Review of engineering education and some technical and non-technical expectations of a new entry level structural engineer

by

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Bachelor of Engineering (Civil)
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Submitted to the Department of Civil and Environmental Engineering
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Abstract

The engineering education is evolving very quickly in this technologically driven era. In this thesis, a review of the current engineering education in the United States is made. The roles of graduates, educators, and professionals in the engineering field are explored with particular attention to the structural engineering education system. Models for four learning styles and the reader’s possible teaching or learning style are explained. Some of the professionals’ expectations are examined for the students to gain an understanding of the problem. The author has explored an environment with integrative teaching model and computer usage to motivate the students for learning. The author has identified three practitioner approaches in structural engineering education. The first practitioner approach deals with the emphasizing the importance of “feel” for structures to the entry level engineer. The second practitioner approach is made with an exploration of an additional stage in educating the structural engineers. The author has chosen tall building design to show the reader the selection process of structural forms with an advanced tool using Artificial Intelligence. In addition, the author has also dealt with some of the technical expectations of the entry engineer by the use of preliminary elementary sizing and design. The third practitioner approach realizes the concerns of the industry towards cost efficiency. A practical study is made on the weight (cost) comparison of W shape sections under three design codes AISC, BS5950, and EC3. The author has found that optimization of W shape sections could be achieved for certain structural elements under particular loading conditions.

Thesis Supervisor: Oral Buyukozturk
Title: Professor of Civil and Environmental Engineering
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Biographical Note

The author had graduated with First Class Honors in the Bachelor of Engineering (Civil) from the National University of Singapore in June 1995. Up until August 1997, the author has been working as a design engineer in KTP Consultants in Singapore.

The two years of challenging experience in the design firm had prepared the author to face various aspects of being a structural engineer. The author had been given opportunities to represent the firm in meetings with clients, architects, and contractors. Besides, the author was also involved in the detailed design of projects ranging from small to large scale. The most meaningful and memorable project that the author was involved has to be the Far East Square project in restoring parts of Chinatown in Singapore. Other projects include a 8-storey residential development, a 6-storey hotel development and also an addition work to the existing Underwater World of Singapore.
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Chapter 1

Introduction

The construction industry in the United States comprises mostly small, local, and very competitive establishments. Business activities in this industry are highly cyclical, especially at the local level, and employee turnover rates are high, so survival in the design and construction industry depends on keeping efficiency up and overhead down. Consequently, most design and construction firms are reluctant to hire untrained engineers and architects or to invest in expensive training, particularly since the trainees are likely to leave in the near future. What then can be done to better prepare students for becoming an entry level engineer?

1.1 Objectives

The main objective of this thesis is to review the engineering education and also identify and relate some of the professionals’ expectations to the entry level engineer. The main audience for this thesis includes the graduating structural engineering students, entry-level structural engineers and the educators in the field of structural engineering.

For the graduating students and entry level structural engineers, we will describe what the are the professionals’ expectations. General questions that have been asked are:
- What can the students do to achieve the expectations?
What are the other things that the students can do beyond just basic structural designing?
What are the tools available that can help the students to achieve the expectations?

For the educators, we will attempt to relate the expectations of professionals with the introduction of an additional stage in educating structural engineer. Accordingly, some technical expectations of the students will also surface.

Many students feel that there is a lack of true engineering practices in their engineering education program. Even with introduction of an open-ended design project for the Design of Concrete Structures in MIT, the students, when asked “Do you think that this design project has equipped you with enough background when you start working?” responded as follows:

“It is enough to start, but assumptions build up and at the end when you really start learning most of it, it’s too late to go back and make changes because of time constraint”
“Sort of. We know the calculations but don’t have a real basis for what is acceptable in the real world.
It has helped, but it would have to be much more detailed/involved.”
“No, you would need more time for in-depth analysis.”
“No, but it makes it stick more. Easy learning is easily forgotten.”
“Working? Um not sure. Don’t know what you need to go to work.”
“Not really, but as time goes, I was able to pick up.”

From these responses, we observe that there are so many uncertainties in what are being expected of the students when they begin to work as entry level structural engineers. The students do not seem to know about the professionals’ expectations.
1.2 Scope of thesis

There is an unbounded list of what might be expected of an entry level structural engineer from the professionals. We have limited the scope of discussion to the following:

First, we review the current state of engineering education in Chapter 2. The objective is to provide preliminary information on the history and current engineering programs accredited in the United States. The current engineering education will be identified in becoming more design oriented. The need for practitioner approaches in engineering education will be covered later.

Next, we will provide a general understanding of the roles of the graduates, the educators, and the professionals in Chapter 3. In addition, various related problems within the current educational systems will be identified and recommendation will be made for rectifying these problems. Investigation of the roles of the professionals will provide the reader with a clear indication of their expectations of new graduate students upon entering the profession.

Following in Chapter 4, we will review the teaching and learning environment within engineering education. We will describe one teaching framework that integrates subject material among individual classes and builds upon prior student experience so that the students will value their learning experiences. In addition, we will discuss the advantages and disadvantages in the use of computer and the basic prerequisites in using computer along with the modeling and verification aspects. We will also discuss the concept of “bringing the site into the classroom” as teaching aids.

In the subsequent chapters, we will approach the lack of practitioner approaches in structural engineering education by looking at three main aspects:
- lacking a sense of “feel” for structures by entry level engineer
- learning about the conceptualization of structural forms and preliminary designs
- learning the use of practical codes for optimization
We will attempt to look at the important issues pertaining to the entry-level structural engineer’s sense or “feel” for structures and structural designs in Chapter 5. Various technical expectations of the entry level engineer will be discussed.

Subsequently, focusing on the need to “feel” for the structure, we will present an additional stage in structural engineering education i.e. teaching and learning of conceptual designs through selection of structural forms and preliminary structural designs. This will be covered in Chapter 6. Tall building design will serve as a case study for such additional stage.

In Chapter 7, we will look at a practitioner approach in using practical codes for optimization. A study has been made on cost efficiency in design of structural steel. We will investigate the optimization of the steel design with just using the American steel (W-shapes only) by comparing the cost efficiency in three different design codes namely American Institute of Steel Construction – Load Resistance Factor Design (AISC), British Standards (BS5950), and European Code (EC3).

Finally, the author will summarize his findings in the conclusion chapter.
Chapter 2
Review of the current educational systems

2.1 Introduction

In the construction industry, proper education systems for the engineering students become vital for their acceptance as an entry engineer. However, a perception has arisen among many federal construction officials that recent graduates in engineering lack sufficient training for professional careers in facility design and construction [1]. In addition, the professional views of senior representatives from the 14 societies and associations believe that there are serious problems with the current system for educating both engineers and architects. This view has been expressed by both the academics and non-academics and by respondents who did not necessarily agree with the idea that problems exist.

This chapter will explore the background of engineering education programs currently available in the United States. We believe that this background information will be beneficial in the process of preparing graduates for entering the engineering profession.
2.2 Engineering education programs

The engineering education programs in the United States are accredited by the Accreditation Board for Engineering and Technology (ABET), a private corporation founded in the early 1930s. ABET accredits three broad categories of programs: engineering programs, engineering technology programs, and engineering related programs. ABET defines the broad categories of programs as:

**Definition of Engineering**
Engineering is defined as a profession in which a knowledge of the mathematical and natural sciences gained by study, experience, and practice is applied with judgment to develop ways to economically utilize the materials and forces of nature for the benefit of mankind.

**Definition of Engineering Technology**
Engineering Technology is that part of the technological field that required the application of scientific and engineering knowledge and methods combined with technical skills in support of engineering activities; it lies in the occupational spectrum between the craftsmen and engineer at the end of the spectrum closest to the engineer.

**Definition of Engineering-Related Programs**
Engineering-related programs in higher technical education are mathematics and science-based programs that do not fit the strict definitions of either engineering or engineering technology but have close practical and academic ties with engineering. With appropriate participation from societies representing specific engineering relayed professional disciplines, engineering-related programs may be structured to prepare graduates for entry into professional practice in a discipline that is neither engineering or engineering technology.

From the statistics on the construction industry, there were 21,000 engineers employed in construction back in 1989. Earlier in the beginning of the 80s, there were 50,000
construction engineers indicating that there had been a dramatic drop in the numbers in one decade. Ten percent of engineers employed by the industry are estimated to be exclusively in the construction phase design and construction process. In 1991 alone, 256 institutions of the ABET-approved civil engineering programs, collectively awarding 7,767 bachelor’s degrees in civil engineering.

Another organization that deals with engineering education is the American Society for Engineering Education (ASEE). ASEE is the successor to the Society for the Promotion of Engineering Education (SPEE), formed in 1893 to improve classroom instruction techniques and promote acceptance of the idea that engineering education should concentrate on teaching scientific and mathematical principles rather than giving hands-on experience.

The history of the SPEE/ASEE has been marked by a continuing debate over certain issues: How should an engineering curriculum be divided between technical, professional, and general education? How much practical engineering work is needed? How long should an engineering education take?

Attempts have been made repeatedly over the years to resolve these questions. There were studies dated to the early 1930s. There have been changes in the emphasis of education over the years. In 1987, ASEE initiated another broad view of engineering education by a task force that called for more emphasis on design and manufacturing in engineering curriculum and for more practice-oriented (rather than research-oriented) master’s programs.

For almost a century, engineering educators have been concerned about high attrition rates in engineering schools. In spite of the adoption of high admissions standards based on Scholastic Aptitude Test (SAT) scores and other criteria, engineering programs have continued to have higher drop-out rates than other educational programs. This indicated that engineering schools are losing many students who begin college with an interest in
and an aptitude for engineering [2]. Based on data collected on more than 27,000 students at 388 4-yr colleges and universities over a 4-year period, the following is revealed:

*Between the freshmen and the senior years, the percent of students majoring in fields of natural science, mathematics, and engineering (SME) declines from 28.7 to 17.4, a 40 percent relative decline. Losses are greatest in the biological sciences (50 percent decline) and engineering (40 percent decline). The net loss in the physical sciences (including mathematics) is substantially less (20 percent decline) in part because these fields recruit substantial numbers of engineering dropouts during the undergraduate years. Indeed, there is evidence to suggest that the presence of a very large program in the physical sciences can accelerate the loss of engineering students by attracting substantial numbers of these students away from engineering into the physical sciences and mathematics. Considered as a career, engineering loses more than half of its students (53 percent decline) during the undergraduate years.*

### 2.3 Engineering education system

The construction industry provides for the following roles for the engineers in the design and construction process:

- **Design firms.** Planners, designers, cost analyst, specification writers, drafters, project managers and field inspectors.
- **Construction firms.** Superintendents, cost estimators, project managers, construction managers and technical advisors.
- **Building owners and developers.** Planners, designers (usually of small projects), writers of technical criteria, cost analysts, field representatives, project managers, and managers of operations and maintenance activities.
- **Fabricators and manufacturers of building products.** Researchers, product designers, applications engineers, sales representatives, technical service representatives, and manufacturing engineers.
- **Academic institutions.** Teachers and researchers.
- Government agencies. Planners, designers, project managers, building code officials, zoning officials, fire marshals, safety inspectors, technical advisors, and researchers.

- Professional societies, trade associations, and standards organizations. Technical coordinators, researchers, and information specialists.

- Independent research and testing organizations. Researchers and test coordinators.

From the list, a newly graduate student in civil and environmental engineering in MIT can choose to enter the job market with plentiful choices depending on his/her interests. With the large list of possible positions, it seem inevitable that the current educational system for engineers may not be able to give students enough practical knowledge and instruction in solving real world problems.

The current engineering education is confronted with two realities that call for a rethinking of the undergraduate curriculum [3]:
- the baccalaureate program, as the terminal degree for practice, has received much criticism in both industry and academia and
- the typical undergraduate student generally requires more than nine semesters to complete a curriculum designed for eight.

It has been proposed [3] that the current densely scheduled curriculum must give way to a reduced program oriented towards engineering science, which can be completed in four years. Such a curriculum must be based on a limited set of core educational outcomes and would educate engineers within an integrated, liberal framework while preparing graduates for a wide variety of career options. Included in that proposal is a formal role for the Masters degree as the first professional degree and a restructuring of the professional registration process which are also stated in a recent November 1998 issue of ASCENews. Compared to other professions such as dentistry, medicine, law and architecture, engineering remains as the only field of study that has not upgraded its primary educational requirement beyond the 4-year baccalaureate degree as the first designated degree for entry into professional practice. The author concurs to such a move
as more time will be allocated to properly train an entry level engineer. However, there is a tendency for students avoiding 5-years program in favor of 4-years program as some employers are unlikely to offer salary premiums to those from 5-years program because the employers believe that engineering schools do not adequately teach design and professional practice [1].

How then can more design, management, and other subjects be incorporated in an undergraduate curriculum of 4 years without displacing other valuable material? Clearly, this is impossible so long as one defines the curriculum as a zero sum game in which discrete subjects must be taught as individual, autonomous courses with no interaction. The committee on Education of Facilities Design and Construction Professionals [1] does not necessary accept that this paradigm cannot be changed.

In view of the above, the author proposed the followings:
- Engineering schools should actively inquire from the professionals about their expectations of an entry-level engineer
- Employers should seriously consider offering higher salary to graduates with 5 years program background

The current system of engineering education [3] had been established for “old-era” industrialization as the framework for engineering and manufacturing and that to meet the needs of the new era, such as lean production processes, a transformation of the curriculum is needed. The coming of the dominantly information-based culture will further increase the demand for the flexible curriculum capable of adapting to fast changing circumstances in the next century.
2.4 Concepts of design and technology

In the engineering education, two essential words used are design and technology. Design is defined [1] as:

- The ultimate goal of the design process is the creation or production of something tangible and functional—such as a building or a system or a product. For financial and other reasons, many designs are never executed; however, every true design efforts begin with the goal of developing a design that could be executed.

- There is no single correct answer to a design problem; there is a range of solutions, each having both advantages and disadvantages.

- Design is a creative process in which an individual or group of individuals with specific technical and analytical knowledge consider a wide range of factors, including function, aesthetics, economics, technology, social, environmental, and legal requirements.

- Design involves the integration and coordination of multiple disciplines and often requires management, marketing, and communications skills.

Engineering education has not considered design central to the current educational process. But the importance of design to an engineering education is being increasingly recognized, and engineering programs in recent years are being criticized for failing to teach design. A 1991 report by the National Research Council compared engineering science education to engineering design education as follows (NRC 1991):

*Engineering education in the United States has undergone many important changes since World War II, leading to impressive improvement in engineering graduates’ knowledge of the engineering sciences, mathematics, and analytical techniques. These changes include restructuring emphasize the engineering sciences as a coherent body of knowledge, the introduction of new disciplines, the creation of an extensive system of research and graduate programs, and the partial integration of computers into curricula. While these improvements were taking place, the state of engineering design education was steadily deteriorating with the result that today’s engineering graduates are poorly equipped to utilize their scientific, mathematical, and analytical knowledge in the design*
of components, processes, and systems. Strengthening engineering design education is critical to the long-term development of engineers who are equipped to become good designers and leaders and who will provide a lasting foundation for US industry’s international competitiveness.

The term technology refers to the application of the practical or industrial arts to the solution of problems. Engineering design is often based on technology – in the form of empirical data and experience derived knowledge – than on science. Therefore, technology emphasizes on applied or empirical science in contrast to pure science. In a 1991 survey by the American Consulting Engineers Council [4], chief executive officers of approximately 1200 consulting engineering firms have found recent graduates to be “well prepared for the work place from an academic standpoint [but] minimal emphasis on practical experience in today’s engineering curricula is reflected by the students’ lack of technical knowledge”.

It seems that educators in school [4] tend to be concerned about the complete elimination of technical courses from the engineering curriculum in favor of courses in mathematics, pure science, and engineering science.

The engineering education comprising of relative value of science-oriented education versus technological training has been going for decades. In mid 50s, the ASEE had succeeded in promoting the science-based engineering curriculum. Again, in mid 60s, the philosophy of engineering education advocating a science-based curriculum was adopted. John Alic of the Congressional Office of Technology Assessment wrote in a 1990 letter published in Issues in Science and Technology: 

*Engineering educators in the United States... have long since won the 100-year-old debate with those, mostly in industry, who would have the schools turn out more practically oriented graduates. Since the 1960s, the theoretically based engineering science perspective has remained unchallenged.*
However, the debate has heated up once again the last few years. A 1990 article in the Engineering News Record reflected a desire on the part of industry for graduates who can “hit the ground running” and civil engineers who know infrastructure but with “higher degree of specialization” [5]. The article also stated that “students who emerge from school with a thorough knowledge of high-tech systems and equipment will have their choice of jobs and companies.”

There are some responses from two of the professional engineers as:

*The person coming out from school- unless he’s had either a co-op program or fairly extensive internships in the summer- doesn’t know what industry is all about.*

*The undergraduates are not well prepared for a job market...*

*They may understand some of the general principles in engineering, but they have difficulty in applying them from a practical standpoint.*

Often, engineers uses trial and error to solve problems. Despite the fact that rational explanation may not be supported by scientific theory, the designer must still know and apply those solutions. Also, the design of most products and systems involves the use of components and materials manufactured by others. Thus, an important aspect of design work involves learning about the use of products available in the marketplace.

For the engineers, technology has been largely eliminated from the curriculum in most schools in order to focus on science and math and basic engineering principles. The result is most engineering schools do a poor job teaching technology and the relationship between technology and design. It is believed that schools could do much more for students in defining technology and its role in engineering practice [1].
2.5 Summary

By now, the reader should have a clear picture of the current engineering education programs in the United States. It has been stated that there is an increasing recognition of design in engineering education. However, there seem to be an elimination of practitioner approach in teaching the entry level engineers. We would look into the use of some practitioner approaches to engineering education later in the thesis.
Chapter 3
Roles of Graduates, educators and professionals

With a basic understanding of the engineering programs and systems available in the United States, there is a need for the reader to identify his/her role within the context of the thesis. This chapter is devoted to review the roles of the various groups of personnel that make up the education system, namely the graduates, the educators and the professionals.

3.1 Roles of Graduates

We approach the roles of graduates by looking at the comments by various organizations. This opens a new standpoint for the reader to learn about their expected roles in the profession.

There is ample evidence of deficiencies in undergraduate design education [1], which fall under the following categories:
- Graduates (mostly engineering graduates) have little understanding of design.
- Graduates lack an understanding of the design and construction process and how it works which indicates a lack of practitioner approach in their engineering education.
- Graduates show deficient knowledge of the business of engineering, architecture, and construction.
- Graduates possess inadequate communication skills, including oral, graphic, and written skills.
- Graduates are prepared to do research but are not prepared to apply their knowledge to practical industry problems.
- Graduates lack experience in teamwork to achieve common goals.

It is not possible to examine graduates’ capabilities or lack thereof without considering what can appropriately be expected of college graduates. Considerations include about how much schools should do to prepare their graduates for professional practice and how much must be learned outside of academia through supplementary experiences such as internships, co-ops, or summer work, or on the job after graduation? How much can be fit into an undergraduate degree before the level of work should be considered graduate level? How much individual initiative and motivation to prepare the student for continuing learning outside the classroom should be expected?

Grades in school are not considered as a good indicator of how well a graduate engineer will do in later life [6,7]; although not always the case, students dropping out because of bad grades may be a reflection of schools providing courses irrelevant to future career success.

When students enter the workforce, they must make a transition from an academic to a professional role. Interviews with professional engineers [8] in supervisory roles have suggested that many basic skills required in the workplace, including the ability to work on a team and to communicate with one’s peers and supervisors, are missing or insufficiently developed in recent college graduates. Many employers have programs to overcome these deficiencies, but the educators should also consider what they can do to better prepare students for their future roles, and what students themselves can do to ease the transition.
In engineering, most preparation for students involves teaching students basic theory and skills in science and technical subjects. However the secondary concern is the professionalization of students. Professionals who were once students make the following opinions about some of the non-technical expectations:

**Ability to work on a team**
Teamwork is a well-recognized fact of engineering life.
" [As a student] you are not encouraged to do too much teamwork.... You work on your own. For one moment, we've all been competing against one another, and now we all have to be friends and form a team. And nobody's really done a lot of that."[8]

Graduates need to understand the nature of working environment in the industry. A NSPE report in 1992 presented the results of a survey of 888 professional engineers. The report has revealed that when judging new engineering graduates approximately 85 percent of the respondent’s organizations placed a “high value” on a candidate’s “ability to work as part of a team” – the highest percentage among all factors on which the candidates were judged. However, the respondents of the survey have rated only 40 percent of new engineers “well prepared” to work as part of a team.

**Ability to communicate**
One graduate student has mentioned that “ some people who have really stayed in the science and engineering – have not really ventured outside of their chosen filed – are lacking in terms of writing and just some of the basic communication skills that you just have to have”.

**Awareness of workplace expectations**
Students when asked about their perceptions of entry level position, they talked about specific tasks they can see themselves doing- things like on-site sampling, reading, and summarizing documents for upper-level engineers or managers, gathering and analyzing field data. They seem to have a narrow view of what the job involves.
What should the students do?

Students could be asked to assume more responsibilities for their own futures. Students should put in efforts such as learn more about the workplace through co-op assignments, attending seminars presented by professionals, and attending interviews. This way, the expectations of the students would be much more closely matched to the expectations of the professionals than those who had not taken advantage of these opportunities to learn. In the Department of Civil Engineering and Construction at North Dakota State University, students have been assigned projects that encourage them to discover the unique engineering practice. By integrating the stories of influential engineers [9] and their achievements into a quantitative analysis course, the students have experienced the technical material within a broader context, helping them to appreciate the practical engineering as a living, human endeavor. It is particularly important for today’s computer generation of engineering students to appreciate the constraints that faced many important engineers to fully appreciate the significance of their contributions. Furthermore, students can reclaim a sense of wonder in their field of study by looking into the works of these influential engineers that would otherwise be lacking in our sophisticated, technologically advanced society.

All in all, another suggestion is to form an ongoing informal “team” of professionals, students and professors with a joint goal of improving the students’ ability to move into a workplace environment. There are benefits to all if this is achieved. The students who are better prepared has an easier time finding a job.

3.2 Roles of Educators

The general thoughts of most educators can be summarized by Woodie Flowers [10], “Engineering educators often think that the intellectually respectable parts of the profession are associated with obscurity and little-known details. Students on the other hand badly need to learn the fundamentals and immediately experience the satisfaction of its application.”
Indeed, most engineering students choose to do engineering [11] because they believe engineering involves real life problems. Some students feel that routine assignments are uninteresting and boring to them, and do not encourage creativity or original thinking. Others believe that the routine assignments do not expose them to solving realistic problems. In recent times, engineering educators have taken a project-based approach to teaching basic subjects in the early years of the course in order to relate basic concepts to real engineering problems.

The conventional design course

The usual format of conventional design course is one where students choose an individual topic under the guidance of a faculty sponsor in their area of specialization [12]. Students are required to submit mid-semester and final reports and to make oral presentations at both stages. Reports are graded by both the faculty sponsor and the course instructor. Oral presentations would be evaluated by all those in attendance which include the students and the faculty.

Some of the common concerns [2] over the curriculum are:
- Inconsistent level of involvement by the faculty sponsors
- Inadequate grading (i.e. faculty specific) of written submissions in a competitively graded course
- Amount of effort required of faculty sponsors (overload) for no credit
- Inequitable work product or outcomes associated with various projects and/or faculty sponsors
- Non-licensed and/or inexperienced faculty tends to require more analysis than design

New Project oriented study

Particularly for engineering schools, this has been proposed as a counterbalance to individual competitiveness and a motivator for holistic problem-solving through project-oriented teamwork. Project-oriented study might provide experience in teamwork such as surveying summer camps, drafting courses and other shared experiences. For example,
surveying courses necessary for the purpose of teaching surveying also have the effect of building esprit de corps among entire class of undergraduates.

This approach might motivate the students to seek answers to solve problems, which may be the strongest factor in their skill acquisition, reinforcing a team approach that includes communication skills and exposes students to different disciplines. It has been suggested that team projects, that is, project-oriented learning and working in teams, require a student-generated creative problem solving that elevates design above analysis and reinforces team design efforts. This is not to say that engineering analysis is to be neglected; rather, it should placed in its proper context within the design project. In this case, design is not so much taught as learnt. It can be learnt more effectively in an environment that demands design efforts.

Project oriented engineering education gives the undergraduates experiences in managing projects, allocating resources, meeting budgets and schedules, evaluating economics, and dealing with others involved in similar enterprises. This experience can be more truly educational than requiring courses in project management, accounting, or business. More important, it is intended to motivate students to take these courses or to study these subjects on their own.

The point of the project team orientation is to provide the motivation for students to learn the technical materials necessary to perform designs, on the basis that a motivated student will learn far more than an unmotivated one.

The project design approach takes advantage of the fact that students can learn from other students as well as professors. It is recognized to be very difficult and labor intensive to implement project design. Although it is perceived that the lecture model is the most efficient form of education, this approach has the potential to increase educational efficiency by establishing the environment in which students can learn from each other. Juniors can learn from seniors and sophomores from juniors. Individuals can learn from those who have different knowledge, backgrounds, or skills. This form of learning is not
the same as graduate teaching assistants delivering lectures; it relies on the natural environment of peer-to-peer group learning.

Issues related to faculty

Another way of exploring the roles of the educators is to look at the issues related to faculty:

- many faculty members have become so research oriented that they have lost interests in teaching generally and teaching undergraduates in particular
- many faculty members are unable to teach design and technology because they have little or no practical experience outside of the academic world and
- many faculty are poor teachers because they have received no training in education.

Research oriented

There is a frequent debate of whether the engineering faculty has become too research oriented. The views below have been expressed in an editorial in the Journal of Engineering Education:

*Perhaps the change in relative weight between teaching and research has gone too far and, rather than strengthening education, the significantly greater attention now accorded faculty research activities has eroded the quality of the undergraduate program. There seems to be much agreement that the culprit is the reward system that recognizes research and publications as the primary-often only-criteria for promotion, tenure and salary increase [13].*

The growing importance of research in civil engineering is reflected in the percentage of programs in which some faculty members are appointed solely to conduct research had increased from 8.2 percent in 1978 to 13.1 percent in 1989 [14]. The report also include: *The single biggest expense a college has is its faculty. Years ago, faculty taught 14 credits per semester and were expected to engage in scholarship, serve on committees, advise students, develop curricula, and perform a few other tasks. Over time, the teaching load was reduced to 12 credits, the to 9 and in many places it is 6 credits or*
A number of faculty avoid teaching altogether by buying out their teaching time with the proceeds from research grants or outside consulting.

An analysis of the different audiences to which professor professes is established [15] and it has been concluded that the problem is under emphasis on teaching. This under emphasis on teaching has resulted from a lack of commitment of resources in evaluating and rewarding, formal classroom teaching, the avoidance of painful decisions, and time pressures. The reason for the reduced faculty teaching loads is to enable the faculty to devote more time to research. However, more than half of all professors [14] have devoted fewer than 5 hours a week to research, while up to a third admit to none at all. It has been found that having a strongly research oriented faculty reduced the perseverance of physical science majors and fostered student dissatisfaction. This could be explained in part by the tendency for strongly research-oriented faculties to rely heavily on teaching assistants in their undergraduate courses.

The faculty needs to accept the challenge to develop and apply an accepted procedure for measuring the quality of teaching, to provide realistic opportunities for continued professional development, and to provide the time required to meet both research and teaching expectations, especially for probationary faculty.

Lacking in practical experience

Another disturbing trend in teaching is a decrease in the level of practical experience among faculty. Faculty members with practical industry engineering, design, and construction experience have been replaced by professors with Ph.Ds. Not only many of the younger engineering educators are from different cultures and may have little understanding of the operation of the construction industry, a significant portion of them are engaged in teaching design oriented courses with little or no industrial experience. Thus, directions for merging professional experience with engineering science in class designs have become a chief concern in engineering educational forums. Solutions suggested or adopted [16] for this problem include:
- urging universities to employ as many faculty as possible with industrial experience
- encouraging faculty development in order to accumulate at least two years of industrial experience
- requiring faculty members to obtain professional registration
- recruiting professionals to teach design courses on an adjunct basis
- inviting professionals to deliver lectures

However, obtaining professional registration does not necessarily equip an inexperienced faculty member with valuable practical experience. On the other hand, although single lectures delivered by practicing engineers can be informative, they often do not emphasize the much needed connection between the theoretical design concepts and professional judgement required in actual practice. Assigning a design course with sizeable engineering science content to an adjunct faculty usually provokes students complaints about inadequate student/teacher interaction and apathy toward engineering science concepts. Part time instructors rarely are on hand a sufficient amount of time to participate in program development, thus throwing an additional burden on regular faculty [17].

Some of the following traditional learning methods could not be considered as teaching in “any complex or complete sense” [18]:
- primarily by reading the text
- secondary by doing problems on their own and comparing solutions
- by duplicating the professor’s problem solving

In addition to the standard homework problems and exams, there are several other ingredients suggested [19] that include design projects, group work, basic competency exams, computational environments for simulating and visualizing phenomena, multimedia instructional tools, hands on experiences and students presentations. In parallel, there are other additional objectives as follows:
- Address a wider set of learning styles
- Provide opportunities for practice in group work and learning
- Emphasize communication skills
- Engage students in the teaching and learning process
- Motivate students to continue their engineering studies

Teaching styles

If professors teach exclusively in a manner that favors their student’s less preferred learning style modes, the students’ discomfort level may be great enough to interfere with their learning. On the other hand, if professor teach exclusively in their students’ preferred modes, the students may not develop the mental dexterity they need to reach their potential for achievement in school and as professionals. An objective of education should thus be to help students build their skills in both their preferred and less preferred modes of learning [20]. There are four learning style models that are identified that can assist educators in professing the class.

1. The Myers-Briggs Type Indicator (MBTI)

Students may have any of the 16 different learning style types combined from below:
- **Extroverts** (try things out, focus on the outer world of people) or **Introvert** (think things through, focus on the inner world of ideas)
- **Sensors** (practical, detail-oriented, focus on facts and procedures) or **Intuitors** (imaginative, concept-oriented, focus on meaning and possibilities)
- **Thinkers** (skeptical, tend to make decisions based on logic and rules) or **Feelers** (appreciative, tend to make decision based on personal and humanistic considerations)
- **Judgers** (set and follow agendas, seek closure even with incomplete data) or **Perceiver** (adapt to changing circumstances, resist closure to obtain more data)

Engineering professor usually orient their courses towards introverts (by presenting lectures and requiring individual assignments rather emphasize active class involvement and cooperative learning), intuitors (by focusing on engineering science rather than design and operations), thinkers (by stressing abstract analysis and neglecting interpersonal considerations), and judgers (by concentrating on following
the syllabus and meeting assignment deadlines rather than on exploring ideas and solving problems creatively).

