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面向可持续发展的土建类工程教育丛书

土木工程专业英语

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机械工业出版社

本书的编写基于同济大学近十几年对土木工程专业英语课程的教学探索和大量的素材积累；全书涵盖土木工程各个领域的基本英语用法，有点有面；紧跟目前科研和工程热点，比如 BIM 技术、住宅工业化技术等；实用性很强，最后一章讲述了科技论文写作。全书分为 19 个单元，并将其归为 5 大部分，分别为结构工程、土木工程其他分支、土木工程新技术、项目管理和案例分析、专业英语写作。

本书可作为土木工程专业及相关专业的教材，也可供专业技术人员了解专业知识、提高英语水平时使用。

本书配有授课 PPT 和 Exercises 部分的参考答案等资源，免费提供给选用本书的授课教师，需要者请登录机械工业出版社教育服务网（www.cmpedu.com）注册下载。

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前言

随着“高铁外交”与中国“一带一路”倡议的实施，我国的土木工程在国际舞台上发挥着越来越重要的作用，涉外工程、科研项目越来越多，国际化趋势持续加速。国内土木工程领域的众多龙头公司已将开拓海外市场作为可持续发展的长期战略并取得了卓越成效。例如我国建筑行业的领军企业——中国建筑工程总公司（简称“中国建筑”），在2016年度新签海外合约额1264亿元，同比增长13%；实现海外营收796亿元，同比增长30%，首次突破百亿美元大关。中国交建、中国铁建、中国中车等国内著名的国际工程承包商在海外的订单额和营收也以每年20%左右的增速高速发展。

繁荣景象的延续需要后备力量的长久支持，土木工程专业英语的教学为国家“一带一路”倡议提供可靠人才保障。我国对外工程的长足发展不能仅依靠成本优势，更要凭借高端竞争优势，而企业竞争力的核心便是“人才”。目前，我国土木工程领域中既能熟练使用专业英语，又有过硬技术本领的人才极度匮乏。因此，紧追企业海外拓展的步伐，打破复合型双语人才匮乏的瓶颈，培养出与国际接轨、具有核心竞争力的人才是当务之急。

另一方面，我国高校毕业生就业形势异常严峻，土木工程专业英语的学习为毕业生的顺利就业增添筹码。目前，我国每年应届毕业生700多万人并逐年增加，众多毕业生为找工作费尽周折。随着企业工程的进一步国际化，精通专业英语的高质量复合型人才必定受到国内外著名企业的追捧。

土木工程专业英语的教学利于国家战略、利于学生发展，培养精通土木工程专业英语的高层次复合人才已成为广大土木工程专业相关管理者和一线教师的共识。

本书选择了19篇专业英语阅读材料，包括结构工程、土木工程其他分支、土木工程新技术、项目管理和案例分析以及专业英语写作等方面的内容。学生通过学习土木工程专业英语这门课程可以掌握英文科技专业书刊中各种句型的表达方式、语法、主要专业词汇及写作技巧，从而为今后阅读英文科技文献、撰写英文科技论文、从事涉外施工和涉外设计等工作奠定基础。

本书内容来源于同济大学土木工程学院结构工程与防灾研究所多年积累的教学讲义。讲义的第1版由蒋欢军和吕西林于2000年选编，第二版由周颖于2007年选编，第3版由鲁正于2015年选编。经过17年的教学实践和积累，最终在2017年12月，由鲁正选编形成本书。由于时间和水平关系，书中不妥之处在所难免，敬请批评指正。

编　者

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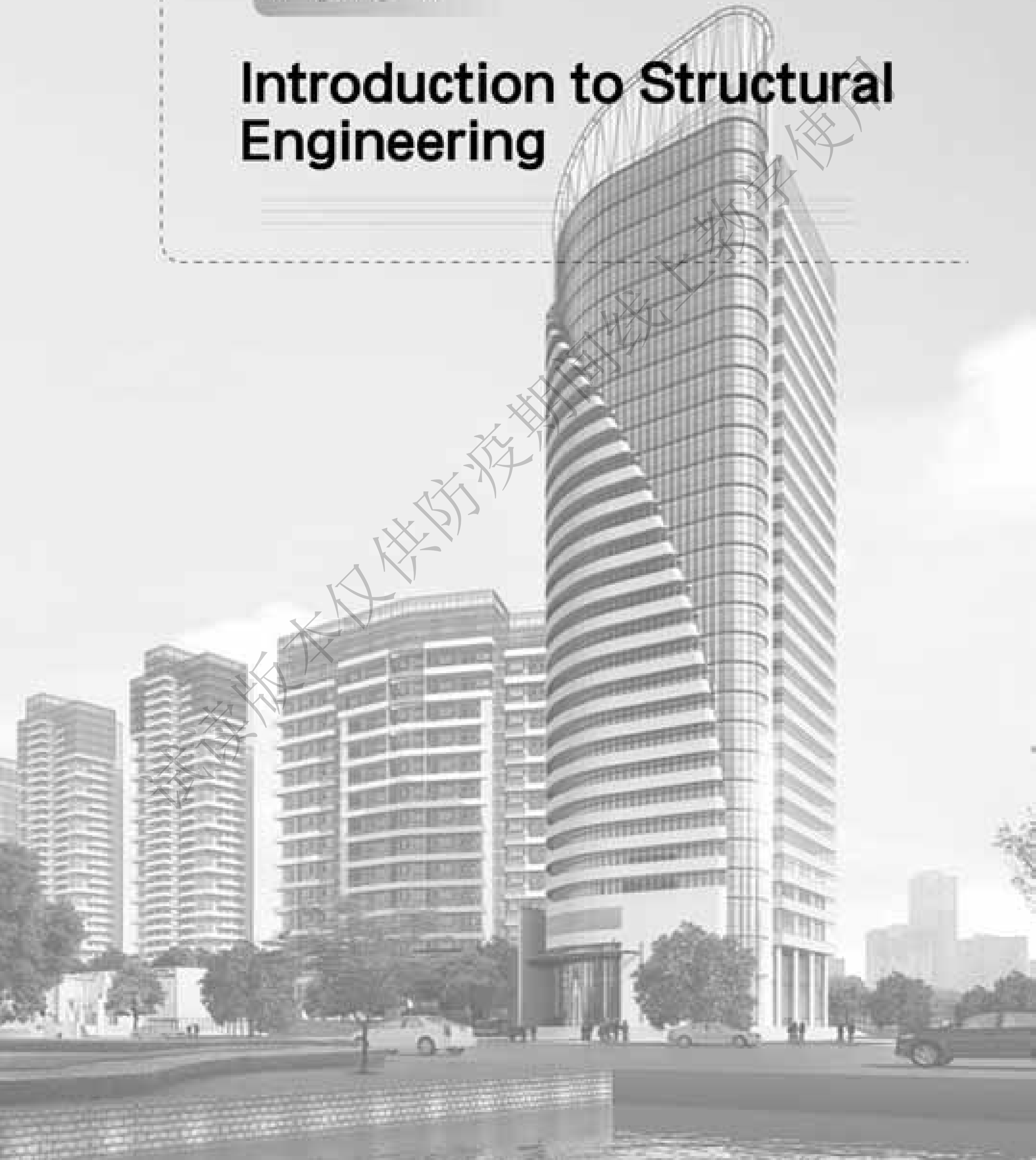
Part 1

Introduction to Structural Engineering



Unit 1

Introduction to Reinforced Concrete Design

1.1 Concrete, Reinforced Concrete, and Prestressed Concrete

Concrete is a stonelike material obtained by permitting a carefully proportioned mixture of cement, sand and gravel or other aggregate, and water to harden in forms of the shape and dimensions of the desired structure. The bulk of the material consists of fine and coarse aggregate. Cement and water interact chemically to bind the aggregate particles into a solid mass. Additional water, over and above that needed for this chemical reaction, is necessary to give the mixture the workability that enables it to fill the forms and surround the embedded reinforcing steel prior to hardening.[1] Concretes with a wide range of properties can be obtained by appropriate adjustment of the proportions of the constituent materials. Special cements (such as high early strength cements), special aggregates (such as various lightweight or heavyweight aggregates), admixtures (such as plasticizers, air-entraining agents, silica fume, and fly ash), and special curing methods (such as steam-curing) permit an even wider variety of properties to be obtained.

These properties depend to a very substantial degree on the proportions of the mix, on the thoroughness with which the various constituents are intermixed, and on the conditions of humidity and temperature in which the mix is maintained from the moment it is placed in the forms until it is fully hardened.[2] The process of controlling conditions after placement is known as *curing*. To protect against the unintentional production of substandard concrete, a high degree of skillful control and supervision is necessary throughout the process, from the proportioning by weight of the individual components, through mixing and placing, until the completion of curing.

The factors that make concrete a universal building material are so pronounced that it has been used, in more primitive kinds and ways than at present, for thousands of years, starting with lime mortars from 12000 to 6000 BCE in Crete, Cyprus, Greece, and the Middle East. The facility with which, while plastic, it can be deposited and made to fill forms or molds of almost any practical shape is one of these factors.[3] Its high fire and weather resistance is an evident advantage. Most of the constituent materials, with the exception of cement and additives, are usually available at low cost locally or at small distances from the construction site. Its compressive strength, like that of natural stones, is high, which makes it suitable for members primarily subject to compression, such as columns and arches. On the other hand, again as in natural stones, it is a relatively brittle material whose tensile strength is small compared with its compressive strength. This prevents its economical use in structural members that are subject to tension either entirely (such as in tie-rods) or over part of their cross sections (such as in beams or other flexural members).

To offset this limitation, it was found possible, in the second half of the nineteenth century, to use steel with its high tensile strength to reinforce concrete, chiefly in those places where its low tensile strength would limit the carrying capacity of the member. The reinforcement, usually round steel rods with appropriate surface deformations to provide interlocking, is placed in the forms in advance of the concrete. When completely surrounded by the hardened concrete mass, it forms an integral part of the member. The resulting combination of two materials, known as reinforced concrete, combines many of the advantages of each: the relatively low cost, good weather and fire resistance, good compressive strength, and excellent formability of concrete and the high tensile strength and much greater ductility and toughness of steel. It is this combination that allows the almost unlimited range of uses and possibilities of reinforced concrete in the construction of buildings, bridges, dams, tanks, reservoirs, and a host of other structures.

In more recent times, it has been found possible to produce steels, at relatively low cost, whose yield strength is 3 to 4 times and more that of ordinary reinforcing steels. Likewise, it is possible to produce concrete 4 to 5 times as strong in compression as the more ordinary concretes. These high-strength materials offer many advantages, including smaller member cross sections, reduced dead load, and longer spans. However, there are limits to the strengths of the constituent materials beyond which certain problems arise. To be sure, the strength of such a member would increase roughly in proportion to those of the materials. However, the high strains that result from the high stresses that would otherwise be permissible would lead to large deformations and consequently large deflections of such members under ordinary loading conditions. Equally important, the large strains in such high-strength reinforcing steel would induce large cracks in the surrounding low tensile strength concrete, cracks that not only would be unsightly but also could significantly reduce the durability of the structure. This limits the useful yield strength of high-strength reinforcing steel to 80 ksi (1ksi = 6.895MPa) according to many codes and specifications; 60ksi steel is most commonly used.

A special way has been found, however, to use steels and concretes of very high strength in combination. This type of construction is known as prestressed concrete. The steel, in the form of wires, strands, or bars, is embedded in the concrete under high tension that is held in equilibrium by compressive stresses in the concrete after hardening.[4] Because of this precompression, the concrete in a flexural member will crack on the tension side at a much larger load than when not so precompressed. Prestressing greatly reduces both the deflections and the tensile cracks at ordinary loads in such structures, and thereby enables these high-strength materials to be used effectively. Prestressed concrete has extended, to a very significant extent, the range of spans of structural concrete and the types of structures for which it is suited.

1.2 Structural Forms

The figures that follow show some of the principal structural forms of reinforced concrete. Pertinent design methods for many of them are discussed later in this volume.

Floor support systems for buildings include the monolithic slab-and-beam floor shown in Fig. 1-1, the one-way joist system of Fig. 1-2, and the flat plate floor, without beams or girders, shown in Fig. 1-3. The flat slab floor of Fig. 1-4, frequently used for more heavily loaded buildings such as warehouses, is similar to the flat plate floor, but makes use of increased slab thickness in the vicinity of the columns, as well as flared column tops, to reduce stresses and increase strength in the support region. The choice among these and other systems for floors and roofs depends upon functional requirements, loads, spans, and permissible member depths, as well as on cost and esthetic factors.

Fig. 1-1 One-way reinforced concrete floor slab with monolithic supporting beams (Portland Cement Association).

Fig. 1-2 One-way joist floor system, with closely spaced ribs supported by monolithic concrete beams; transverse ribs provide for lateral distribution of localized loads (Portland Cement Association).

Fig. 1-3 Flat plate floor slab, carried directly by columns without beams or girders (Portland Cement Association).

Fig. 1-4 Flat slab floor, without beams but with slab thickness increased at the columns and with flared column tops to provide for local stress concentration of forces.

Where long clear spans are required for roofs, concrete shells permit use of extremely thin surfaces, often thinner, relatively, than an eggshell. The folded plate roof of Fig. 1-5 is simple to form because it is composed of flat surfaces; such roofs have been employed for spans of 200 ft (1ft = 0.3048m) and more. The cylindrical shell of Fig. 1-6 is also relatively easy to form because it has only a single curvature; it is similar to the folded plate in its structural behavior and range of spans and loads. Shells of this type were once quite popular in the United Staler and remain popular in other parts of the world.

Fig. 1-5 Folded plate roof of 125 ft span, in addition to carrying ordinary roof loads, carries the second floor as well from a system of cable hangers; the ground floor is kept free of columns.

Fig. 1-6 Cylindrical shell roof providing column-free interior space.

Doubly curved shell surfaces may be generated by simple mathematical curves such as circular arcs, parabolas, and hyperbolas, or they may be composed of complex combinations of shapes. The hyperbolic paraboloid shape, defined by a concave downward parabola moving along a concave upward parabolic path, has been widely used, It has the interesting property that the doubly curved surface contains two systems of straight-line generators, permitting straight-form lumber to be used. The complex dome of Fig. 1-7, which provides shelter for performing arts events, consists essentially of a circular dome but includes monolithic upwardly curved edge surfaces to provide stiffening and strengthening in that critical region.

Bridge design has provided the opportunity for some of the most challenging and creative applications of structural engineering. The award-winning Napoleon Bonaparte Broward Bridge, shown in Fig. 1-8, is a six-lane, cable-stayed structure that spans St. John's River at Dame Point, Jacksonville, Florida. Its 1300 ft center span is the second longest of its type in the western hemisphere. Fig. 1-9 shows the Bennett Bay Centennial Bridge, a four-span continuous, segmentally cast-in-place box girder structure. Special attention was given to esthetics in this award-winning design. The spectacular Natchez Trace Parkway Bridge in Fig. 1-10, a two-span arch structure using hollow precast concrete elements, carries a two-lane highway 155 ft above the valley floor. This structure has won many honors, including awards from the American Society of Civil Engineers and the National Endowment for the Arts.

Fig. 1-7 Spherical shell in Lausanne, Switzerland. Upwardly curved edges provide stiffening for the central dome.

Fig. 1-8 Napoleon Bonaparte Broward Bridge, with a 1300 ft center span at Dame Point, Jacksonville. Florida (HNTB Corporation, Kansas City, Missouri).

Fig. 1-9 Bennett Bay Centennial Bridge, Coeur d' Alene, Idaho, a four-span continuous concrete box girder structure of length 1730 ft (HNTB Corporation, Kansas City, Missouri).

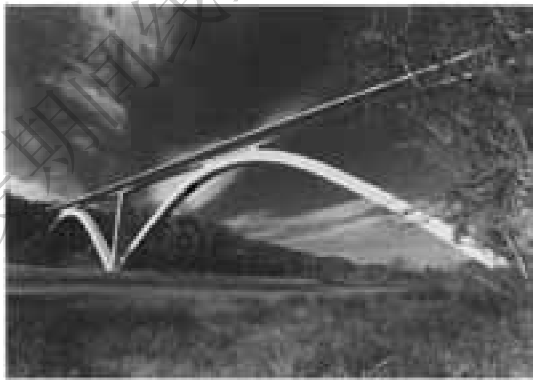


Fig. 1-10 Natchez Trace Parkway Bridge near Franklin, Tennessee, an award-winning two-span concrete arch structure rising 155 ft above the valley floor.

Cylindrical concrete tanks are widely used for storage of water or in waste purification plants. The design shown in Fig. 1-11 is proof that a sanitary engineering facility can be esthetically pleasing as well as functional. Cylindrical tanks are often pre-stressed circumferentially to maintain compression in the concrete and eliminate the cracking that would otherwise result from internal pressures.

Concrete structures may be designed to provide a wide array of surface textures colors, and structural forms. Fig. 1-12 shows a precast concrete building containing both color changed and architectural finishes.

The forms shown in Fig. 1-1 to Fig. 1-12 hardly constitute a complete inventory but are illustrative of the shapes appropriate to the properties of reinforced or prestressed concrete. They illustrate the adaptability of the material to a great variety of one-dimensional (beams, girders, columns), two-dimensional (slabs, arches, rigid frames), and three-dimensional (shells, tanks) structures and structural components. This variability allows the shape of the structure to be adapted to its function in an economical manner, and furnishes the architect and design engineer with a wide variety of possibilities for esthetically satisfying structural solutions.

Fig. 1-11 Circular concrete tanks used as a part of the wastewater purification facility at Howden, England (Northumbrian Water Authority with Luder and Jones, Architects).

Fig. 1-12 Concrete structures can be produced in a wide range of colors, finishes, and architectural detailing (Courtesy of Rocky Mountain Prestress Corp).

1.3 Loads

Loads that act on structures can be divided into three broad categories: dead loads, live loads, and environmental loads.

Dead loads are those that are constant in magnitude and fixed in location throughout the lifetime of the structure. Usually the major part of the dead load is the weight of the structure itself. This can be calculated with good accuracy from the design configuration, dimensions of the structure, and density of the material. For buildings, floor fill, finish floors, and plastered ceilings are usually included as dead loads, and an allowance is made for suspended loads such as piping and lighting fixtures. For bridges, dead loads may include wearing surfaces, sidewalks, and curbs, and an allowance is made for piping and other suspended loads.

Source: From *Minimum Design Loads for Buildings and Other Structures*. Used by permission of the American Society of Civil Engineers.

Live loads consist chiefly of occupancy loads in buildings and traffic loads on bridges. They may be either fully or partially in place or not present at all, and may also change in location. Their magnitude and distribution at any given time are uncertain, and even their maximum intensities throughout the lifetime of the structure are not known with precision. The minimum live loads for which the floors and roof of a building should be designed are usually specified in the building code that governs at the site of construction. Representative values of minimum live loads to be used in a wide variety of buildings are found in *Minimum Design Loads for Buildings and Other Structures*, a portion of which is reprinted in Table 1-1. The table gives uniformly distributed live loads for various types of occupancies; these include impact provisions where necessary. These loads are expected maxima and considerably exceed average values.

