa) throughout a plant when there is minimal likelihood for that requirement other than in discrete areas, would not be cost-effective. Concentrated live loads (most often from wheeled material handling vehicles) are accounted for in the design of the floor slabs as required. The final design of the plant floor will meld the above factors with other requirements so that the sum of all dead and live loads can be safely sustained in the several areas of the floor.

Virtually all slab on-grade concrete floors are cast in place; in rare cases, precast concrete sections are used. Wooden plank flooring is rarely used on new construction. Upper floors are most often constructed with precast concrete planks overlaid with a cement mortar topping coat, or employ ribbed decking. When upper-floor loading capacities require it, those floors can also be cast in place; in those instances, concrete is cast into ribbed metal decking which acts as wet form work and remains in place. End grain wood blocks may be set atop a concrete floor to mitigate against unavoidable spillage of liquids. (See Sec. 12.2.)

The load-carrying capacity of concrete slabs on grade is ultimately limited not only by the strength of the concrete itself, but also by the ability of the subgrade material to resist deformation. The proper design of the floor, especially at grade, will account for all the parameters and result in a floor which will safely sustain design floor loading. Automatic guided vehicle systems and mobile robots (see Sec. 10) require level and smooth floors. The floor surface may be roughened slightly with a float to reduce slipping hazards, or have a steel trowel finish to accommodate AGV and mobile robot navigation. If it is contemplated that products or equipment will be moved by raising and floating them on films of air, the floor must be crack-free. If a slight floor slope can be tolerated from an operational point of view, that will ease cleaning by allowing water to flow to drains. The slab's surface may be coated, treated, or tiled for protection, comfort, ease of maintenance, and/or aesthetics. Expansion joints around column bases will inhibit propagation of small cracks resulting from differential settlement between column footings and floor slabs.

At this stage, the building looks like the one shown in Fig. 12.1.10.

A **flat roof** is always slightly pitched to drain water toward scuppers, gutters, and downspouts. It is constructed with ribbed metal decking or precast concrete planks, topped with a vapor barrier, insulation, a topping surface, flashing, sealants or caulking, and roof drains. The covering of a **built-up roof (BUR)** comprises three to five layers of roofing felt (fiberglass, polyester fabric, or other organic base material), each layer mopped with hot bitumen (asphalt or coal tar pitch). The wearing surface of roofing may include a cap sheet with fine mineral aggregate, reflective coating, or stone aggregate. When extensive roof traffic is anticipated, pavers or wood walkways are installed. A recent successful roof surface is composed of a synthetic rubber sheet, seamless except at lapped, adhesively bonded joints. It has proved to be a superior product, providing long, carefree life.

The seal between roofing and parapet walls or curbed openings is effected with **flashing** bent and cemented in place with bituminous cement. The flashing material may be galvanized steel aluminum, copper, stainless steel, or a membrane material.

Flat roofs lend themselves to ponding water to help cool the plant. Roof ponds (and water sprays) can reduce interior temperatures by 10 to 15°F (6 to 9°C) in the summer. Roof ponds can serve as a backup source of water for fire fighting. If water is ponded on a roof, roof construction must be handled expressly for that purpose. There are structural implications which must be considered as well.

Doors and ramps should be made 2 ft (0.6 m) larger than the largest equipment or material expected and large enough to allow ingress of fire fighting apparatus. It is often convenient to place very large pieces of equipment such as molding machines and presses inside the building envelope before the walls are completed. After exterior walls and roof are in place, the building interior is sufficiently secure for assembling materials for the remaining construction and to receiving smaller equipment.

Interior walls and **partitions** (masonry, wood, wallboard, and metal) follow, after which office flooring and framing are installed (Fig. 12.1.11). The remainder of the construction involves completion of wiring, lighting, and other services; application of paint or wall coverings, floor coverings, and decor; and so on.

Other Considerations

The **applicable standards, regulations, and procedures** of the 1971 Williams-Steiger Occupational Safety and Health Act (OSHA), as amended, must be followed in the construction and operation of the building and the design and use of the plant's equipment; likewise, the regulations of the Environmental Protection Agency (EPA), as amended, and the design standards of the Americans With Disabilities Act (ADA), as amended, must be followed.

Color can serve several purposes in an industrial plant. As an aid to safety, it can be used to identify contents of pipes, dangerous areas, aisles, moving parts of equipment, emergency switches, and fire fighting and first aid equipment. It also can be used to improve illumination, conserve energy, reduce employee fatigue and influence on morale positively.

OSHA, American National Standards Institute (ANSI) specification Z53.1, and specific safety regulations require the use of specific **identity colors** in certain applications. As a general guide, yellow or wide yellow-and-black bands are used to indicate the need for caution and to highlight physical hazards that persons might trip on, strike against, or fall into, such as edges of platforms, low beams, stairways, and trafficked aisles along which equipment moves. Orange designates dangerous parts of machines or energized equipment that may cut, crush, shock, or otherwise injure, such as the inside of gear boxes, gear covers, and exposed edges of gears, cutting devices, and power jaws. Red is the basic color for the identification of fire protection equipment and apparatus, danger and stop signs, fire alarm boxes, fire exit signs, and sprinkler piping. Green designates safety items, such as first aid kits, gas masks, and safety deluge showers, or their locations. Blue designates caution and is limited to warning against starting, use of, or movement of equipment under repair, and utilizes tags, flags, or painted barriers. Purple designates radiation hazards. Black, white, or a combination of both designated traffic control and housekeeping markings, such as aisleways, drinking fountains, and directional signs.

Fig. 12.1.10 A view of a plant during construction. Shown are its columns, roof trusses, and exterior masonry walls. The reinforced-concrete floor slab is being poured over compacted soil. *(Source: VMA, Inc.)*

There are also color conventions for the identification of fluids conducted in pipes. Instead of being painted their entire length, pipes are painted with colored bands and labels at intervals along the lines, at valves, or where pipes pass through walls. ANSI Standard A13.1-1975 specifies the following colors for pipe identification: red for fire protection; yellow for dangerous materials; blue for protective materials; green and white, black, gray, or aluminum for safe materials.

The **noises** created during the operation of a plant can be dealt with at their **sources, along their paths,** and at their **receivers**. At the sources, noise is minimized or eliminated by: changing the process; replacing (with quieter) equipment; modifying equipment by redesign or component changes; moving to another location; installing mufflers and shock mounts; using isolation pads; and shielding and enclosing equipment with material having a high sound transmission loss (STL) value. STL is a measure of the reduction in sound pressure as it passes through a material. Along the paths, attenuation of noise is enhanced by increasing the distance between sources and receivers; introducing discontinuities in the transmission path, such as barriers and baffles which interrupt direct transmission; installing acoustical barriers with an STL of at least 24 dB and with sufficient absorption to prevent reflection of the noise back to its source; and by placing sound-absorbent material on surfaces along its path, such as acoustical ceilings and floor and wall coverings. At the receivers, enclosures, workstation partitions of sound-absorbing material, earplugs, and ''white-noise'' generators can be used. (See Secs. 12.6 and 18.2.)

Provisions must be made for **waste disposal,** including: disposal of refuse and garbage, treatment of process fluids and solids, and waste and recyclables recovery, all in accordance with applicable EPA and other regulations.

Among many other things, the plant design process must address matters such as fuel storage, storm and surface drainage, snow removal facilities and procedures (if the local climate warrants), design for ease of maintenance of the building as well as the grounds and associated landscaping, the articulation of the architect's idiom, energy efficiency in the materials of construction and the equipment installed to service the plant, and diligence in compliance with environmental impact studies (if required).

After the plant is designed, but before a commitment is made to build it, its operations should be simulated to bring forth and correct any errors or omissions. In addition to testing its specifications for normal conditions, extremes (severe storms, power outages, truckers' strikes, etc.) should also be evaluated. This simulation is similar to that done before the plant's site is made final, but comes after the structure and layout have been designed. A sensitivity analysis should be made to see the effect on operations of changes in inputs and operating conditions. Depending on these analyses and best guesses as to the future, adjustments may have to be made to ensure that the plant will operate effectively and economically under all reasonable conditions, and that it has no features that will harm people or do damage to the products or the environment.

Fig. 12.1.11 A view of framing for the first- and second-story offices of a plant, with a roof beam and the steelwork supporting the second floor visible. *(Source: VMA, Inc.)*

CONTRACT PROCEDURES

Cost Estimates

Cost estimates fall into three general classes:

1. Preliminary estimates made usually from sketch drawings and brief outline specifications to determine the approximate total cost of a project.

2. Comparative estimates made usually during the progress of design to determine the relative cost of two or more alternative arrangements of equipment, type of building, type of floor framing, and the like.

3. Detail estimates made from final plans and specifications and based on a careful quantity survey of each component part of the work.

Primary quantity estimates are usually more accurate when comparison with comparable projects is lacking or not feasible. The procedure entails computations based on quantity takeoffs from preliminary plans and specifications, and can include the gross area of exterior walls, interior partitions, floors, and roof. Each is multiplied by known (or estimated) unit cost factors. Other items, such as number of electrical outlets and number of plumbing fixtures and sprinklers, are estimated, and their cost computed. Equipment costs can be based on preliminary quotations from manufacturers.

The following items should be included in preliminary estimates (or in other more detailed estimates which may follow as plans develop to the final stage): land cost; fees to real estate brokers, lawyers, architects, engineers, and contractors; interest during construction; building permits; taxes, including local sales or use taxes; demolition of existing structures including removal of old foundations; yard work including leveling, drainage, fencing, roads, walks, landscaping, yard lighting, and parking spaces; transportation facilities including railroad tracks, wharves, and docks; power supply and source; water supply and source; sewer and industrial waste disposal. A judicious **contingency factor** is included to provide for unforeseen conditions that may arise during the development of the project. It may amount to 5 to 15 percent (or more) depending on the character and accuracy of the estimate, whether the proposed design and construction will follow established procedures or incorporate new state-of-the-art features, and the exact purpose for which the estimate is made.

Especially for complex new or altered construction, estimates may be updated continually as a matter of course. This will serve to monitor the evolving cost of the project and the time constraints placed on construction.

Working Drawings, Specifications, and Contracts

The technical staff of a given industry has special knowledge of trade practices, process requirements, operating conditions, and other fundamentals affecting successful operation in that field and is best fitted to determine the basic factors of process design and plant expansion. Unless the company is extremely large, it is unlikely that they will have the specialized staff or that their own staff will have the time available to undertake the complete layout. The efficient transformation of these requirements into completed construction usually requires experience of a different nature. Therefore, it is generally advisable and economical to employ engineers or architects who specialize in this particular field.

Three general procedures are in common use. First, the employment on a percentage or fixed-fee basis of an engineering and construction organization skilled in the industrial field to prepare the necessary working drawings and specifications, purchase equipment and materials, and execute the work. This has become known as **design/build**. For the duration of the project, such an organization becomes a part of the owner's organization, working under the latter's direction and cooperating closely with the owner's technical staff in the development of the design and in the purchase and installation of equipment and materials. This procedure permits construction work to start as soon as basic arrangements and costs have been determined, but before the time required to complete all working drawings and specifications. This is often termed **fast tracking**. Such a program will result in the earliest possible completion consistent with economical construction.

A second procedure is to employ engineers or architects with wide experience in the industrial field to prepare working drawings and specifications and then to obtain **competitive lump-sum bids** and award separate contracts for each or a combination of several subdivisions of the work, such as foundations, structural steel, and brickwork. This method provides direct competition restricted to units of like character and permits intelligent consideration of the bids received. Provision must be made for proper coordination of these separate contracts by experienced and skilled field supervision. In recent years, a **construction manager** has been hired for this purpose. This procedure requires more time than that first described, since all work of a given class should be completely designed before bids for that subdivision are sought. Where construction conditions are uncertain or hazardous, the first method is likely to be more economical. A combination of the first method for uncertain conditions and the second method for the balance may prove most advantageous at times.

A third procedure is to employ engineers or architects to complete all plans and specifications and then **award a lump-sum contract** for the entire work, or one for all building work, and one or more supplementary major contracts to furnish and install equipment. This method is particularly useful where there are no serious complications or hazards affecting construction operations, but it requires considerably more time, since most of the working drawings and specifications must be complete before construction is started. It has the significant advantage, however, of fixing the total cost within narrow limits before the work starts, provided the contracts cover the complete scope of work intended and no major changes ensue.

CONSTRUCTION

The design, construction, occupancy, and operation of an industrial plant is a complex endeavor involving many people and organizations. **Project control tools and techniques** such as the critical path method (CPM), the program evaluation and review technique (PERT), bar charts, dated start/stop schedules, and daily ''do lists'' and ''hot lists'' of late items are all helpful in keeping construction on time and within budget.

Checkpoints and milestones, with appropriate feedback as the project progresses, are established to monitor progress. Bailout points are established in the event the project must be aborted sometime after the beginning of construction. On the other hand, contingency plans should be in hand and include planned reassignment of financial and personnel resources to keep the project on schedule. Figure 12.1.12 shows the scheduled and actual dates of completion of the phases noted for a small plant depicted in several of the previous figures.

Some portion of the organization (often the construction manager's office), is charged with the delicate task of coordinating contractors and the multiplicity of trades at the job site, and as part of its function, it will generate ''punch lists'' of deficient and/or incomplete work which must be completed to the owner's satisfaction before payment therefore is approved.

	Scheduled	Actual
Project planning	March	March
Basic layout	April	April
Structural design	April	April
Equipment specifications	April	April
Construction contract	May	May
Ground breaking	June	June
Detailed office layout	June	June
Detailed plant layout	July	July
Equipment purchases	August	August
Furniture purchases	September	September
Organization firmed	September	September
Steelwork completed	October	October
Masonry completed	October	October
Roof completed	November	November
Staffing and training	November	November
Systems designed	November	November
Forms designed	December	December
Offices completed	December	December
Equipment received	December	December
Building occupied	December	December

Fig. 12.1.12 The project schedule and completion dates for the single-level 202,800 ft^2 industrial plant used as an example in Figs. 12.1.1 to 12.1.7, 12.1.10, and 12.1.11. *(Source: VMA, Inc.)*

12.2 STRUCTURAL DESIGN OF BUILDINGS

by Aine M. Brazil

REFERENCES: ''Manual of Steel Construction—Allowable Stress Design,'' American Institute of Steel Construction. ''National Design Specification for Wood Construction,'' American Forest and Paper Association. ''Design Values for Wood Construction,'' American Forest and Paper Association. ''Uniform Building Code,'' International Conference of Building Officials. ASCE 7-93, ''Minimum Design Loads for Buildings and Other Structures,'' American Society of Civil Engineers. Blodgett, ''Design of Welded Structures,'' J. F. Lincoln Arc Welding Foundation. ''Masonry Designers Guide,'' The Masonry Society.

LOADS AND FORCES

Buildings and other structures should be designed and constructed to support safely all loads of both permanent and transient nature, without exceeding the allowable stresses for the specified materials of construction. Dead loads are defined as the weight of all permanent construction; live loads are those loads produced by the use or occupancy of the building; environmental loads include the effects of wind, snow, rain, and earthquakes.

Live loads on floors are generally regulated by the building codes in cities or states. For areas not regulated, the following values will serve as a guide for live loads in lb/ft^2 (kPa): rooms for habitation, 40 (1.92); offices, 50 (2.39); halls with fixed seats, 60 (2.87); corridors, halls, and other spaces where a crowd may assemble, 100 (4.76); light manufacturing or storage, 125 (5.98); heavy manufacturing or storage, 250 (11.95); foundries, warehouses 200 to 300 (9.58 to 14.37). Floor decks and beams that support only a small floor area must also be designed for any local concentrations of load that may come upon them. Girders, columns, and members that support large floor areas, except in buildings such as warehouses where the full load may extend over the whole area, may often be designed for live loads progressively reduced as the supported area becomes greater. Where live loads, such as cranes and machinery, produce **impact** or **vibration,** static loads should be increased as follows: elevator machinery, 100 percent; reciprocating machinery, 50 percent; others, 25 percent.

Roof live loads should be taken as a minimum of 20 lb per horizontal ft^2 (957.6 kPa) for essentially flat roofs (rise less than 4 in/ft), varying linearly with increasing slope to 12 lb/ft^2 for steep slopes (rise greater than 12 inches per foot). Reductions may also be made on the basis of tributary area greater than 200 ft^2 (18.58 m^2) of the member under consideration, to a maximum of 40 percent for tributary areas greater than 600 ft^2. The minimum roof live load shall be 12 lb/ft^2 after all reduction factors have been applied. Where **snow loads** occur, appropriate design values should be based on the local building codes.

Dead loads are due to the weight of the structure, partitions, finishes, and all permanent equipment not included in the live load. The weights of common building materials used in floors and roofs are given below (see also Sec. 6).

Material	Weight, lb/ft^2
Asphalt and felt, 4-ply	3
Corrugated asbestos board	5
Glass, corrugated wire	5–6
Glass, sheet, ⅛ in thick	2
Lead, ⅛ in thick	8
Plaster ceiling (suspended)	10
Acoustical tile	1–2
Sheet metal	1–2
Shingles, wood	3
Light weight-concrete over metal deck	30–45
Sheathing, 1 in wood	3
Skylight, 3⁄16 to ¼ in, glass and frame	6–8
Slate, 3⁄16 to ½ in thick	8–20
Tar and gravel, 5-ply	6
Tar and slag, 5-ply	5
Roof tiles, plain, ⅝ in thick	20

NOTE: lb/ft^2 × 0.04788 = kPa.

Earthquake Effects Two distinct methods of designing structures for seismic loads are currently accepted. The more familiar method, employed by the Uniform Building Code (UBC), yields equivalent loading for use with the allowable stress design approach. The National Earthquake Hazard Reduction Program (NEHRP) has developed Recommended Provisions for the Development of Seismic Regulations for New Buildings, which is based on the ultimate strength design approach. The NEHRP approach is the basis for model codes, such as BOCA (Building Officials and Code Administrators International, Inc.) National Building Code.

Following the UBC design approach, the static force procedure represents the earthquake effects as equivalent static lateral forces applied at each floor level. The total design lateral force (called **base shear**) is calculated by the formula

$$V = \frac{ZIC}{R_w} W \qquad \text{where} \qquad C = \frac{1.25\,S}{T^{2/3}} \leqq 2.75$$

V = design base shear; Z = factor representing the degree of regional seismicity, ranging from 0.4 for seismically active areas with proximity to certain earthquake faults (Zone 4) to 0.075 for areas of low seismicity (Zone 1); I = importance factor (1.25 for essential facilities, 1.0 for most others); R_w = coefficient representing the type of lateral-force-resisting system of the building, ranging from 4 for a heavy timber bearing wall system (where bracing carries gravity loads as well as lateral loads) to 12 for buildings with highly ductile systems, such as special moment-resisting frames (this coefficient is a measure of the past earthquake resistance of various structural systems); T = the fundamental period of vibration, seconds, of the building in the direction under consideration (this coefficient represents the acceleration effects of the dynamic response of the structure); S = coefficient representing possible amplification effects of soil-structure interaction and is taken as 1.5 unless a lower value is substantiated by soils data; and W = the total dead load (including partitions) plus snow loads over 30 lb/ft² (1.436 Pa) and 25 percent of any storage or warehouse live loads.

The **fundamental period** T, used to calculate the seismic coefficient C, may be determined by a rational analysis of the structural properties and deformation characteristics of the structure, or it may be estimated by the formula $T = C_t(h_n)^{3/4}$, where C_t = 0.035 (0.0853) for steel moment-resisting frames, 0.030 (0.0731) for reinforced-concrete moment-resisting frames and eccentrically braced frames, and 0.02 (0.0488) for all other buildings.

The **base shear** V is considered to be distributed over the height of the structure according to the formula

$$F_x = \frac{(V - F_t)w_x h_x}{\Sigma w_i h_i}$$

where $F_t = 0.07TV$ is the lateral force at the top and F_x is the lateral force at any level x; w_x is the weight assigned to level x; and h_x is the height of level x above the base.

For stiff, low-rise buildings, such as one- to three-story, steel-braced frame or concrete shear wall structures, it is common practice to compute the design base shear using the maximum values for the coefficients C. Thus, the design base shear for a two-story concrete shear wall building in Zone 4 might be taken as

$$V = ZICW/R_w = (0.4 \times 1.0 \times 2.75 \times W)/8 = 0.1375W.010$$

Where floors are **rigid diaphragms,** such as concrete fill over metal deck or concrete slabs, lateral forces are distributed to the vertical-resistive elements on the basis of their relative stiffness. Where floors or roofs are flexible diaphragms, such as some metal deck with nonstructural fill, plywood, or timber planking, lateral forces are distributed to the resistive elements on the basis of tributary area. Where a rigid diaphragm exists, a torsional moment, equal to the story shear multiplied by the greater of the real eccentricity between the center of mass and the center of rigidity of the resistive elements or 5 percent of the maximum building dimension at that level, is applied to the diaphragm around a vertical axis through the center of rigidity of the resistive elements. Direct shears in the elements are increased by those induced by the torque when additive but are unaltered when subtractive in order to arrive at design lateral seismic loads to the resistive element.

Earthquake forces on portions of structures, such as walls, partitions, parapets, stacks, appendages, or equipment, are calculated by the formula $F_p = ZI_pC_pW_p$, where F_p = the equivalent lateral static force acting at the center of mass of the element; Z and I_p = coefficients previously defined (although I_p for life safety equipment may be greater than that for the parent structure); C_p = horizontal force factor and is taken as 2.0 for cantilever chimneys, stacks, and parapets as well as ornamental appendages and as 0.75 for other elements such as walls, partitions, ceilings, penthouses, and rigidly mounted equipment; and W_p = weight of element. For flexibly mounted equipment C_p may conservatively be taken as 2.0 or may be determined by rational analysis considering the dynamic properties of both the equipment and the structure which supports it. Seismic loads are applied to walls and partitions normal to their surface and to other elements in any horizontal direction at the center of mass.

Wind Pressures on Structures Every building and component of buildings should be designed to resist wind effects, determined by taking into consideration the geographic location, exposure, and both the shape and height of the structure. For structures sensitive to dynamic effects, such as buildings with a height-to-width ratio greater than 5, structures sensitive to wind-excited oscillations, such as vortex shedding or icing, and tall buildings [height greater than 400 ft (121.9 m)] special consideration should be given to design for wind effects and procedures used should be in accordance with approved national standards. Wind loads should not be reduced for the shading effects of adjacent buildings.

Wind pressure on walls of buildings should be assumed to be a minimum of 15 lb/ft² (0.72 kPa) on surfaces less than 50 ft above the ground. For buildings in exposed locations and in locations with high wind velocity (over 70 mi/h or 112.5 km/h), pressures should be calculated based on the following procedure.

Design wind pressure may be determined by the following formula, which is based on the Uniform Building Code: $P = C_eC_qq_sI_w$, where P = design wind pressure; C_e = coefficient which varies with height, exposure, and gust factor (refer to Table 12.2.1); C_q = pressure coefficient (refer to Table 12.2.2), q_s = wind stagnation pressure at standard height of 33 ft (refer to Table 12.2.3); I_w = importance factor (essential or hazardous facilities, 1.15; other structures, 1.0). The **basic wind speed** is the fastest wind speed at 33 ft (10 m) above the ground of terrain Exposure C and associated with an annual probability of occurrence of 0.02 [varies from 70 to 100 mi/h (112 to 161 km/h)]. The exposure category defines the characteristics of ground surface irregularities at the specific site: Exposure B has terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger; Exposure C is flat, open terrain with scattered obstructions having a height of less than 30 ft; Exposure D is flat, unobstructed areas exposed to wind flowing over large bodies of water.

For exceptionally tall, slender or flexible buildings, it is recommended that a wind tunnel test be performed on a model of the building.

Table 12.2.1 Combined Height, Exposure, and Gust Factor Coefficient C_e*

Height above average level of adjoining ground, ft†	Exposure D	Exposure C	Exposure B
0–15	1.39	1.06	0.62
20	1.45	1.13	0.67
25	1.50	1.19	0.72
30	1.54	1.23	0.76
40	1.62	1.31	0.84
60	1.73	1.43	0.95
80	1.81	1.53	1.04
100	1.88	1.61	1.13
120	1.93	1.67	1.20
160	2.02	1.79	1.31
200	2.10	1.87	1.42
300	2.23	2.05	1.63
400	2.34	2.19	1.80

* Values for intermediate heights above 15 ft (4.6 m) may be interpolated.
† Multiply by 0.305 for meters.

Table 12.2.2 Pressure Coefficients C_q

Structure or part thereof	Description	C_q factor
1. Primary frames and systems	**Method 1** (normal force method)	
	Walls:	
	Windward wall	0.8 inward
	Leeward wall	0.5 outward
	Roofs[a]:	
	Wind perpendicular to ridge	
	Leeward roof or flat roof	0.7 outward
	Windward roof	
	Less than 2:12 (16.7%)	0.7 outward
	Slope 2:12 (16.7%) to less than 9:12 (75%)	0.9 outward or 0.3 inward
	Slope 9:12 (75%) to 12:12 (100%)	0.4 inward
	Slope > 12:12 (100%)	0.7 inward
	Wind parallel to ridge and flat roofs	0.7 outward
	Method 2 (projected area method)	
	On vertical projected area	
	Structures 40 feet (12 192 mm) or less in height	1.3 horizontal any direction
	Structures over 40 feet (12 192 mm) in height	1.4 horizontal any direction
	On horizontal projected area[a]	0.7 upward
2. Elements and components not in areas of discontinuity[b]	Wall elements	
	All structures	1.2 inward
	Enclosed and unenclosed structures	1.2 outward
	Partially enclosed structures	1.6 outward
	Parapets walls	1.3 inward or outward
	Roof elements[c]	
	Enclosed and unenclosed structures	
	Slope < 7:12 (58.3%)	1.3 outward
	Slope 7:12 (58.3%) to 12:12 (100%)	1.3 outward or inward
	Partially enclosed structures	
	Slope < 2:12 (16.7%)	1.7 outward
	Slope 2:12 (16.7%) to 7:12 (58.3%)	1.6 outward or 0.8 inward
	Slope > 7:12 (58.3%) to 12:12 (100%)	1.7 outward or inward
3. Elements and components in areas of discontinuities[b,d,e]	Wall corners[f]	1.5 outward or 1.2 inward
	Roof eaves, rakes or ridges without overhangs[f]	
	Slope < 2:12 (16.7%)	2.3 upward
	Slope 2:12 (16.7%) to 7:12 (58.3%)	2.6 outward
	Slope > 7:12 (58.3%) to 12:12 (100%)	1.6 outward
	For slopes less than 2:12 (16.7%)	
	Overhangs at roof eaves, rakes or ridges, and canopies	0.5 added to values above
4. Chimneys, tanks, and solid towers	Square or rectangular	1.4 any direction
	Hexagonal or octagonal	1.1 any direction
	Round or elliptical	0.8 any direction
5. Open frame towers[g,h]	Square and rectangular	
	Diagonal	4.0
	Normal	3.6
	Triangular	3.2
6. Tower accessories (such as ladders, conduit, lights and elevators)	Cylindrical members	
	2 inches (51 mm) or less in diameter	1.0
	Over 2 inches (51 mm) in diameter	0.8
	Flat or angular members	1.3
7. Signs, flagpoles, lightpoles, minor structures[h]		1.4 any direction

[a] For one story or the top story of multistory partially enclosed structures, an additional value of 0.5 shall be added to the outward C_q. The most critical combination shall be used for design.
[b] C_q values listed are for 10-ft² (0.93-m²) tributary areas. For tributary areas of 100 ft² (9.29 m²), the value of 0.3 may be subtracted from C_q, except for areas at discontinuities with slopes less than 7 units vertical in 12 units horizontal (58.3% slope) where the value of 0.8 may be subtracted from C_q. Interpolation may be used for tributary areas between 10 and 100 square feet (0.93 m² and 9.29 m²). For tributary areas greater than 1,000 ft² (92.9 m²), use primary frame values.
[c] For slopes greater than 12 units vertical in 12 units horizontal (100% slope), use wall element values.
[d] Local pressures shall apply over a distance from the discontinuity of 10 ft (3.05 m) or 0.1 times the least width of the structure, whichever is smaller.
[e] Discontinuities at wall corners or roof ridges are defined as discontinuous breaks in the surface where the included interior angle measures 170° or less.
[f] Load is to be applied on either side of discontinuity but not simultaneously on both sides.
[g] Wind pressures shall be applied to the total normal projected area of all elements on one face. The forces shall be assumed to act parallel to the wind direction.
[h] Factors for cylindrical elements are two-thirds those for flat or angular elements.

Table 12.2.3 Wind Stagnation Pressure q_s at Standard Height of 33 ft

Basic wind speed, mi/h*	70	80	90	100	110	120	130
Pressure q_s, lb/ft^{2}†	12.6	16.4	20.8	25.6	31.0	36.9	43.3

* Multiply by 1.61 for kilometers per hour.
† Multiply by 0.048 for kilonewtons per square meter.

Boundary-layer wind tunnels, which simulate the variation of wind speed with height and gusting, are used to estimate the design wind loading.

The effect of the sudden application of **gust loads** has sometimes been blamed for peculiar failures due to wind. In most cases, these failures can be explained from the pressure distributions in a steady wind. If a relatively flexible structure such as a radio tower, chimney, or skyscraper with a natural period of 1 to 5 s is set into vibration, the stresses in the structure may be increased over those calculated from a static-load analysis. Provision should be made for this effect by increasing the static design wind. A rational analysis should be performed to calculate the magnitude of this increase, which will depend on the flexibility of the structure.

Even a steady wind may give rise to periodic forces which may build up into large vibrations and lead to failure of the structure when the frequency of the exciting force coincides with one of the natural frequencies of vibration of the structure. The periodic exciting force may be due to the separation of a system of **Kármán vortices** (Fig. 12.2.1) in the wake of the body. The exciting frequency n in cycles per second is related to d, the dimension of the body normal to the wind velocity V, by the equation $nd/V = C$, where $C \approx 0.207$ for circular cylinders and $C \approx 0.18$ for rectangular plates (Blenk, Fuchs, and Liebers, Measurements of Vortex Frequencies, *Luftfahrt-Forsch.*, 1935, p. 38). Dangerous vibrations related to the ‘‘flutter’’ of airplane wings may arise on bridges and similar flat bodies. These self-induced vibrations may be caused (1) by a negative slope of the curve of lift against angle of attack (Den Hartog, *op. cit.*) or (2) by a dynamic instability which arises when a body having two or more degrees of freedom (such as bending and torsion) moves in such a manner as to extract energy out of the air stream. The first of these vibrations will occur at one of the natural frequencies of the structure. The second type of vibration will occur at a frequency intermediate between the natural frequencies of the structure.

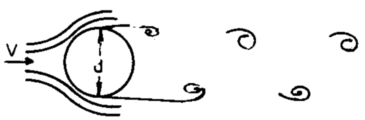


Fig. 12.2.1 Von Kármán vortices.

The wind forces and the **pressure distribution** over a structure corresponding to a design wind V can be determined by **model testing** in a wind tunnel. Extrapolation from model to full scale is based on the fact that at every other point on the body, the pressure p is proportional to the stagnation pressure q (see Sec. 11) and thus the ratio p/q = constant for a fixed point on the body, as the scale of the model or the velocity of the wind is changed. Since the principal component of the wind force is due to the pressures, the force F acting on the surface S is

$$F \approx p_{\mathrm{avg}} S = (p/q)_{\mathrm{avg}} qS$$

and denoting $(p/q)_{\mathrm{avg}}$ by a normal force coefficient or **shape factor** C_N

$$F = C_N \tfrac{1}{2} \rho V^2 S = C_N qS$$

The shape factors so obtained apply to full scale for structures with sharp edges whose principal resistance is due to the pressure forces. For bodies that do not have any sharp edges perpendicular to the flow, such as spheres or streamlined bodies, the factor C_N is not constant. It depends upon the Reynolds number (see Sec. 3). For such bodies, the law for variation of the shape factor C_N must be determined experimentally before safe predictions of full-scale forces can be made from model measurements.

Radio Towers and Other Framed Structures For open frame towers, by using round structural members instead of flat and angular sections, a substantial reduction in wind force can be effected. Refer to note *h* in Table 12.2.2.

Load Combinations Methods of combining types of loading vary with the governing local codes. Dead loads are usually considered to act all the time in combination with either full or reduced live, wind, earthquake, and temperature loads. The reductions in live loads are based on the improbability of fully loaded tributary areas, generally when the tributary area exceeds 150 ft^2. Most codes consider that wind and earthquake loads need not be taken to act simultaneously. There are two basic design methods in use: allowable stress or working stress design and limit states or ultimate strength design. Whereas allowable stress design compares the actual working stresses with an allowable value, the limit states approach determines the adequacy of each element by comparing the ultimate strength of the element with the factored design loads. The following basic load combinations are applicable for allowable stress design: dead plus live (floor and roof); dead plus live plus wind; dead plus live plus earthquake. Most codes permit a one-third increase in stresses for load combinations considering either wind or earthquake effects.

DESIGN OF STRUCTURAL MEMBERS

Members are usually proportioned so that stresses do not exceed allowable **working stresses** which are based on the strength of the material and, in the case of compressive stresses, on the stiffness of the element under compression. Internal forces and moments in **simple beams,** columns, and pin-connected truss bars are obtained by means of the equations of static equilibrium. **Continuous beams,** rigid frames, and other members characterized by practically rigid joints require for analysis additional equations derived from consideration of deflections and rotations.

Design may also be on the basis of the **ultimate strength** of members, the factor of safety being embodied in stipulated increases in the design loads. In steel-frame construction, the procedures of **plastic design** determine points where the material may be allowed to yield, forming *plastic hinges,* and the resulting redistribution of internal forces permits a more efficient use of the material.

