Optimal Conceptual Design of High-Rise Office Buildings

by

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Abstract

Design of a high-rise office building, like any engineering design, is a complex multidisciplinary process with the objective to discover, detail and construct a system to fulfill a given set of performance requirements. The success of this process is highly dependent upon the cooperation taking place between the members of the design team. Although present-day engineering computer technology allows for precise analysis and design of the different subsystems of an high-rise building, it does not readily provide insight for choosing among alternatives of these subsystems to arrive at the best overall design.

This research study presents a computer-based computational method for optimal cost-revenue conceptual design of high-rise office buildings. Specifically, a Multi-criteria Genetic Algorithm (MGA) is applied to conduct Pareto optimization that minimizes capital and operating costs and maximizes income revenue for a given building project, subject to design constraints imposed by building codes and fabrication requirements.

The conceptual design process involves the coordinated application of approximate analysis, design and optimization. To commence the design process, a population of different alternative designs are generated. Using approximate analysis and design based on pre-developed data bases, the values of the conflicting cost-revenue objective criteria are established for each design. Then, a MGA is used to explore the design space and find improved designs having enhanced values of the objective criteria. The results, for a given building project, is a set of Pareto-optimal conceptual designs that

define the 'trade-off' relationships between the three competing objective criteria to minimize capital cost, minimize operating cost and maximize income revenue. The corresponding three-dimensional criteria space is populated by feasible conceptual designs that are 'equal-rank optimal' in the sense that each design is not dominated for all three objective criteria by any other feasible design possible for the building. Life-cycle costing is introduced to investigate the profit potential of building designs over time. The conceptual design of four example office buildings are conducted from a variety of viewpoints to illustrate the capability of the computational procedure to create comprehensive computer-generated colour graphic representations of optimal cost-revenue trade-off relationships for office buildings taking into account architectural, structural, mechanical and electrical systems. While this study focuses on office buildings and corresponding cost-revenue criteria, the proposed computer method for conceptual design is directly applicable to any type of artifact and related objective criteria.

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To my parents

Notation

a	direction of the length of the building
a_{max}	maximum length of the building
ABE	Angle of Building with East (degree)
$ACDT_{max}$	max Average Cold Day Temperature (C°)
$ACDT_{min}$	min Average Cold Day Temperature (C°)
$ADBL_a$, $ADBL_b$	Average Distance Between column Lines in a & b directions
$AHDT_{max}$	max Average Hot Day Temperature (C°)
$AHDT_{min}$	min Average Hot Day Temperature (C°)
AOT_{max}, AOT_{min}	Ave. max. & min Outside Temp. (C°)
AR	Aspect Ratio D_a/D_b
A_{req}	the required floor area of the building (m^2)
b	direction of the width of the building
b_{max}	maximum width of the building
BT	Bracing Type
$BWLP$	Basic Wind Load Pressure (kPa)
C_a, C_b	Core dimensions in a & b directions
$CCLF$	Concrete Cost Location Factor (ratio of local concrete cost to US national average concrete cost)
CDF	Core Dimension Factor: the ratio of a core dimension to the dimension of the building in the same direction
$CDRH$	Cold Day Relative Humidity (%)
$CDTR$	Cold Day Temperature Range (C°)
CFA	Column-Free Area factor
CFT	Concrete Floor Type
C_LC	CLadding Color (greatly effects the HVAC system)
C_LCLF	Cladding Cost Location Factor (ratio of local cladding cost to US national average cladding cost)
CPD_{min}	minimum Core to Perimeter Distance for the building
CSP	Clear Sky Percentage (ratio of clear sky hours to total hours in

	a year) (%)
D_a, D_b	Building Dimensions in a & b directions (m)
DCDD	Direction of the Core Dimension to be Designed first (a or b)
DF	Depth of Floor (m)
DIT	Desired Inside Temperature (C °)
ECLF	Electrical Cost Location Factor (ratio of local electrical cost to US national average electrical cost)
E_LCLF	ELevator Cost Location Factor (ratio of local elevator cost to US national average elevator cost)
FCLF	Forming Cost Location Factor (ratio of local forming cost to US national average forming cost)
F_ICLF	FInishing Cost Location Factor (ratio of local finishing cost to US national average finishing cost)
FLC	Fixed Land Cost ($)
h_{cle}	floor-to-ceiling clearance height
H	Height of the building (m)
H_{max}	maximum Height of the building
HDRH	Hot Day Relative Humidity (%)
HF	Height of Floor
IR	Inflation Rate (%)
IRH	Inside Relative Humidity (%)
ISA	Interior Surface Area of exterior walls
LA	Latitude Angle (degree)
LR_{max}, LR_{min}	max & min Lease Rates ($/m^2 Yr.)
MCLF	Mechanical Cost Location Factor (ratio of local mechanical cost to US national average mechanical cost)
MR	Mortgage Rate (%)
NCL_a, NCL_b	Number of Column Lines between the perimeter and core of the building in a & b directions
NE	total Number of Elevators
NF	Number of Floors

NOPF	Number of Occupants Per Floor
NRF, NMF	Number of Rentable and Mechanical Floors
NRSC	Number of Risers in a Stair Case for one floor
NS_a, NS_b	Number of Spans in a & b directions
NSC, WSC	Number and Width of Stair Cases
NSE, NPE,	Number of Service and Passenger Elevators
NTS_a, NTS_b	Number of perimeter Tube column Spans within spans S_a and S_b
OILSC	Overall Inside Length of Stair Case
OIWSC	Overall Inside Width of Stair Case
RCLF	Reinforcement Cost Location Factor (ratio of local reinforcement cost to US national average reinforcement cost)
R_OCLF	ROofing Cost Location Factor (ratio of local roofing cost to US national average roofing cost)
S_a, S_b	Span distance in a & b directions (m)
SCLF	Steel Cost Location Factor (ratio of local steel cost to US national average steel cost)
SFT	Steel Floor Type
SR	Slenderness Ratio for building
ST	Structural Type
TCS_a, TCS_b	Tube Column Spans in a & b directions
$TMAX_m$	MAX Temperature for sample day m of each month (C^o)
T_{mh}	hourly Temperature for sample day m of each month (C^o)
$TMIN_m$	MIN Temperature for sample day m of each month (C^o)
TNO	Total Number of Occupants
TR	Tax Rate (%)
UCEE, UCGE	Unit Cost of Electrical and Gas Energy ($/mwhr)
ULC	Unit Land Cost ($\$/m^2$)
WAT	exterior WAll Type (Cladding)
WCLF	Window Cost Location Factor (ratio of local window cost to US national average window cost)

WIR	WIndow Ratio(the ratio of window area to the maximum possible window area on the perimeter of a building)
WIT	WIndow Type

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Chapter 1

Introduction

1.1 INTRODUCTION

Basic to the engineering process is the objective to develop, design and construct a system to fulfil a given set of performance requirements. Some of the objective criteria in this multi-level process are entirely rational and quantitative, but others must remain non-quantifiable because of either their enormous analytical complexity or because they involve elements of taste or aesthetics.

The fundamental aim in dealing with rational objective criteria is to find the best or optimal solution to the problem at hand. In a building optimization problem the, best solutions are those that satisfy the requirements of function and integrity for the minimum capital and operating costs and the maximum revenue income while remaining within the aesthetic bounds imposed by the architect.

High-rise buildings are an integral part of modern urban environments, and there are two fundamental differences between designing them and engineering projects of a smaller scale: 1) the consequences of design decisions are more costly; and 2) the environmental technology of a tall building is more complex. High-rise buildings represent enormous private and public investment and, most importantly, they are large consumers of resources in the form of labor and construction materials (Forwood 1975). Because of this enormous investment, research effort has long been devoted over the years to developing optimization techniques that reduce the consumption of resources for building projects (Newmark and Rosenblueth 1971, Cohn et al 1972, McDermott et al 1972, Iyengar 1973, Cohn 1995).

Like all designs, the design of a high-rise building involves the development of the physical description of an artifact subject to a set of given constraints and specifications. There are three phases in the design of a high-rise building; 1) conceptual, 2) preliminary, and 3) detailed design. Conceptual design deals with the identification of different concepts and the selection of overall best subsystems and their configurations. The preliminary design stage involves the initial development of one or a few conceptual models. Finally, the detailed design stage defines a complete solution for all subsystems, and results in final drawings for architectural, structural, electrical and mechanical systems.

Increasing numbers of high-rise buildings are produced each year for commercial use. However, most design procedures are indirect, in that a design concept is proposed and then successively analyzed, evaluated, corrected and reanalyzed until the final results fulfill the designers' demands. The success of such a design process depends very much on the initial design concept proposed and on the opinions, judgements and experience of the designers. As such, the corresponding design process is often relatively ineffective since the structural type and arrangement, architectural layout and electrical/mechanical equipment are often simply devised and copied from previous designs. Because great numbers of such edifices will be required to fulfil the accelerating demands of urban commerce, it is vital to establish comprehensive method for the design of high-rise buildings. This investigation, will focus in particular upon identifying "best concept" designs. Equally significant will be its focus on the development of a general approach by which such designs may be achieved

1.2 DESIGN OF A HIGH-RISE BUILDING

Traditionally, the architect was the master builder with control over the entire design and building process. However, in time, industrialization and increasingly complex projects have required architects to abandon areas of activity that are better served by expert engineers. Such areas in building design and construction include those related to structural, mechanical, electrical and construction engineering. In addition, the services of experts in value engineering and finance are also often required (Holgate, 1986).

The design of a project is the result of a gradually evolving concept commencing from an initial scheme generated by the design team and the owner. Initial concepts are influenced by the required functionality of the project. Further preliminary development of concepts accounting for site conditions, zoning laws and finances, structural, mechanical and electrical systems, as well as the aesthetics features of the project, eventually lead to schematic drawings. At this stage, upon approval from civic authorities, more detailed design is generated. With the input of engineers from different disciplines, the major electrical, mechanical and structural subsystems are sufficiently detailed for each team member to have information regarding the others' requirements and responsibilities, thereby enabling everyone to finalize their respective subsystem. This detailed stage is coordinated by periodic meeting among the different disciplines of the design team and involves significant communication of drawings and documents.

1.3 CONCEPTUAL DESIGN

Conceptual design is the earliest phase of the building design process and commences with a set of initial concepts. Keeping in mind that there is no single solution with optimal performance with respect to all requirements due to the fact there are conflicting objective criteria, designers must evaluate different competing criteria with the view to achieve a good compromise design. That is, the selection of a suitable solution involves making subjective compromises between conflicting objective criteria.

The conceptual design phase involves making decisions that can have the maximum influence on the final design and project cost. One study suggested that as much as 80% of the total resources required to construct a building are committed by the decisions made in the first 10% of the design process (Deiman and Platt, 1993). Albeit, designers often tend to spend most of their working time on the detailed design phase, where the scope for significant improvement is much less. They often only generate a single design concept, or at most a few that satisfy the design criteria, because traditional design practice places severe constraints on time and design costs. Extensive generation and evaluation of alternatives is only possible with the help of computer-based methods. That said, such computer methods for conceptual design are not yet available to designers in practice. One reason for this situation is that conceptual design has not yet evolved into a well-defined procedure.

An overall view of the design process and the design itself is needed when performing conceptual design. The designer at the early stages must understand the many factors affecting the building being designed. Such a global approach to high-rise building design should include account for structural efficiency, erection cost, mechanical and electrical requirements, operating cost, quality of space and comfort, and rental revenue. One should add to this list such things as initial land cost, interest on borrowed money and maintenance cost. Significant complexity comes from the need to determine the relative benefits of all of these various quantities and qualities (Rush 1986).

1.4 LITERATURE REVIEW

This literature review focuses on research concerned with computer-automated conceptual design of buildings and other civil structural entities. It is important to note that this area of research is very current and not yet well investigated and, therefore, that there are not many related documents directly available concerning high-rise buildings. Furthermore, it is necessary to mention that the researches discussed in the following do not cover all aspects involved in the global conceptual design process but do try to address the problem from several important viewpoints.

Akin et al (1988) performed automated space planning using a computer-based system called HeGel, in which the heuristic generation of floor plan layouts is based on a formal model of the architectural design process. Given an outline, HeGel generates alternative locations for design units, say a furniture set, while accounting for constraints like direct access, natural light, privacy, etc..

Baker and Fenves (1989) presented a conceptual structural design system that determines compatible structural and architectural configurations that provide the basis for subsequent analysis and optimization to find 'best' solutions.

Gowri (1990) developed a system to create and analyze the building envelope. The system helps designers to consider a large number of alternative material and construction systems. Generation of feasible alternatives is performed as a constraint-based search problem, for which components of the building envelop must satisfy performance requirements. Selection and ranking of the alternatives is done after evaluation involving an array of performance criteria as well as priorities supplied by the designer.

Haber and Karshenas (1990) developed a system called CONCEPTUAL, an expert system for structural design with emphasize on the conceptual stage of design. The system places emphasis on establishing cost estimates for different building alternatives. The selection of the structural type is made from a predefined database of components. The rational behind the development of the system is based on the argument that designers base their decisions mostly on experience and intuition. As such, the automated knowledge-based expert system is based on a model of the process by which a human expert arrives at a conceptual design for a building.

Liggett (1990) developed a computerized approach to conceptual design in which methods of algorithmic generation of alternatives and manual construction of layouts by means of a graphical interface are used to solve space allocation problems.

Maher and Fenves (1984) presented HI-RISE, an expert system for the preliminary structural design of high-rise buildings which serves as a designer's assistant by generating feasible alternative load carrying systems. HI-RISE utilizes both a rule- and frame-based knowledge representation. Frames are used to represent knowledge of structural systems, subsystems and components in a hierarchical manner. Given a fixed three dimensional layout, alternative load carrying systems are generated by a search through the hierarchy of structural subsystems, using heuristic rules to eliminate infeasible systems. Typical design alternatives are rigid frames, cores, tubes, braced frames, etc., with steel or concrete construction. The user can select one of the feasible alternatives for further detailed investigation.

Moore and Miles (1991) developed a user-oriented knowledge based system (KBS) for use in the conceptual design of bridges. Since the conceptual design of bridges is mainly based on heuristics and personal judgement and therefore depends greatly on the experience of the engineer, the KBS uses an interactive knowledge elicitation (KE) process, with the unusual feature of several experts involved to provide the knowledge in the same domain. The KBS is later being verified by the experts involved in the KE process, and by other independent experts and users. The experience gained from the development, implementation and verification of the system was shown to be relevant to the creation of future KBS's, particularly within the engineering environment. In their next attempt, Miles and Moore (1991) examined the use of KBS technology in civil engineering design, with especial reference to bridge design. Based on assessments of the effectiveness of the systems, and on reasons for the slow acceptance rate of KBS's by industry, the current use of heuristics in KBS's was analyzed and a few broad groupings of heuristics were identified. The utility of KBS's was noted to be as valuable design aids in addition to being simulations of expert thought processes. A further study by Moore and Miles (1996) discussed a method for improving the consideration of costs during conceptual design, in which computer-based techniques allow designers to rapidly explore many options to a high level of detail so as to inform them of the cost implications of their decisions. The method is able, at the very early stages of design, to analyze the impact of large and small changes to the artifact. The paper presents a practical application of the method to bridge design.

A further study by Moore et al (1996) developed a knowledge base system for bridge aesthetics for use in the conceptual stage of design. They described the methodology used to obtain the knowledge, which, due to the subjectivity of the domain, involved supplementing knowledge elicitation with questionnaires as a means to check the correctness of the experts' rules. This KBS was suitable for assessing the aesthetics of small to medium size road bridges.

Moore et al (1997) also developed of a decision support tool for the conceptual design of bridges which incorporated a restructured version of a previous knowledge base (KB). This new version allows the system to be altered and extended by system users who are not expert engineers. This is accomplished by using a novel form of KB in which the knowledge is fragmented into separate concepts associated with design solutions. In addition, the system uses a new type of user interface which involves a critiquing style of interaction in which the KB only interacts with the user when it detects either a possible error in the design or a more suitable design solution.

Sham (1991), as part of a much greater research effort in the computational modeling of conceptual design using artificial intelligence techniques, presented an experiment in knowledge elicitation (KE) which encompassed the entire process of extracting, characterizing and crafting design knowledge into an exploitable form, with special reference to bridge design. The merits and shortcomings of the methods used in the experimentation were compared and contrasted, while the problems involved in acquiring knowledge in a multiple expert environment were discussed.

Borkowski et al (1991) investigated conceptual decisions taken during the preliminary design of structures, use of computer support for such decisions and employment of AI-techniques in computer support modules. Several issues such as intelligent access to previous design experience stored in databases, automatic generation and comparison of plausible alternatives, and acquisition of new knowledge through algorithmic structural optimization are discussed. The difficult task of supporting innovative solutions was an important consideration of the work, and several knowledge-based computer programs illustrated the implementation of the proposed ideas.

Reddy et al (1993) discussed the use of informal methods in the optimization of concrete structures at the conceptual design stage, such as heuristic designers' expertise, and at the design realization stage, such as approximate numerical techniques. While discussing the differences in the optimization of steel and concrete structures, they introduced a formal method for the cost optimization of reinforced-concrete structures using a derived cost function for estimating the optimum sizes of members at the conceptual design stage. The corresponding system, with capabilities limited to the design of columns, beams and slabs, was developed for the optimum-cost design of reinforced-concrete members and made use of heuristic knowledge provided by expert designers.

Fuyama et al (1993) attempted to computerize the conceptual design of structural steel buildings. The motivation behind the work was reported to be the positive effect that the conceptual stage of design has on the quality of a building, as well as the then current lack of computer-bases techniques to handle the conceptual design of building structures. In their research, an interactive design system called Building Engineering and Reasoning Tool (BERT) was developed with the capability to design and "optimize" member sizes through reasoning about the behavior of a steel frame structure while different configurations were evaluated by means of a cost estimating scheme that accounted for material, fabrication and erection costs. A further study (1994) elaborated upon BERT, with focus on a discussion of the object-oriented representation and reasoning schemes employed in the implementation of the design system. Another study (1994) presented a more complete version of BERT which incorporated a behavior-based methodology for designing structural members to meet strength and interstory drift demands caused by equivalent static earthquake loads acting on tall moment-resisting steel frame structures. This work attempts to minimize the total weight of the structural system, with due consideration given to both strength and stiffness requirements.

Mathews and Rafiq (1994, 1995) applied genetic algorithm (GA) technology in the development of a decision-support system for conceptual building structural design. They mentioned that pre-processing enables independent sub-systems to be identified, thereby reducing the complexity of the design space. For example, reinforced concrete columns vary in size and reinforcement detail but, for given load ranges, the cost of optimal sections can be determined beforehand. Using such pre-processing, a GA was applied to minimize the cost of the structural grid layout and floor system for a building of given plan dimensions, while at the same time maximizing floor flexibility and usage comfort. However, the study did not consider the design as a multi-criteria optimization problem but, rather, found an optimal design for each criteria separately.

Mathews, Rafiq and Bullock (1996), in a further attempt to develop an algorithm for the conceptual design of buildings, proposed the integration of different objective criteria, such as the reduction of overall cost and the improvement of final-built quality. To make their task easier, they proposed looking at medium-rise office buildings instead of high-rise buildings. The use of a GA as a search tool was recommended and they proposed a specific algorithm which allowed for interaction with the designer. However, the study only achieved a prototype flowchart of such an algorithm.

Arciszewski (1984) dealt with structural shaping, where qualitative parameters and their feasible states are presented in the form of a morphological field. In this research, computer-aided analysis by means of a nonhomogeneous Markov chain is proposed for the determination of wind bracing types.

Arciszewski and Ziarko (1991) proposed a new approach to structural parametric optimization. The method employs a two-stage case-based optimization process, including learning and production. The system later was tested for four experiments concerning the design and optimization of rigid steel frames under different loadings. The experiments were performed to determine the feasibility of the proposed optimization. It was concluded that the expected feasibility of the method was achieved.

Arciszewski et al (1994) developed a methodology for applying machine learning to problems of conceptual design, and presented a case study of learning design rules for wind bracing in tall buildings in which design rules are generated by induction from examples of minimum weight designs. This research investigated the suitability of machine learning methods involving constructive induction, which automates the search for problem-relevant attributes beyond those originally provided. The final product is a set of decision rules which specify design configurations that are recommended, typical, infeasible, or those that should be discarded. These learned rules capture some of the expert's essential understanding of the design characteristics involved in selecting wind bracing for tall buildings. The results of the case study were promising and demonstrated the potential practical usefulness of the proposed methodology for the automated generation of design rules.

Another attempt by Arciszewski and Michalski (1994) focused on initial ideas for a design theory based on the inferential theory of learning. The theory employs a process that changes design specifications and background knowledge into the desired design. The paper provides the basic tenets of the theory and proposes a system of design knowledge transmutations. To further elaborate on the theory, individual transmutations are established and explained using examples from the area of the conceptual design of wind bracings in steel skeleton frameworks for tall buildings.

Szczepanik et al (1996) presented the results of a performance comparison study of two symbolic learning programs. One program uses single representation space while the other employs constructive induction. The experiment was conducted on a set of optimal designs of wind bracing in steel structures. The paper concludes that constructive induction offers several benefits when compared to learning based on the use of single representation space.

Arciszewski et al (1999) once more presented the basic concepts of inventive design, as applied to structural engineering. The paper discusses the importance of the ability to produce an inventive design in a short time. This study employees genetic algorithms to generate best concept designs in the structural engineering field, with focus on steel frames and outriggers.

Grierson (1994) proposed a computational model for the conceptual design of simple bridges and buildings through the integration of a genetic algorithm (GA) and an artificial neural network (ANN). Later studies (1996, 1997) extended the work to the conceptual design of building structural systems in general. Self-organizing solution paradigms (GA+ANN) were used to develop a computing system that optimized the conceptual design process. Specifically, a preliminary computational model for conceptual design was presented that employs a genetic algorithm in tandem with a neural network to generate best-concept design solutions using directed random search guided by artificial learning. A neural network is used to establish the fitness of each conceptual design. The computational model is illustrated for the routine conceptual design of bridge and building structural systems.

Kunighalli and Russell (1995) provided a framework for the development of Computer-Aided-Design/Computer-Aided-Construction (CAD/CAC) systems which contain software tools related to conceptual design, structural and foundation analysis, design of structural components, routine mathematical and optimization functions, construction management, Computer Aided Process Planning (CAPP), process simulation, and constructability analysis. The research highlights work in domains related to CAD/CAC and investigates the enhanced efficiency and effectiveness of the proposed systems for the design and construction of civil engineering structures.