Table 1-1 Minimum uniformly distributed live loads

Occupancy or Use	Live Load/psf①	Occupancy or Use	Live Load/psf①
Apartments (see residential)		Dining room and restaurants	100
Access floor systems		Dwelling(see residential)	
Office use		Fire escapes	100
Computer use	100	On single-family dwellings only	40
Armories and drill rooms	150	Garages (passenger cars only)	40
Assembly areas and theaters		Trucks and buses②	
Fixed seats (listened to floor)	60	Grandstands (see stadium and arena bleachers)	
Lobbies	100	Gymnasiums, main floors and balconies③	100
Movable seats	100	Hospitals	
Platforms (assembly)	100	Operating rooms, laboratories	60
Stage floors	150	Patient rooms	40
Balconies(exterior)	100	Corridors above first floor	80
On one and two-family residences	60	Hotels (see residential)	
only, and not exceeding $100ft^2$		Libraries	
Bowling alleys, poolrooms, and similar		Reading rooms	60
recreational areas		Stack rooms④	150
Catwalks for maintenance access	40	Corridors above first floor	80
Corridors		Manufacturing	
First floor	100	Light	125
Other floors, same as occupancy		Heavy	250
served except as indicated		Marquees and canopies	75
Dance halls and ballrooms	100	Office buildings	
Decks (patio and roof)		File and computer rooms shall be designed for	
Same as area served, or for the		heavier loads based on anticipated occupancy	
type of occupancy accommodated		Lobbies and first-floor corridors	100
Offices	50	Schools	
Corridors above first floor	80	Classrooms	40
Penal institutions		Corridors above first floor	80
Cell blocks	40	First-floor corridors	100
Corridors	100	Sidewalks, vehicular driveways, and yards	250
Residential		subject to trucking	
Dwellings (one and two-family)		Stadiums and arenas	
Uninhabitable attics without storage	10	Bleachers	100
Uninhabitable attics with storage	20	Fixed seats (fastened to floor)	60
Habitable attics and sleeping areas	30	Stairs and exit ways	100

(Continued)

Occupancy or Use	Live Load/psf①	Occupancy or Use	Live Load/psf①
All other areas except stairs and balconies	40	One and two-family residences only	40
Hotels and multifamily houses		Storage areas above ceilings	20
Private rooms and corridors serving them	40	Storage warehouses (shall be designed for	
Public rooms and corridors serving them	100	heavier loads if required for anticipated storage)	
Reviewing stands, grandstands, and bleachers⑤		Light	125
Roofs		Heavy	250
Ordinary flat, pitched, and curved roofs	20	Stores	
Roofs used for promenade purposes	60	Retail	
Roofs used for roof gardens or assembly purpose	100	First floor	100
Roofs used for other special purposes.⑥		Upper floors	73
Awnings and canopies	5	Wholesale, all floors	125
Fabric construction supported by a		Walkways and elevated platforms	60
lightweight rigid skeleton structure⑦		(other than exitways)	
All other constructions	20	Yards and terraces, pedestrians	100

① Pounds per square foot, 1psf = 47.88Pa.

② Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provisions for truck and bus loadings.

③ In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of seats as follows: 24 lb㊀ per linear ft of seat applied in the direction parallel to each row of seats and 10 lb per linear ft of seat applied in the direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.

④ The loading applies to stack room floors that support nonmobile, double-faced library bookstacks subject to the following limitations: (a) the nominal bookstack unit height shall not exceed 90 in.㊁; (b) the nominal shelf depth shall not exceed 12 in. for each face; and (c) parallel rows of double-faced bookstacks shall be separated by aisles not less than 36 in. wide.

⑤ Other uniform loads in accordance with an approved method that contains provisions for truck loadings shall also be considered where appropriate.

⑥ Roofs used for other special purposes shall be designed for appropriate loads as approved by the authority having jurisdiction.

⑦ Nonreducible。

In addition to these uniformly distributed loads, it is recommended that, as an alternative to the uniform load, floors be designed to support safely certain concentrated loads if these produce a greater stress. For example, office floors are to be designed to carry a load of 2000lb distributed over an area 2.5ft square (6.25ft^2), to allow for the weight of a safe or other heavy equipments, and stair treads must safely support a 300lb load applied on the center of the tread. Certain reductions are often permitted in live loads for members supporting large areas, on the premise that it is not likely that the entire area would be fully loaded at one time.

Tabulated live loads cannot always be used. The type of occupancy should be considered and the probable loads computed as accurately as possible. Warehouses for heavy storage may be designed for loads as high as 500psf or more; unusually heavy operations in manufacturing buildings may require an increase in the 250psf value specified in Table 1-1; special provisions must be made

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㊀ 1lb：磅，1lb = 4.45N。

㊁ in：英寸，1in = 2.54cm。

for all definitely located heavy concentrated loads.

Live loads for highway bridges are specified by the American Association of State Highway and Transportation Officials (AASHTO) in its *LRFD Bridge Design Specifications*. For railway bridges, the American Railway Engineering and Maintenance-of-Way Association (AREMA) has published the *Manual of Railway Engineering*, which specifies traffic loads.

Environmental loads consist mainly of snow loads, wind pressure and suction, earthquake loads (i. e., inertia forces caused by earthquake motions), soil pressures on subsurface portions of structures, loads from possible ponding of rainwater on flat surfaces, and forces caused by temperature differentials. Like live loads, environmental loads at any given time are uncertain in both magnitude and distribution. The book, *Minimum Design Loads for Buildings and Other Structures*, contains much information on environmental loads, which is often modified locally depending, for instance, on local climatic or seismic conditions.

Fig. 1-13, from the 1972 edition of *Minimum Design Loads for Buildings and Other Structures*, which gives snow loads for the continental United States and is included here for illustration only. The 2005 edition gives much more detailed information. In either case, specified values don' t represent average values, but are expected to upper limits. A minimum roof load of 20 psf is often specified to provide for construction and repair loads and to ensure reasonable stiffness.

Much progress has been made in developing rational methods for predicting horizontal forces on structures due to wind and seismic action. The book summarizes current thinking regarding wind forces and has much information pertaining to earthquake loads as well.

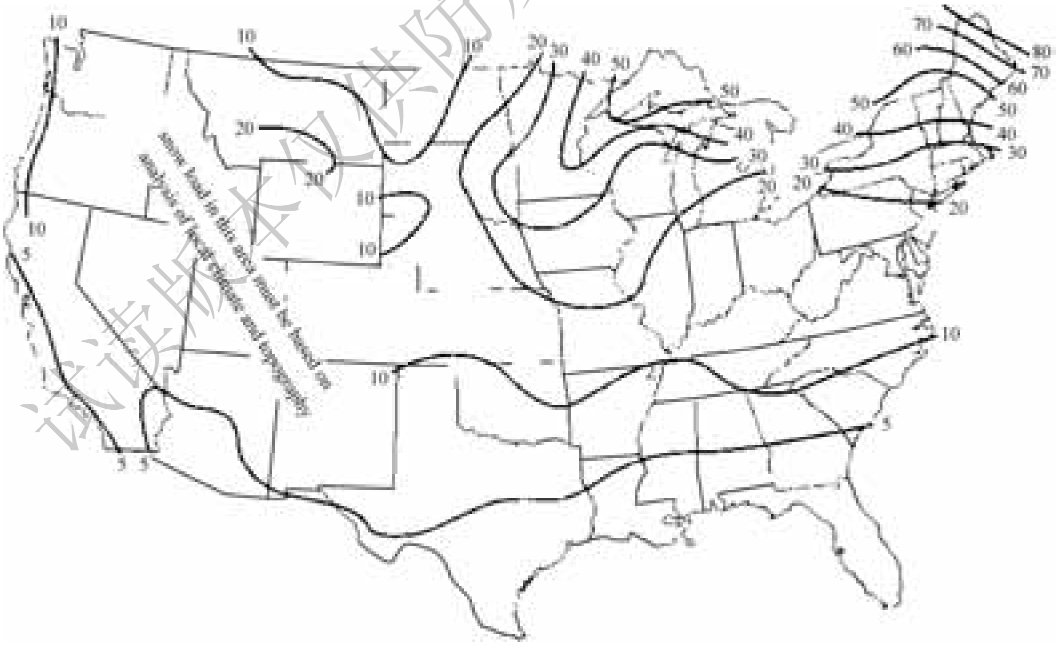


Fig. 1-13 Snow load in pounds per square foot (psf) **on the ground, 50-year mean recurrence interval.**

Wind pressures are specifically designed by per square foot of vertical wall surface. Depending upon locality, these equivalent static forces vary from about 10 to 50psf. Factors include basic wind speed, exposure (urban vs. open terrain, for example), height of the structure, the importance of the structure (i. e., consequences of failure), and gust effect factors to account for the fluctuating nature of the wind and its interaction with the structure.

Seismic forces may be found for a particular structure by elastic or inelastic dynamic analysis, considering expected ground accelerations and the mass, stiffness, and damping characteristics of the construction. However, the design is often based on equivalent static forces. The base shear is found by considering such factors as location, type of structure and its occupancy, total dead load, and the particular soil condition. The total lateral force is distributed to floors over the entire height of the structure in such a way as to approximate the distribution of forces obtained from a dynamic analysis.

1.4 Serviceability, Strength, and Structural Safety

To serve its purpose, a structure must be safe against collapse and serviceable in use. Serviceability requires that deflections be adequately small; that cracks, if any, be kept to tolerable limits; that vibrations be minimized; etc. Safety requires that the strength of the structure be adequate for all loads that may foreseeably act on it. If the strength of a structure, built as designed, could be predicted accurately, and if the loads and their internal effects (moments, shears, axial forces) were known accurately, safety could be ensured by providing a carrying capacity just barely in excess of the known loads.[5] However, there are a number of sources of uncertainty in the analysis, design, and construction of reinforced concrete structures. These sources of uncertainty, which require a definite margin of safety, may be listed as follows:

1. Actual loads may differ from those assumed.
2. Actual loads may be distributed in a manner different from that assumed.
3. The assumptions and simplifications inherent in any analysis may result in calculated load effects−moments, shears, etc. −different from those that, in fact, act in the structure.
4. The actual structural behavior may differ from that assumed, owing to imperfect knowledge.
5. Actual member dimensions may differ from those specified.
6. Reinforcement may not be in its proper position.
7. Actual material strength may be different from that specified.

In addition, in the establishment of a safety specification, consideration must be given to the consequences of failure. In some cases, a failure would be merely an inconvenience. In other cases, loss of life and significant loss of property may be involved. A further consideration should be the nature of the failure, should it occur. A gradual failure with ample warning permitting remedial measures is preferable to a sudden, unexpected collapse.

It is evident that the selection of an appropriate margin of safety is not a simple matter. However, progress has been made toward rational safety provisions in design codes.

1.4.1 Variability of Loads

Since the maximum load that will occur during the life of a structure is uncertain, it can be considered a random variable. In spite of this uncertainty, the engineer must provide an adequate structure. A probability model for the maximum load can be devised by means of a probability density function for loads, as represented by the frequency curve of Fig. 1-14a. The exact form of this distribution curve, for any particular type of loading such as office loads, can be determined only on the basis of statistical data obtained from large-scale load surveys.[6] A number of such surveys have been completed. For types of loads for which such data are scarce, fairly reliable information can be obtained from experience, observation, and judgment.

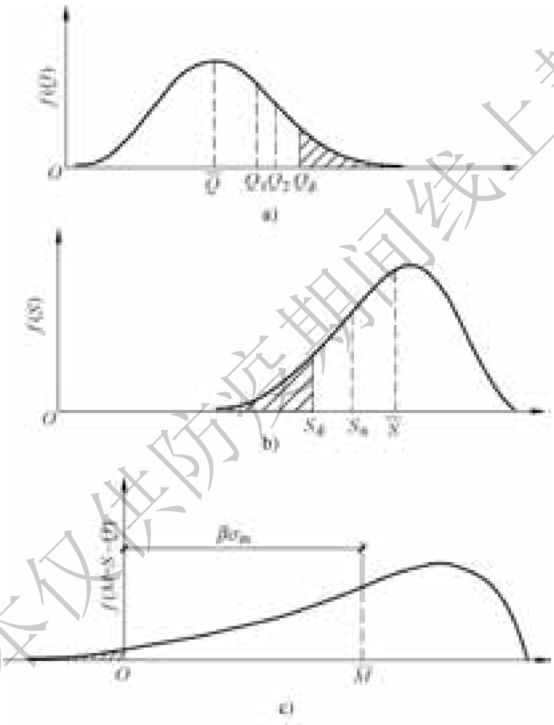


Fig. 1-14 Frequency curves for loads Q, strength S, and safety margin M.

a) Loads Q b) Strength S c) Safety margin $M=S-Q$

In such a frequency curve (Fig. 1-14a), the area under the curve between two abscissas, such as loads Q_1 and Q_2, represents the probability of occurrence of loads Q of magnitude $Q_1<Q<Q_2$. A specified service load Q_d for design is selected conservatively in the upper region of Q in the distribution curve, as shown. The probability of occurrence of loads larger than Q_d is then given by the shaded area to the right of Q_d. It is seen that this specified service load is considerably larger than the mean load Q acting on the structure. This mean load is much more typical of average load conditions than the design load $Q_{d.}$

1.4.2 Strength

The strength of a structure depends on the strength of the materials from which it is made. For this purpose, minimum material strengths are specified in standardized ways. Actual material strengths cannot be known precisely and therefore also constitute random variables. Structural strength depends, furthermore, on the care with which a structure is built, which in turn reflects the quality of supervision and inspection.[7] Member sizes may differ from specified dimensions, reinforcement may be out of position, poorly placed concrete may show voids, etc.

Strength of the entire structure or of a population of repetitive structures, e. g., highway overpasses, can also be considered a random variable with a probability density function of the type shown in Fig. 1-14b. As in the case of loads, the exact form of this function cannot be known but can be approximated from known data, such as statistics of actual, measured materials and member strengths and similar information. Considerable information of this type has been, or is being, developed and used.

1.4.3 Structural Safety

A given structure has a *safety margin* M if

$$M = S - Q > 0 \tag{1-1}$$

i. e., if the strength of the structure is larger than the load acting on it. Since S and Q are random variables, the safety margin $M = S - Q$ is also a random variable. A plot of the probability function of M may appear as in Fig. 1-14c. Failure occurs when M is less than zero. Thus, the probability of failure is represented by the shaded area in the figure.

Even though the precise form of the probability density functions for S and Q, and therefore for M, is not known, much can be achieved in the way of a rational approach to structural safety. One such approach is to require that the mean safety margin M be a specified number β of standard deviations σ_m above zero. It can be demonstrated that this results in the requirement that

$$\psi_s \bar{S} \geqslant \psi_L \bar{Q} \tag{1-2}$$

where ψ_s is a partial safety coefficient smaller than one applied to the mean strength $\bar{S}$ and ψ_L is a partial safety coefficient larger than one applied to the mean load $\bar{Q}$. The magnitude of each partial safety coefficient depends on the variance of the quantity to which it applies, S or Q, and on the chosen value of β, the reliability index of the structure. As a general guide, a value of the safety index β between 3 and 4 corresponds to a probability of failure of the order of 1 : 100000. The value of β is often established by calibration against well-proved and established designs.

In practice, it is more convenient to introduce partial safety coefficients with respect to code-specified loads which, as already noted, considerably exceed average values, rather than with respect to mean loads as in Eq. (1-2); similarly, the partial safety coefficient for strength is applied to nominal strength generally computed somewhat conservatively, rather than to mean strengths as in

Eq. (1-2). A restatement of the safety requirement in these terms is

$$\phi S_n \geqslant \gamma Q_d \tag{1-3a}$$

in which ϕ is a strength reduction factor applied to nominal strength S_n and γ is a load factor applied to calculated or code-specified design loads Q_d. Furthermore, recognizing the differences in variability between, say, dead loads D and live loads L, it is both reasonable and easy to introduce different load factors for different types of loads. The preceding equation can thus be written

$$\phi S_n \geqslant \gamma_d D + \gamma_l L \tag{1-3b}$$

in which γ_d is a load factor somewhat greater than 1.0 applied to the calculated dead load D and γ_l is a larger load factor applied to the code-specified live load L. When additional loads, such as the wind load W, are to be considered, the reduced probability that maximum dead, live, and wind or other loads will act simultaneously can be incorporated by using modified load factors such that

$$\phi S_n \geqslant \alpha(\gamma_d D + \gamma_l L + \gamma_w W + \cdots) \tag{1-3c}$$

Present U.S. design specifications follow the format of Eq. (1-3b) and Eq. (1-3c).

1.5 Design Basis

The single most important characteristic of any structural member is its actual strength, which must be large enough to resist, with some margin to spare, all foreseeable loads that may act on it during the life of the structure, without failure or other distress.[8] It is logical, therefore, to proportion members, i.e., to select concrete dimensions and reinforcement, so that member strengths are adequate to resist forces resulting from certain hypothetical overload stages, significantly above loads expected actually to occur in service.[9] This design concept is known as strength design.

For reinforced concrete structures at loads close to and at failure, one or both of the materials, concrete and steel, are invariably in their nonlinear inelastic range. That is, concrete in a structural member reaches its maximum strength and subsequent fracture at stresses and strains far beyond the initial elastic range in which stresses and strains are fairly proportional. Similarly, steel close to and at failure of the member is usually stressed beyond its elastic domain into and even beyond the yield region. Consequently, the nominal strength of a member must be calculated on the basis of this inelastic behavior of the materials.

A member designed by the strength method must also perform in a satisfactory way under normal service loading. For example, beam deflections must be limited to acceptable values, and the number and width of flexural cracks at service loads must be controlled. Serviceability limit conditions are an important part of the total design, although attention is focused initially on strength.

Historically, members were proportioned so that stresses in the steel and concrete resulting from normal service loads were within specified limits. These limits, known as allowable stresses, were only fractions of the failure stresses of the materials. For members proportioned on such a service load basis, the margin of safety was provided by stipulating allowable stresses under service loads that were appropriately small fractions of the compressive concrete strength and the steel yield stress.[10] We now refer to this basis for design as service load design. Allowable stresses, in practice, were set at about one-half the concrete compressive strength and one-half the yield stress of the steel.

Because of the difference in realism and reliability, the strength design method has displaced the older service load design method. However, the older method provides the basis for some serviceability checks and is the design basis for many older structures. Throughout this text, strength design is presented almost exclusively.

New Words and Expressions

plasticizer	*n.*	塑化剂（砂浆），增强剂（塑料），柔韧剂
air-entraining agent	*n.*	加气剂
strand	*n.*	钢筋（丝）束，钢绞线
wire	*n.*	线材，钢丝索
monolithic	*adj.*	整体的
joist	*n.*	搁栅，小梁，托梁
flared column top		喇叭形（漏斗式）柱顶
esthetic	*adj.*	同 aesthetic，美学的，审美的，艺术的
hyperbolic paraboloid		双曲抛物面
spherical dome		球形屋顶，球形穹顶
sanitary	*adj.*	关于环境卫生的，清洁的
suspended load		悬挂荷载
stair tread		楼梯踏步板
serviceability	*n.*	适用性
abscissas	*n.*	横坐标
overload	*n.*	超载，超负荷

Notes

1. Additional water, over and above that needed for this chemical reaction, is necessary to give the mixture the workability that enables it to fill the forms and surround the embedded reinforcing steel prior to hardening.

本句难点解析：句子主体是 Additional water is necessary to give the mixture the workability，即“多余的水分是为了保证混凝土的和易性”。关键词是 Additional “多余的”，workability “和易性”。其中 over and above that needed for this chemical reaction 是用来修饰 additional water 的，表达“多余的水分”中的“多余”所指的方面。that enables it to fill the forms and surround the embedded reinforcing steel prior to hardening 这句话为 workability 的定语从句，表示“保证和易性是为了让混合物（即混凝土）在硬化前能成型包裹住钢筋”。

本句大意如下：超过化学作用所需的额外的水用来提供混合物的和易性，使之在硬化之

前能够填入模板以及包裹嵌入的钢筋。

2. These properties depend to a very substantial degree on the proportions of the mix, on the thoroughness with which the various constituents are intermixed, and on the conditions of humidity and temperature in which the mix is maintained from the moment it is placed in the forms until it is fully hardened.