Floors and Roofs

Except in reinforced-concrete flat-slab construction, floors and roofs generally consist of flat decks supported upon beams, girders, or trusses. The decks may usually be considered a series of beamlike strips spanning between beams and themselves designed as beams. The design of a beam consists chiefly in proportioning its cross section to resist the maximum bending and shear and providing adequate connections at its supports, without exceeding the unit stresses allowed in the materials (see Sec. 5) and limiting maximum live load deflection at midspan to $1/360$ of the span.

Up to spans of 20 to 30 ft (6.1 to 9.1 m), either wood or steel beams of uniform section are generally more economical than trusses, while for spans above 50 to 70 ft (15.2 to 21.3 m), trusses are usually more economical. Between these limits, the line of economy is not well-defined. Conditions that favor the use of trusses are as follows: (1) identical trusses are repeated many times, (2) the height of the building need not be increased for the greater depth of the truss (3) fire protection of wood or metal is not required.

Roof trusses often have their top chords sloped with the roof. Common trusses for steeply pitched roofs are shown in Figs. 12.2.2 to 12.2.6, the top chord panels equal in each truss. The members shown by heavy lines are in compression under ordinary loads, those in light lines in tension. The trusses of Figs. 12.2.6 to 12.2.9 are adapted for either steel or wood. In wooden trusses, the tensile web members may be steel rods with plates, nuts, and threaded ends. The truss of Fig. 12.2.6 is usually made of steel.

The forces in any member of these trusses under a vertical load uniformly distributed may be found by multiplying the coefficients in Tables 12.2.4 to 12.2.8 by the panel load P on the truss. For other slopes, types of trusses, or loads, see ''General Procedure'' below. Trusses for flat roofs are commonly of one of the types shown in Figs. 12.2.7 to 12.2.13 except that the top chords conform to the slope of the roof.

Floor trusses normally have parallel chords. Common types are shown in Figs. 12.2.7 to 12.2.13 in which heavy lines indicate members in compression, light lines in tension, and dash lines members with only nominal stress, under equal vertical panel loads. The panel lengths l in each truss are equal. The stress in each member is written next to the member in the figure, in terms of the panel load and the lengths of members. For a truss like one of the figures turned upside down, the stresses in the chords and diagonals remain the same in magnitude but

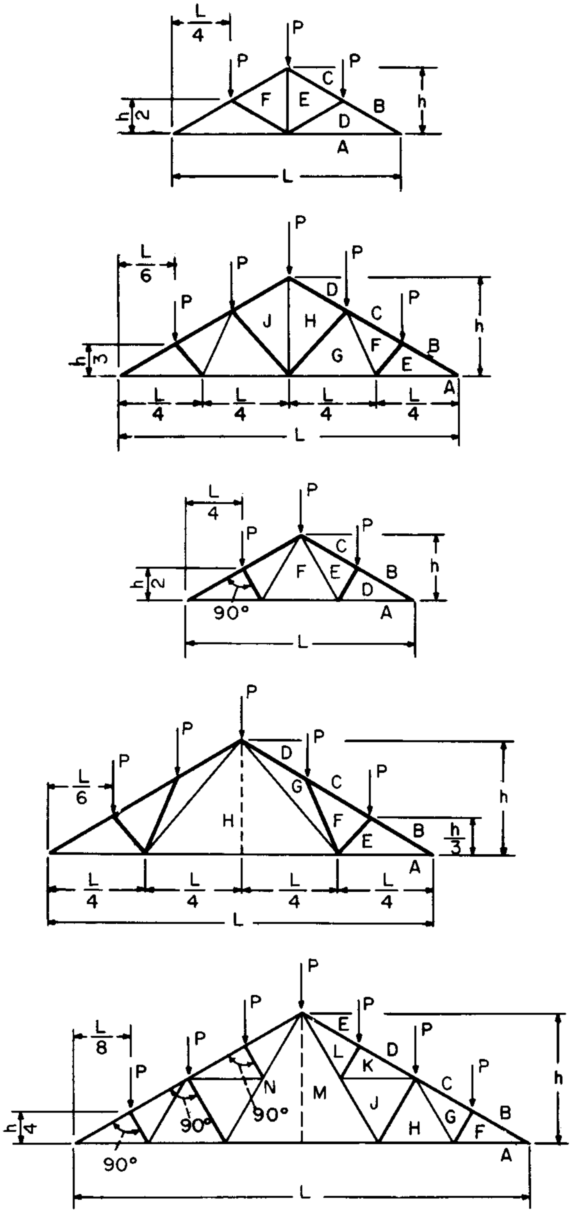


Fig. 12.2.2 to 12.2.6 Types of steep roof trusses.

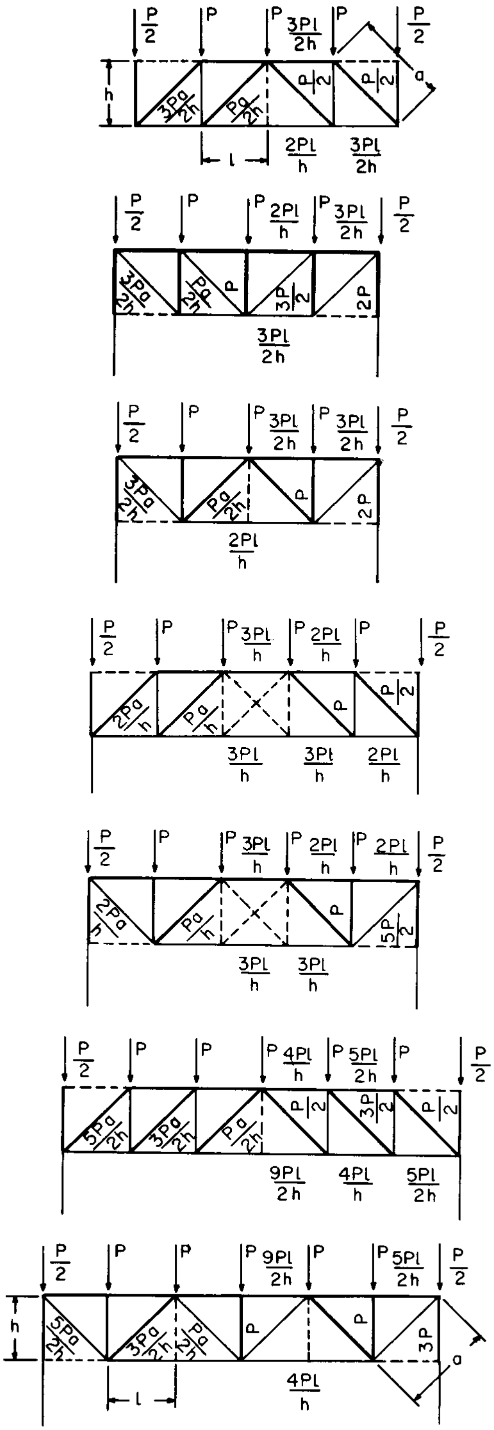


Fig. 12.2.7 to 12.2.13 Types of floor and roof trusses.

Table 12.2.4 Coefficients for Truss Shown in Fig. 12.2.2

	Coefficients of *P* for force in				
Pitch *h*/*L*	*AD*	*BD*	*CE*	*DE*	*EF*
⅓	2.25	2.71	1.80	0.90	1.00
0.288*	2.60	3.00	2.00	1.00	1.00
¼	3.00	3.35	2.24	1.12	1.00
⅕	3.75	4.03	2.69	1.35	1.00

Table 12.2.5 Coefficients for Truss Shown in Fig. 12.2.3

	Coefficients of *P* for force in								
Pitch *h*/*L*	*AE*	*AG*	*BE*	*CF*	*DH*	*EF*	*FG*	*GH*	*HJ*
⅓	3.75	3.00	4.50	2.7	3.60	0.83	0.72	1.25	2.00
0.288*	4.33	3.46	5.00	3.0	4.00	0.88	0.73	1.32	2.00
¼	5.00	4.00	5.59	3.35	4.47	0.94	0.75	1.42	2.00
⅕	6.25	5.00	6.74	4.04	5.39	1.07	0.79	1.60	2.00

Table 12.2.6 Coefficients for Truss Shown in Fig. 12.2.4

	Coefficients of *P* for force in					
Pitch *h*/*L*	*AD*	*AF*	*BD*	*CE*	*DE*	*EF*
⅓	2.25	1.50	2.70	2.15	0.83	0.75
0.288*	2.60	1.73	3.00	2.50	0.87	0.87
¼	3.00	2.00	3.35	2.91	0.89	1.00
⅕	3.75	2.50	4.04	3.67	0.93	1.25

Table 12.2.7 Coefficients for Truss Shown in Fig. 12.2.5

	Coefficients of *P* for force in							
Pitch *h*/*L*	*AE*	*AH*	*BE*	*CF*	*DG*	*EF*	*FG*	*GH*
⅓	3.75	2.25	4.51	3.91	4.51	0.83	1.42	2.50
0.288*	4.33	2.60	5.00	4.33	5.00	0.88	1.45	2.65
¼	5.00	3.00	5.59	4.84	5.59	0.94	1.49	2.83
⅕	6.25	3.75	6.73	5.83	6.73	1.07	1.57	3.20

Table 12.2.8 Coefficients for Truss Shown in Fig. 12.2.6

	Coefficients of *P* for force in										
Pitch *h*/*L*	*AF*	*AH*	*AM*	*BF*	*CG*	*DK*	*EL*	*FG* *KL*	*GH* *JK*	*HJ*	*JM*	*LM*
⅓	5.25	4.50	3.00	6.31	5.75	5.20	4.65	0.83	0.75	1.66	1.50	2.25
0.288*	6.06	5.20	3.46	7.00	6.50	6.00	5.50	0.87	0.87	1.73	1.73	2.60
¼	7.00	6.00	4.00	7.83	7.38	6.93	6.48	0.89	1.00	1.79	2.00	3.00
⅕	8.75	7.50	5.00	9.42	9.05	8.68	8.31	0.93	1.25	1.86	2.50	3.75

* 30 slope.

reversed in sign. Forces in verticals must be computed (compare Figs. 12.2.7 and 12.2.8). For other loads and other types of trusses see ''General Procedure'' below.

Weights of Trusses The approximate weight in pounds of a wooden roof truss may be taken as $W = LS(L/25 + L^2/6{,}000)$, where L is the span and S the spacing of trusses in feet. The approximate weight in pounds of a steel roof truss may be taken as $W = \frac{1}{5}LS(\sqrt{L} + \frac{1}{8}L)$.

Choice of Roof Trusses

WOODEN TRUSSES. For pitched roofs with spans up to 20 ft (6.1 m) the simple king-post truss (Fig. 12.2.2) may be used. For spans up to 40 ft (12.2 m) the trusses of Figs. 12.2.2 and Fig. 12.2.4 are good. For spans up to 60 ft (18.3 m) the trusses of Figs. 12.2.3 and 12.2.5 are good. The number of panels rarely exceeds eight or the panel length, 10 ft (3m). For flat roofs, the Howe truss (Figs. 12.2.7, 12.2.10, and 12.2.12) is built of wood with steel rods for verticals; the depth is one-eighth to one-twelfth the span. Wooden trusses are usually spaced 10 to 15 ft (3.0 to 4.6 m) on centers. Wood is rarely used in roof trusses with span over 60 ft (18.3 m) long. Steel trusses for pitched roofs may well take the form shown in Figs. 12.2.3 to 12.2.6 for spans up to 100 ft (30.5 m). For flat roofs, the Warren truss (Figs. 12.2.9, 12.2.11, and 12.2.13) is usually constructed in steel; the depth of the trusses ranges from one-eighth to one-twelfth the span, with trusses spaced from 15 to 25 ft (4.6 to 7.6 m) on centers.

Stresses in Trusses

An ideal truss is a framework consisting of straight bars or members connected at their ends by frictionless ball-and-socket joints. The external forces are applied only at these ball-and-socket joints. Internal forces and stresses in such straight bars are axial, either tension or compression, without bending. Since frictionless ball-and-socket joints are impossible, and the ends of bars are often bolted or welded, the ideal truss is never realized. For purposes of analysis, the primary stresses, which are always axial, are determined on the assumption that the truss under consideration conforms to the ideal. Secondary stresses are additional stresses, generally flexural or bending, brought about by all the factors that make the actual truss different from the ideal. In the following discussion, only primary forces and stresses will be considered.

Analytical Solution of Trusses

General Procedure After all external forces (loads and reactions) have been determined, the internal force or stress in any member is found (1) by taking a section, making an imaginary cut through the members of the truss, including the one whose stress is to be found, so as to separate the truss into two parts; (2) by isolating either of these parts; (3) by replacing each bar cut by a force, representing the force in the bar; and (4) by applying the equations of statics to the part isolated.

The various ways in which sections can be taken and the equations used to determine the forces are illustrated by a solution of the truss in Fig. 12.2.14.

A section 1–1 (Fig. 12.2.14) may be taken around a joint, L_0. Isolate the forces inside the section (Fig. 12.2.15*a*). Assume the unknown forces to be tension. Since the forces are concurrent and coplanar, two independent equations of statics will establish equilibrium. These may be either $\Sigma x = 0$ and $\Sigma y = 0$ or $\Sigma M = 0$ taken about two axes perpendicular to the plane of the forces and passing through two points selected so that neither point is the intersection of the forces, or so that the line joining the two points is not coincident with either of the unknown forces.

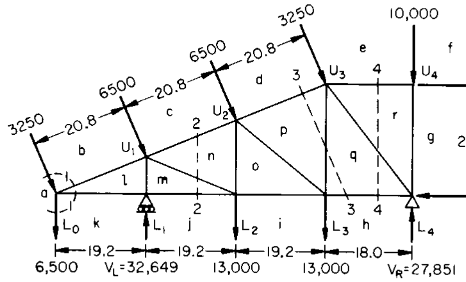


Fig. 12.2.14

Using the first set of equations and taking components of all forces along horizontal and vertical axes; e.g., the horizontal component of the 3,250 lb force is 1,250 and the vertical component is 3,000 lb.

$$\Sigma x = 0 = s_{lk} + s_{bl}(19.2/20.8) + 1,250$$
$$\Sigma y = 0 = s_{bl}(8/20.8) - 9,500$$

From these equations, $s_{lk} = -24,050$ and $s_{bl} = 24,700$.

The minus sign indicates that the force acts opposite to the assumed direction. If all the unknown forces are assumed to be in tension, then a plus sign in the result indicates that the force is tension and a minus sign indicates compression.

Hence, s_{lk} is 24,050 lb compression and s_{bl} is 24,700 lb tension.

Using the $\Sigma M = 0$ twice,

$$\Sigma M \text{ about } U_1 = 0 = -s_{lk} \times 8 - 1250 \times 8 - 9,500 \times 19.2$$

from which $S_{lk} = 24,050$ lb compression.

$$\Sigma M \text{ about } L_1 = 0 = s_{bl}(8/20.8)19.2 - 9,500 \times 19.2$$

from which $s_{bl} = 24,700$ tension.

Instead of taking a section around a joint, a cut may be made vertically or inclined, cutting a number of bars such as Fig. 12.2.15*b* or Fig. 12.2.15*c*. If only three members are cut, and they are neither concurrent nor parallel, the forces can be found by taking moments of all forces on either side of the section about axes passing through the intersections of any two members.

Considering the part to the left of section 2–2, the forces in the three members (Fig. 12.2.15*b*) may be determined by taking moments about L_0, U_1, and L_2 of all forces acting on the part on either side of the section, e.g., on the left of the section because it has the fewer forces:

$$\Sigma M \text{ about } L_0 = s_{mn}(8/20.8)38.4 + 2,500 \times 8 - (32,649 - 6,000)19.2$$
$$\Sigma M \text{ about } U_1 = 0 = -s_{mj} \times 8 - 1,250 \times 8 - 9,500 \times 19.2$$
$$\Sigma M \text{ about } L_2 = 0 = s_{cn}(19.2/20.8)16 + 2,500 \times 8 - 9,500 \times 38.4 + 26,649 \times 19.2$$

from which $s_{mn} = 33,290$ tension, $s_{mj} = 24,050$ compression, and $s_{cn} =$ 11,298 compression.

This method is sometimes called the ''method of moments.''

Considering the part to the right of section 3–3, the forces in the three members (Fig. 12.2.15*c*) may be determined by taking moments about L_0, U_3, L_3 of all the forces acting on part or either side of the section, e.g., on the right of the section because it has the fewer forces:

$$\Sigma M \text{ about } L_0 = 0 = s_{pq} \times 57.6 + 6,250 \times 24 + 3,000 \times 57.6 - (27,851 - 10,000) \times 75.6$$
$$\Sigma M \text{ about } L_3 = 0 = -s_{dp}(19.2/20.8)24 + 6,250 \times 24 - 17,851 \times 18$$
$$\Sigma M \text{ about } U_3 = 0 = s_{qh} \times 24 + 12,500 \times 24 - 17,851 \times 18$$

from which $s_{pq} = 17,825$ tension, $s_{dp} = 7,733$ compression, and $s_{qh} =$ 888 tension.

Considering the part to the right of section 4–4, the forces in the three members (Fig. 12.2.15*d*) may be found by taking moments about axes where two unknowns intersect, e.g., about U_3 and L_4. Since two unknown forces are parallel, their lines of action do not intersect. However, the equation $\Sigma y = 0$ will enable one to find the force in the member which is not parallel to the other two:

$$\Sigma M \text{ about } U_3 = 0 = s_{qh} \times 24 - (27,851 - 10,000)18 + 12,500 \times 24$$
$$\Sigma M \text{ about } L_4 = 0 = -s_{er} \times 24 + 5,000 \times 24$$
$$\Sigma y = 0 = s_{rq}(24/30) + 27,851 - 10,000$$

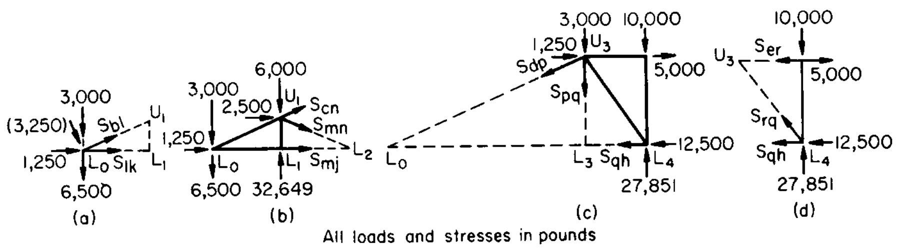


Fig. 12.2.15 Resolution of forces for truss shown in Fig. 12.2.14.

from which $s_{qh} = 888$ tension, $s_{er} = 5{,}000$ tension, and $s_{rq} = 22{,}314$ compression.

Columns and Walls

Vertical elements in building construction consist of columns, posts, or partitions that transmit concentrated loads; walls or partitions that transmit linearly distributed loads (and may, if so designed, transmit lateral loads from story to story); rigid frames that transmit lateral as well as vertical loads; and braced frames that, ideally, transmit only lateral loads from story to story.

Columns may be of timber, steel, or reinforced concrete and should be proportioned for the allowable stresses permitted for the material used and for the flexibility of the column. Care must be taken in the framing of beams and girders to avoid or to provide for the additional stresses due to connections which transfer loads to columns with large eccentricities. For instance, a beam framing to a flange of a steel column, with a seat or web connection, will produce an eccentric moment in the column equal to $Rd/2$, where R is the beam reaction and d the depth of the column section. Columns not adequately restrained against deflection of the top may be subject to considerable additional moment because of the resulting eccentricity of otherwise axial loads.

Rigid frames consist of columns and beams welded, bolted, or otherwise connected so as to produce continuity at the joints and permit the entire frame to behave as a unit, capable of resisting both vertical and lateral loads. Advantages of rigid frames are the ease and simplicity of erection, increased headroom, and more open floor plans free of braced frames. Rigid frames composed of rolled sections are commonly used for spans up to 100 ft (30.5 m) in length. Built-up members have been utilized on spans to 250 ft (76.2 m). Welded fabrication offers particular advantages for frames utilizing variable-depth members and on parabolic-shape roofs. The distribution of moments in the statically indeterminate rigid frame is effected by the relationship of the column height to span and roof rise to column height, as well as the relative stiffness of the various members. The solution for the moments in the frame is obtained from the usual equations of statics plus one or more additional equations pertaining to the elastic deformations of the frame under load.

Bearing walls, which are intended to transmit gravity loads from story to story, are designed as vertical elements of unit width such that the combined effects of axial and/or bending stresses do not exceed their allowable values according to the formula $fa/Fa + fb/Fb \leq 1.0$, where f is the actual stress in the axial or bending mode, and F is the allowable stress in the appropriate mode.

Shear walls, either concrete or masonry, which are intended to transmit earthquake, wind, or other lateral loads from story to story parallel to the plane of the wall, are designed as cantilevered vertical shear beams. Loads are distributed among the various wall elements, created by door and window openings, on the basis of their relative rigidities. Unless special conditions indicate otherwise, the assumption is made, in calculating individual stiffnesses, that the wall is fixed against rotation, in the plane of the wall, at the bottom. The individual wall elements are then designed to resist both their share of the wall shear and the moments it induces as well as any vertical loads due to bearing wall action and/or overall wall overturning. Individual wall elements, then, will have special vertical reinforcing at each side (trim reinforcement) to resist bending moments. Diagonal trim reinforcements, 96 bar diameters in total length, where possible, are placed at corners of openings in concrete walls to inhibit cracking. Trim reinforcing should consist of a minimum of two ⅝-in-diameter (1.6-cm) bars (No. 5 bars).

Uplift and downward forces at opposite ends of the shear walls, due to the tendency of the wall to overturn in its own plane when acted upon by lateral forces, must be resisted by vertical reinforcement at those locations. Special boundary elements are required for shear walls in areas of high seismicity (UBC Zones 3 and 4). Only minimum tributary dead loads may be counted upon to help resist uplift while maximum dead plus live loads should be assumed acting simultaneously with downward overturning loads. **Load reversal,** due to wind or seismic loads, must be considered.

Shear walls should be positioned throughout the plan area of the building in such a manner as to distribute the forces uniformly among the individual walls with approximately one-half of the wall area oriented in each of the two major directions of the building. At least one major wall should be positioned, if possible, near each exterior face of the building. Shear walls should be continuous, where possible, from top to bottom of the building; avoid offsetting walls from floor to floor, and particularly avoid discontinuing walls from one level to the level below. Positioning shear walls near major floor openings that would preclude proper anchorage of floor (or roof) to wall should be avoided so that diaphragm forces in the plane of the floor may have an adequate force path into the wall.

Where **rigid diaphragms** (such as concrete slabs or metal deck with concrete fill) exist, shears are distributed to the individual walls on the basis of their relative rigidity taking into account the additional forces due to the larger of either accidental or actual horizontal torsion resulting from the eccentricity of the building seismic shear (located at the center of mass) or eccentricity of applied wind load with respect to the center of rigidity of the walls. Where **flexible diaphragms** (such as metal deck with nonstructural fill or plywood) exist, shears are distributed to walls on the basis of tributary area. When diaphragms are semirigid, based on type, construction, or spacing between supporting elements, shears could be distributed by both methods (relative rigidity and tributary area) with individual walls designed to resist the larger of the two shears.

Braced frames, which are intended to transmit lateral loads from floor to floor, are designed as trusses that are loaded horizontally instead of vertically and are principally used in steel construction. Lateral loads are distributed to them on the same basis as to shear walls; therefore, the same general rules apply to their ideal placement. Bracing of individual bays may take the form of X-bracing, single diagonal bracing, or K-bracing. Where K-braces are used, the additional bracing forces associated with vertical floor loads to the braced beam must be considered. Alternative floor framing to avoid large beam loads to the braced beam should be considered where possible.

X-bracing may be designed so that either the tension diagonal takes all the lateral load or so that the tension and compression diagonals share the load. When the tension diagonal resists all the lateral load, the minimum slenderness ratio (kl/r) of the member should not exceed 300; the additional axial load in the column to which the top of the brace is connected, the connection load at each end of the brace and the horizontal footing load at the bottom of the brace, is double that obtained when braces share the load. In X-bracing systems where the tension and compression members share the lateral load, maximum slenderness ratios should be limited to 200 (the members may be considered to brace each other about axes that are normal to the plane of the bay), but connection and individual horizontal foundation loads are decreased. With either method of design, it is important to follow the path of the force from its origin through the structure until it is transferred to the ground. With any bracing system, it is wise to sketch to scale the major connection details prior to finalization of calculations; the desirable intersection of member axes is often more difficult to achieve than a single line drawing would indicate. Connection of braces to columns at intermediate points between floors should be avoided because plastic hinges in columns lead to structural instability.

Stud walls consist of wooden studs with one or more lines of horizontal bridging. The allowable vertical load on the wall is a function of the maximum permissible load on each stud as a column and the spacing of the studs.

Corrugated or flat sheet steel or aluminum, are commonly employed for walls of industrial or mill buildings. These sheets are usually supported on steel girts framing horizontally between columns and supported from heavier eave struts by one or more lines of vertical steel sag rods.

Reinforced-concrete or masonry walls should be designed so that the allowable bending and/or axial stresses are not exceeded, but the minimum thicknesses of such walls should not be less than the following:

Material	Max ratio, unsupported height or length to thickness	Nominal min thickness, in*
Reinforced concrete	25	6
Plain concrete	22	7
Reinforced brick masonry	25	6
Grouted brick masonry	20	6
Plain solid masonry	20	8
Hollow-unit masonry	18	8
Stone masonry (ashlar)	14	16
Interior nonbearing concrete or masonry (reinforced)	48	2
Interior nonbearing concrete or masonry (unreinforced)	36	2

* × 25.4 = mm.

Foundations

Bearing Pressure of Soils The bearing pressure which may be allowed on soil may vary over a large range. For important structures, the nature of the underlying soil should be ascertained by borings or test pits. If the soil consists of medium or soft clay, a settlement analysis based on consolidation tests of undisturbed soil samples from the foundation strata is necessary. Structures founded upon mud, soft clay, silt, peat, or artificial filling will almost certainly settle, and no foundation for a permanent structure should rest on or above such material without adequate provision for the resulting settlement. Table 12.2.9 gives a general classification of soils and typical safe pressures which they may support.

These values approximate the pressures allowed by the building law in most cities. Actual allowable bearing pressures should be based on the quality and engineering characteristics of the material obtained from analysis of borings, standard penetration tests, and rock cores. Other laboratory tests, when the cost thereof is warranted by the magnitude of the project, may show higher values to be safe. The foundation for a building housing heavy vibrating machinery such as steam hammers, heavy punches, and shears should receive some allowance for possible compression and rearrangement of soil due to the vibrations transmitted through it. The foundation for a tall chimney should be designed with a comparatively low pressure upon the soil, because of the disastrous results which might occur from local settlement.

Footings The purpose of footings is to spread the concentrated loads of building walls and columns over an area of soil so that the unit pressure will come within allowable limits. Footings are usually constructed of concrete, placed in open excavations with or without sheeting and bracing. In the past, stone, where available in quantity and the proper quality, has been used economically for residential building footings.

Table 12.2.9 Safe Bearing of Soils

Nature of soil	Safe bearing capacity	
	tons/ft²	MPa
Solid ledge of hard rock, such as granite, trap, etc.	25–100	2.40–9.56
Sound shale and other medium rock, requiring blasting for removal	10–15	0.96–1.43
Hardpan, cemented sand, and gravel, difficult to remove by picking	8–10	0.76–0.96
Soft rock, disintegrated ledge; in natural ledge, difficult to remove by picking	5–10	0.48–0.96
Compact sand and gravel, requiring picking for removal	4–6	0.38–0.58
Hard clay, requiring picking for removal	4–5	0.38–0.48
Gravel, coarse sand, in natural thick beds	4–5	0.38–0.48
Loose, medium, and coarse sand; fine compact sand	1.5–4	0.15–0.38
Medium clay, stiff but capable of being spaded	2–4	0.20–0.38
Fine loose sand	1–2	0.10–0.20

Allowable bearing pressures for granular materials are typically determined as 10 percent of the standard penetration resistances (N values) obtained in the field from standard penetration tests (STPs) and are in tons/ft^2.

Concrete footings may be either plain or reinforced. Plain concrete footings are generally limited to one- or two-story residential buildings. The center of pressure in the wall or column should always pass through the center of the footing. Where columns, because of fixity, impose a bending moment on the footing, the maximum soil pressures, due to the combined axial and moment components, shall be positive throughout the area of the footing (i.e., no net uplift) and shall be less than the allowable values. Footings in ground exposed to freezing should be carried below the possible penetration of frost.

Deep foundations are required when a suitable bearing soil is deep below the surface, typically more than 10 to 15 ft. (3.0 to 4.6 m). Sometimes deep foundations are necessary even if the suitable bearing materials are shallower, but groundwater conditions make excavation for footings difficult. The most common types of deep foundations include caissons or drilled piers and piles. Where the bearing soil is clay stiff enough to stand with undercutting, and the material immediately above it is peat or silt, the **open-caisson method** may be economical. In this method, cylindrical steel casings 3 ft and more in diameter are sunk as excavation proceeds, the casings having successively smaller diameters. At the bottom of the shaft thus formed, the soil is undercut to obtain sufficient bearing area. The shaft and the enlargement at the base are then filled with concrete, the cylinders sometimes being withdrawn as the concrete is placed. The open-caisson method cannot be used where groundwater flows too freely into the excavation. Where large foundations under very heavy buildings must be carried to great depth to reach rock or hardpan, particularly where groundwater flows freely, the **pneumatic-caisson method** is used.

Drilled-in piers are formed by drilling with special power augers up to 5 ft (1.5 m) diameter or greater. The holes are drilled to the desired bearing level with or without metal casings, depending on the soil conditions. Belling of the bottom may also be performed mechanically from the surface. In poor soils, or where groundwater is present, the hole may be retained with bentonite clay slurry which is displaced as concrete is placed in the caisson by the **tremie method.**

Pile Foundations Piles for foundations may be of wood, concrete, steel, or combinations thereof. Wood piles are generally dressed and, if required, treated offsite; concrete piles may be prepared offsite or cast in place; steel piles are mill-rolled to section. They can be driven, jacked, jetted, screwed, bored, or excavated. Wood piles are best suited for loads in the range of 15 to 30 tons (133.4 to 177.9 KN) per pile and lengths of 20 to 45 feet (6.6 to 15 m). They are difficult to splice and may be driven untreated when located entirely below the permanent water table; otherwise piles treated with creosote should be used to prevent decay. Wood piles should be straight and not less than 6 in in diameter under the bark at the tip. Concrete piles are less destructible, and hence are adaptable to many conditions, including driving in dense gravels, and can be up to approximately 120 ft (40 m) long. Concrete piles are divided into two classes: those poured in place and those precast, cured, and driven. Cast-in-place piles are made by driving a mandrel into the ground and filling the resulting hole with concrete. In one well-known pile of this type (Raymond), a thin sheet-steel corrugated shell is fitted over a tapered mandrel before driving. This shell, which is left in the ground when the mandrel is removed, is filled with concrete. Prestressing of precast-concrete piles provides greater resistance to handling and driving stresses. With a concrete pile, 25 to 60 tons (222.4 to 533.8 KN) or more per pile are carried. Structural steel H columns and steel pipe with capacities of 40 to 300 tons (350 to 1800 kN) per pile have been driven. Experience has shown that corrosion is seldom a practical problem in natural soils, but if otherwise, increasing the steel section to allow for corrosion is a common solution.

Methods of Driving Piles The drop hammer and the steam hammer are usually employed in driving piles. The steam hammer, with its comparatively light blows delivered in rapid succession, is of advantage in a plastic soil, the speed with which the blows are delivered preventing the readjustment of the soil. It is also of advantage in soft soils where the driving is easy, but a light hammer may fail to drive a heavy pile satisfactorily. The water jet is sometimes used in sandy soils. Water supplied under pressure at the point of the pile through a pipe or hose run alongside it erodes the soil, allowing the pile to settle into place. To have full capacity, jetted piles should be driven after jetting is terminated particularly if the pile is to resist uplift loads.

Determination of Safe Loads for Piles Piles may obtain their supporting capacity from friction on the sides or from bearing at the point. In the latter case, the bearing capacity may be limited by the strength of the pile, considered as a column, to which, however, the surrounding soil affords some lateral support. In the former case, no precise determination of the bearing capacity can be made. Many formulas have been developed for determining the safe bearing capacity in terms of the weight of the hammer, the fall, and the penetration of the pile per blow, the most generally accepted of which is that known as the *Engineering News* formula: $R = 2wh/(s + 1.0)$ for drop hammers, $R = 2wh/(s + 0.1)$ for single-acting steam hammers, $R = 2E/(s + 0.1)$ for double-acting steam hammers, where R = safe load, lb; w = weight of hammer, lb; h = fall of hammer, ft; s = penetration of last blow, in; E = rated energy, ft·lb per blow. This formula and similar ones are based on the determination of the energy in the falling hammer, and from this the pressure which it must exert on the top of the pile. The *Engineering News* formula is currently used typically for timber piles with capacities not exceeding 30 tons. It assumes a factor of safety of 6. It is a wise practice to drive index piles and determine their bearing capacity through pile load tests typically carried to twice the service load of the pile before proceeding with the final design of important structures to be supported on piles. The design then is based on the safe service load capacity so determined, and the piles are driven to the same penetration resistance to which the successfully tested index piles where driven by the same driving hammer.

Spacing of Piles Wood piles are preferably spaced not closer than 2½ ft (0.76 m), and concrete piles 3 ft (0.91 m) on centers. If driven closer than this, one pile is liable to force another up. Piles in a group must not cause excessive pressure in soil below their tips. The efficiency, or supporting value of friction piles when driven in groups, by the Converse-Labarre method, is

$$\frac{1 - d/s[(n-1)m + (m-1)n]}{90mn}$$

where d = pile diameter, in (cm); s = spacing center to center of piles, in (cm); m = number of rows; n = number of piles in a row.