2. *Kolb’s learning style model*

This model classifies students as having a preference for 1) concrete experience or abstract conceptualization (how they take information in), and 2) active experimentation or reflective observation (how they internalize information). The four types of learners in this classification scheme are:

**Type 1** (concrete, reflective). A characteristic question of this learning type “Why?” Type 1 learners respond well to explanations of how course material relates to their experience, their interests, and their future careers. To be effective with Type 1 students, instructor should function as a motivator.

**Type 2** (abstract, reflective) A characteristic question of this learning style is “What?” Type 2 learners respond to information presented in an organized, logical fashion and benefit if they have time for reflection. To be effective, the instructor should function as an expert.

**Type 3** (abstract, active) A characteristic question of this learning type is “How?” Type 3 learners respond to having opportunities to work actively on well-defined tasks and to learn by trial-and-error in an environment that allows them to fail safely. To be effective, the instructor should function as a coach, providing guided practice and feedback.

**Type 4** (concrete, active) A characteristic question of this learning type is “What if?” Type 4 learners like applying course material in new situations to solve real problems. To be effective, the instructor should stay out of the way, maximizing opportunities for students to discover things for themselves.

Traditional engineering instruction focuses almost exclusively on formal presentation of material (lecturing), a style comfortable to only Type 2 learners. To reach all types of learners, a professor should explain the relevance of each new topic (Type 1), present the basic information and methods associated with the topic (Type 2), providing opportunities for practice with the methods (Type 3), and encourage
exploration of applications (Type 4). The term “teaching around the cycle” is originally coined to describe the instructional approach.

3. **Herrmann Brain Dominance Instrument (HBDI)**

   This method classifies students in terms of their relative preferences for thinking in four different modes that are based on the task-specialized functioning of the physical brain. The four modes or quadrants are:
   - **Quadrant A** (Left brain, cerebral) Logical, analytical, quantitative, factual, critical
   - **Quadrant B** (left brain, limbic) Sequential, organized, planned, detailed, structured
   - **Quadrant B** (right brain, limbic) Emotional, interpersonal, sensory, kinesthetic, symbolic
   - **Quadrant C** (right brain, cerebral) Visual, holistic, innovative

   Engineering professors are, on the average, strongly Quadrant A dominant and would like their students to be that way as well, according to Edward and Monika Lumsdaine in a 1995 Journal of Engineering Education article. Most engineering instruction consequently focuses on left brain Quadrant A analysis and Quadrant B methods and procedures associated with that analysis, neglecting important skills representative of Quadrant C (teamwork, communications) and Quadrant D (creative problem solving, system thinking, synthesis, and design). This imbalance is a disservice to all students, but particularly the 20 to 40 percent of engineering students with preferences for C and D quadrant thinking.

4. **Felder-Silverman Learning Style Model**

   This model classifies students as:
   - **Sensing learners** (concrete, practical, oriented toward facts and procedures) or
   - **Intuitive learners** (conceptual, innovative, oriented toward theories and meanings)
   - **Visual learners** (prefer visual representations of presented material – pictures, diagrams, flow charts) or **Verbal learners** (prefer written and spoken explanations)
   - **Inductive learners** (prefer presentations that proceed from the specific to general) or
   - **Deductive learners** (prefer presentations that go from the general to the specific)
Active learners (learn by trying things out, working with others) or Reflective learners (learn by thinking things through, working alone)
Sequential learners (linear, orderly, learn in small incremental steps) or Global learners (holistic, systems thinking, learn in large leaps)

Most engineering instructions have been heavily biased toward intuitive, verbal, deductive, reactive, and sequential learners. However, relatively few engineering students receive an education that is mismatched to their learning styles. This could hurt their performance and their attitudes toward their courses and toward engineering a curriculum and career.

In a study at University of Western Ontario using Felder-Silverman Learning Style Model, results have suggested that professors could improve engineering instruction by increasing the use of methods oriented towards active learners (participatory activities, team projects), sensing learners (guided practice, real world applications of fundamental material), and global learners (providing the big picture, showing connections to relate material in other courses and to the students’ experience)

Another study at North Carolina State University, it has been stressed that active learning experiences in class reduced the time spent lecturing. In homework assignments the traditional formula substitution problems have been re-written with open-ended questions and problem formulation exercises. This has resulted in an unconventional way of problem solving. Extensive cooperative learning has been used along in getting the students to teach one another rather than rely on the lecturer exclusively. It has been found that teaching the full spectrum of learning styles improves the students’ learning, satisfaction with their instruction, and self confidence.

Ability to communicate
Professionals felt that problems of entry level professionals in written communication are serious. “Their communication skills are not good – they’re less than not good, they’re really bad. In most cases, they’re not strong communicators, and ‘that is a problem
because we’re trying to get some of our technical people to participate in [client] presentations.”

Professors have been aware of this problem and one of them had been for the last fifteen years, incorporating extensive preparation for videotaped oral presentation into his coursework. To prepare students for the workplace, the professors have to find ways to incorporate teamwork and communication skills into the technical and academic curricula. The university’s placement ratings improve with the obvious benefit of making the program more attractive.

### 3.3 Roles of Professionals

In a survey in 1992 by the National Society of Professional Engineers (NSPE), there were approximately 74% of the respondents placed high value on preparedness for design work, but only about 35% of respondents thought that new engineers were well prepared in design. This disparity of 40 percent indicates that there is a significant level of dissatisfaction with design in engineering education.

In spite of these views, when respondents were asked which areas would merit more time in a revised curriculum, only 27 percent mentioned design as first or second priority. Conversely, 30 percent of the respondents mentioned an increasing emphasis on basic science as first or second priority, perhaps reflecting a belief that design cannot be realistically taught in schools.

A 1989 survey of civil engineering education suggested that there has been no substantial increase in an emphasis on design in civil engineering [14]. Specifically, the survey results has indicated that the median percentage of the bachelor’s degree curriculum devoted to design had increased to 16 percent in 1989 after dropping from 16 percent in 1978 to 14 percent in 1985. The difference is not significant and merely indicates a return to the 1978 level.
The report states that one of the most desirable goals is to motivate the students in gaining the “ability to identify and define a problem, develop and evaluate alternative solutions, and effect one or more designs to solve the problems”. Part of the report states: 

*Reaching such an attribute has always been a fundamental aim of engineering education... [but] evidence is mounting that engineering curricula nationwide are doing an adequate job of attaining it...*

*Undergraduates and graduate engineering education is the foundation for successful practice, effective teaching, and relevant research in engineering design. The current state of that foundation is attested by employers who find recent engineering graduates to be weak in design. Reasons for the inadequacy of undergraduate engineering design education include weak requirements for design content in engineering curricula.*

There have been three categories of professionals being interviewed, the industry, consulting, and government [1]. Comments about some non-technical qualities of the students that these professionals find missing:

**Ability to work on a Team**

One professional said, “We spend a lot of money every year [on seminars to improve people skills]. Not only are you dealing with external clients, but also you’re dealing with members of team here. You have to be able to listen and communicate and coordinate your tasks with the other discipline. (Consulting)”

In addition to working with a formal team for a specific project, there are aspects of teamwork that arise in other workplace situations:

“One important thing in this early entry level period is for individuals to learn how to take their skill and interact with somebody who has the hands on skill but can’t talk their language – make it work. (Industry)”

**Awareness of workplace expectations**

The professionals talk about the entry level position as a learning experience – they talk about things like working with a mentor, taking courses, recognizing that you can learn
from the “people you serve [for example] the guy you’re inspecting” (government),
learning regulations, coming to “an understanding of what [the company] is all about”
(industry).

Being more specific, this group of professionals say that today’s students “are learning
design and construction, and they’re really not learning about surveillance and oversight
over that process” (Government); that they need to know about budgets, ethics, and
liabilities (Consulting); and they don’t even know what their professional options are –
such as the differences among process, product and project engineers (Industry)

Reparation methods
- On the job training programs for new employees which can range from informal
  encouragement to learning from who are more experience, to a mentor/buddy system
  and occasional workshop, to a formal two-year training program.
- Cooperative education (co-op) and internship programs for colleges provide mutual
  benefits for that students get valuable hands-in experience and the employers get high
  quality temporary employees (often used for special, short term projects) and an
  unparalleled opportunity for recruitment. The professional can spend less time and
  money for initial training
- There are some members of the committee [1], however, believe that it is unrealistic
  to expect engineering schools to teach design. Based on their own experience, these
  committee members believe that design problems involve too many factors and
  considerations to be taught in school. They believe that design can only be learned
  effectively on the job through experience. As a consequence, engineering practice
  experiences a professional and social gulf between those labeled “engineers” and
  those labeled “designers”.

Many of the professionals feel that students should understand the ways that design
problems that are handled by practicing engineers. Especially the many technical and non
technical factors that need to be considered, the various ways computers are used in
design works, and the types and sources of data needed to solve typical design problems.
Even with the use of computer in design, it is found that most computer-aided engineering software, even computer-aided design software, is entirely oriented towards individual use and analysis rather than a team approach.

Professionals [1] are encouraged to consider the following techniques:

- Improve recruitment methods
  Investigate more thoroughly the schools producing candidates to identify curricula that match needs.
  Test candidates on desirable competencies.
  Depart from the practice of hiring only from professional levels engineering and architectural programs. Recruit from schools of construction and schools of technology, many of which have good quality, highly applied curricula.

- Improve post-hiring practices
  Institute continuing education and mentoring programs after graduation.
  Do more frequent evaluation, tracking employees by college, program, and pre-test.
  Clarify expectations of design and construction employees.

- Communicate with colleges regarding expectations

- Provide internship opportunities for undergraduates

There is another recommendation to include a broader definition of scholarship has the potential to raise the level of debate from who should control the educational process – academics or professionals – to how academics and professionals can work together to train graduates that can contribute to their professions. The objective is a mechanism to foster cooperation between the two segments in working to establish a standard for what the proper role of schools in the preparation of professionals should be. This might make the balance less susceptible to such external forces as current trends in research funding.

Professionals involvement in the education
The professionals can also play an important role in the teaching of a capstone course for all students. An example of this involvement of a local industry entity (i.e. consulting
firm) and the CEES faculty has been adopted by the School of Engineering and Environmental Science (CEES) at the University of Oklahoma [12].

A review of that involvement by the industries found that none of the perceived involved the student’s knowledge of technical facts, but they rather focus on the proper utilization, integration and communication of technical information.

Weaknesses identified by industries [12] are:
- Technical arrogance
- Lack of appreciation for considering alternatives
- Narrow view of engineering and related discipline
- Weak communication skills
- Little skill or experience working in teams

In combining the efforts of the professionals and educators in the teaching classes, all the parties involved reaped some form of benefits as indicated below:
- Student Benefits
  Resume: The sponsor will review the resume submitted by the students to let them know of first hand information from the sponsors what they had to change in their resumes.
  Exposure to professional practice -through interaction with the industrial clients, site
- Professional sponsor benefits
  Development of concepts- innovative concepts from the students
  Graduating engineers- better trained in areas
  Industry driven education- provision of input for the engineering education
- Faculty benefits
  Technical writing skills –still a lacking amongst all graduates
  Public speaking – a lack of experience has been found
  Economics – lack of familiarity with the concepts of the time value of money
  Team approach – a new idea instituted and should be encouraged in freshmen courses visits and on-site presentations.
3.4 Summary

This chapter assists the reader in identifying the roles of the graduate or the educator within the engineering education programs. We have looked at the roles of graduates from the standpoint of various organizations. We have identified some non-technical expectations of graduates as ability to work on a team, ability to communicate, and awareness of workplace expectations. Indirectly, we have provided some answers to the general objective question of “What can the students do to achieve the expectations?”. We have consolidated our efforts by looking at the issues about faculty being research oriented and lacking in practical experience. Moreover, the reader is also introduced to the four learning style models and their possible teaching or learning style in each of them. We feel that with some ideas of what the professional is prepared to do and what they are doing now, the educators would be able to assist the students for the coming expectations. Likewise, the students would be able to understand what is fundamentally important to prepare themselves for the professionals’ expectations.
Chapter 4

Teaching and learning environment

4.1 Introduction

In this chapter, we will attempt to introduce an integrative teaching framework where educators can adopt and which the graduates can benefit within the teaching and learning environment. This chapter will also introduce the use of teaching aids. We feel that this teaching and learning environment could be developed into the additional stage in educating structural engineers in Chapter 6.

Integrating engineering education [21] involves connecting subject material among individual classes and building upon prior student experience. Integrative teaching helps students to better synthesize knowledge; facilities easier access of that knowledge in design; provide motivation for learning; and helps students build a strong foundation for future learning.

There are always concerns that dealt with the tension between the need to teach fundamental science principles and the need to teach students design and provide a “big picture” of systems approach to problem solving. Several of the MIT professors think that the relevance of material they teach and how it relates to other subjects is intuitive. However, the viewpoint of a student would be otherwise.
4.2 Integrative teaching framework

In educational research [22], the constructivist model consists of four steps: the invitation, exploration, explanation, and application. The framework of the model and its steps could be applied to engineering education.

Invitation involves discovering students’ prior knowledge and experience, introducing students to new materials, and providing students with a reason for why they should be interested in the material.

Explanation/validation affords students a more in depth interaction with new material with the students “testing” the material with activities like laboratory experiments, design projects, visiting materials in actual sites, etc.

Application is that action where knowledge in the doing and designing that allows students to strengthen links between theory and practice and make the abstract concrete.

The framework of invitation, exploration, explanation, and application is just one model for teaching in an integrated fashion. However, there are two key aspects of engineering classes that hinder the implementation of integrative teaching: the amount of material being taught and the reward structure.

Many MIT civil engineering professors feel that because the time allocated to a particular subject is limited, and because there is too much material to cover, they need to focus first on the basics before any attempt at integration is made. Yet it could be argued that if material is not integrated, students will not be able to effectively use it. Furthermore, if instructors actively point out real world applications and mention connections to other
subjects, they can help students realize that the scientific knowledge they are learning is relevant and applicable to design.

Reward schemes are only limited to the imagination of both the teacher and the students. Grading could be of many forms: teachers require student journals; base a portion of the grade (more than 5%) on in-class participation and questioning; use problem set questions that require students to relate material to other classes, prior experience, or real application problems; base grades on design problems with open-ended questions; require field trips to civil engineering project sites.

4.3 Teaching aids in classrooms

There are many teaching aids that an educator can use in a teaching environment. However, in this section, two innovative areas of teaching aids are explored namely the use of computer and the concept of "bringing the site into the classroom".

Computer Use [21]
Computer is increasingly dominating the engineering analysis, design and drafting. There is a wide spectrum of ways that computers can be used in education. Computers can and should be used to augment education. However, in order to train students to actually think and engage the program, students must be educated in computer use, not just trained to be technicians of the popular software of the day.

Some quotes from Woodie Flowers [10] exemplify the thoughts. "I do not think that "knowledge transfer" can continue to be the core benefit for schools of engineering. Modern communication technologies, commercial firms, and the more progressive educational institutions will take part of the process out of the classroom. Many have already shown that World Wide Web offers pedagogical advantages over texts, videotaped lectures, and many other media once hailed as great wave of change." "... we emphasize to help students learn to learn, and more importantly, learn to think."
"We must provide students with experiences, self images, and insights that cannot be readily obtained via computer screen or even through an elegant virtual environment."

The assumed uniqueness of the relationship between a model and its real-world counterpart has the even more serious consequence. This is reinforced by dozens of examples and examination questions which yield one correct solution, that the whole process of designing in engineering is believed to be precise and exact, and each problem would seem to have a unique solution – the ‘one best way’. This peculiar training concept for engineering for determining the optimum solution often misled students into believing the “supreme” power of the computer in their design work. This leads us immediately to identify the dangers of being over-reliance on computed results.

**Dangers of over-reliance on computed results**

There are several dangers in using computers, for example the illusion of ease, lack of intuition and wrong judgement.

- Computer use in introductory engineering classes may promote the myth that understanding how to run a computer program is equivalent to understanding the underlying theoretical basis.
- Even though, the use of computer aided simulation can provide some form of experience to students, those who are taught on computers how to input data and read output files may fall into the trap of also relying on the computer for intuition. They may fail to develop for themselves a sense of what a deformed structure should look like under a particular set of loads.
- Just as the senior generation of engineers can intuitively make sense of computer output from years of experience in design and manual calculation, young engineers will have to be educated in their intuitive knowledge of system behavior. This will either have to be done in the context of ready computer output or perhaps engineers will need to be taught independence from computer output altogether.
- In addition, students may come to depend too heavily on computer output and interact with computers as black boxes. Instructors introducing computers in classes have experienced students’ manual problem solving skills quickly atrophying, a loss of the
ability to remember fundamental principles and suffering exam scores. In fact, some instructors believe the losses in student performance far outweigh the possible benefits of computer use [23].

Students and other non-engineering designers such as architects and the public at large tend to believe that stresses, strains and deflections in actual structures can be accurately predicted to any number of decimal places, and consequently, that such calculation techniques are infallible in design. This perception by students are so great that young designers can even give calculated results more credentials than even their most basic intuition. One example is that for the entry engineer relying on the calculations which 'proved' that a bridge design is unsafe to conclude that the bridge would lead to collapse, despite the fact that the bridge had already been constructed and has been in full use.

Such problems are likely to become more frequent and serious with the increasing use of computers in analysis and design. The users of computers nowadays do not generally write the programs themselves and are thus likely to be unaware of any assumptions made. It is also becoming more likely that users are less inclined or less able to check the output of computers, even approximately. Furthermore it has become easier to feed a complex structure into a computer, so people are becoming less and less inclined to think about its behavior beforehand, as used to be the case when it was in a designer’s interest to keep the calculations as simple as possible.

Some of the cases of over-reliance on computers include that of a young designer firmly believing the output of a structural analysis computer program, despite the obvious possibility of a building deflecting into the wind; and of another who detailed a two story frame building with genuine pin-joints because that had been how he had analyzed the structure; and of a third who believed the computer’s output showing the sum of foundation reactions greater than the total weight of the structure and imposed load. Two of these errors had been detected before construction commenced; fortunately for the designer of the ‘pin-jointed portal frame’, there was not much wind before a senior engineer happened to arrive on site and notice the error. However, the danger of similar
errors remain undetected seems likely to grow with the use of computers, and the occurrence of a serious collapse seems only to be a matter of time.

Nowadays, it is heard that on many occasions that the way of checking computer results is to compare them with those generated by another piece of software. So what does one do when they give different results?

The senior engineering designers of the current generation still belongs to an era in which they develop their skills and structural attitudes ‘manually’. The difficulty and tedium of complex calculation forced them continually to seek ways of simplifying their task and to have frequent recourse to their qualitative knowledge of the behavior of structures by way of checking calculations which they knew to be approximate. Some design procedures have been particularly preferred because they approach design in terms which closely paralleled the designer’s qualitative understanding of structural behavior.

Graphical statics and moment distribution method are perhaps the best known of these – they both encourage designers to think about structural behavior almost as if they themselves are the structures having to resist the applied loads.

The younger generation of designer engineers has not developed the understanding of structural behavior in this way. Its is encouraged more and more to concentrate upon the ‘behavior’ of computer based mathematical models of structures which are believed to generate ever more accurate results. The fallibility of such result is seldom acknowledged. The understanding, knowledge and assumptions upon which these methods are based tend to be overlooked or ignored and are likely soon to be forgotten. The faith in the results is tending to become blind faith.

*Obvious advantages of computer usage in education*

In an Independent Activities’ Period (IAP) course on the Design Studio of the Future at MIT, there is general consensus that the greatest advantage of computer use in engineering education is the opportunity for students to be freed from the grind of iterative calculations and share with them a glimpse of the big picture. Additional
advantages included increasing student knowledge and breadth, making education more realistic, increasing understanding, and enhancing student creativity.

**Prerequisites of using computer**

In order to safeguard the users in using the computer, the first prerequisite summarized by the president of an engineering firm as: “I do not want you to go to the computer until you have obtained an answer to the design assignment by non-computer means. Computer work is a small part of design, but new graduates think it’s all there is” [24]

Next, the students need to have a basic understanding of the theoretical foundation of the computer program. Then, the students should understand the assumptions (particularly the limiting assumptions) of a computer program and the limitations of the program itself.

Students should be taught what preliminary calculations show that as analyst, they understood the model before even starting computer computations. Students must know that in using a computer program, it is they, not the programmer who will be liable for any mistakes (programming or otherwise).

**Modeling and Verification**

In modeling, it is critical to teach students how to correctly model systems and incorporate boundary conditions and how to plan out analyses instead of wasting time generating useless data.

Teaching modeling involves teaching students how to look at a system as a whole and how to question the model they are developing; question if they could better represent reality another way; and question if there is a better way to accommodate the design requirements. There are three methods for teaching modeling:

- Using a small, simple, physical model [25]
- Exposing students to designs that have been realized or built and the modeling assumptions that are used in analyzing them [26]
- Requiring students to develop models as they do so, interact with professionals or instructors with modeling experience who would both teach and critique students in the art of modeling.

Students should be encouraged to analyze different types of systems under various conditions; perform parametric studies and investigate model sensitivities to input parameters, boundary constraints and mathematical assumptions [27,28].

**Verification**

In a computer-based structural analysis class [28], it is required for students in all projects to conduct global equilibrium checks based on quick manual computations; they deduct 50 to 80% if students fail to include these checks. Incorporating validation in the classroom can teach students to be critical of computer output and refute the idea that computers are infallible.

Part of the validation process will involve teaching students to first review all input data before doing analysis [29]. Second, expose students to various methods of verification. These include (but are not limited to) using simplified problems for validating computer code; checking the program against already existing solutions; manually checking computer output, perhaps on simplified problems, and using other computer programs to check or validate a solution [28].

“Bringing the site into the classroom”[30]

It has been found that student’s lack of preparation for the real world shows, in their senior year, for having trouble in reading a blueprint. Furthermore, novice construction engineers are assumed to “complete” their studies during their first year in practice. Therefore, there is a need to lessen this sense of disorientation.
Recognizing this, a construction engineering laboratory has recently been set up at the Civil Engineering Department, Technion-Israel Institute of Technology. The laboratory includes learning aids as follows:

- Ready-made materials, normally prepared commercially, offer two obvious advantages: they already exist, and they are usually easy to obtain. Some of the materials being used in the labs are:
  1. Catalog
  2. Videotapes
  3. Scale Models
  4. Drawings

- In-house prepared internal material
  1. Equipment data sheets
  2. Slides: photographs of sites all over US.
  3. Informational drawings: whether informative or not?

The laboratory’s learning aid has been used for the learning activities in classroom. For example, for a class assignment: sets of blueprints, specifications, and quantity bills are used to define and calculate the project’s specific requirements (e.g. dimensions, weights, and locations of elements and materials to be conveyed). Site constraints (e.g. topography, access road layout, and proximity of existing facilities) are also learned from blueprints, maps, and specifications. The stock of available equipment of which technical data and features must be considered, is represented by the commercial catalogs and videotapes. Thus a realistic simulation is achieved.

4.4 Summary

In this chapter, we have found that integrated teaching can motivate the students to a full learning experience. The four steps of invitation, exploration, explanation, and application could be applied to engineering education. We have also reviewed the use of computer as a teaching aid. In particular, attention has been focused on the advantages
and disadvantages in the use of computer. We have also looked at the basic prerequisites in using computer along with the modeling and verification aspects. In addition, we have also identified a special way of bringing the site into the classroom to instill some form of realism in the engineering education. With this chapter, we have provided the basic teaching and learning environment that can be used in the development of an additional stage in educating structural engineers.
Chapter 5
A criticism of structural engineering education

5.1 Introduction

At this point of the thesis, we will deviate from the discussion on educational program to introduce some technical professionals' expectations of the entry level engineers. The reason for such deviation is to provide a practitioner approach in establishing an understanding of some technical expectations of an entry level engineer. Besides, information leading from this chapter would be equally important to substantiate the development of the next chapter.

It is important that the structural engineer grasp the knowledge pertaining to design. In most aspects of the construction industry, the term 'design' associated with structural engineers is numbers related, such as the stresses, deflections, etc. Without any doubts, the ability of the structural engineer to model and analyze the model of the structure is imminently important. Nowadays, with the aid of computer, such modeling of structure is easily managed by engineers and sometimes even junior engineer. However, the fear in over-relying on the computer application is the source of another problem. Many of the senior engineers in the industry are trained with 'lesser computed' methods. They trust more on manual calculations. These senior engineers have developed a strong sense or "feel" for structural behavior. The current junior engineers lack this 'sense'. Many of the
senior engineers believe that the ‘harsh computer less’ environment has provided them with opportunities as well as the need to adopt ‘rules of thumb’ in their design works and with experiences, they have become proficient. They have also agreed that this “feel” for structure comes after many years of experience and is not easily taught to junior engineers. Many of them believe that rules of thumb come quite natural to them when they encounter any design problem and as such it is difficult to explain to a junior engineer about their ‘inspirational moments’.

Students are now taught mainly about the mathematics and engineering science relevant to engineering works, but not how to use this knowledge in design. Nor are they taught the importance of other types of engineering knowledge in design, such as qualitative understanding of structural behavior, precedent, empirical data and rules (‘rules of thumb’). In addition, they are poorly educated as to the limitations of theory, how and when its efficacy in design might be suspected, and when it might need to be supported, for instance, by tests on physical models.

Many junior engineers feel lost when they start their profession. There is a need to incorporate qualitative understanding of structural behavior such as empirical rules in structural engineering education. Young engineers will have to develop an intuitive knowledge of system behavior. This chapter will also identify various forms of engineering knowledge applicable to structural engineering. We will distinguish the differences among theory, practice and design. The “feel” for structure as a structural engineer is discussed and the application of rational design procedures for the work of structural engineer is also described and evaluated.

5.2 Ideology of structural engineering

The structure of a building, whilst fulfilling the specific role of giving strength and stiffness to the building enclosure cannot be uncoupled from the design and construction as a whole. The structural elements form part of building elements and the structural
designer needs to know how ‘the structure’ will relate physically to the other elements, often designed by other members of the design team. During the design process the structural designer often has to modify the structural design to accommodate design developments carried out by other members of the design team.

Throughout the history of structural engineering, there have been three main areas of skills which different designers possess to different degrees:

- The ability to come up with innovative and highly appropriate structural solutions to both familiar and new problems
- The ability to take design from a basic or general concept to the level of fine detailed design
- The ability to create designs which can be built easily and cheaply

This has clearly differentiates engineering from any other profession.

Misconceptions about structural engineering

It is many people’s misconception that structural engineers merely ensure that an architect’s design for a building will stand up and be safe, and this involves a process requiring calculations of stresses and deflections alone. Indeed, engineers do that, but they do much more besides. There are many other specific skills that a structural engineer might possess (refer to Appendix A). Architects and structural engineers usually work together closely during the entire process of developing the design of a building from the early concept stage through to the level of fine detail and construction. They are equal parents to their child and, like all parents, they make different contributions. The close and yet diverse contributions from both architects and engineers have led to many innovative structures built.

Another misconception is that engineering design has a certain inevitability about it – if it is based on scientific laws, how can there be any room for choice and subjectivity? This notion are reinforced by the rational explanations and calculations which engineers need to produce as justification for their various decisions. But the convergence and objectivity of these later stages only commence after the earlier, highly divergent and turbulent stage
of the design process during which all members of the design team propose, compare, reject and develop alternative ideas.

The engineer creatively combines various threads of thought, some contradictory and incompatible, to arrive at a specific product. These threads arise from many origins – an understanding of engineering science, knowledge of the behavior of the actual materials and structures, experience of the construction process and, of course, the individual’s own success and failures. The engineer who is a master of the scientific principles can have great freedom in making use of different materials, structural actions and construction techniques, and can bring as much creativity and subjectivity to a project as an architect.

**Structure of buildings**

Some structures such as cranes or bridges primary and only role is to carry the loads, whereas the structure of a building must meet other requirements. Frequently the structural designer of a building is only one member of a team. Each member of the design team is primarily concerned with different aspect of the overall design. A building is essentially a space that is protected from the natural environment and is constructed and plays the role of giving the construction sufficient strength to withstand the loads to which the whole building is subjected. These loads are caused by natural phenomena such as wind and gravity and by the use of the building.

Structures are part of a building and cannot be conceived in isolation but must be conceived as part of the building design. However, they play a specific role, that of providing strength. Whilst the structure of a building is part of the construction the concept of the structure is not. Frequently design decisions are made before the structural concept is clear, often the physical size of structural members is considered without reference to an overall structure. The physical presence of the structure in a building is the concern of many members of the design team as it affects their design decisions. Often the role of structural design is seen as arriving at the physical size of structural elements rather than considering an overall design strategy.
So the structural designer of building structures is faced with a difficult task. Not only is the structural design part of a whole, over which he or she has no direct control, but the size and appearance of individual parts if often prescribed by others who have no concern for their structural action.

5.3 “Feel” for structure

We all have a “feel” for structure, but we cannot experience it directly like a smell. We can feel load through our muscles, although this can be misleading since our muscles get tired whereas loads and structures do not. We can sense movement too, either visually or through our sense of touch and the sensational feelings in our limbs. It might be argued that we can sense stress in some circumstances: a force applied by a drawing pin will cause pain but by a thumb will not; yet a sharp needle can sometimes penetrate the flesh without causing pain.

We all experience structures in other ways too. We can see the materials they are made of, how strong they are and how light. We can see their form, both in man-made and naturally occurring structures such as trees, shells and flower petals. At one level, we perceive these simply as geometrical shapes; at another we might embed them with qualities of cultural, historical or psychological significance. We might also interpret them in terms of the concepts of engineering science. To see a structure in geometric terms is to be able to build a mathematical model of it and be able to manipulate and test it in an abstract way.

Even two engineers may differ – one may describe a structure as a stiffened arch, another as a curved truss (see Figure 5.1); they might even use different mathematical models to investigate its structural behavior.
Architect and engineer will, then, interpret what they see in a structure in different ways and these differences are not unimportant. Their concept of structure is different and this affects the very thoughts and ideas they are likely to have.

Engineers, in addition, have their own set of preferred geometric forms which have their origins in the mathematical models found in structural science. An I or an inverted T-shape are efficient cross sections for a beam; depending on the material and how it is manufactured, efficient cross sections for a column might be a solid circle, a tube, an H or a cross. In order to use minimum amount of material, beams and columns should taper as a parabola or paraboloid from their centers to the ends. Trusses need to be made up of triangles. Some of the triangles can be of identical shapes and sizes while others may be varying shapes and sizes. Suspended structures (and arches, inverted) features catenaries or parabolas. Shells are usually made in the form of paraboloids, hyperboloids or hyperboloid paraboloids, but may also be elliptical, spherical or cylindrical.

A common thread among these structural forms is that they seldom occur naturally to architects – and even if they do, they are likely to be favored for their geometry rather than their structural efficiency. Another feature they share is that they can all be defined using relatively simple mathematical equations. This is essential if the engineer is able to
build the mathematical model that will be needed to try to predict their structural behavior.