本句难点解析：句子主体是 These properties depend on the proportions, the thoroughness and the conditions of humidity and temperature，即“这些性质取决于混合的彻底性与温湿度的条件”。其中 to a very substantial degree 作为状语修饰 depend，表示“很大程度上”。with which the various constituents are intermixed 以及 in which the mix is maintained from the moment it is placed in the forms until it is fully hardened 则分别作为定语从句修饰 thoroughness 和 conditions，表示“彻底性”所指代的是几种材料的混合情况，“条件”所指代的是混合物从放置进模具直至成型的养护条件。

本句大意如下：这些特性很大程度上依赖于混合比例，不同组成成分搅拌的充分性，以及混合物在浇注后直到硬化前的湿度和温度。

3. The facility with which, while plastic, it can be deposited and made to fill forms or molds of almost any practical shape is one of these factors.

本句难点解析：句子主体是 The facility is one of these factors，即“便捷性是这些因素的其中之一”。with which it can be deposited and made to fill forms or molds of almost any practical shape 作为定语从句修饰 facility，表示“便捷性”所指代的是混凝土能放置在几乎任何实际模具中成型。而 while plastic 则是作为状语修饰 deposit 以及 make，表示放置的时候混凝土应处于塑性状态。

本句大意如下：当混凝土处于塑性状态时，它可以很容易地浇注并填充到任何形式的模板或磨具中这一优点便是其中一个因素。

4. The steel, in the form of wires, strands, or bars, is embedded in the concrete under high tension that is held in equilibrium by compressive stresses in the concrete after hardening.

本句难点解析：句子主体是 The steel is embedded in the concrete under high tension。其中 in the form of wires, strands, or bars 作为修饰成分说明钢筋的形式，that is held in equilibrium by compressive stresses in the concrete after hardening 作为定语从句修饰拉应力，表明与钢筋拉应力相平衡的是混凝土中的压应力。

本句大意如下：钢筋的形式通常是钢线、钢绞线或是钢筋，其在较高的拉力作用下埋入混凝土中，且该拉力与硬化后混凝土的受压应力保持平衡。

5. If the strength of a structure, built as designed, could be predicted accurately, and if the loads and their internal effects (moments, shears, axial forces) were known accurately, safety could be ensured by providing a carrying capacity just barely in excess of the known loads.

本句大意如下：如果按设计建造的结构，其结构强度，荷载以及对应的内力（弯矩，剪力，轴力）都能被准确预测的话，则仅需使其承载力比已知荷载稍大便可以确保结构安全。

6. The exact form of this distribution curve, for any particular type of loading such as office loads, can be determined only on the basis of statistical data obtained from large-scale load surveys.

本句难点解析：句子主体是 The exact form can be determined on the basis of statistical data。其中 for any particular type of loading such as office loads 修饰 curve，obtained from large-scale load surveys 作为定语从句修饰 data。

本句大意如下：对于任意一种具体的荷载，比如办公室荷载，其分布曲线的确定仅能以大规模荷载调查所得的统计数据作为基础。

7. Structural strength depends, furthermore, on the care with which a structure is built, which in turn reflects the quality of supervision and inspection.

本句难点解析：句子主体是 Structural strength depends on the care。with which a structure is built 作为定语从句修饰 care，which in turn reflects the quality of supervision and inspection 同样作为定语从句修饰 care。

本句大意如下：此外，结构的强度还取决于建造时的认真程度，这反过来也反映了监管及检查的质量。

8. The single most important characteristic of any structural member is its actual strength, which must be large enough to resist, with some margin to spare, all foreseeable loads that may act on it during the life of the structure, without failure or other distress.

本句难点解析：which 后面的从句作为定语从句修饰 strength，从句主体是 which must be large enough to resist all foreseeable loads，其中 with some margin to spare 作为伴随状语从句修饰前句，表示“强度应该有所富余”，that may act on it during the life of the structure 作为定语从句修饰 loads，without failure or other distress 作为伴随状语从句修饰主句。

本句大意如下：结构构件最重要的特性是其真实强度：强度必须足够大且有所富余，能足以抵抗构件工作过程中可能出现的所有荷载，且不发生破坏。

9. It is logical, therefore, to proportion members, i. e., to select concrete dimensions and reinforcement, so that member strengths are adequate to resist forces resulting from certain hypothetical overload stages, significantly above loads expected actually to occur in service.

本句难点解析：句子主体是 It is logical to proportion members。to select concrete dimensions and reinforcement 这一句可根据前面的 i. e. 得知其是解释 proportion members 的含义。

本句大意如下：应该合理地选择构件尺寸，即混凝土的截面尺寸以及钢筋尺寸，才能让构件有足够的强度抵抗可能出现的超载情况，即大幅超出正常使用期间预期荷载的情况。

10. For members proportioned on such a service load basis, the margin of safety was provided by stipulating allowable stresses under service loads that were appropriately small fractions of the compressive concrete strength and the steel yield stress.

本句难点解析：句子中 on such a service load basis 作为状语修饰 proportioned，表明“构件尺寸按正常使用荷载进行定义”。

本句大意如下：对于以正常使用荷载来定义截面尺寸的构件来说，安全度通过规定正常使用荷载下的应力来实现，限制该应力仅为混凝土抗压强度及钢筋屈服应力的一小部分。

Exercises

Translate the following phrases in to Chinese.

1. stonelike material
2. reinforcing steel
3. Construction site
4. heavyweight aggregate
5. constituent material
6. flexural member
7. prestressed concrete
8. base shear
9. design configuration
10. average value

Translate the following Sentences in to Chinese.

1. Concrete is a stonelike material obtained by permitting a carefully proportioned mixture of cement, sand and gravel or other aggregates, and water to harden in forms of the shape and dimensions of the desired structure.

2. To offset this limitation, it was found possible, in the second half of the nineteenth century, to use steel with its high tensile strength to reinforce concrete, chiefly in those places where its low tensile strength would limit the carrying capacity of the member.

3. The resulting combination of two materials, known as reinforced concrete, combines many of the advantages of each: the relatively low cost, good weather and fire resistance, good compressive strength, and excellent formability of concrete and the high tensile strength and much greater ductility and toughness of steel.

4. Dead loads are those that are constant in magnitude and fixed in location throughout the lifetime of the structure.

5. Seismic forces may be found for a particular structure by elastic or inelastic dynamic analysis, considering expected ground accelerations and the mass, stiffness, and damping characteristics of the construction.

Unit 2

Introduction to Prestressed Concrete

2.1 Introduction

Modern structural engineering tends to progress toward more economic structures through gradually improved methods of design and the use of higher strength materials. This results in a reduction of cross-sectional dimensions and consequent weight savings. Such developments are particularly important in the field of reinforced concrete, where the dead load represents a substantial part of the total design load. Also, in multistory buildings, any saving in depth of members, multiplied by the number of stories, can represent a substantial saving in total height, load on foundations, length of heating and electrical ducts, plumbing risers, and wall and partition surfaces.[1]

Significant savings can be achieved by the use of high-strength concrete and steel in conjunction with present-day design methods, which permit an accurate appraisal of member strength. However, there are limitations to this development, due mainly to the interrelated problems of cracking and deflection at service loads. The efficient use of high-strength steel is limited by the fact that the amount of cracking (width and number of cracks) is proportional to the strain, and therefore the stress, in the steel. Although a moderate amount of cracking is normally not objectionable in structural concrete, excessive cracking is undesirable in that it exposes the reinforcement to corrosion, it may be visually offensive, and it may trigger a premature failure by diagonal tension. The use of high-strength materials is further limited by deflection considerations, particularly when refined analysis is used. The slender members that result may permit deflections that are functionally or visually unacceptable. This is further aggravated by cracking, which reduces the flexural stiffness of members.

These limiting features of ordinary reinforced concrete have been largely overcome by the development of prestressed concrete. A prestressed concrete member can be defined as one in which there have been introduced internal stresses of such magnitude and distribution that the stresses resulting from the given external loading are counteracted to a desired degree. Concrete is basically a compressive material, with its strength in tension being a low and unreliable value. Prestressing applies a precompression to the member that reduces or eliminates undesirable tensile stresses that would otherwise be present. Cracking under service loads can be minimized or even avoided entirely. Deflections may be limited to an acceptable value; in fact, members can be designed to have zero deflection under the combined effects of service load and prestress force. Deflection and crack control, achieved through prestressing, permit the engineer to make use of efficient and economical high-strength steels in the form of strands, wires, or bars, in conjunction with concretes of much higher strength than normal.[2] Thus, prestressing results in overall improvement in performance of structural concrete used for ordinary loads and spans and extends the range of application far beyond old limits, leading not only to much longer spans than previously thought possible, but also permitting innovative new structural forms to be employed.[3]

■ 2.2 Effects of Prestressing

There are at least three alternative ways to look at the prestressing of concrete: (a) as a method of achieving concrete stress control, by which the concrete is precompressed so that tension normally resulting from the applied loads is reduced or eliminated; (b) as a means for introducing *equivalent loads* on the concrete member so that the effects of the applied loads are counteracted to the desired degree; and (c) as a special variation of reinforced concrete in which prestrained high-strength steel is used, usually in conjunction with high-strength concrete. Each of these viewpoints is useful in the analysis and design of prestressed concrete structures, and they will be illustrated in the following paragraphs.

2.2.1 Concrete Stress Control by Prestressing

Many important features of prestressed concrete can be demonstrated by simple examples. Consider first the plain, unreinforced concrete beam shown in Fig. 2-1a. It carries a single concentrated load at the center of its span (The self-weight of the member will be neglected here). As the load W is gradually applied, longitudinal flexural stresses are induced. If the concrete is stressed only within its elastic range, the flexural stress distribution at midspan will be linear, as shown.

At a relatively low load, the tensile stress in the concrete at the bottom of the beam will reach the tensile strength of the concrete f_r, and a crack will form. Because no restraint is provided against upward extension of the crack, the beam will collapse without further increase of load.

Now consider an otherwise identical beam, shown in Fig. 2-1b, in which a longitudinal axial force P is introduced prior to the vertical loading. The longitudinal prestressing force will produce a uniform axial compression $f_c = P/A_c$, where A_c is the cross-sectional area of the concrete. The force can be adjusted in magnitude so that when the transverse load Q is applied, the superposition of stresses due to P and Q will result in zero tensile stress at the bottom of the beam as shown. Tensile stress in the concrete may be eliminated in this way or reduced to a specified amount.

But it would be more logical to apply the prestressing force near the bottom of the beam, to compensate more effectively for the load-induced tension.[4] A possible design specification, for example, might be to introduce the maximum compression at the bottom of the beam without causing tension at the top, when only the prestressing force acts.[5] It is easily shown that, for a beam with a rectangular cross section, the point of application of the prestressing force should be at the lower third point of the section depth to achieve this. The force P, with the same value as before, but applied with eccentricity $e=h/6$ relative to the concrete centroid, will produce a longitudinal compressive stress distribution varying linearly from zero at the top surface to a maximum of $2f_c = P/A_c + Pec_2/I_c$ at the bottom, where f_c is the concrete stress at the concrete centroid, c_2 is the distance from the concrete centroid to the bottom of the beam, and I_c is the moment of inertia of the cross section. This is shown in Fig. 2-1c. The stress at the bottom will be exactly twice the value produced before by axial prestressing.

Consequently, the transverse load can now be twice as great as before, or $2Q$, and still cause no tensile stress. In fact, the final stress distribution resulting from the superposition of load and prestressing force in Fig. 2-1c is identical to that of Fig. 2-1b, with the same prestressing force, although the load is twice as great. The advantage of eccentric prestressing is obvious.

The methods by which concrete members are prestressed will be discussed in Section 2. 3. For present purposes, it is sufficient to know that one practical method of prestressing uses high-strength steel tendons passing through a conduit embedded in the concrete beam. The tendon is anchored, under high tension, at both ends of the beam, thereby causing a longitudinal compressive stress in the concrete. The prestress force of Fig. 2-1b and c could easily have been applied in this way.

A significant improvement can be made, however, by using a prestressing tendon with variable eccentricity with respect to the concrete centroid, as shown in Fig. 2-1d. The load $2Q$ produces a bending moment that varies linearly along the span, from zero at the supports to maximum at midspan. Intuitively, one suspects that the best arrangement of prestressing would produce a countermoment that acts in the opposite sense to the load-induced moment and that would vary in the same way. This would be achieved by giving the tendon with an eccentricity that varies linearly, from zero at the supports to maximum at midspan. This is shown in Fig. 2-1d. The stresses at midspan are the same as those in Fig. 2-1c, both when the load $2Q$ acts and when it does not. At the supports, where only the prestress force with zero eccentricity acts, a uniform compression stress f_c is obtained as shown.

For each characteristic load distribution, there is a best tendon profile that produces a prestress moment diagram that corresponds to that of the applied load. If the prestress countermoment is made exactly equal and opposite to the load-induced moment, the result is a beam that is subject only to uniform axial compressive stress in the concrete all along the span. Such a beam would be free of flexural cracking, and theoretically it would not be deflected up or down when that particular load is in place, compared to its position as originally cast. Such a result would be obtained for a load of $(1/2)\times 2Q = Q$, as shown in Fig. 2-1e, for example.

Some important conclusions can be drawn from these simple examples as follows:

1. Prestressing can control or even eliminate concrete tensile stress for specified loads.
2. Eccentric prestress is usually much more efficient than concentric prestress.
3. Variable eccentricity is usually preferable to constant eccentricity, from the viewpoints of both stress control and deflection control.

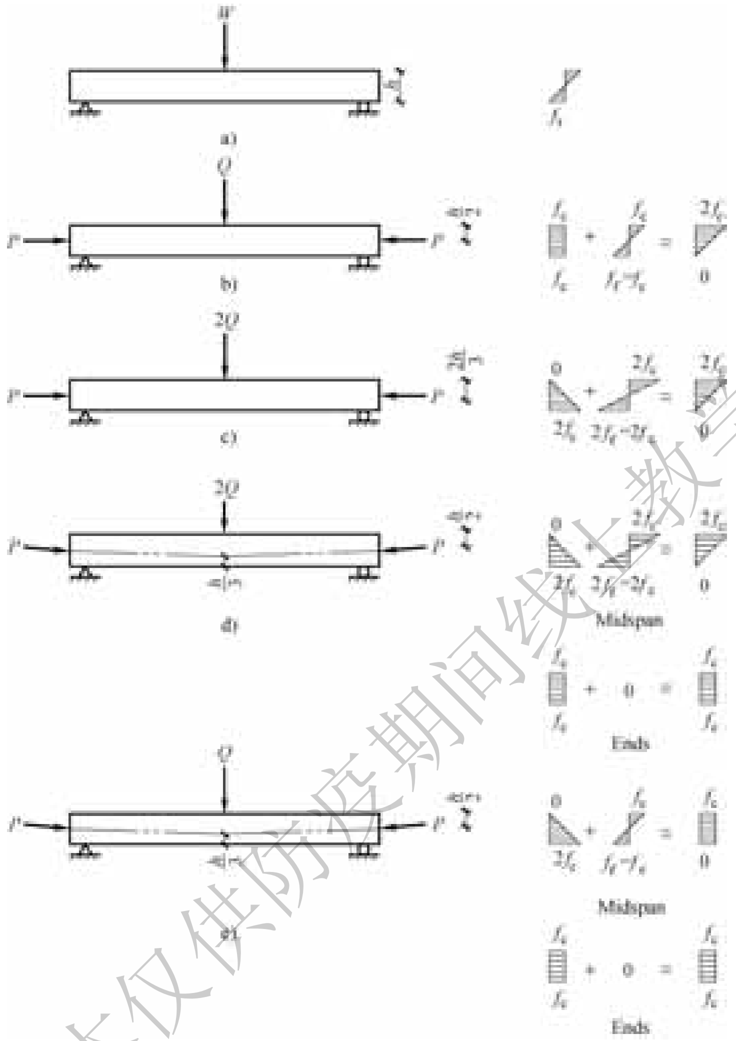


Fig. 2-1 Alternative schemes for prestressing a rectangular concrete beam.

a) Plain concrete beam b) Axially prestressed beam c) Eccentrically prestressed beam
d) Beam with variable eccentricity e) Balanced load stage for beam with variable eccentricity

2.2.2 Equivalent Loads

The effect of a change in the vertical alignment of a prestressing tendon is to produce a vertical force on the concrete beam. That force, together with the prestressing force acting at the ends of the beam through the tendon anchorages, can be looked upon as a system of external loads.[6]

In Fig. 2-2a, for example, a tendon that applies force P at the centroid of the concrete section at the ends of a beam and that has a uniform slope at angle θ between the ends and midspan introduces a transverse force $2P \sin \theta$ at the point of change of slope at midspan. At the anchorages, the vertical component of the prestressing force is $P\sin\theta$ and the horizontal component is $P\cos \theta$. The horizontal component is very nearly equal to P for the usual flat slope angles. The moment diagram for the beam of Fig. 2-2a is seen to have the same form as that for any center-loaded simple span.

The beam of Fig. 2-2b, with a curved tendon, is subject to a vertical upward load from the tendon as well as the forces *P* at each end. The exact distribution of the load depends on the profile of the tendon. A tendon with a parabolic profile, for example, will produce a uniformly distributed load. In this case, the moment diagram will be parabolic, as it is for a uniformly loaded simple span.

If a straight tendon is used with constant eccentricity, as shown in Fig. 2-2c, there are no vertical forces on the concrete, but the beam is subject to a moment *Pe* at each end, as well as the axial force *P*, and a diagram of constant moment results.

The end moment must also be accounted for in the beam of Fig. 2-2d, in which a parabolic tendon is used that does not pass through the concrete centroid at the ends of the span. In this case, a uniformly distributed upward load plus end anchorage forces are produced, as shown in Fig. 2-2b, but in addition, the end moments $M = Pe\cos\theta$ must be accounted for.

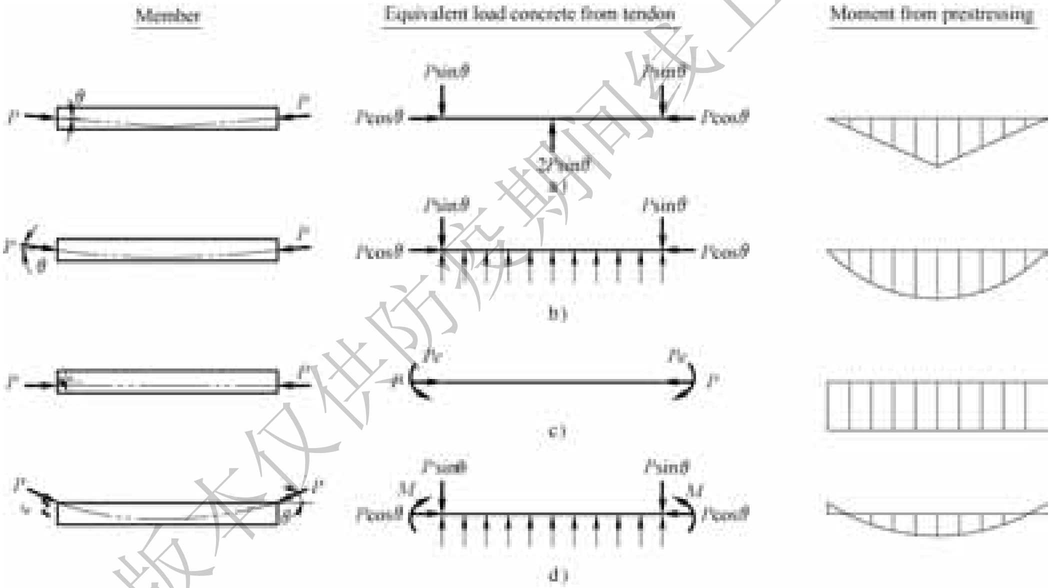


Fig. 2-2 Equivalent loads and moments produced by prestressing tendons.

It may be evident that for any arrangement of applied loads, a tendon profile can be selected so that the equivalent loads acting on the beam from the tendon are just equal and opposite to the applied loads. The result would be a state of pure compressive stress in the concrete, as discussed in somewhat different terms in reference to stress control and Fig. 2-1e. An advantage of the equivalent load concept is that it leads the designer to select what is probably the best tendon profile for a particular loading.