Capping of Piles Piles are usually capped with concrete; wood piles sometimes with pressure-treated timber. Concrete is the most usual material and the most satisfactory for the reason that it gets a full bearing on all piles. The piles should be embedded 4 to 6 in (10 to 15 cm) in the concrete.

Retaining Walls A wall used to sustain the pressure of earth behind it is called a retaining wall. Retaining walls which depend for their stability upon the weight of the masonry are classed as gravity walls. Such walls built on firm soil will usually be stable when they have the following proportions: top of fill level, back vertical, base, 0.4 height; top of fill level, back battered, base, 0.5 height; top of fill steeply inclined, back vertical, base, 0.5 height; top of fill steeply inclined, back battered, base, 0.6 height. An additional factor of safety is obtained by building the face on a batter. Care should be taken in the design of a wall that the allowable soil pressure is not exceeded and that drainage is provided for the back of the wall. The foundations of retaining walls should be placed below the level of frost penetration. Retaining walls of reinforced concrete are made thin, with a broad base, and the wall either cantilevered from the base or braced with buttresses or counterforts.

It is impossible to derive formulas for the earth pressure on the back of the wall which will take account of all the actual conditions. Assuming the earth to be a loose, homogeneous, granular mass, and the coefficient of friction to be independent of the pressure, Rankine deduced the following formula for a wall with vertical back:

$$P = (\tfrac{1}{2}wh^2 + vh)\cos d(\cos d - \sqrt{\cos^2 d - \cos^2 a})/(\cos d + \sqrt{\cos^2 d - \cos^2 a})$$

the center of pressure being at a height $\tfrac{1}{3}H(wh + 3v)/(wh + 2v)$ above the base, where P = earth pressure per lin ft (m) of the wall, lb (kg); h = height of the wall, ft (m); w = weight of earth per ft^3 (m^3), lb (kg); v = weight of superposed load per ft^2 (m^2) of surface, lb (kg); d = the angle with the horizontal of the earth surface behind the wall; and a = angle of internal friction of the earth, deg. (For sands, a = 30 to 38°.) The direction of the pressure is parallel to the earth's surface. The retaining wall should have sufficient thickness at the base so that the resultant of the earth pressure P combined with the weight of the wall falls well within the base. If this resultant falls at the outside edge of the middle third, the maximum vertical pressure on the foundation (at the outer edge of the base) will be equal to $2W/T$ lb/ft^2 (Pa), where W is the total vertical pressure on the base of 1 ft (m) length of wall and T the thickness of the wall at the base, ft (m).

In the design of walls of buildings which must withstand earth pressure and low independent walls, where refinement is not necessary, the earth pressure is frequently assumed to be that of a fluid weighing 40 to 45 lb/ft^3 (641 to 721 kg/m^3).

MASONRY CONSTRUCTION

The term **masonry** applies to assemblages consisting of fired-clay, concrete, or stone units, mortar, grout, and steel reinforcement (if required). The choice of materials and their proportions is based on the required strength (compressive, flexural, and shear), fire rating, acoustics, durability, and aesthetics. The strength and durability of masonry depends on the size, shape, and quality of the unit, the type of mortar, and the workmanship. Resulting bond strength is affected by the initial rate of absorption, texture and cleanliness of the masonry units.

Brick Common red bricks are made of clay burned in a kiln. Quality characteristics are hardness and density. Light-colored brick is apt to be soft and porous. Brick for masonry exposed to the weather or where strength is desired should have a crushing strength of not less than 2,500 lb/in^2 (17.3 MPa) and should absorb not over 20 percent of water by weight, after 5-h immersion in boiling water (see Sec. 6).

Mortar Mortar is the bonding agent that holds the individual units together as an assembly. All mortars contain cement, sand, lime, and water in varying proportions. Mortars are classified as portland-cement-lime mortar or masonry cement mortar. Proportions for Type M portland-cement-lime mortar are 1 part portland cement, ¼ part lime, and 3 parts sand, by volume; for masonry cement mortar, 1 part masonry cement, 1 part portland cement, and 6 parts sand. Portland-cement-lime mortar should be used for all structural brickwork (see Sec. 6).

Laying and Bonding Brick should be laid in a full bed of mortar and shoved laterally into place to secure solid bearing and a bed of even thickness and to fill the vertical joints. Clay brick should be thoroughly wet before laying, except in freezing weather. Concrete bricks/blocks should not be wetted before placement. Brick laid with long dimension parallel to the face of the work are called stretchers, perpendicular to the face, headers. Bats (half-brick) should not be used except where necessary to make corners or to form patterns on the face of the wall. Each continuous vertical section of a wall one masonry unit thick is called a **wythe.** Multiwythe walls, of brick or block faced with brick, are now commonly tied together with ties or anchors between each wythe of brick and block. Such ties are typically provided every other course. When the wythes are adequately bonded or tied; the wall may be considered as a composite wall for strength purposes. Walls may also be tied together longitudinally by overlapping stretchers in successive courses. Transverse bond is obtained by making every sixth course headers, the headers themselves overlapping in successive courses in the interior of thick walls. Variations in the arrangement of headers are often used in the face of walls for appearance. The area of cross section of full-length headers should not be less than one-twelfth the face of the

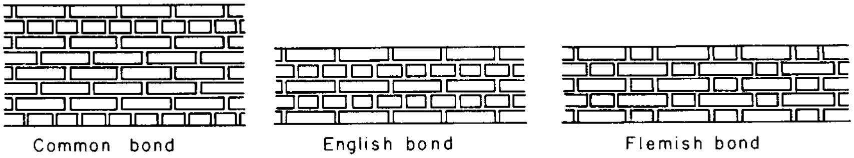


Fig. 12.2.16 Bonds used in bricklaying.

wall, in bonding each pair of transverse courses of brick. Three examples of bond are shown in Fig. 12.2.16.

Arches over windows and doorways are laid in concentric rings of headers on edge, with radial joints. The radius of the arch should be 1 to 1¼ times the width of the opening.

Lateral Support Brick walls should be supported laterally by bonding to transverse walls or buttresses, or by anchoring to floors, at intervals not exceeding 20 times the thickness. Floors and anchors must be capable of transmitting wind pressure and earthquake forces, acting outward, to transverse walls or other adequate supports, and thus to the ground. The height of piers between lateral supports should not exceed 12 times their least dimension.

Concrete Masonry Units (CMUs) Hollow or solid concrete blocks are used in building walls, often faced with brick or stone on the exterior walls. Standard block sizes are based on a brick module, nominally 4 in high.

Allowable compressive stresses in masonry are related to the masonry unit strength and the type of mortar as follows:

Kind of Masonry	Type M or S mortar lb/in^2	Type M or S mortar MPa	Type N mortar lb/in^2	Type N mortar MPa
Stone ashlar	360	2.48	320	2.20
Rubble	120	0.83	100	0.69
Clay brick (2500 lb/in^2)	160	1.10	140	0.97
Hollow CMU (1000 lb/in^2)	75	0.52	70	0.48
Solid CMU (2000 lb/in^2)	160	1.10	140	0.97

Minimum thickness of load-bearing masonry walls shall be 6 in for up to one story and 8 in for more than one story.

Reinforced Masonry The design of masonry with vertical reinforcing bars placed and fully grouted in some of the cells provides increased capacity to resist flexure and axial loads. In addition, in areas where seismic design is required, due to the brittle nature of unreinforced masonry, all masonry must be reinforced to prevent a brittle failure mode. The design principles for reinforced masonry are similar to the design for reinforced concrete.

Reinforced Concrete (See Sec. 12.3.)

TIMBER CONSTRUCTION

Floors The framing of wooden floors may be divided into two general types: joist construction and solid, or mill, construction. The first consists of **joists** 2 to 6 in wide, of the necessary depth, and spaced about 12 to 16 in (30 to 40 cm) on centers. The wall ends should rest on and be anchored to walls and the interior ends carried by a line of girders on columns. These joists should be securely cross-bridged not over 8 ft (2.4 m) apart in each span to prevent twisting and to assist in distributing concentrated loads. Solid blocking should be provided at ends and at each point of support. The floor is formed of a thickness of rough boarding on which the finish flooring is laid. **Solid** or **mill-construction floors** are designed to do away with the small pockets which exist in joist construction and thus reduce the fire hazard. They are generally framed with beams spaced 8 to 12 ft (2.4 to 3.6 m) on centers and spanning 18 to 25 ft (5.5 to 7.6 m). The wall ends of beams rest on and are anchored to the wall, and the interior ends are carried on columns and tied together to form a continuous tie across the building. Ends of timbers in masonry walls should have metal bearing plates and ½ in space at sides and end for ventilation, to prevent rot. The ends should be beveled and the anchors placed low to avoid overturning the wall if the beams drop in a fire. In all cases, care should be taken to provide sufficient bearing at the points of support so that the allowable intensity of compression across the grain is not exceeded. In case it is desirable to omit columns, or the floor load requires a closer spacing of beams, girders are run lengthwise of the building over the columns to take the beams, the ends of which are hung in hangers or stirrup irons and tied together, over or through the girders. This is called **intermediate framing.** Steel beams are sometimes used in place of wooden beams in this type of construction, in which case a wooden strip is bolted to the top flange of the beam to take the nailing of the plank, or the plank is laid directly on top of the beam and secured by spikes driven from below and clinched over the flange. The floor is formed of 3 or 4 in (7.5 or 10 cm) plank grooved in each edge, put together with splines and securely spiked to beams. On

Table 12.2.10 Properties of Plank and Solid Laminated Floors
(b = breadth = 12 in, f = fiber stress)

Nominal thickness or depth in (1)	Actual thickness d, in (S4S) (2)	Area of section $A = bd$, in^2 (3)	Moment of inertia $I = bd^3/12$, in^4 (4)	Section modulus $S = bd^2/6$, in^3 (5)	Safe load, lb/ft^2 on 1-ft span* f = 1,000 lb/in^2† (6)	Safe load, lb/ft^2 on 1-ft span* f = 1,600 (7)	Coef of deflection, uniform load‡ E = 1,000,000 (8)	Coef of deflection, uniform load‡ E = 1,760,000 (9)
1	¾	9.00	0.422	1.13	753	1,205	53.4	93.98
1½	1¼	15.00	1.95	3.13	2,085	3,336	11.51	20.26
2	1½	18.00	3.38	4.50	3,000	4,800	6.66	11.72
2½	2	24.00	8.00	8.00	5,334	8,534	2.82	4.96
3	2½	30.00	15.60	12.50	8,334	13,334	1.441	2.54
4	3½	42.00	42.9	24.5	16,334	26,134	0.524	0.922
5	4½	54.00	91.1	40.5	27,000	43,200	0.247	0.1404
6	5½	66.00	166.4	60.5	40,300	64,500	0.1348	0.0765
8	7½	90.00	422	112.5	75,000	120,000	0.0533	0.0303
10	9½	114.00	857	180.5	120,400	192,500	0.0263	0.0149
12	11½	138.00	1,521	264.5	176,400	282,000	0.0148	0.0084

NOTE: 1 in = 2.54 cm; 1 lb = 4.45 N; 1 lb/in^2 = 6.89 kPa.
* Divide tabular value by square of span in feet.
† For other fiber stress f, multiply tabular value by f/1,000.
‡ For deflection in, multiply coefficient by load, lb/ft^2, and by fourth power of span in ft, and divide by 1,000,000. For other modulus of elasticity E, multiply coefficient of col. 8 by 1,000,000, and divide by E.

top of the plank is laid flooring, with a layer of sheathing paper between. In case the floor loads require an excessive thickness of plank, or in localities where heavy plank is not easily obtainable, the floor is built up of 3 × 6 in (7.5 × 15 cm), or other sized pieces, placed on edge, and securely nailed together.

The roofs of buildings of joist and mill construction are framed in a manner similar to the floors of each type and should be securely anchored to the walls and columns. In case columns are not desired in the top story, steel beams or trusses of either steel or wood are used. For spans up to 35 ft (10.7 m), trussed beams can often be used to advantage.

For **unit stresses in timber,** see Sec. 6. For unit stresses in wooden columns, see Table 12.2.12. Table 12.2.10 gives the properties of mill floors made of dressed plank, and of laminated floors made of planks of edge, laid close.

Timber Beams

Properties of Timber Beams Table 12.2.11 presents those properties of wooden timbers most useful in computing their strength and deflection as beams. (The ''nominal size'' of a timber is indicated by the breadth and depth of the section in inches. The ''actual size'' indicates the size of the dressed timber, according to National Lumber

Table 12.2.11 Properties of Wooden Beams (Surfaced Size)

						Max safe uniform load, lb, based on		
Nominal size, in (1)	Actual size $b \times d$, in, dressed (S4S) size (2)	Area of section bd, in² (3)	Weight at 40 lb/ft³, lb/ft (4)	Moment of inertia $I = bd^3/12$, in⁴ (5)	Section modulus $S = bd^2/6$, in³ (6)	Bending on 1 ft span,* f = 1,000 lb/in² (7)	Shear at 100† lb/in² (8)	Coef‡ of deflection, uniform load E = 1,000,000 (9)
2 × 4	1½ × 3½	5.25	1.46	5.36	3.06	2,040	700	4.20
3 × 4	2½ × 3½	8.75	2.43	8.93	5.10	3,400	1,166	2.52
4 × 4	3½ × 3½	12.25	3.40	12.51	7.15	4,760	1,632	1.80
2 × 6	1½ × 5½	8.25	2.29	20.8	7.56	5,040	1,100	1.082
3 × 6	2½ × 5½	13.75	3.82	34.7	12.60	8,390	1,835	0.648
4 × 6	3½ × 5½	19.25	5.35	48.5	17.65	11,760	2,570	0.464
6 × 6	5½ × 5½	30.3	8.40	76.3	27.7	18,490	4,040	0.295
2 × 8	1½ × 7¼	10.87	3.02	47.6	13.14	8,760	1,445	0.473
3 × 8	2½ × 7¼	18.12	5.04	79.4	21.9	14,600	2,410	0.284
4 × 8	3½ × 7¼	25.4	7.05	111.1	30.7	20,500	3,380	0.202
6 × 8	5½ × 7½	41.3	11.4	193	51.6	34,400	5,500	0.1162
8 × 8	7½ × 7½	56.3	15.6	264	70.3	46,900	7,500	0.0852
2 × 10	1½ × 9¼	13.87	3.85	98.9	21.4	14,290	1,850	0.227
3 × 10	2½ × 9¼	23.1	6.42	164.9	35.7	23,700	3,080	0.1364
4 × 10	3½ × 9¼	32.4	8.93	231	49.9	33,300	4,310	0.0974
6 × 10	5½ × 9½	52.3	14.5	393	82.7	55,200	6,970	0.0573
8 × 10	7½ × 9½	71.3	19.8	536	113	75,200	9,500	0.0421
10 × 10	9½ × 9½	90.3	25.0	679	143	95,300	12,030	0.0332
2 × 12	1½ × 11¼	16.87	4.69	178	31.6	21,100	2,250	0.1264
3 × 12	2½ × 11¼	28.1	7.81	297	52.7	35,100	3,750	0.0757
4 × 12	3½ × 11¼	39.4	10.94	415	73.9	49,300	5,250	0.0543
6 × 12	5½ × 11½	63.3	17.5	697	121	80,800	8,430	0.0323
8 × 12	7½ × 11½	86.3	23.9	951	165	110,200	11,510	0.0237
10 × 12	9½ × 11½	109.3	30.3	1,204	209	139,600	14,570	0.01864
12 × 12	11½ × 11½	132.3	36.7	1,458	253	169,000	17,620	0.01543
4 × 14	3½ × 13¼	46.4	12.88	678	102.4	68,300	6,180	0.0332
6 × 14	5½ × 13½	74.3	20.6	1,128	167	111,400	9,900	0.01987
8 × 14	7½ × 13½	101.3	28.0	1,538	228	152,000	13,500	0.01462
10 × 14	9½ × 13½	128.3	35.6	1,948	289	192,400	17,120	0.01153
12 × 14	11½ × 13½	155.3	43.1	2,360	349	233,000	20,700	0.00953
14 × 14	13½ × 13½	182.3	50.6	2,770	410	273,000	24,300	0.00812
6 × 16	5½ × 15½	85.3	23.6	1,707	220	146,800	11,380	0.01315
8 × 16	7½ × 15½	116.3	32.0	2,330	300	200,000	15,530	0.00967
10 × 16	9½ × 15½	147.3	40.9	2,950	380	254,000	19,610	0.00762
12 × 16	11½ × 15½	178.3	49.5	3,570	460	307,800	23,800	0.00630
14 × 16	13½ × 15½	209	58.1	4,190	541	360,000	27,900	0.00539
16 × 16	15½ × 15½	240	66.7	4,810	621	414,000	32,000	0.00468
8 × 18	7½ × 17½	131.3	36.4	3,350	383	255,000	17,500	0.00672
10 × 18	9½ × 17½	166.3	46.1	4,240	485	323,000	22,200	0.00531
12 × 18	11½ × 17½	201	55.9	5,140	587	391,000	26,800	0.00438
14 × 18	13½ × 17½	236	65.6	6,030	689	459,000	31,500	0.00373
16 × 18	15½ × 17½	271	75.3	6,920	791	528,000	36,200	0.00325
18 × 18	17½ × 17½	306	85.0	7,820	893	595,000	40,800	0.00288
12 × 20	11½ × 19½	224	62.3	7,110	729	485,000	29,900	0.00316
20 × 20	19½ × 19½	380	106	12,050	1,236	824,000	50,700	0.00187
24 × 24	23½ × 23½	552	153	25,400	2,160	1,440,000	73,400	0.000888
26 × 26	25½ × 25½	650	180.6	35,200	2,760	1,840,000	86,700	0.000639
28 × 28	27½ × 27½	756	210	47,700	3,470	2,320,000	100,600	0.000472
30 × 30	29½ × 29½	870	242	63,100	4,280	2,850,000	116,000	0.000356

NOTE: 1 in = 2.54 cm; 1 ft = 0.305 m; 1 lb = 4.45 N; 1 lb/in² = 6.89 kPa.

* For total safe uniform load, pounds, on beam of span L, feet, divide tabular value by L. For fiber stress f other than 1,000 lb/in² multiply by f and divide by 1,000.

† For shearing stress other than 100 lb/in², multiply by stress and divide by 100.

‡ For deflection, inches, multiply coefficient by total load, pounds, and by cube of span, feet, and divide by 1,000,000. For other modulus of elasticity E, multiply coefficient by 1,000,000 and divide by E.

Manufacturers Assoc. The moment of inertia and section modulus are with the neutral axis perpendicular to the depth at the center. The **safe bending moment** in inch-pounds for a given beam is determined from the section modulus S by multiplying the tabular value by the allowable fiber stress. To select a beam to withstand safely a given bending moment, divide the bending moment in inch-pounds by the allowable fiber stress, and choose a beam whose section modulus S is equal to or larger than the quotient thus obtained. For formulas for computing bending moments, see Sec. 5.2. Note that the allowable fiber stress must be modified by adjustment factors: C_D, load duration factor (0.9 for permanent loads); C_M, wet service factor (approximately 0.8 for moisture content greater than 16 percent); C_t, temperature factor (usually 1.0 for temperatures less than 100°F); C_F, size factor (1.0 for members up to 5 in wide by 12 in deep); and other factors which apply to laminated, curved, round, and/or flat use. (See also Sec. 6.7, ''Properties of Lumber Products.'')

Maximum loads in Table 12.2.11, cols. 7 and 8, are for uniform loading. Use half the values of col. 7 for a single load concentrated at midspan; for other loadings compute the bending moment and use the section modulus, col. 6. The values of col. 8 apply to all symmetrical loadings. For unsymmetrical loading, compute the maximum shear, which must not exceed one-half the tabular value.

The **coefficients of deflection** listed in Table 12.2.11 can be used to deduce deflection as indicated in the footnotes to the table. Coefficients of deflection under concentrated loads applied at the middle of the span may be obtained by multiplying the values in the table by 1.6. The results are only approximate, as the modulus of elasticity varies with the moisture content of the wood.

The deflection due to live load of beams intended to carry plastered ceilings should not exceed 1⁄360 of the span.

A convenient rule may be derived by assuming that the modulus of elasticity is 1,000 times the allowable fiber stress, which applies to all woods with sufficient accuracy for the purpose. Beams loaded uniformly to capacity in bending will then deflect 1⁄360 of the span when the depth in inches is 0.90 times the span in feet; and beams with central concentration, when the depth is 0.72 times the span in the same units. For such beams, the deflection in inches is, for uniform load, $0.03L^2/d$; for central concentration, $0.024L^2/d$, where L is the span, ft and d the depth, in. Variation in type of loading affects this result comparatively little.

Timber Columns

Timber columns may be either square or round and should have metal bases, usually galvanized steel, to cut off moisture and prevent lateral displacement. For supporting beams, they should have caps which, at roofs, may be of steel, or wood designed for bearing across the grain. At intermediate floors, caps should be of steel, although in some cases hardwood bolsters may be used. Except when caps or beams are of steel, columns should run down and rest directly on the baseplate. Table 12.2.12 gives **working unit stresses for wood columns** recommended where the building laws do not prescribe lower stresses. Use actual, not nominal, dimension of timbers. The column capacity $P_{\text{col}} = F_c' A_{\text{net}}$, where F_c' is interpolated from Table 12.2.12, and A_{net} is the net cross-sectional area of the column. The values for F_c' in Table 12.2.12 are generally conservative and are based on the factors cited in the table footnotes. In the event $E < 1{,}000F_c$, the value of F_c' must be computed to include the value of C_P as follows:

$$F_c' = F_c C_D C_M C_t C_F C_P$$

(Note that actual design stress $f_c \leq F_c'$.)

$$C_p = \text{column stability factor} = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + F_{cE}/F_c^*}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$

$$F_c^* = F_c C_D C_M C_t C_F$$

$$F_{cE} = \frac{K_{cE}E}{(l_e/d)^2} =$$ critical buckling design stress in compression parallel to the grain for a given wood species and geometrical configuration of column

where F_c = tabulated allowable design value for compression parallel to the grain for the species (Sec. 6.7, Tables 6.7.6 and 6.7.7); C_D = load duration factor (Sec. 6.7, ''Properties of Lumber Products''); C_M = wet use factor (Sec. 6.7, Table 6.7.8); C_t = temperature factor (Sec. 6.7, Table 6.7.9); C_F = size factor (Sec. 6.7, footnotes to Table 6.7.8); K_{cE} = 0.3 for visually and mechanically graded lumber and 0.418 for glulam members; l_e = effective length of column; d = least dimension of the column cross section; E = modulus of elasticity for the species (Sec. 6.7, Tables 6.7.6 and 6.7.7); c = constant for the type member: 0.8 for sawn lumber, 0.85 for timber piles, 0.9 for glulam members.

Glued Laminated Timber Structural glued laminated timber, commonly called glulam, refers to members which are fabricated by pressure gluing selected wood laminations of either ¾ or 1½ in (19 or 38 mm) surfaced thickness. The grain of all the laminations is approximately parallel longitudinally, with exterior laminations being of generally higher-quality wood since bending stresses are greater at the outer fibers. Curved and tapered structural members are available with the recommended minimum radii of curvature being 9 ft 4 in (2.84 m) for ¾-in laminations and 27 ft 6 in (8.4 m) for 1½-in lamination thickness. Laminations should be parallel to the tension face of members; sawn tapered cuts are permitted on the compression face.

Available net (surfaced) widths of members in inches are 2¼, 3⅛, 5⅛, 8¾, 10¾, 12¼, and 14¼; depths are determined by stress requirements. Economical spans (see ''Timber Construction Manual,'' American Institute of Timber Construction) for roof framing range from 10 to 100 ft (3 to 30 m) for simple spans. Floor framing, which is designed for much heavier live loads, economically spans from 6 to 40 ft (1.8 to 12 m) for simple beams and from 25 to 40 ft (7.5 to 12 m) for continuous beams.

Glued laminated members are generally fabricated from either Douglas fir and larch, Douglas fir (coast region), southern pine, or California redwood, depending on availability. Allowable design stresses depend on whether the condition of use is to be wet (moisture content in service of 16 percent or more) or dry (as in most covered structures), the species and grade of wood to be used, the manner of loading, and the number of laminations as well as the usual factors for duration of loading. The cumulative reduction factors described above also apply to glulam beams. Additional factors including C_v, volume factor; C_{fu}, flat use factor; and C_c, curvature factor, also must be applied to glued laminated beams. Refer to the National Design Specification for Wood Construction for further information on glued laminated timber design. (See Sec. 6.7.)

Table 12.2.12 Values of F_c', Working Stresses for Square or Rectangular Timber Columns, lb/in²
(Compression parallel to grain.)*

	l_e/d, in/in												
F_c	10	15	20	25	30	35	40	45	50	55†	60†	70†	80†
1000	680	615	508	389	293	225	176	141	116	96	81	60	46
1300	884	799	660	505	381	292	229	184	150	125	106	78	60
1600	1088	984	812	622	469	359	282	226	185	154	130	96	74
1900	1292	1168	964	739	557	427	335	268	220	183	154	114	88

* Values of F_c' in the table are based on $C_D = 0.9$ (long-duration, permanent, 50-year loading); $C_M = 0.8$ (moisture content > 16%); $C_t = 1.0$ (operating temperature < 100°F); $C_F = 1.0$ (cross section up to 5 in wide × 12 in deep); $K_{cE} = 0.3$; $E \geq 1{,}000F_c$. If $E < 1{,}000F_c$, see text for procedure to compute F_c'; $c = 0.8$.
† Columns should be limited to $l_e/d = 50$, except for individual members in stud walls, which should be limited to $l_e/d = 80$.

A summary of allowable unit stresses may be found in Sec. 6.7 for glued laminated timber.

Connections

Bolted Joints Compression may be transmitted by merely butting the timbers, with splice pieces bolted to the sides to keep alignment and resist incidental bending and shear. The same detail (Fig. 12.2.17) serves in tension, but the entire stress must then be transmitted through the bolts and splice pieces. If of wood, these should have a thickness h equal to $\frac{1}{2}b$. In light, unimportant work, splice pieces may be spiked. Table 12.2.13 gives the allowable load in pounds for one bolt loaded at both ends (double shear) when h is at least equal to $\frac{1}{2}b$. When steel side plates are used for side members, the tabulated loads may be increased 25 percent for parallel-to-grain loading, but no increase should be made for perpendicular-to-grain loads. When a joint consists of two members (single shear), one-half the tabulated load for a piece twice the thickness of the thinner member applies. The safe load for bolts loaded at an angle θ with the grain of the wood is given by the formula $N = PQ/(P \sin^2 \theta + Q \cos^2 \theta)$, where N = allowable load per bolt in a direction at inclination θ with the direction of the grain, lb; P = allowable load per bolt in compression parallel to the grain, lb; and Q = allowable load per bolt in compression perpendicular to the grain, lb.

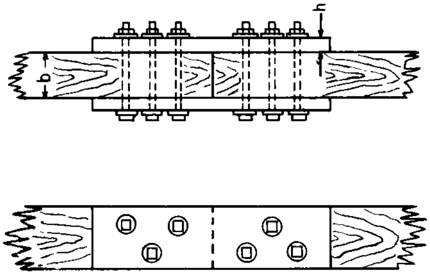


Fig. 12.2.17 Bolted splices for timber framing.

The size, arrangement, and **spacing of bolts** must be such that tension on the net section of the timber through the bolt holes and shear along the grain do not exceed allowable values. Bolts should be at least 7 diameters from the end of the timber for softwoods and 5 diameters for hardwoods and spaced at least 4 diameters on center parallel to the grain. Crossbolting, to prevent splitting the timber end, is sometimes desirable.

The efficiency of bolted timber connections may be greatly increased by the use of **ring connectors.** Split rings and shear plates are fitted into circular grooves, concentric with the bolt, in the contact surfaces, and transmit shear stresses across the joint. Grooves for split rings and shear plates are cut with a special tool, while toothed rings are usually seated by drawing together the timbers with high-strength bolts. Allowable loads for these various connectors are given in the ''Design Values for Wood Construction,'' published by the National Lumber Manufacturers Assoc. Selected values are given in Table 12.2.14.

The **holding power of wire nails** is as follows (''Design Values for Wood Construction''): The resistance to withdrawal is proportional to the length of embedment, to the diameter of the nail (where the wood does not split), and to $G^{2.5}$, where G is the ovendry specific gravity of the wood (see Sec. 6 for G values of various species). The safe resistance to withdrawal of common wire nails driven into the side grain of seasoned wood is given by Table 12.2.15. Nails withdrawn from green wood have generally slightly higher resistance, but nails driven into green wood may lose much of their resistance when the wood seasons; the allowable withdrawal load should be one-fourth of that given in Table 12.2.15. Cement and other coatings on nails may add materially to their resistance in softwoods. Drilling lead holes slightly smaller than the nail adds somewhat to the resistance and reduces danger of splitting. The structural design should be such that nails are not loaded in withdrawal from end grain.

The safe **lateral resistance** of common wire nails driven in side grain to the specified penetrations is given in Table 12.2.15 and is proportional to $D^{1.5}$ where D is the diameter, in. These values are for seasoned wood and should be reduced 25 percent for woods which will remain wet or will be loaded before seasoning. For nails driven into end grain, values should be reduced one-third.

Common wire **spikes** are larger for their lengths than nails. Their resistance to withdrawal and lateral resistance are given by the same formulas as for nails, but greater precautions need to be taken to avoid splitting.

The **resistance of wood screws to withdrawal** from side grain of seasoned wood is given by the formula $P = 2{,}850G^2D$, where P = the allowable load on the screw, lb/in penetration of the threaded portion; G = specific gravity of ovendry wood; D = diameter of screw, in. Wood screws should not be designed to be loaded in withdrawal from end grain.

The **allowable safe lateral resistance** of wood screws embedded 7 diameters in the side grain of seasoned wood is given by the formula $P = KD^2$, where P is the lateral resistance per screw, lb; D is the diameter, in; and K is 4,800 for oak (red and white), 3,960 for Douglas fir (coast region) and southern pine, and 3,240 for cypress (southern) and Douglas fir (inland region).

The following rules should be observed: (1) the size of the lead hole in soft (hard) woods should be about 70 (90) percent of the core or root diameter of the screw; (2) lubricants such as soap may be used without great loss in holding power; (3) long, slender screws are preferable generally, but in hardwood too slender screws may reach the limit of their tensile strength; (4) in the screws themselves, holding power is favored by thin sharp threads, rough unpolished surface, full diameter under the head, and shallow slots.

Table 12.2.13 Allowable Load in Pounds on One Bolt Loaded at Both Ends (Double Shear)

(For additional values and for conditions other than normal, see ''Design Values for Wood Construction'')

Length of bolt in main member, in	Diam of bolt, in	Douglas fir-larch		California redwood (open grain)		Oak, red and white		Western spruce, pine, fir, cedars	
		Parallel to grain	Perpendicular to grain	Parallel to grain	Perpendicular to grain	Parallel to grain	Perpendicular to grain	Parallel to grain	Perpendicular to grain
1½	½	1,050	470	780	310	1,410	730	760	290
	¾	1,580	590	1,170	370	2,110	890	1,140	360
	1	2,100	680	1,560	440	2,810	1,020	1,520	420
2½	½	1,230	730	990	510	1,530	960	980	490
	¾	2,400	980	1,950	620	2,890	1,480	1,900	600
	1	3,500	1,130	2,590	730	4,690	1,700	2,530	700
3½	½	1,230	730	990	580	1,530	960	980	560
	¾	2,400	1,170	2,010	740	2,890	1,770	1,990	720
	1	4,090	1,350	3,110	870	4,820	2,040	3,040	840
5½	¾	2,400	1,170	2,010	740	2,890	1,770	1,990	720
	1	3,180	1,260	2,690	810	3,780	1,920	2,660	790
	1¼	4,090	1,350	3,110	870	4,820	2,040	3,040	840

Table 12.2.14 Allowable Load in Pounds for One-Connector Unit in Single Shear*
(For additional values and for conditions other than normal, see ''Design Values for Wood Construction'')

Connector unit (diam)	Number of faces of piece with connectors on the same bolt	Net thickness of lumber, in	Min. edge distances, in	Group A: Douglas fir-larch and southern pine (dense), oak, red and white, ∥ to grain	Group A: ⊥ to grain	Group B: Douglas fir-larch, southern pine (med. grain), ∥ to grain	Group B: ⊥ to grain	Group C: California redwood (close) grain), western hemlock, southern cypress, ∥ to grain	Group C: ⊥ to grain
2½-in split ring, ½-in bolt	1	1 min,	1¾	2,630	1,900	2,270	1,620	1,900	1,350
		1½ or more		3,160	2,280	2,730	1,940	2,290	1,620
	2	1½ min,		2,430	1,750	2,100	1,500	1,760	1,250
		2 or more		3,160	2,280	2,730	1,940	2,290	1,620
4-in split ring, ¾-in bolt	1	1 min,	2¾	4,090	2,840	3,510	2,440	2,920	2,040
		1½ or more		6,020	4,180	5,160	3,590	4,280	2,990
	2	1½ min,		4,110	2,980	3,520	2,450	2,940	2,040
		3 or more		6,140	4,270	5,260	3,660	4,380	3,050
2⅝-in shear plate, ¾-in bolt†	1	1½ min	1¾	3,110	2,170	2,670	1,860	2,220	1,550
	2	1½ min,		2,420	1,690	2,080	1,450	1,730	1,210
		2½ or more		3,330	2,320	2,860	1,990	2,380	1,650
4-in shear plate, ¾-in or ⅞-in bolt†	1	1½ min,	2¾	4,370	3,040	3,750	2,620	3,130	2,170
		1¾ or more		5,090	3,540	4,360	3,040	3,640	2,530
	2	1¾ min,		3,390	2,360	2,910	2,020	2,420	1,680
		2½		4,310	3,000	3,690	2,550	3,080	2,140
		3½ or more		5,030	3,500	4,320	3,000	3,600	2,510

NOTE: 1 m = 2.54 cm; 1 lb = 4.45 N.
* One connector unit consists of one split ring with its bolt in single shear or two shear plates back to back in the contact faces of a timber-to-timber joint with their bolt in single shear.
† Allowable loads for all loadings, except wind, should not exceed 2,900 lb for 2⅝-in shear plates; 4,400 and 6,000 lb for 4-in shear plates with ¾- and ⅞-in bolts, respectively; multiply values by 1.33 for wind loading.