Without a reasonable knowledge of the mathematical models used to represent loads, materials and structures, only a limited understanding of structures and their behavior is possible. It would, for instance, be difficult to appreciate the significance of the difference between statically determinate and indeterminate (redundant) structures. Generally, the ideas that particularly stiff parts of a structure tend to “attract” load away from other less stiff load paths through a structure. This statement is not a mathematically rigorous statement, but it does guide design. For an example (see Figure 5.2), when a fixed ended beam with varying stiffness is loaded uniformly, the bending moment diagram for the beam changes with the beam’s stiffness. It can be seen as that the stiffer beam’s end has attracted more loads from the middle portion of the beam (lower stiffness). This “attraction” of load is clearly illustrated here with the force being the bending moment of the beam.

Figure 5.2 A uniform stiffness fixed ended beam and a varying stiffness fixed ended beam
Basic conceptions of structural behavior (truss, beam, arch, etc) are one thing, but most problems tend to arise at the level of detail – the minute detail. A cable is one of the easiest structures to conceive, but making it work – in terms of achieving the required strength, stiffness and bending properties, having suitable dynamic behavior, corrosion, creep and fatigue characteristics, and manufacturing it to precisely the right length and fixing the ends – is a complex matter which few architects can, or need to, understand.

It is thus inevitable that an architect largely unaware of many of these complexities, will make choices which sometimes do not make for easy and cost-effective structural solutions. Perhaps most serious are the problems they can inadvertently cause by not understanding the significance of such secondary effects as stress concentrations and eccentrically applied loads that can make a structure prone to torsional instability.

Feel of an engineer
On the other hand, an experienced engineer simply knows – “feel” – the nature of the relationship between floor span and depth, between the shape of a structural section and its deflection, between the rise of an arch and its stability and outward thrust. With partially “visualizing” and partially “feeling” the structure, the engineer can imagine all the different consequences of changing column spacing, floor structure, a material, or the relative dimensions of members. The impression is of an imaginary object that is almost alive, much in the way that drivers of steam trains and old car feel that their machines have their own character and behavior. Much of this type of engineering knowledge cannot be written down and cannot be learnt quickly; it has to be built up gradually and through direct personal experience. The structural engineer will see a structure as something that has to be built easily, cheaply, and quickly, and in a manner which is stable and safe at all times during construction.

Structural design methods
Approaches to structural design representing different attitudes of mind. One way is to design structures by making imaginative use of existing bodies of engineering knowledge
and relatively tried-and-tested structural solutions. The results, needless to say, are not likely to be highly innovative but may nevertheless suit a particular building project. Many designers – both architects and engineers – believe that the best way to achieve good buildings is to go back to first principles and create designs by a combination of inspiration and logic.

Articulating what one is doing is a valuable way of raising one’s own self awareness, and in this lies the road to improvement and the building of self esteem. In this light, it is interesting to wonder why architects tend to talk about their design process more than structural engineers [31]. Figure 5.3 shows how one engineer (Frank Newby) has expressed his view of the engineering design process.

Before the engineer can do any calculations to test a structural idea, a mathematical model of the whole structural system must be created. This model is a combination of three separate and entirely independent primary models – a model of the loads, a model of the material(s) and a model of the structure (see Figure 5.4). The composite model becomes a set of relationships between the various elements of the three primary models – rather like the empty shell of a computer spreadsheet – into which particular values can be entered and certain results calculated. In this way the model can be made to behave and its response studied for a range of stimuli.

The primary models of loads, materials and structure are all idealization and simplification of the real world, and the behavior output of the information contained in the primary models, not of their real-world counterparts. Wind tunnel tests and the building and testing of prototypes or accurate scale models can give valuable quantitative information about loads, stresses and deflections of structures. These will complement and may confirm the predictions made from purely theoretical ‘tests’. Nevertheless, even such physical tests rely on various theories of engineering science for their interpretation and application to the full-size real structures.
Proposed Structural schemes

Developed scheme

Configuration process

Detailed drawings
Contract documents

Erection of building

Initial design process

Integration process

Feedback

Structural climate
- availability and quality of materials
- availability and quality of workmanship
- ground conditions
- weather conditions
- local building regulation

Architect’s brief
- client and planning requirements
- flexibility of space and loading
- life of building
- allowable building time and costs
- proposed mechanical services, etc.
- architect’s concept of the building

Store of information
- properties of materials
- deformation and stress
- characteristics of structural systems and foundations
- capabilities of structural analysis
- methods of manufacture and erection of structural components
- new developments and trends in the building industry
- experience of the integration of structures in architecture
- costs of construction

Figure 5.3 Engineering Design Process by Frank Newby [31]
Prediction of actual stresses, deflections, etc.

Figure 5.4 Flow chart for development of structural system [31]
Qualitative account of an engineer

To think qualitatively about the behavior of a structure is to use one’s imagination and to use it no less creatively than a musician or artist producing ideas out of her head. In addition to the imagining of a geometrical shape or type of material, which can be done largely from memory, there is also the possibility of carrying out experiments in the mind – what physicists call ‘thought experiments’. This type of process clearly calls for more than mere memory – understanding is also necessary. There are thus several ways that an engineer might think about a structure and its behavior, for instance:

- Imagining the geometrical aspects of a structure
- Direct knowledge of structural behavior
- Thinking about possible structural behavior
- Understanding about structural behavior
- Implicitly knowing how to make use of engineering knowledge

The above forms the basic capabilities that an entry level engineer requires. When he/she is able to conceptualize the above, he would then be able to appreciate the behavior of structure.

Of particular value to the engineering professional is the careful observation of full size structures under load. The most dramatic source of direct experience is the detailed study of structural failures. This can, of course, yield an accurate understanding of how a certain structure behave in a certain set of circumstances and can highlight ways in which a structure can fail which the designer had perhaps overlooked. However, failures do not necessarily tell much about how a structure had been behaving before failure.
In general, the engineer needs the ability to anticipate the manner in which an engineering structure might behave when it is loaded in various ways, although qualitative, can be extremely sophisticated – for instance:

- The manner in which the structure will deflect or tend to move
- The ways in which it could fail and eventually collapse
- The direction of forces acting on fixed supports
- The approximate degree to which different parts will be differently stressed or will attract load
- The relative importance of different types of structural action (e.g. arch action in truss bridges and beams)
- The interaction of different components of a structure
- The manner in which different loads, applied simultaneously, can aggregate particularly severely

This type of skill shares with other ‘doing-skills’ such as painting and playing music by ear, the characteristics of being very difficult to explain. Designers often describe how a structural idea or solution to a problem can appear as if from nowhere, like a tune in a composer’s head, and only afterwards be adequately justified. When asked to explain how one knows that a certain structure will deform in a certain ways, the answer might be something like that: that is how structure behaves; there is no basic explanation. It is perhaps for this reason that phrases such as ‘structural intuition’ and ‘a feel for structures’ are often used.

Two differently conceived structures forms with the same functions

Two quite different structures could have been chosen for a single story space [32]. Both structures as shown in Figure 5.5 and 5.6 are structurally feasible as they have been conceived with an understanding of the overall structural behavior. Many other structures could be chosen to support the enclosure of this simple space.
Figure 5.5 shows a structure comprising of portal frames along the short span of the space with the lateral loading supported by wind bracing. Another structure could be 3-D space frame roof systems supported by perimeter columns (pin jointed) and the corner columns constructed such to resist the bending moments from lateral loading as shown in Figure 5.6. In Figure 5.6, the lateral stiffness provided by the wind bracing in Figure 5.5 are replaced with cantilever corner columns that are reinforced by the trusses. In so doing, the perimeter columns can be designed to carry only the vertical loads from the 2-D space roof. The choice of geometry is fully based on an understanding of the overall structural behavior.
Figure 5.6 Alternative structural system for single story space
5.4 Forms of engineering knowledge

In the classification of engineering knowledge, the word 'theory' is variously used to include any or all of the following:

- bodies of mathematical knowledge
- mathematical theory of, for instance, elasticity
- theories of strength of materials
- structural analysis
- mathematical modeling of structures, loads and materials
- empirical data and rules
- empirically derived constants used in design calculations

There are some subtle differences in the various forms of knowledge that an engineer can acquire through their engineering education or by merely working experiences [33]. The best way to identify the differences is to define the various forms of knowledge clearly.

Knowledge related to the quantitative aspects of engineering is itself of several kinds: empirical, explanatory, or mathematical. This classification will enable the means by which design procedures are able to discharge their functions to be better understood.

Empirical knowledge

Empirical knowledge is gathered from experience and experiments in the real world. Empirical knowledge can be qualitative and quantitative. General speaking, qualitative knowledge precedes the quantitative, especially since the latter depends also upon knowledge of one or another branch of mathematics. Strictly speaking, any quantitative empirical knowledge depends for its meaning or interpretation upon the existence of a theory about the world.

Empirical data

Empirical data represent the most basic form of knowledge which can be derived from experience of the real world. Each datum (atomic fact) is, strictly, unique and isolated
since it cannot be related, a priori, to any other datum. Thus, properties of materials and of artifacts made of these materials are, when taken in isolation, simply facts and can hardly be considered as ‘properties’ at all. As such, empirical data alone have very little significance. Their utility is enhanced when they form the basis of various generalizations.

There is one rather special example of a piece of empirical data which does almost have a life of its own – the empirical constant. This is a device developed at least as early as the late 18th century to allow general laws to be modified by an appropriate amount to suit certain particular circumstances. A typical example is the factor by which the general formula relating the maximum load a beam can carry to its dimensions is modified to suit different materials and shape of beams. It is well known long before the development of quantitative theories of bending that the strength of a rectangular beam varies with its length and breadth and the square of its depth. This law had been made useful to designers by the simple application of a numerical factor (an empirical constant) that depended upon the material. Empirical constants are still widely used by engineering designers, especially in those areas of design for which appropriate scientific theories are inadequate or too complex for day-to-day use, and perhaps the most common of these is the ‘safety factor’ concept.

**Empirical laws**

An empirical law constitutes a grouping of data. This may be collected through random experiences and observations or by means of a series of deliberate tests designed to generate grouped data and thus yield a useful law, although strictly speaking, it is not possible to discover a law without first having a hypothesis that one might exist and is ‘awaiting discovery’.

Qualitative empirical laws relating to structural engineering include many which are common knowledge (and have been for millennia) and yet contain the germs of sophisticated engineering ideas, for example:

- A plank bridge is at its most vulnerable when the load it supports is in the center.
The deflection of a plank bridge increases with load and the length of span and decreased with increasing depth and width of the plank; the deflection also depends upon the material of which it is made.

A wall one brick thick is easier to overturn than one which is two or three bricks thick.

Long slender timber props are prone to buckling; short ones are not.

More sophisticated empirical laws had been often observed by craftsmen and builders:

- Semi-circular arches made of thin voussoirs were not as strong or stable as one made of deep voussiors.

- Diagonals in bays near the ends of a cross-braced truss bridge suffer great compression those in the center.

- Two planks make a stronger and stiffer beam if they are joined together by glue or pegs to prevent them sliding relative to one another.

Empirical laws of the quantitative kind had already reached great sophistication and usefulness at least as early as the 1730s. Musschenbroek published many results of tests on beams and columns of many types of materials and presented the results as generalized laws, dependent on the dimensions, raised to the correct powers, and modified only by empirical constants according to the material [33]. One application of such laws is in the area of dimensional analysis.

**Empirical rules**

Empirical data and laws are, in themselves, of no direct use in the process of description in a design procedure since they contain no prescription for human action. A designer can only make use of the information contained in an empirical law if there also exists (or could exist) a corresponding rule which indicates that following a certain course of action will lead to a certain result. A rule can thus be the converse of the process by which data are gathered together to yield a law. The empirical law is used within a rule to generate data as part of the process of description within a design procedure.
Empirical rules have often been referred to as ‘rules of thumb’ and, in this more familiar phrase, are associated with approximate results and with the days before ‘scientific design’. Yet present day Codes of Practice still contain many rules for the use of empirical data based solely on the accumulated experience of the civil and structural engineering industry concerning what is known to work. Examples include recommended working stresses, factors of safety, the use of particular mathematical models of structures and the many empirical constants (such as the end-fixity of beams and columns) which appear in conjunction with mathematical expressions to modify theory for use in practice.

5.5 Bridging the gap between theory and practice

The conventional way of dealing with the problems of the gap between the theory and practice has been to talk of engineering and design as 'applying theory' and 'putting theory into practice', believing that such phrases solves the problems. So popular are such phrases that their precise meaning is nowadays seldom discussed or questioned. The word 'practice' may refer to any or all of:
- construction of buildings
- design of buildings or engineering works
- laboratory experimentation
- testing of actual whole or part structures
- establishment of empirical data and rules

Typically, the results of academics' and researchers' work in engineering science is presented in a form which is believed to make it clear to their practicing colleagues what to do with it. They discuss the theory and the practice of their laboratory experiments and tests, generally proclaim a close agreement between the two, and assume that the results will therefore be of immediate use to engineering designers. The pages of the Proceedings of the Institution of Civil Engineers (to take the longest running example) have been filled with such papers for nearly 150 years. They have also, however,
-contained many a contribution to the discussion of such papers from 'practical men' who have complained at the inappropriateness of the scientists' work.

In general, this type of solution to the theory/practice gap has been unsatisfactory, since it has failed to help the engineering designers understand just how the theory can be used outside the laboratory. Simply using phrases such as 'putting theory into practice' without explaining them, does not constitute a useful bridge between theory and practice. How is an engineer know when and how to 'fill the scheme out with his own thoughts' or to take notice of 'his own good thinking'? Generally, the possible role for theory in the practical side of engineering is not elaborated.

There is yet another problem with the view that design is matter of 'putting theory into practice'. It excludes the possibility that design (or 'practice') could be dependent upon types of knowledge not included in the category called 'theory'. Yet every practicing engineer knows that design depends upon and is influenced by many other types of knowledge. Some examples are:

- Rules of thumb
- The numerous empirical data and rules associated with Codes of Practice
- The properties of particular materials
- Factors of safety
- Intuitive knowledge of structural behavior
- Experience
- Engineering judgement

Such knowledge does not easily fall under the heading 'theory' yet neither does it really belong to 'practice'. It appears to be caught in the same 'excluded middle' as design and, like design, is thereby often rendered invisible.
5.6 Use of design procedure

The notion of the design procedure is not intended to encapsulate or describe the very act of creation by a design engineer at work. The design procedure can be thought of as beginning with or being triggered off by an act of creation – the conception, so as to speak (the French use this very word). Precisely how and why a structural engineer chooses or conceives a particular structure for a particular purpose is a process, if one can call it that, so vague and individual that I doubt if it is possible to study it at all. The process varies from person to person, and project to project. Sometimes, it might be a flash of inspiration; at others it might be a choice following the detailed investigation of several different options; on other occasions it may be a gradual refinement of a single basic idea.

At the conceptual stage of a proposed structure, even the most transient idea is appraised and ‘tested’ in a split second – the way it would carry the loads imposed on it and deflect, its likely suitability to purpose, its appearance, various consequences concerning type of material, time and cost of construction and so on. Such processes are the very essence of what it is to have experience and use it – just as most people can ‘see’ five or six objects on a table with having to count them – the process is almost unconscious. At this early stage it is often possible simply to know that an idea will work in principle and to know that it will be possible to develop the idea and fully work out the details.

However, sooner or later during the development of a design, it will be necessary to formalize the process, to describe the structure using linguistic concepts, to describe it in ways which will enable other people to grasp what the idea is and consciously to work out consequences of a certain choice which are beyond the capacity of a human brain alone. It is at this point, which will vary from person to person and project to project, that the more rigorous stages of developing a design start. Here begins the main body of the design procedure.
The design procedure for a structure is the series of steps which need to be taken in order to develop an idea for a proposed design from its conception to the production of its formal description and justification. For a type of structure which has been designed before, the design procedure will probably be familiar to the engineer in most of its detail. For a new type of structure, however, not only will the engineer face the task of creating a new design, but will have to develop a new design procedure as well. Indeed, this is likely to be a major part of the work and involve just as much creativity as the conception itself.

Two important consequences of the notion of the design procedure are that it is possible to produce very similar structural design using different design procedures and that similar design procedures can lead to significantly different structures – there is no logical connection between the two. Major differences between designs are most likely to arise from different basic conceptions and from the many outside influences (clients, architects, etc.) not as a result of the design procedure itself.

It is worth mentioning that published examples of design procedures are rather rare than might be expected or hoped for, compared to the plethora of material devoted to engineering science. One reason is perhaps that designers are engaged in a process of performing a skill which, like swimming and playing a musical instrument is very difficult to describe. There is also perhaps a certain reluctance for practicing engineers to writing down their thoughts and methods, a phenomenon which has been called the engineer’s ‘papyrophobia’ [34].

Two of the published examples of comprehensive design procedures which the author finds both informative and ideal for students to appreciate the use of flow chart and design procedures are attached in the Appendix B.
Using the idea of the ‘design procedure’ as a way of focusing on what a design engineer does has led to some of the understanding of the design concepts. Advantages of this method are:

- it strongly distinguishes both the aims and methods of engineering designers from those of engineering scientists; engineering scientists and their methods are closer to their ‘purer’ colleagues in the sciences of materials, physics and chemistry, than they are to design engineers
- it focuses attention firmly onto the activity of engineering design, which tends otherwise to be overlooked, and provides a separate identifiable ‘peg’ on which to hang much knowledge and many ideas which to not mix happily with scientists’ ideas and knowledge
- it identifies a philosophical concept which can serve as a parameter in the subject of engineering design in a manner similar to the role of the theory or hypothesis in science
- it enables design to be recognized as a skill rather than simply a body of knowledge
- it recognizes the fact that a design engineer’s creativity comprises more than the ability to conceive of new structures: a designer must also be able to choose an existing design procedure, or to create a new one which will enable an innovative conception to be realized with confidence
- it acknowledges the influence of all types of engineering knowledge (not only science and theory) upon the design of works of engineering
- it elucidates the role which theory plays in the work of a designer, indicating how - it might help him to think about the behavior of structures and to assist in the description and justification of a proposed design: but theory will not lead directly to designs
- it helps to loosen the ropes which have tended to bond the activity of design exclusively to theory and science; as a result, two popular notions, the idea of ‘scientific design’ and the idea that rational design must lead, almost inevitably, to a single solution (the ‘one best way’), both evaporate
- it is equally applicable to all times in history
5.7 Summary

In this chapter, we have deviated from the previous discussion of engineering education program to introduce the conceptual understanding of the structural engineering. We have reviewed the ideology, opinions and misconceptions about being a structural engineer. As a structural engineer, we have indicated that it is crucial for the entry level engineer to gain a "feel" for the structure as early as possible, even in their early engineering education curriculum. We have looked into the feel of an engineer towards structure and structural design. In effect, the qualitative account of being a structural engineer has been made. In addition, the concepts of empirical knowledge and their application in the field of structural engineering have been explored. The idea of looking at the design of structures with a design process/procedure has been investigated together with some real-life examples in Appendix B.

The deviation from the engineering education program is to allow the graduates in understanding what they can do beyond just basic structural designing. In effect, the concepts empirical knowledge will be used in developing an additional stage for engineering education in the next chapter.
Chapter 6
Additional stage in educating structural engineer

6.1 Introduction

To date, structural engineering education emphasizes a lot on design of structural elements. Very little emphasis has been placed on educating the engineers for the initial design phase. Now, with the basic understanding of the engineering educational system and the criticism of structural engineering education, we would like to adopt a practitioner approach in developing an additional stage to the structural engineering educational for the students. This stage can be applied to the study of the all structural systems such as tall building systems, horizontal span systems, cable structures, and prestressed concrete structures. The proposed stage is shown in Figure 6.1. Most of the structural engineering education emphasizes the procedures shown on the right half of the figure. We emphasize the application of the procedures on the left half of the figure. In addition, this additional stage is to be adopted within the framework of the teaching and learning environment as discussed in the earlier chapter.

Most of the currently available literature on structural analysis and structural design still emphasize detailed structural analysis. The intent of this chapter is to reveal to the entry-level engineers another way of looking at design. The introduction of structural forms will enable the entry level engineer not only to see physically but also to sense the
behavior of structures, thus providing them with higher confidence in dealing with the concept of structure. Furthermore, study of preliminary design of structural elements which are often implicitly covered in the engineering education have been otherwise explicitly defined in Appendix D using various mathematical approaches. Practitioners’ practical approaches have been adopted in many of these mathematical formulas.

Figure 6.1 Structural engineering design procedures

The additional stage first approaches the study of structural engineering from a conceptual level of structural systems. This in many engineering schools now are either done with:

- trial and error method or
- with a methodology of adapting the past design experiences available or
- nothing: Educators assumed that it could be learned implicitly through design projects

Initial tries at structural form have always been based on knowledge, past experiences, rules of thumb, intuition and so forth. We will review the latest approaches that make use of Artificial Intelligence algorithms to predict a structural form for any tall building structure. The next step in the scheme is to guide the students how to conduct preliminary
member design. This preliminary member design can be part of the integration of the development of structural engineering education.

**Tall building design methods**

An entry level engineer often works under the supervision of a senior engineer. As a result, the senior engineer would have normally selected the initial designs for him/her to conduct further detailed calculation. This leaves the entry level engineer following "orders" in designing the structural elements accordingly. However, the possibilities for the entry level engineer to conduct structural design right from the beginning of his career have increased as companies may not be willing to spend extra money and time on training the new entry level engineer. The entry level engineers may be overwhelmed with the initial design procedures as they may not have come across such exposure in their educational curriculum. Therefore, they have to be taught/trained to recognize the traditional approach to a structural design.

We will use the design of tall buildings to illustrate how the education of a structural engineer might be improved to better prepare the entry level engineer for design starting at day one.

There is no absolute definition for what is a tall building. It does not depend on the number of stories, a specific height or proportion of the structure. A general definition for a tall building would be one where the design of the structure moves from the field of statics into the field of dynamics. Then, the effects of lateral loads such as wind or seismic loads on the building have to be considered.

As in all design approach, a structural form is first selected to represent the tall building structure. There are various possible structural forms ranging from framed to tube-in-tube structures with the details provided in Appendix C. With the selected structural form, the engineer needs to establish a preliminary sizing of the structural elements by considering both the loading that the building will be stressed and geometry of the building. With these preliminary sized members, the designer will proceed to analyze the structure.
Normally, the loads from the upper floors are accumulated down to the ground supports through the vertical supporting elements such as columns and walls. This is done with consideration of the attributed area of the vertical supporting elements. In this stage, the live load reduction factor may be introduced. In the case of tall building, the analysis will include the effect of the lateral load in uplifting certain columns or that which will increase the vertical compression loads on the columns. With all the accumulated column/wall loading, the engineer will proceed to design the foundation system to support this vertical load. Once the foundation design is completed, the engineer will then proceed to design the structural elements for the floors. Finally, the loading on the columns and walls are checked to reconfirm the foundation design.

6.2 Selection criteria for structural forms

Cost is usually the dominant factor in optimization process, and it is essential that reasonable optimal solutions is established early in the design process so that meaningful decisions can be made. For the case of tall buildings, there are many structural systems and construction materials that make the task of obtaining the optimum solution difficult for the designer. The relative significant structural cost of tall building construction is in the order of 20% to 30% of total cost. This is not a small sum of money. Any improperly planned and designed structural system may raise the total cost of construction by as much as 5% to 10% [35]. Sometimes, the safety and durability of the building may be compromised because of an improperly selected system.

Tall buildings are typically designed for a variety of limit states, ranging from perception of motion, to strength and stability. The magnitude of external loading used to investigate each limit state varies with that state for example the P-delta effects on structures. In addition, the properties of the structure may vary with the magnitude of the load. Uncertainties in quantifying the properties of a structure at a particular state can lead to greater uncertainties in computed behavior. The application of microcomputer is to
determine the different aspects of anticipated response of the structure which includes the human comfort, drift (damageability), strength, and stability.

In general, the constraints identified during the conceptual phase of structural design can be classified as follows:

- **Spatial**: The spatial constraints define areas set aside for circulation and mechanical equipment, and open areas needed for the functional use of the building.
- **Administrative**: The administrative constraints include the zoning laws and height restrictions on the building.
- **Initial Economics**: The initial economic constraints include cost and construction time.
- **Long Term Economics**: The long term economic constraints are concerned with the operation and maintenance involved in the long-term use of the building.
- **Horizontal Compatibility**: The horizontal compatibility constraints include considerations of compatibility between structural system and the building components, such as the foundations, mechanical systems and electrical systems.
- **Vertical Compatibility**: The vertical compatibility constraints include ease of construction, contractor experience, site condition, delivery, erection method, etc.
- **Functional**: The functional constraints are the major structural constraints concerned with the provision of a load path. There are three possible levels of specifying these constraints:
  - Provide a load path
  - Provide the most direct load path
  - Provide alternative load path (redundancy)
- **Stable Equilibrium**: These constraints ensure that the building or any of its components remains in stable equilibrium in its intended environment.
- **Strength and Serviceability**: These constraints include component and system load capacity requirements as well as serviceability requirements such as stiffness. This formal representation of the constraint is contained in the applicable building and material codes, standards, and design specifications. While at the conceptual phase
the designer must make a deliberate selection of a subset of controlling constraints to be used, the final design must obviously satisfy all applicable constraints. The following will be a brief on some of the important selection criteria.

General
There are many factors that need to be considered in the selection of structural forms. In particularly, it is important to realize that the building has to be delivered to the owner at the end of the day. As such, it is always a necessity to include the requirements of the clients into consideration for all designs at no expense for the structural integrity and safety. Hence, in many ways, a structural engineer’s role becomes that of a salesperson who is proficient in the assessment of the structural feasibility of the building.

For most tall building systems, the weight of columns and walls are directly proportional to the height of the building as the axial load increases linearly as the building height increases [36]. In most instances too, the weight of the floor remains constant. In the typical structure of story height of 30-40, the floor weight and subsequently the floor cost constitutes more than half of the entire building. But as the building’s slenderness increases, the materials required to resisting the lateral loads almost doubled. This is illustrated in the Figure 6.2. The figure shows that for buildings in the range of 20-30 story high, the lateral load increases the material by at most about 10%. But once, the building height is higher, the materials need to resist lateral load almost doubled.

The overall efficiency of structure system as shown above seemed to be directly related to the quantity of materials used, i.e. maximum strength and stiffness using the least weight of material. However, there are other components of the construction that can control the efficiency such as fabrication costs and erection time. From the graph, it seemed that innovative structure systems are applicable to certain height regions only. And again, height may not be the limiting concern as other factors such as building shape, slenderness, functional requirements and particular loading conditions and others can affect the choice of structure systems too.
Building shape

Besides the traditional rectangular prisms that can be rather unsusceptible to lateral drift, there are other building shapes that can allow for greater heights at lower cost and also achieving high efficiency. One such shape is a sloped exterior columns that probably resulting in a truncated pyramid. This shape with a slope of only 8% can produce 50% reduction in the lateral displacement of a 40-story building [37]. Similar to this would be to taper the exterior frame to achieve greater stability. Another possible shape is the cylindrical building form whose net surface area exposed to wind is less such that the magnitude of the wind can be reduced from 20% to 40% of the usual values compared to rectangular buildings. Similar argument can be used for the elliptical building offering up to 27% reduction in wind loading attributable to the elliptical shape. Shape in the crescent shell form such as that of a corrugated steel flooring and folded or undulating roof shells is also efficient in resisting the gravity load. However, this crescent shell form maybe rather inefficient when there is asymmetric loading that causes torsional loading.
Functionality of the building

A building can serve several functions such as office, residential and commercial. Accordingly, the more suitable structural systems can be adopted based on its functions. However, it is possible to combine structural systems which might not be the optimum but still remains as a possible design.

A case study of the various functionalities leads to the following:
- Requirement of fixed functions on every floor e.g. residential buildings – use of the bearing walls structure systems as it optimizes the floor space.
- Requirement of flexibility in the layout e.g. office and commercial buildings with flexible wall partition – use of core systems or combined with other structural systems.

With the evolution of new schemes in the early 1920s where the slender tree-like concrete cantilever structures are used to support buildings more than 50 stories high, the term efficiency has another added dimension. The efficiency of concrete structures is graded in terms of the process of construction in addition to the quantity of materials used. In another aspects, the materials and layout of the structure should not provide the stiffness solely by themselves, the form of the structure must also be searched for, with the help of computers, so as to efficiently reduce the use of materials. The efficient use of materials is explored in the comparison of the codes amongst AISC, BS5950 and EC3 in Chapter 7 of the thesis.

Perception of motion/ human comfort criteria

When a tall building is subjected to lateral loading such as wind and seismic load, the resulting oscillating movements of the building may induce discomfort to the building’s occupants ranging from mild discomfort to acute nausea. There are no universally accepted international standards for comfort criteria. Besides acceleration, there are other factors that may influence the comfort of the occupants such as period, amplitude, body orientation, visual and acoustic cues, and even past experience.
Acceleration has been generally recognized as the main parameter in determining the nature of human response to vibration. For tall buildings, the computed accelerations are based on a certain level of inherent or natural damping in the structure. Values of 0.05% to 1.5% critical have been used for welded steel frames and reinforced concrete frames respectively. The wind load to be considered is one with mean recurrence interval of about 10 years and found to be appropriate for most damageability designs.

Several empirical guidelines are provided in various design codes that specify whether the building needs to be considered for dynamic analysis. For the case of the Uniform Building Code of the United States, a range of criteria is specified with respect to the four different seismic zones. Generally, building taller than 400ft (122m) or slender structure (with the height to the least horizontal dimension of the building as five) should be considered as dynamic building. Similarly, the Australian code defined the same slenderness ratio and also the natural frequency in the first mode of vibration as less than 1.0Hz for a building to be classified as a dynamic building. For the Canadian Code, dynamic buildings are considered to have height greater than four times their minimum effective width, or greater than 120m in height. However, these empirical guidelines might not be applicable to the more radical non prismatic “post modern” tall buildings.

Strength and stability
Strength and stability demands of tall building structures are usually based on code-based wind forces (such as Uniform Building Code) with a mean recurrence of 50 to 100 years. For design, these loads are factored up by as much as 30%. For most tall building design, strength is not the limiting design consideration. The main consideration would be that of the stability for the entire building. There are two basic approaches to consider stability. Firstly, an amplification factor can be used on the first order stresses induced that produce an amplification on the first-order deflections and moments due to the second-order P-Delta effects. Secondly, a detailed second-order structural analysis may be required for more slender buildings to accurately model the structural stability.
Combined creep, shrinkage, and temperature effects

The accumulated creep, shrinkage and temperature effects on the structural elements of a very tall building might cause distress to nonstructural elements even though they might satisfy the structural constraints. One common distress is shown in the misalignment of window frames.

In tall building design, the column length changes within a single story of a multistory building may be small, but they are cumulative which could lead to a large magnitude when multiplied by the number of stories. Column length changes are due to:

- elastic stresses caused by gravity loads
- creep caused by gravity loads
- drying shrinkage
- temperature variations of exposed columns

A comparative study [38] had been conducted in the steel and concrete columns of an 80-stories building. It has been found that high stresses due to the gravity load has caused larger elastic shortening of the steel columns than that of the concrete columns.