2.2.3 Prestressed Concrete as a Variation of Reinforced Concrete

In the descriptions of the effects of prestressing in the paragraphs above, it was implied that the prestress force remained constant as the vertical load was introduced, that the concrete responded elastically, and that no concrete cracking occurred.[7] These conditions may prevail up to about the service load level, but if the loads should be increased much beyond that, flexural tensile stresses will eventually exceed the modulus of rupture and cracks will form. Loads can usually be increased much beyond the cracking load in well-designed prestressed beams.

Eventually both the steel and concrete at the cracked section will be stressed into the inelastic range. The condition at incipient failure is shown in Fig. 2-3, which shows a beam carrying a factored load equal to some multiple of the expected service load. The beam undoubtedly would be in a partially cracked state; a possible pattern of flexural cracking is shown in Fig. 2-3a.

At the maximum moment section, only the concrete in compression is effective, and all of the tension is taken by the steel. The external moment from the applied loads is resisted by the internal forces couple $Cz = Tz$. The behavior at this stage is almost identical to that of an ordinary reinforced concrete beam at overload. The main difference is that the very high strength steel used must be prestrained before loads are applied to the beam; otherwise, the high steel stresses would produce excessive concrete cracking and large beam deflections.

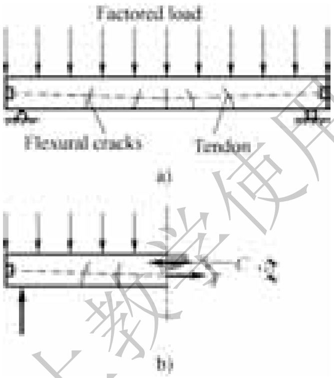


Fig. 2-3 Prestressed concrete beam at load near flexural failure.

a) Beam with factored load applied

b) Equilibrium of forces on left half of beam

Each of the three viewpoints described—concrete stress control, equivalent loads, and reinforced concrete using prestrained steel—is useful in the analysis and design of prestressed concrete beams, and none of the three is sufficient in itself. Neither an elastic stress analysis nor an equivalent load analysis provides information about strength or safety margin. However, the stress analysis is helpful in predicting the extent of cracking, and the equivalent load analysis is often the best way to calculate deflections. Strength analysis is essential to evaluate safety against collapse, but it tells nothing about cracking or deflections of the beam under service conditions.

2.3 Sources of Prestress Force

Prestress can be applied to a concrete member in many ways. Perhaps the most obvious method of precompressing is the use of jacks reacting against abutments, as shown in Fig. 2-4a. Such a scheme has been employed for large projects. Many variations are possible, including replacing the jacks with compression struts after the desired stress in the concrete is obtained or using inexpensive jacks, that remain in place in the structure, in some cases with a cement grout used as the hydraulic fluid.[8] The principal difficulty associated with such a system is that even a slight movement of the abutments will drastically reduce the prestress force.

In most cases, the same result is more conveniently obtained by tying the jack bases together with wires or cables, as shown in Fig. 2-4b. These wires or cables may be external, located on each side of the beam; more usually they are passed through a hollow conduit embedded in the concrete beam. Usually, one end of the prestressing tendon is anchored, and all the force is applied at the other end. After attainment of the desired prestress force, the tendon is wedged against the concrete and the jacking equipment is removed for reuse. Note that in this type of prestressing, the entire system is self-contained and is independent of relative displacement of the supports.

Another method of prestressing that is widely used is illustrated by Fig. 2-4c. The prestressing strands are tensioned between massive abutments in a casting yard prior to placing the concrete in the beam forms. The concrete is placed around the tensioned strands, and after the concrete has attained sufficient strength, the jacking pressure is released. This transfers the prestressing force to the concrete by bond and friction along the strands, chiefly at the outer ends.

Other means for introducing the desired prestressing force have been attempted on an experimental basis. Thermal prestressing can be achieved by preheating the steel by electrical or other means. Anchored against the ends of the concrete beam while in the extended state, the steel cools and tends to contract. The prestress force is developed through the restrained contraction. The use of expanding cement in concrete members has been tried with varying success. The volumetric expansion, restrained by steel strands or by fixed abutments, produces the prestress force.

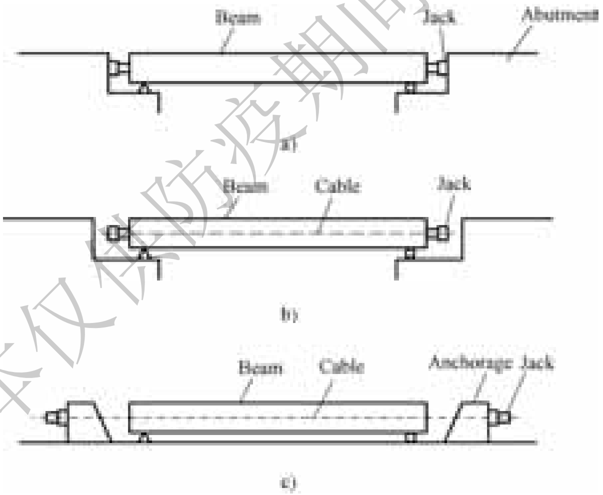


Fig. 2-4 Prestressing methods.

a) Post-tensioning by jacking against abutments b) Post-tensioning with jacks reacting against beam

c) Pretensioning with tendon stressed between fixed external anchorages

Most of the patented systems for applying prestress in current use are variations of those shown in Fig. 2-4b and c. Such systems can generally be classified as pretensioning or post-tensioning systems. In the case of pretensioning, the tendons are stressed before the concrete is placed, as in Fig. 2-4c. This system is well suited for mass production, since casting beds can be made several hundred feet long, the entire length cast at once, and individual beams cut to the desired length in a single casting. Fig. 2-5 shows workers using a hydraulic jack to tension strands at the anchorage of a long pretensioning bed. Although each tendon is individually stressed in this case, large capacity jacks are often used to tension all strands simultaneously.

In post-tensioned construction, shown in Fig. 2-4b, the tendons are tensioned after the concrete is placed and has acquired its strength. Usually, a hollow conduit or sleeve is provided in the beam, through which the tendon is passed. In some cases, hollow box-section beams are used. The jacking force is usually applied against the ends of the hardened concrete, eliminating the need for massive abutments. In Fig. 2-6, six tendons, each consisting of many individual strands, are being post-tensioned sequentially using a portable hydraulic jack.

Fig. 2-5 Massive strand jacking abutment at the end of a long pretensioning bed (Courtesy of Concrete Technology Corporation).

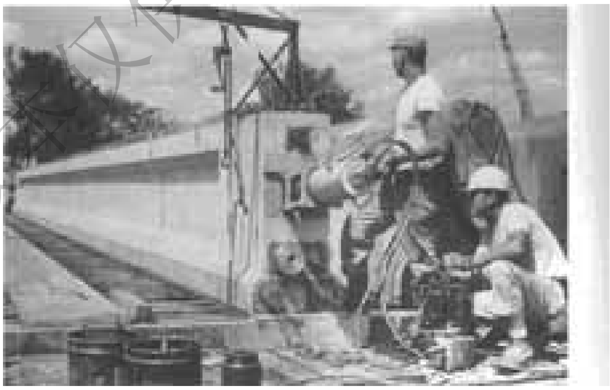


Fig. 2-6 Post-tensioning a bridge girder: use a portable jack to stress multistrand tendons (Courtesy of Concrete Technology Corporation).

A large number of particular systems, steel elements, jacks, and anchorage fittings have been developed in this country and abroad, many of which differ from each other only in minor details. As far as the designer of prestressed concrete structures is concerned, it is unnecessary and perhaps even undesirable to specify in detail the technique that is to be followed and the equipment to be used. It is frequently best to specify only the magnitude and line of action of the prestress force. The contractor is then free, in bidding the work, to receive quotations from several different prestressing subcontractors, with resultant cost savings. It is evident, however, that the designer must have some knowledge of the details of the various systems contemplated for use, so that in selecting cross-sectional dimensions, any one of several systems can be accommodated.[9]

2.4 Prestressing Steels

Early attempts at prestressing concrete were unsuccessful because steel of ordinary structural strength was used. The low prestress obtainable in such rods was quickly lost due to shrinkage and creep in the concrete.

Such changes in length of concrete have much less effect on prestress force if that force is obtained using highly stressed steel wires or cables. In Fig. 2-7a, a concrete member of length L is prestressed using steel bars of ordinary strength stressed to 24000psi (1psi = 6.89kPa). With $E_s = 29\times10^6$psi, the unit strain ε_s, required to produce the desired stress in the steel of 24000psi is

$$\varepsilon_s=\frac{\Delta L}{L}=\frac{f_s}{E_s}=\frac{24000\text{psi}}{29\times10^6\text{psi}}=8.0\times10^{-4}$$

However, the long-term strain in the concrete due to shrinkage and creep alone, if the prestress forces were maintained over a long period, would be on the order of 8.0×10^{-4} and would be sufficient to completely relieve the steel of all stress.

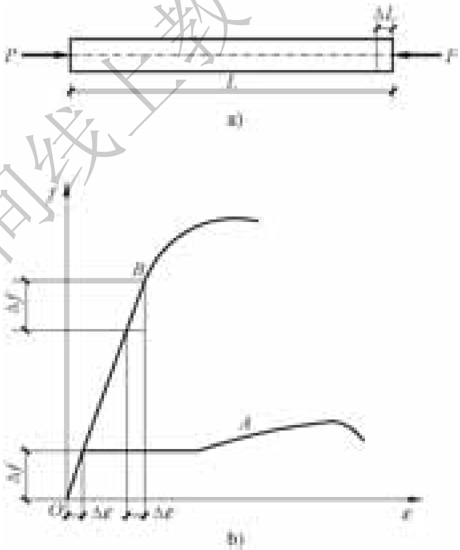


Fig. 2-7 Loss of prestress due to concrete shrinkage and creep.

Alternatively, suppose that the beam is prestressed using high tensile steel stressed to 150000psi. The elastic modulus of steel does not vary greatly, and the same value of 29×10^6psi will be assumed here. Then in this case, the unit strain required to produce the desired stress in the steel is

$$\varepsilon_s=\frac{150000\text{psi}}{29\times10^6\text{psi}}=51.7\times10^{-4}$$

If shrinkage and creep strain are the same as before, the net strain in the steel after these losses is

$$\varepsilon_{s,\text{net}}=(51.7-8.0)\times10^{-4}=43.7\times10^{-4}$$

and the corresponding stress after losses is

$$f_s=\varepsilon_{s,\text{net}}E_s=4.37\times10^{-4}\times29\times10^6\text{psi}=127000\text{psi}$$

This represents a stress loss of about 15 percent, compared with 100 percent loss in the beam using ordinary steel. It is apparent that the amount of stress lost because of shrinkage and creep is dependent upon the original stress in the steel. Therefore, the higher the original stress, the lower the percentage loss. This is illustrated graphically by the stress-strain curves of Fig. 2-7b. Curve *A* is representative of ordinary reinforcing bars, with a yield stress of 60000psi, while curve *B* represents high tensile steel, with an ultimate stress of 250000psi. The stress change Δf resulting from a certain change in strain $\Delta \varepsilon$ is seen to have much less effect when high steel stress levels are attained. Prestressing of concrete is therefore practical only when steels of very high strength are used.

Prestressing steel is most commonly used in the form of individual wires, stranded cable made up of seven wires, and alloy-steel bars.

The tensile stress permitted by ACI Code 18.5 in prestressing wires, strands, or bars is dependent upon the stage of loading. When the jacking force is first applied, a stress of $0.80 f_{pu}$ or $0.94 f_{py}$ is allowed, whichever is smaller, where f_{pu} is the ultimate strength of the steel and f_{py} is the yield strength. Immediately after transfer of prestress force to the concrete, the permissible stress is $0.74 f_{pu}$ or $0.82 f_{py}$, whichever is smaller (except at post-tensioning anchorages where the stress is limited to $0.70 f_{pu}$). The justification for a higher allowable stress during the stretching operation is that the steel stress is known quite precisely at this stage. Hydraulic jacking pressure and total steel strain are quantities that are easily measured. In addition, if an accidentally deficient tendon should break, it can be replaced; in effect, the tensioning operation is a performance test of the material. The lower values of allowable stress apply after elastic shortening of the concrete, frictional loss, and anchorage slip have taken place, when service loads may be applied. The steel stress is further reduced during the life of the member due to shrinkage and creep in the concrete and relaxation in the steel.

The strength and other characteristics of prestressing wire, strands, and bars vary somewhat between manufacturers, as do methods of grouping tendons and anchoring them.

2.5 Concrete for Prestressed Construction

Ordinarily, concrete of substantially higher compressive strength is used for prestressed structures than for those constructed of ordinary reinforced concrete. Most prestressed construction in the United States at present is designed for a compressive strength between 5000psi and 6000psi. There are several reasons for this:

1. High-strength concrete normally has a higher modulus of elasticity (see Fig. 2-3). This means a reduction in initial elastic strain under application of prestress force and a reduction in creep strain, which is approximately proportional to elastic strain. This results in a reduction in loss of prestress.

2. In post-tensioned construction, high bearing stresses result at the ends of beams where prestressing force is transferred from the tendons to anchorage fittings, which bear directly against the concrete. This problem can be met by increasing the size of the anchorage fitting or by increasing the bearing capacity of the concrete, by increasing its compressive strength. The latter is usually more economical.

3. In pretensioned construction, where transfer by bond is customary, the use of high-strength concrete will permit the development of higher bond stresses.

4. A substantial part of the prestressed construction in the United States is precast, with the concrete mixed, placed, and cured under carefully controlled conditions that facilitate obtaining higher strengths.

The strain characteristics of concrete under short-time and sustained loads assume an even greater importance in prestressed structures than in reinforced concrete structures because of the influence of strain on loss of prestress force.[10] Strains due to stress, together with volume changes due to shrinkage and temperature changes, may have considerable influence on prestressed structures.

Table 2-1 Permissible stresses in concrete in prestressed flexural members

Concrete in prestressed flexural members	Permissible stresses
1. Stresses in concrete immediately after prestress transfer, before time-dependent prestress losses, shall not exceed the following:	
a. Extreme fiber stress in compression	$0.60f'_{ci}$
b. Extreme fiber stress in tension except as permitted in (c)	$3\sqrt{f'_{ci}}$
c. Extreme fiber stress in tension at ends of simply supported members	$6\sqrt{f'_{ci}}$
Where computed tensile stresses exceed these values, bonded auxiliary reinforcement (nonprestressed or prestressed) shall be provided in the tensile zone to resist the total tensile force in the concrete computed with the assumption of an uncracked section	
2. Stress in concrete at service loads, after allowance for all prestress losses, shall not exceed the following:	
a. Extreme fiber stress in compression	$0.45f'_{c}$
b. Extreme fiber stress in tension in precompressed tensile zone	$6\sqrt{f'_{c}}$
c. Extreme fiber stress in tension in precompressed tensile zone of members, except two-way slab systems, where analysis based on transformed cracked sections and on bilinear moment-deflection relationships shows that immediate and long-time deflections comply with restrictions stated elsewhere in the ACI Code	$12\sqrt{f'_{c}}$
3. Permissible stresses in concrete given above may be exceeded if it is shown by test or analysis that performance will be impaired	

As for prestressing steels, the allowable stresses in the concrete, according to ACI Code 18.4, depend upon the stage of loading. These stresses are given in Table 2-1. Here f'_{ci} is the compressive strength of the concrete at the time of initial pressure, and the f'_{c} the specified compressive strength of the concrete.

New Words and Expressions

plumbing	*n.*	管道工程，卫生工程
riser	*n.*	竖管，井管，溢水管，提升井，提升装置
tendon	*n.*	钢筋，钢筋束
conduit	*n.*	管道，导线管

profile	*n.*	立面图，剖面图，外形，模型
incipient	*adj.*	早期的，刚出现的
factored load		极限设计荷载
jack	*n.*	千斤顶
abutment	*n.*	支座，支墩，拱座
strut	*n.*	支撑，压杆，对角撑
grout	*n.*	浆，水泥浆；*V.* 灌浆
sleeve	*n.*	套筒（管），管接头
bid	*n.*	投标，标书
quotation	*n.*	报价单，估计单，行情，应用
contemplate	*V.*	仔细考虑，沉思，期待

Notes

1. Also, in multistory buildings, any saving in depth of members, multiplied by the number of stories, can represent a substantial saving in total height, load on foundations, length of heating and electrical ducts, plumbing risers, and wall and partition surfaces.

本句难点解析：句子中 multiplied by the number of stories 作为定语修饰 saving，句子的主谓宾分别是 saving，represent，saving，主语 saving 包括 depth of members，宾语则为 total height，load on foundations，length of heating and electrical ducts，plumbing risers，and wall and partition surfaces 并列。

本句大意如下：同样，在多层建筑中，构件高度的减小值乘以总层数，可以代表总高度、基础上的荷载、供热供电管线长度、管道长度、墙以及隔墙面的减小值。

2. Deflection and crack control, achieved through prestressing, permit the engineer to make use of efficient and economical high-strength steels in the form of strands, wires, or bars, in conjunction with concretes of much higher strength than normal.

本句难点解析：句子中 achieved through prestressing 作为定语修饰 deflection and crack control，句子的主体是 Deflection and crack control permit the engineer to make use of steels。

本句大意如下：通过预应力来实现挠度及裂缝控制的技术使得工程师可以采用高效经济的高强钢筋（以钢绞线，钢线或钢筋形式），并可以协同使用高强混凝土。

3. Thus, prestressing results in overall improvement in performance of structural concrete used for ordinary loads and spans and extends the range of application far beyond old limits, leading not only to much longer spans than previously thought possible, but also permitting innovative new structural forms to be employed.

本句难点解析：句子中 leading not only to much longer spans than previously thought possible, but permitting innovative new structural forms to be employed 作为定语修饰 prestressing results。

本句大意如下：预应力的应用，使得普通荷载和跨度下的结构混凝土性能得到整体提高，并扩大了其应用范围，不但可以增加结构的跨度，而且可以创造出新的结构形式。

4. But it would be more logical to apply the prestressing force near the bottom of the beam, to compensate more effectively for the load-induced tension.

本句难点解析：句子中 to compensate more effectively for the load-induced tension 为 to 引导的状语从句，表示目的。

本句大意如下：但是将预应力施加在梁底部会更加合理，这样可以更有效地抵抗荷载所引起的拉力。

5. A possible design specification, for example, might be to introduce the maximum compression at the bottom of the beam without causing tension at the top, when only the prestressing force acts.

本句难点解析：句子中 when only the prestressing force acts 作为状语从句修饰前句，句子主体是 A possible design specification might be to introduce the maximum compression at the bottom of the beam。

本句大意如下：举个例子，一个可能的设计方案是当只有预应力作用时，在梁底部产生最大压力但在顶部不产生拉力。

6. That force, together with the prestressing force acting at the ends of the beam through the tendon anchorages, can be looked upon as a system of external loads.

本句难点解析：句子的主体是 That force can be looked upon as a system of external loads。其中 together with the prestressing force acting at the ends of the beam through the tendon anchorages 作为定语修饰 force。

本句大意如下：与预应力一起通过锚具作用在梁端的力，可以被视为一个外部荷载。

7. In the descriptions of the effects of prestressing in the paragraphs above, it was implied that the prestress force remained constant as the vertical load was introduced, that the concrete responded elastically, and that no concrete cracking occurred.

本句难点解析：句子中三个 that 从句均对应于 it was implied，表示推断的内容。

本句大意如下：在上个段落对预应力作用的描述中，可以推断在竖向荷载作用下预应力仍保持不变，混凝土为弹性反应，并且没有混凝土裂缝产生。

8. Many variations are possible, including replacing the jacks with compression struts after the desired stress in the concrete is obtained or using inexpensive jacks, that remain in place in the structure, in some cases with a cement grout used as the hydraulic fluid.