Table 12.2.15 Allowable Loads in Pounds for Common Nails in Side Grain* of Seasoned Wood

Type of load	Specific gravity G	Size of nail: d	6	8	10	12	16	20	30	40	50	60
		Length, in	2	2½	3	3¼	3½	4	4½	5	5½	6
		Diam, in	0.113	0.131	0.148	0.148	0.162	0.192	0.207	0.225	0.244	0.263
Withdrawal load per in penetration	0.31		9	10	12	12	13	15	16	18	20	21
	0.40		16	18	20	20	22	27	28	31	33	35
	0.44		20	23	26	26	29	34	37	40	43	46
	0.47		24	27	31	31	34	40	43	47	51	55
	0.51		29	34	38	38	42	49	53	58	63	68
	0.55		34	41	46	46	50	59	64	70	76	81
	0.67		57	66	75	75	82	97	105	114	124	133
Lateral load*†	0.60–0.75		78	97	116	116	132	171	191	218	249	276
	0.50–0.55		63	78	94	94	107	139	154	176	202	223
	0.42–0.50		51	64	77	77	88	113	126	144	165	182
	0.31–0.41		41	51	62	62	70	91	101	116	132	146

NOTE: 1 in = 2.54 cm; 1 lb = 4.45 N.
* The allowable lateral load for nails driven in end grain is two-thirds the values shown above.
† The minimum penetration for full lateral resistance for the four groups listed is 10, 11, 13, and 14 diam from higher to lower specific gravities, respectively. Reduce by interpolation for lesser penetration; minimum penetration is one-third the above.

Table 12.2.16 Allowable Lateral Loads in Pounds on Lag Bolts or Lag Screws

Side member	Length of bolt, in	Diam of bolt at shank, in	Overndry specific gravity of species: 0.60–0.75 ∥	0.60–0.75 ⊥	0.51–0.55 ∥	0.51–0.55 ⊥	0.42–0.50 ∥	0.42–0.50 ⊥	0.31–0.41 ∥	0.31–0.41 ⊥
1½-in wood	4	¼	200	190	170	170	130	120	100	100
	4	½	390	250	290	190	210	140	170	110
	6	⅜	480	370	420	320	360	280	290	220
	6	⅝	860	510	710	430	510	310	410	250
2½-in wood	6	½	620	410	470	310	340	220	270	180
	6	1	1,040	520	790	390	560	280	450	230
	8	¾	1,430	790	1,080	600	780	430	620	340
	8	1	1,800	900	1,360	680	970	490	780	390
½-in metal	3	¼	240	185	210	160	155	120	125	100
	3	½	550	285	415	215	295	155	240	125
	6	½	1,100	570	945	490	770	400	615	320
	6	¾	1,970	865	1,480	650	1,060	460	850	370
	10	⅞	3,420	1,420	2,960	1,230	2,340	970	1,890	785
	12	1	4,520	1,810	3,900	1,560	3,290	1,320	2,630	1,050
	16	1¼	7,120	2,850	6,150	2,460	5,500	2,200	4,520	1,810

NOTE: 1 in = 2.54 cm; 1 lb = 4.45 N.

The allowable **withdrawal load of lag screws** in side grain is given by the formula $p = 1{,}800D^{3/4}G^{3/2}$, allowable load per inch of penetration of threaded portion of lag screw into member receiving the point, lb; D = shank diameter of lag screw, in; G = specific gravity of ovendry wood. Use of lag screws loaded in withdrawal from end grain should be avoided. The allowable load in such case should not exceed 75 percent of that for side grain (see also Sec. 8).

The allowable **lateral resistance of lag screws** for parallel-to-grain loading with screws in side grain is proportional to D^2 and is dependent on species and type of side member. Selected values are given in Table 12.2.16 for one lag screw in single shear in a two-member joint.

Lead holes for lag screws (approximately 75 percent of shank diameter) should be prebored for the threaded portion. Lead holes for the shank should be of the same diameter and length as that of the unthreaded shank. Soap or other lubricant should be used to facilitate insertion and to prevent damage to the screw. Where steel-plate side pieces are used, the allowable loads given by the formula for parallel-to-grain loading may be increased by 25 percent.

The ultimate **withdrawal load per linear inch of penetration of a round drift bolt or pin** from side grain when driven into a prebored hole having a diameter ⅛ in less than that of the bolt diameter may be determined from the formula $p = 6{,}000G^2D$, where p = ultimate withdrawal load of penetration, lb/lin in; G = specific gravity of ovendry wood; D = diameter of drift bolt, in. A safety factor of about 5 is suggested for general use. The allowable load in lateral resistance for a drift bolt should ordinarily be taken as less than that for a common bolt.

STEEL CONSTRUCTION

(Note. In the design of steel structures, 1,000 lb is frequently designated as a kilopound or ''kip,'' and a stress of 1 kip per square inch is designated as 1 ksi.)

Structural steel design was based only on the allowable stress design (ASD) approach until the introduction of the load and resistance factor design (LRFD) technique in the mid 1980s. The LRFD approach is an ultimate strength design approach, similar to that adopted by the American Concrete Institute for concrete design. Both ASD and LRFD are accepted in current codes. The ASD method is still the most commonly used for design.

Specifications The following are in part condensed excerpts from the Specifications of the American Institute of Steel Construction.

Material Ordinary steel for rolled shapes, plates, and bars is typically specified by ASTM A36, with a yield stress of 36,000 lb/in² (248.2 MPa). However, advances in mill production methods have resulted in most steels satisfying the higher strength requirements of ASTM A572, Grade 50, leading to increased use of higher-strength steel at little or no cost premium. Other higher-strength steels used in structures are A440, A441, A588, and A242. Steel materials for pipe and tube, specified by ASTM A53 (welded-seam pipe) and A500 (cold-formed), have yield strengths of 33,000 to 50,000 lb/in² (227.5 to 344.7 MPa).

Ordinary unfinished machine bolts are specified by A307. Bolts used for structural steel connections are typically high-strength bolts specified by A325 or A490. Riveting is no longer used, but may often be encountered in older structures. The most common rivets were A502, Grade 1.

Allowable Stresses* in A36 Steel

	lb/in²	MPa
Tension F_t:		
On gross section	22,000	151.6
On net section, except at pinholes	29,000	200
On net section, at pinholes	16,000	110.2
Compression F_c: See Table 12.2.17		
Bending tension and compression on extreme fibers F_b:		
Basic stress, reduced in certain cases	22,000	151.6
Compact, adequately braced beams	24,000	165.3
Rectangular bearing plates	27,000	186.0
Shear F_v: Web of beams, gross section	14,500	99.9

* Allowable stresses may be increased by one-third when produced by wind or seismic loading alone or when combined with design dead and live loads.

Allowable Stresses* in Riveted and Bolted Connections

	lb/in²	MPa
Bearing: A36 steel		
Pins in reamed, drilled, or bored holes	32,400	223.3
Bolts and rivets	69,000	475.7
Roller, lb/lin in (N/lin cm)	760 × diam (in)	1131 × diam (cm)
Shear: bearing-type connections†		
A502, grade 1 hot-driven rivets	17,500	120.6
A307 bolts	10,000	68.9
A325 bolts when threading is excluded from shear planes (std. holes)	30,000	206.8
A325 bolts when threading is not excluded from shear planes (std. holes)	21,000	144.7
Shear: friction-type connections† (with threads included or excluded from shear plane)		
A325 bolts in standard holes	17,500	120.6
A325 bolts in oversized or short slotted holes	15,000	103.4
A325 bolts in long slotted holes	12,000	82.6
Tension:		
A502, grade 1, hot-driven rivets	23,000	158.5
A307 bolts	20,000	137.9
A325 bolts	44,000	303.3
Bending in pins of A36 steel	27,000	186.1

* Allowable stresses are based on nominal body area of fasteners unless indicated.
† Rivets or bolts may not share loads with welds on bearing-type connections but may do so in friction-type connections.

Table 12.2.17 Allowable Stress, in ksi, for Compression Members of A36 Steel

Main and secondary members, Kl/r not more than 120						Main members, Kl/r, 121–200			
$\frac{Kl}{r}$	F_a	$\frac{Kl}{r}$	F_a	$\frac{Kl}{r}$	F_a	$\frac{Kl}{r}$	F_a	$\frac{Kl}{r}$	F_a
1	21.56	41	19.11	81	15.24	121	10.14	161	5.76
5	21.39	45	18.78	85	14.79	125	9.55	165	5.49
10	21.16	50	18.35	90	14.20	130	8.84	170	5.17
15	20.89	55	17.90	95	13.60	135	8.19	175	4.88
20	20.60	60	17.43	100	12.98	140	7.62	180	4.61
25	20.28	65	16.94	105	12.33	145	7.10	185	4.36
30	19.94	70	16.43	110	11.67	150	6.64	190	4.14
35	19.58	75	15.90	115	10.99	155	6.22	195	3.93
40	19.19	80	15.36	120	10.28	160	5.83	200	3.73

NOTE: 1 ksi = 6.89 MPa.

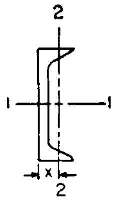


Table 12.2.18 American Standard Channels (C Shapes)

Depth of channel, in	Weight per ft, lb	Area of section, in^2	Width of flange, in	Thickness of web, in	Axis 1–1			Axis 2–2		V*	R*
					I, in^4	r, in	S, in^3	r, in	x, in	1,000 lb	
C 15	50.0	14.64	3.716	0.716	404	5.24	53.6	0.87	0.80	156	121
	40.0	11.70	3.520	0.520	349	5.44	46.2	0.89	0.78	113	88
	33.9	9.90	3.400	0.400	315	5.62	41.7	0.91	0.79	87	55
C 12	30.0	8.79	3.170	0.510	162	4.28	26.9	0.77	0.68	89	76
	25.0	7.32	3.047	0.387	144	4.43	23.9	0.79	0.68	67	55
	20.7	6.03	2.940	0.280	129	4.61	21.4	0.81	0.70	49	30
C 10	30.0	8.80	3.033	0.673	103.0	3.42	20.6	0.67	0.65	98	96
	25.0	7.33	2.886	0.526	90.7	3.52	18.1	0.68	0.62	76	75
	20.0	5.86	2.739	0.379	78.5	3.66	15.7	0.70	0.61	55	54
	15.3	4.47	2.600	0.240	66.9	3.87	13.4	0.72	0.64	35	22
C 9	20.0	5.86	2.648	0.448	60.6	3.22	13.5	0.65	0.59	58	62
	15.0	4.39	2.485	0.285	50.7	3.40	11.3	0.67	0.59	37	33
	13.4	3.89	2.430	0.230	47.3	3.49	10.5	0.67	0.61	30	22
C 8	18.75	5.49	2.527	0.487	43.7	2.82	10.9	0.60	0.57	56	
	13.75	4.02	2.343	0.303	35.8	2.99	9.0	0.62	0.56	35	
	11.5	3.36	2.260	0.220	32.3	3.10	8.1	0.63	0.58	26	
C 7	14.75	4.32	2.299	0.419	27.1	2.51	7.7	0.57	0.53	43	
	12.25	3.58	2.194	0.314	24.1	2.59	6.9	0.58	0.53	32	
	9.8	2.85	2.090	0.210	21.1	2.72	6.0	0.59	0.55	21	
C 6	13.0	3.81	2.157	0.437	17.3	2.13	5.8	0.53	0.52	38	
	10.5	3.07	2.034	0.314	15.1	2.22	5.0	0.53	0.50	27.3	
	8.2	2.39	1.920	0.200	13.0	2.34	4.3	0.54	0.52	17.4	
C 5	9.0	2.63	1.885	0.325	9.0	1.83	3.5	0.49	0.48	23.6	
	6.7	1.95	1.750	0.190	7.5	1.95	3.0	0.50	0.49	13.8	
C 4	7.25	2.12	1.720	0.320	4.5	1.47	2.3	0.46	0.46	18.6	
	5.4	1.56	1.580	0.180	3.8	1.56	1.9	0.45	0.46	10.4	
C 3	6.0	1.75	1.596	0.356	2.1	1.08	1.4	0.42	0.46	15.5	
	5.0	1.46	1.498	0.258	1.8	1.12	1.2	0.41	0.44	11.2	
	4.1	1.19	1.410	0.170	1.6	1.17	1.1	0.41	0.44	7.4	

NOTE: 1 in = 2.54 cm; 1 ft = 0.305 m; 1 lb = 4.45 N.
* V amd R values are for channels of A36 steel.

Proportion of Parts

Most simple beams, columns, and truss members are proportioned to limit the actual stresses to the allowable stresses stipulated above. Other stability or serviceability criteria may control the design. **Deflection** may govern in such members as cantilevers and lightly loaded roof beams. **Buckling,** rather than strength, may govern the design of compression members. The slenderness ratio Kl/r, where Kl is the effective length of the member and r is its radius of gyration, should be limited to 200 in compression members, and L/r limited to 300 in tension members. Kl should not be less than the actual unbraced length l in columns of a frame which depends on its bending stiffness for lateral stiffness. **Width-thickness ratios** are specified for projecting elements under compression. Repeated fluctuations in stress leading to fatigue may be a controlling factor. Rules are given for **combined stresses** of tension, compression, bending, and shear.

Tension members should be proportioned for the gross and net section, deducting for bolt or rivet holes ⅛ in (0.3 cm) larger than the nominal diameter of the fastener.

Columns and other **compression members** subject to eccentric load or to axial load and bending are governed by special rules. A long-established rule is that $f_a/F_a + f_b/F_b$ should be equal to or less than unity, where f_a is the axial stress, f_b the bending stress, and F_a and F_b are the corresponding allowable stresses if axial or bending stress alone exist. This is still considered valid when f_a/F_a is less than 0.15. Joints shall be fully spliced, except that where reversal of stress is not expected and the joint is laterally supported, the ends of the members may be milled to plane parallel surfaces normal to the stresses and abutted with sufficient splicing to hold the connected members accurately in place. Column bases should be milled on top for the column bearing, except for rolled-steel bearing plates 4 in (10 cm) or less in thickness.

Beams and girders, of rolled section or built-up, should in general be sized such that the bending moment M divided by the section modulus S is less than the allowable bending stress F_b. For built-up sections a rule of thumb is, for A36 steel, flanges in compression should have a thickness of 1⁄16 the projecting half width, and webs should have a thickness of 1⁄320 the maximum clear distance between flanges. Web stiffeners should be provided at points of high concentrated loads; additional web stiffeners are required in plate girders. Splices in the webs of plate girders should be made by plates on both sides of the web. When two or more rolled beams or channels are used side by side to form a beam, they should be connected at separators spaced no more than 5 ft (1.52 m); beams deeper than 12 in (30 cm) are to have at least two bolts to each separator.

The **lateral force on crane runways** due to the effect of moving crane trolleys may be assumed as 20 percent of the sum of the weights of the lifted loads and of the crane trolley (but exclusive of the other parts of the crane) applied at the top of the rail, one-half on each side of the runway, and shall be considered as acting in either direction normal to the runway rail. The **longitudinal force** may be assumed as 10 percent of the maximum wheel reactions of the crane applied at the top of the rail.

Bolted or riveted connections carrying calculated stress, except lacing and sag bars, should be designed to support not less than 6,000 lb (27.0 kN). Rivets or high-strength bolts are preferred in all places, and both are implied in these paragraphs wherever ''bolting'' is mentioned; unfinished bolts, A307, may be used in the shop or in field connections of small unimportant structures of secondary members, bracing, and beams.

Members in tension or compression, meeting at a joint, shall have their lines of center of gravity pass through a point, if practicable; if not, provision shall be made for the eccentricity. A group of bolts transmitting stress to a member shall have its center of gravity in the line of the stress, if practicable; if not, the group shall be designed for the resulting eccentricity. Where stress is transmitted from one member to another by bolts through a loose filler greater than ¼ in in thickness, except in slip critical connections using high-strength bolts, the filler shall be extended beyond the connected member and the extension secured by enough bolts or sufficient welding to distribute the total stress in the member uniformly over the combined sections of the member and the filler.

Most bolted connections transmit shearing forces by developing the shearing or bearing values of the bolts, but bolts in certain connections, such as shelf angles and brackets, are required to transmit tension forces.

Bolts shall be proportioned by the nominal diameter. Rivets and A307 bolts whose grip exceeds 5 diam shall be allowed 1 percent less safe stress for each 1⁄16 in (0.16 cm) excess length. The minimum distance between centers of bolt holes shall be 2⅔ diam of the bolt; but preferably not less than 3 diam.

The minimum distance from the center of any bolt hole to a sheared edge shall be 2¼ in (5.7 cm) for 1¼ in (32 mm) bolts, 2 in (5.1 cm) for 1⅛ in (28 mm) bolts, 1¾ in (4.4 cm) for 1 in (25 mm) bolts, 1½ in (3.8 cm) for ⅞ in (22 mm) bolts, 1¼ in (3.2 cm) for ¾ in (19 mm) bolts, 1⅛ in (2.8 cm) for ⅝ in (16 mm) bolts, and ⅞ (2.22 cm) for ½ in (13 mm) bolts. The distance from any edge shall not exceed 12 times the thickness of the plate and shall not exceed 6 in (15 cm).

Design of Members

Properties of Standard Structural Shapes Tables 12.2.18 to 12.2.26 give the properties of American Standard channels and I beams, wide-flange beams and columns, angles, and tees. In these tables, I = moment of inertia, r = radius of gyration, S = section modulus, x = distance from gravity axis to face, V = max web shear in kips, and R = max end reaction on 3½-in (9-mm) seat, based on crippling of web, in kips. R values are omitted where web crippling does not govern.

A great variety of tees is produced by shearing or gas-cutting stan-

Table 12.2.19 American Standard I Beams (S Shapes)

Depth of beam, in	Weight per ft, lb	Area of section, in²	Width of flange, in	Thickness of web, in	Neutral axis perpendicular to web at center: I, in⁴	r, in	S, in³	Neutral axis coincident with center line of web: I, in⁴	r, in	S, in³	V* (1,000 lb)	R* (1,000 lb)
S 24	120.0	35.13	8.048	0.798	3010.8	9.26	250.9	84.9	1.56	21.1	278	162
	105.9	30.98	7.875	0.625	2811.5	9.53	234.3	78.9	1.60	20.0	218	123
	100.0	29.25	7.247	0.747	2371.8	9.05	197.6	48.4	1.29	13.4	260	140
	90.0	26.30	7.124	0.624	2230.1	9.21	185.8	45.5	1.32	12.8	217	117
	79.9	23.33	7.000	0.500	2087.2	9.46	173.9	42.9	1.36	12.2	174	80
S 20	95.0	27.74	7.200	0.800	1599.7	7.59	160.0	50.5	1.35	14.0	232	150
	85.0	24.80	7.053	0.653	1501.7	7.78	150.2	47.0	1.38	13.3	189	124
	75.0	21.90	6.391	0.641	1263.5	7.60	126.3	30.1	1.17	9.4	186	114
	65.4	19.08	6.250	0.500	1169.5	7.83	116.9	27.9	1.21	8.9	145	83
S 18	70.0	20.46	6.251	0.711	917.5	6.70	101.9	24.5	1.09	7.8	186	123
	54.7	15.94	6.000	0.460	795.5	7.07	88.4	21.2	1.15	7.1	120	70
S 15	50.0	14.59	5.640	0.550	481.1	5.74	64.2	16.0	1.05	5.7	120	91
	42.9	12.49	5.500	0.410	441.8	5.95	58.9	14.6	1.08	5.3	89	58
S 12	50.0	14.57	5.477	0.687	301.6	4.55	50.3	16.0	1.05	5.8	120	116
	40.8	11.84	5.250	0.460	268.9	4.77	44.8	13.8	1.08	5.3	80	78
	35.0	10.20	5.078	0.428	227.0	4.72	37.8	10.0	0.99	3.9	74	66
	31.8	9.26	5.000	0.350	215.8	4.83	36.0	9.5	1.01	3.8	61	45
S 10	35.0	10.22	4.944	0.594	145.8	3.78	29.2	8.5	0.91	3.4	86	89
	25.4	7.38	4.660	0.310	122.1	4.07	24.4	6.9	0.97	3.0	45	38
S 8	23.0	6.71	4.171	0.441	64.2	3.09	16.0	4.4	0.81	2.1	51	
	18.4	5.34	4.000	0.270	56.9	3.26	14.2	3.8	0.84	1.9	31	
S 7	20.0	5.83	3.860	0.450	41.9	2.68	12.0	3.1	0.74	1.6	46	
	15.3	4.43	3.660	0.250	36.2	2.86	10.4	2.7	0.78	1.5	25	
S 6	17.25	5.02	3.565	0.465	26.0	2.28	8.7	2.3	0.68	1.3	40.5	
	12.5	3.61	3.330	0.230	21.8	2.46	7.3	1.8	0.72	1.1	20	
S 5	14.75	4.29	3.284	0.494	15.0	1.87	6.0	1.7	0.63	1.0	35.8	
	10.0	2.87	3.000	0.210	12.1	2.05	4.8	1.2	0.65	0.82	15.2	
S 4	9.5	2.76	2.796	0.326	6.7	1.56	3.3	0.91	0.58	0.65	18.9	
	7.7	2.21	2.660	0.190	6.0	1.64	3.0	0.77	0.59	0.58	11.0	
S 3	7.5	2.17	2.509	0.349	2.9	1.15	1.9	0.59	0.52	0.47	15.2	
	5.7	1.64	2.330	0.170	2.5	1.23	1.7	0.46	0.53	0.40	7.4	

NOTE: 1 in = 2.54 cm; 1 ft = 0.305 m; 1 lb = 4.45 N.
Lightweight beams of each depth are usual stock sizes.
* V and R values are for beams of A36 steel.

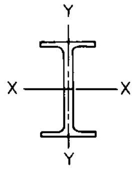


Table 12.2.20 Properties of Wide-Flange Beams and Columns (W Shapes)

Nominal size, in	Weight per ft, lb†	Area of section, in^2	Depth of section, in	Flange		Web thickness, in	Neutral axis perpendicular to web at center			Neutral axis parallel to web at center			V, 1,000 lb*	R, 1,000 lb*
				Width, in	Thickness, in		I, in^4	S, in^3	r, in	I, in^4	S, in^3	r, in		
W 36	300	88.3	36.74	16.66	1.68	0.945	20,300	1,110	15.2	1,300	156	3.83	500	237
	280	82.4	36.52	16.6	1.57	0.885	18,900	1,030	15.1	1,200	144	3.81	465	215
	260	76.5	36.26	16.55	1.44	0.84	17,300	953	15	1,090	132	3.78	439	198
	245	72.1	36.08	16.51	1.35	0.8	16,100	895	15	1,010	123	3.75	416	186
	230	67.6	35.9	16.47	1.26	0.76	15,000	837	14.9	940	114	3.73	393	170
	194	57	36.49	12.12	1.26	0.765	12,100	664	14.6	375	61.9	2.56	402	163
	182	53.6	36.33	12.08	1.18	0.725	11,300	623	14.5	347	57.6	2.55	379	152
	170	50	36.17	12.03	1.1	0.68	10,500	580	14.5	320	53.2	2.53	354	137
	160	47	36.01	12	1.02	0.65	9,750	542	14.4	295	49.1	2.5	337	124
	150	44.2	35.85	11.98	0.94	0.625	9,040	504	14.3	270	45.1	2.47	323	113
W 33	241	70.9	34.18	15.86	1.4	0.83	14,200	829	14.1	932	118	3.63	409	177
	221	65	33.93	15.81	1.275	0.775	12,800	757	14.1	840	106	3.59	379	159
	201	59.1	33.68	15.75	1.15	0.715	11,500	684	14	749	95.2	3.56	347	142
	152	44.7	33.49	11.57	1.055	0.635	8,160	487	13.5	273	47.2	2.47	306	122
	141	41.6	33.3	11.54	0.96	0.605	7,450	448	13.4	246	42.7	2.43	290	109
	130	38.3	33.09	11.51	0.855	0.58	6,710	406	13.2	218	37.9	2.39	276	98
W 30	211	62	30.94	15.11	1.315	0.775	10,300	663	12.9	757	100	3.49	345	162
	191	56.1	30.68	15.04	1.185	0.71	9,170	598	12.8	673	89.5	3.46	314	141
	173	50.8	30.44	14.99	1.065	0.655	8,200	539	12.7	598	79.8	3.43	287	128
	148	43.5	30.67	10.48	1.18	0.65	6,680	436	12.4	227	43.3	2.28	287	131
	132	38.9	30.31	10.55	1	0.615	5,770	380	12.2	196	37.2	2.25	268	115
	124	36.5	30.17	10.52	0.93	0.585	5,360	355	12.1	181	34.4	2.23	254	103
	116	34.2	30.01	10.5	0.85	0.565	4,930	329	12	164	31.3	2.19	244	95
	108	31.7	29.83	10.48	0.76	0.545	4,470	299	11.9	146	27.9	2.15	234	87
W 27	178	52.3	27.81	14.09	1.19	0.725	6,990	502	11.6	555	78.8	3.26	290	141
	161	47.4	27.59	14.02	1.08	0.66	6,280	455	11.5	497	70.9	3.24	262	126
	146	42.9	27.38	13.97	0.975	0.605	5,630	411	11.4	443	63.5	3.21	239	111
	114	33.5	27.29	10.07	0.93	0.57	4,090	299	11	159	31.5	2.18	224	100
	102	30	27.09	10.02	0.83	0.515	3,620	267	11	139	27.8	2.15	201	82
	94	27.7	26.92	9.99	0.745	0.49	3,270	243	10.9	124	24.8	2.12	190	73
W 24	162	47.7	25	12.96	1.22	0.705	5,170	414	10.4	443	68.4	3.05	254	143
	146	43	24.74	12.9	1.09	0.65	4,580	371	10.3	391	60.5	3.01	232	126
	131	38.5	24.48	12.86	0.96	0.605	4,020	329	10.2	340	53	2.97	213	113
	117	34.4	24.26	12.8	0.85	0.55	3,540	291	10.1	297	46.5	2.94	192	94
	104	30.6	24.06	12.75	0.75	0.5	3,100	258	10.1	259	40.7	2.91	173	77
	103	30.3	24.53	9	0.98	0.55	3,000	245	9.96	119	26.5	1.99	194	97
	94	27.7	24.31	9.065	0.875	0.515	2,700	222	9.87	109	24	1.98	180	84
	84	24.7	24.1	9.02	0.77	0.47	2,370	196	9.79	94.4	20.9	1.95	163	70
	76	22.4	23.92	8.99	0.68	0.44	2,100	176	9.69	82.5	18.4	1.92	152	60
W 21	147	43.2	22.06	12.51	1.15	0.72	3,630	329	9.17	376	60.1	2.95	229	140
	132	38.8	21.83	12.44	1.035	0.65	3,220	295	9.12	333	53.5	2.93	204	124
	122	35.9	21.68	12.39	0.96	0.6	2,960	273	9.09	305	49.2	2.92	187	110
	93	27.3	21.62	8.42	0.93	0.58	2,070	192	8.7	92.9	22.1	1.84	181	106
	83	24.3	21.43	8.355	0.835	0.515	1,830	171	8.67	81.4	19.5	1.83	159	85
	73	21.5	21.24	8.295	0.74	0.455	1,600	151	8.64	70.6	17	1.81	139	67
	68	20	21.13	8.27	0.685	0.43	1,480	140	8.6	64.7	15.7	1.8	131	59
	62	18.3	20.99	8.24	0.615	0.4	1,330	127	8.54	57.5	13.9	1.77	121	51
W 18	119	35.1	18.97	11.27	1.06	0.655	2,190	231	7.9	253	44.9	2.69	179	123
	106	31.1	18.73	11.2	0.94	0.59	1,910	204	7.84	220	39.4	2.66	159	106
	97	28.5	18.59	11.15	0.87	0.535	1,750	188	7.82	201	36.1	2.65	143	94
	86	25.3	18.39	11.09	0.77	0.48	1,530	166	7.77	175	31.6	2.63	127	76
	76	22.3	18.21	11.04	0.68	0.425	1,330	146	7.73	152	27.6	2.61	111	60

NOTE: 1 in = 2.54 cm; 1 ft = 0.305 m; 1 lb = 4.45 N.
Flanges of wide-flange beams and columns are not tapered, have constant thickness.
Sections without values of V and R are used chiefly for columns.
Lightweight beams for each nominal size, and beams with depth in even inches, are most usually stocked.
Designation of wide-flange beams is made by giving nominal depth and weight, thus W8 × 40.
* V and R values are for beams of A36 steel.
† Some sections listed are no longer rolled but may be encountered in existing construction. Others currently rolled are not listed. Refer to producers' catalogs for sections currently available.

Table 12.2.20 Properties of Wide-Flange Beams and Columns (W Shapes) (*Continued*)

Nominal size, in	Weight per ft, lb†	Area of section, in²	Depth of section, in	Flange		Web thickness, in	Neutral axis perpendicular to web at center			Neutral axis parallel to web at center			V, 1,000 lb*	R, 1,000 lb*
				Width, in	Thickness, in		I, in⁴	S, in³	r, in	I, in⁴	S, in³	r, in		
W 18	71	20.8	18.47	7.635	0.81	0.495	1,170	127	7.5	60.3	15.8	1.7	132	81
	65	19.1	18.35	7.59	0.75	0.45	1,070	117	7.49	54.8	14.4	1.69	119	67
	60	17.6	18.24	7.555	0.695	0.415	984	108	7.47	50.1	13.3	1.69	109	58
	55	16.2	18.11	7.53	0.63	0.39	890	98.3	7.41	44.9	11.9	1.67	102	51
	50	14.7	17.99	7.495	0.57	0.355	800	88.9	7.38	40.1	10.7	1.65	92	42
W 16	100	29.4	16.97	10.43	0.985	0.585	1,490	175	7.1	186	35.7	2.51	143	107
	89	26.2	16.75	10.37	0.875	0.525	1,300	155	7.05	163	31.4	2.49	127	92
	77	22.6	16.52	10.3	0.76	0.455	1,110	134	7	138	26.9	2.47	108	71
	67	19.7	16.33	10.24	0.665	0.395	954	117	6.96	119	23.2	2.46	93	53
	57	16.8	16.43	7.12	0.715	0.43	758	92.2	6.72	43.1	12.1	1.6	102	63
	50	14.7	16.26	7.07	0.63	0.38	659	81	6.68	37.2	10.5	1.59	89	49
	45	13.3	16.13	7.035	0.565	0.345	586	72.7	6.65	32.8	9.34	1.57	80	41
	40	11.8	16.01	6.995	0.505	0.305	518	64.7	6.63	28.9	8.25	1.57	70	32
	36	10.6	15.86	6.985	0.43	0.295	448	56.5	6.51	24.5	7	1.52	67	29
W 14	426	125	18.67	16.7	3.035	1.875	6,600	707	7.26	2,360	283	4.34	—	—
	398	117	18.29	16.59	2.845	1.77	6,000	656	7.16	2,170	262	4.31	—	—
	370	109	17.92	16.48	2.66	1.655	5,440	607	7.07	1,990	241	4.27	—	—
	342	101	17.54	16.36	2.47	1.54	4,900	559	6.98	1,810	221	4.24	—	—
	311	91.4	17.12	16.23	2.26	1.41	4,330	506	6.88	1,610	199	4.2	—	—
	283	83.3	16.74	16.11	2.07	1.29	3,840	459	6.79	1,440	179	4.17	—	—
	257	75.6	16.38	16	1.89	1.175	3,400	415	6.71	1,290	161	4.13	—	—
	233	68.5	16.04	15.89	1.72	1.07	3,010	375	6.63	1,150	145	4.1	—	—
	211	62	15.72	15.8	1.56	0.98	2,660	338	6.55	1,030	130	4.07	—	—
	193	56.8	15.48	15.71	1.44	0.89	2,400	310	6.5	931	119	4.05	—	—
	176	51.8	15.22	15.65	1.31	0.83	2,140	281	6.43	838	107	4.02	—	—
	159	46.7	14.98	15.57	1.19	0.745	1,900	254	6.38	748	96.2	4	161	145
	145	42.7	14.78	15.5	1.09	0.68	1,710	232	6.33	677	87.3	3.98	145	127
	132	38.8	14.66	14.73	1.03	0.645	1,530	209	6.28	548	74.5	3.76	136	118
	120	35.3	14.48	14.67	0.94	0.59	1,380	190	6.24	495	67.5	3.74	123	106
	109	32	14.32	14.61	0.86	0.525	1,240	173	6.22	447	61.2	3.73	108	92
	99	29.1	14.16	14.57	0.78	0.485	1,110	157	6.17	402	55.2	3.71	99	82
	90	26.5	14.02	14.52	0.71	0.44	999	143	6.14	362	49.9	3.7	89	69
	82	24.1	14.31	10.13	0.855	0.51	882	123	6.05	148	29.3	2.48	105	92
	74	21.8	14.17	10.07	0.785	0.45	796	112	6.04	134	26.6	2.48	92	72
	68	20	14.04	10.04	0.72	0.415	723	103	6.01	121	24.2	2.46	84	61
	61	17.9	13.89	9.995	0.645	0.375	640	92.2	5.98	107	21.5	2.45	75	50
	53	15.6	13.92	8.06	0.66	0.37	541	77.8	5.89	57.7	14.3	1.92	74	49
	48	14.1	13.79	8.03	0.595	0.34	485	70.3	5.85	51.4	12.8	1.91	68	41
	43	12.6	13.66	7.995	0.53	0.305	428	62.7	5.82	45.2	11.3	1.89	60	33
	38	11.2	14.1	6.77	0.515	0.31	385	54.6	5.87	26.7	7.88	1.55	63	34
	34	10	13.98	6.745	0.455	0.285	340	48.6	5.83	23.3	6.91	1.53	57	29
	30	8.85	13.84	6.73	0.385	0.27	291	42	5.73	19.6	5.82	1.49	54	26
W 12	190	55.8	14.38	12.67	1.735	1.06	1,890	263	5.82	589	93	3.25	—	—
	152	44.7	13.71	12.48	1.4	0.87	1,430	209	5.66	454	72.8	3.19	—	—
	136	39.9	13.41	12.4	1.25	0.79	1,240	186	5.58	398	64.2	3.16	—	—
	120	35.3	13.12	12.32	1.105	0.71	1,070	163	5.51	345	56	3.13	134	136
	106	31.2	12.89	12.22	0.99	0.61	933	145	5.47	301	49.3	3.11	113	112
	96	28.2	12.71	12.16	0.9	0.55	833	131	5.44	270	44.4	3.09	101	99
	87	25.6	12.53	12.13	0.81	0.515	740	118	5.38	241	39.7	3.07	93	89
	79	23.2	12.38	12.08	0.735	0.47	662	107	5.34	216	35.8	3.05	84	79
	72	21.1	12.25	12.04	0.67	0.43	597	97.4	5.31	195	32.4	3.04	76	68
	65	19.1	12.12	12	0.605	0.39	533	87.9	5.28	174	29.1	3.02	68	56
	58	17	12.19	10.01	0.64	0.36	475	78	5.28	107	21.4	2.51	63	48
	53	15.6	12.06	9.995	0.575	0.345	425	70.6	5.23	95.8	19.2	2.48	60	44
	50	14.7	12.19	8.08	0.64	0.37	394	64.7	5.18	56.3	13.9	1.96	65	51
	45	13.2	12.06	8.045	0.575	0.335	350	58.1	5.15	50	12.4	1.94	58	42
	40	11.8	11.94	8.005	0.515	0.295	310	51.9	5.13	44.1	11	1.93	51	32

NOTE: 1 in = 2.54 cm; 1 ft = 0.305 m; 1 lb = 4.45 N.
Flanges of wide-flange beams and columns are not tapered, have constant thickness.
Sections without values of V and R are used chiefly for columns.
Lightweight beams for each nominal size, and beams with depth in even inches, are most usually stocked.
Designation of wide-flange beams is made by giving nominal depth and weight, thus W8 × 40.
* V and R values are for beams of A36 steel.
† Some sections listed are no longer rolled but may be encountered in existing construction. Others currently rolled are not listed. Refer to producers' catalogs for sections currently available.