Column length changes in a multistory building have both structural and non structural implications. Structural effects are caused by differential movements only and not by total movements. The effects included moments induced into the forcibly distorted slabs or beams, and the accompanying moments in the columns. This physical tilting of slabs can also cause non-structural stresses on claddings, pipes and elevator rails that are attached to the concrete surfaces.

Vertical compatibility

A commonly used term to classify the vertical compatibility constraints is constructability. The Construction Institute has defined constructability as:

"... the optimum integration of construction knowledge and experience during conceptual planning, engineering, procurement and field operation to achieve overall project objectives." (CII 1986). The primary objective of constructability [39] is to
enhance the performance of construction projects through effective input of Constructability Knowledge (CK) to all the phases of the lifecycle of a project, especially during the initial phases: planning, conceptual and schematic design. CK is the result of the combination of a strong technical base with a wealth of construction experience. The development of this knowledge is a complex effort that requires collaboration, and a substantial amount of time and dedication from all members of the complete project team (i.e. owners, designers, and constructors).

6.3 Computer-aided approach in selection process

Before looking at the selection process, it is important to identify the various structural forms that are applicable to tall buildings. The 16 structural forms identified are braced-frame, rigid-frame, infilled-frame, flat-plate and flat slab, shear wall, coupled shear wall, wall-frame, framed-tube, tube-in-tube or hull-core, bundled-tube, braced-tube, outrigger-braced, suspended, core, space, and hybrid structures. These structural forms are presented in Appendix C. It is apparently important for the entry level engineers to know about the existing types of structural forms available for their decisions making. This form of learning from the existing designs has been discussed earlier as one possible avenue in learning. Figure 6.3 and Figure 6.4 show the possible economical heights of the steel and concrete structural systems [40] respectively.

Having decided on the various design criteria on tall building and also an understanding of the structural forms available, the next step is to identify the method of selection of the structural forms. This section deals with the day to day decision-making process of a structural engineer in deciding the feasibility of his design. In particularly, the initial selection of the type of structural form that represents the building can be a challenge to an entry level engineer.
Figure 6.3 Comparison of steel structural systems

Figure 6.4 Comparison of concrete structural systems
The early days of using simplified methods of approaching the building system concept are becoming obsolete. The use of computer to generate three-dimensional building unit to fully utilize the material’s strength seemed to be the trend towards evaluating the efficiency of building systems. It may be concluded that there is an optimum solution for any building type for a given situation. However, in the case of selection of optimum solution, it is inevitable to make comparisons amongst different structure systems. These comparisons are only applicable within a certain height limits. As the scales of the building structure change, the different structures are required.

Structural design is difficult not only because it requires broad knowledge and years of experience in structural engineering but also because it is a laborious process. With the introduction of the computers, there is an increasing move to develop and incorporate the expertise and knowledge of the experienced engineer into a framework where the information will be readily available to others including the entry engineer. We will introduce the entry level engineer the available computer-aided tools in choosing a structural form. The author’s argument will be based on the fact the mere sense of accomplishment can be presented to the engineer when at the end of the day, he/she is able to make use of the tool to select an initial structural form for the problem that she is facing. In this subtle way, the engineer will also appreciate the past designs and initiate the path for a “feel” for the structure as the shape is being formed.

**Expert design system**

The first introductory tool that a design engineer can use is called “expert design system” by applying the expert system technique together with conventional programming. Expert systems can deal with problems involving knowledge, heuristics, and decision making which conventional programs cannot. Expert systems can show their line of reasoning thereby making the design process “transparent” to its user whereas the conventional programs do not offer.
**Expert systems**

Expert systems are defined as computer programs that use logical relationships to incorporate knowledge and expertise about a specific problem area to perform specialized tasks that typically require human judgement [41]. The most attractive characteristics of expert systems that conventional programs do not have can be summarized as follows:

- the ability to explain and justify answers
- the ability to grow gradually by adding new pieces of knowledge
- the ability to deal with uncertain or incomplete information
- the use of verbal or symbolic encoding for knowledge

**The Expert Design System**

The main aim of "expert design system" is to replace the manual work in code checking and to reduce the time and effort in learning and teaching structural designs. The knowledge in the design codes and the design procedures are represented by production rules, which is one of the expert system techniques, and stored in the knowledge bases.

The expert design system for design can consist of three parts: one knowledge base for code checking; one knowledge base for design procedures; and one computing tools bank.

**Code checking KB (Knowledge-Base)**

The code checking KB contains the knowledge in building codes such as standard loads including vertical, wind, seismic loads, allowable stresses including axial, flexural, and shear stresses, and other limitations including height to thickness ratio, etc.

The user can travel along the appropriate path from the root to an end (of a tree), i.e. to the appropriate formula to be used in each particular case.

This knowledge base can be used independently to perform the following:

- pre-design code checking – provided the necessary data such as standard live loads, allowable stresses, and steel requirements
- post-design code checking – checking for consistency of a finished design and the code specifications/standards
- as a subroutine to be called by the design KB or other design program for data
Design KB
In this knowledge base, the user can see the flow of the design and participate in the whole design process while still making use of the high-speed computation. The user can be prompted to specify a piece of information or let the system direct him/her to find the information, either through the code checking KB, the computing bank, or by assigning a commonly used value.

The computing tools bank
The computing tool bank comprises of many conventional computation methods. These conventional programming are able to handle the heavy computation within a short time and yet allowing the students to obtain the member forces. The assumptions made is that the students have sufficient structural analysis knowledge prior so that more time is devoted to the design methodology than detailed calculations.

This expert system approach provides a combination of the excellent reasoning capability in the knowledge base and efficient computation capability by the conventional programming. However, like most expert systems, there is a lacking in knowledge acquisition, flexibility and also learning process within the system. The need to improve on these areas lead to the integration of advanced Artificial Intelligence Tools.

Advanced Artificial Intelligence Tools
Many CAD systems are available in the market but mostly are computer aided drafting, or (and) calculation software rather than Computer-Aided Design. In recent years, the approach to the selection of forms has moved on to the use of artificial intelligence. Some of the artificial intelligence techniques are machine learning algorithms, reasoning mechanisms, traditional artificial intelligence, and distributed artificial intelligence. Combining these techniques with knowledge discovery in databases, data mining research, and conceptual structural design theory and principles, the artificial intelligence techniques can be integrated to enable structural designers to perform conceptual design. The designers are able to avoid limiting themselves to the repeated application of few standard structural configurations by providing the ability to generate and explore a much
wider range of alternative structural solutions. This will help engineers to design high-rise buildings (concrete and steel) in preliminary design with higher quality.

The input variables include information on physical dimensions of the building, building, building use, loading on the building (typical live load, earthquake loading), design fundamental period, design acceleration, maximum tolerable lateral deflection, spans, story height, and other client requirements. Then using the design criteria such as functional, architectural and aesthetics requirements, the conceptual design is accomplished through a series of possible configurations for the structural systems to sustain load distribution for gravity and dead loads based on geometry of the systems.

The preliminary conceptual design can also be adjusted according to the available information pertaining to the selected preliminary materials, such as producing a rough dimensional layout by considering technological possibilities.

The framework for such advanced artificial intelligence techniques approach is normally divided into the following parts managed by a central program manager [42]:

- Determination of the structural system by a machine learning algorithm
  The initial input data are used to classify the structural system with a machine learning algorithm such as Neural Networks, C4.5, and Genetic Algorithms.

- Producing the preliminary schemes by a reasoning mechanism
  The preliminary schemes selected by the machine learning algorithm will be checked against those stored in the storage base with a reasoning mechanism such as Case-Based Reasoning (CBR), Fuzzy Systems and Fuzzy Logic, Heuristic Systems (Expert Systems), and Qualitative Reasoning. The reasoning mechanism will search for one or many cases which are most relevant to the given condition in the database. The cases can be stored in both graphic and text.

- Analytical reduction of the selected schemes
  The program may include a fast analysis system of the mechanical model. One example is the Ordinary Differential Equation (ODE) solver [42] based on the
modeling of the high rise building as a cantilever beam. This concept of cantilever beam model has four advantages:

- it is a continuous mechanic model without mass lumping which is more close to the real condition
- both static and dynamic problem can be unified through the ordinary differential equation:
- The accuracy of the method is controllable under ODE solver.
- It is practical and efficient for all structural systems as the main inputs are the two parameters EI (flexural stiffness) and GA (shear stiffness) which can be easily represented for all structural systems.

- Adjusting and estimation of structural schemes by a reasoning algorithm
  Adjustment of structure dimensions is accomplished through a reasoning algorithm that provides some advises to the designer to adjust the rigidity of the structure. When information on the input variables is limited, the reasoning algorithm allows for a mix of information to be incorporated such that an adapted solution is proposed. With a few iterations, the system would be able to find a satisfying scheme.

With this basic framework, we have found some application such as the M-RAM (Multi-Reasoning Artificial Mind) model and HIPRED (An integrated Expert System for Structural Preliminary Design of High Rise Buildings) model. The M-RAM model is developed in MIT and we will explain its function as a tool for the entry engineer.

**M-RAM modeling**
An M-RAM model [43] aims to assist engineers in the initial stages of the structural design of tall buildings. The structural subsystems being considered are moment resistant frames, braced frames, shear walls, core and outriggers, tubular, and hybrid. These subsystems are not complete structural systems, because they do not specify the building to the extent needed for evaluation of alternatives or input to the next stage of design i.e. analysis.
With the constraints in mind, the nineteen variables involved in the M-RAM interface are classified as follows:

- Height from the street to the roof – meters
- Number of stories
- Number of levels below ground
- Building Use – Choose one of the five available building use
- Frame material – Choose one of the three available building materials
- Typical floor live load – kPa
- Basic wind velocity – m/s
- Maximum lateral deflection – mm
- Fundamental period of building in transverse direction – s
- Fundamental period of building in longitudinal direction – s
- Design acceleration – mg
- Design damping – %
- Earthquake loading – value of C
- Typical floor story height – m
- Typical floor beam span – m
- Typical floor beam depth - mm
- Typical floor beam spacing – m
- Column size at ground floor – cm²
- Column spacing – m

The M-RAM model proposes a design solution through the use of the following steps: proposal, recommendation, intent, artifact, justification, and a computer designer divided in a manager, a user interface, a classification engine, a past experience engine, and an adaptation engine.

M-RAM performs three basic functions: classification, retrieval of past experience, and adaptation of past experience. The classification function uses a decision tree builder to be able to classify the problem. The retrieval of past experience function uses a case based reasoning engine to retrieve matching past experience. Finally, the case adaptation
function uses a genetic algorithm engine to mix bits and parts of past examples creating adapted solution to the design problem being solved.

The M-RAM is identified as not only useful for the professional designer but also a very powerful tool to support structural engineering education. The students can easily assimilate the knowledge learning through past design examples just like the idea on the appreciation of the amazing accomplishments of important engineers in the past as stated in the earlier chapter.

The unique characteristic of the M-RAM model is the possibility of geographical dispersion of the reasoning mechanisms where the reasoning mechanisms are developed all over the world. The idea is to allow the individual agents to be developed and maintained in different parts of the world using a "plug-and-play" architecture leaving the agents where they are developed. The convenient program manager in this case is the link using World Wide Web server.

*Ease and detail operation of M-RAM*

Operation of M-RAM is user friendly as all decision makings are already incorporated in the model. The URL for using M-RAM is currently under construction in the World Wide Web server environment (http://cee-ta.mit.edu as shown in Figure 6.5). The user can then proceed with opening the M-RAM interface that leads to a new html screen with the user entering the various variables for the design. The user can type in the values for the design problem requirements (typing -1 for unknown values within Figure 6.6).

After which, the designer clicks on the M-RAM C4.5 Agent button the attribute values are sent to the server where the C4.5 engine is located. The classification engine will generate three outputs. The first set of data is the input data. The second being the actual decision tree and the third being the classification results. The third output are the most informative as it provides some possible structural systems with a certainty factor, CF attached to each chosen structural system (Figure 6.7). This certainty factor includes the probability of the chosen systems, the highest number indicating the best guess.
The next step is to run the M-RAM CBR agent that will retrieve from its database of building designs, one that is the best fit if compared with the example design (Figure 6.8). The CBR agent will retrieve from its memory the URL of the best matching case and send this information back to the manager, which will display it to the user in the M-RAM interface text area. From this home page, the user can go to a text description of the retrieved case, or the structural information of the case, or the graphical structural details of the project. The main observation is that the retrieved structural information is approximately close to the data input specified earlier.

The next possible option for the user is to exercise the third M-RAM agent that will transform the retrieved solution by mixing parts of partial matches available in the M-RAM’s database and retrieved by the CBR agent (Figure 6.9). This will allow the genetic algorithm to mix bit and parts of the design cases provided to match as close to the original input data requirement. The first part of the output specifies the statistic information about the genetic algorithm and the second part presents the genetic algorithm proposed solution to the design problem.

Until this stage, the user is only required to establish as much information on the 18 variables for initial input data. Then the model will generate the possible outcomes and even suggest some of the variables for the designer to consider.
The design process is an information processing activity. More detailed information about the task itself, about the constraints, about possible solutions, principles, and about known solutions for similar problems is extremely useful in the process of finding a solution to the design problem. This research presents the M-RAM (Multi-Reasoning Artificial Mind) model that aims to assist engineers in the conceptual phase of the structural design of tall buildings by providing him/her with organized and reliable information. This preliminary conceptual design involves selecting preliminary materials, selecting the overall structural form of the building, producing a rough dimensional layout, and considering technological possibilities. Decisions are made on the basis of such information as height of the building, building use, typical live load, wind velocity, earthquake loading, design fundamental period, design acceleration, maximum lateral deflection, spans, story height, and other client requirements. The M-RAM objective is to provide designers with adapted past design solutions with the help of a distributed multi-reasoning mechanism creating a support system to enhance creativity, engineering knowledge and experience of designers. To test the feasibility of the proposed model a prototype of a distributed artificial intelligence system was developed where the Internet was used as a communication backbone among the different systems that implemented the reasoning mechanisms employed. Each different reasoning mechanism was considered as an autonomous module acting as an intelligent agent.

Figure 6.5 M-RAM interface homepage
Figure 6.6 Initial M-RAM interface

Figure 6.7 M-RAM interface with C4.5 agent output
Figure 6.8 M-RAM interface with CBR agent output

Figure 6.9 M-RAM interface with GA agent output
Limitations

It is also important to learn about the limitations of the advanced tool. It seemed that there is unlimited information that can be stored in the knowledge database. But, as the database gets larger, the operational time for the selection increase. Besides, there is a constant need to update new data into the database.

Other limitations include the capability to add new structural systems. For instance, the classification of the hybrid structure may in the future be broken down to several components with each of them representing another specific structural form.

Most of these programs integrating the artificial intelligence tools are only recently completed. The test over time may still be a question despite the fact that the M-RAM model allows for the upgrading of the available agents for more powerful ones.

Perhaps, the best way to integrate the approach is to ensure that the entry level engineer understand the variables involved. In this way, the basic assumptions and the methodology used in the model would be clear to them. Again, the engineers are also advised against heavily reliance on the outcome of the model. The engineers need to be able to challenge and compare with the computed output with their own alternative designs and thus verifying the use of the model might be more superior.

It is to be recognized that the system can be a powerful tool for the engineering students to get a preliminary structural subsystem for the entire tall building structure easily. In reality, similar approaches can be made towards other forms of structural systems such as the horizontal span structures, cables and prestressed concrete structures too.
6.4 Preliminary sizing of elements

With the use of the artificial intelligence techniques in predicting the structural forms applicable to the design problem, the next step in additional stage is the preliminary sizing and design of structural elements. The sizing of elements require both knowledge of the construction building as well as an understanding of the structural behavior of the structure. We would like to remind the reader that this is only one portion of the entire design process. But critical as it is, it will affect the subsequent results and a good initial sizing of elements will eradicate re-iteration later.

The process of proportioning structural members according to the respective code requirements is often quite complex and time consuming. In order to be able to concentrate on the building structure as a whole, simple processes and formulas for the preliminary sizing of elements have been developed.

Though there are many handbooks available for the sizing of beams and columns, it is the intention here to develop some feeling for structural behavior and some self confidence for being able to quickly proportion members, and thus be in full control of the design without having to depend on ready-made solutions as provided by handbooks and computer software.

Simple equations are used to make quick estimate of member proportion possible. This is especially important at various times of design and construction. For example, when a sense for the order of magnitude of forces and stresses must be available at the developmental stage of architectural design, when a fast judgement for size is needed for checking drawings and computer outputs, or for the designer to sense, on the construction site, when a member size don’t seem right, or finally, when first trial sections may be needed for the solution of indeterminate structures.

The ability to design individual structural elements is fundamentally required for all structural designers. There is a need to fully understand the behavior of individual
elements and their main structural purpose in each case. There are many ways that a structural engineering student can be taught to design the simple structural elements. An outline for practitioners’ preliminary design of common structural elements are shown in Appendix D. We will not endeavor into the detailed design procedures and design aids as they are easily available from textbooks, and design codes and guidelines. The main task is then to expose the students to the empirical rules so that he/she can use it upon entering the design profession. We will only generalize the findings of the preliminary design parameters.

The most important part of the preliminary sizing of the elements is that the designer has to be totally aware of the forces and the types of elements [31] that are suitable to resist the forces (see Tables 6.1 and 6.2).

The following section is organized to provide the readers information on the rule of thumbs for general building aspects. The detailed rules of thumbs for the design of structural elements such as concrete, steel and composite elements will only be discussed in Appendix D.

**General**

Ordinary buildings up to 20 and 30 story is usually controlled by gravity loading such that the gravity structure can absorb the lateral forces so that the member sizes may not have to be increased. For ordinary building up to approximately 40 stories or occasionally up to 60 stories in height, the lateral force resisting structure can still be contained within the building volume and this does not have to influence on the appearance of the architecture.
Table 6.1 Forces and the equivalent resisting basic structural elements

However, when the building’s slenderness increases, i.e. the height exceeds about 5 times the minimum base dimension, the lateral deformation of the building due to wind becomes the primary design consideration. When the building height increase more than 60 stories, the overall building form must be activated so that the structure must be concentrated along the perimeter to efficiently resist overturning by lateral forces. Normally when the plan aspect ratio is not greater than 1.5 or so, the free form shape (whether rectangular, square, circular or other regular shapes) and closed form is able to achieve the expected efficiency. However, when the plan aspect ratio exceeds 1.5 may suffer three drawbacks:

- The elongated plan shape, because of a large exposure area, acts like a large sail collecting large magnitudes of wind loads.
These large forces present structural design problems because the frames on the narrow faces of the building most usually not adequate to provide the required shear resistance.

Efficient tube action is difficult to achieve in elongated shapes because the shear lag phenomenon is more pronounced.

The spacing of parallel frames is in the range from 16 to 20ft and up to 40ft, depending on the building type. The building form determines where resultant wind pressure acts, and the building masses define the location of the resultant seismic force. By realizing that stiffness requirement for the upper portion of the building, one can purposely drop off or step back the core walls at the termination of the low- and mid-rise elevator banks.

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<thead>
<tr>
<th>Table 6.2 Complex structural actions and structures</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Complex structural actions</strong></td>
</tr>
<tr>
<td>Combinations</td>
</tr>
<tr>
<td>Composite materials</td>
</tr>
<tr>
<td>Composite action</td>
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<tr>
<td>Stiffening</td>
</tr>
<tr>
<td>Stabilizing</td>
</tr>
<tr>
<td>Bracing</td>
</tr>
</tbody>
</table>
Weight of structures

Ordinary steel building’s overall average gross weight can be in the range of 50psf to 80psf (lb/ft²) or approximated as 5pcf to 8pcf (lb/ft³). Non prestressed concrete building may have a density of twice as much. The overall weight (including the live loads) ranges from 10pcf for steel office building to about 14pcf to 18pcf for concrete office buildings and 20pcf for concrete apartment buildings. By varying the materials like use of high strength materials, or lightweight concrete, the overall weight can be reduced. For instance, the dead load of the structure can be reduced by as much as 10pcf to 20psf when using lightweight concrete for the floors.

The weight of the structure itself only constitutes a relatively small portion of the total building dead load; it may be in the range of 20% to 50% for frame buildings, but varies with height. A 10-story steel frame structure may weigh as little as 6psf in contrast to a 100-story building with about 30psf. This can be attribute to the fact that volume (weight is proportional to volume) increases to the power of 3 while the supporting area increased by order of 2.

The live loads of approximately 80psf for an office building are twice as high as for a residential building. According to some codes, live loads can be reduced for members supporting an area bigger than 150ft². In the case of multistory structures, it is improbable that every floor is fully loaded so that the live loads on columns or walls may be reduced by up to 60%.

Serviceability

In practice, there is an optimum sag-to-span ratio in the range of 1/10 to 1/20 for buildings and 1/8 to 1/12 for suspension bridges. In practice, it is not economical to design for high boundary support structures when sag-to-span ratio is greater than 0.2. For cable roofs, the typical ratios are in the range from 1/10 to 1/20. In this case, when the lower support is at the ground, the maximum sag should not be more than a quarter of the height h in order that the cable clears off the surface.
General factors that affect floor framing design

In the ‘laying out’ of the floor framing of any structural system, there are several factors that influence the various structural design criteria on the floor framing and they include the following:

- The effect of span direction
- The effect of bay proportion
- The effect of cantilever beam construction
- The effect of beam spacing
- The effect of framing floor openings for slabs, elevators, or other vertical shafts
- The effect of column layout
- The effect of cantilevering
- The effect of a slant corner
- The effect of scale
- The effect of corner or intersecting building units
- The effect of a partition wall or other heavy load layout

Low rise buildings

Seismic forces is usually critical with respect to stiff low- and mid-rise structures, while wind loading generally dominates the design of tall, slender buildings. The optimum design of high-rise buildings in areas of strong earthquakes conflicts with that for wind loading. Here, seismic action calls for ductility with much redundancy, while the wind resistance requires stiffness for occupant comfort. For low building, it may vibrate in their first mode, taller ones may deflect in higher modes similar to the movement of a snake.

For the preliminary design of a multibay post-beam structure, one can assume that the uniform gravity loading controls the design of the beams (i.e. the effect of live load arrangement for roofs in general can be ignored), while the lateral force case together with the dead loading governs the design of the cantilevering columns.
For low-rise structures, it is considered long when the span is at least 100-ft (depend on the material and structure system). The typical depth-to-span ratio of horizontal, double layer space frames used as roofs for ordinary conditions, is in the range of:

\[
\frac{1}{18} \leq \frac{d}{L} \leq \frac{1}{25} \text{ or } \frac{1}{9}
\]

for the cantilever span

The maximum bay size is often assumed not larger than a fifth of the span or 30ft.

\[
\frac{L}{5} \geq a \leq 30 \text{ ft}
\]

An optimum bay size seems to be in the range of:

\[
1.2d \geq a \leq 2.5d
\]

**Tall buildings**

There are several aspects that need to be considered for tall buildings such as the structural systems, lateral load effects, slenderness effects, torsional effects and general analysis and design considerations.

*Structural systems*

There are several types of tall building forms shown in Appendix C. In tall building, there is a prominent feature where the free ground level space with minimum columns being incorporated for grand entrances and wide lobby spaces, loading docks, and parking isles to open public plazas. As such, many geometrical pattern of the building cannot be extended to the foundation walls; it becomes discontinuous and is replaced by another structure system.

The other type of tall buildings responds as shear cantilever when the resisting elements are rigid frames, since shear can only be resisted by the girders and columns in bending. In this case, the effect of rotation (i.e. axial shortening and lengthening of columns) is secondary and may be ignored for preliminary design purposes.
The hidden, non-visual response of the structure to the accumulation of the loads is usually done by the thickening of members and/or increasing the material strength. Another approach is to decrease the density of cross section, for example from a hollow concrete tube to a solid section at the base. But standard carbon steels may be more economical when stability, deflection, elongation, or stiffness is the governing factor as in the use for exterior columns of tubular buildings that must resist lateral forces occasionally.

Framed tube concepts should be given serious consideration for building taller than about 40 stories. The feature of this system includes a closely spaced columns connected by a relatively deep spandrel. The resulting system works as a giant vertical cantilever and is very efficient since there exists a large separation distance between the windward and the leeward columns. If the plan aspect ratio is larger, say much in excess of 1:1.5, it is likely that a supplementary bracing is necessary to limit the drift.

The terminology used for a tube system will refer to a system of closely spaced columns 8 to 15ft on center (2.43 to 4.57m) tied together with a relatively deep spandrel. In general, the truss tube system has about 14kPa of structural steel with a cantilever efficiency of about 80%.

**Strength and stability**

The calculation of the critical buckling strength of columns $P_{cr}$ is normally with lots of uncertainties, not only in the determination of appropriate section properties, but also in the selection of the effective length factor $k$ used in the buckling load calculations.

$$\delta = \frac{1}{1 - \frac{P}{P_{cr}}} = \frac{1}{1 - \frac{K_G}{K_E}}$$

Equation 6.1

The current trend in the US specifications for concrete and steel construction is to use P-delta techniques in lieu of the approximate moment magnifier approach. The steel specifications give little guidance as to what properties should be used for analysis. ACI 318-95 suggests $0.35EI_g$ for girders, and $0.70EI_g$ for the columns, where $E$ is the modulus
of elasticity and $I_g$ is the gross section moment of inertia [44]. ACI also suggests that the gross section area be used in the analysis. These factors appear to be calibrated on the basis of the behavior of relatively low rise frames, with span to depth ratios in the range of 8 to 12.

**Slenderness effects**
The new breed of skyscraper of slenderness up to about 12 to 1 may be influenced by a lower wind speed than 200mph. An efficient steel tubular building typically has a narrow range of height-to-width ratio of 4 to 6. Experience with various framed tube systems has indicated the flange width of these channels to be no more than half the depth of the web or 10% of the height of the building.

**Lateral load effects**
Typical influences of the lateral load include wind and seismic loading. Typical uniform wind pressure used for the inland United States ranges from 20 to 40psf for ordinary high rise buildings. The ideal shape to counteract against wind loading would be the tear-drop shape on plan. Surprisingly, very tall round buildings are found to be much more vulnerable than rectangular buildings, where the wind turbulence acts as a damping agent.

It is common practice to express the magnitude of the seismic forces as a percentage of the building weight. Typical values for high-rise buildings in major seismic zones may range from about 5% for flexible rigid frame structures, to about 20% for stiff bearing-wall buildings.

The trend of setting back as the building height increases has the following advantages. The reducing mass in the upper floors is a structural advantage in earthquake prone region. Also the sway due to wind is reduced due to less exposed surface areas. In addition, the undulating created at the top of the building reduces vortex shedding because of turbulence created at the upper levels.
Torsional effects
The lateral load can cause twist if the resultant force do not pass through the center of rigidity. For effective torsional resistance, closed tubular sections are much stronger and stiffer than the equivalent perforated open shafts. The round, closed thin-walled tube is the ideal shape for resisting twisting. In the current trend, however, building forms comprise of compound and hybrid nature such that often causes extensive eccentric loading due to lack of symmetry.

Approximate element designs
The building design specifications of the AISC and the ACI have been the basis for structural design; they have been incorporated into most building codes in the United States. For the preliminary sizing of steel members, the traditional allowable stress approach has been used, while for the sizing of reinforced concrete elements, the strength method has been employed, as derived from the loading conditions. In this case, the design strength must be at least equal to the required strength, as derived from the loading conditions. The design strength is equal to the nominal strength reduced by the capacity reduction, $\phi$ which is shown in Appendix D as well.

6.5 Summary
The aim of the chapter is to identify for both the educators and the students an additional stage in educating structural engineers. We have capitalized on the prior knowledge from the previous chapters concerning the existing engineering education programs in the United States, the roles of the various personnel involved in the education programs, the possible teaching and learning environment that educators can adopt, and the technical expectations of the graduates.

This additional stage primarily evolved from the conceptual understanding of structures. We have emphasized that this stage can be adopted for all structural systems. We have focused on the example of tall building design for the selection of structural forms and
the appropriate structural selection criteria have been discussed. In effect, we have looked into the evolution for selection of structural forms together with preliminary elementary design.

In attempting to answer the general objective question: “What are the tools available that can help the students to achieve technical expectations?”, we have also introduced to the readers an advanced computing tool MRAM using Artificial Intelligence, in assisting them in the selection process for structural forms of tall building. We have illustrated the simplicity in using MRAM for engineer to appreciate past designs and initiate a “feel” for structural forms. In this way, the entry engineer can participate in the selection of structural forms. We have generalized the findings of primary design parameters focusing on the technical expectations of the entry engineer. The more practitioners’ technical details are shown in Appendix D.
Chapter 7
Comparison of codes for optimization

7.1 Introduction

It is crucial that a design ultimately be considered as an efficient design only if it has achieved minimum cost. Especially in this highly competitive nature of the construction industry, it becomes advantageous to the entry level engineer to consider finding various ways to reduce cost and achieve higher efficiency. In this chapter, we will adopt a practitioner approach to study cost optimization of steel design with the use of practical codes.

We aim to optimize the structural steel design by comparing the design codes American Institute of Steel Construction – Load Resistance Factor Design (AISC), British Standards (BS5950), and European Code (EC3). One way of measuring the cost efficiency in steel structures is the weight of steel. As mentioned in the earlier chapter that an efficient design can reduce the cost to as much as 5-10 % of the total cost.

We believe that the optimization of the steel design based on different design codes can be achieved by just using the American steel (W-shapes only) by comparing the cost efficiency in the three different design codes. As the infrastructure system continue to improve over the years, the different sources of steel would be more readily available to
all different parts of the world. In addition, nowadays there are also many international consortiums bidding for large construction jobs all over the world. It would be useful for the American counterparts to understand if there is a cost advantage to use American steel in the different countries practicing different codes.

This method of investigation can also be applied to steel manufactured elsewhere. For instance, the recent economic crisis in Asia has led to many structural steel purchasing order being cancelled and projects are put on hold which has resulted in large amount of foreign steel being “dumped” in the America. The engineer can also make use of the different steels in the region to optimize their design by following similar investigation method.

The rest of the chapter is devoted to the study of the cost efficiency of steel in terms of the weight of steel in laterally supported beams, laterally unsupported beams, concentric columns, concentric columns versus slenderness, and beam-columns. The specific designs according to various design codes are shown in Appendix E.

### 7.2 Load combination and assumptions

For the rest of the chapter, the structural loads that has been considered consist of only the gravity loads that include the dead load and live imposed load. Each load type is multiplied by a relevant load factor, and the most severe load combination is embraced. These load factors and combinations are summarized as follows:

AISC (A.4)
- 1.4 x dead load
- 1.2 x dead load + 1.6 x imposed live load

BS5950 (cl. 2.4, Table 2)
- 1.4 x dead load
- 1.4 x dead load + 1.6 x imposed live load
EC3 (EC3 2.3.3)

1.35 x dead load
1.35 x dead load + 1.5 x imposed live load

Besides the load combination, the same W-shaped steel element can be classified differently under the various codes. The detail section classification according to the three design codes are shown in the Appendix C. Steel grade for W shape is taken as 36ksi (248MPa). For example, the case of a W14 x370 would be classified as follows:

<table>
<thead>
<tr>
<th>Member under</th>
<th>Classification of members according to code</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AISC</td>
</tr>
<tr>
<td>Flexure alone</td>
<td>Compact</td>
</tr>
<tr>
<td>Compression alone</td>
<td>Not slender</td>
</tr>
<tr>
<td>Compression with bending</td>
<td>Compact</td>
</tr>
</tbody>
</table>

Table 7.1 Comparison of classification of W14x370 under three codes

The classification of the section is a tool for engineer to predict the type of behavior of that particular member given its properties, geometry, dimensions and in particular its thickness.