本句大意如下：有许多不同的做法，包括当混凝土获得预期应力后将千斤顶换成压杆，或者采用不贵的千斤顶，直接将其留在结构中，在某些情况下还可以将水泥浆作为液压体。

9. It is evident, however, that the designer must have some knowledge of the details of the various systems contemplated for use, so that in selecting cross-sectional dimensions, any one of several systems can be accommodated.

本句大意如下：然而，设计人员必须掌握可能使用的几种体系的相关知识，才能在选择截面尺寸时推荐采用相应的体系。

10. The strain characteristics of concrete under short-time and sustained loads assume an even greater importance in prestressed structures than in reinforced concrete structures because of the influence of strain on loss of prestress force.

本句难点解析：句子的主体是 The strain characteristics of concrete assume an even greater importance in prestressed structures than in reinforced concrete structures。其中 under short-time and sustained loads 作为定语修饰 concrete。

本句大意如下：混凝土在短期及长期荷载作用下的应变问题对预应力结构产生的影响要大于钢筋混凝土结构，因为应变损失将引起预应力损失。

Exercises

Translate the following phrases into Chinese.

1. structural engineering
2. longitudinal axial force
3. equivalent load
4. transverse load
5. tendon profile
6. under service condition
7. portable hydraulic jack
8. cross-sectional dimension
9. diagonal bracing
10. extreme fiber stress

Translate the following sentences into Chinese.

1. Modern structural engineering tends to progress toward more economic structures through gradually improved methods of design and the use of higher strength materials.

2. Because no restraint is provided against upward extension of the crack, the beam will collapse without further increase of load.

3. Strength analysis is essential to evaluate safety against collapse, but it tells nothing about cracking or deflections of the beam under service conditions.

4. Some important conclusions can be drawn from these simple examples as follows:

1) Prestressing can control or even eliminate concrete tensile stress for specified loads.

2) Eccentric prestress is usually much more efficient than concentric prestress.

3) Variable eccentricity is usually preferable to constant eccentricity, from the viewpoints of both stress control and deflection control.

5. It may be evident that for any arrangement of applied loads, a tendon profile can be selected so that the equivalent loads acting on the beam from the tendon are just equal and opposite to the applied loads.

Unit 3

Introduction to Steel Structures

3. 1 Structural Design

Structural design may be defined as a mixture of art and science, combining the experienced engineer's intuitive feeling for the behavior of a structure with a sound knowledge of the principles of statics, dynamics, mechanics of materials, and structural analysis, to produce a safe economical structure that will serve its intended purpose.[1]

Until about 1850, structural design was largely an art relying on intuition to determine the size and arrangement of the structural elements. Early man-made structures essentially conformed to those which could also be observed in nature, such as beams and arches. As the principles governing the behavior of structures and structural materials have become better understood, design procedures have become more scientific.

Computations involving scientific principles should serve as a guide to decision making and not be followed blindly . The art or intuitive ability of the experienced engineer is utilized to make the decisions, guided by the computational results.

3. 2 Principles of Design

Design is a process by which an optimum solution is obtained. In this text the concern is with the design of structures-in particular, *steel* structures. In any design, certain criteria must be established to evaluate whether or not an optimum has been achieved. For a structure, typical criteria may be (a) minimum cost; (b) minimum weight; (c) minimum construction time; (d) minimum labor; (e) minimum cost of manufacture of owner's products; and (f) maximum efficiency of operation to owner. Usually several criteria are involved, each of which may require weighting. Observing the above possible criteria, it may be apparent that setting clearly measurable criteria (such as weight and cost) for establishing an optimum frequently will be difficult, and perhaps impossible. In most practical situations, the evaluation must be qualitative.

If a specific objective criterion can be expressed mathematically, then optimization techniques may be employed to obtain a maximum or minimum for the objective function. Optimization procedures and techniques comprise an entire subject that is outside the scope of this text . The criterion of minimum weight is emphasized throughout, under the general assumption that minimum material represents minimum cost. Other subjective criteria must be kept in mind, even though the integration of behavioral principles with design of structural steel elements in this text utilizes only simple objective criteria, such as weight or cost.

The design procedure may be considered to be composed of two parts—functional design and structural framework design. Functional design ensures that intended results are achieved, such as (a) adequate working areas and clearances; (b) proper ventilation and/or air conditioning; (c) adequate transportation facilities, such as elevators, stairways, and cranes or materials handling equipment; (d) adequate lighting; and (e) aesthetics.

The structural framework design is the selection of arrangement and sizes of structural elements so that service loads may be safely carried, and displacements are within acceptable limits.

The iterative design procedure may be outlined as follows:

1. Planning. Establishment of the functions for which the structure must serve. Set criteria against which to measure the resulting design for being an optimum.

2. Preliminary structural configuration. Arrangement of the elements to serve the functions in step 1.

3. Establishment of the loads to be carried.

4. Preliminary member selection. Based on the decisions of steps 1, 2, and 3 selection of the member sizes to satisfy an objective criterion, such as least weight or cost.

5. Analysis. Structural analysis involving modeling the loads and the structural framework to obtain internal forces and any desired deflections.

6. Evaluation. Are all strength and serviceability requirements satisfied and is the result optimum? Compare the result with predetermined criteria.

7. Redesign. Repetition of any part of the sequence 1 through 6 found necessary or desirable as a result of evaluation. Steps 1 through 6 represent an iterative process. Usually in this text only steps 3 through 6 will be subject to this iteration since the structural configuration and external loading will be prescribed.

8. Final decision. The determination of whether or not an optimum design has been achieved.

■ 3.3 Historical Background of Steel Structures

Metal as a structural material began with cast iron, used on a 100ft (about. 30m) arch span which was built in England in 1777—1779. A number of cast-iron bridges were built during the period 1780—1820, mostly arch-shaped with main girders consisting of individual cast-iron pieces forming bars or trusses. Cast iron was also used for chain links on suspension bridges until about 1840.

Wrought iron began replacing cast iron soon after 1840, the earliest important example being the Brittania Bridge over Menai Straits in Wales, which was built in 1846—1850. This was a tubular girder bridge having spans 230-460-460-230 ft (about 70-140-140-70m), which was made from wrought-iron plates and angles.

The process of rolling various shapes was developing as cast iron and wrought iron received wider usage. Bars were rolled on an industrial scale beginning about 1780. The rolling of rails began about 1820 and was extended to I-shapes by the 1870s.

The development of the Bessemer process (1855), the introduction of a basic liner in the Bessemer converter (1870), and the open-hearth furnace brought widespread use of iron ore products in building materials. Since 1890, steel has replaced wrought iron as the principal metallic building material. Currently (1989), steels having yield stresses varying from 24000 to 100000 pounds per square inch, psi (165 to 690 megapascals, MPa), and available for structural uses.

■ 3.4 Loads

The accurate determination of the loads to which a structure or structural element will be subjected is not always predictable.[2] Even if the loads are well known at one location in a structure, the distribution of load from element to element throughout the structure usually requires assumptions and approximations. Some of the most common kinds of loads are discussed in the following sections.

3.4.1 Dead Load

Dead load is a fixed-position gravity service load, so called because it acts continuously toward the earth when the structure is in service.[3] The weight of the structure is considered dead load, as well as attachments to the structure such as pipes, electrical conduit, air-conditioning and heating ducts, lighting fixtures, floor covering, roof covering, and suspended ceilings; that is, all items that remain throughout the life of the structure.

Dead loads are usually known accurately but not until the design has been completed. Under steps 3 through 6 of the design procedure, the weight of the structure or structural element must be estimated, preliminary section selected, weight recomputed, and member selection revised if necessary. The dead load of attachments is usually known with reasonable accuracy prior to the design.

3.4.2 Live Load

Gravity loads acting when the structure is in service, but varying in magnitude and location, are termed live loads.[4] Examples of live loads are human occupants, furniture, movable equipment, vehicles, and stored goods. Some live loads may be practically permanent, others may be highly transient. Because of the unknown nature of the magnitude, location, and density of live load items, realistic magnitudes and the positions of such loads are very difficult to determine.

Because of the public concern for adequate safety, live loads to be taken as service loads in design are usually prescribed by state and local building codes. These loads are generally empirical and conservative, based on experience and accepted practice rather than accurately computed values. Wherever local codes do not apply, or do not exist, the provisions from one of several regional and national building codes may be used. One such widely recognized code is the *American national Standard Minimum Design Loads for Buildings and Other Structures* ANSI A58. 1 of the American National Standards Institute (ANSI), from which some typical live loads are presented. The code will henceforth be referred to as the ANSI Standard. This standard is updated from time to time, most recently in 1982.

Live load when applied to a structure should be positioned to give the maximum effect, including partial loading, alternate span loading, or full span loading as may be necessary.[5] The simplified assumption of full uniform loading everywhere should be used only when it agrees with reality or is an appropriate approximation. The probability of having the prescribed loading applied uniformly over an entire floor, or over all floors of a building simultaneously, is almost nonexistent. Most codes recognize this by allowing for some percentage reduction from full loading. For instance, for live loads of 100 psf or more ANSI Standard allows members having an influence area of 400 ft^2 or more to be designed for a reduced live load according to Eq. 3-1, as follows:

$$L=L_0\left(0.25+\frac{15}{\sqrt{A_{\mathrm{I}}}}\right) \tag{3-1}$$

where L—reduced live load per sq ft of area supported by the member;

L_0—unreduced live load per sq ft of area supported by the member (from Table 3-1);

A_{I}—influence area (ft^2)。

Table 3-1 Typical minimum uniformly distributed live loads

Occupancy or use	Live load	
	psf	Pa①
1.Hotel guest rooms	40	1900
School classrooms		
Private apartments		
Hospital private rooms		
2.Offices	50	2400
3.Assembly halls, fixed seat	60	2900
Library reading rooms		
4.Corridors, above first floor in schools.libraries and hospitals	80	3800
5.Assembly areas: theater lobbies	100	4800
Dining rooms and restaurants		
Office building lobbies		
Main floor retails stores		
Assembly hall movable seats		
6.Wholesale stores, all floors	125	6000
Manufacturing light		
Storage warehouses, light		
7.Armories and drill halls	150	7200

(Continued)

Occupancy or use	Live load	
	psf	Pa①
Stage floors		
Library stack rooms		
8.Manufacturing, heavy	250	12000
Sidewalks and driveways subject to trucking		
Storage warehouse, heavy		

① SI values are approximate conversions. 1psf (1b/ft^2) = 47.9Pa.

The influence area A_I is four times the tributary area (the area which distributes load to member being considered) for a column, two times the tributary area for a beam, and is equal to the panel area for a two-way slab. The reduced live load L shall not be less than 50% of the live load L_0 for members supporting one floor, nor less than 40% of the live load L_0 otherwise.

The live load reduction referred to above is not permitted in areas to be occupied as places of public assembly and for one-way slabs, when the live load L is 100 psf or less. Reductions are permitted for occupancies where L_0 is greater than100 psf and for garages and roofs only under special circumstances (ANSI-4.7.2).

3.4.3 Highway Live Loads

Highway vehicle loading in the United States has been standardized by the American Association of State Highway and Transportation Officials (AASHTO) into standard truck loads and lane loads that approximate a series of trucks. There are two systems, designated H and HS, that are identified by the number of axles per truck. The H system has two axles, whereas the HS system has three axles per truck. There are several classes of loading; however, the usual ones are known as H20 and HS20, shown in Fig. 3-1.

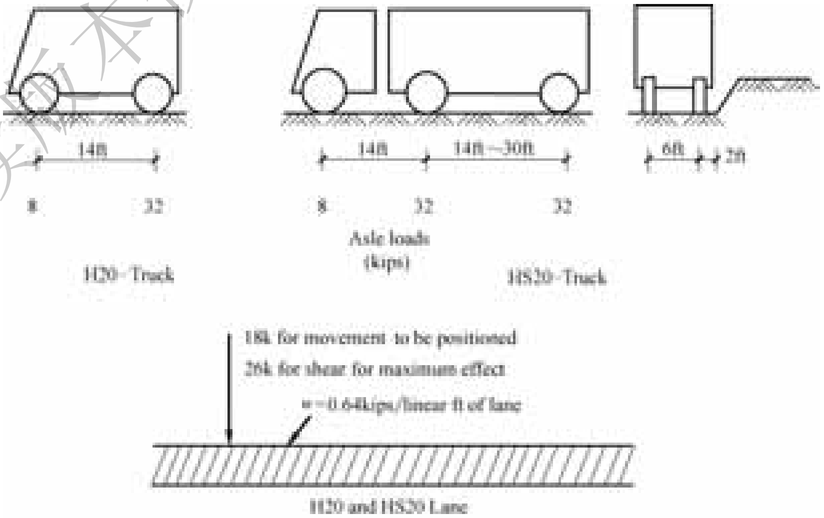


Fig. 3-1 AASHTO highway H20 and HS20 loading.

Note: 1kip = 4.45kN.

In designing a given bridge, either one truck loading is applied to the entire structure, or the lane loading is applied. When the lane loading is used, the uniform portion is distributed over as much of the span or spans as will cause the maximum effect. In addition, the one concentrated load (for maximum negative moment on continuous spans, a second concentrated load is also used) is positioned for the most severe loading effect. The load distribution across the width of a bridge to its various supporting members is taken in accordance with semiempirical rules that depend on the type of bridge deck and supporting structure.

The single truck loading provides the effect of a heavy concentrated load and usually governs on relatively short spans. The uniform lane load is to simulate a line of traffic, and the added concentrated load is to account for the possibility of one extra heavy vehicle in the line of traffic. These loads have been used with no apparent difficulty since 1944, before which time a line of trucks was actually used for the loading. On the interstate system of highways, a military loading is also used that consists of two 24kip (about 107kN) axle loads spaced 4ft (about 1.2m) apart.

Railroad bridges are designed to carry a similar semiempirical loading known as the Cooper E72 train, consisting of a series of concentrated loads a fixed distance apart followed by uniform loading. This loading is prescribed by the American Railway Engineering Association (AREA).

3.4.4 Impact

The term impact as ordinarily used in structural design refers to the dynamic effect of a suddenly applied load. In the building of a structure, the materials are added slowly; people entering a building are also considered a gradual loading. Dead loads are static loads; i.e., they have no effect other than weight. Live loads may be either static or they may have a dynamic effect. Persons and furniture would be treated as static live load, but cranes and various types of machinery also have dynamic effects.

Consider the spring-mass system of Fig. 3-2a, where the spring may be thought of as analogous to an elastic beam. When load is gradually applied (i.e., static loading) the mass (weight) deflects an amount x and the load on the spring (beam) is equal to the weight W. In Fig. 3-2b the load is suddenly applied (dynamic loading), and the maximum deflection is $2x$; i.e., the maximum load on the spring (beam) is $2W$. In this case the mass vibrates in simple harmonic motion with its neutral position equal to its static deflected position. In real structures, the harmonic (vibratory) motion is damped out (reduced to zero) very rapidly. Once the motion has stopped, the force remaining in the spring is the weight W. To account for the increased force during the time the member is in motion, a load equal to twice the static load W should be used—add 100% of the static load to represent the dynamic effect. This is called a 100% impact factor.

Any live load that can have a dynamic effect should be increased by an impact factor. While a dynamic analysis of a structure could be made, such a procedure is unnecessary in ordinary design. Thus empirical formulas and impact factors are usually used. In cases where the dynamic effect is small (say where impact would be less than about 20%), it is ordinarily accounted for by using a conservative (higher) value for the specified live load. The dynamic effects of persons in buildings

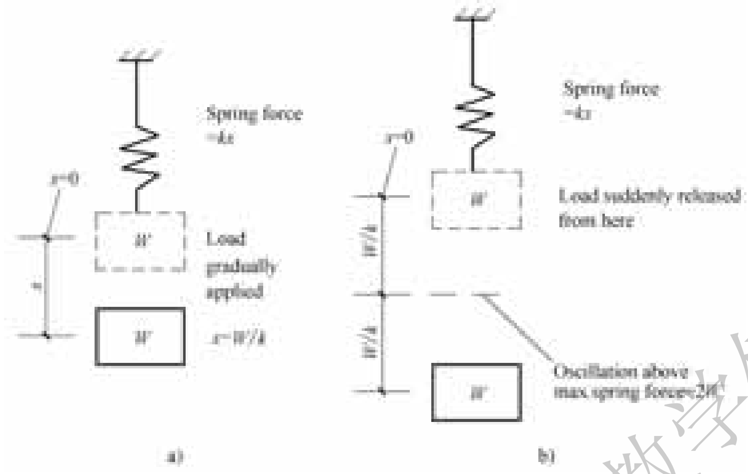


Fig. 3-2 Comparison of static and dynamic loading.

a) No vibration max. spring force = W b) Free vibration max. Spring force = $2W$

and of slow-moving vehicles in parking garages are examples where ordinary design live load is conservative, and usually no explicit impact factor is added.

For highway bridge design, however, impact is always to be considered. AASHTO prescribes empirically that the impact factor expressed as a portion of live load is

$$I = 50/(L+125) \leqslant 0.30 \qquad (3\text{-}2)$$

In Eq. 3-2, L (expressed in feet) is the length of the portion of the span that is loaded to give the maximum effect on the member. Since vehicles travel directly on the superstructure, all parts of it are subjected to vibration and must be designed to include impact. The substructure, including all portions not rigidly attached to the superstructure such as abutments, retaining walls, and piers, are assumed to have adequate damping or be sufficiently remote from the application point of the dynamic load so that impact might not be considered. Again, conservative static loads may account for the smaller dynamic effects.

In buildings, it is principally in the design of supports for cranes and heavy machinery that impact is explicitly considered. The American Institute of Steel Construction (AISC) Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) Specifications (ASD and LRFD-A4. 2) state that if not otherwise specified, the impact percentage shall be:

For supports of elevators and elevator machinery, 100%.

For supports of light machinery, shaft or motor driven, not less than, 20%.

For supports of reciprocating machinery or power driven units, not less than, 50%.

For hangers supporting floors and balconies, 33%.

For cab-operated traveling crane support girders and their connections, 25%.

For pendant-operated traveling crane support girders and their connections, 10%.

In the design of crane runway beams and their connections (Fig. 3-3), the horizontal forces caused by moving crane trolleys must be considered. Both LFRD and ASD-A4. 3 prescribe using a minimum of "20% of the sum of the lifted load and the crane trolley (but exclusive of other parts of the crane). The force shall be assumed to be applied at the top of the rails, acting in either direction normal to the runway rails, and shall be distributed with due regard for lateral stiffness of the structure supporting the rails."

In addition, due to acceleration and deceleration of the entire crane, a longitudinal tractive force is transmitted to the runway girder through friction of the end truck wheels with the crane rail.[6] LRFD and ASD-A 4. 3 require this force, unless otherwise specified, to be a minimum of 10% of the maximum wheel loads of the crane applied at the top of the rail.

The reader will find continued reference to the AISC Specifications (ASD and LRFD) which are contained, respectively in the *AISC ASD Manual* and *AISC LRFD Manual*. These two books may be purchased from AISC, 400 North Michigan Avenue, Chicago, IL 60611-4185.