Table 12.2.20 Properties of Wide-Flange Beams and Columns (W Shapes) (*Continued*)

Nominal size, in	Weight per ft, lb†	Area of section, in²	Depth of section, in	Flange		Web thickness, in	Neutral axis perpendicular to web at center			Neutral axis parallel to web at center			*V*, 1,000 lb*	*R*, 1,000 lb*
				Width, in	Thickness, in		*I*, in⁴	*S*, in³	*r*, in	*I*, in⁴	*S*, in³	*r*, in		
W 12	35	10.3	12.5	6.56	0.52	0.3	285	45.6	5.25	24.5	7.47	1.54	54	33
	30	8.79	12.34	6.52	0.44	0.26	238	38.6	5.21	20.3	6.24	1.52	46	25
	26	7.65	12.22	6.49	0.38	0.23	204	33.4	5.17	17.3	5.34	1.51	40	20
W 10	112	32.9	11.36	10.42	1.25	0.755	716	126	4.66	236	45.3	2.68	—	—
	100	29.4	11.1	10.34	1.12	0.68	623	112	4.6	207	40	2.65	—	—
	88	25.9	10.84	10.27	0.99	0.605	534	98.5	4.54	179	34.8	2.63	—	—
	77	22.6	10.6	10.19	0.87	0.53	455	85.9	4.49	154	30.1	2.6	—	—
	68	20	10.4	10.13	0.77	0.47	394	75.7	4.44	134	26.4	2.59	70	78
	60	17.6	10.22	10.08	0.68	0.42	341	66.7	4.39	116	23	2.57	62	68
	54	15.8	10.09	10.03	0.615	0.37	303	60	4.37	103	20.6	2.56	54	54
	49	14.4	9.98	10	0.56	0.34	272	54.6	4.35	93.4	18.7	2.54	49	45
	45	13.3	10.1	8.02	0.62	0.35	248	49.1	4.32	53.4	13.3	2.01	51	48
	39	11.5	9.92	7.985	0.53	0.315	209	42.1	4.27	45	11.3	1.98	45	39
	33	9.71	9.73	7.96	0.435	0.29	170	35	4.19	36.6	9.2	1.94	41	33
	30	8.84	10.47	5.81	0.51	0.3	170	32.4	4.38	16.7	5.75	1.37	45	35
	26	7.61	10.33	5.77	0.44	0.26	144	27.9	4.35	14.1	4.89	1.36	39	26
	22	6.49	10.17	5.75	0.36	0.24	118	23.2	4.27	11.4	3.97	1.33	35	22
W 8	67	19.7	9	8.28	0.935	0.57	272	60.4	3.72	88.6	21.4	2.12	—	—
	58	17.1	8.75	8.22	0.81	0.51	228	52	3.65	75.1	18.3	2.1	—	—
	48	14.1	8.5	8.11	0.685	0.4	184	43.3	3.61	60.9	15	2.08	49	61
	40	11.7	8.25	8.07	0.56	0.36	146	35.5	3.53	49.1	12.2	2.04	43	53
	35	10.3	8.12	8.02	0.495	0.31	127	31.2	3.51	42.6	10.6	2.03	36	41
	31	9.13	8	7.995	0.435	0.285	110	27.5	3.47	37.1	9.27	2.02	33	35
	28	8.25	8.06	6.535	0.465	0.285	98	24.3	3.45	21.7	6.63	1.62	33	34
	24	7.08	7.93	6.495	0.4	0.245	82.8	20.9	3.42	18.3	5.63	1.61	28	26
	21	6.16	8.28	5.27	0.4	0.25	75.3	18.2	3.49	9.77	3.71	1.26	30	26

NOTE: 1 in = 2.54 cm; 1 ft = 0.305 m; 1 lb = 4.45 N.
Flanges of wide-flange beams and columns are not tapered, have constant thickness.
Sections without values of *V* and *R* are used chiefly for columns.
Lightweight beams for each nominal size, and beams with depth in even inches, are most usually stocked.
Designation of wide-flange beams is made by giving nominal depth and weight, thus W8 × 40.
* *V* and *R* values are for beams of A36 steel.
† Some sections listed are no longer rolled but may be encountered in existing construction. Others currently rolled are not listed. Refer to producers' catalogs for sections currently available.

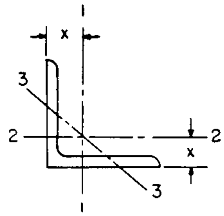


Table 12.2.21 Selected Standard Angles (L Shapes), Equal Legs
(One to three intermediate thicknesses in each size group are available, varying by 1/16 in)
A single angle should never be used as a beam. Two angles, bolted at frequent intervals, may be used.

Size, in	Weight per ft, lb	Areas of section, in^2	Axis 1-1 and axis 2-2				Axis 3-3, *r* min, in	Net areas after deducting holes for ⅞-in rivets	
			I, in^4	*r*, in	*S*, in^3	*x*, in		1 hole	2 holes
8 × 8 × 1⅛	56.9	16.73	98.0	2.42	17.5	2.41	1.56	15.60	14.48
1	51.0	15.00	89.0	2.44	15.8	2.37	1.56	14.00	13.00
⅞	45.0	13.23	79.6	2.45	14.0	2.32	1.57	12.36	11.48
¾	38.9	11.44	69.7	2.47	12.2	2.28	1.57	10.69	9.94
⅝	32.7	9.61	59.4	2.49	10.3	2.23	1.58	8.98	8.36
½	26.4	7.75	48.6	2.50	8.4	2.19	1.59	7.25	6.75
6 × 6 × 1	37.4	11.00	35.5	1.80	8.6	1.86	1.17	10.00	9.00
⅞	33.1	9.73	31.9	1.81	7.6	1.82	1.17	8.86	7.98
¾	28.7	8.44	28.2	1.83	6.7	1.78	1.17	7.69	6.94
⅝	24.2	7.11	24.2	1.84	5.7	1.73	1.18	6.48	5.86
½	19.6	5.75	19.9	1.86	4.6	1.68	1.18	5.25	4.75
⅜	14.9	4.36	15.4	1.88	3.5	1.64	1.19	3.98	3.61
5 × 5 × ⅞	27.2	7.98	17.8	1.49	5.2	1.57	0.97	7.10	6.23
¾	23.6	6.94	15.7	1.51	4.5	1.52	0.97	6.19	5.44
⅝	20.0	5.86	13.6	1.52	3.9	1.48	0.98	5.24	4.61
½	16.2	4.75	11.3	1.54	3.2	1.43	0.98	4.25	3.75
⅜	12.3	3.61	8.7	1.56	2.4	1.39	0.99	3.24	2.86
4 × 4 × ¾	18.5	5.44	7.7	1.19	2.8	1.27	0.78	4.69	3.94
⅝	15.7	4.61	6.7	1.20	2.4	1.23	0.78	3.98	3.36
½	12.8	3.75	5.6	1.22	2.0	1.18	0.78	3.25	2.75
⅜	9.8	2.86	4.4	1.23	1.5	1.14	0.79	2.48	2.11
¼	6.6	1.94	3.0	1.25	1.1	1.09	0.80	1.70	1.45
3½ × 3½ × ½	11.1	3.25	3.6	1.06	1.5	1.06	0.68	2.75	2.25
⅜	8.5	2.48	2.9	1.07	1.2	1.01	0.69	2.10	1.73
¼	5.8	1.69	2.0	1.09	0.79	0.97	0.69	1.44	1.19
3 × 3 × ½	9.4	2.75	2.2	0.90	1.1	0.93	0.58		
⅜	7.2	2.11	1.8	0.91	0.83	0.89	0.58		
¼	4.9	1.44	1.2	0.93	0.58	0.84	0.59		
2½ × 2½ × ½	7.7	2.25	1.2	0.74	0.72	0.81	0.49		
⅜	5.9	1.73	0.98	0.75	0.57	0.76	0.49		
¼	4.1	1.19	0.70	0.77	0.39	0.72	0.49		
2 × 2 × ⅜	4.7	1.36	0.48	0.59	0.35	0.64	0.39		
¼	3.19	0.94	0.35	0.61	0.25	0.59	0.39		
⅛	1.65	0.48	0.19	0.63	0.13	0.55	0.40		
1¾ × 1¾ × ¼	2.77	0.81	0.23	0.53	0.19	0.53	0.34		
⅛	1.44	0.42	0.13	0.55	0.10	0.48	0.35		
1½ × 1½ × ¼	2.34	0.69	0.14	0.45	0.13	0.47	0.29		
⅛	1.23	0.36	0.08	0.47	0.07	0.42	0.30		
1¼ × 1¼ × ¼	1.92	0.56	0.08	0.37	0.09	0.40	0.24		
⅛	1.01	0.30	0.04	0.38	0.05	0.36	0.25		
1 × 1 × ¼	1.49	0.44	0.04	0.29	0.06	0.34	0.20		
⅛	0.80	0.23	0.02	0.30	0.03	0.30	0.20		

NOTE: 1 in = 2.5 cm; 1 ft = 0.305 m; 1 lb = 4.45 N.

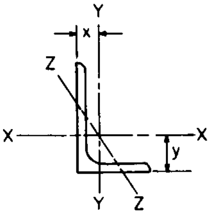


Table 12.2.22 Selected Standard Angles (L Shapes) Unequal Legs

(Intermediate thicknesses are available in each size group, varying by 1/16 in among the thinner angles)
A single angle should never be used as a beam. Two angles, bolted at frequent intervals, may be used.

Size, in	Thickness, in	Weight per ft, lb	Area of section, in²	Axis *X-X*				Axis *Y-Y*				Axis *Z-Z*	Net areas after deducting holes for ⅞-in rivets	
				I, in⁴	*S*, in³	*r*, in	*y*, in	*I*, in⁴	*S*, in³	*r*, in	*y*, in	*r*, in	1 hole	2 holes
8 × 6	1	44.2	13.00	80.8	15.1	2.49	2.65	38.8	8.9	1.73	1.65	1.28	12.00	11.00
	¾	33.8	9.94	63.4	11.7	2.53	2.56	30.7	6.9	1.76	1.56	1.29	9.19	8.44
	½	23.0	6.75	44.3	8.0	2.56	2.47	21.7	4.8	1.79	1.47	1.30	6.25	5.75
	7/16	20.2	5.93	39.2	7.1	2.57	2.45	19.3	4.2	1.80	1.45	1.31	5.49	5.06
8 × 4	1	37.4	11.00	69.6	14.1	2.52	3.05	11.6	3.9	1.03	1.05	0.85	10.00	9.00
	¾	28.7	8.44	54.9	10.9	2.55	2.95	9.4	3.1	1.05	0.95	0.85	7.69	6.94
	½	19.6	5.75	38.5	7.5	2.59	2.86	6.7	2.2	1.08	0.86	0.86	5.25	4.75
	7/16	17.2	5.06	34.1	6.6	2.60	2.83	6.0	1.9	1.09	0.83	0.87	4.62	4.18
7 × 4	⅞	30.2	8.86	42.9	9.7	2.20	2.55	10.2	3.5	1.07	1.05	0.86	7.98	7.11
	¾	26.2	7.69	37.8	8.4	2.22	2.51	9.1	3.0	1.09	1.01	0.86	6.94	6.19
	½	17.9	5.25	26.7	5.8	2.25	2.42	6.5	2.1	1.11	0.92	0.87	4.75	4.25
	⅜	13.6	3.98	20.6	4.4	2.27	2.37	5.1	1.6	1.13	0.87	0.88	3.62	3.24
6 × 4	⅞	27.2	7.98	27.7	7.2	1.86	2.12	9.8	3.4	1.11	1.12	0.86	7.10	6.23
	¾	23.6	6.94	24.5	6.3	1.88	2.08	8.7	3.0	1.12	1.08	0.86	6.19	5.44
	½	16.2	4.75	17.4	4.3	1.91	1.99	6.3	2.1	1.15	0.99	0.87	4.25	3.75
	⅜	12.3	3.61	13.5	3.3	1.93	1.94	4.9	1.6	1.17	0.94	0.88	3.24	2.86
	5/16	10.3	3.03	11.4	2.8	1.94	1.92	4.2	1.4	1.17	0.92	0.88	2.72	2.40
6 × 3½	½	15.3	4.50	16.6	4.2	1.92	2.08	4.3	1.6	0.97	0.83	0.76	4.00	3.50
	⅜	11.7	3.42	12.9	3.2	1.94	2.04	3.3	1.2	0.99	0.79	0.77	3.04	2.67
	¼	7.9	2.31	8.9	2.2	1.96	1.99	2.3	0.85	1.01	0.74	0.78	2.06	1.81
5 × 3½	¾	19.8	5.81	13.9	4.3	1.55	1.75	5.6	2.2	0.98	1.00	0.75	5.06	4.31
	½	13.6	4.00	10.0	3.0	1.58	1.66	4.1	1.6	1.01	0.91	0.75	3.50	3.00
	¼	7.0	2.06	5.4	1.6	1.61	1.56	2.2	0.83	1.04	0.81	0.76	1.81	1.56
5 × 3	½	12.8	3.75	9.5	2.9	1.59	1.75	2.6	1.1	0.83	0.75	0.65	3.25	2.75
	⅜	9.8	2.86	7.4	2.2	1.61	1.70	2.0	0.89	0.84	0.70	0.65	2.48	2.11
	¼	6.6	1.94	5.1	1.5	1.62	1.66	1.4	0.61	0.86	0.66	0.66	1.69	1.44
4 × 3½	⅝	14.7	4.30	6.4	2.4	1.22	1.29	4.5	1.8	1.03	1.04	0.72	3.68	3.05
	½	11.9	3.50	5.3	1.9	1.23	1.25	3.8	1.5	1.04	1.00	0.72	3.00	2.50
	⅜	9.1	2.67	4.2	1.5	1.25	1.21	3.0	1.2	1.06	0.96	0.73	2.30	1.92
	¼	6.2	1.81	2.9	1.0	1.27	1.16	2.1	0.81	1.07	0.91	0.73	1.56	1.31
4 × 3	⅝	13.6	3.98	6.0	2.3	1.23	1.37	2.9	1.4	0.85	0.87	0.64	3.36	2.73
	½	11.1	3.25	5.1	1.9	1.25	1.33	2.4	1.1	0.86	0.83	0.64	2.75	2.25
	¼	5.8	1.69	2.8	1.0	1.28	1.24	1.4	0.60	0.90	0.74	0.65	1.44	1.19
3½ × 3	½	10.2	3.00	3.5	1.5	1.07	1.13	2.3	1.1	0.88	0.88	0.62	2.50	
	¼	5.4	1.56	1.9	0.78	1.11	1.04	1.3	0.59	0.91	0.79	0.63	1.31	
3½ × 2½	½	9.4	2.75	3.2	1.4	1.09	1.20	1.4	0.76	0.70	0.70	0.53	2.25	
	¼	4.9	1.44	1.8	0.75	1.12	1.11	0.78	0.41	0.74	0.61	0.54	1.19	
3 × 2½	½	8.5	2.50	2.1	1.0	0.91	1.00	1.3	0.74	0.72	0.75	0.52		
	⅜	6.6	1.92	1.7	0.81	0.93	0.96	1.0	0.58	0.74	0.71	0.52		
	¼	4.5	1.31	1.2	0.56	0.95	0.91	0.74	0.40	0.75	0.66	0.53		
3 × 2	½	7.7	2.25	1.9	1.0	0.92	1.08	0.67	0.47	0.55	0.58	0.43		
	3/16	3.07	0.90	0.84	0.41	0.97	0.97	0.31	0.20	0.58	0.47	0.44		
2½ × 2	⅜	5.3	1.55	0.91	0.55	0.77	0.83	0.51	0.36	0.58	0.58	0.42		
	3/16	2.75	0.81	0.51	0.29	0.79	0.76	0.29	0.20	0.60	0.51	0.43		
2 × 1½	¼	2.77	0.81	0.32	0.24	0.62	0.66	0.15	0.14	0.43	0.41	0.32		
	⅛	1.44	0.42	0.17	0.13	0.64	0.62	0.09	0.08	0.45	0.37	0.33		
1¾ × 1¼	¼	2.34	0.69	0.20	0.18	0.54	0.60	0.09	0.10	0.35	0.35	0.27		
	⅛	1.23	0.36	0.11	0.09	0.56	0.56	0.05	0.05	0.37	0.31	0.27		

NOTE: 1 in = 2.54 cm; 1 ft = 0.305 m; 1 lb = 4.45 N.

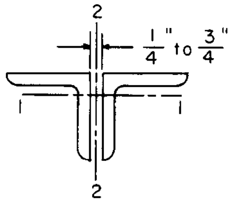


Table 12.2.23 Radii of Gyration for Two Angles, Unequal Legs

Single angle		Two angles	Radii of gyration, in							
			Long legs vertical				Short legs vertical			
				Axis 2-2				Axis 2-2		
Size, in	Weight per ft, lb	Area, in²	Axis 1-1	In contact	⅜ in apart	¾ in apart	Axis 1-1	In contact	⅜ in apart	¾ in apart
8 × 6 × 1	44.2	26.00	2.49	2.39	2.52	2.66	1.73	3.64	3.78	3.92
¾	33.8	19.9	2.53	2.35	2.48	2.62	1.76	3.60	3.74	3.88
8 × 4 × 1	37.4	22.00	2.52	1.47	1.61	1.76	1.03	3.95	4.10	4.25
½	19.6	11.50	2.59	1.38	1.51	1.64	1.08	3.86	4.00	4.14
7 × 4 × ¾	26.2	15.4	2.22	1.48	1.62	1.76	1.09	3.35	3.48	3.64
⅜	13.6	7.96	2.27	1.43	1.55	1.68	1.13	3.28	3.42	3.56
6 × 4 × ¾	23.6	13.9	1.88	1.55	1.69	1.83	1.12	2.80	2.94	3.09
⅜	12.3	7.22	1.93	1.50	1.62	1.76	1.17	2.74	2.87	3.02
5 × 3½ × ¾	19.8	11.62	1.55	1.40	1.54	1.69	0.98	2.34	2.48	2.63
5⁄16	8.7	5.12	1.61	1.33	1.45	1.59	1.03	2.26	2.38	2.53
4 × 3½ × ½	11.9	7.00	1.23	1.44	1.58	1.72	1.04	1.76	1.89	2.04
5⁄16	7.7	4.50	1.26	1.42	1.55	1.69	1.07	1.73	1.86	2.00
4 × 3 × ½	11.1	6.50	1.25	1.20	1.33	1.48	0.86	1.82	1.96	2.11
¼	5.8	3.38	1.28	1.16	1.29	1.43	0.90	1.78	1.92	2.06
3½ × 3 × ⅜	7.9	4.59	1.09	1.22	1.36	1.50	0.90	1.53	1.67	1.82
¼	5.4	3.12	1.11	1.21	1.34	1.48	0.91	1.52	1.65	1.80
3 × 2½ × ⅜	6.6	3.84	0.93	1.02	1.16	1.31	0.74	1.34	1.48	1.63
¼	4.5	2.62	0.95	1.00	1.13	1.28	0.75	1.31	1.45	1.60
2½ × 2 × ⅜	5.3	3.10	0.77	0.82	0.96	1.11	0.58	1.13	1.27	1.43
¼	3.6	2.12	0.78	0.80	0.94	1.09	0.59	1.11	1.25	1.40

NOTE: 1 in = 2.54 cm; 1 ft = 0.305 m; 1 lb = 4.45 N.

dard beams (S shapes) or wide-flange sections (W shapes) length-wise at midheight of the web, making two similar shapes of T section. Table 12.2.25 lists a selection of such tees.

For additional data regarding structural shapes, their strengths as beams and columns, and means of making connections, see ''AISC Manual of Steel Construction.''

Welding The main advantage of assembling and connecting steel frames by welding is the reduction in the amount of metal used. The saving in metal is achieved by (1) elimination of bolt holes which reduce the net section of tension members, (2) simplification of details, and (3) elimination of splice plate and gusset plate material. (See also Sec. 13.3.)

Allowable stresses in welds depend on the type of weld, the manner of loading, and the relative strengths of the weld metal and the base metal. For **complete-penetration groove welds** in which the full edge thickness of the thinner part to be joined is beveled in preparation for welding, allowable stresses due to tension or compression normal to the effective area or parallel to the axis of the weld are the same as the base metal, and the allowable shear stress on the effective area is 0.3 times the nominal tensile strength of the weld metal (limited by 0.4 times yield stress in the base metal). The effective area for a complete-penetration groove weld is the width of the part joined times the thickness of the thinner part.

For **partial-penetration groove welds,** allowable stresses due to compression normal to the effective area and tension or compression parallel to the axis of the weld are the same as the base metal. For tension normal to the effective area, the allowable stress is 0.3 times the nominal tensile strength of the weld metal (limited by 0.6 times the yield stress of the base metal); for shear parallel to the axis of the weld, the allowable stress on the effective area is 0.3 times the nominal tensile strength of weld metal (limited by 0.4 times yield stress of the base metal). The effective thickness of partial-penetration groove welds depends on the welding process, welding position, and the included angle of the groove but may be safely taken as the depth of the chamfer less ⅛ in.

For **fillet welds,** allowable shear stresses on the effective area are taken as 0.3 times nominal tensile strength of the weld (limited by 0.4 times yield stress of the base metal), and for tension or compression parallel to the axis of the weld, allowable stresses are the same as the base metal. Fillet welds are not to be loaded in tension or compression normal to the effective area; they transfer loads between members in shear only. The effective area of a fillet weld is the overall length of the full-size weld times the shortest distance from the root to the face (normally leg size × sin 45°). Allowable shear in a fillet weld is taken as 930 lb/in of length (163 N/mm) for each 1⁄16 in (1.59 mm) of leg size. Fillet welds should be limited to ½ in (12.5 mm) leg size in normal construction and to the thickness of the material up to ¼ in and thickness less 1⁄16 in for thicker material. The minimum-size fillet weld for ¼-in-thick material should

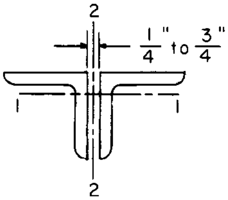


Table 12.2.24 Radii of Gyration for Two Angles, Equal Legs

Single angle		Two angles	Radii of gyration, in			
				Axis 2-2		
Size, in	Weight per ft, lb	Area, in²	Axis 1-1	In contact	⅜ in apart	¾ in apart
8 × 8 × 1⅛	56.9	33.46	2.42	3.42	3.55	3.69
½	26.4	15.50	2.50	3.33	3.45	3.59
6 × 6 × 1	37.4	22.00	1.80	2.59	2.72	2.87
⅜	14.9	8.72	1.88	2.49	2.62	2.76
5 × 5 × ⅞	27.2	15.96	1.49	2.17	2.31	2.45
⅜	12.3	7.22	1.56	2.09	2.22	2.35
4 × 4 × ¾	18.5	10.88	1.19	1.74	1.88	2.03
¼	6.6	3.88	1.25	1.66	1.79	1.93
3½ × 3½ × ⅜	8.5	4.97	1.07	1.48	1.61	1.75
¼	5.8	3.38	1.09	1.46	1.59	1.73
3 × 3 × ½	9.4	5.50	0.90	1.29	1.43	1.58
¼	4.9	2.88	0.93	1.25	1.38	1.53
2½ × 2½ × ⅜	5.9	3.47	0.75	1.07	1.21	1.36
¼	4.1	2.38	0.77	1.05	1.19	1.34
2 × 2 × ⅜	4.7	2.72	0.59	0.87	1.02	1.18
¼	3.19	1.88	0.61	0.85	0.99	1.14

NOTE: 1 in = 2.54 cm; 1 ft = 0.305 m; 1 lb = 4.45 N.

be ⅛ in; for over ¼- to ½-in material, 3⁄16 in; for over ½-in to ¾-in material, ¼ in; and for over ¾-in-thick material, 5⁄16 in. Typical details of welded connections are indicated in Fig. 12.2.18.

Safe Loads for Steel Beams To determine the safe load uniformly distributed, as limited by bending, for a structural steel beam on a given span, apply the formula $W = 8F_b s/l$, where W is the total load, lb.; F_b is the allowable fiber stress (24,000 lb/in² or any other); S is the section

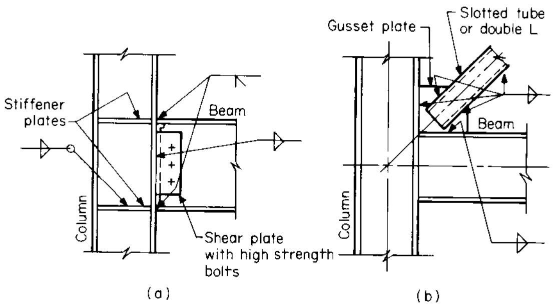


Fig. 12.2.18 Welded connections. (*a*) Moment connection; (*b*) bracing connection.

modulus for the beam in question, given in Tables 12.2.18 to 12.2.26; and l is the span, in. (This formula may also be used with equivalent metric units.) The safe load concentrated at midspan is one-half this amount. For other safe loads, note that $F_b S$ is the safe resistance to bending in inch-pounds (or newton-meters) afforded by the beam. Compute the load, of whatever type or distribution, which will produce a maximum bending moment equal to safe moment of resistance (see Sec. 5 for bending-moment formulas).

To select a beam to support a given load, compute the maximum bending moment in inch-pounds, divide by the allowable fiber stress F_b, and refer to the table for a beam having a section modulus which is not smaller than the quotient.

Formulas for the safe loads and deflections of beams with various methods of support and of loading are given in Sec. 5.

Short beams should be investigated for crippling of the web. In the tables are given the safe end reactions for beams of A36 steel resting on a seat 3½ in (9 cm) long along the axis of the beam. Short beams should also be investigated for shear, by dividing the maximum shear, in pounds, by the area of the web, excluding the flanges.

Single angles used as beams and loaded in the plane of axis *X-X* or *Y-Y* tend to deflect laterally as well as in the plane of the loads. Unless this is prevented, as by pairing the angles back to back and securing them together, the unit fiber stress due to bending may be as much as 40 percent above that computed by dividing the bending moment by S for the axis perpendicular to the plane of the loads. The relation $f = M/S$ does not hold for single angles, and Z bars, which are unsymmetrical about both axes.

Deflection of I Beams and Other Structural Shapes Table 12.2.27 gives **coefficients of deflection** for steel shapes under uniformly distributed loads, and is based on the formula; deflection in inches = $30fL^2/Ed$, the table giving the values of $30fL^2/E$. (f = fiber stress, lb/in², L = span, ft; d = depth of section, in; E = modulus of elasticity = 29,000,000 lb/in².)

To find the deflection in inches of a section symmetrical about the neutral axis, such as a beam, channel, etc., divide the coefficient in the table corresponding to given span and fiber stress by the depth of the section in inches.

To find the deflection in inches of a section which is not symmetrical about the neutral axis but which is symmetrical about an axis at right angles thereto, such as a tee or pair of angles, divide the coefficient corresponding to given span and fiber stress by twice the distance of extreme fiber from neutral axis obtained from table of elements of sections.

To find the deflection in inches of a section for any other fiber stress than those given, multiply this fiber stress by either of the coefficients in the table for the given span and divide by the fiber stress corresponding to the coefficients used.

I beams and channels loaded to a fiber stress of 24,000 lb/in^2 (165.5 MPa) will not deflect in excess of 1⁄360 of the span (allowed for plastered ceilings) if the depth in inches (cm) is not less than 0.74 (6.21) times the span in feet (m) for uniform loads and 0.60 (5.00) times the span for central concentration.

Beam Supports Steel beams are supported at the ends generally (1) by means of web connections to girders and columns, (2) by resting on structural-steel seats, or (3) by resting on masonry. Limiting values of end reactions of the second type, for seats 3½ in (9 cm) long, are given in Tables 12.2.18 to 12.2.20. Standard AISC web connections of the first type are called *framed beam* connections and are designated by the number of rows of bolts. Examples of connections are given in Fig. 12.2.19. These connections may be specified as ''Standard 3 row, 4 row, etc., connections.'' Connections must always be designated and detailed for the calculated design reaction.

The capacity of web connections is governed by the shearing of the fastener, or the bearing of the fastener on the web or on the material to which the beam is connected, or by the strength of the connecting angles. The supporting values of standard framed beam connections, using ⅞ in fasteners in members of A36 material, are given in Table 12.2.28. For fasteners in webs thicker than 0.34 in use the values in the column headed Double Shear; for thinner webs, bearing limits the value, and the coefficients for web bearing are to be used. For ¾-in fasteners, multiply tabular bearing values by 6⁄7 and shear values by 36⁄49. Fasteners connecting the outstanding legs to the supporting metal are in double shear if two beams are framed opposite or in single shear if a beam is connected on one side only. If the supporting material of A36 steel is thinner than 0.34 in in double-shear connections or thinner than 0.17 in in single-shear connections, the capacity is limited by bearing. The value of any ⅞-in fastener in bearing on A36 material is 60,900t, where t is the thickness of the plate. The value of ⅞ in A502, grade 1 rivets or A325 HS bolts (slip-critical connections) is 9,020 lb in single shear and 20,400 lb in double shear. The corresponding values for A307 unfinished bolts are 6,000 lb and 12,000 lb, respectively.

Cast-iron columns were often used in the past instead of wood, to save space, in the lower stories of heavy buildings. Their use is now obsolete, but they are occasionally encountered in repair and alteration work to older buildings. The ratio of length to least radius of gyration l/r should not exceed 70, and the average unit stress under axial compression should not exceed $9{,}000 - 40l/r$ lb/in^2.

Steel joists consisting of lightweight rolled sections, thin for their height, or open-web trussed members fabricated by welding or otherwise, are used with economy in buildings where spans are long and loads are light, and where a plaster ceiling affords sufficient fire protection. They are rarely used in industrial buildings, except to support roof loads.

Steel pipe is often used for columns under light loads. Table 12.2.29 gives the safe loads on standard size pipes (ASTM A501 or A53, grade B) used as columns. For extra-strong and double extra-strong pipe used as columns, the safe loads will increase approximately in the same proportion as the weight per foot. (See Sec. 8.7)

Structural steel tubing (ASTM A500, grade B), in square or rectangular cross section with $F_y = 46$ Ksi (317.1 MPa), is also used for columns, bracing members, and unbraced members subjected to large torsional loads. The closed box shape makes tube sections especially suited for resisting torsional loads.

Corrugated metal deck and siding is used for roofs and walls, respectively, to span between purlins for roof loads or between girts for wind loads. The decking is sized to resist the bending caused by these loads. Roof decking is often also used as a diaphragm to transfer wind or seismic loads to the lateral bracing system below. Load tables specifying safe loads for different spans are available from metal deck manufacturers.