Chierdhast and Ambo (1995) discussed a practical approach for solving topology optimization problems for planar cross-sections, which involved a rigorous method for the conceptual design of structural components. A standard nonlinear programming algorithm involving continuous-valued design variables is used to solve the optimization problem.

Shiva Kumar et al (1995) presented a knowledge-based system for the design of concrete bridges with emphasis on the initial modeling of the problem, which is an important step in KBS development. KBS's for bridge design have been traditionally implemented as simple production rule systems. However, since a thorough examination of the application domain and identification of the required artificial intelligence techniques are necessary for full KBS development, this research argues that an integration of AI-based problem-solving techniques is necessary to address the various tasks of bridge design. Furthermore, it argues that a design process model based on rule-based inference, synthesis and critical evaluation techniques is required to address the knowledge-intensive tasks of site selection, bridge layout planning, conceptual design and preliminary design of concrete road bridges, in an efficient manner.

In a work that applied a GA to conceptual design, Hudson and Parmee (1995) summarized methodologies for a grammar-based chromosome system. The study pointed out that when applying a GA to a conceptual design problem that neither the structure of the final solution nor the design space to be searched should be fixed, that the evaluation of concepts does not involve simple qualitative or quantitative comparison, and that a range of good solutions is more important than a single solution. The stated goal of the work was to develop practical systems for the qualitative, rather than quantitative, solution of realistic conceptual design problems.

In a recent study, Beck and Parmee (1998) focus on the conceptual design of a building and attempt to design such as to minimize both project cost and heat loss and maximize both heat gain and occupant comfort. A fine-grained GA technique was employed. This study, however, did not include account for alternative structural layouts.

Fruchter et al (1996) presented an interdisciplinary communication medium for collaborative conceptual building design which involved intensive cross-disciplinary communication of design concepts and decisions. The requierment to overcome extensive delays, miscommunication and confusion caused by difficulty in producing and expressing information, which often have a negative impact upon the time required to achieve design consensus and on the quality of the final design, triggers the need for a framework for interdisciplinary communication to support collaborative conceptual design. Since then-current computer tools provided little support for the special needs of representation and reasoning posed by cross-disciplinary communication in collaborative conceptual building design, this study proposed a method for interdisciplinary communication. The method enabled designers to propose a shared model, interpret the model for various disciplines, critique the discipline models to derive behaviour and compare it to function, and explain the results to other members of the team. Such a propose-interpret-critique-explain paradigm as a communication cycle for collaborative conceptual building design is presented as an experimental software prototype that integrates graphic representations and AI reasoning for evolving building design concepts and uses a graphic environment as the central interface to reasoning tools that support the collaborative design process. In a second study (1996), Fruchter elaborated on collaborative conceptual building design using the previously developed system to integrate a shared graphic modeling environment together with network-based services to accommodate the many perspectives of an architecture/engineering/construction team.

Jo and Gero (1996) employed a GA for space layout planning. In the study they optimize the distribution of available space among different activities in a building in order to minimize the cost of taxiing between those activities. They concluded that a GA is able to generate good designs for complex design problems.

Wang and Gero (1997) discussed the application of machine learning techniques in a knowledge support system for the conceptual design of bridges. A sequence-based prediction method is used in which the most recent numbers of similar design cases are used in predicting the characteristics of the next design, and more recent cases are given stronger influence on decision making in the new design situation than older cases. This research developed a prototype of a sequence-based prediction tool and carried out a number of experiments comparing results with those for other methods and concluded that the method has potential for success in engineering design.

Chinowsky (1996) introduced an cooperative conceptual design environment that supports interdisciplinary design teams with enhanced information access and object manipulation capabilities. This research demonstrates the need for such s system through arguing that, while computers provide significant assistance in the storage of project documents and the creation of detailed drawings, pivotal economic and quality enhancement benefits are generally lost during the conceptual design stage, and that such benefits have great opportunity to economically, aesthetically and qualitatively impact final design solutions.

Park and Grierson (1997) developed an algorithm for the optimal conceptual design of medium-rise buildings accounting for the cost of the structure and the quality of occupant space. The approach generates best-concept designs by simultaneously optimizing two conflicting criteria concerning the project cost and the flexibility of floor space usage. Specifically, Pareto optimal equal-rank designs that are not dominated in both criteria by any other feasible design are found using a multi-criteria genetic algorithm. The MGA process resembles that of a simple GA (Goldberg 1987), except that the fitness evaluation of candidate designs is based on a distance metric related to the Pareto-optimal set. The study considered only one type of structural system and assumed that the building is supported laterally by means other than the structural frame. They found that there is a performance trade-off between the objective criteria and that it is up to the designer to make some compromises to arrive at an acceptable design.

Ravi (1997), in an attempt to create a knowledge-based system for the integrated design of multistory office buildings at the preliminary stage, developed an interactive program which poses questions to the designer as it generates a desirable design. However, while being user-friendly and able to generate promising results, the system requires a designer with broad knowledge of the different aspects of the design in order to be successfully applied. A further limitation of this system is the lack of inclusion of heating, ventilation and air conditioning (HVAC) considerations in the design procedure.

Shrestha and Ghaboussi (1998) discussed a methodology for the evolution of optimum structural shapes in which a genetic algorithm is used to evolve optimum shape designs that are free to assume any geometry and topology and do not necessarily resemble any conventional design. The approach has the potential to generate new and innovative designs, especially when more complex design problems are attempted. The methodology addresses configurational and topological aspects of the design, and considers discrete member sizes and multiple loading cases for planar and space structures.

Shea and Cagan (1999) used shape annealing techniques as the basis of a computational method for structural configuration design that supports structural designers with varying design intent. The work involved studying roof truss designs conceived by architects and engineers as well as those generated using shape annealing, with the purpose of evaluating the capabilities of shape annealing techniques to meet the varying needs of designers and, as well, to generate spatially intriguing and functional structures at the conceptual design stage. The study concluded that shape annealing generates alternatives that appeal to designers while providing insight into relations between structural form and function.

1.5 OBJECTIVES AND SCOPE OF RESEARCH

As presented in the foregoing literature review, most studies to date concerning the conceptual design of buildings have been limited to but a few aspects of a building project. This research proposes to achieve optimal conceptual designs of high-rise buildings while accounting for all major aspects and conflicting cost-revenue objective criteria. The study will not address the spatial arrangements of functional zones within areas, but will take into account the efficiency of architectural and structural layouts and mechanical and electrical systems.

The primary objective of the research is to develop a computer-based technique for the optimal conceptual design of high-rise buildings, while accounting for competing objective criteria related to capital cost, operating cost and income revenue. Self-organizing computing methods (e.g., genetic algorithms) are employed as the computational basis for modeling the unstructured and evolving nature of the conceptual design process. In particular, a genetic algorithm is used to develop a computational capability to exhaustively explore design domains and evaluate possible design scenarios. A multi-objective approach is taken whereby optimization techniques are employed to establish Pareto-optimal curves/surfaces representing the trade-offs between the three competing cost-revenue objectives for a design. The trade-off relationships between the design objectives provide clear guidelines for the selection of structure type, configuration, layout, materials, windows and elevators (i.e., general of the design concept). Importantly, this trade-off information permits designers to study the gains and losses incurred when selecting one design concept over another, which consequently provides the ability to know the approximate effect of design decisions on the capital cost, operating cost and income revenue for high-rise buildings.

The achievements of the research are summarized as follows:

I. Account for three important and conflicting objective criteria related to capital cost, operating cost and income revenue for the global optimal conceptual design of high-rise office buildings.

II. Development of evaluation functions for the cost-revenue objective criteria.

III. Account for a wide variety of architectural space layouts.

IV. Account for different gravity and lateral load structural systems.

V. Development of a set of rational design constraints for buildings conceptual design.

VI. Development of a practical computer-based design tool for the optimal conceptual design of high-rise office buildings.

VII. Capability to create Pareto trade-off curves/surfaces to facilitate designers in their task to choose good compromise designs for high-rise buildings.

VIII. Capability to estimate the potential profitability of building designs over time.

1.6 ASSUMPTIONS AND IDEALIZATIONS

The assumptions and idealizations adopted in this study for the conceptual design of high-rise office buildings are as follows:

1. Mean's manuals are used to estimate the cost of construction (R.S. Means 1999).
2. Beams and slabs span over columns to form floor systems.
3. Columns are arranged in lines in two orthogonal directions.
4. Structural grid lines defining bay sizes are regularly spaced.
5. Only one type of floor layout prevails over all stories of the structure.
6. Only static or equivalent static environmental loads, as permitted by building codes, are considered. Dead loads are assumed to be invariant with changes in member sizes.
7. The material behaviour of steel and concrete is assumed to be linear elastic, and second-order geometric nonlinear (P-Δ) effects are not considered.
8. All structural members are taken to be prismatic and straight.
9. Connections between members are assumed to be either fully fixed or simply pinned. Member lengths are measured using centre to centre dimensions.

10. It is assumed that the floor area occupied by columns is negligible.

11. In lieu of exact analysis, the portal method of approximate analysis (Hibbeler 1997) that locates the points of flexural inflections at mid spans of members is used to estimate forces in beams and columns of systems relying on the beam-column connections to carry the lateral loads. (The cantilever method of approximate analysis is more accurate than the portal method for slender buildings. However, the latter method is computationally less expensive and, through a separate study, was found to produce comparable results to the cantilever method for the building considered by this study). In structural systems carrying lateral loads by means other than the framing action of beams and columns, such as bracings and shear walls, the forces induced in these stabilizing elements are found through determinant modeling and analysis (e.g., a shear wall acts as a cantilevered column). Member sizing is performed based on the worst-case combination of internal forces induced in members due to dead (D), live (L) and wind loads (W), as follows:

    a) 1.25 D+ 1.5 L

    b) 1.25D + 1.5 W

    c) 1.25D + 0.70 (1.5 L + 1.5 W)

12. The distribution of lateral loads between different lateral load resisting systems is as explained in Chapter 2.

13. It is assumed that windows are installed one meter above floor level (task level) and stretch to the ceiling.

14. For calculation of energy gain or loss, it is assumed that minimum and maximum temperatures occur in January and July, respectively.
15. The building mass is considered to have no impact on HVAC energy consumption (Canadian Institute of Steel Construction & Canadian Steel Construction Council).
16. The building working hours are assumed to start at 8 am and end at 6pm.
17. It is assumed that the artificial lighting system and office equipment are fully functioning during working hours and only half operating during off-work hours.
18. The ventilation system is assumed to be working at full power in working hours and at half power at other times.
19. It is assumed that the HVAC system only employs gas boilers and electrical chillers.
20. It is assumed that designs having larger spans, more window area and bigger floor area that benefits from natural day light are more likely to generate higher revenue income.

1.7 THESIS OUTLINE

Presented in Chapter 1 are an overview of the conceptual design of high-rise building, a literature review of the state of previous work concerning computerized tools for the conceptual design of engineered structures, and an outline of the study.

Presented and discusses in Chapter 2 are the major structural, mechanical and electrical systems involved in the design of high-rise buildings and their suitability for different building scenarios. The chapter is concluded by identifying the parameters and variables adopted by the study as the basis of the computer-based method for conceptual design of high-rise buildings that is developed and applied in Chapters 3 and 4.

Developed in Chapter 3 is the computer-based conceptual design method for office buildings, based on Pareto optimization using a multi-criteria genetic algorithm. A major portion of the chapter is devoted to describing the means by which the capital cost, operating cost and revenue income are evaluated for a building design. The chapter is concluded by a detailed description of the conceptual design procedure.

Presented in Chapter 4 are four design examples to illustrate the effectiveness, efficiency and practicability of the developed computerized tool for the conceptual design of high-rise office buildings. Results are presented in colour graphics that identify the trade-off relationships between cost and revenue for office buildings.

Summarized in Chapter 5 are the conclusions resulting from the study and directions for future research.

Chapter 2

High-Rise Office Buildings: Systems, Parameters and Variables

2.1 INTRODUCTION

Factors that affect decisions made in the design of high-rise buildings are primarily initiated by the interests of the different parties involved, as follows (Guise 1990):

1. Owner:
   - Market Feasibility
   - Financial feasibility
2. Architect:
   - Spatial requirements of building envelope and services
   - Quality and cost of internal environment
3. Structural Engineer:
   - Gravity and Lateral load systems
   - Foundations
4. Mechanical/ Electrical Engineer:
   - Hydraulics/ Piping
   - Electrical/ Lighting
   - Elevators/ lifts
   - HVAC
   - Energy consumed by service systems
5. Construction Engineer:
   - Labor/ Equipment
   - Time/ Climate

Globalization of building optimality is yet difficult to achieve because of the lack of agreement across the industry for standard global models. Often, the optimization interests of the parties involved in the design are in conflict. For example, an architect wants maximum flexibility of floor space usage and high comfort level while a structural engineer desires the most economical and safe structure. It is apparent that optimum floor flexibility may conflict with having the lightest structure as column and girder layouts that achieve a least-weight structure may have an adverse impact on floor space usage. As another example, by increasing the height of a building for constant required area, the building footprint and, hence, the land cost will decrease but the structural, vertical transportation and façade costs will increase. Moreover, even the type of structural system and material may change with height of a building, as illustrated in Figure 2.1 (Khan 1974). For constant floor area, a taller building means a smaller footprint, which then implies the use of mat foundations or piles in lieu of less expensive spread footings. Furthermore, for a fixed required floor area, the more slender a building is in one direction the greater is its surface area on the perimeter, which causes increased capital and operating costs for heating, ventilation and air conditioning (HVAC) systems. At the same time, a greater perimeter means more access to daylight which decreases the lighting expenses and the heat generated by the lighting system and increases the quality of the space and the comfort level of occupants. This results in decreased HVAC cost during summertime and increased HVAC costs during wintertime. Conversely, the increased absorbed energy from the sun causes more spending on HVAC systems during summertime and less spending during wintertime. It is also known that the occupants of a high-rise building are generally negligent in turning off lights, even if there is enough light from outside, and, therefore, to benefit from daylight it is necessary to install an automated system which dims the lights in the presence of enough daylight, which will itself increase the lighting capital costs.

Considering the interactions noted in the forgoing for but a few examples, one can see that the prediction of optimal conceptual design scenarios for a high-rise building is a very complicated task indeed. Fortunately, relatively recent advances in distributed computing paradigms have been shown to be well suited for the complex task of modelling the global conceptual design optimization problem.

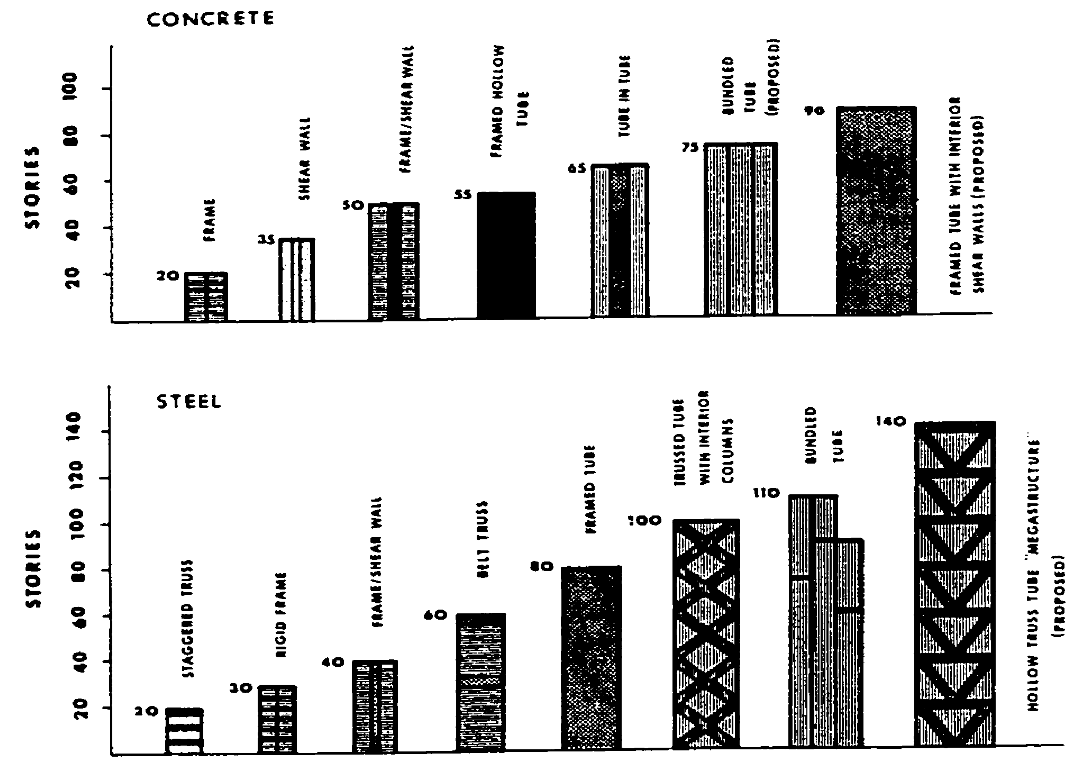


Figure 2.1: Different Structural Systems

2.2 SYSTEMS IN HIGH-RISE BUILDINGS

The first step towards optimizing a building is to identify its major systems. While an optimized high-rise building does not necessarily result from individually optimized systems, the identification of optimum individual major systems must be the first step prior to integrating these systems into the whole building. Structural, mechanical and electrical systems are the major systems for a high-rise building that are of primary concern to engineers.

2.2.1 Structural Systems for High-Rise Buildings

In general, the structural system of a building is a complex three-dimensional assemblage of various combinations of interconnected structural elements. The primary function of a structural system is to carry effectively and safety all the loads acting on the building, and eventually to transmit them to the foundations. A structural system is therefore expected to: carry dynamic and static vertical loads; carry horizontal loads due to wind and seismic effects; resist stresses caused by temperature and shrinkage effects; resist external or internal blast and impact loads; and resist vibration and fatigue effects. At the same time, the structural system is subject to the following requirements: it should conform with architectural requirements and those of the building's users and owner; it must interact with and facilitate service systems, such as heating, ventilating, air conditioning, horizontal and vertical transport, and other electrical and mechanical systems; it should facilitate simple and fast erection of the building; it must be resistant to fire; it must enable the building, foundations, and the ground to interact properly; and it should be economical.

A variety of factors has to be considered in the process of selecting the most suitable structural system for a high-rise building. The selection is a complicated process, and no simple clear-cut design procedures are available. The design team must use every available means, such as imagination, previous experience, and relevant literature to arrive at the best possible solution in each particular case.

There are several sub-systems common to all types of structural systems (steel. concrete, composite), namely:

1. Vertical load resisting systems: a)Floor systems; b)Columns
2. Horizontal load resisting systems
3. Structural joints
4. Energy dissipation systems (dampers)

In this study, only the first two subsystems will be investigated. The most frequently used structural systems for high-rise steel and concrete buildings are shown in Figure 2.1 (Khan, 1974). It can be observed that Figure 2.1 recommends different types of structural systems depending on the number of stories and the building material. In general, however, it is extremely difficult to apply accurately a classification system for structural systems of high-rise buildings.

As the height of a building increases, the design of its structural system becomes increasingly specialized and complex. A variety of factors, many of them difficult to identify at the schematic level, can have a major influence on the selection and design of a structural system; the immense vertical loads on the structure, the character of wind and earthquake forces applied to a building specific to the building site, the local foundation conditions and, on top of all, the relative cost of various construction systems within the region are all important factors that a structural engineer has to consider. For these reasons no serious attempt at the design of a high-rise structure should be made without the participation of a qualified structural engineer, even in the early phases of design. In general, for high-rise buildings designed for a similar purpose and of the same material and height, the efficiency of different structures can be compared roughly by their weight per unit floor area. In these terms, the weight of the floor framing is influenced mainly by the floor span and is virtually independent of the building height, while the weight of the columns, considering gravity load only, is approximately proportional to the height of the building, see Figure2.2 (Smith and Coull 1991). Buildings up to 10 stories designed for gravity loading can usually accommodate wind loading without any increase in design stresses for combined loading. For buildings of more than 10 stories, however, the additional material required for lateral load resistance increases nonlinearly with height so that for buildings of 50 stories and more the selection of an appropriate structural form may be critical for the economy and, indeed, the feasibility of the building (Smith and Coull 1991).

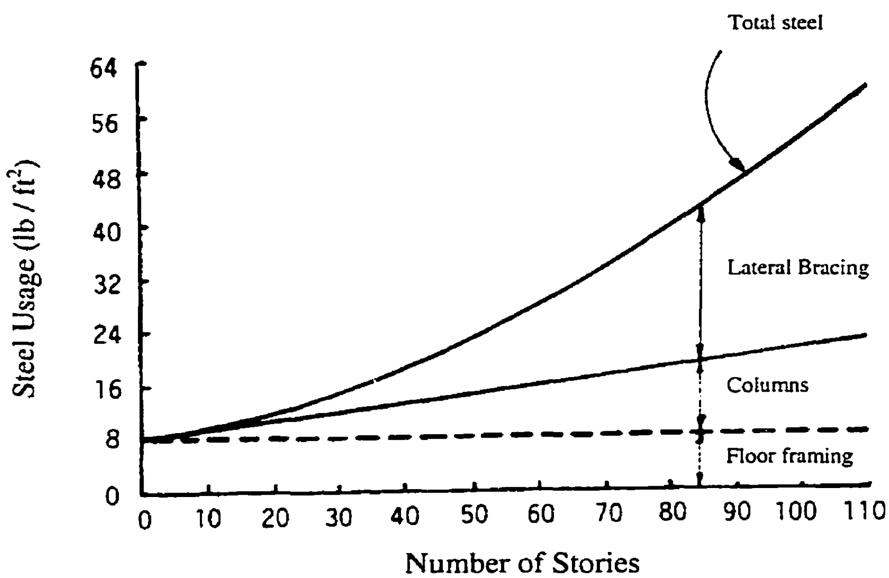


Figure 2.2: Use of Steel in Tall Buildings (Smith and Coull 1991)

Refer to Appendix 2.A for a description of the basic structural systems for tall buildings, and their relationships to the total design of the building, considered by this study.

2.2.2 Mechanical Systems

The most important mechanical systems in a tall building are: 1) Heating Ventilation and Air Conditioning (HVAC); and 2) vertical distribution services (elevators). Refer to Appendix 2.B where the mechanical systems considered by this study are discussed with the view to establish appropriate rules for their design within the context of high-rise office buildings.