Assumptions for the comparison of the three design codes are:
- The ends of the members are simply supported.
- The W-shape steel section has a modulus of elasticity of 29,000ksi (200,000MPa) and mild steel section of yield strength of 36ksi (248 MPa).
- Spans of beams are varied from 2 to 10 meters with increment of 0.5m while the heights of the columns are from 3 to 10m.
- Beams and columns are considered to bend about the major axis.
7.3 Laterally supported beams

A range of laterally supported beams are assumed to be loaded [45] with dead load of magnitude 40kN/m² and imposed live load of 60kN/m² as shown in Figure 7.1. A series of beams ranging from 2m to 10m are designed according to the loading criteria mentioned.

![Laterally supported beam](image)

Figure 7.1 Laterally supported beam

<table>
<thead>
<tr>
<th>Length (m)</th>
<th>AISC (kN)</th>
<th>BS5950 (kN)</th>
<th>EC3 (kN)</th>
<th>AISC % higher than 5950</th>
<th>EC3 % higher than 5950</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2.5</td>
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<td>0</td>
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<td>6.1</td>
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<tr>
<td>6</td>
<td>6.7</td>
<td>6.0</td>
<td>6.7</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>6.5</td>
<td>8.0</td>
<td>8.0</td>
<td>8.0</td>
<td>0</td>
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<tr>
<td>7</td>
<td>9.6</td>
<td>8.6</td>
<td>8.6</td>
<td>12</td>
<td>0</td>
</tr>
<tr>
<td>7.5</td>
<td>10.8</td>
<td>9.9</td>
<td>9.9</td>
<td>10</td>
<td>0</td>
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<tr>
<td>8</td>
<td>12.6</td>
<td>11.6</td>
<td>12.6</td>
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<td>14.4</td>
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<td>9</td>
<td>17.1</td>
<td>15.5</td>
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<td>9.5</td>
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<td>10</td>
<td>21.9</td>
<td>19.7</td>
<td>19.7</td>
<td>11</td>
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</tr>
</tbody>
</table>

![Table 7.2](image)

Table 7.2 Weight of laterally supported beam using three codes
The designed beam weights for the three design codes for various lengths of beams are sorted as shown in Table 7.2. A graphical representation of the data is shown in Figure 7.4. From the Table, the averaging of the differences of the steel weights for various codes are based on non-zero figures. Using this method, W-shape steel weight for AISC is about 10% higher than BS5950 and that of EC3 is about 11% higher than BS5950.

The main differences can be accounted by the different loading conditions used in the design codes. For BS5950, no additional load resistance factor or partial safety factor is imposed. For AISC, there is a load resistance factor of 0.9 for beam design. Therefore, intuitively, there is a $1/0.9 = 11.1\%$ differences between the AISC and BS5950 loading. And for the case of EC3, there is a partial safety factor $\gamma_M$ taken as:

- Resistance of Class 1,2 or 3 cross section $\gamma_M = 1.1$
- Resistance of Class 4 cross section: $\gamma_M = 1.1$
- Resistance of member to buckling: $\gamma_M = 1.1$

This factor accounts for the 11% average differences. It is to be noted that the total loads considered for the three cases are as:

- BS5950 is $1.4DL+1.6LL = 152 \text{ kN/m}$
- AISC is $1.2DL + 1.6LL = 144 \text{ kN/m}$
- EC3 is $1.35DL+ 1.5LL = 144 \text{ kN/m}$

By considering the load resistance factor of 0.9 for AISC and the partial safety factor of 1.1 for EC3. We can find an equivalent load as:

- BS5950 is $1.4DL+1.6LL = 152 \text{ kN/m}$
- AISC is $(1.2DL + 1.6LL)/0.9 = 160 \text{ kN/m}$
- EC3 is $1.1(1.35DL+ 1.5LL) = 158 \text{ kN/m}$

It is obvious then that design code BS5950 constitutes to the lowest equivalent load. As such, the design of laterally supported beam using W shape steel for BS5950 would be the most economical amongst the three design codes.
7.4 Laterally unsupported beams

Using the same loading conditions, the case of the laterally unsupported beam is examined using the three codes. Various lengths of laterally unsupported beam are considered ranging also from 2m to 10m. The laterally unsupported condition is defined as one where at the supports, the compression flange is assumed to be laterally unrestrained, both flanges are free to rotate on plan, and restraint against torsion is provided only by the positive connection of the bottom flange to supports. The laterally unsupported length is considered as the actual length of the beam.

The result shows that the order of magnitude of differences is significant for the BS5950 and EC3 when compared to AISC. The BS5950 design using W shape beam shows an average of 18% heavier steel than the AISC design while that of EC3 shows 8% heavier steel beam weight than AISC.

<table>
<thead>
<tr>
<th>Length (m)</th>
<th>AISC (kN)</th>
<th>BS5950 (kN)</th>
<th>EC3 (kN)</th>
<th>BS5950 % higher than AISC</th>
<th>EC3 % higher than AISC</th>
<th>BS5950 % higher than EC3</th>
</tr>
</thead>
<tbody>
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<td>0.6</td>
<td>0.6</td>
<td>0.0</td>
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<td>12.6</td>
<td>5.2</td>
<td>7.0</td>
</tr>
</tbody>
</table>

sum 271.2   117.7   143.1
average 18    8       10

Table 7.3 Weight of laterally unsupported beams using three design codes
Again, the designed beam weights for the laterally unsupported beams using three design codes are sorted as shown in Table 7.3. A graphical representation is illustrated in Figure 7.5. From both the Table and the Figure, the designed beam weight using EC3 is almost the average of the designed beam weight by BS5950 and AISC. It is interesting to note that AISC code takes the effective length for laterally unsupported beams as the span of the beam, with no allowance being made to the restraint conditions at the supports and the position of the applied load with respect to shear center.

Even though for the same loading conditions as the laterally supported beam, the design using AISC seemed to produce more economical sizes despite that the equivalent loading being smallest for BS5950 amongst the three design codes. Clearly there are other factors within the design codes that cause this variation.

In practice, most beam lengths are between the limiting laterally unsupported length for full plastic capacity and the limiting unsupported length for inelastic lateral-torsional buckling. Despite the fact that same laterally unsupported length is used for each beam length in all three design codes, the deciding buckling design strength of the beam still varies. Some of these deciding factors then include:

- BS5950 evaluates the limiting equivalent slenderness ratio \( \lambda_{LT} \) directly, without the intermediate step of evaluating the value of elastic critical moment for lateral torsional buckling, \( M_{cr} \). For the case of EC3, the ratio \( \lambda_{LT} \) is found by incorporating the finding of \( M_{cr} \) in an equation. For the case of AISC, the limiting equivalent slenderness ratio is the same as the slenderness of the beam. It is found that [46] expression for \( M_{cr} \) require greater care when dealing with parameters that have powers of 10. The possibility of error with calculating \( \lambda_{LT} \) directly is lesser.

- The limiting laterally unsupported length for inelastic lateral-torsional buckling is also calculated differently in all three design codes.

In general, most beams are laterally supported. However, if there are situations where the beams cannot be laterally restrained, the design code that optimally uses W shape beams would be the AISC.
7.5 Concentric columns

W-shape columns are used in the design of concentric columns for the three design codes. The design loading on each column is 1200kN (DL) and 800kN (LL) only as shown in Figure 7.2. It is assumed that there are no imposed moments on the columns. The same load combination is considered under the relevant codes. The weights of the W-shaped columns to resist the loads for the three codes are shown Table 7.4.

It is to be noted that the total loads considered for the three cases are as:
BS5950 is 1.4DL+1.6LL = 2960 kN
AISC is 1.2DL + 1.6LL = 2720 kN
EC3 is 1.35DL+ 1.5LL = 2820 kN

By considering the load resistance factor of 0.85 for AISC and the partial safety factor of 1.1 for EC3.
The equivalent load would otherwise be:
BS5950 is 1.4DL+1.6LL = 2960 kN
AISC is (1.2DL + 1.6LL)/0.85 = 3200 kN
EC3 is 1.1(1.35DL+ 1.5LL) = 3102 kN

From the equivalent load, it seemed apparent that the design using BS5950 would result in lighter column weights.
(Also seen from Figure 7.6).
<table>
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<tr>
<th>Length (m)</th>
<th>AISC (kN)</th>
<th>BS5950 (kN)</th>
<th>EC3 (kN)</th>
<th>AISC % higher than BS5950</th>
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Table 7.4 Weight of concentric columns using three codes

Figure 7.6 also illustrates that the designed column weights using AISC is somewhat the average of that using EC3 and BS5950 for column heights above 7 meters. However, the percentage weight increase shown in Table 7.4 does not reflect the proportional increase in the equivalent load. In this case, besides the influences from the load resistance factor in AISC and the partial safety factor in EC3, the other contributing factors include:

- Compression elements and beams are classified in the ASIC as compact, non-compact and slender components, whereas the EC3 classifies them in four categories of Class 1 (Plastic), Class2 (Compact), Class 3 (Semi-compact) or Class 4 (Slender). For the case of BS5950, the members are classified as Class 1 (Plastic), Class2 (Compact), Class 3 (Semi-compact) only.

- Columns are designed in the AISC by using three equations: the first dealing with long columns (elastic buckling); the second, with intermediate columns (inelastic range); and the third, with short columns. The estimated effects of residual stresses and out-of-straightness are allowed for in an empirical manner. For instance, the
coefficient of 0.877 (Appendix C) on the critical compressive strength (for the case of \( \lambda c > 1.5 \)) covers the effect of initial crookedness and residual stress, although the latter has little or no influence for long columns, especially for larger slenderness ratio. According to AISC, the coefficient is not be affected by the type of sections and axes of buckling. Column designs by EC3 [47] are based on European column curves and provide explicit (rather than empirical) allowances for an out-of-straightness residual stresses, etc. For the case of BS5950, the Perry Robertson constant is used to instead.

- The development of the column curves for all three codes adopted different approaches. The BS5950 curves are generated from the modified Perry-Robertson formula. They produce results that are lower bounds to physical test results, whereas the AISC has carried out regression analysis of the physical test results to obtain a single curve that coincides somehow with the test results. EC3 is based on the work of Beer & Schulz [48] and has been correlated to a large series of full-scale column tests. EC3 also uses a full-blown multiple column curve approach.

- In Europe, EC3 and BS5950, both codes represent the maximum permissible crookedness to be L/1000. In the USA, the decision is made on the value of L/1500. This value is chosen on the premise that all of the other column strength parameters are based on mean values. This is in agreement with the first order, second moment approach that has been used to develop the AISC code. The basic column philosophy is therefore that of a member always subjected to bending, albeit with a small moment.

- According to the AISC there is a boundary of separation between the inelastic and the elastic buckling region, whereas in the British Standards only one equation is employed to deal with both cases. Hence, AISC is concerned with two slendernesses while BS5950 is concerned with only one limiting slenderness that separates the stocky beam from the intermediate one, which undergoes inelastic responses.
- The multiple curve approaches by both BS5950 and EC3 has resulted in design curves depending on the different column sections, the buckling of a section about a different axis and sections with different thicknesses.

7.6 Concentric column versus slenderness

For this section, the axial capacity of a concentric column of W12x79 is calculated with respect to the slenderness ratio. The result is shown in the Table 7.5. The AISC (N) refers to the nominal axial capacity while AISC (D) refers to the design axial capacity after considering the load resistance factor of $\phi = 0.85$. The data on the Table 7.5 are plotted in Figure 7.7 and Figure 7.8 that shows the variation of the axial capacity with respect to the slenderness ratio and the height of the column respectively under the three design codes.

It may be convenient to compare the data by tabulating the percentage increase in the axial capacity of each column at the individual slenderness level based on the lowest (conservative) design axial capacity. As such, the Table 7.6 is formed on the basis for comparison of the three design codes based on the percentage differences. For AISC method, the design axial capacity [AISC (D)] is used for comparison with the other two codes.

From Table 8.6, the interested ranges of column heights are:
- For the first 3 meters of the column, the variation of the column loads decreases as the slenderness increases for the case of BS5950 in comparison with the AISC. Similar trend is also observed for the percentage increase of EC3 compared to AISC. The significance of the 3 meters zone height can be explained from the perspective of BS5950. For BS5950, the limiting slenderness of this W-shape column is 18.06 marking the separation zone for stocky column from the intermediate one. This slenderness ratio corresponds to a height of 1.4m. This explains the sharp drop in axial capacities when the column height exceeds 1.4m.
<table>
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<th>Height (in.)</th>
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Table 7.5 Axial capacities of concentric columns versus their slendernesses
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<td>-</td>
<td>-0.6</td>
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</tr>
</tbody>
</table>

Table 7.6 Percentage difference between the axial capacity recorded

- For the section of column from 3 meters to 10.5m, the AISC records higher column capacity than the BS5950. However, the percentage increases as the slenderness increases with a drop in the percentage increase if the slenderness increases further. The largest percentage difference is about 25 percent. Similar trend is also observed.
for the case of EC3 in comparison with BS5950 with the maximum percentage difference at about 9%. Furthermore, the axial capacity of columns designed using EC3 seemed to be about the average of the other two codes. What then is the significance of columns in the range of 3 meters to 10.5m height? It seemed that according to AISC, this range of column height is within the inelastic buckling range of slenderness ratio of less than 135.5. The deviation of the capacity curve for AISC from the BS5950 increases as the slenderness increases within the elastic buckling region until it reached the boundary that separates the two. When it enters the elastic range, the deviation decreases as the slenderness increases.

- As the column height increases from 11 meters to 17 meters, it can be observed that the design of the column using EC3 gives more conservative design. The percentage changes when compared to AISC can be observed to be in the trend of decreasing as the column’s slenderness increases. For the case of the BS5950, there seemed to be increasing percentage changes as the column’s slenderness increases. In this range of column heights, it is clearly within the elastic buckling zone for AISC.

### 7.7 Beam-column comparisons

For the case of beam-columns, the axial loading on the column are 1200kN (DL) and 800kN (LL). The column moment loads as shown in Figure 7.3 are:

- Dead load moment of 30kN.m and live load moment of 40kN.m imposed at the top of the column.
- Dead load moment of 50kN.m and live load moment of 70kN.m imposed at the bottom of the column.

From Table 7.7, it can be deduced that most of the W shape steel sections designed based on AISC is about 10% heavier than those designed based on BS5950. On the other hand, those columns designed based on EC3 showed an average of 18.2 % heavier than that of
BS5950. The percentage difference for the EC3 and BS5950 are mainly in the range of 20-21% with the maximum difference of 33%. The data are plotted in Figure 7.9.

<table>
<thead>
<tr>
<th>Length (m)</th>
<th>AISC (kN)</th>
<th>BS5950 (kN)</th>
<th>EC3 (kN)</th>
<th>AISC % higher than BS5950</th>
<th>EC3 % higher than BS5950</th>
<th>EC3 % higher than AISC</th>
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<tbody>
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<td>8</td>
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<td>21.2</td>
<td>25.7</td>
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</tbody>
</table>

| Sum        | 63        | 273         | 202      |
| Average    | 9         | 18.2        | 13.5     |

Table 7.7 Weight of beam-columns using three design codes

Similar to design of concentric columns, most of the beam-columns' weight of AISC is somewhat at the average of that of BS5950 and EC3. The explanation for the differences are similar to the ones for concentric columns in particular to the load resistance factors and partial safety factors that vary from code to code and those other factors discussed in the section for the concentric columns comparison.

In the same context as that of the concentric column, the design of W-shape columns is optimal for the case of BS5950.
7.8 Summary and recommendation

In this chapter, we have looked into a practitioner approach in optimization of steel design. We have achieved this by looking into the possible cost savings at the element designs by using W shapes under the various design codes of AISC, BS5950 and EC3.

The findings for the given loading conditions are:
- The nature of Eurocode is aimed to harmonize in three ways [49]: between countries, between materials, and between industries. As such, we observed that designs using EC3 has attained almost an average position when compared with the AISC and BS5950 in both the laterally unsupported beam and the comparison of the axial load capacity of a single column section.
- One general observation is that BS5950 constitutes the lowest effective loading compared to the other two codes. This is mainly because of the partial safety factor in EC3 and the load resistance factor introduced by AISC.
- The design of laterally supported beams is most economical for BS5950, follow up by EC3 and then finally AISC.
- The AISC method is most efficient in designing laterally unsupported beams.
- It seemed that there is a cost advantage to use American W-shape steel for designing concentric columns and beam columns under the BS5950.

For better comparison of the codes, we propose the following recommendations:
- Consideration of the effects of some other steel such as British or Japanese steel. Different availability of sections from the each steel source and their comparison could provide more competition that may eventually lead to optimization of design pertaining to certain section size.
- Steel grades may also play another important role. Nowadays, steels of 36ksi specification are manufactured with strength of up to 43ksi or so. If the designer can prove the strength of steel to be almost a grade higher, then the design of the structural elements can be optimized. However, it is important to note that for the case of some flexible structure such as the tall building design, the effect of strength
may not be as important issue as the stiffness, thus the steel grade of 36ksi may be more than adequate to support the structural elements.

- This comparison is still at elementary level. There is a need to conduct comparison in particular to the global structural system. This is in particular to tall building systems since there are other added consideration such as lateral loading and drift.

- The prediction of lateral buckling strength of beams can possibly progress by using multiple design curves as it has been used for the column design.
Figure 7.4 Comparison of laterally supported W shaped beams

- AISC
- BS5950
- EC3

Weight (kN) vs. Beam Length (m)
Figure 7.5 Comparison of laterally unsupported W shaped beams
Figure 7.6 Comparison of concentrically loaded W shaped columns

- AISC
- BS5950
- EC3
Figure 7.7 Axial load versus slenderness ratio

- AISC (N) strength
- AISC (D) strength
- BS5950 strength
- EC3 Nc,Rd

Axial Force (kN)

Slenderness ratio

6.5 19.4 32.3 45.2 58.1 71.0 83.9 96.8 109.7 122.6 135.5 148.4 161.4 174.3 187.2 200.1 213.0
Figure 7.8 Axial load versus column height

Axial force (kN) vs. Column height (m)

- AISC (N) strength
- AISC (D) strength
- BS5950 strength
- EC3 Nc,Rd
Figure 7.9 Comparison of beam-column using W shaped columns

- AISC
- BS5950
- EC3

Weight (kN)

Column Height (m)
Chapter 8
Conclusion

In conclusion, we have reviewed the engineering education and also identified some of the technical and non-technical expectations of a new entry level structural engineer.

When reviewing the engineering education in the United States, it has been found that while the current engineering education is becoming more design oriented, there is a lack of practitioner approach in the engineering education.

Looking at the roles of graduates, educators, and professionals in the engineering education, we have found the followings:
- Various organizations highlighted the non-technical concerns of students in their ability to work on a team, ability to communicate, and awareness of workplace expectations.
- Issues related to the faculty include being research oriented and lacking in practical experience.
- Four learning style models have been classified to assist educators in meeting the students’ needs.

We have proposed an integrative teaching model looking at the four steps of invitation, exploration, explanation, and application that could be applied to engineering education. In addition, the advantages and disadvantages of computer usage in the teaching and
learning environment have been discussed together with some prerequisites of using computer and the modeling and verification when using computer. We have also identified a special way of bringing the site into the classroom to instill realism in engineering education. This discussion has formed the basic teaching and learning environment for the additional stage in structural engineering education.

We have approached the lack of practitioner approaches in structural engineering education by looking at three main aspects:

- lacking a sense of “feel” for structures by entry level engineer
  Some of the things that the students can do beyond just basic structural designing are perceived from professionals’ expectations of an entry level engineer. We have provided the entry level engineers a correct perception of their engineering profession through the review of the ideology, opinions, and misconceptions about being a structural engineer. In addition, entry level engineers have been encouraged to develop a “feel” towards structure and structural design. It has been stressed that concepts of empirical knowledge in the field of structural engineering are crucial for the entry level engineer besides technical knowledge. Also the concepts of design process/procedures have been presented in the form of some real-life examples so that they can realize its importance during design.

- learning about the conceptualization of structural forms and preliminary designs
  With these developed ideas and concepts, we have proposed an additional stage in structural engineering education that evolved from the conceptual understanding of structures. This additional stage is recommended for teaching of all structural systems. It starts with the selection of structural form. We have used tall building structural form selection with an explanation of technical selection criteria and procedures. The selection process is substantiated with an advanced computing tool using Artificial Intelligence (AI). The working mechanism with AI is explored and the explicit use of one such tool, M-RAM model has been identified. The steps in using M-RAM have been explained to show the simplicity in its use. We have identified this as one of the tools that can help students to achieve technical
We introduce the next step in the additional stage as preliminary sizing and design of structural elements with some general information on structures and the technical details shown in Appendix D. We believe that current available literature in preliminary designs, if used in the proper engineering education context, would be indeed very beneficial to the entry engineers in enhancing their appreciation of structural behavior.

- learning the use of practical codes for optimization

We have adopted a practitioner approach to account for economics in the entry level engineer’s design. We have investigated for an economically efficient steel element design. We have made use of the three practical design codes to illustrate a simple weight (cost) comparison study on the W-shape steel sections. It has been found that it might be economical to use W-shape sections under certain conditions. However, a complete comparison study has been recommended to incorporate the effects of different sources of steel and global frame systems instead of element level.

Ultimately, we believe that looking at the practitioner approaches in the engineering education, this thesis will contribute to the understanding of some technical and non-technical expectations of a new entry level engineer. In addition, we trust this thesis will add value to the existing structural education program in MIT.
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Appendix A

The list below is a random list of the more specific skills that a structural engineer might possess. There are approximately according to their generality. Although many of the skills have been expresses using modern concepts, all of them could be applied to people engaged in building design at any time in the past – one or ten or twenty centuries ago (remember, ‘mathematical model includes geometrical model).

- To have intention and assess progress towards a predetermined goal: to be able to detect which way is forward
- To analyze a problem logically
- To choose and mix appropriate skills in order to progress towards the desired goal
- To learn in different ways, both from one’s own experience and from the experience of others
- To communicate ideas and information to others using appropriate technical language and concepts
- To develop peer group identity for instance for the purposes of developing common standards and work methods, and to facilitate learning by the group as a whole
- To recognize and evaluate, at any stage of a project, issues of relevance to the structure of the building
- To select, from an almost infinite amount of possible data about materials, products, processes and costs, that data which is appropriate at a particular time
- To understand the implications of the concerns of other professionals (such as cost, services, buildability, durability, fire resistance, etc) upon the building structure, and vice versa
- To have a “feel” for materials, their properties and behavior
- To imagine the likely loading on a structure and the corresponding behavior of a structure under load, both as a whole and at the level of individual elements
- To imagine an unbuilt structure both geometrically and in terms of chosen structural actions
- To imagine possible collapse mechanisms for individual elements, due to particular loading conditions, and their consequences leading up to total structural collapse
- To imagine, and hence be able to prevent, other different ways in which a building structure can fail (in the wildest sense of the world)
- To conceive innovative and highly appropriate structural solutions to both familiar and new problems, either drawing from precedent and experience or \textit{ab initio}
- To conceive a structural solution at a level of detail appropriate to different stages of a project
- To imagine as many as possible of the consequences of a structural decision or choice between alternate designs
- To think both qualitatively and quantitatively about loads, materials and structures and to switch easily, appropriately and at will between both modes of thought
- To create one or more mathematical models of an imagined or actual structure, using geometry, algebra, trigonometry, statics and so on
- To make appropriate assumptions, approximations, idealizations and simplifications of actuality in order to be able to build both mathematical and physical models for use in a design procedure
- From a variety of possible mathematical models of loads, materials and structure, to evaluate and select ones appropriate to a given case
- To know and use a range of standard design procedures
- To recognize when a proposed structure can or cannot be designed using standard or existing design procedures
- To select or modify an appropriate existing design procedure, or create a new one for established types of structure
- To develop an entirely new design procedure for a type of structure which has never before been designed or built
- To evaluate the relative power of different methods of justifying a proposed design using precedent or physical or mathematical models
- To know when a physical model will help to develop a design or to understand a structure’s behavior
- To be able to build scale models to test structural ideas and behavior
- To interpret the results obtained from a scale model for use at full size
- To choose an appropriate performance criteria against which a structure can be assessed, and limits which it must not transgress
- To evaluate the performance of a proposed structure and hence compare the performance of alternative proposals
- To ensure that a proposed solution satisfies non-structural criteria such as thermal and acoustic behavior, buildability and durability
- To communicate and know when to communicate with specialists from other disciplines such as architect, contractor, service engineer
- The ability to take a design from an outline conception to the level of fine detail design
- To recognize, from experience, ways of simplifying the problems of design and seeing short cuts to appropriate answers, for instance by identifying highly stresses, sensitive or problematic parts of a building structure which will require particular attention, allowing time and effort to be saved by treating the rest of the structure as a relatively standard solution
- To select and use a variety of tools, such as computer programs, codes of practice and physical models, to assist in the design procedure, for instance by making it quicker, less arduous or more thorough
- To carry out and record calculations justifying a design, for instance, for independent checking
- To use a variety of calculating devices and nowadays, types of computer software
- To imagine how an artifact could be manufactured, assembled and, where appropriate, maintained
- To assess the buildability of a proposed design: to create designs which can be built easily and cheaply
- To draft appropriate and sufficient drawings to represent and communicate a building design to those persons who need to know it
- To draw effective written specifications for materials, workmanship and matters relating to method of construction and temporary works
- To know when the process of design has been completed in a satisfactory manner
- To fall back on ‘engineering judgement’ when all else fails
Appendix B

Application of flow chart for elementary design purposes

This section shows the integrated relationship for using flow chart as the basis for elementary design. It shows a comprehensive step-wise approach which would be useful to engineers. However, this is at the last section of the chapter so that the readers has a basic understanding of the elementary basic design before they approach the designs with values and decision making. The section captures the preliminary flow chart designs for steel elements. The author would concentrate on the design flow charts for steel structures. The reason being that there are already extensive design procedures incorporated into concrete designs handbook provided by the ACI committee.

Significance of the flowchart symbols

- Decision Requirement: A flow chart location where a specification criteria is either met, or not met, and the answer determines which of two alternative paths must be followed in making an exit from that particular location.

- Process requirement or statement: A flow chart location where an made, or where an operation, as stated, is to be performed.

- Connector: Represents a junction in the line of flow. An alphabetic character in the circle identifies a unique location to transfer to within a flow chart.

- Offpage- connector: Entry to or exit from a page. A numeral in the offpage connector symbol shows the matching locations of entries and exits.
Two rare flow diagrams which have appeared in print are included below to illustrate the idea of the design procedure, to demonstrate the effectiveness of a flow diagram as a way of summarizing a design procedure, and to serve as models which others may choose to emulate.

The first appeared in an article describing the design of a reinforced concrete road bridge within the Ove Arup Partnership [Ranawake et al. 1970]. The project was the first such structure the company had undertaken and at the time (late 1960s) the company was also beginning to expand its use of computer programs to help with the structural analysis. Both factors meant that the staff involved on the project was learning new engineering skills and developing new engineering knowledge. In order to record this knowledge and to pass it on to others in the company, they used a flow diagram to summarize the design procedure they had developed.

The second example is from a book that resulted from the unusual circumstance facing one engineering design community when it needed to communicate its skills and knowledge to another in a different culture [Grob et al. 1983]. A number of bridge engineers were engaged in a project with the Swiss Company for International Technical Cooperation and Development (ITECO) to help to provide a number of similar pedestrian suspension bridges across rivers in mountainous regions of Nepal. Rather than simply construct the bridges and withdraw, the Swiss engineers chose to use the project to train local people with technical knowledge how to undertake every aspect of the project from the decision to build a suspension bridge, through surveying, design, contractual arrangements, and procurement, to the construction itself. The result is an unusually clear and complete summary of how to design and build a particular type of structure – just the sort of information which is lacking about so many projects in the past. The required information and the many activities, such as surveying, and the calculations involved in designing the various parts of the bridges, are related in a number of flow diagrams. The one reproduced below deals with the design stages of a wire cable suspension bridge, laterally braced by windguy cables.
Design procedure for a reinforced concrete bridge

1. Evolve structural system and make preliminary sizing of deck
2. Section properties using computer programme
3. Deck analysis as beam using plane frame programme
   - Torsion influence lines
   - Shear influence lines
   - Moment influence lines
   - Finite element analysis of simple two cell box to establish warping factors
4. Calculate maximum and minimum design moment envelope
5. Prestressing scheme and estimation of forces
6. Find zone, draw trial profile and calculate parasitic moments
7. Check longitudinal stresses
   - Y: Alter structure
   - N: Check longitudinal stresses
     - Y: Unsatis.
     - N: Unsatis.
9. Check ultimate load, deflection. Estimate reinforcement
   - Y: Satis.
   - N: Satis.

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Detailed cable profiles

Produce cable data on profiles, friction factors, estimates of creep and shrinkage, stressing force

Run cable prestressing programme; output total prestressing force and centroid of transfer & working states

Plot PE and P diagrams, calculate parasitic moments

Check final longitudinal stresses using computer

Ultimate load, deflection, principal tensile stress checks

Finite element analysis of side span

Analysis of transverse deck moments. Deck reinforcement

Plot cable positions from computer output

Print cable setting-out tables

Is adjustment of cable forces adequate?

Y

N

satis.

unsatis.
Design procedure for a trail suspension bridge for remote areas

<table>
<thead>
<tr>
<th>Results of the site survey</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge site in plan and cross section 1:200 with high flood level, axis points, benchmarking</td>
</tr>
</tbody>
</table>

Check bridge site selection and axis line | Check triangulation and tacheometric survey | Check high flood level | Check bridge type selection |

Fix free-board

Draw rock face and profiles of exploration pits in cross section

Fix walkway and tower foundations on both banks - tower axes - elevation of top of foundation

Determine camber and span
Select appropriate tower design. Determine dead load sag.