The spacing of purlins on roofs and girts on wall is usually 4 to 6 ft. Numbers 20 and 22, U.S. Standard gage, are generally used for roofing; No. 24 for siding.

Fire Resistance The resistance to fire of building materials has been tested extensively by various agencies. Table 12.2.30 gives the fire-resistance rating of a few of the common building materials and methods of construction as established by the Uniform Building Code from standard fire tests.

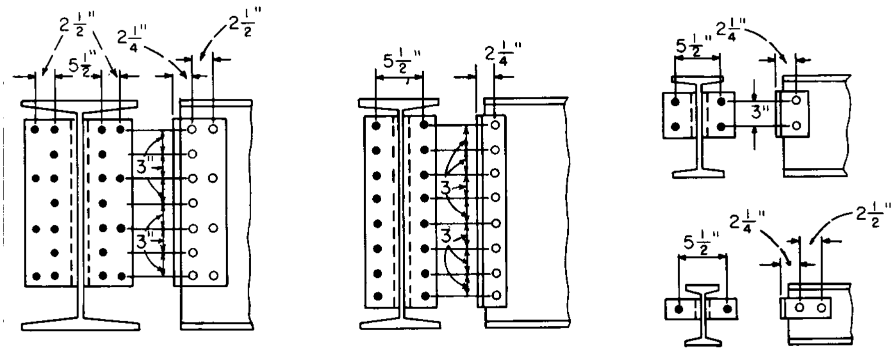


Fig. 12.2.19 Framed beam connections.

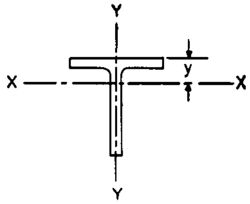
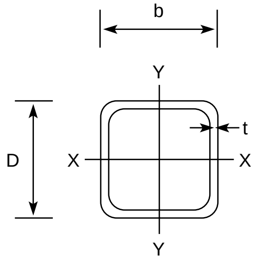


Table 12.2.25 Tees Cut from Standard Sections (WT and ST Shapes)*

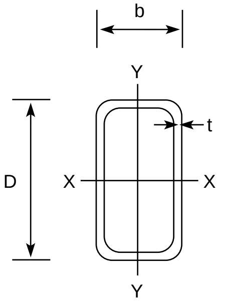
				Flange			Axis *X-X*				Axis *Y-Y*		
Nominal depth, in	Weight per ft, lb	Area, in^2	Depth, of tee, in	Width in	Avg thick-ness, in	Stem thick-ness, in	I, in^4	S, in^3	r, in	y, in	I, in^4	S, in^3	r, in
WT18	150	44.1	18.37	16.66	1.68	0.95	1,230	86.1	5.27	4.13	648	77.8	3.83
	115	33.8	17.95	16.47	1.26	0.76	934	67.0	5.25	4.01	470	57.1	3.73
	97	28.5	18.25	12.12	1.26	0.77	901	67.0	5.62	4.80	187	30.9	2.56
	75	22.1	17.93	11.98	0.94	0.63	698	53.1	5.62	4.78	135	22.5	2.47
WT16.5	120.5	35.4	17.09	15.86	1.40	0.83	875	65.0	4.96	3.85	466	58.8	3.63
	110.5	32.5	16.97	15.81	1.28	0.78	799	60.8	4.96	3.81	420	53.2	3.59
	100.5	29.5	16.84	15.75	1.15	0.72	725	55.5	4.95	3.70	375	47.6	3.56
	65	19.2	16.55	11.51	0.86	0.58	513	42.1	5.18	4.36	109	18.9	2.39
WT15	106.5	31.0	15.47	15.11	1.32	0.78	610	50.5	4.43	3.40	378	50.1	3.49
	86.5	25.4	15.22	14.99	1.07	0.66	497	41.7	4.42	3.31	299	39.9	3.43
	62	18.2	15.09	10.52	0.93	0.59	396	35.3	4.66	3.90	90.4	17.2	2.23
	49.5	14.5	14.83	10.45	0.67	0.52	322	30.0	4.71	4.09	63.9	12.2	2.10
WT113.5	97	28.5	14.06	14.04	1.34	0.75	444	40.3	3.95	3.03	309	44.1	3.29
	80.5	23.7	13.80	14.02	1.08	0.66	372	34.4	3.96	2.99	248	35.4	3.24
	57	16.8	13.65	10.07	0.93	0.57	289	28.3	4.15	3.42	79.4	15.8	2.18
	42	12.4	13.36	9.96	0.64	0.46	216	21.9	4.18	3.48	52.8	10.6	2.07
WT12	81	23.9	12.50	12.96	1.27	0.71	293	29.9	3.50	2.70	221	34.2	3.05
	52	15.3	12.03	12.75	0.75	0.50	189	20.0	3.51	2.59	130	20.3	2.91
	42	12.4	12.05	9.02	0.77	0.47	166	18.3	3.67	2.97	47.2	10.5	1.95
	27.5	8.1	11.79	7.01	0.51	0.40	117	14.1	3.80	3.50	14.5	4.15	1.34

WT10.5	41.5	12.2	10.72	8.36	0.84	0.52	127	15.7	3.22	2.66	40.7	9.75	1.83
	34	10.0	10.57	8.27	0.69	0.43	103	12.9	3.20	2.59	32.4	7.83	1.80
	28.5	8.37	10.53	6.56	0.65	0.41	90.4	11.8	3.29	2.85	15.3	4.67	1.35
	22	6.49	10.33	6.50	0.45	0.35	71.1	9.7	3.31	2.98	10.3	3.18	1.26
WT9	65	19.1	9.63	11.16	1.20	0.67	127	16.7	2.50	2.02	139	24.9	2.70
	53	15.6	9.37	11.20	0.94	0.59	104	14.1	2.59	1.97	110	19.7	2.66
	30	8.82	9.12	7.56	0.70	0.42	64.7	9.3	2.71	2.16	25	6.63	1.69
	20	5.88	8.95	6.02	0.53	0.32	44.8	6.73	2.76	2.29	9.6	3.17	1.27
WT8	38.5	11.3	8.26	10.30	0.76	0.46	56.9	8.59	2.24	1.63	69.2	13.4	2.47
	25	7.37	8.13	7.07	0.63	0.38	42.3	6.78	2.40	1.89	18.6	5.26	1.59
	20	5.89	8.01	7.00	0.51	0.31	33.1	5.35	2.37	1.81	14.4	4.12	1.57
	15.5	4.56	7.94	5.23	0.44	0.28	27.4	4.64	2.45	2.02	6.20	2.24	1.17
WT7	155.5	45.7	8.56	16.23	2.26	1.41	176	26.7	1.96	1.97	807	99.4	4.20
	60	17.7	7.24	14.67	0.94	0.59	51.7	8.61	1.71	1.24	247	33.7	3.74
	41	12.0	7.16	10.13	0.89	0.51	41.2	7.14	1.85	1.39	74.2	14.6	2.48
	17	5.0	6.99	6.75	0.46	0.29	20.9	3.83	2.04	1.53	11.7	3.45	1.53
WT6	95	27.9	7.19	12.67	1.74	1.06	79.0	14.2	1.68	1.62	295	46.5	3.25
	48	14.1	6.36	12.16	0.90	0.55	32.0	6.12	1.51	1.13	135	22.2	3.09
	32.5	9.54	6.06	12.00	0.61	0.39	20.6	4.06	1.47	0.99	87.2	14.5	3.02
	20	5.89	5.97	8.01	0.52	0.30	14.4	2.95	1.57	1.08	22.0	5.51	1.93
WT5	44	12.9	5.42	10.27	0.99	0.61	20.8	4.77	1.27	1.06	89.3	17.4	2.63
	30	8.82	5.11	10.08	0.68	0.42	12.9	3.04	1.21	0.88	58.1	11.5	2.57
	22.5	5.73	4.96	7.99	0.53	0.32	8.84	2.16	1.24	0.88	22.5	5.64	1.98
	15	4.42	5.24	5.81	0.51	0.30	9.28	2.24	1.45	1.1	8.35	2.87	1.37
WT4	33.5	9.84	4.50	8.28	0.94	0.57	10.9	3.05	1.05	0.94	44.3	10.7	2.12
	20	5.87	4.13	8.07	0.56	0.36	5.73	1.69	0.99	0.74	24.5	6.08	2.04
	14	4.12	4.03	6.34	0.47	0.29	4.22	1.28	1.01	0.73	10.8	3.31	1.62
	7.5	2.22	4.06	4.02	0.32	0.25	3.28	1.07	1.22	1.0	1.7	0.85	0.87
ST 9	35	10.3	9.00	6.25	0.69	0.71	84.7	14.0	2.87	2.94	12.1	3.86	1.08
6	25	7.35	6.00	5.48	0.66	0.69	25.2	6.05	1.85	1.84	7.85	2.87	1.03
4	11.5	3.38	4.00	4.17	0.43	0.44	5.03	1.77	1.22	1.15	2.15	1.03	0.80
3	6.25	1.83	3.00	3.33	0.36	0.23	1.27	0.56	0.83	0.69	0.91	0.55	0.71

NOTE: 1 in = 2.54 cm; 1 ft = 0.305 m; 1 lb = 4.45 N.
* The availability of WT sections listed is governed by the basic W sections from which they are cut. See footnote under Table 12.2.20.



Square



Rectangular

Table 12.2.26 Properties of Square and Rectangular Tubing (TS Sections)*

Nominal Size D, in × b, in	t, in	Weight, lb/ft	Area of metal, in^2	I_{xx}, in^4	S_{xx}, in^3	r_{xx}, in	I_{yy}, in^4	S_{yy}, in^3	r_{yy}, in
TS 12 × 12	0.5	76.07	22.4	485	80.9	4.66	485	80.9	4.66
12	0.375	58.1	17.1	380	63.4	4.72	380	63.4	4.72
TS 10 × 10	0.5	62.46	18.4	271	54.2	3.84	271	54.2	3.84
10	0.375	47.9	14.1	214	42.9	3.9	214	42.9	3.9
10	0.25	32.63	9.59	151	30.1	3.96	151	30.1	3.96
TS 8 × 8	0.5	48.85	14.4	131	32.9	3.03	131	32.9	3.03
8	0.375	37.69	11.1	106	26.4	3.09	106	26.4	3.09
8	0.25	25.82	7.59	75.1	18.8	3.15	75.1	18.8	3.15
TS 6 × 6	0.5	35.24	10.4	50.5	16.8	2.21	50.5	16.8	2.21
6	0.375	27.48	8.08	41.6	13.9	2.27	41.6	13.9	2.27
6	0.25	19.02	5.59	30.3	10.1	2.33	30.3	10.1	2.33
6	0.1875	14.53	4.27	23.8	7.93	2.36	23.8	7.93	2.36
TS 5 × 5	0.5	28.43	8.36	27.0	10.8	1.80	27.0	10.8	1.80
	0.375	22.37	6.58	22.8	9.11	1.86	22.8	9.11	1.86
	0.25	15.62	4.59	16.9	6.78	1.92	16.9	6.78	1.92
	0.1875	11.97	3.52	13.4	5.36	1.95	13.4	5.36	1.95
TS 4 × 4	0.5	21.63	6.36	12.3	6.13	1.39	12.3	6.13	1.39
4	0.375	17.27	5.08	10.7	5.35	1.45	10.7	5.35	1.45
4	0.25	12.21	3.59	8.22	4.11	1.51	8.22	4.11	1.51
4	0.1875	9.42	2.77	6.59	3.3	1.54	6.59	3.3	1.54
TS 3 × 3	0.375	10.58	3.11	3.58	2.39	1.07	3.58	2.39	1.07
	0.25	8.81	2.59	3.16	2.10	1.10	3.16	2.10	1.10
	0.1875	6.87	2.02	2.60	1.73	1.13	2.60	1.73	1.13
TS 2 × 2	0.3125	6.32	1.86	0.815	0.815	0.662	0.815	0.815	0.662
2	0.25	5.41	1.59	0.766	0.766	0.694	0.766	0.766	0.694
2	0.1875	4.32	1.27	0.668	0.668	0.726	0.668	0.668	0.726
TS 20 × 12	0.5	103.3	30.4	1,650	165	7.37	750	125	4.97
12	0.375	78.52	23.1	1,280	128	7.45	583	97.2	5.03
8	0.5	89.68	26.4	1,270	127	6.94	300	75.1	3.38
8	0.375	68.31	20.1	988	98.8	7.02	236	59.1	3.43
8	0.3125	57.36	16.9	838	83.8	7.05	202	50.4	3.46
TS16 × 12	0.5	89.68	26.4	962	120	6.04	618	103	4.84
12	0.375	68.31	20.1	748	93.5	6.11	482	80.3	4.9
8	0.5	76.07	22.4	722	90.2	5.68	244	61	3.3
8	0.375	58.1	17.1	565	70.6	5.75	193	48.2	3.36
8	0.3125	48.86	14.4	481	60.1	5.79	165	41.2	3.39
TS 12 × 6	0.625	67.82	19.9	337	56.2	4.11	112	37.2	2.37
6	0.5	55.66	16.4	287	47.8	4.19	96	32	2.42
6	0.375	42.79	12.6	228	38.1	4.26	77.2	25.7	2.48
6	0.25	29.23	8.59	161	26.9	4.33	55.2	18.4	2.53
6	0.1875	22.18	6.52	124	20.7	4.37	42.8	14.3	2.56
TS 12 × 4	0.625	59.32	17.4	257	42.8	3.84	41.8	20.9	1.55
4	0.5	48.85	14.4	221	36.8	3.92	36.9	18.5	1.6
4	0.375	37.69	11.1	178	29.6	4.01	30.5	15.2	1.66
4	0.25	25.82	7.59	127	21.1	4.09	22.3	11.1	1.71
4	0.1875	19.63	5.77	98.2	16.4	4.13	17.5	8.75	1.74

* On special order, TS sections currently are available in sizes up to 30 × 30 and 30 × 24.

Table 12.2.26 Properties of Square and Rectangular Tubing (TS Sections)* (*Continued*)

Nominal Size D, in × b, in	t, in	Weight, lb/ft	Area of metal, in²	I_{xx}, in⁴	S_{xx}, in³	r_{xx}, in	I_{yy}, in⁴	S_{yy}, in³	r_{yy}, in
TS 10 × 4	0.5	42.05	12.4	136	27.1	3.31	30.8	15.4	1.58
4	0.375	32.58	9.58	110	22	3.39	25.5	12.8	1.63
4	0.25	22.42	6.59	79.3	15.9	3.47	18.8	9.39	1.69
TS 8 × 6	0.5	42.05	12.4	103	25.8	2.89	65.7	21.9	2.31
6	0.375	32.58	9.58	83.7	20.9	2.96	53.5	17.8	2.36
6	0.25	22.42	6.59	60.1	15	3.02	38.6	12.9	2.42
4	0.625	42.3	12.4	85.1	21.3	2.62	27.4	13.7	1.49
4	0.5	35.24	10.4	75.1	18.8	2.69	24.6	12.3	1.54
4	0.375	27.48	8.08	61.9	15.5	2.77	20.6	10.3	1.6
4	0.25	19.02	5.59	45.1	11.3	2.84	15.3	7.63	1.65
2	0.375	22.37	6.58	40.1	10	2.47	3.85	3.85	0.765
2	0.25	15.62	4.59	30.1	7.52	2.56	3.08	3.08	0.819
TS 6 × 4	0.5	28.43	8.36	35.3	11.8	2.06	18.4	9.21	1.48
4	0.375	22.37	6.58	29.7	9.9	2.13	15.6	7.82	1.54
4	0.25	15.62	4.59	22.1	7.36	2.19	11.7	5.87	1.6
4	0.1875	11.97	3.52	17.4	5.81	2.23	9.32	4.66	1.63
2	0.375	17.27	5.08	17.8	5.94	1.87	2.84	2.84	0.748
2	0.25	12.21	3.59	13.8	4.6	1.96	2.31	2.31	0.802
TS 4 × 3	0.25	10.51	3.09	6.45	3.23	1.45	4.1	2.74	1.15
3	0.1875	8.15	2.39	5.23	2.62	1.48	3.34	2.23	1.18
2	0.375	12.17	3.58	5.75	2.87	1.27	1.83	1.83	0.715
2	0.25	8.81	2.59	4.69	2.35	1.35	1.54	1.54	0.77
2	0.1875	6.87	2.02	3.87	1.93	1.38	1.29	1.29	0.798
TS 3 × 2	0.25	7.11	2.09	2.21	1.47	1.03	1.15	1.15	0.742
2	0.1875	5.59	1.64	1.86	1.24	1.06	0.977	0.977	0.771
2	0.125	3.9	1.15	1.38	0.92	1.1	0.733	0.733	0.8

* On special order, TS sections currently are available in sizes up to 30 × 30 and 30 × 24.

Table 12.2.27 Coefficients of Deflection for Steel Beams under Uniformly Distributed Loads

Span, ft	Fiber stress, lb/in²		Span, ft	Fiber stress, lb/in²		Span, ft	Fiber stress, lb/in²		Span, ft	Fiber stress, lb/in²	
	24,000	10,000		24,000	10,000		24,000	10,000		24,000	10,000
1	0.026	0.011	14	4.87	2.029	27	18.1	7.54	39	37.7	15.7
2	0.098	0.041	15	5.59	2.328	28	19.5	8.12	40	39.8	16.6
3	0.223	0.093	16	6.36	2.648	29	20.9	8.71	41	41.8	17.4
4	0.398	0.166	17	7.18	2.990	30	22.4	9.32	42	43.9	18.3
5	0.621	0.259	18	8.04	3.35	31	23.9	9.94	43	45.8	19.1
6	0.892	0.372	19	8.97	3.74	32	25.4	10.60	44	48.0	20.0
7	1.23	0.507	20	9.93	4.14	33	27.0	11.27	45	50.4	21.0
8	1.59	0.662	21	10.9	4.56	34	28.7	11.96	46	52.6	21.9
9	2.01	0.838	22	12.1	5.01	35	30.5	12.7	47	54.7	22.8
10	2.48	1.034	23	13.1	5.47	36	32.2	13.4	48	57.1	23.8
11	3.00	1.251	24	14.3	5.96	37	34.1	14.2	49	59.5	24.8
12	3.58	1.489	25	15.6	6.47	38	35.8	14.9	50	62.2	25.9
13	4.20	1.748	26	16.8	7.00						

NOTE: For a load concentrated at midspan, use ⅘ of the coefficient given. 1 ft = 0.305 m; 1 lb/in² = 6.89 kPa.

Table 12.2.28 Values of Standard Framed-Beam Connections
(⅞-in A325 HS bolts in standard holes,* A36 members)

AISC designation	Two angles thickness × length	Shear 1,000 lb	Bearing on beam web (t), 1,000 lb
10 rows	5⁄16 × 2′5½″	204	609t
9 rows	5⁄16 × 2′2½″	184	548t
8 rows	5⁄16 × 1′11½″	164	487t
7 rows	5⁄16 × 1′8½″	143	426t
6 rows	5⁄16 × 1′5½″	123	365t
5 rows	5⁄16 × 1′2½″	102	304t
4 rows	5⁄16 × 0′11½″	81.8	243t
3 rows	5⁄16 × 0′8½″	61.3	182t
2 rows	5⁄16 × 0′5½″	40.9	121t

NOTE: 1 in = 2.54 cm; 1 lb = 4.45 N.

* Values indicated are for slip-critical connections or bearing type where threads are not excluded from the shear plane. For bearing-type connections where threads are excluded from the shear plane, shear values may be increased by 1.47.

† If the web of the supporting beam is thinner than 0.17 in (0.42 in if beams frame on both sides), bearing must also be investigated.

Table 12.2.29 Safe Axial Loads for Standard Pipe Columns, kips
(Stress according to AISC specification for A501 pipe*)

Nominal pipe size, in	Outside diam, in	Wall thickness, in	Effective length of column K1, ft										
			6	7	8	9	10	11	12	14	16	18	20
3	3.500	0.216	38	36	34	31	28	25	22	16	12	10	
3½	4.000	0.226	48	46	44	41	38	35	32	25	19	15	12
4	4.500	0.237	59	57	54	52	49	46	43	36	29	23	19
5	5.563	0.258	83	81	78	76	73	71	68	61	55	47	39
6	6.625	0.280	110	108	106	103	101	98	95	89	82	75	67
8	8.625	0.322	171	168	166	163	161	158	155	149	142	135	127
10	10.750	0.365	246	243	241	238	235	232	229	223	216	209	201
12	12.750	0.375	303	301	299	296	293	291	288	282	275	268	261

NOTE: 1 in = 2.54 cm; 1 ft = 0.305 m. For dimensions of standard pipe see Sec. 8. Safe loads above underscore lines are for values of K1/r more than 120 but not over 200.

* Yield stress is 36 ksi (248.2 MPa). Pipe ordered to ASTM A53, type E or S grade B, or to API standard 5L grade B will have a yield point of 35 ksi (241.3 MPa) and may be designed at stresses allowed for A501 pipe.

Table 12.2.30 Selected Fire-Resistance Ratings

Type	Details of construction	Rating
Reinforced-concrete beams and girders	Grade A concrete, 1½ in clear to reinforcement	4 h
	Grade B concrete, 1½ in clear to reinforcement	3 h
Steel beams, girders, and trusses	2½ in cover to steel	4 h
	1 in gypsum-perlite plaster on metal lath, 1¼ in clear of steel	3 h
	Ceiling of 1⅛ in gypsum-perlite plaster on metal lath with 2½ in min air space between lath and structural members	4 h
Reinforced concrete columns	Grade A concrete 1½ in clear to reinforcement; 12-in columns or larger	4 h
	Grade B concrete 2 in clear to reinforcement; 12-in columns or larger	4 h
Steel columns, 8 × 8 in or larger	Concrete (siliceous gravel):	
	2½ in clear to steel	4 h
	2 in clear to steel	3 h
	1 in clear to steel	2 h
	1⅛ in gypsum-perlite plaster on metal lath spaced from flanges with 1¼-in steel furring channels	4 h
	⅞ in portland-cement plaster on metal lath over ¾-in channels	1 h
Reinforced concrete slabs	5 in concrete (expanded clay, shale, slate, or slag) 1 in clear to reinforcement	4 h
	6½ in concrete (all other aggregate) 1 in clear to reinforcement	4 h
Heavy-timber floors	3 in tongue-and-groove plank floor with 1 in finish flooring	1 h
Wood joists	Wood floor; 1 in tongue-and-groove subfloor and 1 in finish floor with asbestos paper between. Ceiling of ⅝ in Underwriters' Laboratories listed wallboard	1 h
Brick walls	Solid walls, unplastered, with no combustible members framed in wall:	
	8 in nominal	4 h
	4 in nominal	1 h
Concrete masonry units	8 in Underwriters' Laboratories listed concrete blocks, laid as specified in Underwriters' Laboratories listing	4 h
	4 in Underwriters' Laboratories listed concrete blocks; laid as specified in Underwriters' Laboratories listing	3 h
Steel-stud partitions	¾ in gypsum-perlite plaster both sides on metal lath	2 h
	⅝ in gypsum wallboard on 3⅝-in steel studs; attached with 6 d nails; joints taped and cemented	2 h
Wooden-stud partitions	Exterior walls: one side covered with ½ in gypsum sheathing and wood siding; other side faced with ½ in gypsum-perlite plaster on ⅜-in perforated gypsum lath	1 h
	Interior Walls: 2 × 4 in studs with ⅝ in gypsum wallboard on each side	1 h
Plain or reinforced concrete walls	Solid, unplastered:	
	7 in thick	4 h
	6½ in thick	3 h
	5 in thick Grade B Grade A	2 h
	3½ in thick	1 h

NOTE: 1 in = 2.54 cm.
Grade A concrete is made with aggregates such as limestone, calcareous gravel, trap rock, slag, expanded clay, shale, or slate or any other aggregates possessing equivalent fire-resistance properties. Grade B concrete is all concrete other than Grade A concrete and includes concrete made with aggregates conaining more than 40 percent quartz, cherts, or flint.

12.3 REINFORCED CONCRETE DESIGN AND CONSTRUCTION

by William L. Gamble

REFERENCES: Breen, Jirsa and Ferguson, ''Reinforced Concrete Fundamentals,'' Wiley. Winter and Nilson, ''Design of Concrete Structures,'' McGraw-Hill. Lin and Burns, ''Design of Prestressed Concrete Structures,'' Wiley. Park and Gamble, ''Reinforced Concrete Slabs,'' Wiley. ''Building Code Requirements for Reinforced Concrete (318–95),'' ''Commentary on Building Code Requirements for Reinforced Concrete,'' ''Formwork for Concrete, SP-4,'' and ''Manual of Standard Practice for Detailing Reinforced Concrete Structures,'' American Concrete Institute. ''Standard Specifications for Highway Bridges,'' American Association of State Highway and Transportation Officials (AASHTO). ''Minimum Design Loads for Buildings and Other Structures (ASCE 7),'' American Society of Civil Engineers. ''Uniform Building Code (UBC),'' International Conference of Building Officials.

The design, theory, and notation of this chapter are in general accord with the 1995 Building Code Requirements for Reinforced Concrete of the American Concrete Institute, though many detailed provisions have been omitted.

Standard Notation

Load Factors

D = dead load of structure or force caused by dead load
E = earthquake load or force
L = live load of structure or force caused by live load
W = wind load or force
U = required strength of structure to resist design ultimate loads
ϕ = understrength or capacity reduction factor

Beams and General Notation

a = depth of compression zone, using approximate method
A_b = area of individual reinforcing bar, in^2
A_{ps} = area of prestressing steel
A_s = area of tension reinforcement
A'_s = area of compression reinforcement
A_v = area of steel in one stirrup
b = width of compression face of beam
b_w = width of stem of T beam
$c = k_u d$ = depth to neutral axis at ultimate, from compression face
d = effective depth of beam, compression face to centroid of tension steel
d' = depth of compression steel, from compression face
d_b = diam of individual reinforcing bar, in
E_c = Young's modulus of concrete
E_s = Young's modulus of steel
f'_c = compressive strength of concrete from tests of 6×12 in cylinders, lb/in^2
$\sqrt{f'_c}$ = measure of shear and tensile strength of concrete, lb/in^2, i.e., if $f'_c = 4{,}900$ lb/in^2, then $\sqrt{f'_c} = 70$ lb/in^2
f_{pu} = ultimate stress of prestressing steel, lb/in^2
f_y = yield stress of reinforcing steel, lb/in^2
h = overall height or thickness of member
k_u = ratio of ultimate neutral axis depth to effective depth
l_d = development length of reinforcing bar, in
s = spacing of shear reinforcement
M_u = ultimate moment of section or required ultimate moment
v_c = shear stress in concrete
V_c = shear force resisted by concrete
V_s = shear force resisted by web reinforcement
V_u = shear strength of section or required ultimate shear
β_1 = factor relating neutral axis position to depth of equivalent approximate stress block (see Fig. 12.3.2)
ε_{su} = reinforcement strain at time of failure of member
ε_y = yield strain of reinforcement
ρ = tension reinforcement ratio $= A_s/bd$
ρ' = compression reinforcement ratio $= A'_s/bd$
ρ_{bal} = balanced reinforcement ratio

Columns

$A_c = A_g - A_s$ = net area of concrete in cross section
A_{core} = area within spiral
A_g = gross area of column
A_s = area of steel in column
e = eccentricity of axial load on column
$M = Pe$ = applied bending moment
P = axial thrust
P_0 = failure load of short column under concentric load
ρ_g = gross steel ratio $= A_s/A_g$
ρ_s = spiral steel ratio = volume of spiral steel/volume of core

Floor Systems

b_0 = effective shear perimeter around column in flat plate, flat slab, or footing
c_1 = width of supporting column or capital, in direction of span being considered
c_2 = width of supporting column, in transverse direction
I_b = moment of inertia of beam
K_b = flexural stiffness of beam, moment per unit rotation
K_c = flexural stiffness of columns at joint, moment per unit rotation
K_s = flexural stiffness of slab of width l_2, moment per unit rotation
l_1 = span, center to center of supports, in direction considered
l_2 = span, center to center of supports, in transverse direction
$l_n = l_1 - c_1$ = clear span in direction considered
M_0 = static moment
w = distributed design load, including load factors
α_1 = EI of beam in direction 1/EI of slab of width l_2

Footings

A_1 = loaded area
A_2 = area of same shape and concentric with A_1
f_b = ultimate bearing stress on concrete
β = ratio of long side/short side of footing

Walls

l_c = unbraced height of wall

Prestressed Concrete

A_{ps} = area of prestressed reinforcement
A_t = area of anchorage-zone reinforcement
f'_{ci} = compressive strength of concrete at time of prestressing
f_{ps} = stress in reinforcement at time of failure of member
f_{pu} = ultimate stress of prestressing reinforcement
$f_{se} = f_{si} - \Delta f_s$ = stress in reinforcement at service load
f_{si} = reinforcement stress at time of tensioning steel
Δf_s = loss of prestress from initial tensioning value
$\rho_p = A_{ps}/bd$

MATERIALS

Reinforced concrete is a combination of concrete and steel acting as a unit because of bond between the two materials. Concrete has a high compressive strength but a relatively low tensile strength. Beams of plain concrete fail by tension at very low stresses, but if properly reinforced by embedment of steel in their tensile regions, they may be loaded to utilize the much higher compressive strength of the concrete. Reinforced concrete structures are practically monolithic, are more rigid than steel structures of the same strength, and are inherently fire-resistant. Reinforcement in the concrete also controls cracking caused by temperature changes and shrinkage.

Prestressed concrete is a form of reinforced concrete in which initial stresses opposite those caused by the applied loads are induced by tensioning high-strength steel embedded in the concrete. Members may be *pretensioned,* in which the steel is tensioned and the concrete then cast around it, or *posttensioned,* in which the concrete is cast and cured, after which steel placed in ducts through the concrete is tensioned.

Concrete For reinforced concrete work, only high-quality portland cement concrete may be used, and the aggregates must be carefully selected. The proportions are governed by the required strength, durability, economy, and the quality of the aggregates. Concretes for building construction normally have compressive strengths of 3,000 to 5,000 lb/in^2 (20 to 35 MPa), except that concrete for columns may be considerably stronger, with 12,000 to 14,000 lb/in^2 common in some geographic areas. Most concrete for prestressed members will have 5,000 lb/in^2 or higher compressive strength. The higher-strength concretes require thorough quality control if the strengths are to be consistently obtained. Generally, lean, harsh mixes should be avoided because the bond with the steel will be poor, permeability will be high, and durability of the concrete may be poor. A consistency of concrete that will flow sluggishly but not so wet as to produce segregation of the materials when transported must be used for all reinforced concrete work in order to embed the steel and completely fill the molds or forms. The use of vibration is almost mandatory, and enables the use of stiffer, more economical, concretes than would otherwise be possible (see also Sec. 6.9).

Steel Reinforcing steel may be deformed or plain bars, welded-wire fabric, or high-strength wire and strand for prestressed concrete. Bars with deformations on their surfaces are designed to produce mechanical bond and greater adhesion between the concrete and steel, and are used almost universally in the United States. Welded-wire fabric is suitable in many cases for slabs and walls, and may result in cost savings through labor savings. Fabric is made with both smooth and deformed wires, and the deformed fabric may have some advantage in terms of better crack control. Deformed reinforcing bars having minimum yield stresses from 40,000 to 75,000 lb/in^2 (275 to 517 MPa) are currently manufactured. The higher-strength steels have the advantage of allowing higher working stresses, but their ductility may be less and it may be difficult to cold-bend them successfully, especially in the larger sizes. The chemical makeup of most reinforcing steels is such that it is not readily weldable without special techniques, including careful preheating and controlled cooling. Furthermore, the variation of the material from batch to batch is so great that separate procedures must be devised for each batch. Tack welding in assembling bar cages can be particularly troublesome because of the stress raisers introduced. A weldable steel is produced with the specification ASTM A-706, but it is not widely available. Prestressing steel is heat-treated high-carbon steel, and 7-wire strand will have a breaking stress of 250,000 or 270,000 lb/in^2 (1,720 or 1,860 MPa). The usual steel is Grade 270, low-relaxation strand. The strength of solid wire for prestressing is slightly less. ASTM specifications cover the various steels, and all steel used as reinforcement should comply with the appropriate specification.

Reinforcing Steel Sizes The sizes of reinforcing bars have been standardized, and the designation numbers are approximately the bar diameter, in ⅛-in units. Table 12.3.1 gives the nominal diameter, cross-sectional area, and perimeter for each bar size. The areas of the four most common 7-wire prestressing strands are given below:

Diam, in	Area, in^2	
	Grade 250	Grade 270
7⁄16	0.109	0.115
½	0.144	0.153

Table 12.3.1 Dimensions of Deformed Bars

No.	Diam, in	Area, in^2	Perimeter, in
2*	0.250	0.05	0.786
3	0.375	0.11	1.178
4	0.500	0.20	1.571
5	0.625	0.31	1.963
6	0.750	0.44	2.356
7	0.875	0.60	2.749
8	1.000	0.79	3.142
9	1.128	1.00	3.544
10	1.270	1.27	3.990
11	1.410	1.56	4.430
14	1.693	2.25	5.32
18	2.257	4.00	7.09

* No. 2 bars are obtainable in plain rounds only.

Plain and deformed wires are used as reinforcement, usually in the form of welded mats. Plain wires are made in sizes W0.5 through W31, where the number designates the cross-sectional area of the wire in hundredths of square inches. Deformed wires are made in sizes D1 through D31, and the number has the same meaning. Not all sizes are made by all manufacturers, and local suppliers should be consulted about availabilities of sizes. The formerly used wire gage numbers are no longer used for size specifications.