2.2.3 Electrical Systems

The main components of the electrical system in an office buildings are: electrical outlets; lighting; and the electrical parts related to mechanical systems. Since the electrical parts related to mechanical systems are directly dealt with in the design of HVAC and elevator systems, this study is only concerned with electrical outlet and lighting systems. The electrical outlet system is dependent on the total area and function of the building and its cost for an office building is a function of total area and the unit cost for electrical outlets. The design of good lighting in buildings, daylight or artificial, is a matter of both quality and quantity. The architect in collaboration with the lighting engineer is concerned not only with providing enough light for the given tasks in each space but also with visual efficiency and comfort.

Lighting systems in a building can be categorized as: artificial lighting; and daylight. Identifying the best artificial lighting system is a straightforward task since it is an accepted fact that fluorescent lamps generate the best kind of lighting at a low cost for office buildings. Such lighting generally demands a level of illumination that consumes only about 20w/hr if recent lighting fixtures are employed (Reid, 1984). On the other hand, natural lighting or daylight is not available in all times, is less predictable and controllable than artificial lighting, varies with place, time and weather, and is not necessarily free because of the heat gain it causes through the windows. Daylight does have some significant advantages, however, such as decreasing internal energy consumption on sunny days and increasing the efficiency of the occupants of the building. The ratio of window area to the perimeter surface area of the building is an important factor in providing daylight of appropriate quantity and quality.

2.3 PARAMETERS AND VARIABLES FOR CONCEPTUAL DESIGN

The structural, mechanical and electrical systems discussed in this Chapter and related Appendices 2.A and 2.B give rise to the parameters and variables that govern and define the computer-based method for the conceptual design of high-rise office buildings developed in Chapter 3 and applied in Chapter 4.

2.3.1 Design Parameters

The basic design parameters considered by this study are defined by local location information, and are (e.g., see Table 4.1): land cost and property tax rates; office space lease rates; mortgage and inflation rates; electrical and gas energy unit costs; daylight factors; inside and outside temperatures and humidities; building geographical location and orientation; gravity and lateral loads; and cost location factors which relate US national average costs to the local cost of building components, see Table 2.1 (refer to Table 4.1 for representative number of these cost location parameters).

Table 2.1: Cost Location Factors

Cost Location Factor	Description
CCLF	Concrete Cost Location Factor (ratio of local concrete cost to US national average concrete cost)
C_LCLF	CLadding Cost Location Factor (ratio of local cladding cost to US national average cladding cost)
ECLF	Electrical Cost Location Factor (ratio of local electrical cost to US national average electrical cost)
E_LCLF	ELevators Cost Location Factor (ratio of local elevators cost to US national average elevators cost)
FCLF	Forming Cost Location Factor (ratio of local forming cost to US national average forming cost)
F_ICLF	FInishing Cost Location Factor (ratio of local finishing cost to US national average finishing cost)
MCLF	Mechanical Cost Location Factor (ratio of local mechanical cost to US national average mechanical cost)
RCLF	Reinforcement Cost Location Factor (ratio of local reinforcement cost to US national average reinforcement cost)
R_OCLF	ROofing Cost Location Factor (ratio of local roofing cost to US national average roofing)
SCLF	Steel Cost Location Factor (ratio of local steel cost to US national average steel cost)
WCLF	Windows Cost Location Factor (ratio of local windows cost to US national average windows cost)

Additional design-specific parameters considered by this study are defined by the building restriction limits, and are (e.g., see Table 4.1): a_{max}, b_{max} = maximum allowable footprint dimensions in the a and b directions for the building; H_{max} = maximum building height; A_{req} = minimum required area of lease/rental office space; h_{cle} = minimum permitted floor-to-ceiling clearance height; CPD_{min} = minimum permitted distance between building core and perimeter; $C_a \times C_b$ = core area as a fixed percentage of footprint area; $D_a/D_{b\ min}$ = minimum aspect ratio allowed for the building; and $H/D_{a\ min}$ = maximum slenderness ratio allowed for the building.

2.3.2 Primary Design Variables

For given parameter values, the computer-based method for conceptual design developed by this study initially finds the values of a number of primary variables that define the architectural and structural systems for a high-rise office building. The primary variables (along with the ranges of possible alpha-numeric values they may be assigned) that are adopted by this study are listed in Table 2.2 in concise form, and are further elaborated upon in the following: ST = structural type (steel rigid frame, concrete rigid frame, steel frame and bracing, steel rigid frame and bracing, steel frame and concrete shear wall, steel rigid frame and concrete shear wall, concrete rigid frame and concrete shear wall, steel frame with bracing and outrigger trusses, steel framed tube, and concrete framed tube); BT = bracing type (K&K and K&X); CFT = concrete floor type (flat plate, flat slab, beam and slab, and waffle slab); SFT = steel floor type (steel joist and beam with steel deck and concrete slab, composite beam & cast-in-place slab, W-shape composite beam with steel deck and concrete slab, and composite beam with steel deck and concrete slab); S_a, S_b = the span distances between columns in the two orthogonal directions a and b of the building footprint (from 4.5m to 12m in increments of 0.5m); NS_a, NS_b = the number of column bays (from 3 to 10 in increments of 1); NTS_a, NTS_b = the number of tube column bays within the span distances S_a and S_b (from 2 to 5 in increments of 1); $DCDD$ = direction of randomly chosen core dimension to be designed first (a or b); CDF = fraction of building dimension to be assigned to the $DCDD$ core dimension (from 25%

to 80% in increments of 7.86%); *WIT* = window type (standard, insulated, standard heat absorbing and insulated heat absorbing); *WIR* = ratio of window area to maximum window area available on the surface of the building perimeter (from 25% to 100% in increments of 5%); and *WAT* = cladding type (pre-cast concrete, metal siding panel, stucco wall, glazing panel).

Table 2.2: Ranges of Primary Variable Values for the Conceptual Design of Office Buildings

Index	*ST*	*BT*	*CFT*	*SFT*	S_a,S_b (m)	NS_a NS_b	NTS_a, NTS_b (m)	*DCDD*	*CDF*	*WIT*	*WIR* %	*WAT*
0	([1]c) [3]Rigid frame	K&K	Flat plate	Steel joist & beam	4.5	3	2	*a*	0.250	Standard	25	Pre-cast concrete
1	(c)Rigid frame & shear wall	K&X	Flat slab	Com. beam & [5]CIP slab	5.0	4	3	*b*	0.329	Insulated	30	Metal siding panel
2	(c)Framed tube		Slab & beam	W & com. deck & slab	5.5	5	4		0.407	Standard [6]HA	35	Stucco wall
3	([2]s)Rigid Frame		Waffle slab	Com. beam, deck & slab	6.0	6	5		0.486	Insulated HA	40	Glazed panel
4	(s)[4]Frame & bracing				6.5	7			0.564		45	
5	(s)Rigid frame & bracing				7.0	8			0.643		50	
6	(s)Frame & (c)shear wall				7.5	9			0.721		55	
7	(s)Rigid frame & (c)shear wall				8.0	10			0.800		60	
8	(s)Frame, bracing & outriggers				8.5						65	
9	(s)Framed tube				9.0						70	
10					9.5						75	
11					10.0						80	
12					10.5						85	
13					11.0						90	
14					11.5						95	
15					12.0						100	

ST = Structural type (10 choices); *BT*= Bracing type (2 choices); *CFT* = Concrete floor type (4 choices); *SFT* = Steel floor type (4 choices); S_a, S_b = span distances between *columns along the building width a and length b (16 choices in each direction); NS_a, NS_b = number of column bays along the building width a and length b (8 choices in each direction); NTS_a, NTS_b = number of perimeter tube column spans within S_a and S_b; DCDD = direction of the core dimension to be designed first (2 choices); CDF = ratio of the core dimension to the overall length of the building in the same direction (8 choices); WIT= Window type (4 choices); WIR= Window ratio (16 choices); WAT = Wall cladding type (4 choices); [1]c = Concrete; [2]s = Steel; [3] Rigid frame = framework participates in carrying lateral loads; [4] Frame = framework does not participate in carrying lateral loads; [5] CIP = cast-in-place concrete; [6] HA = heat absorbing.*

2.3.3 Secondary Design Variables

For given values of the design parameters and determined values of the primary design variables, the values of a number of secondary variables are calculated to complete the description of the conceptual design of an office building. These secondary variables, which are concisely listed in Table 2.3, are described in the following.

Table 2.3: Secondary Variables

Secondary Variables	Description
$ADBL_a$, $ADBL_b$	Average Distance Between column Lines in *a* & *b* directions
C_a, C_b	Core dimensions in *a* & *b* directions
CFA	Column-Free Area factor
D_a, D_b	Building Dimensions in *a* & *b* directions (m)
DF	Depth of Floor (m)
H	Height of the building (m)
HF	Height of Floor
NCL_a, NCL_b	Number of Column Lines between the perimeter and core of the building in *a* & *b* directions
NE	total Number of Elevators
NF	Number of Floors
NOPF	Number of Occupants Per Floor
NRF, *NMF*	Number of Rentable and Mechanical Floors
NRSC	Number of Risers in a Stair Case for one floor
NSC, *WSC*	Number and Width of Stair Cases
NSE, *NPE*,	Number of Service and Passenger Elevators
OILSC	Overall Inside Length of Stair Case
OIWSC	Overall Inside Width of Stair Case
TCS_a, TCS_b	Tube Column Spans in *a* & *b* directions
TNO	Total Number of Occupants

Knowing the values of the primary variables S_a, S_b, NS_a and NS_b the building width D_a and length D_b are found as,

$$D_a = NS_a \times S_a \tag{2.1a}$$

$$D_b = NS_b \times S_b \tag{2.1b}$$

Having D_a and D_b from Eq. (2.1), and knowing the required floor area A_{req} (see Appendix 2.A), it is assumed that 20% of the floor area is taken by the core, and that 4% of the total area of a building is needed for mechanical floors, such that the number of rentable floors NRF and mechanical floors NMF are found as,

$$NRF = (A_{req} \times 1.25 \,/\, (D_a \times D_b))_{\text{Rounded up}} \tag{2.2a}$$

$$NMF = (A_{req} \times 1.25 \times 0.04 \,/\, (D_a \times D_b))_{\text{Rounded}} \tag{2.2b}$$

where the minimum acceptable value of NMF is unity (1), and total number of floors NF is then found as,

$$NF = NRF + NMF \tag{2.2c}$$

To find the height of the building, this study assumes that the depth of false ceiling is one-half meter (0.5m) and, for known depth of floor DF and specified floor-to-ceiling clearance height h_{cle}, finds the height of each floor HF to be

$$HF = (h_{cle} + DF + 0.5) \tag{2.3a}$$

Then, having NF and HF from Eqs. (2.2c) and (2.3a), the total height H of the building is found as,

$$H = HF \times NF \tag{2.3b}$$

Core dimensions are chosen to satisfy the requirement that the core area be 20% of the total floor area at each story level. This is achieved by randomly choosing one dimension of the core to be a fraction CDF of the dimension D_a or D_b of the building footprint in that direction (see Appendix 2.A), and then calculating the other core dimension to meet the required core area. For example, if the randomly chosen core direction $DCDD = a$ (see Table 2.2), the dimensions C_a and C_b of the core area are found as follows, in the order shown,

$$C_a = D_a \times CDF \tag{2.4a}$$

$$C_b = (0.2 \times D_a \times D_b) / C_a \tag{2.4b}$$

For known number of tube column spans NTS_a and NTS_b, within S_a and S_b, the corresponding distances between the tube columns are found as,

$$TCS_a = S_a / NTS_a \tag{2.5a}$$

$$TCS_b = S_b / NTS_b \tag{2.5b}$$

The minimum number of service elevators NSE and passenger elevators NPE are found as, see Appendix 2.B (Allen and Iano 1995),

$$NSE = ((NRF \times ((D_a \times D_b) - (C_a \times C_b)) / 24600)_{\text{Rounded up}} \tag{2.6a}$$

$$NPE = ((NRF \times ((D_a \times D_b) - (C_a \times C_b)) / 3250)_{\text{Rounded up}} \tag{2.6b}$$

The total number of elevators NE for the building is,

$$NE = NSE + NPE \tag{2.6c}$$

The number of occupants per floor *NOPF* and the total number of occupants *TNO* are found as, see Appendix 2.B (Allen and Iano 1995),

$$NOPF = ((D_a \times D_b - C_a \times C_b) / 9.3)_{Rounded\ up} \tag{2.7a}$$

$$TNO = NOPF \times NRF \tag{2.7b}$$

The number of stair cases *NSC* and their widths *WSC* are a function of the number of occupants per floor, and are found as (NBCC, 1990),

$$NSC = (NOPF / 500)_{Rounded\ up} + 1 \tag{2.8a}$$

$$WSC = (NOPF / NSC) \times 0.0092 \tag{2.8b}$$

Eq (2.8a) is accurate for buildings with footprints as large as 130m by 130m, which is in keeping with the upper bounds set on the primary variables NS_a, NS_b, S_a, and S_b in this study (see Table 2.2). The number of risers for each two-flight stair case *NRSC* is a function of the height of floor *HF* and is found as (Figure 2.3),

$$NRSC = (5.42 \times HF + 0.25)_{Rounded\ up} \tag{2.8c}$$

Allowing for 0.15 m of space between ramps, and taking a landing area to be as wide as the stair itself, the overall inside length *OILSC* and width *OIWSC* of a stair case are found as (Allen and Iano 1995),

$$OILSC = (NRSC \times 0.280) / 2 + WSC \times 2 \tag{2.8d}$$

$$OIWSC = WSC \times 2 + 0.15 \tag{2.8e}$$

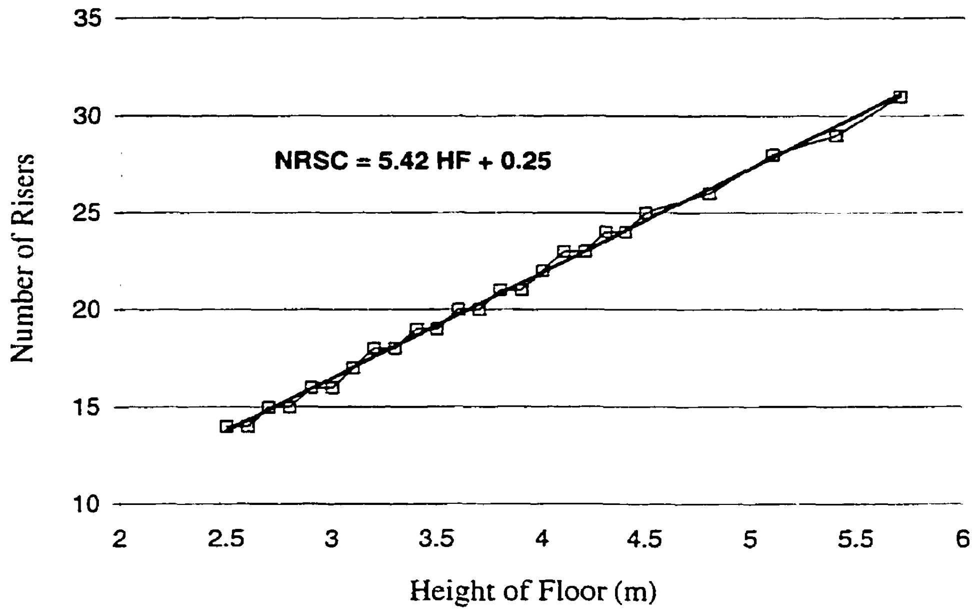


Figure 2.3: Relationship Between Height of Floor and Number of Stair Risers

To facilitate optimum usage of floor area, it is desirable to have columns spaced as far apart as possible. Specifically, longer floor spans are generally more beneficial than shorter spans since they provide greater flexibility for internal layout and unexpected future changes of floor use. In this study, a factor that corresponds to the amount of free-column area for the floor plan (Figure 2.4) is calculated to quantify the flexibility of floor space usage. To this end, the number of column lines between the building perimeter and the core in the a and b directions for the building, NCL_a and NCL_b, are first found as (Figure 2.4),

$$NCL_a = ((D_a - C_a)/(2 \times S_a) - 1)_{\text{Rounded up}} \tag{2.9a}$$

$$NCL_b = ((D_b - C_b)/(2 \times S_b) - 1)_{\text{Rounded up}} \tag{2.9b}$$

Then, the average distances between the column lines, $ADBL_a$ and $ADBL_b$, are found as,

$$ADBL_a = (D_a - C_a) / (2 \times (NCL_a + 1)) \tag{2.9c}$$

$$ADBL_b = (D_b - C_b) / (2 \times (NCL_b + 1)) \tag{2.9d}$$

Finally, the column-free area factor for the floor plan, CFA, is found as (Figure 2.4),

$$CFA = \frac{AreaA \times \sqrt{\dfrac{ADBL_a \times (D_b + C_b)}{2}} + AreaB \times \sqrt{\dfrac{ADBL_b \times (D_a + C_a)}{2}}}{AreaA + AreaB} \tag{2.9e}$$

For a fixed total floor area, Eq. (2.9e) yields larger values of the columns-free area factor CFA for buildings having larger footprints and widely spaced columns, and smaller values for buildings having smaller footprints and closely spaced columns. (As explained in Chapter 3, the CFA value is used to quantify the quality of space for a building).

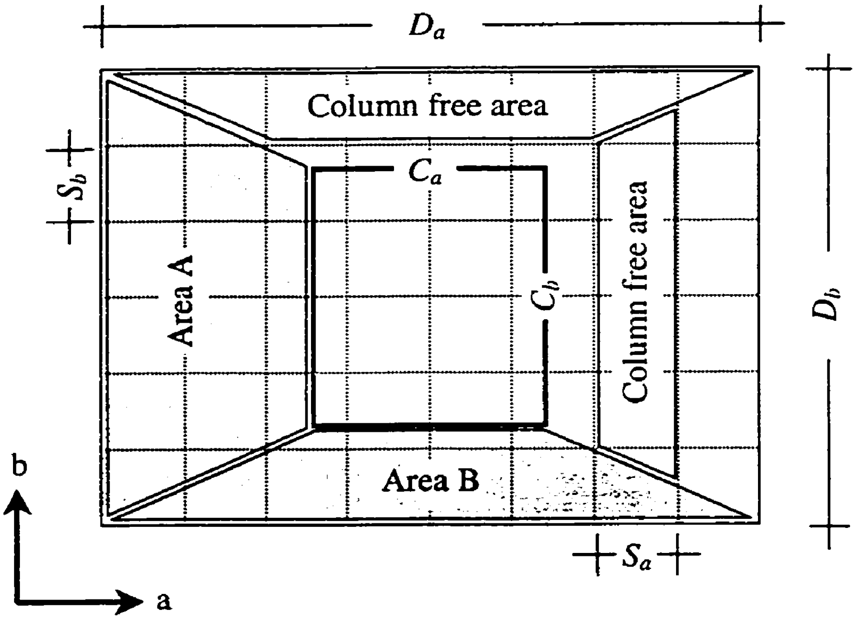


Figure 2.4: Schematic of a Typical Floor Plan

Appendix 2.A - Structural Systems

2.A.1 Vertical Load Resisting Systems

The vertical load resisting systems for high-rise buildings are essentially the same as those for low-rise structures, namely: 1) floors; 2) columns; and 3) load bearing walls. A suitable floor system is an important factor in the overall economy of the building. Some factors that effect choosing the floor system are architectural. For example, shorter floor spans are possible in residential buildings due to the permanent division of area into smaller spaces, while in modern office buildings longer span systems are preferred because their design philosophy leans toward more open and temporarily sub-divisible areas. Hence, in an office building, the structure's main vertical components are generally arranged as far apart as possible so as to leave large column-free areas available for office space planning. Other factors affecting the choice of a floor system are related to its intended structural performance, such as whether it is to participate in the lateral load resisting system. Floor systems can be categorized into three types (Cristiansen et al, 1980):

1. One-way systems: a) one-way slab, b) closely spaced joists
2. Two-way concrete systems: a) flat plate, b) flat slab with drop panel, c) slab and beam, and d) waffle slab
3. Two-way steel systems: a) beam and slab, and b) joists, girders and slab; in both of these systems the slab can be comprised of concrete with or with out steel deck and act as a non- or composite system.

Since one-way systems demand shorter spans and, as discussed earlier, it is desirable for office buildings to have large column-free spaces, this study only considers the two-way floor systems in concrete and steel shown in Figure 2.A.1.

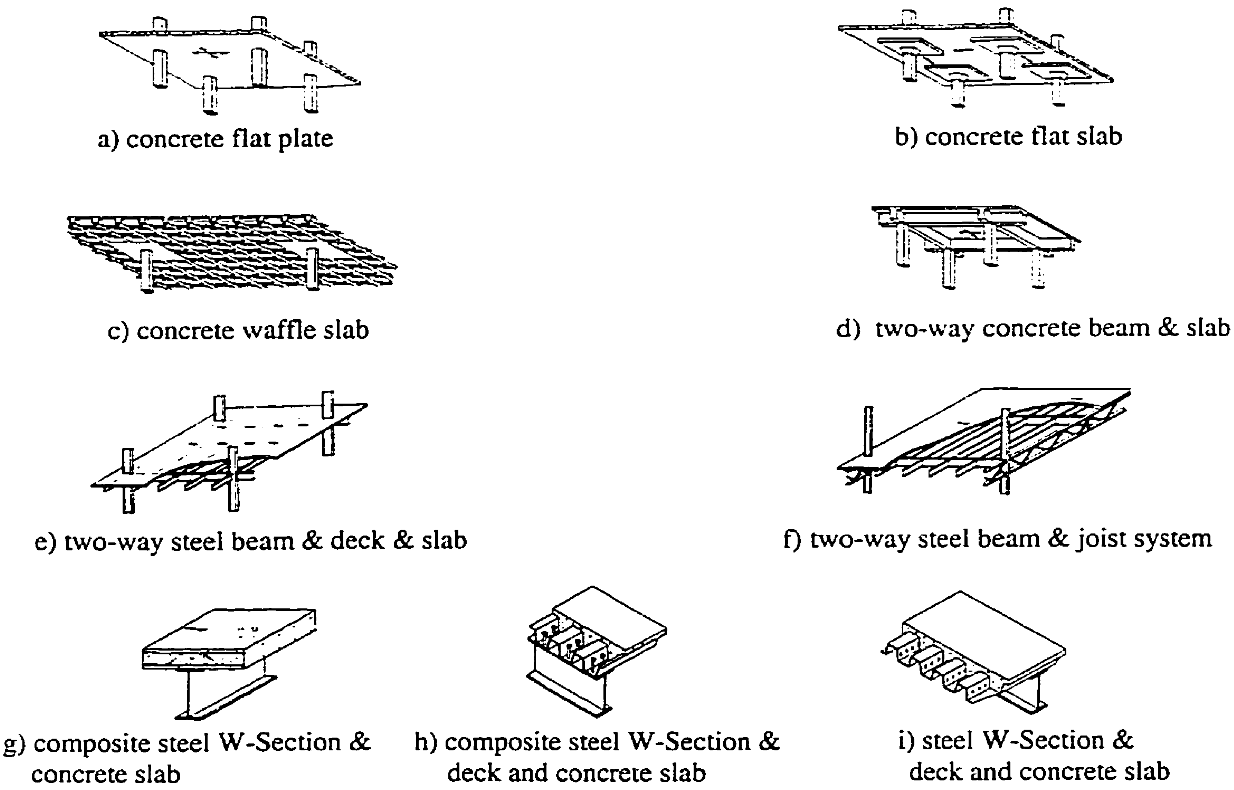


Figure 2.A.1: Floor Systems

In taller buildings, columns and beams are the predominant load bearing systems due to their efficient use of space, versatility as structural systems, and ease of construction. Because of the large gravity loads associated with tall buildings, special care should be taken that major structural elements are not interrupted vertically. Whenever possible, the building's cores, columns and loadbearing walls should not shift laterally from story to story but should be continuous from the roof down to the foundation of the building. Some structural configurations may occur, however, for which all loads do not have direct and continuous paths to the foundations.