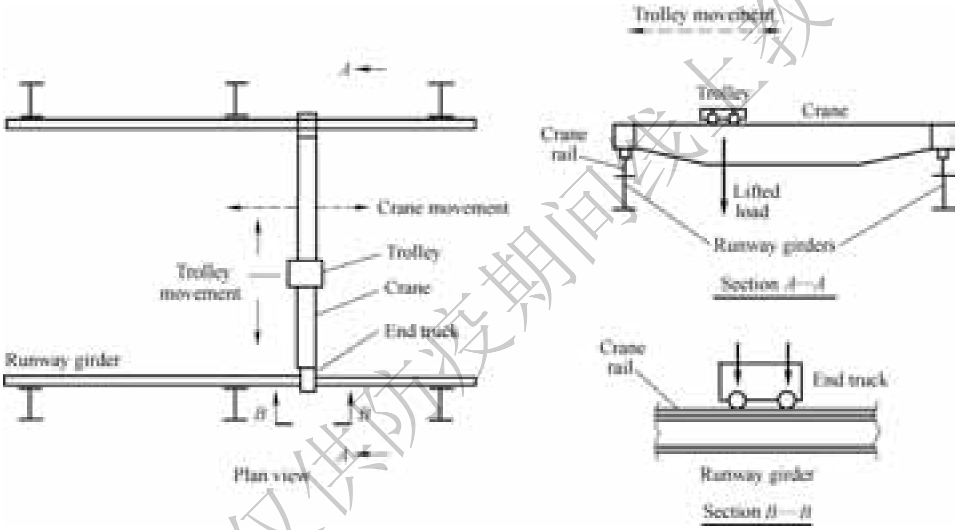


Fig. 3-3 Crane arrangement showing movements that contribute impact loading.

3. 4. 5 Snow Load

The live loading for which roofs are designed is either totally or primarily a snow load. Since snow has a variable specific gravity, even if one knows the depth of snow for which design is to be made, the load per unit area of roof is at best only a guess.

The best procedure for establishing snow load for design is to follow the ANSI Standard. This Code uses a map of the United States giving isolines of ground snow corresponding to a 50-year mean recurrence interval for use in designing most permanent structures. The ground snow is then multiplied by a coefficient that includes the effect of roof slope, wind exposure, nonuniform accumulation on pitched or curved roofs, multiple series roofs, and multilevel roofs and roof areas adjacent to projections on a roof level.

It is apparent that the steeper the roof is, the less snow can accumulate. Also, partial snow loading must be considered in addition to full loading, if it is believed such loading can occur and would cause maximum effects. Wind may also act on a structure that is carrying snow load. It is unlikely, however, that maximum snow and wind loads would act simultaneously.

In general, the basic snow load used in design varies from 30 to 40psf (1400 to 1900MPa) in the northern and eastern states to 20psf (960MPa) or less in the southern states. Flat roofs in normally warm climates should be designed for 20psf (960MPa) even when such accumulation of snow may seem doubtful. This loading may be thought of as due to people gathered on such a roof. Furthermore, though wind is frequently ignored as a vertical force on a roof, nevertheless it may cause such an effect. For these reasons, a 20 psf (960MPa) minimum loading, even though it may not always be snow, is reasonable. Local codes, actual weather conditions, ANSI, or the *Canadian Structural Design Manual*, should be used when designing for snow.

Other snow load information has been provided in the *Building Structural Design Handbook* and related studies.

3.4.6 Wind Load

All structures are subject to wind load, but it is usually only those more than three or four stories high, other than long bridges, for which special consideration of wind is required.[7]

On any typical building of rectangular plan and elevation, wind exerts pressure on the windward side and suction on the leeward side, as well as either uplift or downward pressure on the roof. For most ordinary situations, vertical roof loading from wind is neglected on the assumption that snow loading will require a greater strength than wind loading. This assumption is not true for southern climates where the vertical loading due to wind must be included. Furthermore, the total lateral wind load, windward and leeward effect, is commonly assumed to be applied to the windward face of the building.

In accordance with Bernoulli's theorem for an ideal fluid striking an object, the increase in static pressure equals the decrease in dynamic pressure, or

$$q=\frac{1}{2}\rho V^2 \tag{3-3}$$

Where q is the dynamic pressure on the object, ρ is the mass density of air [specific weight $w=0.07651$ pcf (1pcf = 16.02kg/m^3) at sea level and 15℃], and V is the wind velocity. In terms of velocity V in miles per hour, the dynamic pressure q (psf) would be

$$q=\frac{1}{2}\left(\frac{0.07651}{32.2}\right)\left(\frac{5280V^2}{3600}\right)=0.0026V^2 \tag{3-4}$$

In design of usual types of buildings, the dynamic pressure q is commonly converted into equivalent static pressure p, which may be expressed

$$p=qC_eC_gC_p \tag{3-5}$$

Where C_e is an exposure factor that varies from 1.0 (for 0 ~ 40ft height) to 2.0 (for 740 ~ 1200ft height); C_g is a gust factor, such as 2.0 for structural members and 2.5 for small elements including cladding; and C_p is a shape factor for the building as a whole. Excellent details of application of wind loading to structures are available in the ANSI Standard and in the *National Building Code of*

Canada.

The commonly used wind pressure of 20 psf, as specified by many building codes, corresponds to a velocity of 88 miles per hour (mph) from Eq. 3-4. An exposure factor C_e of 1.0, a gust factor C_g of 2.0, and a shape factor C_p of 1.3 for an airtight building, along with a 20 psf equivalent static pressure p, will give from Eq. 3-5 a dynamic pressure q of 7.7 psf, which corresponds using Eq. 3-4, to a wind velocity of 55 mph. For all buildings having nonplanar surfaces, plane surfaces inclined to the wind direction, or surfaces having significant openings, special determination of the wind forces should be made using such sources as the ANSI Standard, or the *National Building Code of Canada*. For more extensive treatment of wind loads, the reader is referred to the Task Committee on Wind Forces.

3.4.7 Earthquake Load

An earthquake consists of horizontal and vertical ground motions, with the vertical motion usually having much the smaller magnitude.[8] Because the horizontal motion of the ground causes the most significant effect, it is that effect which is usually thought of as earthquake load. When the ground under an object (structure) having a certain mass suddenly moves, the inertia of the mass tends to resist the movement, as shown in Fig. 3-4. A shear force is developed between the ground and the mass. Most building codes having earthquake provisions require the designer to consider a lateral force CW that is usually empirically prescribed. The dynamics of earthquake action on structures is outside the scope of this text, and the reader is referred to Chopra, Clough and Penzien.

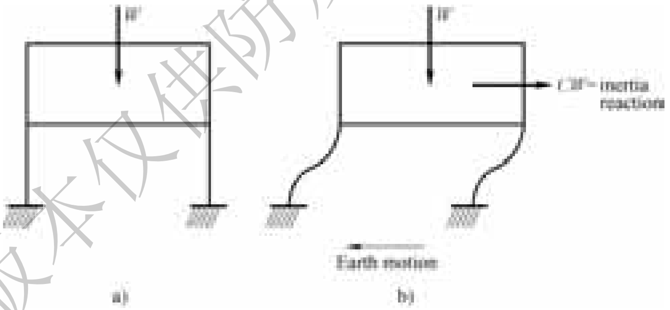


Fig. 3-4 Force developed by earthquake.

a) At test b) Under horizontal motion from earthquake

In order to simplify the design process, most building codes contain an equivalent lateral force procedure for designing to resist earthquakes. One of the most widely used design recommendations is that of the Structural Engineers Association of California (SEAOC), the latest version of which is 1974. Since that time, the Applied Technology Council (ATC) prepared a set of design provisions. Some recent rules for the equivalent lateral force procedure are those given by the ANSI Standard. In ANSI the lateral seismic force V, expressed as follows, are assumed to act nonconcurrently in the direction of each of the main axes of the structure.

$$V = ZIKCSW \tag{3-6}$$

Where Z—seismic zone coefficient, varying from 1/8 for the zone of lowest seismicity, to 1 for the zone of highest seismicity;

I—occupancy importance factor, varying from 1.5 for buildings designated as "essential facilities", 1.25 for buildings where the primary occupancy is for assembly for greater than 300 persons, to 1.0 for usual buildings;

K—horizontal force factor, varying from 0.67 to 2.5, indicating capacity of the structure to absorb plastic deformation (low values indicate high ductility);

$$C=\frac{1}{15\sqrt{T}}\leqslant 0.12 \tag{3-7}$$

C—the seismic coefficient, equivalent to the maximum acceleration in term of acceleration due to gravity;

T—fundamental natural period, i.e., time for one cycle of vibration, of the building in the direction of motion;

S—soil profile coefficient, varying from 1.0 for rock to 1.5 for soft to medium-stiff clays and sands;

W—total dead load of the building, including interior partitions.

When the natural period T cannot be determined by a rational means from technical data, it may be obtained as follows for shear walls or exterior concrete frames utilizing deep beams or or wide piers, or both:

$$C=\frac{0.05h_{\mathrm{n}}}{\sqrt{D}} \tag{3-8}$$

where D is the dimension of the structure in the direction of the applied forces, in feet, and h_{n} is the height of the building.

Once the base shear V has been determined, the lateral force must be distributed over the height of the building.

More details of the ANSI Standard procedure are available in the *Building Structural Design Handbook*. Various building code formulas for earthquake-resistant design are compared by Chopra and Cruz. Many states have adopted the *Uniform Building Code* (UBC), the most recent version of which is 1985, which contains provisions for design to resist earthquakes generally based on the ANSI Standard.

3.5 Types of Structural Steel Members

The function of a structure is the principal factor determining the structural configuration. Using the structural configuration along with the design loads, individual components are selected to properly support and transmit loads throughout the structure. Steel members are selected from among the standard rolled shapes adopted by the American Institute of Steel Construction (AISC) (also given by American Society for Testing and Materials [ASTM] A6 Specification). Of course, welding permits combining plates and/or other rolled shapes to obtain any shape the designer may require.

Typical rolled shapes, the dimensions for which are found in the AISC Manual, are shown in Fig. 3-5. The most commonly used section is the wide-flange shape (Fig. 3-5a) which is formed by hot rolling in the steel mill. The wide-flange shape is designated by the nominal depth and the weight per foot, such as a W18×97 which is nominally 18 in. deep (actual depth = 18.59 in. according to AISC Manual) and weighs 97 pounds per foot (In SI units the W18×97 section could be designated W460×142, meaning nominally 460mm deep and having a mass of 142kg/m). Two sets of dimensions are found in the AISC Manual, one set stated in decimals for the designer to use in computations, and another set expressed in fractions (1/16 in. as the smallest increment) for the detailer to use on plans and shop drawings. Rolled W shapes are also designated by ANSI/ASTM A6 in accordance with web thickness as Groups I through V, with the thinnest web sections in Group 1.

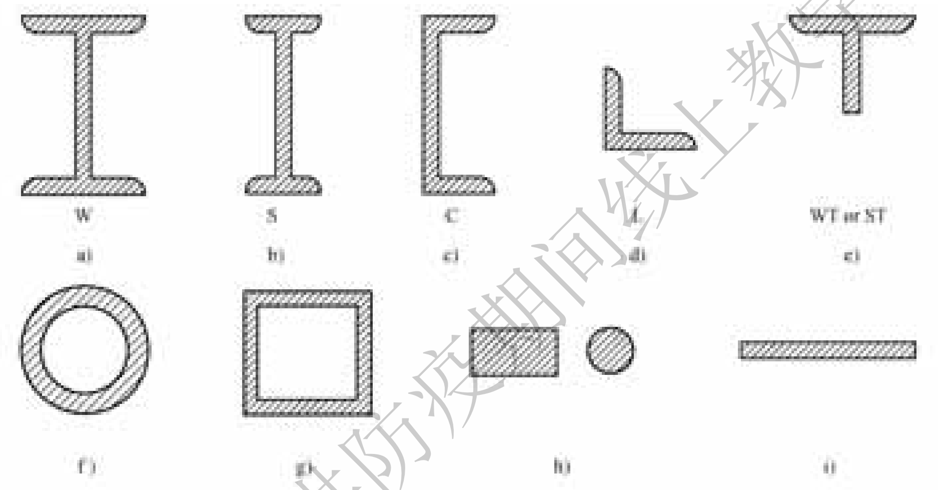


Fig. 3-5 Standard rolled shapes.

a) Wide-flange shape b) American standard beam c) American standard channel d) Angle
e) Structural tee f) Pipe section g) Structural turbing h) Bars i) Plates

The American Standard beam (Fig. 3-5b), commonly called the I-beam, has relatively narrow and sloping flanges and a thick web compared to the wide-flange shape. Use of most I-beams has become relatively uncommon because of excessive material in the web and relative lack of lateral stiffness due to the narrow flanges.

The channel (Fig. 3-5c) and angle (Fig. 3-5d) are commonly used either alone or in combination with other sections. The channel is designated, for example, as C12×20.7, a nominal 12in. deep channel having a weight of 20.7 pounds per foot. Angles are designated by their leg length (long leg first) and thickness, such as, L6×4×3/8.

The structural tee (Fig. 3-5e) is made by cutting wide-flange or I-beams in half and is commonly used for chord members in trusses. The tee is designated, for example, as WT5×44, where the 5 is the nominal depth and 44 is the weight in pounds per foot; this tee being cut from a W10×88.

Pipe sections (Fig. 3-5f) are designated "standard" "extra strong" and "double-extra strong" in accordance with the thickness and are also nominally prescribed by diameter; thus 10in.-diam double-extra strong is an example of a particular pipe size.

Structural tubing (Fig. 3-5g) is used where pleasing architectural appearance is desired with exposed steel. Tubing is designated by outside dimensions and thickness, such as structural tubing, 8×6×1/4.

The sections shown in Fig. 3-5 are all hot-rolled; that is, they are formed from hot billet steel (blocks of steel) by passing through rolls numerous times to obtain the final shapes.

Many other shapes are cold-formed from plate material having a thickness not exceeding 1 in., as shown in Fig. 3-6.

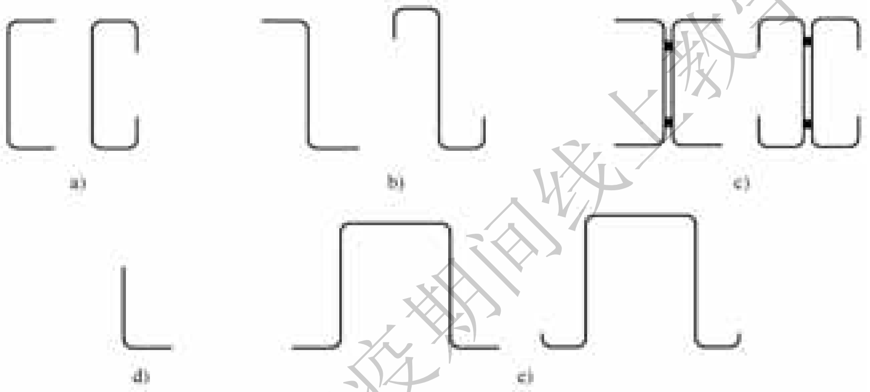


Fig. 3-6 Some cold-formed shapes.

a) Channels b) Zees c) I-shaped double channels d) Angle e) Hat sections

Regarding size and designation of cold-formed steel members, there are no truly standard shapes even though the properties of many common shapes are given in the *Cold-Formed Steel Design Manual*. Various manufacturers produce many proprietary shapes.

3.5.1 Tension Members

The tension member occurs commonly as a chord member in a truss, as diagonal bracing in many types of structures, as direct support for balconies, as cables in suspended roof systems, and as suspension bridge main cables and suspenders that support the roadway. Typical cross-sections of tension members are shown in Fig. 3-7.

3.5.2 Compression Members

Since compression member strength is a function of the cross-sectional shape (radius of gyration), the area is generally spread out as much as is practical. Chord members in trusses, and many interior columns in buildings are examples of members subject to axial compression. Even under the most ideal condition, pure axial compression is not attainable; so, design for "axial" loading assumes the effect of any small simultaneous bending may be neglected. Typical cross-sections of com-

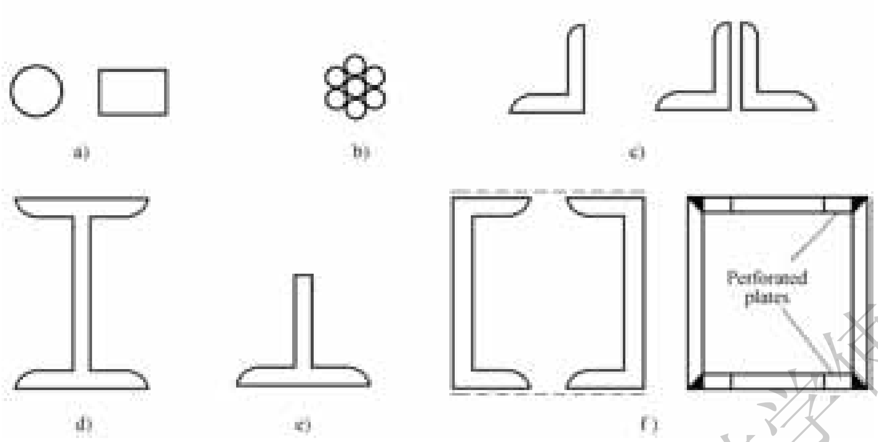


Fig. 3-7 Typical tension members.

a) Round and rectangular bars, including eye bars and upset bars b) Cables composed of many small wires
c) Single and double angles d) Rolled W-and S-sections e) Structural tee f) Built-up box sections

pression members are shown in Fig. 3-8.

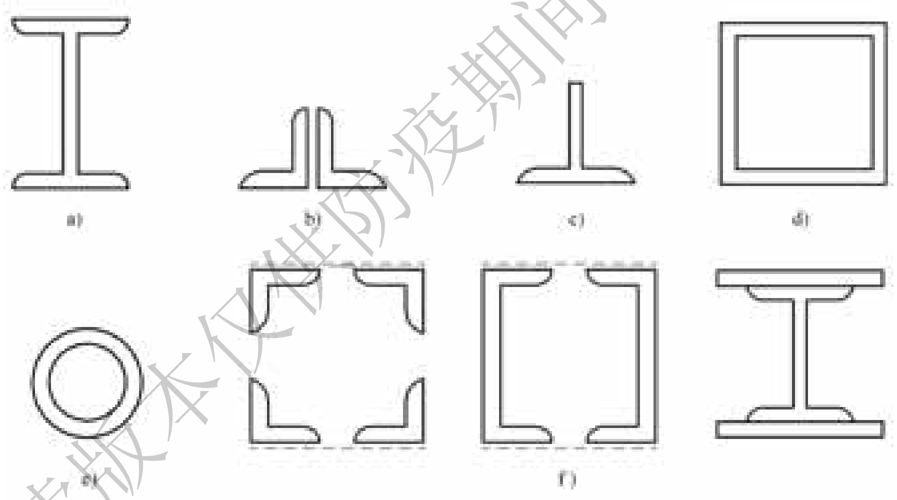


Fig. 3-8 Typical Compression members.

a) Rooled W-and S-shapes b) Double angle c) Structural tee
d) Structural tubing e) Pipe section f) Built-up sections

3.5.3 Beams

Beams are members subjected to transverse loading and are most efficient when their area is distributed so as to be located at the greatest practical distance from the neutral axis.[9] The most common beam sections are the wide-flange (W) and I-beams (S) (Fig. 3-9a), as well as smaller rolled I-shaped sections designated as "miscellaneous shapes" (M).

For deeper and thinner-webbed sections than can economically be rolled, welded I-shaped sections (Fig. 3-9b) are used, including stiffened plate girders.

For moderate spans carrying light loads, open-web "joists" are often used (Fig. 3-9c). These are parallel chord truss-type members used for the support of floors and roofs. The steel may be hot-rolled or cold-formed. Such joists are designated "K-Series" "LH-Series" and "DLH-Series" . The K-Series is suitable for members having the direct support of floors and roof decks in buildings. The LH-Series and DLH-Series are known as Longspan and Deep Longspan, respectively. Longspan Steel Joists are shop-fabricated trusses used "for the direct support of floor or roof slabs or decks between walls, beams, and main structural members". Deep Longspan Joists are used " for the direct support of roof slabs or decks between wall, beams and main structural members". The design of the chords for K-Series trusses is based on a yield strength of 50ksi (about 345MPa), while the web sections may use either 36ksi (about 248MPa) or 50ksi (about 345MPa). For the LH-and DLH-Series the chord and web sections design must be based on a yield strength of at least 36 ksi (about 248MPa) but not greater than 50ksi (about 345MPa).