Moduli of Elasticity For concrete the modulus of elasticity E_c may be taken as $w^{1.5}33\sqrt{f'_c}$ in lb/in^2, where w is the weight of the concrete between 90 and 155 lb/ft^3. Normal weight concrete may be assumed to weigh 145 lb/ft^3. For steel the modulus of elasticity may be taken as 29,000,000 lb/in^2 (200 GPa), except for prestressing steel for which the modulus shall be determined by tests or supplied by the manufacturer, but is usually about 28,000,000 lb/in^2.

The **modular ratio** $n = E_s/E_c$ is of importance in designing reinforced concrete. It may be taken as the nearest whole number but not less than six. The value of n for lightweight concrete may be taken as the same for normal weight concrete of the same strength, except in calculations for deflections.

Protection of Reinforcement Reinforcement, for both regular and prestressed concrete, must be protected by the concrete so as to prevent corrosion. The amount of cover needed for various degrees of exposure is as follows:

Member and Exposure	Cover, in
Concrete surface deposited against the ground	3
Concrete surface to come in contact with the ground after casting:	
Reinforcement larger than No. 5	2
Reinforcement smaller than No. 5	1½
Beams and girders not exposed to weather:	
Main steel	1½
Stirrups and ties	1
Joists, slabs, and walls not exposed to weather	¾
Column spirals and ties	1½

The clear cover and clear bar spacings should ordinarily exceed 4⁄3 times the maximum sized aggregate used in the concrete.

The amount of protection recommended is a minimum, and when corrosive environments or other severe exposure occurs, the cover should be increased. The concrete in the cover should be made as impermeable as possible. Fire-resistance requirements may also control the cover requirements.

LOADS

The dead and live loads are combined in determining the cross sections of the members. The dead load includes the weight of the structure, all finishing materials, and usually the installed equipment. The live load includes the contents and ordinarily refers to the movable items.

The **live load** (pounds per square foot) to be used for design depends upon the loadings that will occur in the particular structure as well as on the requirements of the local building code. The Boston Building Code illustrates good practice and is as follows for floor loads:

Heavy manufacturing, sidewalks, heavy storage, truck garages, 250; public garages, intermediate manufacturing, and hangars, 150; stores, heavy merchandise, light storage, 125; armories, assembly halls, gymnasiums, grandstands, public portions of hotels, theaters, and public buildings, corridors and fire escapes from public assembly buildings, light merchandising stores, stairs, first and basement floors of office buildings, theater stages, 100; upper floors of public buildings, office portions of public buildings, stairs, corridors and fire escapes except from public assembly buildings, theater and assembly halls with fixed seats, light manufacturing, locker rooms, stables, 75; church auditoriums, 60; office buildings above first floor including corridors, classrooms with fixed seats, 50; residence buildings and residence portions of hotels, apartment houses, clubs, hospitals, educational and religious institutions, 40. [Note: 100 lb/ft^2 = 4.79 kPa.]

Live loads affecting structural members supporting considerable tributary floor areas are sometimes reduced in recognition of the low probability that the entire area will be loaded to the full design load at the same time. Roofs are commonly designed to support live loads of 20 to 30 lb/ft^2. Wind loads are commonly from 15 to 30 lb/ft^2 (higher in areas subject to hurricanes) of vertical projection. In the case of heavy moving loads, allowance should be made for impact by increasing the live load by 25 to 100 percent.

Dead loads include the weight of both structure and finishing materials, and the weights of some typical wall, floor, and ceiling materials are as follows:

Description	Weight, lb/ft^2
Granolithic finish, per in of thickness	12
⅞-in hardwood, 1⅛-in plank intermediate floor, and tar base	16
3-in wood block in coal-tar pitch	10
Lightweight concrete fill, 2 in thick	14
Plaster on concrete, tile, or concrete block, two coats	5
Plaster on lath	10
6-in concrete block wall	25
Suspended ceiling	12

SEISMIC LOADINGS

Earthquakes induce forces in structures because of inertial forces which resist the ground motions. These forces have damaged or destroyed large numbers of structures throughout the world. The art and science of designing to resist seismic effects are still evolving, and may be thought of in several steps.

1. Determine the ground motion to be expected at the site of interest. This usually requires consulting a map in ASCE-7 or the prevailing local building code, such as the Uniform Building Code (UBC). Several steps may be required, including finding the expected acceleration of the bedrock, followed by an accounting for the soil types between the bedrock and the structure. There is an implicit or explicit assumption of a probability that this ground motion will not be exceeded in some particular time interval, such as 50 or 100 years.

2. Determine the effects of the ground accelerations on the structure which is being analyzed. This will require consideration of the natural period of vibration of the structure and of structural characteristics such as damping and ductility, and more elaborate analyses will be required for large and important structures than for smaller, less crucial cases. Member forces, moment, shear, thrust, torsion, and deformations such as story drift and member end rotations will result from this analysis. The major building codes all have specific requirements and provisions for this analysis, and the UBC is the most widely used in seismically active regions. In most cases, an equivalent static horizontal load will be applied to the structure, with the force in the range of 10 percent or less to as much as 30 percent of the mass of the structure.

3. Design and detail members and connections for the imposed forces. Chapter 21 of the ACI Code has many requirements for the detailing of the reinforcement in the members and joints. When compared with designs for static loadings which include only dead, live, and wind loads, seismic designs will have much more lateral reinforcement in columns, and often much heavier shear reinforcement (especially near the joints), and will usually have extra reinforcement within the joint regions. Much of this added reinforcement is referred to as **confinement reinforcement,** and the intent is to utilize the fact that triaxially confined concrete may be able to sustain much greater strains before failure than unconfined concrete. Obviously, steps 2 and 3 are parts of an iterative process which converges to an acceptable structural design solution.

The current seismic design philosophy includes the assumption that it is not economically possible to design structures to resist extremely large earthquakes without damage. It is expected that structures will resist smaller earthquakes, such as might be expected several times during the life of a structure, with minimal damage, while a very large earthquake which might be expected to occur once during a structure's design life would cause significant damage but would leave the structure still standing. The assignment of an importance factor for buildings is one consequence of this design philosophy. Hospitals, school buildings, and fire stations, for example, would be designed to a higher seismic standard than would a three-story apartment building.

LOAD FACTORS FOR REINFORCED CONCRETE

The current ACI Building Code for Reinforced Concrete is based on a **strength design** concept, in which the strength of a member or cross section is the basis of design. This approach has been adopted because of the difficulty in assigning reasonable and consistent allowable stresses to the concrete and steel. Factors of safety are expressed in terms of *overload* and *understrength* factors. The overload factors, reflecting the uncertainty of the applied loads, are expressed as

No wind or earthquake loads:

$$U = 1.4\,D + 1.7\,L \qquad (12.3.1)$$

With wind acting, use the larger of (12.3.2) or (12.3.3):

$$U = 0.75\,(1.4\,D + 1.7\,L + 1.7\,W) \qquad (12.3.2)$$

$$U = 0.9\,D + 1.3\,W \qquad (12.3.3)$$

No section may be weaker than required by Eq. (12.3.1). In case earthquake loadings are considered. 1.1 E is substituted for W in the above equations. Liquids are treated as dead loads, and other provisions are made for earth-pressure loadings. The understrength factors reflect the ductility of failure in the mode considered, and the consequences of a failure on the rest of the structure. These are expressed as ϕ factors with the following values:

Bending and tension	$\phi = 0.90$
Shear and torsion	$\phi = 0.85$
Spiral columns	$\phi = 0.75$
Tied columns and bearing	$\phi = 0.70$

These factors are used to reduce the computed ideal strengths of members, reflecting possible weaknesses related to materials, dimensions, and workmanship.

In addition to strength requirements, serviceability checks must be made to ensure freedom from excessive cracking, deflections, vibrations, etc., at working loads. Prestressed-concrete members must also satisfy a set of allowable stresses at this load level.

Deflections are usually controlled by specifying minimum member depths. The values of $l/16$ and $l/21$, for beams which are simply supported and continuous at both ends, respectively, are typical, unless an investigation is conducted to show that shallower members will result in acceptable deflections. Crack control is obtained by careful distribution of the steel through the tensile zone. In this regard, several small bars are superior to one large bar.

The approach to design is not a ''limit design'' or ''plastic design'' concept, as the ultimate forces are derived from elastic analyses of structures, using maximums for each critical section. The plastic col-

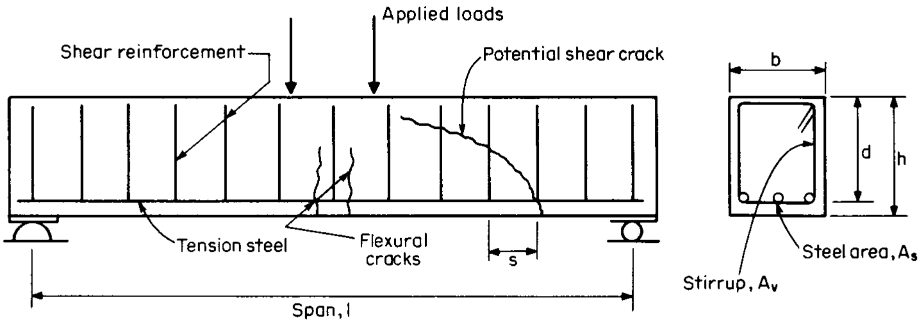


Fig. 12.3.1 Typical arrangement of reinforcement in beams.

lapse loads and mechanisms currently used in the ''plastic design'' of some steel structures in the United States are not considered.

Bridges may be designed using the same general concepts, but different overload and understrength factors are used, and the serviceability requirements include checks on the fatigue strength.

REINFORCED CONCRETE BEAMS

Concrete beams are reinforced to resist both flexural and shear forces. A typical reinforcement scheme for a simply supported beam is shown in Fig. 12.3.1.

As the load on a reinforced concrete beam is increased, vertical tension cracks appear in the maximum moment regions, and gradually grow in length, width, and number. By the time the working load is reached, some of the cracks extend to the neutral axis and the contribution of the tensile strength of the concrete to the flexural capacity of the beam has become very small. As the load is increased further, the reinforcement eventually yields. This load is nearly the maximum the member can sustain. Further attempts at loading produce large increases in deflection and crack widths with very small increases in load and a gradual reduction in the remaining concrete compression area. When a limiting concrete strain of about 0.003 is reached at the compression face of the beam, the concrete starts crushing and the capacity of the beam starts dropping. For static design purposes, the achievement of the 0.003 strain is usually regarded as the end of the useful life of the beam.

Within limits to be checked later, at the time of flexural failure of a beam the stress in the steel is equal to the yield stress, which greatly simplifies the analysis of the member. The strain and stress distributions in a beam are shown in Fig. 12.3.2, at failure.

It is assumed that plane sections remain plane, that adequate bond exists, that the stress-strain relationships for concrete and steel are known, and that tension in the concrete has a negligible influence.

The **flexural strength** of a cross section may be written as, including the understrength factor and rounding the terms in the parentheses to one significant figure:

$$\begin{aligned} M_u &= \phi A_s f_y d(1 - 0.4\,k_u) \\ &= \phi A_s f_y d(1 - 0.6\rho f_y / f'_c) \end{aligned} \tag{12.3.4}$$

where $\rho = A_s/bd$ = reinforcement ratio. The reinforcement ratio will ordinarily be between 0.005 and 0.02.

A satisfactory approximate stress distribution is shown in Fig. 12.3.2*d*. Since $T = C = 0.85 f'_c ba$, then $a = A_s f_y / 0.85\, f'_c b$ and the flexural capacity is

$$M_u = \phi A_s f_y (d - a/2) \tag{12.3.5}$$

It must be demonstrated for each case that the stress in the reinforcement has reached f_y, and the simplest approach is to show that $\varepsilon_{su} \geq \varepsilon_y$. From Fig. 12.3.2*b*, $\varepsilon_{su} = 0.003\,(1 - k_u)/k_u$. From equilibrium (see Fig. 12.3.2*c*), $k_u = A_s f_y / 0.85\, \beta_1 f'_c bd = \rho\, f_y / 0.7 f'_c$. The limiting case for the validity of Eqs. (12.3.4) and (12.3.5) is a balanced condition in which the yield strain in the steel and the 0.003 compressive strain in the concrete are reached simultaneously, and this can be found using Fig. 12.3.2 and assuming $\varepsilon_{su} = \varepsilon_y$. It can then be shown that

$$\rho_{bal} = \frac{0.85\beta_1 f'_c}{f_y} \frac{0.003}{0.003 + \varepsilon_y} \tag{12.3.6}$$

Values of ρ_{bal} are plotted vs. f'_c in Fig. 12.3.3 for three values of f_y. The

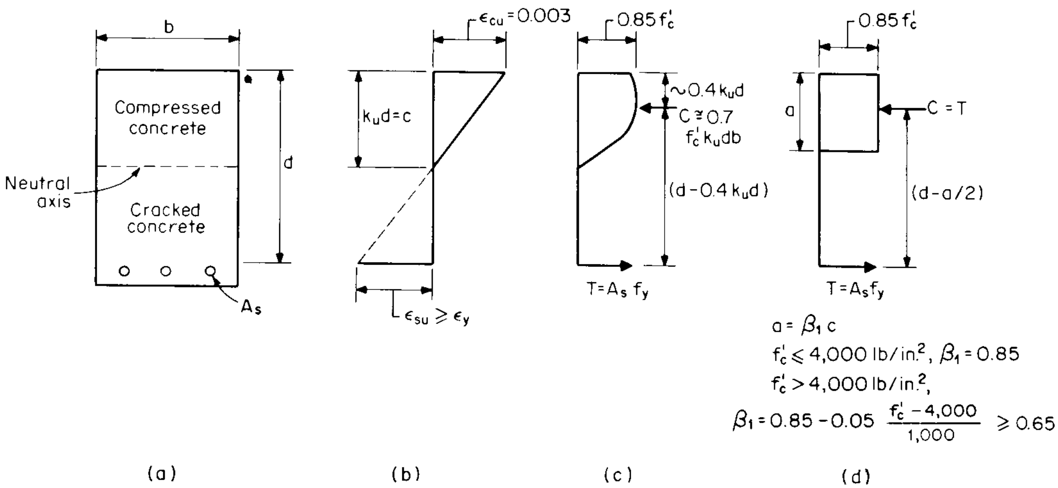


Fig. 12.3.2 Stress and strain distribution in reinforced concrete beam at failure. (*a*) Section; (*b*) strains; (*c*) stresses; (*d*) approximate stresses.

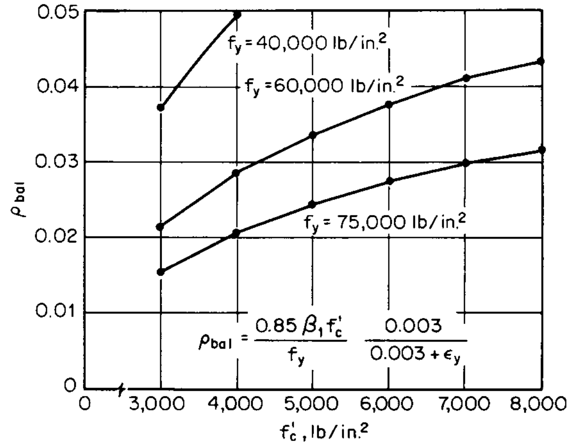


Fig. 12.3.3 Balanced steel ratio as a function of f_y and f_c'.

steel ratio should not exceed 0.75 ρ_{bal}, in order to ensure that at least some yielding, with resultant large deflections, occurs before a member fails.

Beams may contain both compression and tension reinforcement, especially when the beam must be kept as small as possible or when the long-term deflections must be minimized. The forces at ultimate are shown in Fig. 12.3.4, and the moment may be calculated as

$$M_u = \phi[A_s' f_y (d - d') + (A_s - A_s') f_y (d - a/2)] \qquad (12.3.7)$$

where $a = (A_s - A_s') f_y / 0.85 f_c' b$. The net steel ratio $\rho - \rho' = (A_s - A_s')/bd$ should not exceed 0.75 of the balanced value given by Eq. (12.3.6).

Many reinforced concrete beams are flanged sections, or T beams, by virtue of having a slab cast monolithically with the beam, and such a cross section is shown in Fig. 12.3.5. It will be found that the neutral axis lies within the flange in most instances, and if $k_u d \leq t$, then the beam is treated as a rectangular beam of width b. This is checked using $k_u d = A_s f_y / 0.85\ \beta_1\ f_c' b$, developed from equilibrium (Fig. 12.3.2). If $k_u d > t$, the flexural capacity is computed by

$$M_u = \phi[(A_s - A_{sw}) f_y (d - a/2) + A_{sw} f_y (d - t/2)] \quad (12.3.8)$$

where $A_{sw} = (b - b_w) t 0.85 f_c' / f_y$; and $a = (A_s - A_{sw}) f_y / 0.85 f_c' b_w$.

Continuous T beams are treated as rectangular beams of width b_w in the negative moment regions where the lower surface of the beam is in compression. Most continuous beams will have compression steel in the negative moment regions, as some bottom steel is always continued into the support regions.

Both T beams and beams with compression steel may be thought of in terms of dividing the beam into two components—one a rectangular beam containing part of the tension steel and the other a couple with the rest of the tension steel at the bottom and either the compression steel or the T-beam flanges at the top. Both components of the beam must satisfy horizontal equilibrium.

The reinforcement ratio should not be less than $\rho_{min} = 3\sqrt{f_c'}/f_y \geq 200/f_y$. This is to ensure that the ultimate moment is somewhat

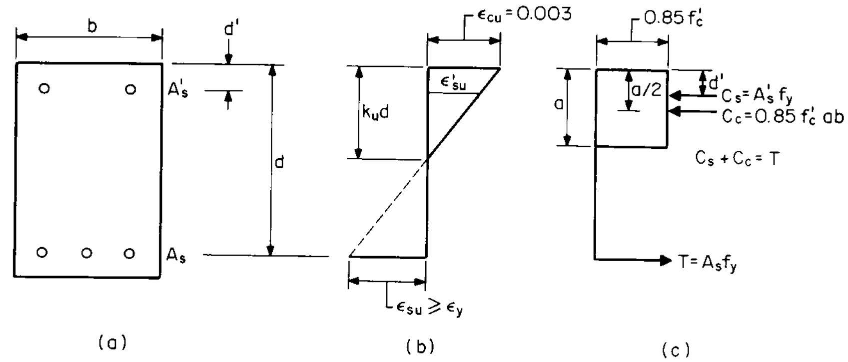


Fig. 12.3.4 Stress and strain distributions in a doubly reinforced concrete beam. (*a*) Section; (*b*) strains; (*c*) approximate stresses.

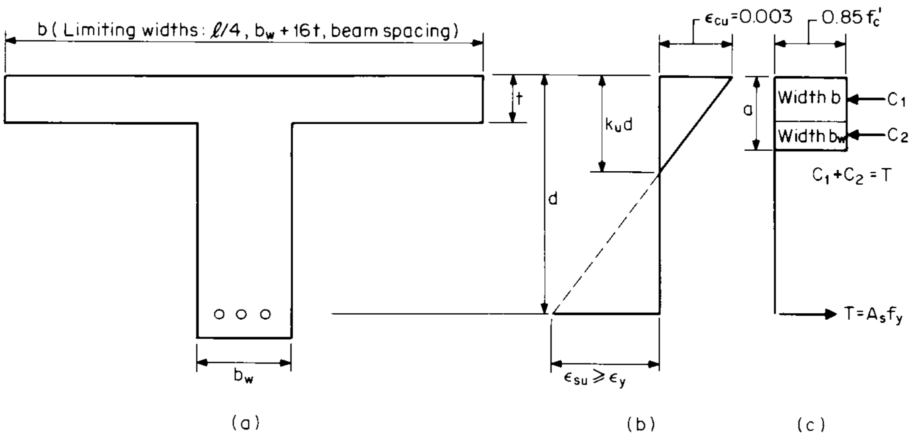


Fig. 12.3.5 Strain and stress distribution in a T beam. (*a*) Section; (*b*) strains; (*c*) approximate stresses.

greater than the initial cracking moment. For T beams, $\rho = A_s/b_w d$ for purposes of the minimum steel requirement.

In the selection of a cross section, it is convenient to transform Eq. (12.3.4) by substituting ρbd for A_s and rearranging to obtain

$$M_u/\phi bd^2 = \rho f_y(1 - 0.6\rho f_y/f'_c) \qquad (12.3.9)$$

With given materials f_y and f'_c selection of a ρ value enables calculation of the required bd^2, from which a cross section may be selected. For convenience, values of $M_u/\phi bd^2$, are plotted against ρf_y in Fig. 12.3.6, for several values of f'_c.

The strengths of prestressed-concrete beams are treated much the same as reinforced concrete beams, and the differences will be noted later.

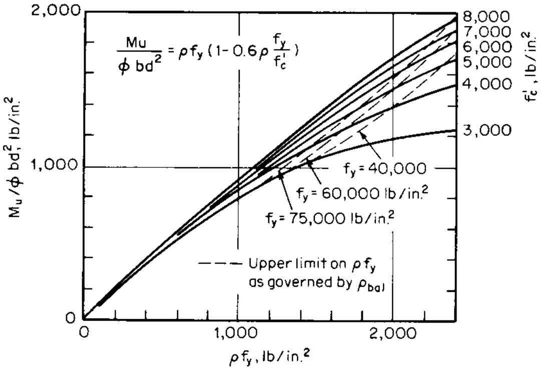


Fig. 12.3.6 Design factors for single reinforced concrete beams.

In addition to the tension stresses caused by bending forces, **shear forces** cause inclined tension stresses which may lead to inclined cracking such as is sketched in Fig. 12.3.1. Unless web reinforcement is present, the formation of such a crack usually leads directly to the complete collapse of the member at the load which caused the crack, or only slightly higher.

As a result of this undesirable behavior, shear reinforcement is required in all major beams, and the normal design procedure is to proportion the beam for the flexural requirements and then add shear steel, usually in the form of stirrups, to make the shear strength adequate.

The concrete can be assumed to resist a shear stress of $v_c = \phi 2\sqrt{f'_c}$, or a shear force of

$$V_c = \phi 2\sqrt{f'_c}\, bd \qquad (12.3.10)$$

(or $V_c = \phi 2\sqrt{f'_c}\, b_w d$ for T beams). The shear reinforcement must resist the force in excess of V_c, so that

$$V_u - V_c = V_s \qquad (12.3.11)$$

The area of shear reinforcement is then selected to satisfy

$$V_s = \phi A_v f_y d/s \qquad (12.3.12)$$

Shear reinforcement to satisfy this requirement is provided at every section of the beam, except that the region between the support and the section at the distance d from the support is supplied with the steel required at d from the support.

The stirrup spacing should not exceed $d/2$, and is reduced to a maximum of $d/4$ if $V_u > \phi 6\sqrt{f'_c}\, bd$. V_u must not exceed $\phi 10\sqrt{f'_c}\, bd$. The minimum area of shear reinforcement allowed is $A_v = 50 b_w s/f_y$. Closed stirrups of the form shown in Fig. 12.3.1 are recommended, and are essential in areas subjected to earthquake loadings.

Shear in prestressed-concrete members is handled in a similar manner, and the designer is referred to the ACI Code for details. However, it will be found that most prestressed members designed for buildings and which do not support major concentrated loads will have adequate shear strength if the minimum steel given by the following expression is supplied:

$$A_v = \frac{A_{ps}}{80}\frac{f_{pu}}{f_y}\frac{s}{d}\sqrt{\frac{d}{b_w}} \quad \text{in}^2 \qquad (12.3.13)$$

The maximum stirrup spacing is 0.75 of the member depth, or 24 in, and for constant-depth members, d is measured at the section of maximum moment.

At least the minimum area of shear reinforcement must be supplied over the full length of most reinforced and prestressed members unless it can be shown, by a test acceptable to the building official, that the members can sustain the required ultimate loads without the steel.

Recent ACI Codes have undergone major revisions to reflect the effects of bar spacing and bar cover on development length, l_d, and on splice lengths. In some common cases the current development lengths will be much greater than in ACI Codes from 1983 and earlier. The 1995 Code development lengths for straight bars are given in Code Sec. 12.2.2, and a more complex, less conservative, set of requirements are in Sec. 12.2.3. When the available anchorage lengths are less than l_d, hooks are often used. The conservative Sec. 12.2.2 requirements may be summarized as follows:

$$\frac{l_d}{d_b} = K\frac{f_y \alpha\beta\lambda}{\sqrt{f'_c}}$$

in which K is as follows:

	No. 6 and smaller bars and deformed wire	No. 7 and larger bars
1. Clear spacing of bars being developed or spliced $\geq d_b$ and clear cover $\geq d_b$ with minimum ties or stirrups or	1⁄25	1⁄20
2. Clear spacing $\geq 2d_b$ and clear cover $\geq d_b$	1⁄25	1⁄20
3. Other cases	3⁄50	3⁄40

The other terms are defined as:

d_b = diameter of bar or wire, in
α = bar location factor
= 1.3 for top bars (horizontal with more than 12 in of concrete cast below)
= 1.0 for other bars
β = coating factor
= 1.5 for epoxy-coated bars or wires with cover $\leq 3d_b$, or clear spacing $\leq 6d_b$
= 1.2 for all other epoxy-coated bars or wires
= 1.0 for uncoated reinforcement
λ = lightweight concrete factor
= 1.3 when light-weight aggregate concrete is used
= 1.0 when normal weight aggregate is used

The product $\alpha\beta$ need not be taken greater than 1.7.

Bars No. 11 and smaller are commonly spliced by simply lapping them for a distance. These splices should be avoided in regions of high computed stress, and should be spread out so that not many bars are spliced near the same section. If the computed tensile stress is less than 0.5 f_y at the design ultimate load and not more than half the bars are spliced at one section, the lap length is 1.0 l_d. For other cases the minimum lap length is 1.3 l_d. For lower stress levels, a lap of l_d is sufficient.

Development lengths in compression, important in columns and compression reinforcement, are somewhat shorter.

Requirements for reinforcement of beams subjected to torsional moments are contained in the ACI Code. The requirements are too complex for discussion here, but the basic reinforcement scheme consists of closed stirrups with longitudinal bars in each corner of the stirrup. The most efficient method of dealing with torsion in many instances will be to rearrange the structure so as to reduce or eliminate the torsional moments.

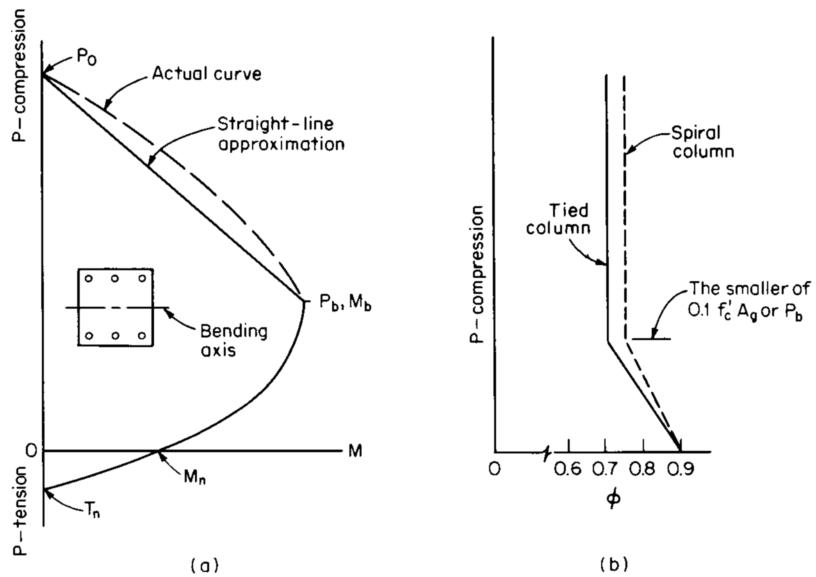


Fig. 12.3.7 Column capacity diagrams. (*a*) Typical moment-thrust interaction diagram; (*b*) variation of ϕ with P.

REINFORCED CONCRETE COLUMNS

Reinforced concrete compression members are proportioned taking into account the applied thrust, the bending moments, and the relationship between length and thickness for the member. The strength of a cross section or a short column can conveniently be shown with the aid of a *moment-thrust interaction diagram* such as is shown in Fig. 12.3.7*a*, which is drawn without considering the ϕ factor. The variation in ϕ with thrust is shown in Fig. 12.3.7*b* for the same column.

The load P_0 is the strength of a short column under a concentric load, and is expressed as

$$P_0 = 0.85\, A_c f'_c + A_s f_y \tag{12.3.14}$$

The contribution of the concrete is slightly less than the cylinder strength because of differences in workmanship, curing, and position of casting. M_u is simply the strength in flexure, as was discussed for beams. The $M_b - P_b$ point is the ''balance point,'' at which simultaneous crushing of the concrete and yielding of the reinforcement occur. Failure is initiated by crushing of the concrete at loads higher than P_b, and by yielding of the reinforcement in tension at lower loads. Only the reinforcement contributes to the tensile capacity of $T_u = A_s f_y$

The balance-point moment and thrust are found using the strain and stress distributions shown in Fig. 12.3.8, for a symmetrical section with steel in two faces. From equilibrium,

$$P_b = C_s + C_c - T \tag{12.3.15}$$

Summing moments about the centroidal axis gives

$$M_b = A_s f_y (d - d')/2 + 0.85\beta_1 f'_c k_u db(h/2 - \beta_1 k_u d/2) \tag{12.3.16}$$

This assumes that the compression steel has yielded, and this will be true except for very small members or members with exceptionally large cover over the steel.

The portion of the interaction diagram below P_b may be constructed in a point-by-point manner. For any value of $\varepsilon_s > \varepsilon_y$, or for any value of $k_u d < k_u d_{bal}$, the strain and stress distributions are defined. Once the forces are defined, the moments and thrusts are computed using the same formulas as for the balance point. A modification will have to be made when the compression-steel stress is less than f_y.

Care must be used in the selection of the axis about which moments are to be summed, especially if the section is not symmetrical or symmetrically reinforced, since the force system is not a pure couple. The most important thing is to remain consistent, and to be certain that the internal and external moments are summed about the same axis.

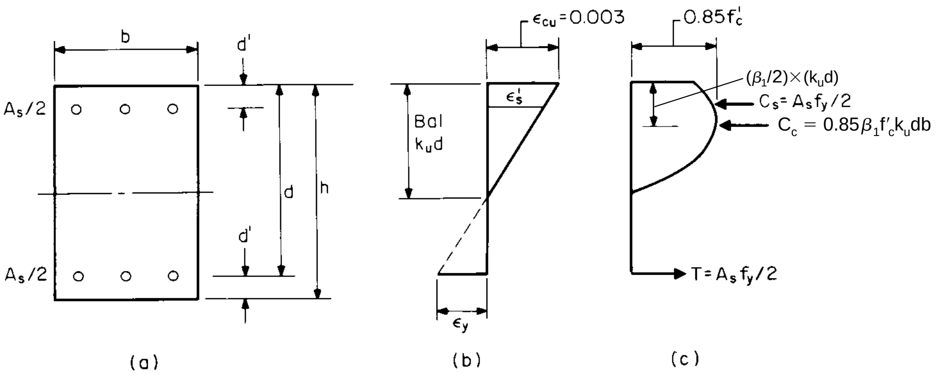


Fig. 12.3.8 Stress and strain distribution in a column at balanced moment and thrust. (*a*) Section; (*b*) strains; (*c*) stresses.

In practice, either the moment-thrust curve can be reduced by the appropriate ϕ factor, or the computed ultimate M and P increased by dividing by ϕ. If the required $M - P$ point lies on or slightly inside the interaction curve, the design is acceptable. The maximum permitted factored thrust is $0.80\ \phi P_0$, which limits the applied thrust capacity in cases where the computed moments are relatively small. This limitation is intended to provide resistance to accidental eccentricities. Column reinforcement consists of longitudinal bars and lateral ties or spiral bars. The ratio of longitudinal steel ρ_g should be between 0.01 and 0.08. Ties are usually No. 3 or No. 4 bars which are bent to enclose the longitudinal bars, and are spaced at not more than 16 longitudinal bar diam, 48 tie diam, or the least thickness of the column. Ties are arranged to bind each corner bar and alternate intermediate bars. Several typical arrangements are shown in Fig. 12.3.9. Ties hold the longitudinal bars in place during construction, and may provide some shear resistance and improve the behavior of the column at loads near failure.

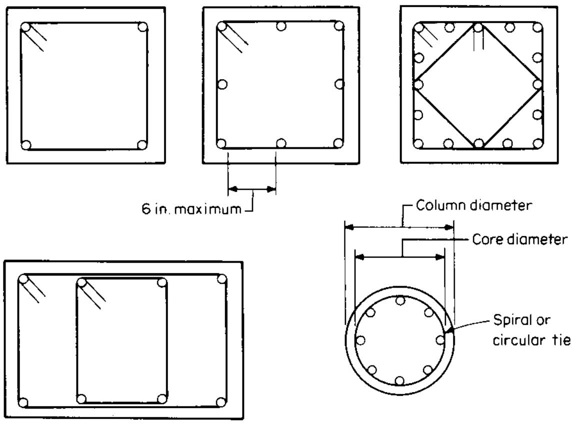


Fig. 12.3.9 Typical arrangement of steel in columns.

Spiral bars serve to confine the concrete core of the section as well as holding the longitudinal bars. The minimum amount of spiral steel, if advantage is to be taken of the higher ϕ factor for spiral columns, is

$$\rho_s = 0.45(A_g/A_{core} - 1)f'_c f_y \tag{12.3.17}$$

where f_y is the yield stress of the spiral, but not more than 60,000 lb/in^2. The clear spacing of the turns of the spiral must be between 1 and 3 in.