Determine appropriate location of main cable anchorages

Check layout

Select related walkway and tower foundation design. Estimate number and diameter of main cables and windguy cables; determine loads

Sag calculation

Check results

Sag calculation

Check results

Draw towers, top of walkway and tower foundations, walkway, and main cables in plan and section. Transfer the results of the sag calculation to General Arrangement drawing

Compile loading for walkway and tower foundations on both banks
  - tower loads for two loading cases
  - spanning cable tension for two loading cases
  - earth pressure parameters, ground water level

1

2
Determine preliminary type (with/without foot) and dimensions of walkway and tower foundations

Calculate walkway and lower foundation for both banks

Check results

Draw walkway and tower foundations in plan and cross section for both banks, including dimensions and levels

Select related main cable anchorage design. Determine location of main cable anchorage on both banks with final cable inclination:
- distance from tower axis
- cable elevation

Compile loading for main cable anchorages on both banks:
- full load main cable tension
- surcharges (soil, walls, etc.)
- earth pressure parameters
- ground water level

Estimate required dimensions of main cable anchorages on both banks

Calculate main cable anchorages for both banks

Check results

Draw main cable anchorages in plan and cross section for both banks, including dimensions, distances and levels
Calculate suspenders

Draw suspenders in section. Transfer results of suspender calculation to standard design drawing “Suspenders”

Select, design and draw in General Arrangement as far as required:
- side stay cables
- side stay cable anchorages
- stabilizing cables
- diagonal stabilizers

Select the appropriate temporary tower stay arrangement for tower erection

Layout windguy cables. Select appropriate system and position of vertex according to the anchorage conditions in the river banks

Draw windguy cables in plan. Compile design parameters for calculation

Draw cross section of the river bank along all four cable ends. Fix front of anchorages

Calculate windguy cables, windties; transfer results to General Arrangement drawing

Select appropriate windguy cable anchorage drawing for all four cable ends
Compile anchorage loading for all four anchorages:
- full wind load cable tension
- surcharges (soil, walls, etc.)
- earth pressure parameters
- ground water levels

Estimate dimensions for all four anchorages

Calculate windguy cable anchorage

Check results

Draw windguy cable anchorages in plan and cross section, including dimensions and levels

Design and structural analysis of adjacent works:
- retaining walls
- drainages
- river bank protection
- slope protection
- approach trails

Complete 1:200 General Arrangement drawing

Produce standard structural design

Produce standard working and assembly drawings

Compute quantities

Prepare tender documents for steel works

END OF DESIGN
Appendix C

Different structural forms for tall buildings [50]

C.1 Braced-frame structures

In braced frames, the lateral resistance of the structure is provided by diagonal members that, together with the girders, form the “web” for the vertical truss, with columns acting as the “chords” (refer to Figure C.1). Because the horizontal shear on the building is resisted by the horizontal components of the axial tensile or compressive actions in the web members, bracing systems are highly efficient in resisting lateral loads. This type of structural forms are suitable for resisting lateral loads for buildings up to 40 or 50 stories.

Bracing is generally regarded as an exclusively steel system because the diagonals are inevitably subjected to tension for one or the other directions of lateral loading. Concrete bracing are sometimes used but in the form of double diagonals with each diagonal designed as a compression member to carry the full external shear. There is minimum additional material needed for bracing thus it is an economical choice for any height of building, up to the very tallest. In addition, generally the bracing has no influence on the girder which means that the floor framing design is independent of its level in the structure that enable repetitive and economical design of the girders.

However, bracing usually interferes with the internal planning and the locations of windows and doors. For this reason, the braces are normally hidden within the walls or partition lines, and also around elevators, stairs and service shafts. In order to provide more planning spaces, braces can be either concentric braced or eccentric braced.

In areas of low seismic activity, concentric bracing is preferred as there is no need for ductility consideration. The eccentric bracing where the axis offsets introduce flexure and shear into the frame has lowered the stiffness to weight ratio but increases ductility. It is also possible to have large scale bracing as shown in Figure C.2.
Figure C.1 Braced frames and other various different types of bracing

Figure C.2 Large-scale braced framing
C.2 Rigid-frame structures

Rigid frame structures consist of only columns and girders to provide the rigidity of the structure with moment resistant connections (refer to Figure C.3). This framing is economical for buildings up to about 25 stories only in its typical 20ft (6m) – 30 ft (9m) bay size. For above that height, large members would be required to resist the lateral stiffness required which turned out to be uneconomical.

Rigid frame construction is ideally suited for reinforced concrete buildings because of the inherent rigidity of reinforced concrete joints. For steel construction, the moment resistant connections tend to be expensive. As the columns and girders participate in the overall sidesway due to lateral loading, the designs at each level cannot be repetitive under economical constraints. Sometimes, it is not possible in the lowest stories to accommodate the required depth of girder within the normal ceiling space.

The strength and stiffness of the frame is proportional to the column and beam size and inversely proportional to the story height and column spacing. In general, an increase in beam stiffness has a greater effect on the frame stiffness than an increase in column stiffness.

![Figure C.3 Rigid frame](image-url)
C.3 Infilled-frame structures

Frames infilled with materials such as panels of brickwork, blockwork, or cast-in-place concrete can support building up to 30 stories in height. The infills (Figure C.4) serve as struts along its compression diagonal to brace the frames when the frame is subjected to lateral loading. However, due to the complex interactive behavior and the probability of removal of the infills during the life time of the building, these structures tend to be designed according to the typical frame structures ignoring the contribution effects from the infills.

![Infilled frame](image)

Figure C.4 Infilled frame

C.4 Flat-plate and flat slab structures

Flat plate systems comprise of uniform slabs of 5-8 inches (12-20 cm) thickness connected rigidly to supporting columns. It is one of the economical way of construction as there are no complicated formwork as the soffit can be used as the ceiling and create a minimum possible floor depth. This system is economical for spans up to 25ft (8m) and can be increased to 38ft (12m) if drop panels are added to create a flat-slab structure. In
terms of story height, the lateral resistance relying entirely on flat-plate or flat-slab system can support building up to 25 stories economically. However, in situations where the wind design are less stringent, buildings up to 40 stories are found to be performing satisfactorily using flat plate system.

This system resist the lateral loads in similar way as the rigid frame system with the slab acting as the girders in the rigid frame system i.e. the lateral resistance depends on the flexural stiffness of the columns and the slabs and their connections.

C.5 Shear wall structures
The walls in shear wall structures are used to resist the lateral loading. For a linear member to be considered as a shear wall, the aspect ratio of its section must be greater than 5 and the length greater than 10 times the smaller dimensions of the section.

Normally, the walls act as cantilever from the ground in the form of separate planar walls or/and as nonplanar assemblies of connected walls around elevator, stair, and service shafts (Figure C.6). Due to the higher rigidity horizontally compared to rigid frame, shear wall structures can support buildings up to about 35 stories.

The requirement of continuity of the shear walls from ground to top of building has restricted the floor planning of the building. However, this system is ideal for hotels and residential buildings where the floor to floor layout are similar and in addition, the walls provide excellent acoustic and fire insulators between rooms and apartments.

In consideration of low and medium-rise buildings where shear walls are combined with frames, it may be reasonable to assume that all the lateral loads are supported by the shear walls with the frames supporting only the gravity loads. To avoid tensile forces in the walls (leading to more reinforcement), the shear walls need to be strategically placed to ensure that the tensile stresses (due to lateral loads) are suppressed by the gravity load stresses.
C.6  Coupled shear wall structures

Coupled shear wall structures are walls that are aligned in the same plane, or almost the same plane, connected at the floor levels by beams or stiff slabs (Figure C.7). This set of walls with the shear resisting connecting members are considered as a composite cantilever that provides greater horizontal stiffness compared to the uncoupled cantilevers. Coupled shear wall structures can be found in residential construction where lateral load-resistant cross walls, which separates the apartments, consist of in-plane couples pairs or trios of shear walls, between which there are corridor or window openings. There are occasions that coupled shear wall structures make use of heavy steel plate in the form of massive vertical plate or box girders that resist very high shear as in the case of the base of elevator shafts.
C.7  Wall-frame structures

The combination of shear wall and rigid frame can support buildings in the 40 to 60 story range with the efficiency decreasing as the height increases (Figure C.8). The decrease in efficiency is due to the large amount of material needed to make the shear wall (either concrete or steel braces) sufficiently stiff and strong.

The unique combination of the flexural behavior of the walls and the shear behavior of rigid frames have led to a stiffer and stronger structure especially at the top of the building. The interaction of the two systems is shown in Figures C.9 and C.10. In addition, some of this system has enabled a uniform shear in the frame over the height of the building and thus making it to possible to use the concept of repetitive floor framing.

In addition to concrete structural form, as a substitute, steel braced frames and steel rigid framing can be used that also resulted in the similar flexural and shear behavior respectively.
Figure C.8 Wall-frame structure

Figure C.9(a) Uniform lateral loading on wall; (b) Uniform lateral loading on frame; (c) wall-frame structure subjected to uniform lateral loading
Figure C.10 (a) Deflection diagram for laterally loaded wall-frame structure; (b) typical moment diagrams for components of wall-frame structure; (c) typical shear diagrams for components of wall-frame structure

C.8 Framed-tube structures

Framed-tube structures consist of very stiff moment resisting frames at the perimeter of the structure thus forming a “tube” around it. The frame consists of closely spaced columns, 6-12 ft (2-4m) between centers, joined by deep spandrel girders (Figure C.11). The tube can be considered as a thin-wall structure that is highly efficient in resisting lateral loading. In addition, the tube also shares the gravity loading with the interior columns and walls.

Under lateral loading, the frames in the direction of the loading can be considered as the “webs” of a massive tube cantilever and those perpendicular to the loading direction as the “flanges”. There is an imminent inefficiency in the use of the “flange” columns due to the effect of shear lag that cause the mid-faced “flange” columns are being stressed less than that of the corner columns, therefore not contributing as fully as they could to the flange action. Despite this inefficiency, the overall efficiency of this system still remains high. In fact, this tube system is suitable for both steel and concrete construction and has

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been used for buildings ranging from 40 to more than 100 stories. However, the economical range is found to be from 40 to 70 floors. Again, the repetitive nature of the framing system has allowed fast construction.

Figure C.11 Framed-tube

The close spacing of the columns in the frame-tube poses a problem to the entrance of the ground floor. This is normally accommodated with columns terminating at a transfer beam, a few stories above the base so that only a few, larger, more widely spaced columns continue to the base. The tube forms are applicable to square, triangular or circular base configurations.

C.9 Tube-in-tube or hull-core structures

In addition to having an exterior perimeter tube, it is also possible to include an internal elevator or service core creating a tube-in-tube system (Figure C.12). This core and the perimeter hull act together in resisting both the vertical and lateral loading. Usually, the perimeter hull (with a larger dimension) will dominate the lateral stiffness with the inner core providing more gravity resistance and also contributes to the flexural resistance as in the core wall structures.
**C.10 Bundled-tube structures**

The prominent bundled-tube structure is the Sears Tower in Chicago. Figure C.13 shows the plan diagram of the bundled-tube structure. The interconnected nine “bundled” tubes interact such a way that it reduces the shear lag effects as mentioned in the frame-tube system. This is accomplished by the additional internal “webs” of the vertical cantilever that reduces the shear lag in the middle of the windward and leeward flange frames. As such, the columns are more evenly stressed than in the single-tube structure.

Furthermore, the columns can be spaced further apart and thus create more planning spaces and less obstructions. As shown in Figure C.13, the bundled tubes can be reduced and yet remain effective as the building height increases.

This system is economically applicable from 55 to 110 stories and the potential for greater heights is feasible.
C.11 Braced-tube structures

A braced-tube structure will include diagonal bracing at the faces of the tube (Figure C.14). This enables better distribution of the loading to the mid-face columns such that they virtually eliminate the effects of shear lag in both the flange and web frames. Such arrangement allows the building height to increase (from the single tube structure) and greater spacing between the columns. The bracing can be of steel in the form of bracing that traverses across the faces of the rigid frames or in the concrete structure as a diagonal pattern of concrete window-size panels poured integrally with the frame. The economical height of this system ranges from 40 to 80 stories.

Besides the improved lateral resistance, the braced-tube structure also improve the gravity resistance as the differences between the axial load stresses in the columns are
even out by the braces transferring axial loading from the more highly stressed columns to the lesser stressed ones.

![Figure C.14 Steel-braced tube (left) and concrete-braced tube (right)](image)

**C.12 Outrigger-braced structures**

Outrigger-braced structures comprise of either braced frames or shear walls and horizontal cantilever "outrigger" trusses or girders that are connecting a central core to the outer columns (Figure C.15).

Upon lateral loading, the outrigger-braced structure is restrained from lateral movement by the tension in the windward columns and the compression in the leeward columns. The outrigger joins the columns to the core to make the structure behave as a partly composite cantilever. All the perimeter columns can thus participate by having horizontal truss or girder around the faces of the building. At the outrigger level of normally one or two-story depth, it is also used as the location for plant levels in the building.
Outrigger-braced structure has been used for buildings from 40 to 70 stories high with great potential for greater heights. The efficiency of the system depends on the number of outriggers being used in the structure. However, the increase in efficiency decreases with each additional outrigger and four or five levels of outrigger has been found to be most economical.

C.13 Suspended structures
Suspended structures consist of central core or cores that has a horizontal cantilevers at roof level, to which vertical hangers of steel cable, rod or plate are attached and the floor slabs are suspended from these hangers (Figure C.16).
The advantages of this type of structural form include:
- Ground story can be free of obstructive columns
- Hangers being in tension can be designed more effectively thus possibly making the cross section of high strength steel small provided that the hangers are fire and rust proofed. The latter usually caused the hangers to be substantially bulky instead.
- The construction of the core, cantilevers and hangers can be done concurrently with the floor slabs being casted on top of each other on the ground level waiting to be lifted in sets to be fixed in position.

The disadvantage of the system include:
- With the limited size of the core, the building height is also limited.
- The transferring of gravity load via the hangers to the upper level cantilevers and then returning to the ground via the core is inefficient.
- An extension of the hangers (from zero strain to maximum strains) due to loading of the live loading may lead to significant changes in the levels of the edges of the slabs.
- This excessive strain increments lead to a restriction of up to 10 number of floors that can be supported by a single length of hanger. As such, multilevel cantilever system may need to be explored (Figure C.17). These cantilever levels usually coincide with the plant levels.
The other possible configuration for this structural form is to include two- and four-core structures (instead of single core) in which the vertical hangers are suspended from massive girders that span between the cores or in which hangers are draped in a cantenary fashion between the cores.

C.14 Core structures

Only one core is used to carry the entire gravity and horizontal loads for core structures (Figure C.18). The layout of the structure could be:

- Slabs being supported at each level by cantilevers from the core
- Slabs are supported between the core and the perimeter columns that then terminate either on major cantilevers at intervals down the height or on a single massive cantilever a few stories above the ground.
Due to the limited structural depth in the main core, this type of structural system does not deliver as an efficient cantilever structure to resist lateral loads. In addition, there are many components of the structure being cantilever that is also inefficient. However, the merit of the system is more architectural that allows for column free perimeter at the ground level and at other levels just below the cantilevers.

C.18 Core structure

C.15 Space structures

Space structures essentially consist of three-dimensional triangulated frame whose members are designed to resist both the gravity and lateral loading. Due the incorporation of the third dimension, the structure tends to be relative lightweight with a potential for achieving the greatest heights with high efficiency.

A classic example of this structural form is the 76-story Hong Kong Bank of China Building (Figure C.19 left). The unique geometry of the structure requires considerable structural ingenuity in transferring both the gravity and lateral loading from the floors to the main structure. One possibility as illustrated in Figure C.19 (right) is to have an inner braced core that collects the lateral loading and the inner gravity loading, from the slabs
over a number of multistory regions. At the bottom of each region, the lateral and gravity loads are transferred out to the main joints of the space frame.

Figure C.19 Space structure of Hong Kong Bank of China Building (left); Space structure with unique bracing (right)

C.16 Hybrid structures
Many of the “modern” buildings have prismatic shape, tower or block such that the entire building can be identified making up of single system such as a tube or a wall-frame structural form. However, the “postmodern” buildings are more non regular in shape with larger scale cut-outs, flutings, facets, and crowns that defy classifications in their intricacy and variety. Thus, the engineer would improvise combinations of two or even more of the basic structural forms being used for such non prismatic shaped buildings. Besides, with larger computational power, it is now feasible to consider combinations on
the basis of their effects on the appearance and functioning of the building and also its constructability.

Figure C.20 Hybrid structures

This could be done as direct combination in a superimposed tube and outrigger system (Figure C.20 left) or by adopting different forms in different parts of the structure in a tube system on three faces of the building and a space frame on a faceted forth face (Figure C.20 right).

The combination of the various structural forms requires the engineer to have a sound knowledge of both the individual structural form and its behavior. Some of the possible risks include the abrupt discontinuities in building stiffness, the long-term effects of
differential axial shortening, and other possible side effects when combining mixed systems and materials. All the above will provide the entry engineer an idea of the types of structural forms that are used and also currently being adopted in some tall building designs.
Appendix D

Preliminary sizing of structural elements

Strength method for estimating concrete elements
Design strength ≥ Required strength
Bending Strength $\phi M_n \geq M_u$ (e.g. $1.4M_D + 1.7M_L$)
Axial strength $\phi P_n \geq P_u$
Shear Strength $\phi V_n \geq V_u$
Torsional Moment Strength $\phi T_n \geq T_u$

The capacity reduction factors are:
- Flexure and axial tension in reinforced concrete $\phi = 0.90$
- Shear, torsion, bond and anchorage $\phi = 0.85$
- Axial compression for tied columns and bearing on concrete $\phi = 0.70$
- Flexure in plain concrete $\phi = 0.65$

D.1 Concrete elements - slabs

D.1.1 Introduction of concrete elements

For this section of concrete elements, there are many flowcharts pertaining to the design procedures for reinforced concrete as in the Design handbooks in Accordance with the strength design method of ACI 318-89. There are flow charts pertaining to the design of concrete beams, one way slabs, fanged section and also the tension development length for reinforcement. In addition, comprehensive design procedures are provided for columns and two-way slabs together with design tables and design curves. As such, this section will only explore the simple preliminary element sizing and the rules of thumbs involved.

Concrete covers

The minimum clear cover to the reinforcement for cast-in-place concrete, as protection against corrosion by weather or loss of strength from fire exposure, is:
Concrete cast against and exposed to earth: 3 in.
Concrete exposed to weather:

#6 though #18 bars 2 in.
#5 bars and smaller 1½ in.

Concrete not exposed to weather:
Slabs, walls, joists #11 bar and smaller ¾ in.
Slabs, walls, joists #14 bar and larger 1½ in.
Beams, columns, primary reinforcement, ties, stirrups, spirals 1½ in.

Concrete element’s effective depth

The effective depth, from the compression face to the center of the steel reinforcement can be approximated and related to the member thickness, t, as:

Beams (interior, for exterior exposure subtract 0.5 in.):

Single layer (always for top steel of T-beam): \( d = t - 2.50 \)
Double layer: \( d = t - 3.50 \)

Joists: \( d = t - 1.25 \)

Slabs:

One-way slabs \( d = t - 1.00 \)
Two-way slabs (center of upper layer): \( d = t - 1.50 \)

Economical span ranges for cast-in-place reinforced concrete floor systems are:

20-25 ft: Flat plate construction for typical high rise buildings
25-30 ft: Banded slab construction
Flat slab construction for heavy loading conditions, such as found in industrial buildings and warehouses.
35-40 ft: One-way beam and slab construction
Joist slab and skip-joist construction
Waffle slab construction when exposure is desired or heavy loads must be supported
L> 40ft: Post tensioned flat slab construction
D.1.2 One way slab

Preliminary slab thickness estimate where \( t \) (in.) and \( L \) (ft.):

Simply supported: \( t = L/2 \geq 4\text{in. for fireproofing} \)

Cantilever: \( t = L \geq 4\text{in.} \)

Continuous both ends: \( t = L/3 \geq 4\text{in.} \)

For ordinary solid concrete slabs, which have a much lower steel ratio than beams, often \( z = 0.95d \) is taken as first approximation, which results in the following required moment reinforcing:

\[
A_s = \frac{M}{0.85f_y d} \geq A_{s_{\text{min}}} \quad \text{Equation D.1}
\]

D.1.3 Two-way slabs

For the two-way slab, it may be possible to imagine perpendicular strips along the axes of symmetry, each of the shallow slab beams behaves as approximately a one-way, single curvature system, sharing the load equally. Then, the approximate moment at the center in each direction is:

\[
M_{\text{max}} = \frac{(w/2)L^2}{8} = 0.0625wL^2 \quad \text{Equation D.2}
\]

In reality, however, the strips are not interconnected and isolated, they are continuous with their boundaries, which provide torsional resistance to bending. The ACI empirical moment coefficient is then used (refer to Building Code Requirements for Reinforced Concrete ACI 318-63). Typically, the two-way slab proportion is in the range of \( m = 1 \) to \( 0.7 \) where \( m = L_s/L_L, L_s \) being the length of the shorter span and \( L_L \) being the longer span of the two-way slab. Then the maximum moment (dependent on value of \( m \)) is empirically shown as:

\[
M_{\text{max}} = (\text{coeff.})W_uL_s^2 \quad \text{Equation D.3}
\]

In this case, the approximate maximum moment is:
D.1.4 Precast concrete slab

Most common is the hollow core slab and double tee sections which tend to be pretensioned and have a thin, cast-in-place concrete topping from 2 to \(3\frac{1}{2}\) in. thick that will act compositely with the precast slab. The typical 4-ft wide hollow core slabs range in thickness from 4 to 12 in., and have a simple span range from 15 to about 38 ft. The maximum recommended span-to-depth ratio is 40 or \(t = \frac{L}{3.33} = 0.3L\).

The weight of hollow core slabs without topping varies with the manufacturer. For preliminary design purposes, a 45psf slab weight may be assumed for 6 in. hollow core slabs, and 57psf for 8 in. slab.

D.1.5 Composite one-way slab system

The cast-in-place concrete can act together with steel decking or precast concrete components. The most common steel decks for floors are 1½ and 3 inches in depth, with rib spacing of 6 to 12 in., but also 2 in. deep decks are available – they range between 0.0596 in. (16 gauge) to 0.0295 in. (22 gauge) in thickness. Their capacity depends on the depth, profile, and metal thickness and type of temporary or permanent supports.

The minimum overall slab thickness should be 3½ in. with a minimum concrete cover of 2 in. above the deck; this concrete cover varies typically from 2½ to 4 in. Economical span ranges for 1½ in. composite cellular rib decks up to about 10 ft. and, for 3 in. decks up to about 15 ft. Most composite floor decks are treated as simple supported slabs and thus the minimum thickness should be \(t \geq \frac{L}{2} \geq 3\frac{1}{2}\) in.

The flexural stress for the steel deck during the non-composite construction stage under the dead load, \(w_d\), which includes the wet concrete and construction loads of 20psf are:

\[
F_b = \frac{M_y}{S_d} = \left(\frac{w_d L^2}{8S_d}\right)12 = 1.5w_d L^2/S_d \leq 0.6F_y
\]

Equation D.5
If the stresses are found to be larger than the allowable stresses, either a stronger deck must be selected or shoring must be provided during construction to reduce bending.

**D.1.6 Flat slabs and flat plates**

Typical flat slab/plate thicknesses in high rise construction are 5 to 10 inches. The flat slab/plates system without beams are envisioned as hidden slab beams that span from column to column and the plate is to behave similarly to the two-way slab on beam. Hence, the approximation also includes the column strips as:

Considering a typical interior span, the ACI code would have 65 percent of the total static moment distributed to the supports while the rest of 35 percent is distributed on middle third span. Accordingly, the empirical ACI code specifies that the stiffer column strips resist 75 percent of the support moment $0.65M_0$ (Figure D.1)

$$M_1 = 0.75(0.65M_0) = 0.49M_0$$  \hspace{1cm} \text{Equation D.6}

The more flexible middle strip resists the remaining 25 percent.

$$M_3 = 0.25(0.65M_0) = 0.16M_0$$  \hspace{1cm} \text{Equation D.7}

![Figure D.1 Moments on the interior of a flat slab (Courtesy of Schueller)](image)

Similarly, but with a smaller portion of the total, the column strip resists 60 percent of the positive field moment $0.35M_0$. 

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The shear in the flat slab/plate is critical as the columns tend to punch through the relatively thin slab i.e. – critical punching shear. The shear strength of the concrete, in two-way action at \(d/2\) distance from the edges of the column is:

\[
\phi V_e = \phi (4 \sqrt{f'_c}) b_0 d 
\]

where \(\phi = 0.85\),

\[b_0 = \text{perimeter of critical section (in.)}\]

\[f'_c = \text{concrete strength (psi)}\]

\[d = \text{effective depth of slab (in.)}\]

**D.1.7 Plates and shells**

The usual height-to-span ratio for folded plate structures is in the range of \(h/L=1/10-1/15\). A reasonable slope would be \(25°-45°\) so that the assumed behavior is possible and the material is efficiently used as for the V- and W-shaped folding [37].

Typical concrete shell thickness range from 3 to \(4\frac{1}{2}\) in. for concrete domes having a span range of 100-200 ft, with an increase of shell thickness of approximately 50-75% near the periphery.

The usual height-to-span ratio for long shell varies from 1/10 to 1/15, although there is no precise point at which a cylindrical shell’s shape can be considered as “long”. In general, the beam theory may be applied to long cylindrical shells that are of symmetrical cross-section and under uniform loading, and which meet the following span-to-radius of curvature conditions:

**Single shells without edge beams:** \(L/R>5\)

**Single shells with edge beams that are not too deep:** \(L/R>3\)

**Interior shells of a multiple system:** \(L/R>2\)

**Interior shells of a multiple system with edge beam:** \(L/R>3\)
D.2 Concrete elements - beams

D.2.1 Deflection limit
The preliminary sizing of flexural members depends on stiffness and strength considerations. Usually, stiffness controls the design of flexible elements such as slabs, joists, shallow beams, and long spans beams. For this condition, the minimum member depth can be determined from flexibility considerations. The ratio of deflection to span $\Delta/L$ of a beam can be expressed in terms of its span-to-depth ratio, $L/t$, multiplied by a constant $C$

$$\Delta/L = C(L/t)$$  

Equation D.11

In order to avoid the complex deflection calculations for reinforced concrete, limiting $L/t$ ratios are given. Hence, the following approximate minimum member thickness with $t$ (in.) and $L$ (ft) for various cases are:

- Cantilevers: $L(12)/t = 8$ or $t_{min} = 1.5L$
- Simple span beams: $t = L/1.33 = 3L/4$
- Continuous-span beams: $t = L/1.5 = 2L/3$

These values are based on normal-weight concrete and Grade 60 steel. For grade 40 reinforcement, the beam depth may be reduced by 20 percent, but it must be increased by 20 or 10 percent for 90pcf or 120pcf lightweight concrete respectively.

D.2.2 Moment capacity
Size of ordinary floor beams is controlled by the flexural compressive strength at the maximum moment location, where the resisting compressive cross section is the smallest. In this context, the equation for selecting the concrete depth is base on the typical condition of Grade 60 steel, together with an average reinforcement ratio of $\rho = \rho_{max}/2 = 1\%$, so that the bars can be easily placed and also provide reasonable deflection control.

For this condition, the coefficient of resistance is: $R_n = \phi R_n = M_u/b_n d^2 = 0.52ksi$. This expression is now changed in form as given below, but using mixed units – it is
applicable to beams with common concrete strengths of 3000 to 6000 psi. The equation can also be represented in terms of the service moment, $M$, by assuming an average load factor of 1.5:

$$bd^2 = 23Mu = 35M$$  \text{Equation D.12}

where $M_u =$ ultimate bending moment (ft-k) = 1.5$M$

$b =$ beam web width (in.)

$d =$ effective depth of beam (in.)

Note that the expression is in mixed units.

From this equation, the beam depth can easily be found by assuming a typical beam width of about 10 to 16 inches. For Grade 40 steel, an approximate minimum reinforcement of 1.6 percent is required, with 1.3 percent for Grade 50 ksi steel, rather than the assumed 1 percent for 60 ksi steel.

Another possible approach is by using the working stress on the balanced condition of a rectangular section and developed an empirical formula of:

$$d = 1.45l_n \sqrt{w/u/b_w}$$  \text{Equation D.13}

where $w_u =$ uniform ultimate load (k/ft)

$l_n =$ clear beam span (ft)

$d =$ effective beam depth (in.)

$b_w =$ beam stem width (in.)

For the preliminary design purpose of a general typical beam width $b = 10$ to 16 in., with at least 1 percent of Grade 60 steel, but using the span length, $L$, from center to center of supports.

$$d = 0.4L \sqrt{w_u}$$  \text{Equation D.14}

It must be kept in mind that when selecting the beam proportions that wide, shallow beams may be more economical from an overall construction point of view than the narrow deep beams that the designer might be more used to.
When the cross section is known, the moment reinforcement can be found as shown below. Rotational equilibrium, as shown in Figure D.2, necessitates the balance of the acting moment $M_u$ and the steel strength $A_s f_y$, which is then reduced by the capacity reduction factor, $\phi = 0.9$.

$$M_u \leq \phi M_n = \phi T_z = \phi A_s f_y z$$

Equation D.15

From this equation, the approximate moment reinforcement for rectangular beams, joists, and T-beams can be found by using an average internal lever arm length of $z = 0.9d$.

$$A_s = \frac{M_n}{(0.8 f_y d)} \geq p_{min} b_n d = 0.2(b_n d)/f_y$$

Equation D.16

where

- $A_s =$ moment reinforcement (in.$^2$)
- $M_u =$ ultimate moment (in.k)
- $f_y =$ yield stress of steel (ksi)
- $d =$ effective beam depth (in.)

![Figure D.2](image)

Figure D.2 Equivalent stresses of a typical single reinforced concrete beams under ultimate load (Courtesy of Schueller)

This equation gives reasonable results for typical beam steel ratios of 1 percent to about 1.6 percent. For this range, it is quite insensitive to the various common concrete strengths and the variation of the steel ratio, as is apparent from the closely bundled and nearly straight lines of the strength curves for a given steel. For fast approximation
purposes, however, this equation may even be used for its entire permitted range, as long as the steel ratio is less than $p_{\text{max}}$, which is given in below for various steel concrete combinations. Attention must be given to 3000psi concrete together with 60ksi steel, where the equation becomes less precise beyond a steel ratio of about 1.1 percent.

<table>
<thead>
<tr>
<th>Fy (psi)</th>
<th>fc' (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3000 ($\beta_1=0.85$)</td>
</tr>
<tr>
<td>40000</td>
<td>0.0278</td>
</tr>
<tr>
<td>50000</td>
<td>0.0206</td>
</tr>
<tr>
<td>60000</td>
<td>0.0160</td>
</tr>
</tbody>
</table>

Table D.1 Maximum reinforcement ratios for singly reinforced rectangular beams

Similar equation is used for T-beam behavior of the composite beam-slab at midspan (Figure D.2), where the flexural capacity of the concrete is so large that the stress block depth, $a$, usually lies within the flange (slab), so that the T-section can be treated as a wide, shallow rectangular beam section of width, $b_e$, for preliminary design purposes; here only the minimum steel ratio $p_{\text{min}}$ must be checked.