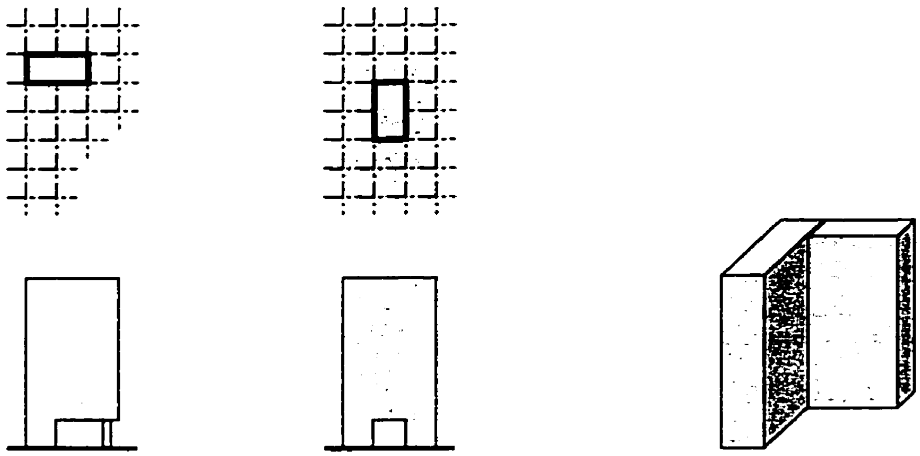
In some cases it is desirable to redistribute vertical loads out towards the perimeter of the building so to improve resistance to overturning. Special spaces in the lower levels of tall buildings, such as auditoriums, lobbies, atriums and mezzanines, often require longer span systems that must interrupt the paths of loadbearing elements from above. This sudden change in the arrangement or spacing of structural elements cause changes in the mass distribution along the height of a building. In extreme cases, a drastic change in the mass distribution requires reconsideration of the basic structural system for a building.

2.A.2 Horizontal Load Resisting Systems

Increasing the height of a building increases its sensitivity to both wind and earthquake forces. The taller the building, the more these forces will dominate the design of the entire structure, and the more attention should be given to the designing of them. Discussed in the following are guidelines important to the design of lateral load resisting systems for high-rise buildings.

Tall, narrow buildings are more difficult to stabilize against lateral forces than broader buildings. More effective bracing mechanisms may be required and bracing elements may assume more importance in the final design of such buildings. The most efficient structure is one in which the forces induced in the members due to lateral and gravity loadings do not greatly surpass those induced by gravity loading alone (Schueller 1977). In areas of great lateral loads (high seismic activity or hurricanes), tall buildings that are non-symmetrical or unbalanced in either weight distribution or the arrangement of bracing elements (Figure 2.A.2a) should be avoided in favour of symmetrical and balance buildings (Figure 2.A.2b).

Parts of a building that have independent mass can be expected to move differently under dynamic loads associated with earthquakes. The leg of an L-shaped building (Figure 2.A.2c), the stem of a T-shaped building, the wide base of a narrow tower, or any other form composed of discrete masses, may react in potentially destructive ways under such load conditions. All such masses should be designed as separate structures, with independent vertical and lateral load resisting systems, to minimize these effects.



a) unsymmetrical and unbalanced building   b) symmetrical and balanced building   c) unsymmetrical building designed as two structures

Figure 2.A.2: Symmetry in Buildings

Buildings with inherently unstable massing should be avoided. Discontinuities in the stiffness of a structure at different levels may lead to excessive deflections or other unfavourable responses to lateral loads. For instance, an open space in the long horizontal span direction at the base of a tall building may produce excessive flexibility at that level. If such a "soft story" cannot be avoided, the addition of special bracing elements at that level may be required.

Tall buildings may interact with winds in unpredictable ways. With buildings of irregular or unusual form, or building sites where adjacent structures or other features may produce unusual air movements, specialized studies of the building's response to local wind pressures and fluctuations may be required.

The conventional arrangements of stabilizing elements used in low-rise buildings may be extended for use in buildings up to 20 to 25 stories in height (Allen and Iano 1995). The same considerations that apply to low-rise buildings apply to taller buildings as well. Stabilizing elements should be arranged so as to resist lateral forces along all major axes of the building. These elements should be arranged in a balanced manner either within the building or at the perimeter, and such elements must be integrated with the building plan of elevation.

Shear walls and braced frames are the stabilizing elements most commonly used in buildings of medium height, due to their structural efficiency. They may be used either separately or in combinations. The use of rigid frames as the sole means of stabilizing structures of medium height is possible, although this may be less than desirable because of the large size of the beams and columns that are generally required. For steel structures, the fabrication of welded joints required for rigid frame behaviour also becomes increasingly uneconomical as the number of connections increases. Rigid frames may also be used in combination with either shear walls or braced frames to enhance the total lateral resistance of a structure.

The proper arrangement of shear walls, diagonal braces, or rigid joints in a structure is crucial to their effectiveness in resisting lateral forces acting on the building. As illustrated in the schematic floor plans in Figure 2.A.3, these elements may be placed within the interior of the buildings or at the perimeter, and they may be combined in a variety of ways. However, they must be arranged so as to resist lateral forces acting from all directions. This is usually accomplished by aligning one set of stabilizing elements along each of the two perpendicular plan axes of a building. Stabilizing elements must also be arranged in as balanced a fashion as possible in relation to the mass of the building (Figures 2.A.3a, b, d and e). Unbalanced arrangements of these elements result in the displacement of the centre of stiffness of the building away from its centre of mass (Figure 2.A.3c and e). Such a condition causes torsional building movements under lateral loads that may be difficult or impossible to control.

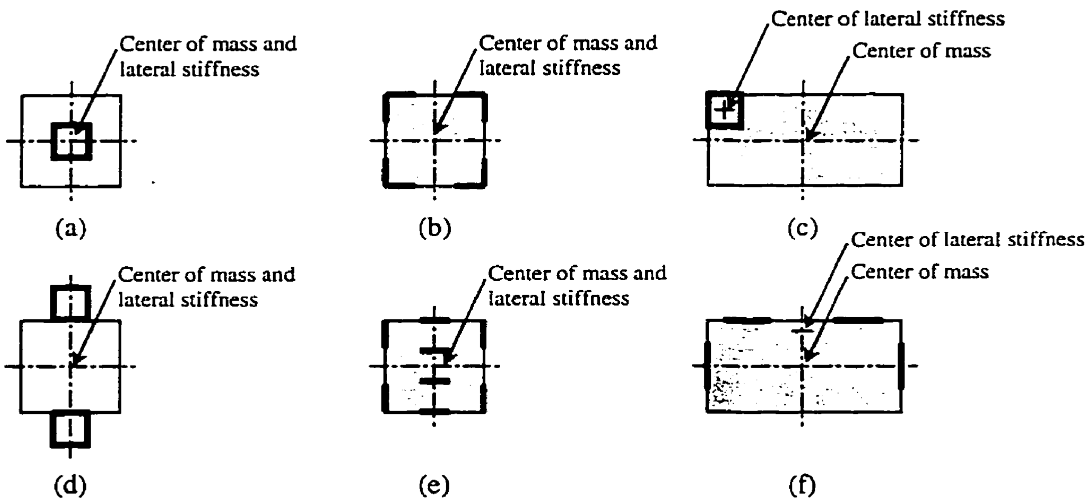


Figure 2.A.3: Arrangement of Stabilizing Elements in Buildings

All buildings must include structural elements designed specifically to resist lateral forces, such as those due to wind and earthquakes. The choice and location of these elements can influence building design in important ways even at the preliminary stage. The three stabilizing mechanisms used in buildings are the rigid frame, the braced frame, and the shear wall. Any one of these can be used to stabilize a building, or they may be used together in a variety of combinations.

The systems shown in elevation and plan view in Figure 2.A.4 are presented in left-to-right order of increasing resistance to lateral forces.

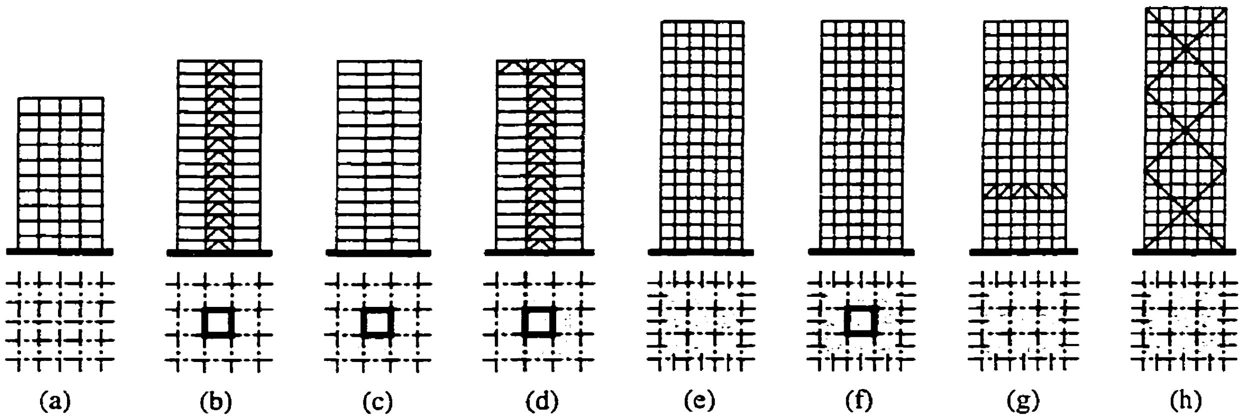


Figure 2.A.4: Schematic Representation of Different Structural Systems

The horizontal load resisting systems in Figure 2.A.4 can be categorized into the following groups (Cristiansen et al, 1980):

1. Moment resistant (rigid) frames (Figure 2.A.4a)
2. Braced frames (Figure 2.A.4b)
3. Shear wall systems (Figure 2.A.4c)
4. Combination systems: braced rigid frame (Figure 2.A.4b);
   rigid frame and shear wall (Figure 2.A.4c);
   braced frame and outrigger trusses (Figure 2.A.4d);
   tube (Figure 2.A.4e);
   tube-in-tube (Figure 2.A.4f);
   tube and belt trusses (Figure 2.A.4g);
   tube and external bracing (Figure 2.A.4h);
   bundled tube (not shown)

Moment Resistant (Rigid) Frames. Rigid frames depend on rigid connections between columns and beams (or slabs) to develop resistance to lateral forces. Rigid frame skeletons generally consist of a rectangular grid of horizontal beams and vertical columns connected together in the same plane by means of rigid joints. Though the least efficient of the three basic stabilizing mechanisms, rigid frames find use in buildings that require relatively modest lateral resistance (e.g., low, broad buildings), or in buildings where the presence of stabilizing walls or braces is undesirable. The frame may be in-plane with an interior wall of the building, or in-plane with the façade. The rigid frame is economical up to approximately 30 stories for steel buildings and up to 20 stories for concrete buildings (Schueller 1977).

Compared to shear wall or braced frame systems, the use of rigid frames may set greater restrictions on the arrangement and sizing of the structural frame. Column spacing often must be reduced, variations or irregularities in column placement may be limited, and the size of columns and depths of beams may need to be increased. The size of the columns and girders at any level of a rigid frame are directly influenced by the magnitude of the external shear at that level (Smith and Coull 1991) and, therefore, they increase in size toward the base of the structure. Consequently, the design of the floor framing system cannot be repetitive as it is in some braced frames. Also, in the lowest stories it is sometimes not even possible to accommodate the required dept of girder within the normal ceiling space. The rigid joints necessary in this system can be easily constructed in steel (at added cost compared to hinge-connections), or in sitecast concrete, where they are formed as a normal part of the construction process. Though possible, rigid joints are difficult to construct in precast concrete and are rarely used.

Rigid frames are often combined with either shear walls or bracing for improved results compared to either system acting alone.

Because of the type of connections between the structural elements, a rigid frame responds to lateral loads primarily through flexure of the beams and columns. This continuous character of the rigid frame is dependent on the resistance of the member connections against any rotational slippage. The load capacity of the frame relies very much on the strength of the individual beams and columns, and its capacity decreases as story height and columns spacing become larger. The lateral deflection of rigid frames is caused generally by two factors:

1. Deflection due to cantilever bending: This phenomenon is known as chord drift, where, in resisting the over-turning moment, the frame acts as a vertical cantilever beam that bends through axial deformation of its fibres. In this case, lengthening and shortening of the columns produce the lateral sway of the frame. This mode of lateral deflection accounts for about 20% of the total drift of structures (Schueller 1977).
2. Deflection due to bending of beams and columns: This phenomenon is known as frame racking, where shear forces cause bending moments to be introduced into columns and beams such that as they bend, the entire frame distorts. This mode of deformation accounts for about 80% of the total sway of the structure; 65% is due to beam flexure and 15% is due to column flexure (Schueller 1997). The curvature of the deflection corresponds to the external shear diagram; the slope of the deflection curve is maximum at the base of the structure, where the largest shear occurs.

Braced Frames. Braced frames are quite effective in resisting lateral forces. They may be constructed from steel or, occasionally, from concrete. The diagonal bracing elements that comprise these systems act similar to shear walls in transferring lateral forces between floors of a building. Diagonal bracing is inherently obstructive to the architectural plan and can pose problems in the organization of internal spaces and access as well as in locating window and door openings. For this reason, bracing is usually concentrated in vertical panels or bents that are located near the centre of the building to cause minimum obstruction while satisfying the structural requirements to resist shear and torque forces on the building. The most efficient, but also the most obstructive, types of bracing are those that form a fully triangulated vertical truss. These include single-diagonal, double-diagonal and K-braced types (Figures 2.A.5a, b, c, and d).

The full diagonal types of braced bent are usually located where passage is not required, such as between elevator, service and stair shafts, which entities are unlikely to be relocated in the lifetime of the building.

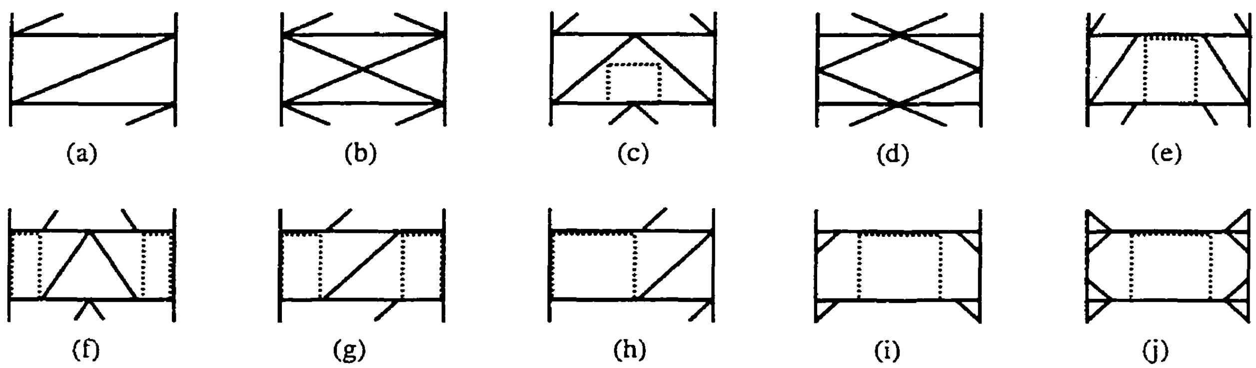


Figure 2.A.5: Different Bracing Types

Other types of braced bents that allow for window and door openings, but whose arrangement cause bending in the girders, are shown in Figures 2.A.5e, f, g, h, i, and j. Because lateral (wind, seismic) loading on a building is reversible, braces can be subjected to both tension and compression forces, but they are generally designed for the more severe case of compression loading. For this reason, bracing systems with shorter braces, e.g., the K-type, may be preferred to full-diagonal types. As an exception to designing braces for compression, the braces in the double-diagonal system are sometimes assumed to buckle in compression and each diagonal is designed to carry in tension the full shear in the panel.

A significant advantage of the fully triangulated bracing types, Figures 2.A.a, b, c, and d, is that the girder moments and shears are independent of the lateral loading on the structure. Consequently, the floor system can be designed for gravity loading alone and, as such, can be repetitive throughout the height of the structure with obvious economic benefit. Generally, the types of braced bent that respond to lateral loading by bending of the girders, or the girders and columns, are laterally less stiff and therefore less efficient, than the fully triangulated braced bent that develops axial forces alone in the members (Smith and Coull 1991).

Shear Walls. Shear walls are extremely effective in resisting lateral forces. They are easily constructed from concrete, masonry or wood and, sometimes in tall buildings, from steel. The superior resistance of shear walls to lateral forces often makes them a good choice in situations where the maximum resistance to lateral forces is required, such as across the narrow dimension of a tall, slender building. Shear walls are commonly integrated into the enclosure of vertical building cores or stair cases. They may or may not carry gravity loads. When shear walls are incorporated into the interior of a building their locations must be coordinated with the building's plan. Shear walls placed at the perimeter of a building can restrict the size, number or arrangement of openings, and this is generally not desirable for proper access and natural lighting for the building.

Shear wall systems can assume a number of geometrical configurations, which may be subdivided into open and closed systems. Open systems are made up of single linear shear wall elements, or a combination of such elements, that do not completely enclose a geometric space. Such shapes are L, X, V, Y, T and H (Schueller 1977). Conversely, closed systems enclose a geometrical space, common forms of which are square, triangular, rectangular and circular cores of buildings. Shear wall systems may be arranged symmetrically or asymmetrically so as to minimize the effect of eccentricity of lateral loads.

The shape and location of shear walls have significant effects on their structural behaviour under lateral loads. A core that is eccentrically located with respect to the building shape has to carry torsion as well as bending and direct shear. Moreover, torsion may even develop in buildings featuring symmetrical shear wall arrangements when the wind loads act on facades of different surfaces texture and roughness (Schueller 1977), or when the building's centre of mass and stiffness do not coincide.

Optimal torsional resistance is obtained with closed core sections. When evaluating core section resistance, however, the tosional rigidity must be reduced to account for door, window and other openings. For maximum performance, shear walls should have a minimum of perforations or openings. In fact, walls having large openings to accommodate mechanical and electrical systems might not be able to carry lateral loads.

Floors acting as horizontal diaphragms transmit lateral loads to the shear walls. If the floors have no major openings, they are generally assumed to be infinitely stiff and the distribution of lateral forces to the shear walls is strictly a function of the geometrical arrangement of the resisting wall systems.

If the resultant of the lateral forces acts through the centre of stiffness for a building, only translation reaction will be generated. The most obvious case in this regard is the symmetrical pure shear wall building (Figures 2.A.3a, b, d, and e). In a rigid frame shear wall building, the shear may be assumed to be resisted completely by the core as a first approximation (Schueller 1977). This is because core lateral stiffness is generally much greater than the lateral stiffness of the frame. If the shear wall arrangement is asymmetrical, the resultant of the lateral forces does not act through the stiffness centeriod of the building, and rotation of the shear walls will occur in addition to translation.

When the loads acting on an individual shear wall have been determined, the next stage of the design process is to determine the corresponding wall stresses. The distribution of stresses in a shear wall is dependent on the shape of the system. If the wall is rectangular in elevation and has a height-to-width ratio greater than five, a close estimate of the axial stresses is given by simple bending theory (Smith and Coull 1991). The same methodology can be extended to coupled shear walls, where the forces induced in the connecting beams can be approximated from the sum of shear flows for the coupled walls.

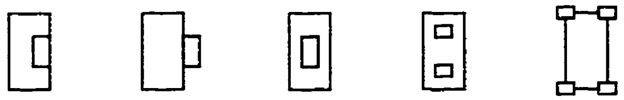
Core Structures. Cores typically take up approximately 20% to 25% of the total floor area of a high-rise building (Allen and Iano 1995). They should be formed as closed elements, approximately square or cylindrical, with openings in the core kept to a minimum.

Core structures are perhaps the systems that are most commonly used to laterally stabilize all but the tallest buildings (Schueller 1977). These structures integrate stabilizing elements into the vertical shafts that house the circulation and mechanical service systems for a tall building. One of the principal advantages of these structures is that interference with the surrounding usable space in the building is minimized. In concrete construction, core walls intended to enclose building service systems can be readily designed to also act as shear walls, in many cases with no increase in size. In steel construction, core structures are usually designed as braced frames.

In buildings with more than one core, the cores should be located symmetrically in the building plan so as to provide balanced resistance under lateral loads from any direction. A single core servicing an entire building should be located at the centre of the building, which typically provides the overall best solution to meet various architectural and structural criteria for office buildings, as indicated in Table 2.A.1 (Allen and Iano 1995).