The K-Series joists have depths from 8 to 30 in. for clear spans to 60 ft. The Longspan joists (LH-Series) have depths from 18 to 48 in. for clear spans to 96 ft. The Deep Longspan joists (DLH-Series) have depths from 52 to 72 in. for clear spans to 144 ft.

All of these joists are designed according to Specifications adopted by the Steel Joist Institute (SJI), which generally are in agreement with the AISC Specifications for hot-rolled steels and AISI Specifications for cold-formed steels.

For beams (known as lintels) carrying loads across window and door openings, angles (Fig. 3-9d) are frequently used; and for beams (known as girts) in wall panels, channels (Fig. 3-9e) are frequently used.

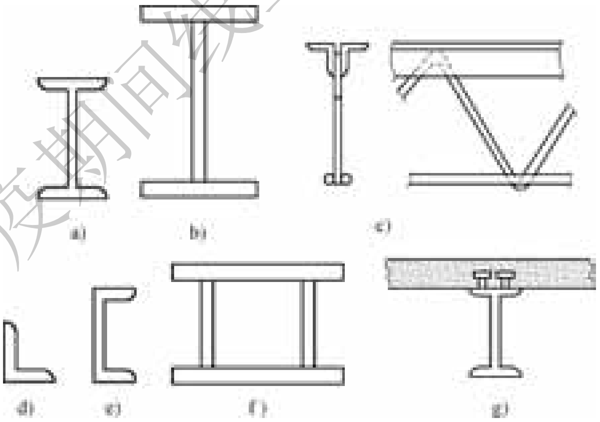


Fig. 3-9 Typical beam members.

a) Rooled W-and other I-shaped sections b) Welded I-shape (plate girder) c) Open web joists d) Angle e) Channel f) Built-up members g) Composite steel-concrete

3.5.4 Bending and Axial Load

When simultaneous action of tension or compression along with bending occurs, a combined stress problem arises and the type of member used will be dependent on the type of stress that predominates.[10]

The aforementioned illustration of types of members to resist various kinds of stress is intended only to show common and representative types of members and not to be all inclusive.

New Words and Expressions

ventilation	*n.*	通风换气设备
cast iron		铸铁
wrought iron		熟铁，锻铁
Bessemer process		酸性转炉法
converter	*n.*	炼钢炉，吹风转炉
tractive	*adj.*	牵引的
isoline	*n.*	等值线，等位线
lee	*n.*	背风处，下风
windward	*n.*	迎风面
partition	*n.*	隔墙，分割
radius of gyration		回转半径
miscellaneous	*adj.*	混杂的，多方面的，有各种特点的
channel	*n.*	槽钢，槽，沟
lintel	*n.*	过梁

Notes

1. Structural design may be defined as a mixture of art and science, combining the experienced engineer's intuitive feeling for the behavior of a structure with a sound knowledge of the principles of statics, dynamics, mechanics of materials, and structural analysis, to produce a safe economical structure that will serve its intended purpose.

本句难点解析：句子中 combining... analysis 作为定语修饰 structural design，to produce a safe economical structure that will serve its intended purpose 则为状语从句，表示目的。

本句大意如下：结构设计可以被定义为一种艺术与科学的结合体，它将有经验的工程师对建筑性能的直觉，与静力、动力、材料力学以及结构分析的相关知识联合起来，产生了一个可以实现其既定目标的安全经济的结构。

2. The accurate determination of the loads to which a structure or structural element will be subjected is not always predictable.

本句难点解析：句子的主体是 The accurate determination of the loads is not always predictable。其中 to which a structure or structural element will be subjected 作为定语从句修饰 loads。

本句大意如下：结构或者结构单元的受力通常是不可精确预测的。

3. Dead load is a fixed-position gravity service load, so called because it acts continuously toward the earth when the structure is in service.

本句难点解析：句子中 so called 表示“之所以这么称呼”，起转折连接作用。

本句大意如下：恒荷载是一个固定位置的重力荷载，之所以这么称呼，是因为在结构使用阶段其作用方向一直指向地面。

4. Gravity loads acting when the structure is in service, but varying in magnitude and location, are termed live loads.

本句难点解析：句子的主体是 Gravity loads are termed live loads。其中 acting when the structure is in service, but varying in magnitude and location 作为定语修饰限定 gravity loads，表明其范围。

本句大意如下：在结构使用期间作用于其上，但大小以及位置变化的重力荷载，称为活荷载。

5. Live load when applied to a structure should be positioned to give the maximum effect, including partial loading, alternate span loading, or full span loading as may be necessary.

本句难点解析：句子的主体是 Live load should be positioned to give the maximum effect。

本句大意如下：当活荷载施加于结构时，应该将其放置在产生最大效应的位置上，包括部分加载、分跨加载或者全跨加载（若有需要）。

6. In addition, due to acceleration and deceleration of the entire crane, a longitudinal tractive force is transmitted to the runway girder through friction of the end truck wheels with the crane rail.

本句难点解析：其中 through 作为介词表明了牵引力的传导方式。

本句大意如下：此外，由于整个起重机的加速以及减速，纵向的牵引力将通过末端车轮与起重机轨道之间的摩擦力进行传递。

7. All structures are subject to wind load, but it is usually only those more than three or four stories high, other than long bridges, for which special consideration of wind is required.

本句难点解析：句子中 for which special consideration of wind is required. 作为定语从句修饰前句。

本句大意如下：所有的结构均承受风荷载，但除了长大桥以外，通常仅有高于三四层的建筑需要对风荷载进行专门考虑。

8. An earthquake consists of horizontal and vertical ground motions, with the vertical motion usually having much the smaller magnitude.

本句难点解析：句子中 with the vertical motion usually having much the smaller magnitude 为伴随状语。进一步解释说明竖向运动的情况。

本句大意如下：地震包含水平以及竖向运动，其中竖向运动幅度较小。

9. Beams are members subjected to transverse loading and are most efficient when their area is distributed so as to be located at the greatest practical distance from the neutral axis.

本句难点解析：句子中 subjected to transverse loading 作为定语修饰 members。so... axis 则是对 distributed 的进一步说明。

本句大意如下：梁是用于承受横向荷载的构件，且当它们面积分散，位于离轴心处实际距离最远的位置时最高效。

10. When simultaneous action of tension or compression along with bending occurs, a combined stress problem arises and the type of member used will be dependent on the type of stress that predominates.

本句难点解析：句子中 along with 表示同时发生，即拉、压、弯同时作用。

本句大意如下：当同时作用拉、压、弯时，将产生复合应力问题，此时构件类型的选取

将取决于哪种应力起主要作用。

Exercises

Translate the following phrases into Chinese.

1. structural design
2. computations involving scientific principles
3. measurable criteria
4. adequate transportation facilities
5. preliminary structural configuration
6. yield stress
7. require assumptions and approximations
8. fixed-position gravity service load
9. ANSI Standard
10. maximum negative moment

Translate the following Sentences into Chinese.

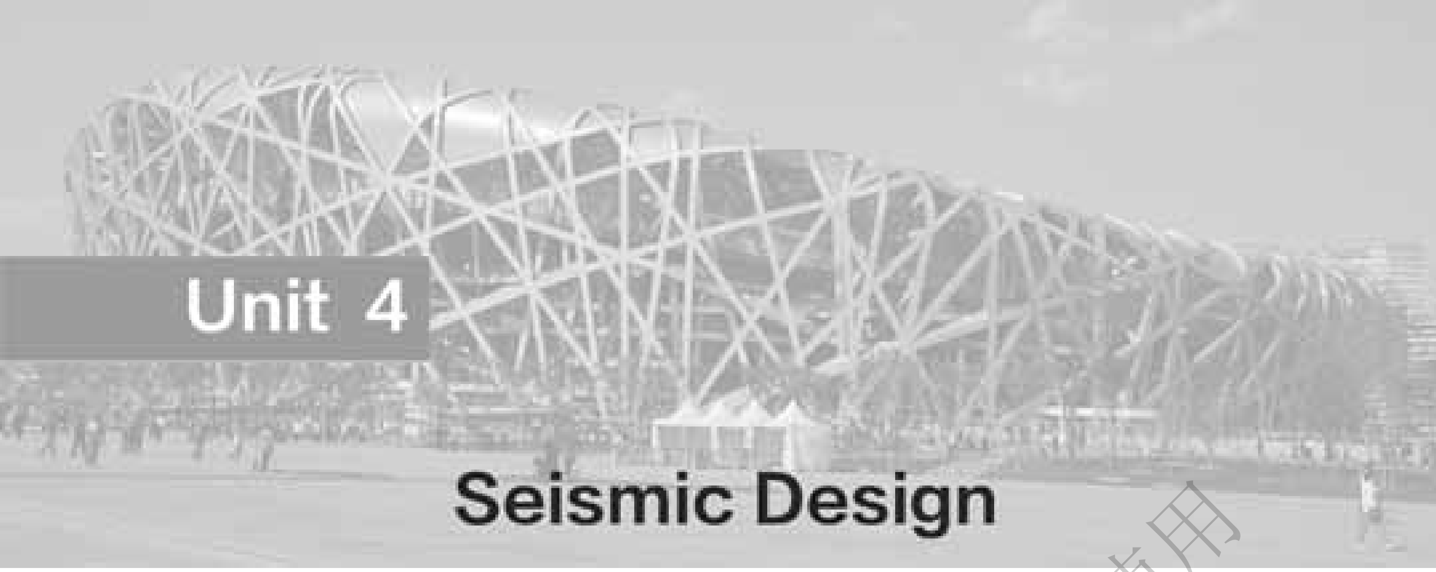
1. If a specific objective criterion can be expressed mathematically, then optimization techniques may be employed to obtain a maximum or minimum for the objective function. Optimization procedures and techniques comprise an entire subject that is outside the scope of this text . The criterion of minimum weight is emphasized throughout, under the general assumption that minimum material represents minimum cost.

2. The structural framework design is the selection of arrangement and sizes of structural elements so that service loads may be safely carried, and displacements are within acceptable limits.

3. The process of rolling various shapes was developing as cast iron and wrought iron received wider usage.

4. Because of the public concern for adequate safety, live loads to be taken as service loads in design are usually prescribed by state and local building codes.

5. When the ground under an object (structure) having a certain mass suddenly moves, the inertia of the mass tends to resist the movement.



Unit 4

Seismic Design

4.1 Introduction

Earthquakes result from the sudden movement of tectonic plates in the earth's crust. The movement takes place at fault lines, and the energy released is transmitted through the earth in the form of waves that cause ground motion many miles from the epicenter.[1] Regions adjacent to active fault lines are the most prone to experience earthquakes. The values, expressed as a percent of gravity, represent the expected peak acceleration of a single-degree-of-freedom system with a 0.2 sec period and 5 percent of critical damping.[2] Known as the 0.2 sec spectral response acceleration S_s (subscript s for short period), it is used, along with the 1.0 sec spectral response acceleration S_1 (mapped in a similar manner), to establish the loading criteria for seismic design. Accelerations S_s and S_1 are based on historical records and local geology. For most of the country, they represent earthquake ground motion with a "likelihood of exceedance of 2 percent in 50 years", a value that is equivalent to a return period of about 2500 years.

As experienced by structures, earthquakes consist of random horizontal and vertical movements of the earth's surface. As the ground moves, inertia tends to keep structures in place (Fig. 4-1), resulting in the imposition of displacements and forces that can have catastrophic results.[3] The purpose of seismic design is to proportion structures so that they can withstand the displacements and the forces induced by the ground motion.

Historically in North America, seismic design has emphasized the effects of horizontal ground motion because the horizontal components of an earthquake usually exceed the vertical components and because structures are usually much stiffer and stronger in response to vertical loads than they are in response to horizontal loads. Experience has shown that the horizontal components are the most destructive. For structural design, the intensity of an earthquake is usually described in terms of the peak ground acceleration as a fraction of the acceleration of gravity, i.e., $0.1g$, $0.2g$, or $0.3g$. Although peak acceleration is an important design parameter, the frequency characteristics and duration of an earthquake are also important; the closer the frequency of the earthquake motion is to the natural frequency of a structure and the longer the duration of the earthquake, the greater the potential for damage.

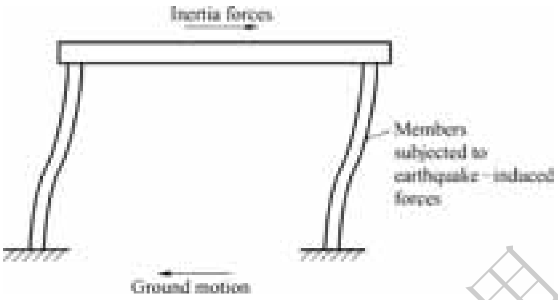


Fig. 4-1 Structure subjected to ground motion.

Based on elastic behavior, structures subjected to a major earthquake would be required to undergo large displacements. However, North American practice requires that structures be designed for only a fraction of the forces associated with those displacements. The relatively low design forces are justified by the observations that buildings designed for low forces have behaved satisfactorily and those structures dissipate significant energy as the materials yield and behave inelastically. This nonlinear behavior, however, usually translates into increased displacements, which may require significant ductility and result in major nonstructural damage. Displacements may also be of such a magnitude that the strength of the structure is affected by stability considerations.

Designers of structures that may be subjected to earthquakes, therefore, are faced with a choice: (1) providing adequate stiffness and strength to limit the response of structures to the elastic range or (2) providing lower-strength structures, with presumably lower initial costs, that have the ability to withstand large inelastic deformations while maintaining their load-carrying capability.

4.2 Structural Response

The safety of a structure subjected to seismic loading rests on the designer's understanding of the response of the structure to ground motion. For many years, the goal of earthquake design in North America has been to construct buildings that will withstand moderate earthquakes without damage and severe earthquakes without collapse. Building codes have undergone regular modification as major earthquakes have exposed weaknesses in existing design criteria.

Design for earthquakes differs from design for gravity and wind loads in the relatively greater sensitivity of earthquake-induced forces to the geometry of the structure. Without careful design, forces and displacements can be concentrated in portions of a structure that are not capable of providing adequate strength or ductility. Steps to strengthen a member for one type of loading may actually increase the forces in the member and change the mode of failure from ductile to brittle.

4.2.1 Structural Considerations

The closer the frequency of the ground motion is to one of the natural frequencies of a structure, the greater the likelihood of the structure experiencing resonance, resulting in an increase in both displacement and damage.[4] Therefore, earthquake response depends strongly on the geometric properties of a structure, especially height. Tall buildings respond more strongly to long-period (low-frequency) ground motion, while short buildings respond more strongly to short-period (high-frequency) ground motion. Fig. 4-2 shows the shapes for the principal modes of vibration of a three-story frame structure. The relative contribution of each mode to the lateral displacement of the structure depends on the frequency characteristics of the ground motion. The first mode (Fig. 4-2a) usually provides the greatest contribution to lateral displacement.

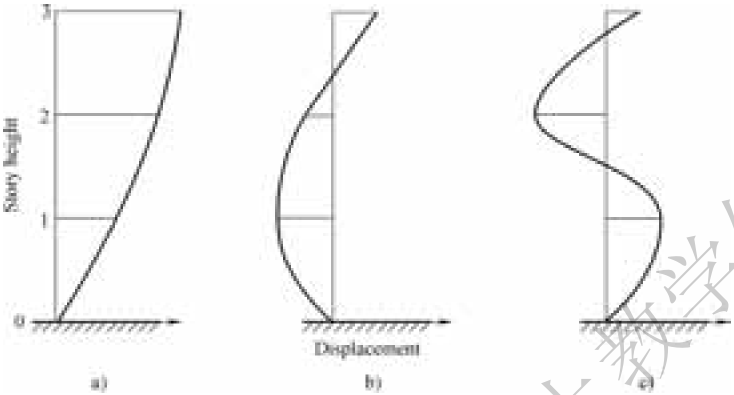


Fig. 4-2 Modal shapes for a three-story building.

a) First mode b) Second mode c) Third mode.

The taller a structure, the more susceptible it is to the effects of higher modes of vibration, which are generally additive to the effects of the lower modes and tend to have the greatest influence on the upper stories.[5] Under any circumstances, the longer the duration of an earthquake, the greater the potential for damage.

The configuration of a structure also has a major effect on its response to an earthquake. Structures with a discontinuity in stiffness or geometry can be subjected to undesirably high displacements or forces. For example, the discontinuance of shear walls, infill walls, or even cladding at a particular story level, such as shown in Fig. 4-3, will have the result of concentrating the displacement in the open, or "soft", story. The high displacement will, in turn, require a large amount of ductility if the structure is not to fail. Such a design is not recommended, and the Fig. 4-4 illustrates structures with vertical geometric and plan irregularities, which result in torsion induced by ground motion.

Within a structure, stiffer members tend to pick up a greater portion of the load. When a frame is combined with a shear wall, this can have the positive effect of reducing the displacements of the structure and decreasing both structural and non-structural damage. However, when the effects of higher stiffness members, such as masonry infill walls, are not considered in the design, unexpected and often undesirable results can occur.

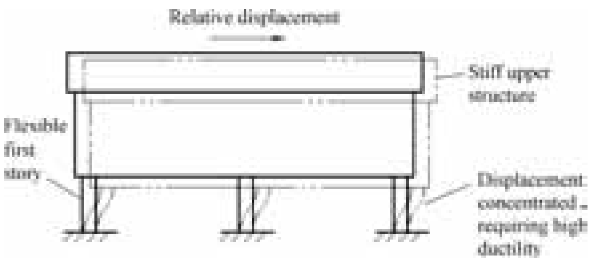


Fig. 4-3 Soft first story supporting a stiff upper structure.

Finally, any discussion of structural considerations would be incomplete without emphasizing the need to provide adequate separation between structures. Lateral displacements can result in structures coming in contact during an earthquake, resulting in major damage due to hammering.

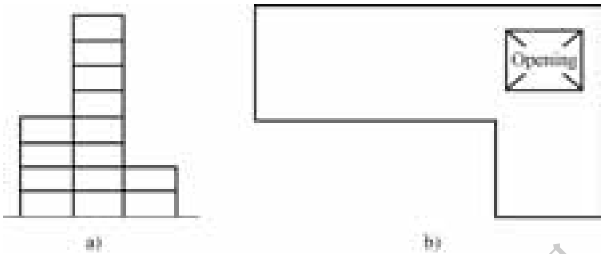


Fig. 4-4 Structures with vertical geometric and plan irregularities.

a) Vertical geometric b) Plan irregularities.

4. 2. 2 Member Considerations

Members designed for seismic loading must perform in a ductile fashion and dissipate energy in a manner that does not compromise the strength of the structure.[6] Both the overall design and the structural details must be considered to meet this goal.

The principal method of ensuring ductility in members subject to shear and bending is to provide confinement for the concrete. This is accomplished through the use of closed hoops or spiral reinforcement, which enclose the core of beams and columns. When confinement is provided, beams and columns can undergo nonlinear cyclic bending while maintaining their flexural strength and without deteriorating due to diagonal tension cracking. The formation of ductile hinges allows reinforced concrete frames to dissipate energy.

Successful seismic design of frames requires that the structures be proportioned so that hinges occur at locations that least compromise strength.[7] For a frame undergoing lateral displacement, such as shown in Fig. 4-5a, the flexural capacity of the members at a joint (Fig. 4-5b) should be such that the columns are stronger than the beams. In this way, hinges will form in the beams rather than the columns, minimizing the portion of the structure affected by nonlinear behavior and maintaining the overall vertical load capacity. For these reasons, the "weak beam-strong column" approach is used to design reinforced concrete frames subject to seismic loading.

When hinges form in a beam, or in extreme cases within a column, the moments at the end of the member, which are governed by flexural strength, determine the shear that must be carried, as illustrated in Fig. 4-5c. The shear V corresponding to a flexural failure at both ends of a beam or column is

$$V=\frac{M^{+}+M^{-}}{l_{\mathrm{n}}} \tag{4-1}$$

where M^{+} and M^{-}—flexural capacities at the ends of the member;

l_{n}—clear span between supports.