Generally, Grade 60 bars are used – Grade 40 requires 50 percent more steel. Substituting Grade 60 steel in the equation results in an expression for beam design which, however, is also often used for slab design, when fast estimates are needed; the equation is in mixed units.

$$A_t = M_u / 4d \geq b_e d / 300$$

Equation D.17

where $M_u =$ ultimate moment (ft-k)

$A_t =$ moment reinforcement (in.$^2$)

$d =$ effective beam depth (in.)

Notice that the minimum reinforcement for this condition is $p = -0.335$, the maximum reinforcement ratio, $p_{\text{max}}$, which ensures that the beam is under-reinforced and fails in tension, does not have to be checked if the moment reinforcement equation is only
applied to beams with reinforcement ratios of up to about 1.6 percent; however, one must watch out for the combination of 3000psi concrete and Grade 60 steel with \( p_{\text{max}} = 1.61\% \).

According to the working stress method, the following is used as first approximation
\[
z = jd = \frac{7d}{8}
\]
\[
A_s = \frac{M}{f_s z} = \frac{M}{f_s jd} = \frac{M}{0.875 df_s}
\]
Equation D.18

In this case, the tensile stresses in the reinforcement are limited to \( f_s = 20\text{ksi} \) for Grade 40 and Grade 50 steel, and \( f_s = 24\text{ksi} \) for Grade 60 and higher strength steel.

Another equation often used for the approximate design of bending reinforcement, as based on the service loads and ordinary loading conditions of \( D/L = 2/1 \), can be derived as follows:
\[
M_u = 1.4 (2M/3) + 1.7 (M/3) = 1.5M
\]
Equation D.19
\[
A_s = M/0.8f_s d = \frac{1.5M}{(0.8)(60)d} = \frac{M}{32d}
\]
Equation D.20

Notice that the equation does not have any mixed units, the moment is in (in.k) and \( d \) is in inches.

### D.2.3 Shear capacity

The stirrup spacing should be made in increments of not less than \( \frac{1}{2} \) in. Using the usual maximum stirrup spacing of \( s = d/2 \), smaller spacing intervals can be continued for standard conditions of \( d/3 \) and for, under most conditions, the closest spacing of \( d/4 \), considering that the stirrups should not be closer than about 3 inches.

General design process may be used:
- There is no shear reinforcement required for beams if
  \[V_u \leq \phi V_c / 2 = \phi \theta_v d \sqrt{f'c'}\]
  Equation D.21
- There is a minimum shear reinforcement required for beams (but not for shallow beams, slabs, footings, and joist construction), if
  \[\phi V_c / 2 < V_u \leq \phi V_c\]
  Equation D.22
There is shear reinforcement if

\[ V_u > \phi V_e \]  

Equation D.23

D.2.4 General considerations for sizing of beams

With respect to the various approximate beam sizes, it may be concluded that:

- Any beam proportion is possible, as long as the depth is at least equal to the one required for deflection control.

- The shear strength is directly related to the cross-sectional area of beam and this does not influence the beam proportion, but only its size; it only becomes critical for short span beams under heavy loads or beams with unreinforced webs. The moment capacity is, however, primarily affected by the square of the depth, making deep beams more efficient from a local material point of view, as reflected by the lower steel ratios. The usual depth-to-width ratio for shorter spans is 1.5 to 2.0, while for larger spans the ratio may be 2.5 to 3 or larger. However, it must be kept in mind that, from an overall point of view, it may still be more economical to use wide, shallow beams rather than narrow deep beams. For instance, in pan joist construction, the supporting beams often have the same depth as the joist, in order to reduce formwork cost rather than material costs, and to reduce the overall building height. Furthermore, by changing the widths of the beams, only the bottom forms are affected, but not the side forms and shores. Shallow, wide beams are first checked with respect to depth according to deflection control, and then the width is found from flexural requirements. The beam widths are usually multiples of 2 or 3 inches. Often, constant beam sizes are used for one building story by only changing the reinforcement to the span and load variations.

- The beam proportions are also influenced by the placement of flexural reinforcement. In narrow beams, several layers of longitudinal steel may be required. For different combination of longitudinal reinforcement steel and stirrups, there would be a minimum beam widths to comply with the cover required for the beam. Also the shear reinforcement has an effect upon the beam sizes, small beams, for instance, may need very close stirrup spacing. Additionally, it may be advantageous to make beams
at least 2 inches wider than narrow columns, so that the bars in the beam corners can pass unobstructed.

- Beams width are frequently made equal to the double-layered bricks such that the edges of the beams flushes with the plastering of the bricks thus improving on the aesthetics at the intersections of beam and bricks.

**D.2.5 Concrete wall beams**

The deep concrete beams are considered as one with depth-to-span ratio $h/L_n$ exceeding 0.2. At approximately $h/L_n > 0.4$ for continuous spans, and $h/L_n > 0.8$ for simple spans, loads can be transferred to the supports, primarily in direct arch action with the corresponding thrust. A deep beam acts more like a surface element or plate where the strain distribution is no longer liner and the shear deformations cannot be ignored, as in the common shallow beams. The elastic flexural stress distribution is likewise no longer linear as is illustrated in Figure D.3. Here the positive moment range, the compressive stresses in the concrete are usually not critical. They are small in comparison to the magnitude of the tensile stress trajectories at midspan.

For preliminary design purposes, it may be taken for the deep beam case as $z = 0.5L$ or $z = 0.6h$, whichever is smaller. For example, for a simply supported wall beam that carries a uniform load on top and has a span equal to its height, $h/L = 1$, the approximate flexural reinforcement at the bottom can be determined as follows:

\[
M_u \leq \phi M_n = \phi A_d f_y z
\]

\[
W_u L^2/8 = 0.9 A_d f_y (0.5L), \quad \text{let} \ W_u = w_u L
\]

\[
A_s = 0.28 W_u f_y \geq A_{\text{min}} = 0.2 b d / f_y
\]

Equation D.24

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D.3 Steel beams

D.3.1 Introduction
Typical steel beams for floor framing in high rise building construction are the common W-sections, open web steel joists, castellated beams, stub girders, plate girders, and tapered and haunched-taper beams.
For long spans, for instance beyond 100ft, deeper and lighter beam sections should be considered: plate girders (single-web or double-web) or trusses (that is, built-up members) can be employed. A useful way to start a design would be to know the limits of the span range for the structural system to be adopted as shown in the table below.

<table>
<thead>
<tr>
<th>System</th>
<th>Span range(m)</th>
<th>Approximate depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate girder</td>
<td>6&lt;L&lt;25</td>
<td>L/15-L/20</td>
</tr>
<tr>
<td>Open web joists</td>
<td>3&lt;L&lt;36</td>
<td>L/18-L/22</td>
</tr>
<tr>
<td>Fink Truss</td>
<td>10&lt;L&lt;?</td>
<td>L/4-L/5</td>
</tr>
<tr>
<td>Howe truss</td>
<td>10&lt;L&lt;33</td>
<td>L/4-L/5</td>
</tr>
<tr>
<td>Bow string truss</td>
<td>18&lt;L&lt;36</td>
<td>L/6-L/10</td>
</tr>
<tr>
<td>Special trusses</td>
<td>23&lt;L&lt;?</td>
<td>L/4-L/5</td>
</tr>
<tr>
<td>Arches</td>
<td>18&lt;L&lt;?</td>
<td>L/3-L/5</td>
</tr>
<tr>
<td>Cables</td>
<td>22&lt;L&lt;?</td>
<td>L/5-L/11</td>
</tr>
</tbody>
</table>

Table D.4 Configuration versus span tradeoffs for steel systems
D.L. Schodek,ting, Structures, Prentice-Hall, 1980

D.3.2 Moment capacity
The design of beam is generally controlled by bending. The familiar flexural stress for symmetrical rolled beam section is

\[ f_b = \frac{M}{S} \leq F_b, \text{ or } S \geq \frac{M}{F_b} \]  
Equation D.25

The section modulus necessary to resist the given moment can easily be looked up. The allowable bending stress, \( F_b \), is dependent upon the lateral support of the compression flange, in addition to the section properties. The section properties refer to slenderness considerations of the compression flange \( (F_y') \) and the web \( (F_y'') \) when an axial load is present. The yield stresses in the parentheses are hypothetical values, above which the flange and web are non-compact.

Most floor beams are fully laterally braced by the concrete slab (i.e. the unbraced length, \( L_b = 0 \)) and nearly all A36 beams have \( F_y \leq F_y' \), so that they can be considered as compact. In other words, the sections are capable of developing their plastic moment
capacity before any buckling occurs. For these conditions, the allowable bearing stress for strong axis bending of doubly-symmetrical members is:

\[ F_b = 0.66F_y \]  
Equation D.26

For weak-axis bending of doubly-symmetrical members, including solid rectangular and round bars, the allowable bend stress is:

\[ F_b = 0.75F_y \]  
Equation D.27

When the compression flange is not braced, as is typical for building columns that have an unbraced length of \( L_b \), larger than the theoretical value of \( L_e \), but which should be kept at less than \( L_u \) for this preliminary approach \( (L_e < L_b \leq L_u) \). Further, should the beam-column web be non-compact \( F_y > F_y' \), or the flange be non compact \( F_y > F_y' \), then the allowable bending stress for those conditions may be taken as:

\[ F_b = 0.60F_y \]  
Equation D.28

**D.3.3 Shear capacity**

The shear stresses for rolled and fabricated shapes, \( f_v = V/A_w = V/(d t_w) \leq 0.4F_y \) rarely control the preliminary design of the beam, as long as the web is not weakened by holes or larger loads are not acting adjacent to the support.

**D.3.4 Deflection limits**

Deflection limitation is generally based on the live load and equal to \( 1/360 \) for beams supporting a plaster ceiling, which should not be fractured. The Commentary on the AISC Specs proposes, as deflection limits for fully stressed floor beams and girders, \( L(F_y/800) \), or \( L/22 \) for A36 steel as a minimum beam depth. When the floor framing is subjected to vibrations with no sources of damping available, a beam depth of at least \( L/20 \) is suggested. Long span beams are usually cambered for dead load deflection.

The deflection of a simple span beam is:

\[ \Delta = 5wL^4/(384EI) = 5ML^3/(48EI) \text{ or } \]
\[ \Delta = ML^2/16EI \]  
Equation D.29
This expression can be further simplified by letting $f_b = (M/t)/I$ as

$$\Delta_e = f_b \frac{(L^2/t)}{1000}$$

Equation D.30

where $f_b$ in ksi, $L$ in ft and $t$ in inches. In the case of full loading conditions, let $f_b = F_b = 24$ksi, so that one obtains,

$$\Delta_e = 0.0248\frac{L^2}{t}$$

Equation D.31

where $L =$ beam span (ft.)

$t =$ beam depth (in.)

$\Delta_e =$ beam deflection due to full loading (in.)

For preliminary design purposes, this expression can also be used for floor framing where the beams carry the concentrated loads of the supporting filler beams. Notice that the deflection is primarily dependent on the square of the span and the beam depth for a maximum bending stress $F_b$; otherwise the deflection increases with an increase in bending stress. Hence, beams of the same span and depth have the same deflection when they are stressed to their allowable limit under full loading conditions.

The corresponding deflection for a fixed beam is $\Delta = 0.2\Delta_e$ and the maximum deflection for a continuous beam may be assumed to be 25 percent of that of the simple beam,

$$\Delta = 0.25\Delta_e$$

Equation D.32

The following rule of thumb (with mixed units) are often found in practice:

- The section modulus of a W-section, $S$, is roughly equal to the product of beam weight (lb/ft), $w$, and its nominal depth (in.), $t$, divided by 10.
  
  $$S_x = \frac{wt}{10} \text{ or } I_x = S_x(t/2) = \frac{wt^2}{20}$$

- The nominal depth (in.), $t$, of a W section is approximately equal to one-half of its span (ft), $L$.
  
  $$T = \frac{L}{2}$$

For the primary beams supporting filler beams, the nominal depth is often assumed as:
\[ T = \frac{L}{1.5} \]

- The beam section weight (lb/ft), \( w \), is roughly 1.25 times the total load (k), \( W \), that the beam must support,

\[ w = 1.25W \]

The maximum deflection (in.), \( \Delta \), is one-tenth of the nominal depth (in.), \( t \),

\[ \Delta_{\text{max}} = \frac{t}{10} \]

## D.4 Composite beams

### D.4.1 Introduction

The composite action of steel beams and concrete slabs, with the aid of shear connectors, is a common construction practice in high rise buildings today for spans larger than 25 to 30 ft; it usually increases the ultimate strength and stiffness of the members by more than 50 percent. The bonding between the interface of the two materials is generally achieved by studs or channel connectors, which are welded to the flanges and resist the horizontal shear, thus prohibiting slippage; limited slippage is allowed in partial composite action, which may be more economical.

For the preliminary design of composite beams with shear connectors, a rough rule of thumb is that the capacity of the steel beam in composite action is increased by \( \frac{1}{3} \), that is, the steel beam alone can be designed for 75 percent of the moment. Often, the steel beam depth is estimated as 80 percent of the non-composite section.

The designer must be aware of the construction stage where the beam alone if not shored must be able to resist the floor dead load and construction loads in non-composite action.

### D.4.2 Estimation of number of connectors

For preliminary design purposes, the neutral axis is assumed to fall within the concrete slab if the composite section, so that the concrete slab is adequate in resisting the total compressive force at ultimate load. Hence, for full composite action, the shear connectors
must be able to transfer the tensile capacity of the steel beam $T_y = A_s F_y$ (see Figure 7.2); in other words, $T_y$ is balanced by the shear load capacity of the connectors $V_h u = 2Vh$ by using a load factor of 2.

$$T_y = A_s F_y = 2Vh$$  \hspace{1cm} \text{Equation D.33}

Therefore, the total horizontal shear to be resisted between the points of maximum positive moment and zero moment is

$$V_h = A_s F_y / 2$$  \hspace{1cm} \text{Equation D.34}

where $A_s =$ areas of the steel beam cross section

The total number of connectors, $N$, for a simple supported beam, for example is equal to the shear, $V_h$, divided by the allowable shear load, $q$ for one connector, and multiplied by 2 for symmetrical loading conditions:

$$N = 2(V_h / q) = A_s F_y / q$$  \hspace{1cm} \text{Equation D.35}

The connectors may be evenly spaced when no concentrated loads are present. For the condition of partial composite action, fewer shear connectors are required than for fully composite behavior.

\section*{D.5 Columns}

The vertical supports for all structures that carry the gravity load to the ground support are columns. In addition, columns can also function to resist the lateral loading by considering it as a cantilever beam from the ground. Columns are frequently made of the steel, reinforced concrete or composite materials.

\subsection*{D.5.1 Steel}

Typical steel columns used in ordinary multistory frame construction are W10, W12, and W14 sections, occasionally pipes and structural tubing are formed. Steel columns can be classified as short or slender.
For short columns, approximately below a slenderness ($Kl/r$) of 30, there is only a decrease of the allowable compressive stress of $0.6F_y$ by approximately 10 percent. For long columns of slenderness more than about 120 but less than 200, Euler’s formula can be used as:

$$F_a = \frac{12\pi^2E}{23(Kl/r)^2} = 149000(Kl/r)^2 \text{ (ksi)}$$

Equation D.36

For columns where the slenderness ratio falls between 30 to 120, a preliminary empirical design is used:

$$F_a = 23 - 0.1KL/r$$

Equation D.37

where $F_a =$ allowable axial compressive stress (ksi)

$K =$ effective length factor

$L =$ actual unbraced length of member (in.)

$R =$ governing radius of gyration (in.)

For preliminary design purposes, it may be convenient to express the required column size in terms of its weight, which is equal to:

$$w_c = \frac{A\gamma}{A_s} = A_s(\frac{490}{12^2}) = A_s(3.41b/in.2/ft) \text{ or } A_s = 0.294w_c$$

Equation D.38

using a unit steel weight of $\gamma = 490\text{pcf}$. The approximation can be simplified as:

$$w_c = P(7 - 0.03K/r)$$

Equation D.39

where $w_c =$ column weight (lb/ft)

$P =$ axial load (k)

D.5.2 Concrete

Generally, concrete columns are rectangular or round. Columns are distinguished mainly as short columns and slender/long columns. As a preliminary design, slenderness can be ignored in ordinary braced buildings. For unbraced buildings, the initial column sizes are predicted using lower steel content (e.g. 1% steel reinforcement) so that more reinforcement can be added later in the final design stages where the slenderness is taken into account.

In general, for columns of rectangular cross sections ($t$-inches and $l_u$ in feet):
for typical columns above the 1st level
\( t \geq l_u/14 \)

for 1st floor columns with zero end restraint
\( t \geq l_u/10 \)

The first approximation for short, tied rectangular column is:
\[
P_u \leq \phi P_n
\]
\[
\leq \phi \left[ 0.80 \left( 0.85 f_c' (A_g - A_{st}) + f_y A_s \right) \right]
\]
\[
\leq \phi \left[ 0.80 A_g \left( 0.85 f_c' (1 - \rho_g) + \rho_g f_y \right) \right]
\]
\[
\leq \phi \left[ 0.80 A_g \left( 0.85 f_c' + \rho_g (f_y - 0.85 f_c') \right) \right]
\]

where \( P_u \) = factored axial load (k)
\( A_g \) = area of column cross section (in.\(^2\)) = cross sectional area of concrete \( A_c \) plus area of longitudinal reinforcement \( A_{st} \) (i.e. \( A_g = A_c + A_{st} \))
\( f_c' \) = compressive strength of concrete (ksi)
\( \rho_g = A_{st}/A_g \) = column reinforcement ratio
\( f_y \) = yield strength of longitudinal reinforcement (ksi)

For sizing of the short concrete column, the simple approximation is then:
\[
A_g = P_u / (0.5 f_c' + 0.3 \rho_g), \text{ for } e = M_u/P_u \leq 0.1 t
\]

Equation D.41

where \( f_y - 0.85 f_c' = 60 - 0.85(4) = 56.6 \text{ksi} \) (for typical Grade 60 steel grade and concrete strength of 4000psi)

Another approximation method is based on working stress approach. By using a column reinforcement ratio of 1% and an allowable compressive stress of 0.22\( f_c' \),
\[
A_g = P/0.25 f_c' = 4P/f_c'
\]

Equation D.42

Using the typical concrete strength of 4000psi, preliminary column sizes for typical loading of 100psf dead load and a reduced live load of 50psf is:
\[
A_g = nA/10
\]

Equation D.43

where \( A_g \) = column area (in.\(^2\))
\( nA \) = total floor area that column supports (ft\(^2\)) with \( n \) being the stories

For smaller building loads, the formula is revised to \( A_g = nA/12 \) (Equation D.44)

Or with the same 1% reinforcement,
when the concrete strength has increased to 6000psi, $A_g = nA/15$ and  
Concrete strength as 8000psi, $A_g = nA/20$  

**D.5.3 Composite columns**

Composite columns can be constructed by filling a hollow steel section with concrete or by encasing rolled or built up steel shapes with concrete. The latter columns are especially useful because concrete both strengthens and fireproofs the member. The design of composite column is performed in compliance with Sec.I2 of AISC. There are several limitations and restrictions stated in that section.

Following are the criteria that need to be checked for the design of composite columns when the reinforcing steel is neglected. This is a conservative assumption. The basic concept in the design of such columns is to define modified yield strengths $F_{my}$ and modified modulus of elasticity $E_m$, which are then used in the column formulas in AISC Sec.E2.

For concrete filled pipes or rectangular tubes:

$$F_{my} = F_{ys} + 0.85 f'_c A_c / A_t$$  
$$E_m = E_s + 0.4 E_s A_c / A_t$$  
$$r = r_s$$  

For concrete encased shapes:

$$F_{my} = F_{ys} + 0.6 f'_c A_c / A_s$$  
$$E_m = E_s + 0.2 E_s A_c / A_s$$  
$$r = \max (r_s, 0.3 h_2)$$  

where  
$F_{ys}$ = yield stress of steel, ksi; if $F_{ys} > 55$ksi, use $F_{ys} = 55$ksi  
$f'_c$ = concrete crushing strength, ksi  
$A_c$ = area of concrete  
$A_s$ = area of steel  
$E_s$ = modulus of elasticity of steel, $E_s = 29,000$ksi  
$E_c$ = modulus of elasticity of concrete
\[ r = \text{radius of gyration} \]
\[ r_s = \text{radius of gyration of steel section} \]
\[ h_2 = \text{dimension of the rectangular concrete encasement perpendicular to the direction of buckling} \]
Appendix E

The design checks for AISC-LRFD, BS5950 and EC3 are according to the different manuals listed below. The detailed design checks are selected based on the design requirement for W-shape sections.


Check/Design for AISC-LRFD93

Design loading combinations

The common design loading combinations are as follows:

1.4DL  
1.2DL + 1.6LL  

0.9DL ± 1.3WL  
1.2DL ± 1.3WL  
1.2DL + 0.5LL ± 1.3WL  
0.9DL ± 1.0EL  
1.2DL ± 1.0EL  
1.2DL + 0.5LL ± 1.0EL  

Classification of Sections

The W shaped or I-shaped beams are being explored. Limiting Width-thickness ratio for Flexure classification of sections according to AISC-LRFD.

<table>
<thead>
<tr>
<th>Description of Section</th>
<th>Check (λ)</th>
<th>Compact (λ&lt;sub&gt;p&lt;/sub&gt;)</th>
<th>Noncompact (λ&lt;sub&gt;c&lt;/sub&gt;)</th>
</tr>
</thead>
</table>
| I-Shape  
For P<sub>u</sub>/φP<sub>y</sub>≤0.125,  
\[ \leq \frac{640}{\sqrt{F_y}} \left( 1 - \frac{2.75P_u}{\varphi_P P_y} \right) \]  
For P<sub>u</sub>/φP<sub>y</sub>≥0.125,  
\[ \leq \frac{191}{\sqrt{F_y}} \left( 2.33 - \frac{P_u}{\varphi_P P_y} \right) \geq \frac{253}{\sqrt{F_y}} \]  
| h<sub>c</sub>/t<sub>w</sub> | \[ \leq \frac{970}{\sqrt{F_y}} \left( 1 - 0.74 \frac{P_u}{\varphi_P P_y} \right) \] |
For limiting width-thickness ratios for classification of sections according to AISC-LRFD

<table>
<thead>
<tr>
<th>Description of Section</th>
<th>Width-Thickness Ratio (λ)</th>
<th>Compact (Seismic Zone) (λ_p)</th>
<th>Noncompact (Uniform Compression) (M_{22}=M_{33}=0) (λ_t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-Shape</td>
<td>b_f/2t_f (rolled)</td>
<td>≤ 52/\sqrt{F_y}</td>
<td>≤ 95/\sqrt{F_y}</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>b_f/2t_f (rolled)</td>
<td>≤ 52/\sqrt{F_y}</td>
<td>≤ 95/\sqrt{F_y}</td>
</tr>
<tr>
<td></td>
<td>h_c/t_w</td>
<td>For P_u/\varphi P_y ≤ 0.125,</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>\leq \frac{520}{\sqrt{F_y}} \left(1 - 1.54 \frac{P_u}{\varphi \beta P_y}\right)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>For P_u/\varphi P_y &gt; 0.125,</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>\leq \frac{191}{\sqrt{F_y}} \left(2.33 - \frac{P_u}{\varphi \beta P_y}\right) \geq \frac{253}{\sqrt{F_y}}</td>
<td>≤ \frac{253}{\sqrt{F_y}}</td>
</tr>
</tbody>
</table>

**Calculation of Forces and Moments (factored)**

The forces induced on the elements are factored accordingly:

For loading combinations that cause compression in the member, the factored moment \( M_u \) (\( M_{u33} \) and \( M_{u22} \) in the corresponding directions) is magnified to consider second order effects. The magnified moment in a particular direction is given by:

\[ M_u = B_1 M_{nt} + B_2 M_{lt} \]  \hspace{1cm} (LRFD C1-1)

where

\[ B_1 = \text{Moment magnification factor for non-sidesway moments}, \]
\[ B_2 = \text{Moment magnification for sidesway moments}, \]
\[ M_{nt} = \text{Factored moments not causing sidesway}, \]
\[ M_{lt} = \text{Factored moments causing sidesway}. \]

The moment magnification factors are associated with corresponding directions. The moment magnification factor \( B_1 \) for moments not causing sidesway is given by:

\[ B_1 = \frac{C_m}{(1 - P_u/P_r)} \geq 1.0 \]  \hspace{1cm} (LRFD C1-2)
where $P_e$ is the Euler buckling load  

$$P_e = \frac{A_g F_y}{\lambda^2}, \lambda = \frac{Kl}{r \pi \sqrt{E}}$$

$$C_m = 0.6 - 0.4 M_a/M_b$$  \hspace{1cm} (LRFD C1-3)

where $M_a/M_b$ is the ratio of the smaller to the larger moment at the ends of the member, $M_a/M_b$ being positive for double curvature bending and negative for single curvature bending. For compression members with transverse load on the members, $C_m$ is assumed as 1.0. When $M_b$ is zero, $C_m$ is taken as 1.0.

### Calculation of Nominal Strengths

The nominal strengths of the sections are reduced by a reduction factor, $\varphi$, according to the following (LRFD A5.3):

- $\varphi_t =$ Resistance factor for tension, 0.9 \hspace{1cm} (LRFD D3)
- $\varphi_c =$ Resistance factor for compression, 0.85 \hspace{1cm} (LRFD E2)
- $\varphi_b =$ Resistance factor for bending, 0.9 \hspace{1cm} (LRFD F1)
- $\varphi_s =$ Resistance factor for shear, 0.9 \hspace{1cm} (LRFD F2)

### Nominal Strength in Tension

The nominal axial tensile strength value $P_n$ is based on the gross cross sectional area and the yield stress.

$$P_n = A_g F_y$$  \hspace{1cm} (LRFD D1-1)

### Nominal Strength of Compression

The nominal axial compressive strength, $P_n$, depends on its critical value, $X_c$. $Kl/r$ is the larger of $K_{33} l_{33}/r_{33}$ and $K_{22} l_{22}/r_{22}$, and

$$\lambda = \frac{Kl}{r \pi \sqrt{E}}$$  \hspace{1cm} (LRFD E2-4)

$P_n$ for compact and non-compact section is evaluated for flexural buckling as follows:

$$P_n = A_g F_{cr}$$  \hspace{1cm} (LRFD E2-1)

where

$$F_{cr} = \begin{cases} 0.658 \lambda^{1.2} F_y & \text{for } \lambda c \leq 1.5 \\ \frac{0.877}{\lambda c^{1.2}} F_y & \text{for } \lambda c > 1.5 \end{cases}$$

$$P_n = A_g F_{cr}$$  \hspace{1cm} (LRFD E2-3)
### Nominal Strength in shear

The nominal shear strength, $V_{n2}$, for major direction shears in I-shapes, boxes and channels is evaluated as follows:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Nominal Shear strength $V_{n2}$</th>
<th>Code reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\frac{h}{t_w} \leq \frac{418}{\sqrt{F_y}}$</td>
<td>$0.6F_yA_w$</td>
<td>LRFD F2-1</td>
</tr>
<tr>
<td>$\frac{418}{\sqrt{F_y}} &lt; \frac{h}{t_w} \leq \frac{523}{\sqrt{F_y}}$</td>
<td>$0.6F_yA_w \frac{418}{\sqrt{F_y}} \sqrt{\frac{h}{t_w}}$</td>
<td>LRFD F2-2</td>
</tr>
<tr>
<td>$\frac{523}{\sqrt{F_y}} &lt; \frac{h}{t_w} \leq 260$</td>
<td>$132000 \frac{A_w}{\sqrt{\frac{h}{t_w}}}$</td>
<td>LRFD F2-3 and A-F2-3</td>
</tr>
</tbody>
</table>

### Nominal Strength in Bending

The nominal bending strength depends on the following criteria: the geometric shape of the cross section, the axis of bending, the compactness of the section, and a slenderness parameter for lateral-torsional buckling. The nominal bending strength is the minimum value obtained from yielding, lateral-torsional buckling, flange local buckling, and web buckling, as follows:

**Yielding**

For laterally braced compact (and seismic) members with $L_b \leq L_p$,

\[ M_p = Z F_y \leq 1.5 S F_y \] (LRFD F1-1)

$L_b$ = Laterally unbraced length, $l_{22}$ and

$L_p$ = Limiting laterally unbraced length for full plastic capacity

\[ L_p = \frac{300 r_{22}}{F_y} \] for I-shapes and channels (LRFD F1-4)

**Lateral-torsional buckling**

For I-shapes, channels, boxes and rectangular bars bent about the major axis, if $L_b \leq L_t$

\[ M_{n33} = C_b \left[ M_{p33} - \left( M_{r33} - M_{33} \right) \frac{L_b - L_p}{L_r - L_p} \right] \leq M_{p33} \] (LRFD F1-2)
and if $L_b > L_r$,

$$M_{n33} = M_{r33} \leq M_{p33}$$  \hspace{1cm} (LRFD F1-12)$$

where

- $M_{n33}$ = Nominal major bending strength
- $M_{p33}$ = Major plastic moment, $Z_{33}F_y \leq 1.5S_{33}F_y$
- $M_{r33}$ = Major limiting buckling moment, $(F_y-F_r)S_{33}$ for I shapes and channels
- $M_{cr33}$ = Critical elastic moment,
  $$= \frac{C_b}{L_b} \left[ EI_{22}GJ + (\frac{\pi E}{L_b})^2 I_{22}C_w \right]$$
  for I-shapes and channels  \hspace{1cm} (LRFD F1-13)
- $C_w$ = warping constant, in$^6$ for I-shapes and channels
- $L_b$ = Laterally unbraced length, $l_{22}$
- $L_p$ = Limiting laterally unbraced length for full plastic capacity
  $$= \frac{300r_{22}}{\sqrt{F_y}}$$
  for I-shapes and channels  \hspace{1cm} (LRFD F1-4)
- $L_r$ = Limiting laterally unbraced length for inelastic lateral-torsional buckling,
  $$= \frac{r_{22}X_1}{F_y-F_r} \left\{ 1 + [1 + X_2(F_y-F_r)^{1/2}]^{1/2} \right\}^{1/2}$$
  \hspace{1cm} (LRFD F1-6)
- $X_1 = \frac{\pi}{S_{33}} \sqrt{\frac{EGJA}{2}}$  \hspace{1cm} (LRFD F1-8)
- $X_2 = 4 \frac{C_w}{I_{22}} \left( \frac{S_{33}}{GJ} \right)^2$  \hspace{1cm} (LRFD F1-9)
- $C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$  \hspace{1cm} (LRFD F1-3)

$M_{max}$, $M_A$, $M_B$, and $M_C$ are absolute values of maximum moment, ¼ point, center of span and ¾ point major moments respectively, in the member. $C_b$ should be taken as 1.0 for cantilevers.