Simple core structures can be used in buildings as high as 35 to 40 stories (Allen and Iano 1995). The lateral stability of simple core structures can be enhanced with the addition of bracing in the form of "hat" trusses which serve to also engage the perimeter columns of the building in the task of resisting lateral loads, thus significantly improving the overall performance of the building. Albeit, such trusses may influence the design of the building façade or the location of mechanical floors. Columns at the perimeter of the building may also increase in size with this system. These core-interactive structures are suitable for buildings up to approximately 55 stories in height (Allen and Iano 1995).

Table 2.A.1: Characteristics of Core Placements



1 = Best, 5 = Worst	Edge	Detached	Central	Two	Corners
Flexibility of typical rental area	2	1	3	4	2
Perimeter for rental area	4	3	1	1	5
Ground floor high-rent area	3	1	3	4	2
Typical distance of travel from core	4	5	2	1	3
Clarity of circulation	3	4	2	1	3
Daylight and view for core spaces	2	1	5	5	1
Service connection at roof	3	5	1	2	4
Service connection at ground	3	4	2	1	5
Suitability for lateral bracing	4	5	1	1	2
Total	28	29	20	20	27
Overall ranking	3rd	4th	1st	1st	2nd

Shear Core Structures. The linear shear wall system works quite well for apartment buildings in which functional and utilitarian needs are fixed. Commercial buildings, however, require maximum flexibility in layout, calling for large open spaces that can be subdivided by movable partitions. A common solution is to gather together vertical transportation and energy distribution systems, such as elevators, stairs, toilets and mechanical shafts, to form a core or cores depending on the size and function of the building. These cores are then also utilized as shear wall systems to provide the necessary lateral stability for the building.

Cores can be made of steel, concrete, or a combination of both. In a steel framed core, diagonal bracing is used to achieve the necessary lateral stiffness for taller buildings. The advantage of steel framed cores lies in the relatively rapid assemblage of the core using prefabricated members. The concrete core, in addition to carrying loads, completely encloses the space such that no further considerations need to be given to fireproofing. At the same time, the lake of ductility inherent in concrete as a material is a disadvantage when responding to earthquake loading.

Lateral-loads resisting shear core structures may be visualized somewhat as huge beams cantilevering out of the ground, for which bending and shear stresses are similar to those of a box Section beam. Since the core also carries gravity loads it has the advantage of being prestressed by the induced compressive stresses, and thus may not need to be designed for tensile stresses due to bending caused by lateral loads (this is especially true for heavy concrete cores). In addition, the capacity of the core material to resist shear stress is increased in the presence of compressive stresses.

The response of a core structure to lateral loading is dependent on its shape, degree of homogeneity and rigidity, and the direction of the load. At every floor level there are openings in the core, and the amount of continuity provided by the coupling beams determines the behavior of the core. The design must avoid having the core act like an open section that distorts (warps) in its upper portion with no restraint, especially under asymmetrical loading causing twisting.

Frame-Shear Wall Building Systems. Pure rigid frame systems are not practical in buildings higher than 30 stories (Schueller 1977). Thereafter, such systems generally also employ a shear wall of some type within the frame to resist lateral loads. The shear walls are either concrete or trussed-steel bracing . They may be closed interior cores. as around elevator shafts or stair wells, or parallel walls within the building, or they may be vertical façade trusses.

Frame-shear wall systems are classified with respect to their response to lateral loading, which may be one of the following two types: 1) Hinged frame-shear wall systems; 2) Fixed frame-shear wall systems. In the hinged frame-shear wall system the column-girder connections do not take any bending moment, such that the frame only carries gravity loads while the shear walls resist all the lateral loads. For such systems, however, it may not be possible at times to make the shear walls sufficiently strong to resist the lateral forces by themselves alone. In such cases, the fixed frame-shear wall system is used where both shear walls and the rigid frame act together to resist the lateral forces. Here, the lateral deflection of the combined shear wall and rigid frame is obtained by superimposing their individual modes of deformation, as shown in Figure 2.A.6.

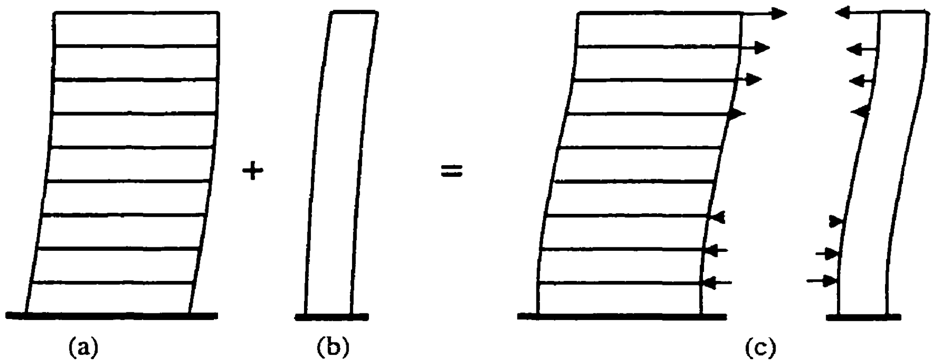


Figure 2.A.6: Frame-Shear Wall Interaction

The rigid frame shear mode deformation is indicated schematically in Figure 2.A.6a. Note that the slope of the deflection curve is greatest at the base of the structure where the maximum shear is acting. The shear wall system is assumed to act as a vertical cantilever beam in bending. The shear wall bending deformation mode is indicated in Figure 2.A.6b. Note that the slope of the deflection curve is greatest at the top of the building, indicating that the shear wall system contributes the least stiffness in this region. The combined frame and shear wall deformation is obtained by superimposing the two separate deflection modes, resulting in the flat S-curve shown in Figure 2.A.6c (Schueller 1977). Because of the different deflection characteristics of the shear wall and frame, the shear wall is pulled back by the frame in the upper portion of the building, and pushed forward near its base. As a consequence, the lateral shear force is carried mostly by the frame in the upper portion of the building and by the shear wall in the lower portion.

It is desirable in tall buildings to proportion the wall and frame components so as to optimize the overall desirable effect of wall-frame interaction. Such an optimization aims to not only achieve significant reductions in lateral deflections and wall moments, but also to cause an approximately uniform distribution of shear over the height of the frame. This then permits the repetitive design and construction of the floor system. To achieve such a well proportioned shear wall-frame structure, a common rule is to size the shear walls in the preliminary stage of design to carry their gravity loading together with two-thirds of the total horizontal loading, leaving the frame to carry one-third of the total lateral load on the building (Smith and Coull 1991).

Flat Slab Building Structures. Flat slab systems consist of solid or waffle-type concrete slabs supported directly on columns, thus eliminating the need for floor framing. This results in minimum story height, an obvious economic advantage. The systems are adaptable to an irregular support layout. Drop panels and/or column capitals are frequently used because of high shear concentrations around the columns. Slabs without drop panels are commonly called flat plates. Some disadvantages of flat slab systems are: a) undesirable large dead load; b) small depth-to-span ratios can cause the appearance of excessive deflection; and c) their relatively short span capability.

Usually for multi-story buildings, flat slab structures rely on the shear walls to provide the necessary lateral stiffness. Albeit, the monolithic character of such concrete structures requires the entire building to react to lateral loads as a unit, and it is not realistic to assume that lateral loads are resisted entirely by the more rigid core or shear wall and that the slabs and columns contribute no resistance at all. In fact, the flat slab itself, though relatively flexible, provides lateral stiffness to the structure because of its continuity with the shear walls. As well, a portion of the slab will act as a shallow beam continuous with the columns such that the behavior of the total structure is similar to that of a core-frame system (e.g., see Figure2.A.6)

Frame-Shear Wall Systems with Belt Trusses. The braced frame becomes inefficient above about 40 stories because excessive bracing is required beyond that point to provide adequate lateral stiffness to the structure. The efficiency of the building structure may be improved by about 30% through the use of horizontal belt trusses that tie the frame to the core (Schueller 1977). The trusses are fixed rigidly to the core and simply connected to the exterior columns. When the shear core tries to bend, the belt trusses act as lever arms that directly transfer axial stresses into the perimeter columns. The columns, in turn, act as struts to resist the lateral deflection of the core. That is, the core fully develops the horizontal shear and the belt trusses transfer the vertical shear from the core to the façade frame. Thus, the building is made to act as a unit that is very similar to a cantilever tube.

The building can have one or several belt truss; the more trusses used, the better the integration of core and façade columns. They should be placed at locations within the building where the diagonal bracing will not interfere with the building's function. The structural principle of employing belt trusses at the top and mid-height of a building seems to be economical in applications up to approximately 60 stories (Schueller 1977).

The stress diagram in Figure 2.A.7 illustrates the relative efficiency of hinging the belt trusses to the perimeter columns rather than fixing them rigidly. If the trusses were to be continuously connected to the columns, the entire system would act as a unit, thus utilizing only a small percentage of the moment-resisting capacity of the core, whose walls are relatively close to the neutral axis of the building. This is indicated by the continuous distribution of stresses shown for the rigid frame in Figure 2.A.7a. On the other hand, belted trusses that are cantilevered from the core and hinged to the perimeter columns better develop the moment resisting capacity of the core while still engaging the exterior columns as in the rigid system (Figure 2.A.7b). In fact, since the hinged shear connections induce no bending moments into the columns, the axial capacity of the columns is increased relative to that for the case of fixed shear connections.

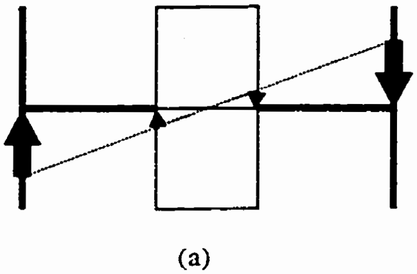


Figure 2.A.7: Stress Distribution in Frame-Shear Wall Systems with Belt Trusses

The response of a core frame building with belt trusses to lateral loading is shown in Figure 2.A.8. This Figure schematically shows the reduction of moment in the shear-core for a one-outrigger system (Figure 2.A.8b) and a two-outrigger system (Figure 2.A.8c) compared to that for a no-outrigger system (Figure 2.A.8a).

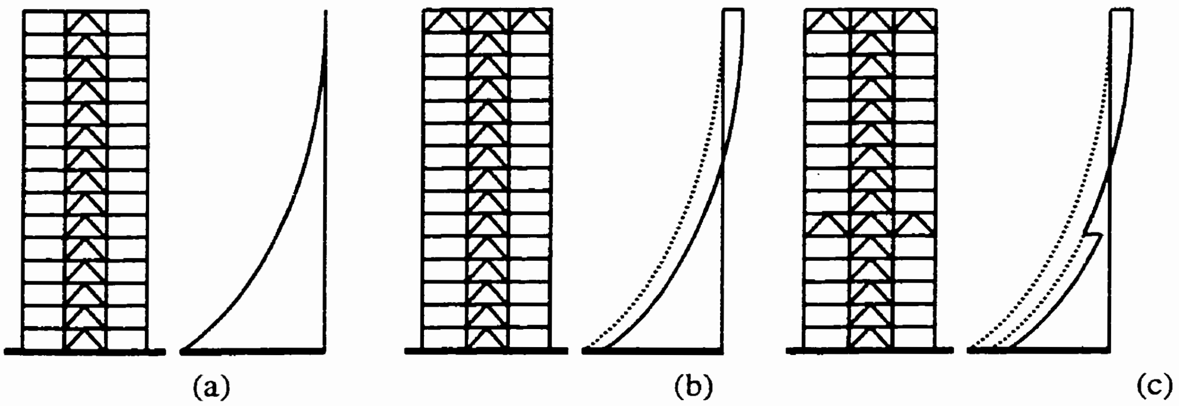


Figure 2.A.8: The Effect of Outriggers on Core Moment

When the frame is hinged to the core of the structure, the core behaves like a cantilever and its top is free to rotate. The frame itself hardly resists any rotation. If the frame is tied to the core by a belt truss, however, any rotation at the top of the system is restricted, since the perimeter columns tie the belt truss down. There is then no bending moment in the columns. The partial fixity provided at the top of the system by the belt truss is reflected in the moment diagram in Figure 2.A.8b. The system no longer acts as a pure cantilever because it is restrained at the top as well as at the bottom. The resulting deflection is a flat S-curve, with a zero moment at a point of inflection above the mid point of the building. The bending moment in the shear wall at the base of the building is less than that for the no-outrigger case in Figure 2.A.8 a. The strength and stiffness of the system is further increased by adding additional belt trusses at intermediate levels within the building. At each truss level the system is restrained from rotating. The fixity provided at these levels pulls the moment diagram back, as shown in Figures 2.A.8c, such that the bending moment at the base of the building is further reduced (along with building sway).

Smith and Coull (1991) studied the optimum location of outriggers by considering hypothetical structures whose outriggers were flexurally rigid. They found that a single outrigger in a one-outrigger system should be located at approximately half height of the building, that the outriggers in a two-outrigger system should be located roughly at one-third and two-thirds height, and that in a three-outrigger system they should be at approximately one-quarter, one-half, and three-quarters height, and so on. Generally for the optimum performance of an n-outrigger structure, the outriggers should be placed at the 1/(n+1), 2/(n+1), up to the n/(n+1) height locations. The Smith and Coull study found that the reduction in core base bending moment is approximately 58%, 70%, 77% and 81% for one-outrigger, two-outrigger, three-outrigger and four-outrigger structures, respectively. Unexpectedly, contrary to a traditional location for outriggers (Shueller 1977), they found that it is structurally inefficient to locate an outrigger at the top of a building. In an optimally arranged outrigger system, the moment carried by any one outrigger is approximately 58% of that carried by the outrigger below. However, if an additional outrigger is placed at the top of the building, it carries a moment that is roughly only 13% of that carried by the outrigger below, which clearly shows the inefficiency of this outrigger location.

Tubular Systems. A relatively recent development in tall building design is the concept of tubular behaviour introduced by Fazlur Khan (Schueller 1977). The tallest buildings currently being constructed are designed as tube structures. In fact, four of the world's tallest buildings are tubular systems: the Hancock Building, Sears Building and Standard Oil Building in Chicago, and the World Trade Center in New York, Figures 2.A.9 a, b, c and d, respectively.

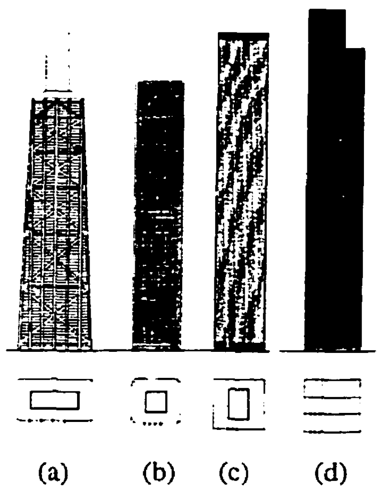


Figure 2.A.9: Four As-Built Tube Structures

In tubular systems, stabilizing elements are located at the perimeter of the structure, leaving the layout of the interior of the building virtually unrestricted by concerns for lateral stability. Tubular systems are so efficient that in most cases the amount of structural material used per square meter of floor space is comparable to that used in conventionally framed buildings of half the size (Schueller 1977)

Tubular design assumes that the façade structure responds to lateral loads as a closed hollow box beam cantilevering out of the ground. Since the exterior walls resist all or most of the wind load, costly interior diagonal bracing or shear walls are eliminated.

The use of rigid frame tubes may effect the size and spacing of framing elements at the perimeter of the building. Beams may need to be deeper and columns may need to be larger and more closely spaced than would otherwise be required. When constructed of steel, the welded joints required in tube systems may be more costly to construct, although construction techniques have been developed that allow for the off-site fabrication of these joints, thus minimizing this disadvantage. The walls of a tube system consist of closely spaced columns around the perimeter of the building that are tied together by deep spandrel beams. This façade structure looks like a perforated wall. The stiffness of the façade wall may be further increased by adding diagonal braces to cause truss-like action (Figure 2.A.9a). The rigidity of a tube is so high that it responds to lateral loading in a way similar to a cantilever beam. As we will see in the following, an exterior tube can resist all of the lateral loads on its own, or it can be further stiffened by adding interior bracing of some kind.

Framed Tube. The framed tube, the earliest application of the tubular concept, was first used in a 43-story apartment building in Chicago in 1961 (Schueller 1977). In this particular tube system, the exterior walls of the building consist of a closely spaced rectangular grid of beams and columns rigidly connected together, which resist lateral loads through cantilever tube action without using interior bracing. Interior columns are assumed to carry gravity loads alone and do not contribute to the lateral stiffness of the building. Stiff floor systems act as rigid diaphragms that distribute lateral forces to the perimeter walls.

Other examples of hollow framed tube buildings are the 83-story Standard Oil Building in Chicago and the 110 story World Trade Center in New York (Figures 2.A.9c and d). Although these buildings have interior cores, they act as hollow tubes because the cores are not designed to resist lateral loads. Such a system possesses excellent lateral stiffness and torsional qualities while retaining flexible interior space layout possibilities. In some framed tube buildings, the façade grid is so closely spaced that it can serve as mullions for the glazing.

It would be ideal in the design of framed tube systems if the exterior walls were to act as a unit, responding to lateral loads in pure cantilever bending. If this were the case, all columns that make up the tube would be either in direct axial tension or compression. The linear stress distribution that would result is indicated by the broken lines in Figure 2.A.10.

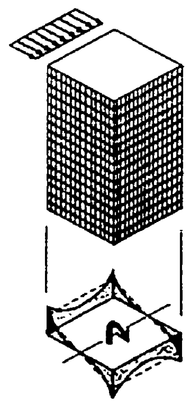


Figure 2.A.10: Stress Distribution for Façade Columns

However, the true behavior of the tube lies somewhere between that of a pure cantilever and a pure frame. Due to the flexibility of the spandrel beams, the sides of the tube parallel to the lateral force tend to act as independent multi-bay rigid frames. This flexibility results in racking of the frame due to shear (shear lag). Hence, bending takes place in the columns and beams. The effect of shear lag on the tube action results in a nonlinear pressure distribution over the column envelope, where the columns at the corners of the building are forced to take a higher share of the load than the columns in between; see solid-line stress distribution in Figure 2.A.10. Furthermore, the total deflection of the building no longer resembles a cantilever beam, as shear mode deformation becomes more significant. However, it has been suggested (Smith and Coull 1991) that for approximate analysis it is reasonable to assume that lateral forces cause shear in web panels parallel to the direction of the lateral load, and axial forces alone in flange columns perpendicular to the lateral load, Figure 2.A.11.

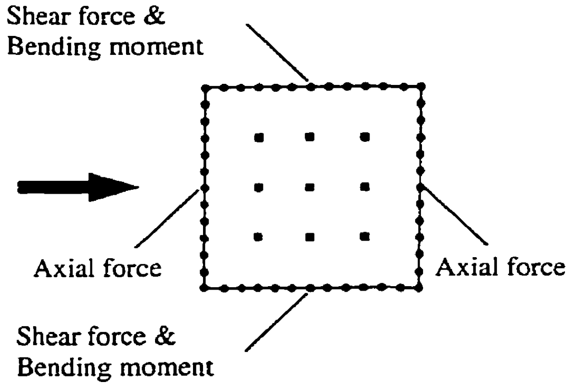


Figure 2.A.11: Forces Induced in the Columns and Spandrel Beams of a Tubular Structure

The shear problem severely affects the efficiency of tubular systems, and many developments in tubular design have attempted to overcome it. The framed tube principle seems to be economical for steel buildings up to 80 stories and for concrete buildings up to 60 stories (Schueller 1977). However, there is no obvious optimum height for this structural system as other sources report that simple tube structures perform very well up to 50 to 55 stories (Allen and Iano 1995).

Braced Tube. The performance of rigid frame tube structures may be enhanced with the addition of belt trusses located on the perimeter of the structure, Figure 2.A.4g. These trusses may be located at various levels on the structure, and they may influence the location of mechanical floors and overall façade design. The framed exterior tube may be stiffened in plane by adding diagonals, Figure 2.A.4h, or it may be stiffened from within the building by adding shear walls or interior cores, Figure 2.A.4f.

Braced frame tubes are very efficient lateral load resisting systems. When built in steel, these structures usually rely on easily constructed bolted connections. The diagonal braces that are an integral part of this system can have a significant impact on the appearance of the building façade, (e.g., see the Hancock Building shown if Figure 2.A.9a).

Tube-in-Tube. Variations on the tube structure are also possible. Tube-in-tube structures, in which perimeter tubes interact with interior rigid cores, may be designed for enhanced structural performance. In fact, the stiffness of a hollow tube system is very much improved by using the core not only for gravity loads but also to resist lateral loads as well. The floor systems tie together the exterior and interior tubes such that they respond to lateral forces as a unit. The response of a tube-in-tube system to wind is similar to that of a frame and shear wall structure. However, the framed exterior tube is much stiffer than a simple rigid frame.

Figure 2.A.6, which was previously used to explain frame and shear wall structures can be viewed to clarify the interaction between the core and tube for tube-in-tube systems. The approach has been used in the 38-story Brunswick building in Chicago, and the 52-story One Shell Plaza building in Houston (Schueller 1977). Moreover, taking the tube-in-tube concept one step further, the designer of a 60-story office building in Tokyo used a triple tube. In this system, the exterior tube alone resists the wind loads, but all three tubes are connected by the floor systems and act as a unit in resisting earthquake loads, a significant design factor in Japan. Finally, bundled-tube structures have been developed that permit great variation in the massing of a structure so as to enhance the overall performance of the structure, (e.g., see the World Trade Center building shown in Figure 2.A.9d).

Except for the braced tube and tube-in-tube systems presented in the immediate forgoing, for which appropriate methods of approximate analysis are not readily available, it is noted all of the structural systems that have been discussed in this Section are accounted for in Chapters 3 and 4 concerning the implementation and application of the proposed computer based method for conceptual design of high-rise office buildings.