The member must be checked for adequacy under the shear V in addition to shear resulting from dead and live gravity loads. Transverse reinforcement is added, as required. For members with inadequate shear capacity, the response will be dominated by the formation of diagonal cracks, rather than ductile hinges, resulting in a substantial reduction in the energy dissipation capacity of the

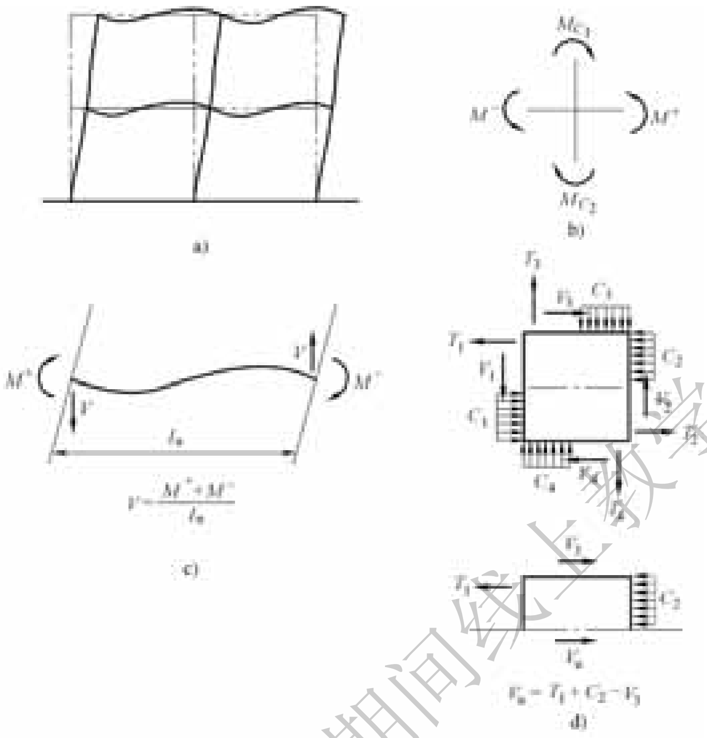


Fig. 4-5 Frame subjected to lateral loading.

a) Deflected shape b) Moments acting on beam-column joint c) Deflected shape and forces acting on a beam d) Forces acting on faces of a joint due to lateral load

member.[8]

If short members are used in a frame, the members may be unintentionally strong in flexure compared to their shear capacity. An example would be columns in a structure with deep spandrel beams or with "nonstructural" walls with openings that expose a portion of the columns to the full lateral load. As a result, the exposed region, called a captive column, responds by undergoing a shear failure, as shown in Fig. 4-6.

The lateral displacement of a frame places beam-column joints under high shear stresses because of the change from positive to negative bending in the flexural members from one side of the joint to the other, as shown in Fig. 4-5d. The joint must be able to withstand the high shear stresses and allow for a change in bar stress

Fig. 4-6 Shear failure in a captive column without adequate transverse reinforcement.

from tension to compression between the faces of the joint. Such a transfer of shear and bond is often made difficult by congestion of reinforcement through the joint. Thus, designers must ensure that joints not only have adequate strength but are also constructable. Two-way systems without beams are especially vulnerable because of low ductility at the slab-column intersection.

4.3 Seismic Loading Criteria

In the United States, the design criteria for earthquake loading are based on design procedures developed by the Building Seismic Safety Council and incorporated in *Minimum Design Loads for Buildings and Other Structures* (ASCE/SEI 7). The values of the spectral response accelerations S_S and S_1, are obtained from detailed maps produced by the United States Geological Survey and included in ASCE/SEI 7. The values of S_S and S_1 are used to determine the spectral response accelerations S_{Ds} and S_{D1} that are used in design.

$$S_{Ds}=\frac{2}{3}F_aS_s \tag{4-2}$$

$$S_{D1}=\frac{2}{3}F_vS_1 \tag{4-3}$$

where F_a and F_v are site coefficients that range from 0.8 to 0.25 and from 0.8 to 0.35, respectively, as a function of the geotechnical properties of the building site and the values of S_s and S_1, respectively. Higher values of F_a and F_v are possible for some sites. The coefficients F_a and F_v increase in magnitude as site conditions change from hard rock to thick, soft clays and (for softer foundations) as the values of S_s and S_1 decrease.

Both S_{Ds} and S_{D1} are used to construct the design response spectrum shown in Fig. 4-7, which relates the spectral response acceleration S_a, used to calculate the earthquake force, to the fundamental period of the structure T. In the spectrum, $T_0=0.2\,S_{D1}/S_{Ds}$, $T_s=S_{D1}/S_{Ds}$, and T_L is the site-specific long-period transition period.

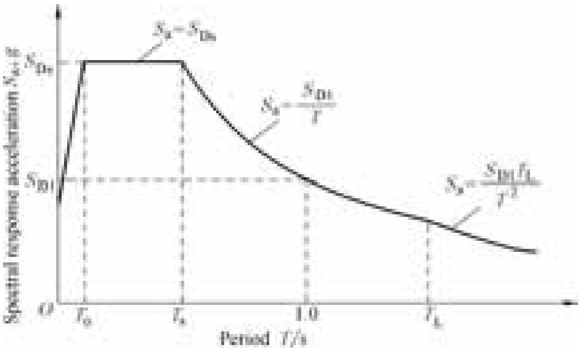


Fig. 4-7 Design response spectrum.

Structures are assigned to one of six *Seismic Design Categories* (SDCs) A through F as a function of (1) structure occupancy and use and (2) the values of S_{Ds} and S_{D1}. Requirements for seismic design and detailing are minimal for SDCs A and B but become progressively more rigorous for SDCs C through F.

Earthquake loading is included in two combinations of factored load.

$$U=1.2D+1.0E+1.0L+0.2S \tag{4-4}$$

$$U=0.9D+1.0E+1.6H \tag{4-5}$$

where D—dead load;

E—earthquake load;

H—weight or pressure from soil;

L—live load;

S—snow load.

For SDC A, the earthquake load E is a horizontal load equal to 1 percent of the dead load D assigned to each floor. For SDCs B through F, the values of the earthquake load E used in Eq. (4-4) and Eq. (4-5) are, respectively,

$$E=\rho Q_{\mathrm{E}}+0.2S_{\mathrm{Ds}}D \tag{4-6a}$$

$$E=\rho Q_{\mathrm{E}}-0.2S_{\mathrm{Ds}}D \tag{4-6b}$$

where Q_{E}—effect of horizontal seismic forces;

ρ—reliability factor.

The factor ρ is taken as 1.0 for structures assigned to SDCs B and C and as 1.3 for structures assigned to SDCs D though F, in which case ρ may be taken as 1.0.

Combining Eq. (4-4) with Eq. (4-6a) and Eq. (4-5) with Eq. (4-6b) gives

$$U=(1.2+0.2S_{\mathrm{Ds}})D+\rho Q_{\mathrm{E}}+1.0L+0.2S \tag{4-7}$$

$$U=(0.9-0.2S_{\mathrm{Ds}})D+\rho Q_{\mathrm{E}}+1.6H \tag{4-8}$$

Eq. (4-4) and Eq. (4-6a) are used when dead load adds to the effects of horizontal ground motion, while Eq. (4-5) and Eq. (4-6b) are used when dead load counteracts the effects of horizontal ground motion. Thus, the total load factor for dead load is greater than 1.2 in Eq. (4-7) and less than 0.9 in Eq. (4-8).

ASCE/SEI 7 specifies six procedures (if SDC A is included) for determining the horizontal earthquake load Q_{E}. These procedures include three progressively more detailed methods that represent earthquake loading through the use of equivalent static lateral loads, *modal response spectrum analysis*, *linear time-history analysis*, and *nonlinear time-history analysis*. The method selected depends on the seismic design category. All but the most basic reinforced concrete structures in Seismic Design Categories B through F must be designed using equivalent lateral force analysis (the most detailed of the three equivalent static lateral load procedures), modal response analysis, or time-history analysis. These procedures are discussed next.

4.3.1 Equivalent Lateral Force Procedure

According to ASCE/SEI 7, equivalent lateral force analysis may be applied to all structures with S_{Ds} less than $0.33g$ and S_{D1} less than $0.133g$, as well as structures subjected to much higher design spectral response accelerations, if the structures meet certain requirements. More sophisticated dynamic analysis procedures must be used otherwise.

The equivalent lateral force procedure provides for the calculation of the total lateral force, defined as the design base shear V, which is then distributed over the height of the building. The design base shear V is calculated for a given direction of loading according to the equation

$$V=C_{\mathrm{s}}W \tag{4-9}$$

where W is the total dead load plus applicable portions of other loads and

$$C_s = \frac{S_{Ds}}{R/I} \tag{4-10}$$

which need not be greater than

$$C_s = \frac{S_{D1}}{T(R/I)} \quad for\ T \leqslant T_L \tag{4-11}$$

or

$$C_s = \frac{S_{D1} T_L}{T^2(R/I)} \quad for\ T \leqslant T_L \tag{4-12}$$

but may not be less than

$$C_s = 0.44 I S_{Ds} \geqslant 0.01 \tag{4-13}$$

or where $S_1 \geqslant 0.6g$,

$$C_s = \frac{0.5 S_1}{R/I} \tag{4-14}$$

where R—response modification factor (depends on structural system), Values of R for most reinforced concrete structures range from 4 to 8, based on ability of structural system to sustain earthquake loading and to dissipate energy;

I—occupancy important factor, $I = 1.0$, 1.25, or 1.5, depending upon the occupancy and use of structure;

T—fundamental period of structure.

According to ASCE/SEI 7, the period T can be calculated based on an analysis that accounts for the structural properties and deformational characteristics of the elements within the structure. Approximate methods may also be used in which the fundamental period of the structure may be calculated as

$$T = C_t h_n^x \tag{4-15}$$

where h_n—height above the base to the highest level of structure (ft);

C_t—0.016 for reinforced concrete moment-resisting frames in which frames resist 100 percent of required seismic force and are not enclosed or adjoined by more rigid components that will prevent frame from deflecting when subjected to seismic forces, and 0.020 for all other reinforced concrete buildings;

x—0.90 for $C_t = 0.016$ and 0.75 for $C_t = 0.020$.

Alternately, for structures not exceeding 12 stories in height, in which the lateral-force-resisting system consists of a moment-resisting frame and the story height is at least 10 ft,

$$T = 0.1N \tag{4-16}$$

where N—number of stories.

For shear wall structures, ASCE/SEI 7 permits T to be approximated as

$$T = \frac{0.0019}{\sqrt{C_w}} h_n \tag{4-17}$$

where

$$C_{\mathrm{w}} = \frac{100}{A_{\mathrm{B}}}\sum_{i=1}^{n}\left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{1+0.83(h_i/D_i)^2} \tag{4-18}$$

where A_{B}—base area of structure (ft^2);

A_i—area of shear wall (ft^2);

D_i—length of shear wall i (ft);

n—number of shear walls in building that are effective in resisting lateral forces in direction under consideration.

The total base shear V is distributed over the height of the structure in accordance with Eq. (4-19).

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{4-19}$$

where F_x—lateral seismic force induced at level x;

w_x, w_i—portion of W at level x and level i, respectively;

h_x, h_i—height to level x and level i, respectively;

k—exponent related to structural period, $k=1$ for $T \leqslant 0.5$ sec and $k=2$ for $T \geqslant 2.5$ sec, for 0.5sec $<T<$ 2.5sec, k is determined by linear interpolation or set to a value of 2.

The design shear at any story V_x equals the sum of the forces F_x at and above that story. For a 10-story building with a uniform mass distribution over the height and $T = 1.0$ sec, the lateral forces and story shears are distributed as shown in Fig. 4-8.

At each level, V_x is distributed in proportion to the stiffness of the elements in the vertical lateral-force-resisting system. To account for unintentional building irregularities that may cause a horizontal torsional moment, a minimum 5 percent eccentricity must be applied if the vertical lateral-force-resisting systems are connected by a floor system that is rigid in its own plane.

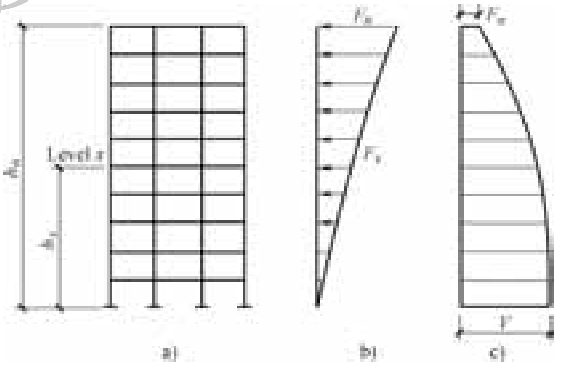


Fig. 4-8 Forces based on ASCE/SEI 7 equivalent lateral force procedure.

a) Structure b) Distribution of lateral forces over height c) Story shears

In addition to the criteria just described, ASCE/SEI 7 includes criteria to account for overturning effects and provides limits on story drift. P-Δ effects must be considered, and the effects of upward loads must be accounted for in the design of horizontal cantilever components and prestressed members.

4. 3. 2 Dynamic Lateral Force Procedures

ASCE/SEI 7 includes dynamic lateral force procedures that involve the use of (1) response spectra, which provide the earthquake-induced forces as a function of the natural periods of the structure, or (2) time-history analyses of the structural response based on a series of ground motion acceleration histories that are representative of ground motion expected at the site. Both procedures require the development of a mathematical model of the structure to represent the spatial distribution of mass and stiffness. Response spectra, are used to calculate peak forces for " a sufficient number of nodes to obtain the combined modal mass participation of at least 90 percent of the actual mass in each of two orthogonal directions". Since these forces do not always act in the same direction, as shown in Fig. 4-2, the peak forces are averaged statistically, in most cases using the square root of the sum of the squares to obtain equivalent static lateral forces for use in design. In cases where the periods in the translational and torsional modes are closely spaced and result in significant cross correlation of the modes, the complete quadratic combination method is used.[9] When time-history analyses, which may include a linear or nonlinear representation of the structure, are used, design forces are obtained directly from the analyses.[10] Both modal response spectrum and time-history procedures provide more realistic representations of the seismically induced forces in a structure than do equivalent lateral force analyses.

New Words and Expressions

tectonic plates		构造板块，地壳板块
fault lines		裂纹线，断层线
critical damping		临界阻尼
spectral response		谱响应
subscript	*n.*	下标，脚注
dissipate	*v.*	消散
ductile to brittle		韧性到脆性
discontinuity	*n.*	不连续
infill wall		填充墙
incorporated in		纳入，包括
cantilever	*n.*	悬臂
response spectra		反应谱
orthogonal	*adj.*	正交的

Notes

1. The movement takes place at fault lines, and the energy released is transmitted through the earth in the form of waves that cause ground motion many miles from the epicenter.

本句难点解析：句子中 through 作为介词表明能量传播途径，in the form of 表明传播方式，that cause ground motion many miles from the epicenter 作为定语从句修饰 waves。

本句大意如下：运动发生在断层线上，能量释放后以波的形式在地层传播，引起了离震源数英里之外的地方产生地面运动。

2. The values, expressed as a percent of gravity, represent the expected peak acceleration of a single-degree-of-freedom system with a 0.2sec period and 5 percent of critical damping.

本句难点解析：句子主体是 The values represent the expected peak acceleration。其中 with a 0.2sec period and 5 percent of critical damping 作为定语修饰 system，expressed as a percent of gravity 作为定语修饰 values。

本句大意如下：以重力百分数表示的值代表了在 0.2s 自振周期，临界阻尼为 0.05 的单自由度体系下的加速度峰值。

3. As the ground moves, inertia tends to keep structures in place (Fig. 4-1), resulting in the imposition of displacements and forces that can have catastrophic results.

本句难点解析：句子中 resulting in the imposition of displacements and forces that can have catastrophic results 作为定语从句表示结果。

本句大意如下：随着地面的运动，惯性将会使结构倾向于原地不动，从而在结构上施加了位移及力，这将导致严重的破坏后果。

4. The closer the frequency of the ground motion is to one of the natural frequencies of a structure, the greater the likelihood of the structure experiencing resonance, resulting in an increase in both displacement and damage.

本句难点解析：句子中 the closer... the greater 表示越接近……越大……。resulting in an increase in both displacement and damage 作为定语从句表示结果。

本句大意如下：地面运动的频率越接近结构自振频率，结构发生自振的可能性便越大，这将导致结构位移以及破坏情况的增加。

5. The taller a structure, the more susceptible it is to the effects of higher modes of vibration, which are generally additive to the effects of the lower modes and tend to have the greatest influence on the upper stories.

本句难点解析：句子中 the taller... the more 表示越高……越……。which 引导定语从句修饰 effects。

本句大意如下：结构越高，就越容易受高阶振型的影响，这种影响一般作为低阶模态作用的一个附加部分，且对上部楼层有较大的影响。

6. Members designed for seismic loading must perform in a ductile fashion and dissipate energy in a manner that does not compromise the strength of the structure.

本句难点解析：句子中 in a manner 表示以一种……的方式，并通过定语从句对 manner 进行说明。

本句大意如下：按地震荷载设计的构件必须以延性的方式工作，并以不会折减结构强度的方式进行耗能。

7. Successful seismic design of frames requires that the structures be proportioned so that hinges occur at locations that least compromise strength.

本句大意如下：成功的框架抗震设计要求结构比例合适，这样塑性铰才会出现在合适的位置，从而使得强度折减量最小。

8. For members with inadequate shear capacity, the response will be dominated by the formation of diagonal cracks, rather than ductile hinges, resulting in a substantial reduction in the energy dissipation capacity of the member.

本句难点解析：句子中 resulting in a substantial reduction in the energy dissipation capacity of the member 作为定语从句，表示目的。

本句大意如下：对于抗剪强度不足的构件，在地震作用下其主要先产生斜裂缝，而不是塑性铰，这将导致构件耗能能力的直接下降。

9. In cases where the periods in the translational and torsional modes are closely spaced and result in significant cross correlation of the modes, the complete quadratic combination method is used.

本句难点解析：句子中 In cases where 引导状语从句，其中 spaced 一词表示两者十分接近，即平移与扭转模态。

本句大意如下：在平移和扭转的模态周期比较接近，从而导致模态发生明显的交叉相关的情况下，可以采用完全二次型组合法。

10. When time-history analyses, which may include a linear or nonlinear representation of the structure, are used, design forces are obtained directly from the analyses.

本句难点解析：句子中 When 引导状语从句，从句主体是 When time-history analyses are used。which 引导定语从句修饰 time-history analyses。

本句大意如下：当使用时程分析，即包括结构的线性及非线性特征时，设计力可直接从分析中获得。

Exercises

Translate the following phrases into Chinese.

1. earth's crust
2. a single-degree-of-freedom system
3. keep structures in place
4. mode of Vibration
5. lateral displacement
6. equivalent lateral force
7. fundamental period of the structure
8. non-structural damage
9. distribution of mass and stiffness
10. time-history procedure

Translate the following Sentences into Chinese.

1. Known as the 0.2 sec spectral response acceleration S_s (subscript s for short period), it is used, along withe the 1.0 sec spectral response acceleration S_1 (mapped in a similar manner), to establish the loading criteria for seismic design.

2. Although peak acceleration is an important design parameter, the frequency characteristics and duration of an earthquake are also important.

3. The configuration of a structure also has a major effect on its response to an earthquake.

4. According to ASCE/SEI 7, equivalent lateral force analysis may be applied to all structures with S_{Ds} less than $0.33g$ and S_{D1} less than $0.133g$, as well as structures subjected to much higher design spectral response accelerations, if the structures meet certain requirements.

5. The equivalent lateral force procedure provides for the calculation of the total lateral force, defined as the design base shear V, which is then distributed over the height of the building.