The strengths of compression members may be reduced below the cross-sectional strengths by length effects. Most columns in unbraced frames (frames in which the columns resist all horizontal forces) will have some strength reduction because of length effects, and most columns in braced frames will not, but this depends on the precise details of the length, width, restraint by other frame members, reinforcement, and amount of creep expected. A comprehensive method of taking column length into account is contained in the ACI Code.

Columns are also made by encasing structural steel sections in concrete, in which case the covering concrete must contain at least some steel in order to control cracks and maintain the integrity of the concrete in case of fire. Heavy steel pipes filled with concrete may also be used as columns. These are also used as piling, in which case the pipe is filled with concrete, usually after being driven to the final location.

REINFORCED CONCRETE FLOOR SYSTEMS

Several types of reinforced concrete floors are used, with the choice depending on a number of factors such as span, live load, deflection limits, cost, story-height limitations, local custom, the nature of the rest of the structural frame or system, and the probability of future alterations.

The floor systems may be divided, somewhat arbitrarily, into one-way and two-way systems. One-way systems include solid and hollow slabs and joists spanning between parallel supporting beams or walls. Floors in which panels are subdivided into a grid by subbeams spanning between main girders have usually been designed as one-way slabs when the grid length is several times the width.

Two-way systems include slabs supported on all four sides on beams or walls, traditionally called *two-way slabs.* Slabs supported only on columns located at the corners of the panels also carry loads by developing stresses in the two major directions, and are usually termed *flat plates* if the slab is supported directly on the columns and *flat slabs* if there are capitals on the columns to increase the effective support size. A waffle slab is usually designed as a flat plate or flat slab, with pockets of concrete omitted from the lower surface, and the slab appears as a series of crossing joists.

One-way slabs and joists are designed as beams. A 12-in or other convenient width of slab is selected, analyzed as an isolated beam, and a depth and steel are picked. The main steel is perpendicular to the supports. Additional steel, parallel to the supports, is placed to control cracking and help distribute minor concentrated loads. This steel is usually one-fourth to one-third of the main steel, but not less than a gross steel ratio of 0.0018 for Grade 60 and 0.002 for lower-grade steels. These minimums govern in both directions, and in two-way systems as well.

One-way joists, such as that shown in Fig. 12.3.10, are usually cast with reusable sheet metal or fiberglass forms owned or rented by the contractor. Standard form sizes range from about 20 to 36 in wide and 8 to 20 in deep. The web thicknesses are made to suit the shear and fireproofing requirements, and special tapered end sections may be available to increase the web widths in the regions of high shear near the supports. Joists are exempted from the requirement that web steel be supplied regardless of the concrete shear stress. The allowable shear stress for the concrete is 1.1 times that for beams. The top slabs usually range from 2.5 to 4.5 in thick, and are reinforced to span from rib to rib. The joists are essentially designed as isolated T beams, and may be supported on girders or walls. Joist systems are suitable for reasonably long spans and heavy loads, and have low dead weights for the effective depths attainable.

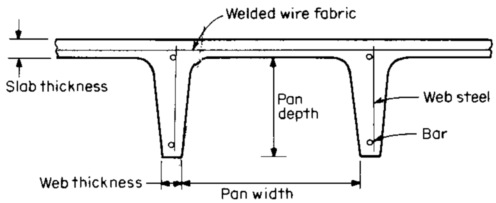


Fig. 12.3.10 Cross section of concrete floor joist.

Slabs spanning in two directions are all designed taking into account the shape of the panel and the relative stiffnesses of the supporting beams, if any. The choice of types is a matter of loadings and economics. For residential and light office loadings, the flat plate is frequently the choice, as the very simple formwork may lead to substantial economy and the story height is minimized. For heavier loads or longer spans, punching shear around the columns becomes a limiting factor, and the flat slab, with its column capitals, may be the most suitable.

In case of extremely heavy floor loads or very stringent limits on deflections, slabs with beams on all four sides of each panel will be most satisfactory. The formwork is more complex than for the other slabs, but there will be some compensating savings in the amounts of steel because of the greater depths of the beams. In addition, the two-way slab may be much more efficient if the building is to resist major lateral loads by frame action alone, because of the difficulties in transferring large moments between columns and flat plates or slabs. The design procedure is the same for slabs with and without beams. The basic steps, for each direction of span in each panel, are:

1. Compute static moment M_0.
2. Distribute M_0 to positive and negative moment sections.
3. Distribute section moments to column and middle strips and beams.

Most buildings slabs are designed for uniformly distributed loads, and the *static moment*, defined as the absolute sum of the midspan positive plus average negative moments, is

$$M_0 = wl_2(l_n)^2/8 \tag{12.3.18}$$

The dimensions are illustrated in Fig. 12.3.11.

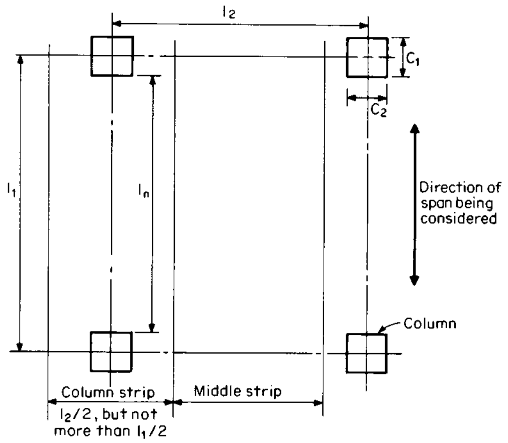


Fig. 12.3.11 Arrangement of typical slab panel.

Relatively simple rules for the distribution of the static moment to various parts of the panel exist as long as the structure meets several simple limits:

1. Minimum of three spans in each direction.
2. Panel length no more than twice panel width.
3. Successive spans differ by not more than one-third the longer span.
4. Columns on a rectangular grid, or offset no more than 10 percent of the span.
5. Live load not more than two times the dead load.
6. If beams are used, they are used on all four sides of each panel and are approximately the same size, except that spandrel beams only are acceptable.

The following is for slabs meeting these restrictions. Information on other cases is contained in the ACI Code.

The positive-negative moment distribution for interior spans is

$$+M = 0.35\,M_0 \tag{12.3.19}$$
$$-M = 0.65\,M_0 \tag{12.3.20}$$

For end spans, the stiffness of the exterior support must be taken into account. Table 12.3.2 gives the fractions of M_0 to be assigned to the three critical sections for five common cases.

Once the section moments have been determined, they are distributed to the column and middle strips and beams, taking into account the panel shape and the beam stiffness. The beam relative stiffness coefficient α_1 is calculated using an effective beam cross section as shown in Fig. 12.3.12, and the full width of the slab panel l_2. The locations of the column and middle strips are shown in Fig. 12.3.11.

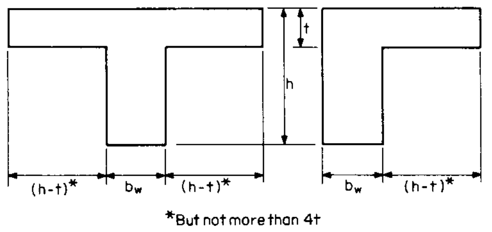


Fig. 12.3.12 Beam sections for calculation of I_b and α_1.

The interior negative and positive moments are distributed to the column strips in the proportions shown in Fig. 12.3.13, with the remainder of the moment going to the middle strip. Linear interpolations are made for intermediate beam stiffnesses, but in most instances where there are beams, they will be found to have

$$\alpha_1 l_2/l_1 \geq 1.0$$

At the exterior supports, the distribution of the negative moments is a complex function of the flexural and torsional stiffnesses of the beams

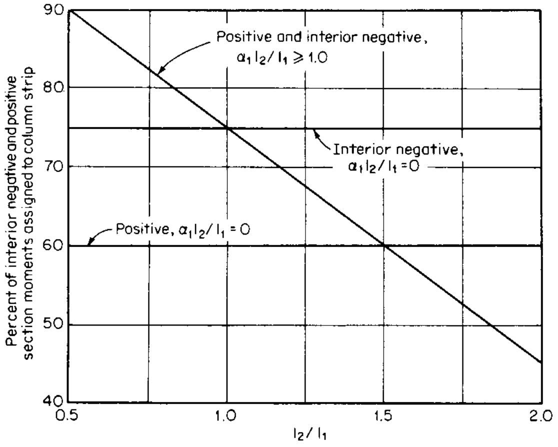


Fig. 12.3.13 Percentage of interior negative and positive moments assigned to a column strip.

and of the panel shape, but satisfactory designs can usually be achieved by assigning all the negative moment to the column strip and detailing the edge beam or edge strip of the slab for torsion, by using closed stirrups at relatively small spacings, and by placing bars parallel to the edge of the structure in each corner of the stirrups. At fully restrained edges, use the distribution for an interior negative moment section.

Table 12.3.2 Moments in End Spans

	Exterior edge unrestrained (1)	Slab with beams between all supports (2)	Slab without beams between interior supports: Without edge beam (3)	Slab without beams between interior supports: With edge beam (4)	Exterior edge fully restrained (5)
Interior negative factored moment	0.75	0.70	0.70	0.70	0.65
Positive factored moment	0.63	0.57	0.52	0.50	0.35
Exterior negative factored moment	0	0.16	0.26	0.30	0.65

Beam moments are found by dividing the column strip moment between beam and slab. If $\alpha_2 l_2/l_1 \geq 1.0$, the beam moment is 85 percent of the column strip moment. This moment is reduced linearly to zero as $\alpha_1 l_2/l_1$ approaches zero.

This design method assumes that all panels are loaded with the same uniformly distributed load at all times. This is obviously a gross simplification, and the requirement that the live load be no greater than twice the dead load limits the potential overstress caused by partial loadings.

In addition, there is a requirement for column design moments, and unless a more complete analysis is made, the following moment, divided between the columns above and below the slab in proportion to their stiffnesses, must be provided for:

$$M = 0.07[(w_d + 0.5w_l)l_2 l_n^2 - w_d' l_2'(l_n')^2] \qquad (12.3.21)$$

The loads w_d and w_l are the distributed dead and live loads including the overload factors. The terms w_d', l_2', and l_n' are for the shorter of the two spans meeting at the column considered.

The shear strength of slab structures must always be checked, and shear stresses often govern the thickness of beamless slabs, especially flat plates.

If there are beams with $\alpha_1 l_2/l_1 \geq 1.0$, all shear is assigned to the beams, and stirrups are provided to make the shear capacity adequate, as was described in earlier coverage on beams. The beam shear is linearly reduced to zero as $\alpha_1 l_2/l_1$ is reduced to zero.

For the case of no beams, punching shear around the columns becomes a controlling factor. In this case the average shear stress, calculated as

$$v_u = V_u/b_0 d \qquad (12.3.22)$$

must not exceed the smaller of $\phi 4\sqrt{f_c'}$, $\phi(2 + 4/\beta_c)\sqrt{f_c'}$, or $\phi(\alpha_s\, d/b_0 + 2)\sqrt{f_c'}$, where β_c = ratio of long side of critical shear perimeter to short side and $\alpha_s = 40$ for interior columns, 30 for edge columns, and 20 for corner columns.

The critical shear perimeter b_0 is defined by a section located $d/2$ away from and extending all around the column, as shown in Fig. 12.3.14. It is very important that holes in the slab in the vicinity of the column be taken into account in reducing the value of b_0, and that no unauthorized holes, such as for piping, be made either during or after construction.

It is possible to increase the shear resistance by the use of properly designed shear reinforcement, but this is not often done and is not recommended as a standard practice.

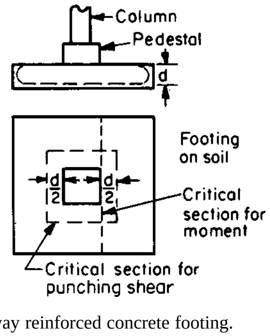


Fig. 12.3.14 Two-way reinforced concrete footing.

Transfer of moments between columns and slabs sets up shear and torsional stresses which must also be considered in the analysis of the shear strength.

FOOTINGS

Footings (Fig. 12.3.14) may be classified as wall footings, isolated column footings, and combined column footings. The bending moments, shears, and bond stresses in such footings should be determined by the principles of statics on the basis of assumed or known soil-pressure distribution over the area of the footing. The bending moment on any projecting portion of a footing may be computed as the moment of the forces acting on the area to one side of a vertical plane through the critical section.

The critical section for bending in a concrete footing supporting a concrete column, pedestal, or wall should be taken at the face of the column, pedestal, or wall. For footings under metallic column bases or under masonry walls where bond with the footings is reduced to the friction value, the critical section is assumed midway between the middle and edge of the base or wall.

Shear stresses must be considered on two sections. The footing may act as a wide beam, for which the critical section is a vertical plane located d away from the critical section for moment, and the stresses must satisfy those for a beam. Punching shear will often govern, and the critical section lies at a distance $d/2$ from the face of the column or other critical section for moment, as shown in Fig. 12.3.14, and as was the case for flat plates and slabs. Footings supported on a small number of high-capacity piles present special shear problems since the conventional critical sections for shear may not be meaningful.

The critical section for bond should be taken at the same plane as for bending. Other vertical planes where abrupt changes of section occur should also be investigated for bond and shear stresses.

In sloped or stepped footings, sections other than the critical ones may require consideration. A square footing, reinforced in two directions, should have the reinforcement uniformly distributed across the entire width. Rectangular footings, reinforced in two directions, should have the reinforcement in the long direction uniformly distributed; in the short direction a portion, Eq. (12.3.23), should be uniformly distributed across a strip equal in width to the short side and centered on the structural element supported and the remainder distributed uniformly in the outer portions. The amount included in the center strip may be computed as follows:

Reinforcement in center strip

$$= \frac{2\ (\text{total reinforcement in short direction})}{\beta + 1} \qquad (12.3.23)$$

where β is the ratio of the long side to the short side.

Combined Footings Footings supporting two or more columns may be designed with sufficient accuracy by assuming uniform soil pressure and applying the laws of statics. The footing shape must be such that the center of gravity coincides with the center of gravity of the superimposed loads; otherwise unequal settlement may occur. The longitudinal and diagonal tension reinforcement should be designed by the ordinary rules of beam design. Lateral reinforcement should be designed as for isolated footings and should preferably be concentrated in bands under and near the columns proportionate in area to the column loads. The lateral reinforcement at each column should be uniformly distributed within a width centered on the column and should not be greater than the width of the column plus twice the effective depth of the footing.

Spread or raft foundation, consisting of a slab extending over the entire area under the columns or of a slab supported by beams, may be considered as loaded by a uniform upward reaction of the ground. The principle of design is exactly the same as that applied to a floor system, except that the load acts upward instead of downward.

Concrete piles of various types are widely used for foundations as they have larger carrying capacity and greater durability under many conditions of exposure than wooden piles. Precast piles are designed as columns with allowance for driving and handling stresses. Cast-in-place piles are constructed either by driving a steel shell and filling it with concrete or by filling the hole formed by a shell as it is withdrawn. Another method forms a bulb at the bottom by means of a ram which forces the concrete into the ground. The design load or capacity of cast-in-place concrete piles is largely empirical, being based on load-test data. The concrete for precast piles is usually over 5,000 lb/in^2 strength and for cast-in-place piles, over 3,000 lb/in^2 strength.

Dowels and Bearing Plates The stress in the longitudinal reinforcement of concrete columns should be transferred to the footing by means of dowels, equal in number and area to the column rods and of sufficient length to transfer the stress as in a lap splice in the column.

Bearing stresses in concrete, under design ultimate loads, should not exceed the following values:

$$\text{Entire surface loaded: } f_b = 0.85\ \phi f'_c \tag{12.3.24}$$

$$\text{Part of surface loaded: } f_b = 0.85\ \phi f'_c \sqrt{A_2/A_1} \tag{12.3.25}$$

where A_1 = loaded area; and A_2 = surface area of same shape and concentric with A_1. $\sqrt{A_2/A_1}$ should not exceed 2.0.

WALLS AND PARTITIONS

Reinforced concrete is well suited to the construction of walls, especially where they have to withstand heavy pressures, such as the retaining walls of a cellar or basement, walls for coal pockets, silos, reservoirs, or grain elevators. Such walls must be designed for flexural shear and bond stresses as well as stability against overturning, sliding, and soil pressure. Drainage should be provided for by weep holes or drains. Partitions may be built of solid concrete 4 to 6 in thick, reinforced to control temperature and shrinkage cracks. Reinforced concrete walls need to be anchored by reinforcement to adjacent structural members. All walls must be reinforced for temperature with steel placed horizontally and vertically.

The horizontal reinforcement shall not be less than 0.25 percent and the vertical reinforcement not less than 0.15 percent of the area of the reinforced section of the wall when bars are used and three-fourths of these amounts when welded fabric is used. Adequate reinforcement must be provided around all openings for windows and doors.

Retaining walls of reinforced concrete are used to resist the pressures of earth, water, and other retained materials and are usually of T or L shape. The base must be so proportioned that there is sufficient resistance to sliding and overturning and that the safe bearing strength of the soil is not exceeded. The dimensions of the concrete section and the position and amount of steel reinforcement are determined by the moments and shears at critical vertical and horizontal sections at the junction of the wall and the base. Particular attention should be given to drainage to prevent excessive water pressure behind walls retaining earth or other materials. Walls retaining water, such as tanks, should have steel tensile stresses limited to 12,000 lb/in^2 unless special consideration is given to controlling cracks and should have ample reinforcement to provide for effects caused by shrinkage of the concrete and temperature change.

Bearing Walls The allowable compressive force for reinforced concrete bearing walls subject to concentric loads can be computed as follows:

$$P_u = 0.55\ \phi f'_c A_g[1 - (l_c/40h)^2] \tag{12.3.26}$$

For the case of concentrated loads, the effective width for computational purposes can be considered as the width of the bearing plus four times the wall thickness but not greater than the distance between loads. The wall thickness should be at least 1⁄25 of the unsupported height or width, whichever is smaller. For the upper 15 feet, bearing walls must be at least 6 in thick and increase at least 1 in in thickness for each successive 25 feet downward, except that walls of a two-story dwelling need to be only 6 in thick over the entire height, provided that the strength is adequate.

PRESTRESSED CONCRETE

Prestressed concrete members have initial internal stresses, set up by highly stressed steel tendons embedded in the concrete, which are generally opposite those caused by applied loads. Prestressed members are constructed in one of two ways: (1) *Pretensioned* members are factory precast products, made by tensioning steel tendons between abutments and then casting concrete directly around the steel. After the concrete has reached sufficient strength, the steel is cut and the force transferred to the concrete by bond. (2) *Posttensioned* members, either factory or site cast, contain steel in ducts cast in the concrete. After the concrete has cured, the steel is tensioned and mechanically anchored against the concrete. The ducts are preferably pumped full of grout after tensioning to provide bond and corrosion protection, or the tendons may be coated with corrosion inhibitors.

Very high strength steel is used for prestressing in order to overcome the losses of steel stress due to creep and shrinkage of concrete, and as a result of the strength, relatively small amounts of steel are required. The concrete for pretensioned members will usually be at least 5,000 lb/in^2 compressive strength, and at least 4,000 lb/in^2 for posttensioned members.

Because of the initial stress conditions, prestressed concrete members are generally crack-free at working loads and consequently are quite suitable for water-containing structures. Circular tanks and pipes are posttensioned by wrapping them with highly stressed wires, using specialized equipment.

Losses of prestress occur with time owing to creep and shrinkage of concrete and relaxation of steel stress. Pretensioned members also have an initial elastic shortening loss accompanying transfer of force to the concrete, and posttensioned members have losses due to friction between ducts and tendons and anchor set. These losses must be taken into account in the design of members.

Prestressed concrete members are checked for both strength and stresses at working loads. Because of the built-in stresses, the condition of dead load only may also govern. The flexural strength is computed using the same equations as for reinforced concrete which were developed earlier. The steel does not have a well-defined yield stress, and the steel stress at failure can be predicted from the following expressions:

Bonded tendons, low relaxation steel and no compression steel:

$$f_{ps} = f_{pu}\left(1 - \frac{0.28}{\beta_1}\,\rho_p \frac{f_{pu}}{f'_c}\right) \tag{12.3.27}$$

Unbonded, $l/h \leq 35$:

$$f_{ps} = f_{se} + 10{,}000 + f'_c/100\ \rho_p \tag{12.3.28}$$

but not more than f_{se} or $f_{py} + 60{,}000$.

The stresses are used directly in Eqs. (12.3.4), (12.3.7), or (12.3.8), as appropriate, substituting f_{ps} for f_y and A_{ps} for A_s.

Allowable Stresses at Working Load

Steel:	
Maximum jacking stress, but not to exceed recommendation by steel or anchorage manufacturer	$0.8\ f_{pu}$
Immediately after transfer or posttensioning	$0.7\ f_{pu}$
Concrete:	
Temporary stresses immediately after prestressing	
Compression	$0.6\ f'_{ci}$
Tension in areas without reinforcement	$3\sqrt{f'_{ci}}$
Design load stresses (after losses)	
Compression	$0.45 f'_c$
Tension in precompressed tensile zones	$6\sqrt{f'_c}$

The allowable tension may be increased to $12\sqrt{f'_c}$ if it is demonstrated, by a comprehensive analysis taking cracking into account, that the short- and long-term deflections will be satisfactory.

The final steel stress, at working loads, will usually be 30,000 to 45,000 lb/in^2 less than the initial stress for pretensioned members. Posttensioned members will have slightly lower losses. Losses, from initial tensioning values, for pretensioned members may be predicted satisfactorily using the following expressions from the AASHTO, Bridge Specifications, Sec. 9.16.2;

$$\Delta f_s = SH + ES + CR_c + CR_s = \text{prestress loss} \quad \text{lb/in}^2 \tag{12.3.29}$$

where SH = shrinkage loss = 17,000 − 150 RH; ES = elastic shortening loss = $(E_s/E_c)f_{cir}$; CR_c = creep loss = $12f_{cir} - 7f_{cds}$; CR_s = relaxation loss = $5{,}000 - 0.1ES - 0.05(SH + CR_c)$, for low relaxation strands; f_{cir} = concrete stress at level of center of gravity of steel (cgs) at section considered, due to initial prestressing force and dead load; f_{cds} = change in concrete stress at cgs due to superimposed composite or noncomposite dead load; and RH = relative humidity, percent. The loss calculations are carried out for each critical moment section. The average annual relative humidity of the service environment should be used in the SH calculation.

The shear reinforcement requirements for simple cases are covered in an earlier section. In addition to the shear steel, a few stirrups or ties should be placed transverse to the member axis as close to the ends as possible, to control potential splitting cracking. The area of steel, from the AASHTO Bridge Specification, should be $A_t = 0.04\ f_{si}A_{ps}/20{,}000$. In posttensioned beams, end blocks will often have to be used to provide space for anchorage bearing plates.

The minimum clear spacing between strands in pretensioned members is three times the strand diameter near the ends of the beam, but many plants are set up to handle only 2-in spacings. Strands may be closer together in central positions of members, which will help in maximizing member effective depths and steel eccentricity.

Few precast, pretensioned members are solid rectangular sections, and single and double T beams and hollow floor slab units are used extensively in buildings. Hollow box beams and I-section beams are used extensively in bridges and in buildings with heavy design loads. Square piling with the prestressing strands arranged in a circular pattern is widely used. Because of the large number of possible sections, it is necessary to check availability of any particular section with local producers before designing any precast structure.

PRECAST CONCRETE

Precast slabs, beams, walls, and partitions as well as piles, retaining wall units, light standards, railroad crossings, and bridge slabs are being increasingly used because of the saving in time and labor cost. Such units vary in size from small slabs for use in floors or residences to large frames for industrial buildings. The small units, such as roof slabs, are cast in steel forms at central plants. Some of the larger units, such as bridge or highway slabs and wall units, are cast in wood forms at or near the place of use. A method by which wall or partition slabs are cast so that they are simply tilted into position has found wide use in housing and industrial construction. Another special adaptation is the method of casting complete floors on top of each other, then lifting into position vertically at the columns.

Particular attention must be given in the design of precast units to reduction in weight and to details to minimize the cost of erection and installation. Reduction in weight is obtained by the use of lightweight aggregates, high-strength concrete, and hollow units. Precast reinforced concrete units are seldom designed for concrete strengths of less than 4,000 lb/in^2. They are often combined with cast-in-place concrete so as to obtain the advantages of continuity. The combination of precast beams with cast-in-place slabs gives the advantage of T-beam action. Wall units are tied together by interlocking joints or by bolts. Care must be taken in shipping and handling to avoid damage to the precast units, and the design must take care of the stresses that come from such causes. All lifting devices built into the units should be designed for 100 percent impact. All units must be identified as to proper location and orientation in the structure.

Because precast units are made under conditions which allow good control of dimensions, certain restrictions can be relaxed that must be observed for cast-in-place concrete. Cover over the reinforcement for members not exposed to freezing need not be more than the nominal diameter of the steel but not less than ⅝ in. The maximum size of the coarse aggregate can be as large as one-third of the smaller dimension of the member. Precast wall panels are not limited to the minimum thickness requirements for cast-in-place walls.

To reduce the number of connections, precast units should generally be cast as large as can be properly handled. However, some joints will be needed to transfer moments, torsion, shear, and axial loads from one member to another. The integrity of the structure depends on the adequacy of the design of the various joints and connections. They may be made by use of bolts and pins or clips and keys, by welding the reinforcement or steel insert, or by a number of other methods limited only by the ingenuity of the designer. The connections should not be the weak links in the structure. Thought as to their location will avoid many problems.

JOINTS

Contraction and expansion joints may be needed at intervals in a structure to help care for movement due to temperature changes and shrinkage. Joints at 20- to 30-ft intervals provide good crack control. A weakened plane, formed in the tension side of the member by a slot ¼ in wide and ½ in deep, will induce the formation of contraction cracks at selected points. Structures over 200 ft in length should have special consideration given to contraction provisions.

Construction joints are necessary in most structures because all sections cannot be cast continuously. They should be made at points of minimum shearing stress and reinforced across the joint with a steel area of not less than 0.5 percent of the area of the section cut. Provision must be made for the transfer of shear and other forces through the construction joint. Joints in columns should be made at the underside of the floor members, haunches, T beams, and column capitals.

The hardened concrete at a joint should be properly prepared for bonding with the new concrete by being cleaned, roughened, and wetted. On this surface, a coat of neat cement grout or other bonding agent should be applied just before depositing the new concrete.

FORMS

Forms are usually built of wood or metal but in special cases may be made of plastic or fiberglass reinforced plastic. Wooden forms may be the most economical unless the construction allows for the repeated use of the same forms. Plywood and compressed wood fiber sheets, specially treated to make them waterproof, are frequently used for form faces where good surfaces are required. Forms must be designed so that they can be easily erected, removed, and reerected. The usual order of removing forms is (1) column sides, (2) joists, (3) girder and beam sides, (4) slab bottoms, and (5) girder and beam bottoms. Column forms are held together by clamps made of wood or steel, the spacing of which is smallest at the bottom and increases with the decrease in pressure. Beam forms consist of the bottom and two sides held together by clamps or cleats and supported by posts. Slab forms consist of boards or other form material supported by joists spaced 2 or 3 ft apart or other means. The joists either rest on a horizontal joist bearer fastened to the clamps of the beam or girder or are supported by stringers, or posts, or both.

Special consideration must be given to forms for prestressed concrete members. For pretensioned members the form must be constructed such that it will permit movement of the member during release of the prestressing force. For posttensioned members, the form should provide a minimum of resistance to shortening of the member. It is also necessary to consider the deflection of the members due to the stressing force.

Design in Formwork The formwork is an appreciable portion of the cost of most concrete structures. Any efforts, however, to reduce the cost of the forms must not go beyond the point of safe design to prevent failures which would in themselves raise the cost of construction.

All forms must conform to the dimensions and shape of the members and must be sufficiently tight to prevent leakage of the mortar. They must be properly braced and tied together to maintain their position and shape during the construction procedure.

The formwork must support all the vertical and lateral loads that may be applied until these loads can be carried by the concrete structure. Loads on the form include the weight of the forms, reinforcing steel, fresh concrete, and various construction live loads. The construction live load varies with conditions but is often assumed to be 75 lb/ft^2 of floor area. The formwork should also be designed to resist lateral loads produced by wind and movement of construction equipment. Most frequently the steel and concrete will not be placed in a symmetrical pattern and frequently large impact loads will occur. Because of the many varied conditions, it is frequently impossible to determine with any great precision the loads which the form must carry. The designer must therefore make safe assumptions by which the forms can be designed such that failure will not result.

Lateral pressures in forms for walls and columns are influenced by a number of factors: weight of concrete, height of placing, vibration, temperature, size and shape of form, amount and distribution of reinforcing steel, and several other variables. Formulas have been suggested for computing safe lateral pressures to be used in form design. However, because of limited test data, they are not generally accepted by all engineers.

Form Liners Absorbent form liners are occasionally applied to the surface of forms to extract the water from the surface of the concrete, eliminate air and water voids, and produce a concrete of uniform appearance with surfaces which are superior in durability and resistance to abrasion.

The vacuum process, whereby water is absorbed from the concrete through a special form liner made of two layers of screen or wire mesh covered by a layer of cloth, has a similar effect and if properly used reduces the water content of the concrete to a depth of several inches.

Neoprene and other types of rubber have been successfully used as liners in precasting work in which a number of units are made from one form. Rubber is particularly suited for patterned work.

Plastic form liners make it easy to obtain a textured surface or a glossy smooth surface. Generally speaking, plastic liners are easily cleaned and if not too thin are suitable for a number of reuses.

Removal of Forms The time that forms should remain in place depends on the character of the members and weather conditions. The strength of concrete must be ascertained before removing the forms. Unless special precautions are taken, concrete should not be placed below 40°F. Fresh concrete should never be subjected to temperatures below freezing. As an approximate guide for the minimum time for form removal, the following rules, which assume moist curing at not less than 70°F for the first 24 h, may be observed.

WALLS IN MASS WORK. In summer, 1 day; in cold weather, 3 days.

THIN WALLS. In summer, 1 day; in cold weather, 5 days.

COLUMNS. In summer, 1 day; in cold weather, 4 days, provided girders are shored to prevent appreciable weight reaching the columns.

SLABS UP TO 7-FT SPAN. In summer, 4 days; in cold weather, 2 weeks.

BEAMS AND GIRDER SIDES. In summer, 1 day; in cold weather, 5 days.

BEAMS AND GIRDERS AND LONG-SPAN SLABS. In summer, 7 days; in cold weather, 2 weeks.

CONDUITS. 2 or 3 days, provided there is not a heavy fill upon them.

ARCHES. If a small size, 1 week; large arches with a heavy dead load, 3 weeks.

Forms for prestressed members may be removed when sufficient prestressing has been applied to enable them to carry their dead loads and the expected construction loads.

EVALUATION OF EXISTING CONCRETE STRUCTURES

This section is intended to give some guidance to the persons responsible for inspections of structures, whether these are done routinely, or before changing a loading or use, or during remodeling, or when something suspicious is found. Serious problems clearly need to be evaluated by a structural engineer experienced in such evaluations, testing, and renovation; such problems should not be evaluated by one who is primarily a structural designer.

The following are some signs of distress. Any cracks wider than about 0.02 in (0.5 mm) are potentially serious, particularly if there are more than a few. If the cracks are inclined like the shear crack shown in Fig. 12.3.1, they are an additional warning sign and should be investigated promptly. At interior supports and other locations where the beams are continuous with the columns, these cracks may also start at the top surface. Any cracking parallel to the member axis is potentially serious.

Rust stains and streaks from cracks indicate corrosion problems. Corrosion, especially from salt, disrupts the concrete surrounding the steel long before the bar areas are significantly reduced, because rust occupies much more volume than the steel it replaces. This rusting causes internal cracking, which can often be detected by tapping on the concrete surface with a hammer (1-lb size), which produces a distinctive ''hollow'' sound. ''Stalactites'' growing from the bottoms of members may be either salt or lime, but both indicate water-penetration problems and the need for waterproofing work.

Concrete exposed to freezing and thawing or to attack by some chemicals may crumble and disintegrate, leading to loss of section area and protective cover on the reinforcement. Large deflections and/or slopes in members may be indicative of impending distress or disaster, but sometimes members were not built very straight, and in such cases the warning sign is actually a *change* in the conditions.

Any reinforced concrete building designed before about 1956 has a potential weakness in the shear strength of the beams and girders because of a deficiency in the codes of that era, and major changes in loading and seemingly minor signs of distress should be investigated carefully. This problem may also exist in highway bridges designed before 1974. Prestressed members should not have this problem.

Repairs are made by injecting epoxy into cracks, by surface patching, by chipping away significant volumes of concrete and replacing it, and by other means. The repair materials range from normal concretes to highly modified concrete and polymer materials.

12.4 AIR CONDITIONING, HEATING, AND VENTILATING

By Norman Goldberg

REFERENCES: ASHRAE Handbooks ''Fundamentals,'' ''Systems and Equipment,'' and ''Applications.'' Stamper and Koral, ''Handbook of Air Conditioning, Heating and Ventilating,'' Industrial Press. Carrier, ''Handbook of Air Conditioning Design,'' McGraw-Hill.

Air conditioning is the process of treating air to meet the requirements of a conditioned space by controlling its temperature, humidity, cleanliness, and distribution. This section presents standards, basic data, and physical laws for use in the design of air conditioning and related heating and ventilating systems.

COMFORT INDEXES

The human body generates heat and disipates that heat to the surrounding air by sensible flow and the evaporationg of moisture.

Effective Temperature Effective temperature (ET) combines the effect of ambient temperature and humidity into a single index.

ASHRAE Comfort Chart The American Society of Heating, Refrigeration, and Air Conditioning Engineers (ASHRAE) comfort envelope shown in Fig. 12.4.1 is based on clothing and activity. Comfort varies with skin temperature and skin wettedness.