Appendix 2.B - Mechanical Systems

2.B.1 HVAC Systems

The HVAC system must fit the overall objectives of the building and, in this sense, must be thought of as an integral part of the building rather than as an appendage to be placed after the architectural design has been fixed. In most cases for tall buildings, the mechanical floors are strategically located over the height of the building so as to reduce the distance between the fan rooms and the boiler and chillers rooms. Generally, the designer must consider a variety of architectural, structural, occupancy, environmental, energy and cost issues for HVAC systems (Baum et at 1980):

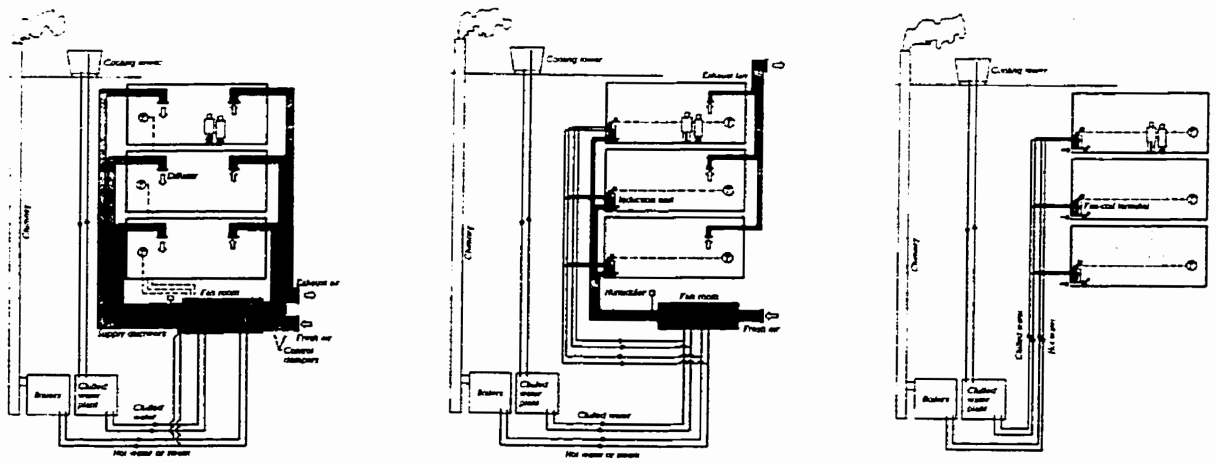
In this study, while taking into account occupancy requirements, architectural and structural constraints, and the internal and external environment, the initial cost, annual operating cost and annual maintenance cost of HVAC systems are used to evaluate the overall optimality of a building. These three costs are functions of the loads applied on the HVAC system, of which there are two types:

1. Heating loads: the amount of energy to be provided to the building by boilers to arrive at a suitable temperature for the occupants during the cold season.

2. Cooling loads: the amount of energy to be taken from the building by chillers to arrive at a suitable temperature for the occupants during the hot season.

There are three different types of HVAC systems for large buildings(Allen and Iano, 1995), as described in the following.

All-Air Systems. In this system, air is conditioned (mixed with a percentage of outdoor air, filtered, heated or cooled, and humidified or dehumidified) at a central source. Supply and return fans circulate the conditioned air through ducts to the occupied spaces of the building. In each individual zone of the building a thermostat regulates the temperature by controlling the heating and cooling coils. In one type of multi-zone system, dampers blend hot and cold air in the fan to send air into the ducts at the temperature requested by the thermostat in each zone. In another type of system, shown in Figure 2.B.1a, reheat coils in the fan room regulate the temperature of the air supplied to each zone. This system offers a high degree of control of air quality and is comparatively simple and easy to maintain, its only drawback being that it requires a large amount of space for ductwork in the vicinity of the fan (however, this problem is not critical in tall building design since there is generally a core area existing to contain such systems).



(a) (b) (c)

Figure 2.B.1: Schematics of Different HVAC Systems

Air and Water Systems. In this system, fresh air is conditioned (heated or cooled, filtered, and humidified or dehumidified) at a central source and circulated in small high-velocity ducts to the occupied spaces of the building, Figure 2.B.1b. Each outlet is designed so that the air discharged from the duct (primary air) draws a much larger volume of room air through a filter. The mixture of primary air and room air passes over a coil that is either heated or cooled by secondary water pipes from the boiler room or the chilled water plant. The primary air (about 15% to 25% of the total airflow through the outlet) and the heated or cooled room air that has been induced into the outlet (75% to 85% of the total airflow) are mixed and discharged into the room. A local thermostat controls the water flow through the coil to regulate the temperature of the air in the space. Condensate that drips from the chilled water coil is caught in a pan and removed through a system of drainage piping. This system is very suitable for exterior spaces of buildings having a wide range of heating and cooling loads where close control of humidity is not required. As well, this system offers good local temperature control and the space required for its ductwork and fans are less than that for all-air systems. However, such systems are relatively complicated to design, install, maintain and manage. They tend to be noisy, inefficient in their use of energy and unable to closely control humidity. In fact, due to these disadvantages, this type of HVAC system is rarely designed or specified at the present time (Allen and Iano 1995).

All-Water Systems. In this system, hot and/or chilled water is pumped through pipes to fan-coil terminals, Figure 2.B.1c. At each terminal, a fan draws a mixture of room air and outdoor air through a filter and blows it across a coil of heated or chilled water and then back into the room. A thermostat controls the flow of hot and chilled water to the coils so as to control the room temperature. The same technique as is used in air-water systems conveys the condensate away from the occupied space. In most installations, the additional volume of air brought from the outdoors is used to pressurize the building to prevent infiltration of outside unconditioned air. The system can be used in buildings having many zones located on exterior walls, such as schools. It does not need a fan room or ductwork and allows control of temperature in different spaces individually. However, as for the air-water system, there is no control over the degree of humidity. As well, the system requires considerable maintenance, most of which must take place in the occupied spaces of the building.

Choosing an HVAC System. Each of the three HVAC systems described in the foregoing has its pro's and con's in terms of needed space and control over temperature and/or humidity in the various zones of a building. As mentioned, an all-air system needs much more space compared to the other two systems but it offers excellent control of interior air quality. Its central air-handling equipment can be designed for precise control of fresh air, filtration, humidification, dehumidification, heating, and cooling. When the outdoor air is cool, an all-air system can switch to an economizer cycle, in which it cools the building by circulating a maximum amount of outdoor air. Unlike the other systems, all-air systems concentrate maintenance activities in unoccupied areas of the building because there are no water pipes, condensate drains, valves, fans, or filters outside the mechanical equipment rooms.

Alternatively, air-and-water and all-water systems require less space and offer better individual control of temperature in the occupied spaces than some all-air systems. However, they are inherently more complicated and much of their maintenance work must carried out in occupied spaces of the building. For these reasons, an all-air system is generally the most suitable HVAC system for high-rise office buildings, and is the only system considered hereafter in this study.

Major Components in All-Air HVAC Systems. The designer must consider the following major components involved in the design of an all-air HVAC system (Allen and Iano 1995): boilers and chimneys; chillers; cooling tower; fan room; outdoor fresh air and exhaust louvres; and vertical and horizontal supply and return ducts, supply diffuser and return grills.

Horizontal ducting is usually concealed between a false ceiling and the ceiling. As shown in Figure 2.B.2, the wiring and ductwork share the above-ceiling space with lighting fixtures and sprinkler piping, which requires careful planning. Generally the lowest layer, about 200mm thick, is reserved for the sprinkler piping and lighting fixtures. Lighting fixture selection plays an important role in determining the thickness of this lower layer because some types of lighting fixtures require more space than others. The HVAC ducts, which are usually 200 to 250 mm deep, run above the lower layer and just below the beams and girders for the floor system above.

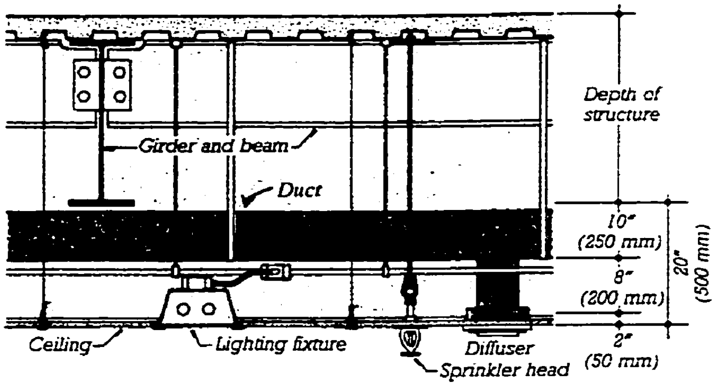


Figure 2.B.2: Schematic of Mechanical/Electrical Assembly in False Ceiling

Adding about 50 mm to account for the thickness of the suspended ceiling, it is generally the case that a minimum height of about 450 to 500 mm must be added to the thickness of the floor system in a typical building to allow for mechanical and electrical services. This causes the depth of the ceiling-floor assembly in the average tall office building to be about 1150 mm.

Other bigger components of the HVAC system cannot be concealed within the floors due to their size and demand their own special place in the building. The cooling tower is usually placed on top of the roof and a fan room is located on each floor within the core area. In fact, the fan room(s) may be located anywhere in the building, as shown in Figure 2.B.3.

The boilers and chillers for the HVAC system require special areas separate from the occupied spaces of the building due to their excessive noise. A boiler room for a large building normally contains at least two boilers, so that one may be in service while the other is being cleaned or repaired. The boiler room may be placed anywhere in a building, and common locations are in the basement, a mechanical room on grade, a mechanical floor, or on the roof. To reduce needed space, it is helpful to locate the boiler room next to the chilled water plant. The two facilities are often combined in the same space on a mechanical floor. The ceiling height in a chilled water plant varies from a minimum of 3.7 m for a building of a moderate size to a maximum of 4.9 m for a very large building, and the total space for the boiler room and chilled water plant is almost 4% of the total floor area for a large building (Allen and Iano 1995).

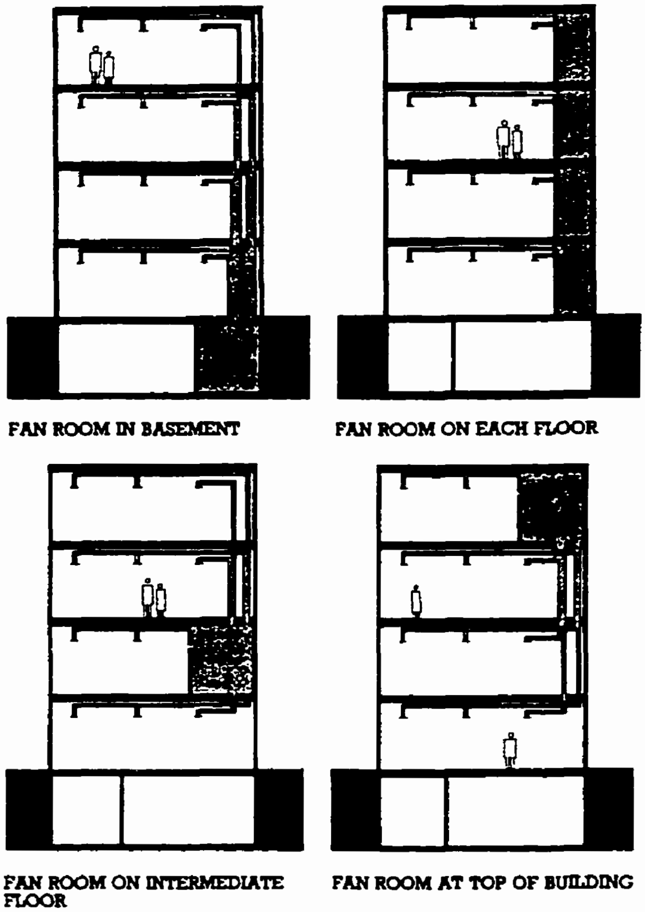


Figure 2.B.3: Different Locations for Fan Room(s)

The maximum vertical "reach" of a fan room is approximately 25 stories up and/or down; however, more typically, fan rooms are located so that none need to circulate air more than 11 to 13 stories in each direction (Allen and Iano 1995). Multiple fan rooms distributed throughout the building are often desirable because they allow the building to be zoned for better local control and tend to reduce the total volume of ductwork in the building. It is often advantageous to have a separate fan room for each floor of a building because such an arrangement saves floor space by eliminating most or all of the vertical runs of ductwork. The space on each floor occupied by the fan room is approximately 2.7% of the floor area. For an HVAC system having but a few fan rooms that serve the entire building, the total area needed for the fan rooms is 2.7% of the total floor area of the building (Allen and Iano 1995).

2.B.2 Elevator Systems

Because of its many complexities, an elevator system is usually designed by an elevator consultant or the engineering department of an elevator manufacturer. Discussed in the following are guidelines for the preliminary determination of the number of elevators needed and the allocation of corresponding space in the building. It is first noted that vertical elevator systems have a severe impact on the design of a building. Secondly, as elevators become extremely expensive as the height of a building increases, it is prudent to accurately estimate their cost a priori in order to arrive at an overall optimal conceptual design of the building. The following are the constraints and costs to be considered for the design of the vertical elevator system for a high-rise building: architectural and structural constraints (for placement within the core); initial cost; and annual operating cost (maintenance and energy).

Table 2.B.1 presents the minimum requirements for the arrangement of elevators in an office building (i.e., number of elevators and appropriate size of the cars). In very tall buildings, the number of shafts can be reduced somewhat by grouping together express and local elevators. Local elevators in high and low zones of the building can even run in the same shaft to save floor space. In some buildings, two-story lobbies served by two-story elevators can reduce the number of shafts by as much as one third. This study, however, will only consider regular use of elevators over the entire height of the building.

Table 2.B.1: Minimum Number of Elevator Shafts and Elevator Dimensions

Number of Elevators	Capacity of Elevator (lb)	Inside Car Dimensions	Inside Shafts Dimensions
1 per 3250 m^2 of area served, plus 1 service elevator for 24,600 m^2 of area served	3000	2032 x 1448 mm	2540 x 2261 mm

From Table 2.B.1, it is clear that the number of elevators in a tall building is not a function of the height of the building but, rather, of its total floor area. This is because the speed of elevators is increased as the height of building increases, rather than increasing the number of elevators, which demands less space on each floor. Walking distance from the elevator lobby to any location on a floor in an office building should not exceed 45m; the minimum width of an elevator lobby serving a single bank of elevators should be 2.45 m; while the minimum width for a lobby with banks of elevators on both sides is 3m (Allen and Iano 1995). For most buildings, including very tall ones, the most widely used elevator type is an electric traction elevator having its machine room at the top of the shaft.

Stair Cases. While stair cases are not part of the mechanical systems for a building, it is appropriate to mention the rules that govern their design here, just after discussing vertical elevator systems.

Stair width and exit discharges widths are based on the occupant load of the largest single floor. Occupant loads do not accumulate from one floor to the next, except at the floor of exit discharge for people who converge there from adjacent floors (Allen and Iano 1995). The minimum numbers of stair cases and exits required by NBCC (National Building Code of Canada 1990) are presented in Table 2.B.2. Based on NBCC guidelines, the occupancy load for an office building is estimated to be 9.3m^2 per person, and the minimum width of each stair should be at least 9.2 mm per number of persons assigned to that stair case and not less than 1.1 meter.

Table 2.B.2: Minimum Number of Stair Cases and Exits

Occupancy Load per Story	Number of Stairs Cases and Exits
500 Persons or fewer	2
501 to 1000 persons	3
More than 1000 persons	4

Chapter 3

Computer-Based Conceptual Design for High-Rise Buildings

3.1 INTRODUCTION

It is proposed in this study that the optimal conceptual design of the major systems for a high-rise building is effectively done through the following three objective criteria: 1) ***minimize initial capital cost***, which consists of the cost of land, structure, façade (cladding and windows), HVAC and elevator systems, lighting, and finishing (painting, carpets, etc); 2) ***minimize annual operating cost***, which consists of maintenance and upkeep costs, the cost of energy consumed per year by the HVAC, elevator and lighting systems, and annual property taxes; and 3) ***maximize annual revenue income***, which is quantified by accounting for the impact that flexibility of floor space usage and occupant comfort level has on lease/rental income. The conceptual design process to achieve these objectives is controlled by multiple constraints concerned with the feasibility, functionality and performance of the building. For this study, explicit constraints are imposed on the building footprint dimensions and height to satisfy available land restrictions and zoning regulations, on the available lease office space to meet anticipated occupancy demands, on the service core area to meet lateral bracing and vertical service requirements, on the distance between the building perimeter and the service core to meet horizontal occupancy requirements, and on the building aspect ratio and slenderness ratio to ensure that designs are compliant with accepted office space layout principles and structural stability requirements. Further implicit constraints are imposed by the limits that are placed on the values of the design variables for an office building, such as the restrictions placed on the type and number of different structural systems, floor systems, cladding types, window types, window ratios and floor plan layouts that may be considered for the design of the building. Additional implicit constraints are imposed by rules of good design practice that ensure architectural, structural, mechanical and electrical systems are feasible and practical.

Recalling the multitude of structural, mechanical and electrical systems discussed in Chapter 2, one can see that optimizing a high-rise office building is extremely complex, and that sometimes the input may seem unmanageable. In fact, generating the best possible design concepts for a building while considering a variety of competing criteria requires the use of numerical algorithms capable of multi-criteria optimization. In this regard, the relatively recent development of search and prediction engines such as Genetic Algorithms (GA's) has created a unique opportunity to solve complex multi-criteria optimization problems. Studies to date have shown that such adaptive search techniques with emergent solution characteristics provide a computing paradigm that is well suited to the complicated and unstructured nature of the conceptual design process (Grierson 1997). The basic features of GA's are briefly elaborated upon in Appendix 3.C. This study proposes to employ a Multicriteria Genetic Algorithm (MGA) for solution of the multi-objective optimization problem posed by the conceptual design of a high-rise office building.

3.2 MULTI-CRITERIA OPTIMIZATION

As building design problems generally have several to many conflicting and non-commensurable criteria, the designer must look for good compromise designs by trading off performance between the various requirements. Multi-criteria optimization offers a flexible approach for the designer to treat this decision-making process in a systematic way.

The two general approaches to solving multi-criteria optimization problems are 'preference' and 'non-preference' methods. The preference method makes use of explicit information about the relative importance of the different objective criteria in order to identify a best overall solution. A difficulty with this approach is that it is not always possible to assess the relative weightings of the different objective criteria so as to achieve a single (combined) criterion objective. The non-preference method makes no assumptions about the relative importance of the different objective criteria, but, instead, identifies a field of solutions that are all considered to be of equal rank in the sense that no one solution is better than any other solution in the field for all objective criteria. A difficulty with this approach is that the number of these non-dominated solutions is often quite large.

In the absence of specific information about preferred relative weighting of costs and revenues for office buildings, non-preferential optimization is adopted in this study for solution of the multi-criteria conceptual design problem to minimize capital cost, minimize operating cost and maximum income revenue. The basic principles of the non-preferential approach, referred to in the literature as 'Pareto' optimization, are described in Appendix 3.D.

The multi-criteria optimization problem posed by this study for the conceptual design of a high-rise office building is concisely stated as,

Minimize:	*{Capital Cost, Operating Cost, 1/(Revenue Income)}*	(3.1a)
Subject to:	*Explicit Constraints ; Implicit Constraints*	(3.1b)

Note that minimizing the inverse function *1/(Revenue Income)* is equivalent to maximizing revenue income, as desired. The explicit functional forms of the objective and constraint functions in Eqs. (3.1) are first developed in the following sections. Then described is the multi-criteria genetic algorithm (MGA) and overall computational procedure employed by this study to solve the problem posed by Eqs. (3.1)

3.2.1 Capital Cost

The assessment of design alternatives at the conceptual stage of design involves comparison of estimated costs. In general, cost estimates can be produced in increasing level of detail and accuracy by the following approaches:

1. Use unit area cost indices published by reputable organisations (e.g., Means manuals (1999)).
2. Use unit volume cost indices for assemblies, also published by reputable organizations (e.g., Means manuals (1999)).
3. Interface the computer-based design system to a cost-analysis software package (such as *Precision Estimating* (1999) by Timberline Software Corporation) to

perform detailed cost estimates of the design alternatives based on material and labour estimates.

4. Also perform life-cycle cost analysis of design alternatives so that, in addition to construction costs, cost factors such as taxes, mortgage and inflation rates, maintenance and energy costs, in addition to revenue income, are also taken into account.

The first and second cost estimation methods noted above are initially employed, based on cost data extracted from Means Manuals (R.S. Means, 1999), as the means to identify the Pareto optimality of different design alternatives. The fourth cost estimation method noted above is subsequently used to account for life-cycle costing so as to estimate the potential profitability of Pareto-optimal designs over time. The effect that different construction materials have on the duration of construction is neglected when estimating costs since it can be argued that the overall project times for both steel and concrete building construction are very similar (Glover 1991). Furthermore, it is assumed that no interest is accumulated on borrowed money during the construction period, i.e., that life-cycle costing only commences upon completion of the project.

The calculation of initial capital cost at the time of building construction accounts for the cost of land and that of estimated structural (floors, columns, lateral load resisting system, and stair cases), mechanical and electrical (HVAC, elevators, lighting and power outlets) systems found through corresponding approximate analyses, in addition to the cost of the building exterior envelope (facade and roofing) and interior environment (finishing and partitioning), i.e.,

Capital Cost = Cost{Land, Floors, Columns, Lateral load system, Stairs, Façade, Roof, Finishing, Partitions, HVAC, Elevators, Lighting} (3.2)

In Appendix 3.A, the capital costs of the individual components in Eq. (3.2) are expressed as explicit functions of the parameters and variables defined in Section 2.3 of Chapter 2 (the reader is also encouraged to refer to the Notation list at the beginning of this study as the written definitions of these parameters and variables are not repeated in Appendix 3.A for the sake of brevity). The total capital cost of a particular conceptual design of an office building is taken by this study to be the sum of the capital costs of the individual building components described in Sections 3.A.1 to 3.A.10 of Appendix 3.A, plus 6% for engineering fees and 25% for contract fees broken down as 10% general requirements + 5% overhead + 10% profit (Mean's Manuals 1999), plus the cost of land.

3.2.2 Annual Operating Cost

The calculation of annual operating cost (after completion of building construction) accounts for the annual cost of energy consumed, maintenance work done and property taxes, i.e.,

$$Operating\ Cost = Cost\{\ Energy,\ Maintenance,\ Taxes\ \} \tag{3.3}$$

Where: the cost of energy is a function of the energy consumed by the HVAC, elevator and lighting systems, as well as by electrical office equipment (which, in turn, is a function of the lease office space); the cost of building maintenance work is a function of the upkeep costs for the HVAC, elevator and lighting systems, and the cleaning and upkeep costs for the building; and the cost of property taxes is a function of the tax rate (as defined by local location information) and the building value.

Refer to Appendix 3.B for a description of the operating costs of the individual components in Eq. (3.3). The total annual operating cost for any particular conceptual design of an office building is calculated by this study as the sum of the annual costs identified in Sections 3.B.1 to 3.B.3 of Appendix 3.B.