Local Buckling

For non-compact I-shapes, channels and boxes, the nominal bending strengths are given by the lowest value calculated in the formulas below for the various local buckling modes possible for these sections. The nominal flexural strength $M_n$ for the limit state of flange and web local buckling is:

$$M_{n33} = M_{p33} - (M_{p33} - M_{r33}) \left( \frac{\lambda - \lambda_0}{\lambda - \lambda_0} \right)$$
for major direction bending  \hspace{1cm} (LRFD A-F1-3)
\[ M_{n22} = M_{p22} - (M_{p22} - M_{r22}) \left( \frac{\lambda - \lambda_p}{\lambda - \lambda_p} \right) \text{ for minor direction bending} \]  
\[ \text{ (LRFD A-F1-3)} \]

where

- \( M_{n33} \) = Nominal major bending strength
- \( M_{n22} \) = Nominal minor bending strength
- \( M_{p33} \) = Major plastic moment, \( Z_{33}F_y \leq 1.5SZ_{33}F_y \)
- \( M_{p22} \) = Minor plastic moment, \( Z_{22}F_y \leq 1.5SZ_{22}F_y \)
- \( M_{r33} \) = Major limiting buckling moment, \( (F_y-F_r)S_{33} \) for I shapes and channels
- \( M_{r22} \) = Minor limiting buckling moment, \( F_yS_{22} \) for I shapes and channel
- \( \lambda \) = Controlling slenderness parameter
- \( \lambda_p \) = Largest value of \( \lambda \) for which \( M_n = M_p \) and
- \( \lambda_r \) = Largest value of \( \lambda \) for which buckling is inelastic.

**Calculation of Capacity Ratios**

**Axial and Bending Stresses**

The interaction ratio is determined based on the ratio \( P_u/\varphi P_n \). If \( P_u \) is tensile, \( P_n \) is the nominal axial tensile strength and \( \varphi = \varphi_t = 0.9 \); and the procedure is also applied for compressive, and bending stresses with the corresponding \( \varphi \) values.

For \( P_u/\varphi P_n \geq 0.2 \), the capacity ratio is given as

\[
\frac{P_u}{\varphi P_n} + 8 \left( \frac{M_{n33}}{\varphi M_{n33}} + \frac{M_{n22}}{\varphi M_{n22}} \right) = \text{(LRFD H1-1a)}
\]

For \( P_u/\varphi P_n < 0.2 \), the capacity ratio is given as

\[
\frac{P_u}{2\varphi P_n} + \left( \frac{M_{n33}}{\varphi M_{n33}} + \frac{M_{n22}}{\varphi M_{n22}} \right) = \text{(LRFD H1-1b)}
\]

Similarly, the shear stresses in the major and minor axis are shown as

\[
\left( \frac{V_{u3}}{\varphi V_{n3}} \right) \text{ And } \left( \frac{V_{u2}}{\varphi V_{n2}} \right) \text{ respectively with } \varphi = 0.9.
\]
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Design Loading Combinations

If a structure is subjected to dead load and live load only, the design will need only one loading combination, namely 1.35DL + 1.5LL. If, however, there are wind (WL) and earthquake forces (EL) imposed on the structure, the following load combinations may have to be considered (EC3 2.3.3):

- 1.35DL
- 1.35DL + 1.5LL
- 1.35DL ± 1.50WL
- 1.00DL ± 1.50WL
- 1.35DL ± 1.35LL ± 1.35WL
- 1.00DL ± 1.00EL
- 1.00DL + 1.5*0.3LL ± 1.0EL

Classification of Sections

The Eurocode defines the sections classification according to Class 1 (Plastic), Class 2 (Compact), Class 3 (Semi-compact), or Class 4 (Slender). One of the major factors in determining the limiting width-thickness ratio is \( \varepsilon \). This parameter is used to reflect the influence of yield stress on the section classification.

\[
\varepsilon = \sqrt{\frac{235}{f_y}}
\]

(Class 3.5.2)

Classification of Sections According to Eurocode 3 Limiting Width-Thickness ratios for compression elements

<table>
<thead>
<tr>
<th>Section</th>
<th>Element</th>
<th>Ratio Checked</th>
<th>Class 1</th>
<th>Class 2</th>
<th>Class 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-shape</td>
<td>web</td>
<td>d/t_w</td>
<td>33( \varepsilon )</td>
<td>38( \varepsilon )</td>
<td>42( \varepsilon )</td>
</tr>
<tr>
<td></td>
<td>flange</td>
<td>c/t_f (rolled)</td>
<td>10( \varepsilon )</td>
<td>11( \varepsilon )</td>
<td>15( \varepsilon )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>c/t_f (welded)</td>
<td>9( \varepsilon )</td>
<td>10( \varepsilon )</td>
<td>14( \varepsilon )</td>
</tr>
</tbody>
</table>

Classification of Sections According to Eurocode 3 Limiting Width-Thickness ratios for flexure elements

<table>
<thead>
<tr>
<th>Section</th>
<th>Element</th>
<th>Ratio Checked</th>
<th>Class 1</th>
<th>Class 2</th>
<th>Class 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-shape</td>
<td>Web</td>
<td>d/t_w</td>
<td>72( \varepsilon )</td>
<td>83( \varepsilon )</td>
<td>124( \varepsilon )</td>
</tr>
<tr>
<td></td>
<td>Flange</td>
<td>c/t_f (rolled)</td>
<td>10( \varepsilon )</td>
<td>11( \varepsilon )</td>
<td>15( \varepsilon )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>c/t_f (welded)</td>
<td>9( \varepsilon )</td>
<td>10( \varepsilon )</td>
<td>14( \varepsilon )</td>
</tr>
</tbody>
</table>
For the sections pertaining to compression elements where there is a combination of axial compression and bending, the classification of sections can be found in Table 5.3.1 of EC3 5.3.2.

**Calculation of Forces and Moments (factored)**

The design moments and forces need to be corrected for second order effects. This correction is different for the so-called “sway” and “nonsway” components of the moments. The code requires that the additional sway moments introduced by the horizontal deflection of the top of a story relative to the bottom must be taken into account in the elastic analysis of the frame in one of the following ways (EC3 5.2.6.2):

Directly – by carrying out the global frame analysis using P-Δ analysis. Member design can be carried out using inplane buckling lengths for nonsway mode.

Indirectly – by modifying the results of a linear elastic analysis using an approximate method which makes allowance for the second order effects. There are two alternative ways to do this – “amplified sway moment method” or “sway mode inplane buckling method”.

Thus the moment can be expressed as:

\[ M_{sd} = M_{e, sd} + \psi_s M_{s, sd} \]  

(EC3 5.2.6.2)

where

- \( M_{e, sd} \) = Design moments not causing translation
- \( M_{s, sd} \) = Design moments causing sidesway

**Calculation of Nominal Strengths**

The material partial safety factors are:

\( \gamma_{M0} = 1.1 \) and
\( \gamma_{M1} = 1.1 \)

**Tension capacity**

The design tension resistance for all classes of sections is:

\[ N_{t, Rd} = Af_y / \gamma_{M0} \]  

(EC3 5.4.3)

**Compression resistance**

The design compressive resistance of the cross section is taken as the smaller of the design plastic resistance of the gross cross-section (\( N_{pl,Rd} \)) and the design local buckling resistance of the gross section (\( N_{b,Rd} \)).

\[ N_{c,Rd} = \min (N_{pl,Rd}, N_{b,Rd}) \]  

(EC3 5.4.4)

The plastic resistance of Class 1, 2 and 3 sections is given by:

\[ N_{pl,Rd} = Af_y / \gamma_{M0} \]  

(EC3 5.4.4)

The design buckling resistance of a compression member is taken as:

\[ N_{b,Rd} = \frac{\chi \min \beta A f_y}{\gamma_{M1}} \]  

(EC3 5.5.1)
where $\beta_A=1.0$ for class 1, 2 or 3 cross-sections

$\chi$ is the reduction factor for the relevant buckling mode. This factor is calculated below based on the assumption that all members are of uniform cross section.

$$\chi = \frac{1}{\varphi + (\varphi^2 - \overline{\lambda}^2)^{1/2}} \leq 1 \quad \text{(EC3 5.5.1.2)}$$

where

$$\varphi = 0.5[1 + \alpha(\overline{\lambda} - 0.2) + \overline{\lambda}^2]$$

$$\overline{\lambda} = \left[ \frac{\lambda}{\lambda_1} \right]^{1/5}$$

$$\lambda = \frac{K_{33}I_{33}}{I_{33}} \text{ or } \frac{K_{22}I_{22}}{I_{22}}$$

The two values of $\lambda$ give $\chi_3$ and $\chi_2$. $\chi_{\text{min}}$ is the lesser of the two.

$K=1/L \leq 1.0$

$$\lambda_1 = \pi \left[ \frac{E}{f_y} \right]^{1/2}$$

The $\alpha$ is an imperfection factor for different sections and different axes of buckling.

<table>
<thead>
<tr>
<th>Section (rolled)</th>
<th>Limits</th>
<th>$\alpha$ (major axis)</th>
<th>$\alpha$ (minor axis)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-shape</td>
<td>$t_f \leq 40$ mm</td>
<td>0.21</td>
<td>0.34</td>
</tr>
<tr>
<td>h/b &gt; 1.2</td>
<td>$t_f &gt; 40$ mm</td>
<td>0.34</td>
<td>0.49</td>
</tr>
<tr>
<td>I-shape</td>
<td>$t_f \leq 100$ mm</td>
<td>0.34</td>
<td>0.49</td>
</tr>
<tr>
<td>h/b ≤ 1.2</td>
<td>$t_f &gt; 100$ mm</td>
<td>0.76</td>
<td>0.76</td>
</tr>
<tr>
<td>I-shape (welded)</td>
<td>$t_f \leq 40$ mm</td>
<td>0.34</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>$t_f &gt; 40$ mm</td>
<td>0.49</td>
<td>0.76</td>
</tr>
</tbody>
</table>

### Shear capacity

The design shear resistance of a section is the minimum of the plastic shear capacity and the buckling shear capacity.

The plastic shear capacity for all sections is:

$$V_{pl} = V_{pl,Rd} = \frac{A_f}{\sqrt{3}} / \gamma_M$$

where $A_f$ is the effective shear area for the section and the appropriate axis of bending.

The buckling shear capacities for I-, Box-, and Channel sections if the width thickness ratio is large ($d/t_w > 69\varepsilon$) are as follows:
where $\tau_{ba}$ is the simple post-critical shear strength which is determined as follows:

$$\tau_{ba} = \frac{f_{yw}}{\sqrt{3}}$$

for $\bar{\lambda}_w \leq 0.8$  

(EC3 6.6.3)

$$\tau_{ba} = [1 - 0.625(\bar{\lambda}_w - 0.8)]\frac{f_{yw}}{\sqrt{3}}$$

for $0.8 < \bar{\lambda}_w < 1.2$  

(EC3 6.6.3)

$$\tau_{ba} = [0.9 / \bar{\lambda}_w]\frac{f_{yw}}{\sqrt{3}}$$

for $\bar{\lambda}_w \geq 1.2$  

(EC3 6.6.3)

in which $\bar{\lambda}_w$ is the web slenderness ratio,

$$\bar{\lambda}_w = \frac{d/t_w}{37.4 \epsilon \sqrt{k_r}}$$

(EC3 6.6.3)

and $k_r$ is the buckling factor for shear. For webs with transverse stiffeners at the supports but no intermediate transverse stiffeners,

$$k_r = 5.34$$

(EC3 6.6.3)

**Moment resistance**

The moment resistance of a cross section depends also on the shear forces and axial forces at the cross section.

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Moment resistance</th>
<th>Code reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_{sd} \leq 0.5V_{pl,Rd}$</td>
<td>$M_{c,Rd}=M_{pl,Rd}$</td>
<td>EC3 5.4.5.2</td>
</tr>
<tr>
<td>$V_{sd} \leq 0.5V_{pl,Rd}$</td>
<td>$M_{c,Rd}=M_{pl,Rd}$</td>
<td>EC3 5.4.5.2</td>
</tr>
<tr>
<td>$V_{sd} \leq 0.5V_{pl,Rd}$</td>
<td>$M_{c,Rd}=M_{el,Rd}$</td>
<td>EC3 5.4.5.2</td>
</tr>
</tbody>
</table>

$M_{c,Rd} = W_{pl}f_y/\gamma_M$  

$M_{c,Rd} = W_{pl}f_y/\gamma_M$  

$M_{c,Rd} = W_{el}f_y/\gamma_M$  

where $\rho = \left[\frac{2V_{sd}}{V_{pl,Rd}} - 1\right]^2$  

EC3 5.4.5.2
Lateral torsional buckling

The lateral torsional buckling resistance of a beam is evaluated as:

\[ M_{b, rd} = \chi_{LT} \beta_{v} W_{pl, 33} f_{y} / \gamma_{M1} \]  

(EC3 5.5.2)

where

\[ \beta_{v} = \beta_{v1} \]  for Class 1 and Class 2 sections

\[ \beta_{v} = \frac{W_{v, 33}}{W_{pl, 33}} \]  for Class 3 sections

\[ \chi_{LT} = \frac{1}{\varphi_{LT} + (\varphi_{LT}^{2} - \lambda_{LT}^{2})^{1/2}} \leq 1 \]

in which \( \varphi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^{2}] \) where

\[ \alpha_{LT} = 0.21 \]  for rolled sections

\[ \alpha_{LT} = 0.49 \]  for welded sections

\[ \lambda_{LT} = \left[ \frac{\beta_{v} W_{pl, 33} f_{y}}{M_{cr}} \right]^{0.5} \]  where

\[ M_{cr} = C_{1} \frac{\pi^{2} EI_{32}}{L^{2}} \left[ \frac{I_{w}}{L_{32}} + \frac{L^{2} G I_{e}}{\pi^{2} EI_{32}} \right]^{0.5} \]  

(EC2 F1.1)

\( I_{w} = \) The torsion constant
\( I_{w} = \) The warping constant
\( L = \) Laterally unbraced length for buckling about the minor axis
\( C_{1} = 1.88 - 1.40\psi + 0.52\psi^{2} \leq 2.7 \)
\( \psi = \) The ratio of smaller to larger end moment of unbraced segment, \( \frac{M_{a}}{M_{b}} \Psi \)
\( \psi \) varies between \(-1\) and \(1\) \((-1 \leq \psi \leq 1)\).
Calculation of Capacity ratios

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Capacity ratio</th>
<th>Code reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending, axial compression, and low shear</td>
<td>$N_{c,ld} + \frac{M_{33,ld}}{N_{pl,ld}} + \frac{M_{22,ld}}{M_{pl,ld}}$</td>
<td>EC3 5.4.9</td>
</tr>
<tr>
<td>$V_{sd} \leq 0.5V_{pl,Rd}$ and</td>
<td>$N_{c,ld} + \frac{M_{33,ld}}{N_{pl,ld}} + \frac{M_{22,ld}}{M_{pl,ld}}$</td>
<td>EC3 5.4.8.1</td>
</tr>
<tr>
<td>$V_{sd} \leq 0.5V_{ba,Rd}$</td>
<td>$N_{c,ld} + \frac{M_{33,ld}}{N_{pl,ld}} + \frac{M_{22,ld}}{M_{pl,ld}}$</td>
<td></td>
</tr>
<tr>
<td>Bending, axial compression, and high shear</td>
<td>$\frac{N_{c,ld}}{N_{pl,ld}} + \frac{M_{33,ld}}{M_{pl,ld}} + \frac{M_{22,ld}}{M_{pl,ld}}$</td>
<td>EC3 5.4.9</td>
</tr>
<tr>
<td>$V_{sd} &gt; 0.5V_{pl,Rd}$ or</td>
<td>$N_{c,ld} + \frac{M_{33,ld}}{N_{pl,ld}} + \frac{M_{22,ld}}{M_{pl,ld}}$</td>
<td>EC3 5.4.8.1</td>
</tr>
<tr>
<td>$V_{sd} &gt; 0.5V_{ba,Rd}$</td>
<td>$N_{c,ld} + \frac{M_{33,ld}}{N_{pl,ld}} + \frac{M_{22,ld}}{M_{pl,ld}}$</td>
<td></td>
</tr>
<tr>
<td>Bending, compression and flexural buckling</td>
<td>$\frac{N_{c,ld}}{N_{b,ld}} + \frac{k_{33}M_{33,ld}}{\eta M_{c,ld}} + \frac{k_{22}M_{22,ld}}{\eta M_{c,ld}}$</td>
<td>EC3 5.5.4</td>
</tr>
<tr>
<td>Bending, compression, and lateral torsional</td>
<td>$\frac{N_{c,ld}}{N_{b,ld}} + \frac{k_{LT}M_{33,ld}}{\eta M_{b,ld}} + \frac{k_{22}M_{22,ld}}{\eta M_{c,ld}}$</td>
<td>EC3 5.5.4</td>
</tr>
<tr>
<td>buckling</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$k_{LT} = 1 - \frac{\mu_{LT}N_{c,ld}}{\chi_{22}A_{fy}} \leq 1$ and $\mu_{LT} = 0.15 \lambda_{22} \beta_{M,LT} - 0.15 \leq 0.9$</td>
<td></td>
</tr>
<tr>
<td>Criteria</td>
<td>Capacity ratio</td>
<td>Code reference</td>
</tr>
<tr>
<td>----------</td>
<td>----------------</td>
<td>----------------</td>
</tr>
</tbody>
</table>
| **Bending, axial tension, and low shear** | \[ \frac{N_t, Sd}{N_t, Rd} + \frac{M_{33, Sd}}{M_{pl, 33, Rd}} + \frac{M_{22, Sd}}{M_{pl, 22, Rd}} \] | EC3 5.4.9  
EC3 5.4.8.1 |
| \( V_{sd} \leq 0.5V_{pl, Rd} \) and \( V_{sd} \leq 0.5V_{ba, Rd} \) | \[ \frac{N_t, Sd}{N_t, Rd} + \frac{M_{33, Sd}}{M_{pl, 33, Rd}} + \frac{M_{22, Sd}}{M_{pl, 22, Rd}} \] | \( \frac{N_t, Sd}{A_{fsd}} + \frac{M_{33, Sd}}{W_{el, 33, Rd}} + \frac{M_{22, Sd}}{W_{el, 22, fsd}} \) |
| **Bending, axial tension, and high shear** | \[ \frac{N_t, Sd}{N_t, Rd} + \frac{M_{33, Sd}}{M_{V, 33, Rd}} + \frac{M_{22, Sd}}{M_{V, 22, Rd}} \] | EC3 5.4.9  
EC3 5.4.8.1 |
| \( V_{sd} > 0.5V_{pl, Rd} \) or \( V_{sd} > 0.5V_{ba, Rd} \) | \( \frac{N_t, Sd}{N_t, Rd} + \frac{M_{33, Sd}}{M_{V, 33, Rd}} + \frac{M_{22, Sd}}{M_{V, 22, Rd}} \) | \( \frac{N_t, Sd}{A_{fsd}} + \frac{M_{33, Sd}}{W_{el, 33, Rd}} + \frac{M_{22, Sd}}{W_{el, 22, fsd}} \) |
| **Bending, axial tension, and lateral torsional buckling** | \( \frac{N_t, Sd}{N_t, Rd} + \frac{k_{LT} M_{33, Sd}}{M_{b, Rd}} + \frac{k_{22} M_{22, Sd}}{\eta M_{c, 22, Rd}} - \Psi_{vec} k_{LT} \frac{N_t, Sd W_{com, 33}}{A_{M_{b, Rd}}} \) | **  
EC3 5.5.4 |
| **Shear** | \[ \frac{V_{2, Sd}}{V_{2, Rd}} \quad \text{and} \quad \frac{V_{3, Sd}}{V_{3, Rd}} \] | |

*\( * \) Bending, compression, and flexural buckling
Bending, compression, and flexural buckling

\( \text{Nb.min.Rd} = \min\{\text{Nb.33.Rd}, \text{Nb.22.Rd}\} \)

\( \eta = \gamma M0/\gamma M1 \)

\( k_{33} = 1 - \frac{\mu_{33} N_{c,3d}}{X_{33} A_{f}} \leq 1.5 \)

\( k_{22} = 1 - \frac{\mu_{22} N_{c,2d}}{X_{22} A_{f}} \leq 1.5 \)

\( \mu_{33} = \lambda_{33}(2\beta_{M,33} - 4) + \left[ \frac{W_{pl,33} - W_{el,33}}{W_{el,33}} \right] \leq 0.9 \)  
(Class 1 and Class 2)

\( \mu_{22} = \lambda_{22}(2\beta_{M,22} - 4) + \left[ \frac{W_{pl,22} - W_{el,22}}{W_{el,22}} \right] \leq 0.9 \)  
(Class 1 and Class 2)

\( \mu_{33} = \lambda_{33}(2\beta_{M,33} - 4) \leq 0.9 \)  
(Class 3)

\( \mu_{22} = \lambda_{22}(2\beta_{M,22} - 4) \leq 0.9 \)  
(Class 3)

\( \beta_{M,33} \) = Equivalent uniform moment factor for flexural buckling about the 3-3 major axis between points braced in 2-2 direction

\( \beta_{M,22} \) = Equivalent uniform moment factor for flexural buckling about the 2-2 minor axis between points braced in 3-3 direction

The equivalent uniform moment factors, \( \beta_{M,33} \) and \( \beta_{M,22} \) are determined from

\( \beta_{M} = (1.8 - 0.7\Psi) + \frac{M_{O}}{\Delta M}(0.7\Psi - 0.5) \)

where \( M_{O} = \) Absolute maximum moment due to lateral load only assuming simple support at the ends

\( \Psi = \) Absolute value of the ratio of smaller to larger end moment. \( \Psi \) varies between -1 and 1 (-1 ≤ \( \Psi \) ≤ 1). A negative value implies double curvature.

\( \Delta M = \) Absolute maximum value of moment for moment diagram without change of sign

\( \Delta M = \) Sum of absolute maximum and absolute minimum value of moments for moment diagram with change of sign.
** Bending, axial tension, and lateral-torsional buckling

\[
M_{\text{eff}, 33.5d} = M_{33.5d} - \Psi_{\text{vec}} \frac{N_{c,d} W_{\text{com}, 33}}{A}
\]

\(\Psi_{\text{vec}} = 0.8\) (according to the EC3 box value), and
\(W_{\text{com}, 33}\) is the elastic section modulus for the extreme compression fiber.
Check/Design for BS59590

Design load combinations

The following load combination may have to be considered (BS 2.4)
1.4DL
1.4DL + 1.6LL (BS 2.4.1.1)

1.0DL ± 1.4WL
1.4DL ± 1.4WL
1.2DL ± 1.2LL ± 1.2WL (BS 2.4.1.1)

1.0DL ± 1.4EL
1.4DL ± 1.4EL
1.2DL ± 1.2LL ± 1.2EL

The code also requires that all buildings should be capable of resisting a notional design horizontal load applied at each floor or roof level. The notional load should be equal to the maximum of 0.01 times the factored dead load and 0.005 times the factored dead plus live loads (BS 2.4.2.3). The notional forces should be assumed to act in any one direction at a time should be taken as acting simultaneously with the factored dead plus vertical imposed live loads. They should not be combined with any other horizontal load cases (BS 5.1.2.3).

Classification of sections

The I-shapes are classified in the table with the variables as:

R is the ratio of mean longitudinal stress in the web to \( p_y \) in a semi-compact section. This implies that for a semi-compact section in pure bending R is zero. In calculating R, compression is taken as positive and tension is taken as negative.

\( \alpha \) is given as \( 2\gamma_c/d \), where \( \gamma_c \) is the distance from the plastic neutral axis to the edge of the web connected to the compression flange. For \( \alpha > 2 \), the section is treated as having compression throughout.

\( \varepsilon \) is defined as \( (275/p_y)^{1/3} \).
Calculation of factored forces and moments

The moment magnification for non-sidesway moments is included in the overall buckling interaction equations.

\[ M = M_s + \left( \frac{1}{1 - 200\phi_{s,\text{max}}} \right) M_t \]

where

- \( \phi_{s,\text{max}} \) = Maximum story drift divided by the story height
- \( M_s \) = Factored moments not causing translation
- \( M_t \) = Factored moments causing sidesway

Calculation of Section Capacities

Tension Capacity

The tension capacity of a member is given by:

\[ P_t = A_g \rho_y \]  

(BS 4.6.1)
The main member in tension, the slenderness, $\lambda$, should not be greater than 250 (BS 4.7.3.2).

**Compression Resistance**

The compression resistance for plastic, compact, or semi-compact sections is evaluated as follows:

$$P_c = A_g \rho_c$$

Where $\rho_c$ is the compressive strength given by:

$$\rho_c = \frac{\rho E}{\varphi + (\varphi^2 - \rho E)\frac{1}{2}}$$

where

$$\varphi = \frac{\rho E + (\eta + 1)\rho E}{2}$$

$\rho E$ = Euler strength, $\pi^2 E / \lambda^2$

$\eta$ = Perry factor, 0.001$a(\lambda - \lambda_0) \geq 0$

$a$ = Robertson constant, refer to table below

$\lambda_0$ = Limiting slenderness, 0.2

$\lambda$ is the slenderness ratio in either the major or the minor direction (BS 4.7.3.1).

$$\lambda = \max(\lambda_33, \lambda_{22}) = \max(\frac{l_{33}}{r_{33}}, \frac{l_{22}}{r_{22}})$$

**Table for Robertson constant**

<table>
<thead>
<tr>
<th>Description of Section</th>
<th>Thickness</th>
<th>Axis of Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Major</td>
</tr>
<tr>
<td>I-shape (rolled)</td>
<td>Any</td>
<td>2.0</td>
</tr>
<tr>
<td>H-shape (rolled)</td>
<td>$\leq 40$</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>$&gt; 40$</td>
<td>5.5</td>
</tr>
<tr>
<td>I-shape (welded)</td>
<td>$\leq 40$</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>$&gt; 40$</td>
<td>3.5</td>
</tr>
</tbody>
</table>

**Shear Capacity**

The shear capacities for both the major and minor direction shears are evaluated as follows:

$$P_{v2} = 0.6 \rho_y A_{v2}$$

$$P_{v3} = 0.6 \rho_y A_{v3}$$

This shear capacity is only valid if $d/t \leq 63 \varepsilon$. If $d/t > 63 \varepsilon$, then shear buckling of the thin members should be checked.
Moment Capacity

For plastic and compact sections where the bending about the 3-3 axis the moment capacities considering the effects of shear forces are computed as:

\[ M_c = \rho_1 S \leq 1.2 \rho_2 Z \quad \text{for } F_v \leq 0.6 P_v \quad (\text{BS 4.2.5}) \]

\[ M_c = \rho_1 (S - S_v) \leq 1.2 \rho_2 Z \quad \text{for } F_v > 0.6 P_v \quad (\text{BS 4.2.5}) \]

where

- \( S \) = Plastic modulus of the gross section about the relevant axis
- \( Z \) = Elastic modulus of the gross section about the relevant axis
- \( S_v \) = Plastic modulus of the gross section about the relevant axis less the plastic modulus of that part of the section remaining after deduction of shear area i.e. plastic modulus of shear area. For example, for rolled I-shapes \( S_v \) is taken to be \( tD^2/4 \) and for welded I-shapes it is taken as \( td^2/4 \).
- \( P_v \) = The shear capacity
- \( \rho_1 = 2.5F_v/P_v - 1.5 \)

For semi-compact section, the reduction of moment capacity due to coexistent shear does not apply,

\[ M_c = \rho_2 Z \quad (\text{BS 4.2.5}) \]

Lateral torsional buckling moment capacity

The lateral torsional buckling moment capacity, \( M_b \) can be found as follows:

\[ M_b = \frac{\rho_5 S_{33} M_E}{\varphi_0 + (\varphi_0^2 - \rho_5 S_{33} M_E)^{1/2}} \quad (\text{BS B.2.1}) \]

where

\[ \varphi_0 = \frac{\rho_5 S_{33} + (\eta_{LT} + 1) M_E}{2} \]

\( M_E \) = The elastic critical moment, \[ S_{33}\pi^2 E \]

\( \lambda_{LT} \) = The Perry coefficient

Rolled sections, \( \eta_{LT} = \alpha_6 (\lambda_{LT} - \lambda_{L0}) \geq 0 \) and

Welded sections, \( \eta_{LT} = 2\alpha_6 \lambda_{L0} \geq 0 \) \quad (BS B.2.2)

with \( \alpha_6 (\lambda_{LT} - \lambda_{L0}) \leq \eta_{LT} \leq 2\alpha_6 (\lambda_{LT} - \lambda_{L0}) \)

\( \alpha_6 = 0.007 \)
For flanged members symmetrical about at least one axis and uniform throughout their length, $\lambda_{LO}$ and $\lambda_{LT}$ are defined as:

$$\lambda_{LO} = 0.4 \sqrt{\frac{\pi^2 E}{\rho_s}}$$  \hspace{1cm} (BS B2.4)

$$\lambda_{LT} = nuv\hat{\lambda}$$  \hspace{1cm} (BS B2.5)

where

$\lambda$ is the slenderness and is equivalent to $le_{22}/r_{22}$

$n$ is the slenderness correction factor. For flanged members in general, not loaded between adjacent lateral restraints, and for cantilevers without intermediate lateral restraints, $n$ is taken as 1.0. For members with equal flanges loaded between adjacent restraints, the value of $n$ is conservatively taken as given by the following formula (BS Table 13).

$$n = \frac{1}{\sqrt{C_b}}$$

$$C_b = \frac{12.5M_{\text{max}}}{2.5M_{\text{max}} + 3M_A + 4M_B + 3M_C}$$

where the various moments have been explained in the earlier section.

$u$ is the buckling parameter. It is conservatively taken as 0.9 for rolled I-shapes and channels.

$v$ is the slenderness factor. It is given by the following formula (BS B.2.5)

$$v = \left[1 + \frac{1}{20} \left(\frac{\lambda T}{D}\right)^2\right]^{-1/2}$$  \hspace{1cm} (BS B.2.5)

### Calculation of Capacity ratios

For members under axial load and moments,

$$\frac{F}{A_s\rho_s} + \frac{M_{33}}{M_{c33}} + \frac{M_{22}}{M_{c22}}$$

with $F=F_t$ (for axial tension) or

$F=F_c$ (axial compression)  \hspace{1cm} (BS 4.8.2)

$F=F_t$ (axial tension) or

$F=F_c$ (axial compression)  \hspace{1cm} (BS 4.8.3.2)
Overall buckling check

In addition to local capacity checks, which are carried out at section level, a compression member with bending moments is also checked for overall buckling in accordance to the following interaction ratio:

\[
\frac{F_c}{A_{c\phi}} + \frac{m_{33}M_{33}}{M_{b33}} + \frac{m_{22}M_{22}}{\rho Z_{22}}
\]  

(BS 4.8.3.3.1)

The equivalent uniform moment factor, \( m \), for members of uniform section and with flanges, not loaded between adjacent lateral restraints is defined as

\[
m = 0.57 + 0.33\beta + 0.10\beta^2 \geq 0.43
\]  

(BS Table 18)

Shear capacity check

For each section, the shear capacity must be less than 1.0.

\[
\frac{F_{v2}}{P_{v2}} \quad \text{and} \quad \frac{F_{v3}}{P_{v3}} < 1.0
\]