3.2.3 Annual Revenue Income

The calculation of annual income revenue after completion of building construction is premised on the concept that higher quality of office space commands higher lease rates, and that income revenue can be quantified in terms of quality of office space and building lease rates, i.e.,

$$\textit{Income Revenue} = \textit{Revenue}\{\textit{Space Quality, Lease Rate}\} \tag{3.4}$$

The functional forms for space quality and lease rates employed by this study are developed in the following.

3.2.3.1 Quality of Space

The space quality term in Eq (3.4) is taken to be a function of the flexibility of floor space usage, as defined by the extent of column free area, and the comfort level of the occupants, as defined by the ratio of floor area benefiting from natural lighting to the total rentable floor area. The column free area is defined by the *CFA* factor found through Eq. (2.9e) developed in Section 2.3.3 of Chapter 2. Occupant comfort is defined by the area shown in grey in Figure 3.1 that benefits from natural lighting for a window ratio *WIR* =

100%; the depth of natural light penetration is considered to be twice the clear height h_{cle} of the story (Reid 1990).

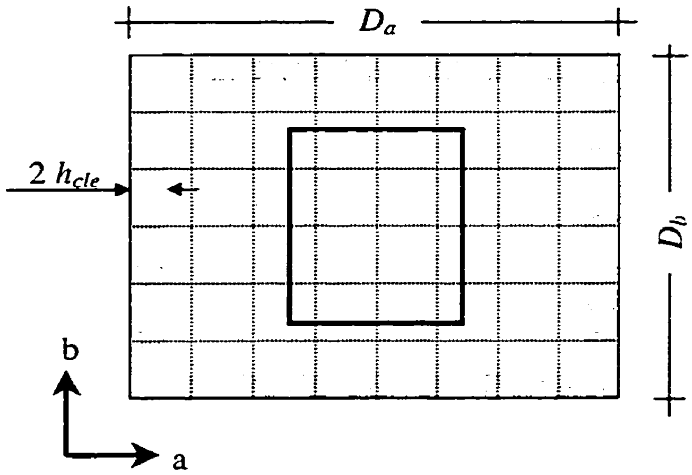


Figure 3.1: Schematic of Floor Area Benefiting from Natural Lighting

This study assumes equal importance of both floor flexibility and comfort level to determine the quality of space for a building. To this end, the maximum and minimum values for floor flexibility and comfort level are found through investigating all possible feasible conceptual designs that can be formed as combinations of the primary design variables given in Table 2.2. These extreme values are then used to normalize all floor flexibilities and comfort levels between 1 and 10, where 1 represents the lowest quality and 10 the highest quality. The normalized floor flexibility and comfort level values for a building are found as,

$$Floor\,flexibility = 1 + 9 \times \frac{CFA - 9}{28} \tag{3.5a}$$

$$Comfort\ level = 1 + 9 \times \frac{\dfrac{4 \times h_{cle} \times (D_a + D_b) - 16 \times h_{cle}^{\ 2}}{D_a \times D_b - C_a \times C_b} \times WIR - 0.05}{0.85} \tag{3.5b}$$

Figure 3.2 demonstrates the normalized floor flexibility vs. comfort level relationships for about six thousands randomly chosen designs. The quality of space for the building is taken to be the product of the floor flexibility and comfort level given by Eqs (3.5), i.e.,

$$\textit{Space Quality} = \textit{Floor flexibility} \times \textit{Comfort level} \tag{3.6}$$

Eq. (3.6) yields bounding values of 3.5 and 37 to define the minimum and maximum quality of space for all possible feasible conceptual designs that can be found through combinations of the primary design variable (alpha-numeric) values given in Table 2.2 (see Figure 3.2).

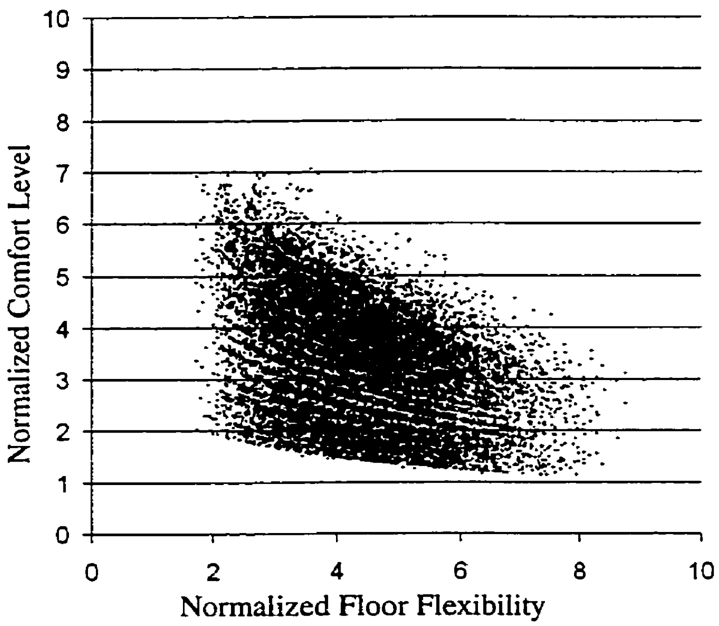


Figure 3.2: Floor Flexibility vs. Comfort Level

3.2.3.2 Annual Lease Rates

The lease rate term in Eq (3.4) is a function of the building location and the demand for office space (as defined by industry). The annual lease rate (*LR*) for any given building design is found by a linear mapping between local lease rates and space quality as given by Eq.(3.7), and is given by (see Figure 3.3),

$$LR = LR_{min} + (LR_{max} - LR_{min}) \times \frac{SQ - 3.5}{37 - 3.5} \tag{3.7}$$

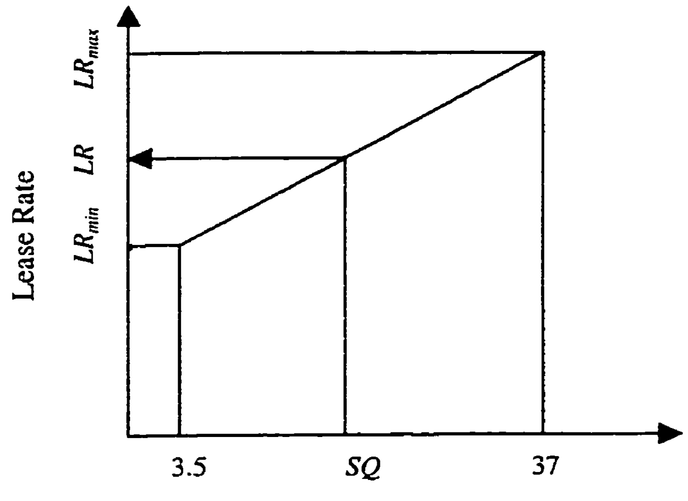


Figure 3.3: Space Quality vs. Lease Rate

3.2.3.3 Total Annual Revenue Income

The total rentable floor area and the annual lease rate define the annual revenue income as,

$$\textit{Revenue Income} = RNF \times (D_a \times D_b - C_a \times C_b) \times LR \tag{3.8}$$

where *RNF* is the rentable number of floors, and D_a, D_b, C_a, and C_b are the building and core dimensions in the *a* and *d* directions, respectively. It is noted that assumed in this study is that the occupancy rate does not vary from one design to another and, therefore, that it is reasonable to take revenue income calculated through Eq.(3.8) for an occupancy rate of 100% as the basis to compare different designs. However, in order to establish the potential profitability of each individual design over time it is necessary to account for more realistic occupancy rates that vary over time (see Section 3.3).

Table 3.1: Ranges of Primary Variable Values for the Conceptual Design of Office Buildings

Index	*ST*	*BT*	*CFT*	*SFT*	S_a,S_b (m)	NS_a NS_b	NTS_a, NTS_b (m)	*DCDD*	*CDF*	*WIT*	*WIR* %	*WAT*
0	([1]c) [3]Rigid frame	K&K	Flat plate	Steel joist & beam	4.5	3	2	*a*	0.250	Standard	25	Pre-cast concrete
1	(c)Rigid frame & shear wall	K&X	Flat slab	Com. beam & [5]CIP slab	5.0	4	3	*b*	0.329	Insulated	30	Metal siding panel
2	(c)Framed tube		Slab & beam	W & com. deck & slab	5.5	5	4		0.407	Standard [6]HA	35	Stucco wall
3	([2]s)Rigid Frame		Waffle slab	Com. beam, deck & slab	6.0	6	5		0.486	Insulated HA	40	Glazed panel
4	(s)[4]Frame & bracing				6.5	7			0.564		45	
5	(s)Rigid frame & bracing				7.0	8			0.643		50	
6	(s)Frame & (c)shear wall				7.5	9			0.721		55	
7	(s)Rigid frame & (c)shear wall				8.0	10			0.800		60	
8	(s)Frame, bracing & outriggers				8.5						65	
9	(s)Framed tube				9.0						70	
10					9.5						75	
11					10.0						80	
12					10.5						85	
13					11.0						90	
14					11.5						95	
15					12.0						100	

ST = Structural type (10 choices); *BT*= Bracing type (2 choices); *CFT* = Concrete floor type (4 choices); *SFT* = Steel floor type (4 choices); S_a, S_b = span distances between *columns along the building width a and length b (16 choices in each direction); NS_a, NS_b = number of column bays along the building width a and length b (8 choices in each direction); NTS_a, NTS_b = number of perimeter tube column spans within S_a and S_b; DCDD = direction of the core dimension to be designed first (2 choices); CDF = ratio of the core dimension to the overall length of the building in the same direction (8 choices); WIT= Window type (4 choices); WIR= Window ratio (16 choices); WAT = Wall cladding type (4 choices); [1]c = Concrete; [2]s = Steel; [3] Rigid frame = framework participates in carrying lateral loads; [4] Frame = framework does not participate in carrying lateral loads; [5] CIP = cast-in-place concrete; [6] HA = heat absorbing.*

Table 3.2: Binary Representation of Primary Design Variables

Base-10 Index	Base 2 Value											
	ST	*BT*	*CFT*	*SFT*	S_a, S_b (m)	NS_a NS_b	NTS_a, NTS_b (m)	*DCDD*	*CDF*	*WIT*	*WIR* %	*WAT*
0	0000	0	00	00	0000	000	00	0	000	00	0000	00
1	0001	1	01	01	0001	001	01	1	001	01	0001	01
2	0010		10	10	0010	010	10		010	10	0010	10
3	0011		11	11	0011	011	11		011	11	0011	11
4	0100				0100	100			100		0100	
5	0101				0101	101			101		0101	
6	0110				0110	110			110		0110	
7	0111				0111	111			111		0111	
8	1000				1000						1000	
9	1001				1001						1001	
10					1010						1010	
11					1011						1011	
12					1100						1100	
13					1101						1101	
14					1110						1110	
15					1111						1111	

3.2.4 Design Constraints

The implicit and explicit constraints in the conceptual design optimization problem posed by Eqs. (3.1) ensure the feasibility, functionality and performance of the conceptual design. For this study, explicit constraints are imposed on the building footprint dimensions D_a , D_b and height H to satisfy available land restrictions and zoning regulations, on the available lease office space $(D_a \times D_b)$ - $(C_a \times C_b)$ to meet anticipated occupancy demands, on the service core area $C_a \times C_b$ to meet lateral bracing and vertical service requirements, on the distances D_a - C_a and D_b - C_b between the building service core and perimeter to meet horizontal occupancy requirements, and on the building aspect ratio D_a/D_b and slenderness ratio H/D_a (assuming $D_a < D_b$) to ensure that designs are compliant with accepted office space layout principles and structural stability requirements, respectively, i.e.,

$$D_a \leq a_{max} \ ; \quad D_b \leq b_{max} \ ; \quad H \leq H_{max} \tag{3.9a,b,c}$$

$$(D_a \times D_b) - (C_a \times C_b) \geq A_{req} \ ; \quad C_a \times C_b = Percentage(D_a \times D_b) \tag{3.9d,e}$$

$$D_a - C_a \geq 2(CPD_{min}) \ ; \quad D_b - C_b \geq 2(CPD_{min}) \tag{3.9f,g}$$

$$D_a/D_b \geq (D_a/D_b)_{Lower} \ ; \quad H/D_a \leq (H/D_a)^{Upper} \tag{3.9h,i}$$

where a_{max} and b_{max} = maximum allowable building footprint dimensions, H_{max} = maximum height permitted for the building, A_{req} = minimum required lease office space for the building, $Percentage(D_a \times D_b)$ = fixed percentage of building footprint area assigned as service core area, CPD_{min} = specified minimum core-perimeter distance, and $(D_a/D_b)_{Lower}$ = minimum aspect ratio and $(H/D_a)^{Upper}$ = maximum slenderness ratio permitted for the building (assuming $D_a < D_b$).

Implicit constraints are additionally imposed on the conceptual design process by the limits that are placed on the possible values that the primary design variables may take on for an office building. In this regard, Table 3.1 (same as Table 2.2) lists the ranges of possible primary variable values adopted by this study for the design examples presented in Chapter 4; i.e., the conceptual design of an office building may be selected from among 10 different structural types, 2 different bracing types, 4 different floor types for concrete structures, 4 different floor types for steel structures, 4 different window types, 16 different window ratios, 4 different cladding types, a large number of different regular-orthogonal floor plans having from 3 to 10 column bays with span distances of 4.5 to 12 meters in the length and width directions for the building, from 2 to 5 times more column bays on the perimeter of framed tube structures than the interior of the building, and up to 8 different core dimensions in each of the length and width directions for the building.

Further implicit constraints are imposed by rules of good design practice that ensure architectural and structural layouts are feasible and practical. For example, one rule is that there must be at least two columns on each side of the service core for braced structural systems. Other rules ensure that particular types of floor systems are only matched with certain types of structural systems, that particular types of bracing are used in certain places to ensure proper access to the service core area, and that the distances between perimeter tube columns are not too small or too large.

Even though the number of constraints is significant, the typical ranges of variable values for an office building still allow for a large number of viable conceptual designs.

In fact, The data in Table 3.1 allows for more than 11.5×10^9 different conceptual design scenarios (albeit, many are infeasible).

3.2.5 Multi-Criteria Genetic Algorithm (MGA)

A design is Pareto-optimal for the multi-criteria optimization problem posed by Eqs. (3.1) if there exists no other feasible design satisfying Eqs. (3.1b) which dominates it for all three cost-revenue objective criteria. The explicit constraints in Eqs. (3.1b) are defined by Eqs. (3.9), while the implicit constraints are defined by the limits placed on the values of the primary design variables in Table 3.1 and by rules of good design practice. Pareto-optimal design satisfying Eqs. (3.1) define the trade-off relationships between the competing cost-revenue objective criteria.

The problem posed by Eqs. (3.1) is complex and difficult, if not impossible, to solve using procedural-based optimization algorithms that rely on gradient information for solution. On the other hand, the problem is readily solved using adaptive search techniques based on self-learning solution methodologies that do not rely on gradient information. This study applies the adaptive search strategy of a multi-criteria genetic algorithm (MGA) to solve the Pareto optimization problem Eqs. (3.1).

The MGA solves the conceptual design optimization problem using the basic procedures of a conventional GA (see Appendix 3.C). Namely, the genetic operators of selection, crossover and mutation are progressively applied to a population of conceptual designs encoded as binary bit strings until, guided by design fitness evaluations with account for constraint violations, convergence occurs to the Pareto-optimal design set (see Appendix 3.D) after a number of generations.

For any one generation of the genetic search, designs found to violate the constraints Eqs. (3.9) are excluded from the population to ensure that Pareto-optimal designs are identified from among feasible designs alone. The fitness of each feasible design x is based on its (Euclidean) distance $D(x)$ from the nearest Pareto design x^o (Osyczka, 1995), i.e.,

$$D(x) = Min\left[\left(1-\frac{CapitalCost(x)}{CapitalCost(x_j^o)}\right)^2 + \left(1-\frac{OperatingCost(x)}{OperatingCost(x_j^o)}\right)^2 + \left(1-\frac{RevenueIncome(x_j^o)}{RevenueIncome(x)}\right)^2\right]^{0.5} \quad (j=1,2,..,p) \tag{3.10}$$

where $D(x) > 0$ for each non-Pareto design x, while $D(x_j^o) = 0$ for each of the j = 1,2,..,p Pareto designs x_j^o. The fitness of each design x is calculated as,

$$F(x) = F_{max} - D(x) \tag{3.11}$$

where, to ensure that Eq. (3.11) does not produce a negative fitness for any design, F_{max} = the maximum $D(x)$ value found for Eq. (3.10) from among all feasible designs for the current generation. Note from Eqs. (3.10) and (3.11) that $F(x_j^o) = F_{max}$ for each Pareto design x_j^o, while that for each non-Pareto design x lies somewhere in the range $0 \leq F(x) < F_{max}$ depending on its distance from the Pareto-optimal set.

Having the fitness of all designs in the current generation, this study uses roulette wheel selection, two-point crossover and single-bit mutation (see Appendix 3.C) to identify the next generation of feasible designs. An elitist strategy is employed to ensure that current Pareto designs survive into the next generation, where they then compete with all other newly created feasible designs to become members of the new Pareto-optimal set. The genetic search procedure is repeated until there is no change in the Pareto set for a pre-assigned number of consecutive generations, at which point the MGA is deemed to have converged to the optimal Pareto set.

3.2.6 Design Computational Procedure

The flow chart for a single run of the multi-criteria genetic algorithm for Pareto-optimal conceptual design of an office building is shown in Figure 3.4. To begin, the building design project is specified by the information and limitations defined by the parameters for the design (e.g., see Table 4.1), by the ranges of possible values of the primary design variables (see Table 3.1), by the values of the lower and upper bounds for the constraint Eqs.(3.9) controlling the secondary design variables (e.g., see Table 4.1), and by rules of good design practice (e.g., see the following). As well, to facilitate the genetic search, values are assigned for population size and crossover and mutation probabilities (e.g., see Examples in Chapter 4).

Each member of the initial genetic population is a randomly generated string of binary (base-2) values of the primary design variables which, when decoded to their base-10 index values, define the structure and floor systems, the cladding and window types, the window ratio, and the numbers of column bays and corresponding span distances in the width and length directions for a particular conceptual design of the building. For example, from Table 3.2 (the binary representation of the primary variable values in Table 3.1), the 39-bit binary string 0011|0|10|11|1010|0111|101|011|01|10|1|100|10|1001|11 decodes to the base-10 indices 3,0,2,3,10,7,5,3,1,2,1,4,2,9,3 which, from Table 3.1, identify: structural type = ST = steel rigid frame; bracing type = BT = K&K; concrete floor type = CFT = two-way slab and beam; steel floor type = SFT = composite beam,

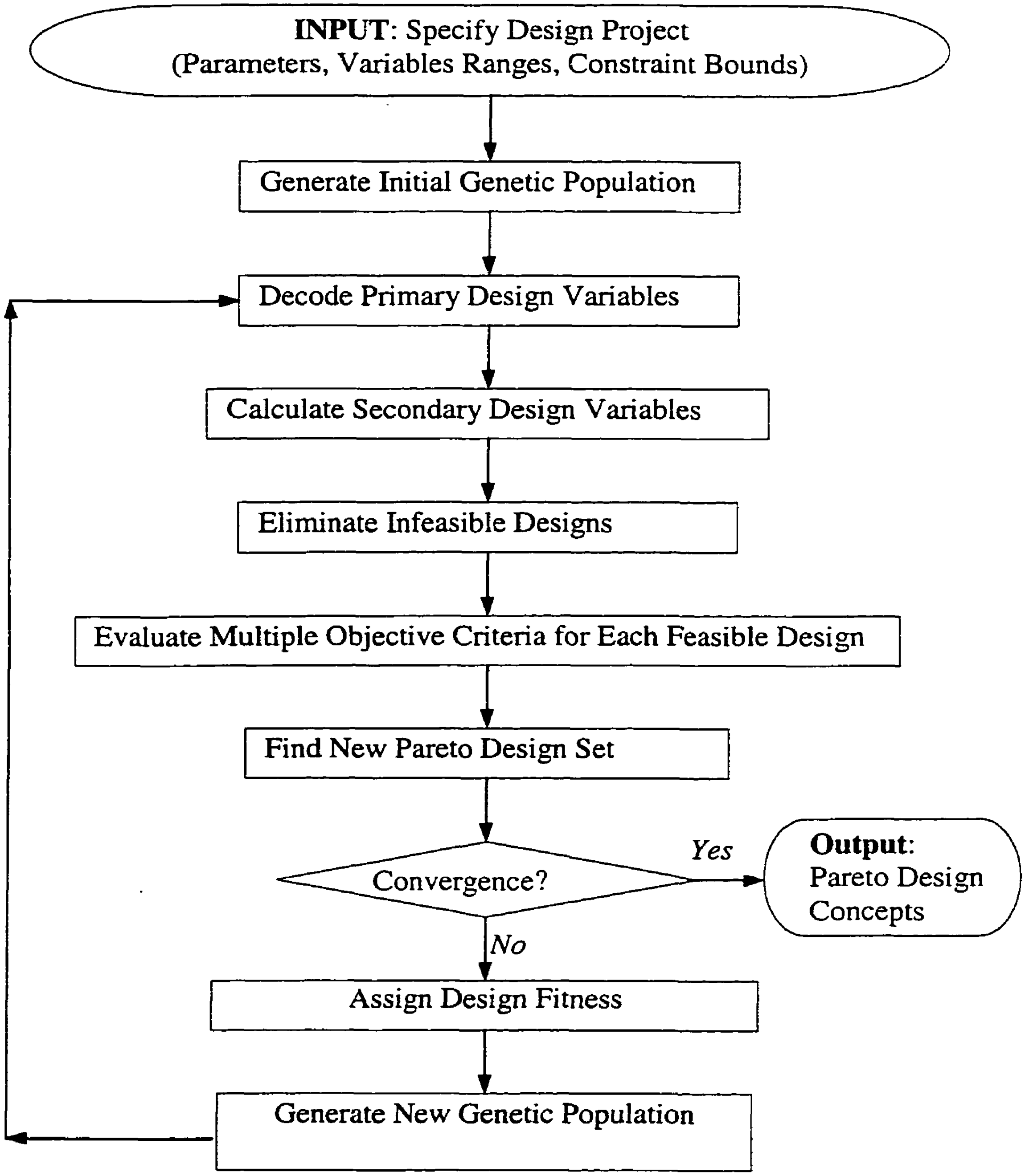


Figure 3.4: Conceptual Design Computational Procedure

deck and concrete slab; span distances between columns along the building width $a = S_a$ = 9.5m; span distances between columns along the building length $b = S_b$ = 8.0m; number of column spans along the building width $a = NS_a$ = 8; number of columns spans along the building length $b = NS_b$ = 6; number of perimeter tube column spans within S_a = NTS_a = 3; number of perimeter tube column spans within S_b = NTS_b = 4; direction of the core dimension to be assigned first = *DCDD* = *b;* ratio of the core dimension in the *b* direction to the overall dimension of the building in the *b* direction = *CDF* = 0.564; window type = *WIT* = standard heat absorbing; window ratio = *WIR* = 70%; and wall cladding type = *WAT* = glazed panel.

Rules of good design practice are then invoked for each design scenario to exclude any primary variables values that are not applicable for the chosen structure type. For the foregoing design scenario, for example, since the structural type is a steel rigid frame, the values for the variables *BT*, *CFT*, *NTSa*, and *NTSb* are deemed not applicable and are excluded from further consideration for the design. As another example, if the design is such that the structural type = *ST* = steel frame and bracing, the bracing is always selected to be either K-bracing on all four sides of the service core or K-bracing on the two sides of the core having the larger bay widths and X-bracing on the two sides having the smaller bay widths, but never X-bracing on the sides having the larger bay widths because this would then prevent ready access to elevators and stairways in the core area.

Having the values of the applicable primary design variables for a particular conceptual design of the building, the corresponding values of the secondary design variables are found to establish the dimensions of the building footprint and service core, the number of stories, the available lease office space, the floor depth, the building height, and the aspect and slenderness ratios for the building. For example, from the foregoing, the building footprint dimensions are found as $D_a = NS_a \times S_a =$ 8 x 9.5 = 76.0m, and $D_b = NS_b \times S_b$ = 6 x 8.0 = 48.0m. The service core area is found as a specified percentage of the footprint area = $D_a \times D_b$ = 76.0 x 48.0 = 3648m^2. For example, for *Percentage* = 20% , the core area is $C_a \times C_b$ = 0.20 x 3648 = 729.6m^2. Knowing the fraction that one service core dimension is to be of the footprint dimension in the same direction, the other core dimension is calculated to meet the required service core area. For example, from the foregoing, for the core width dimension randomly selected to be $C_b = 0.564D_b$ = 0.564 x 48.0 = 26.51m, the core length dimension C_a = 729.6 /26.51 = 27.50m. The number of stories is found to meet the minimum lease office space required for the building. For example, for A_{req} = 60,000m^2, the available lease office space per floor = 3648 - 729.6 = 2918.4 m^2, and the number of mechanical taken to be 4% of the number of rentable floors, the number of rentable floors = *NRF* = 60,000 /2918.4 = 20.55 = 21, the number of mechanical floors $NMF = NRF \times 0.04$ = 21 x 0.04 = 0.84 = 1, and the total number of floors *NF* = *NRF* + *NMF* = 21 + 1 = 22. The actual total amount of available rental/lease space = 21 x 2918.4 = 61,286m^2. For initial calculations, the floor depth is considered common for all stories and is defined by the type of floor and the bay area. For example, from the foregoing, for *SFT* = composite beam, deck and concrete slab, and bay area = $S_a \times S_b$ = 9.5 x 8.0 = 76.0m^2, the depth of floor = *DF* = 0.63m from Table 3.A.3b. The height of the building is defined by the number of floors *NF*, the floor depth *DF*, the specified floor-to-ceiling clearance height and the depth of false ceiling. For example, for 3m clearance height and 0.5m false ceiling depth common for all 22 stories, the overall building height *H* = 22 x (3 + 0.63 + 0.5) = 90.86m. The building aspect ratio $D_b / D_a = 48.0 / 76.0 = 0.63$, while the slenderness ratio = $H / D_b = 90.86 / 48.0 = 1.89$.

Designs which violate any of the constraint Eqs. (3.9) concerning plan and height restrictions, office space requirements, and appropriate aspect and slenderness ratios for the building, are deemed infeasible and eliminated from the population of conceptual designs, as are any building concepts not in keeping with the rules of good design practice (e.g., tube structures with spans between perimeter columns smaller than 2.25m and larger than 4.25m would be eliminated because those particular structural layouts are not practical). Eliminated designs are replaced by other, randomly generated, feasible designs so as to maintain a fixed population size.

The capital cost, operating cost and income revenue for each feasible conceptual design are calculated as described in Chapter 3 and related Appendices 3.A and 3.B. Having the cost and revenue values for the entire population of feasible conceptual designs for the building, the Pareto-optimal design set is formed by those designs that each have the characteristic that there is no other design in the population that completely dominates it in the sense of having both smaller capital and operating costs and larger income revenue.

Having the Pareto-optimal design set, the fitness of each design in the population is calculated through Eqs. (3.10) and (3.11). Then, while invoking an elitist strategy to retain the binary strings defining the Pareto-optimal designs (Figure 3.5), the genetic operations of reproduction, crossover and mutation are carried out to create a new population of binary design representations to commence the next generation of the genetic search.

Convergence of a single run of the multi-criteria genetic algorithm occurs when the Pareto-optimal design set is found to remain (relatively) the same for a specified number of consecutive generations and no improvement is noticed in the values of the cost-revenue objective criteria. Multiple runs of the MGA starting from different initial genetic populations are conducted, and the Pareto-optimal sets found at convergence of the different runs are combined together to form the overall Pareto-optimal design set (e.g., see Examples in Chapter 4).

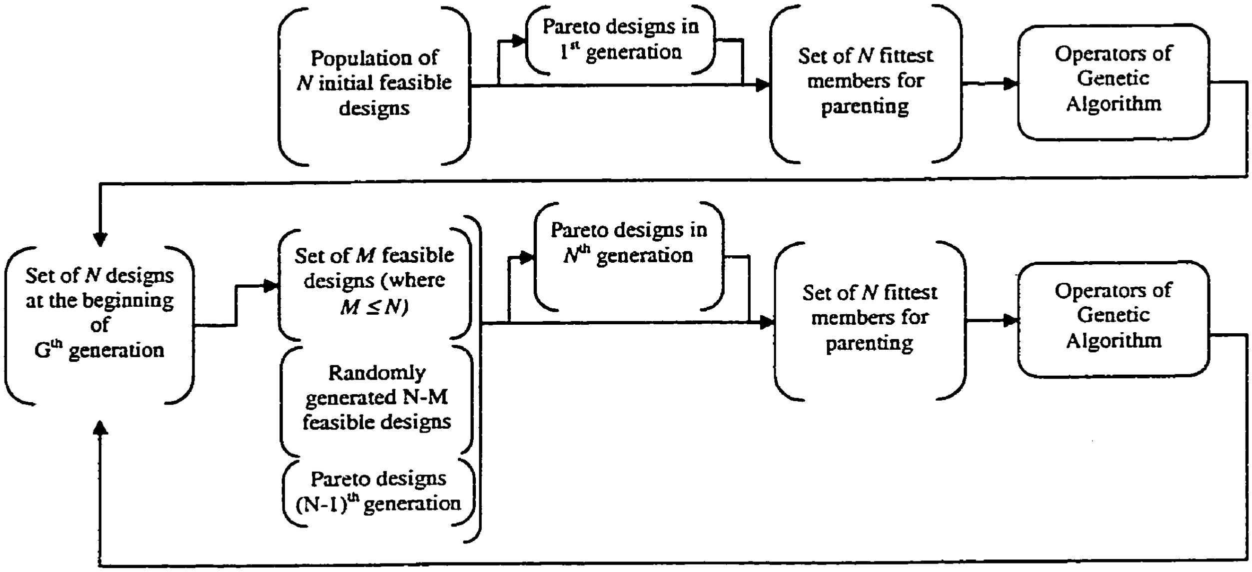


Figure 3.5: Elitist Strategy for Multi-criteria Genetic Algorithm

3.2.7 Design Profitability

It should be noted that the cost-revenue criteria evaluated for the genetic search described in the foregoing do not account for inflation and mortgage interest rates because such life-cycle costing does not affect the Pareto optimality of a building design. Once the Pareto-optimal design set has been found, the combined cost-revenue function described in the following (which does account for life-cycle inflation and mortgage interest rates) can then be applied to assess the potential profitability of each Pareto-optimal building design over time.

The profit potential of a building design over time is assessed using life-cycle costing analysis to estimate cash flows on an annual basis after completion of the project construction phase. To this end, this study assumes that the building occupancy rates vary in time as shown in Table 3.3.

Table 3.3: Variable Occupancy Rates

Time	Occupancy Rate (OR)
Throughout Year 1	50% leased
Throughout Year 2	70% leased
Throughout Year 3	85% leased
Year 4 and after	95% leased

The potential profitability of a building design can be assessed by evaluating the following cost-revenue function,

$$\begin{aligned} Profitability = & \left(RI \sum_{k=1}^{n} OR_k (1+MR)^{n-k} (1+IR)^{k-1}\right) \\ & - \left(OC \left(\sum_{k=1}^{n} (1+MR)^{n-k} (1+IR)^{k-1}\right)\right) \\ & - \left(CC\,(1+MR)^{n}\right) \end{aligned} \tag{3.12}$$

where: the values of *CC* (capital cost), *OC* (operating cost) and *RI* (revenue income) are found through Eqs. (3.2), (3.3) and (3.4) for the building design; *MR* (mortgage rate) and *IR* (inflation rate) are fixed annual life-cycle rates; k = a yearly counter; OR_k = occupancy rate (Table 3.3); and n = the number of years after completion of construction. If $Profitability > 0$ from Eq. (3.12), the design is profitable in year n and all years thereafter; otherwise, if $Profitability < 0$ the design is not profitable in year n or in any year previous. As illustrated for the design examples, in Chapter 4, Eq. (3.12) can also be used to predict the year n in which a building design first becomes profitable.

Appendix 3.A - Capital Cost (Eq. 3.2)

3.A.1 Cost of Land

The land cost is a function of unit land rates (as defined by local location information) and the footprint dimensions of the building, i.e.,

$$Cost_{Land} = (D_a \times D_b) \times Land_{unit\,cost} \tag{3.A.1}$$

3.A.2 Cost of Floor System

For known column layout, floor system, and applied live and dead gravity loads, the cost, depth and selfweight of the floor system per unit area are found using prepared databases based on bay area. For structural systems that do not engage the flooring system as part of the lateral load resisting system, such as tubes, the floor system is usually designed only for gravity loading (the databases used for floor systems in this study are generated based on this condition).

Table 3.A.1 defines the gravity loading considered by this study. Table 3.A.2a and 3.A.2b identify the percentage costs of different components of floor system construction. Tables 3.A.3a and 3.A.3b represent the cost, depth and selfweight of different concrete and steel floor systems under gravity loads, for bay areas up to 149 m^2.

Table 3.A.1: The Intensity of Applied Gravity Loads

Load	Intensity (kN/m^2)
Live load	2.80
Self weight	Depends on type of floor and bay area: see Tables 3.A.3a & 3.A.3b
Superimposed Dead Load	
Partitions	1.00
Plumbing and ducting	0.20
False ceiling and fixtures	0.15
Floor finishing	0.10
Total	1.45

Table 3.A.2: Percentage of Different Construction Components in Floor Unit Cost

Floor Type	Average percentage of the unit cost			
	Forming %	Reinforcement %	Concrete %	Structural Steel %
Flat plate	50	20	30	-
Flat slab	51	19	30	-
Beam and slab	54	21	25	-
Waffle slab	54	18	28	-

(a) Concrete Structures

Floor Type	Average percentage of the unit cost			
	Forming %	Reinforcement %	Concrete %	Structural Steel %
Steel joist & beams with deck and slab	-	5	20	75
Com. Beam & CIP Slab	41	6	15	38
W Shape Com. Beam Deck & Slab	-	3	16	81
Composite beam, deck and slab	-	3	26	71

(b) Steel Structures

Table 3.A.3: Floor Information

Bay Area (m^2)	Flat Plate			Flat Slab			Beam & Slab			Waffle Slab		
	Cost $/m^2	Depth m	Self W kN/m^2	Cost $/m^2	Depth m	Self W kN/m^2	Cost $/m^2	Depth m	Self W kN/m^2	Cost $/m^2	Depth m	Self W kN/m^2
21	85.36	0.14	3.29	92.35	0.22	3.73	101.61	0.22	3.25	110.44	0.25	4.97
28	94.18	0.19	4.49	97.41	0.27	4.16	111.73	0.29	4.01	113.02	0.25	4.97
37	94.40	0.19	4.78	100.75	0.28	4.44	117.33	0.29	4.40	115.60	0.25	4.97
46	100.21	0.23	5.40	109.04	0.37	5.21	126.91	0.36	5.07	117.43	0.25	5.07
58	104.73	0.24	5.69	110.87	0.39	5.54	126.26	0.36	5.26	120.88	0.30	5.26
70	109.25	0.25	5.98	118.19	0.42	6.12	131.64	0.43	6.02	122.39	0.30	5.45
84	113.77	0.27	6.26	123.57	0.46	6.79	141.65	0.43	6.55	126.80	0.36	6.17
98	118.30	0.28	6.55	131.32	0.52	7.46	147.47	0.51	7.17	126.15	0.36	6.17
114	122.82	0.29	6.84	134.55	0.58	7.89	152.31	0.51	7.56	134.55	0.36	6.41
130	127.34	0.31	7.12	139.93	0.65	8.32	159.31	0.58	8.23	139.39	0.41	6.50
149	131.86	0.32	7.41	150.69	0.71	8.75	166.30	0.65	8.90	143.70	0.41	6.50

(a) Concrete Structures

Bay Area (m^2)	Steel Joist & Beam Deck & Slab			Com. Beam & CIP Slab			W Shape Com. Beam Deck & Slab			Com. Beam, Deck & Slab		
	Cost $/m^2	Depth m	Self W kN/m^2	Cost $/m^2	Depth m	Self W kN/m^2	Cost $/m^2	Depth m	Self W kN/m^2	Cost $/m^2	Depth m	Self W kN/m^2
21	77.93	0.48	2.05	118.40	0.53	2.62	112.70	0.41	2.10	100.97	0.58	1.67
28	84.28	0.48	2.10	125.40	0.53	2.62	121.85	0.41	2.10	103.76	0.58	1.67
37	90.63	0.66	2.10	132.40	0.53	2.62	131.00	0.53	2.43	106.56	0.58	1.91
46	95.58	0.66	2.15	139.39	0.53	2.62	144.24	0.53	2.48	109.36	0.60	1.91
58	104.41	0.66	2.15	146.39	0.57	2.91	149.08	0.74	2.53	112.16	0.60	2.05
70	106.78	0.74	2.15	154.46	0.57	2.91	159.31	0.74	2.58	112.50	0.60	2.10
84	114.21	0.81	2.15	152.85	0.63	2.67	163.61	0.74	2.58	112.59	0.67	1.95
98	126.91	0.81	2.19	165.23	0.71	3.06	174.38	0.89	2.67	117.54	0.75	2.00
114	135.95	0.97	2.19	167.92	0.71	2.77	179.22	0.89	2.67	128.74	0.75	2.19
130	144.99	1.08	2.24	177.60	0.71	2.82	186.22	0.97	2.67	134.87	0.75	2.19
149	154.03	1.27	2.24	187.29	0.71	2.82	193.21	1.04	2.72	141.01	0.75	2.38

(b) Steel Structures

** Prices shown are calculated based on US national average costs. For bay areas different from those listed, the cost, depth and selfweight are interpolated or extrapolated.*

The cost of the floor system is the product of the unit cost of floor system times the built floor area including the area covered by elevators and stair cases, i.e.,

$$Cost_{Floor} = NF\,(D_a \times D_b - (NSC \times OILSC \times OIWSC - NE \times 5.9)) \times Floor_{Unit\,Cost} \qquad (3.A.2)$$

For any given type of floor, the unit cost accounts for the cost of the different construction components (i.e., steel, concrete, reinforcement, forming), which can be established as the product of their percentage cost share (Table 3.A.2) times the floor unit cost (Table 3.A.3). The US national average floor unit cost in Table 3.A.3 can be modified for any specific location (city) by accounting for the different cost location factors that relate the cost of different materials to their US national average cost (see Table 2.1). As well, the US national average costs of the individual components can also be modified to account for the building location (city). For an example, the cost of components of a flat plate flooring system are: $Cost_{forming} = 0.50 \times Floor_{unit\ cost}$; $Cost_{reinforcement} = 0.20 \times Floor_{unit\ cost}$; $Cost_{concrete} = 0.30 \times Floor_{unit\ cost}$; and $Cost_{steel} = 0.00 \times Floor_{unit\ cost}$. Therefore, the modified floor unit cost is found as:

$$\begin{aligned} Mod.\ Floor_{unit\ cost} = (&Cost_{forming} \times FCLF + Cost_{reinforcement} \times RCLF + \\ &Cost_{concrete} \times CCLF + Cost_{steel} \times SCLF) \end{aligned} \qquad (3.A.3)$$

where $FCLF$, $RCLF$, $CCLF$ and $SCLF$ are cost location factors for forming, reinforcement, concrete and steel, respectively (Table 2.1).

The data in Table 3.A.3 can also be used for structural systems that take advantage of the flooring system to resist lateral loads. This is done by choosing a larger bay area than reality such that the gravity loading induces moments in the floor that are approximately equivalent to those that would be caused by a combination of gravity and lateral loads. For these types of structural systems where floors contribute to lateral stiffness, this study changes the size of the flooring system every four floors to account for the increased forces induced in the flooring system over the height of the building. Hence, contrary to that for structural systems that do not rely on floor elements to carry lateral forces, the cost of the flooring system is not constant for all stories of the building for these structural systems.

3.A.3 Cost of Columns

In order to achieve a fair estimate of the cost of columns in a building at the conceptual stage of the design, it is necessary to find a reasonably accurate approximation of the column sizes necessary to resist the axial forces induced by the different design load combinations. This approximation should account for both dead and live gravity loads in addition to applied lateral loads. In this study, estimated axial forces in columns are found from the results of approximate determinate analysis for different combinations of gravity dead and live floor loadings, and from the results of approximate indeterminate analysis (Portal Method) for applied lateral loads (Smith and Coull 1991). Additional axial forces induced in perimeter columns by vertical bracing systems and outrigger trusses are also accounted for.

Having the factored axial forces, the column sizes are found based on the Handbook of Steel Construction (1997) and the Concrete Design Handbook (1995). For the purpose of choosing appropriate sizes, it is assumed that columns are four meter (4m) tall on average. Tables 3.A.4 and 3.A.5 represent the sections, dimensions and costs adopted by this study for steel and concrete columns, respectively. Figures 3.A.1, 3.A.2, 3.A.3 and 3.A.4 demonstrate the relationships between factored axial resistance and material mass, area and volume for steel and concrete columns (based on the Canadian design standards: Concrete Design Handbook 1995 and Handbook of Steel Construction 1997). These Tables and Figures are based on the following material properties for steel and concrete: yield stress of structural steel F_y = 350 MPa; compressive strength of concrete f'_c = 400MPa; yield stress of reinforcement steel f_y = 400 MPa.

Table 3.A.4: Steel Column Costs

Designation	[1]FR (kN)	[2]CSA mm²	Mass/[3]vlm kg/m	Cost[4] $/m	Designation	[1]FR (kN)	[2]CSA mm²	Mass/[3]vlm Kg/m	[4]Cost $/m	Designation	[1]FR (kN)	[2]CSA mm²	Mass/[3]vlm Kg/m	[4]Cost $/m
WWF650-864	34500	110000	858.00	1750.32	WWF450-342	13600	43600	340.08	693.76	W310-179	5770	22800	177.84	362.79
WWF600-793	31700	101000	787.80	1607.11	WWF500-306	12200	39000	304.20	620.57	WWF350-137	5300	17500	136.50	278.46
WWF650-739	29600	94100	733.98	1497.32	WWF400-303	11900	38600	301.08	614.20	W310-158	5040	20000	156.00	318.24
WWF550-721	28800	92000	717.60	1463.90	WWF500-276	11000	35200	274.56	560.10	W310-143	4580	18200	141.96	289.60
WWF600-680	27200	86600	675.48	1377.98	WWF450-274	10900	35000	273.00	556.92	W310-129	4130	16500	128.70	262.55
WWF500-651	25900	83000	647.40	1320.70	WWF400-273	10700	34800	271.44	553.74	W310-118	3750	15000	117.00	238.68
WWF550-620	24800	79100	616.98	1258.64	WWF350-263	10200	33600	262.08	534.64	W310-107	3390	13600	106.08	216.40
WWF650-598	23900	76200	594.36	1212.49	WWF500-254	10100	32300	251.94	513.96	W310-97	3060	12300	95.94	195.72
WWF500-561	22400	71600	558.48	1139.30	WWF450-248	9820	31600	246.48	502.82	W310-86	2420	11000	85.80	175.03
WWF600-551	22000	70200	547.56	1117.02	WWF400-243	9540	31000	241.80	493.27	W310-79	2180	10000	78.00	159.12
WWF550-503	20100	64200	500.76	1021.55	WWF350-238	9180	30200	235.56	480.54	W250-73	2060	9280	72.38	147.66
WWF650-499	20000	63600	496.08	1012.00	WWF450-228	9000	29000	226.20	461.45	W310-67	1470	8500	66.30	135.25
WWF600-460	18400	58600	457.08	932.44	WWF400-220	8620	28000	218.40	445.54	W200-59	1380	7530	58.73	119.82
WWF500-456	18200	58200	453.96	926.08	WWF350-212	8150	27000	210.60	429.62	W250-58	1320	7420	57.88	118.07
WWF400-444	17500	56600	441.48	900.62	WWF450-201	7950	25600	199.68	407.35	W200-52	1210	6620	51.64	105.34
WWF550-420	16800	53600	418.08	852.88	WWF350-192	7390	24400	190.32	388.25	W250-49	1080	6250	48.75	99.45
WWF450-409	16300	52200	407.16	830.61	W310-226	7380	28900	225.42	459.86	W200-46	1050	5820	45.40	92.61
WWF650-400	15500	51000	397.80	811.51	WWF400-178	6980	22700	177.06	361.20	W200-42	737	5280	41.18	84.02
WWF500-381	15200	48600	379.08	773.32	WWF350-176	6770	22400	174.72	356.43	W200-36	627	4540	35.41	72.24
WWF600-369	14700	47000	366.60	747.86	W310-202	6550	25800	201.24	410.53	W150-30	477	3790	29.56	60.31
WWF400-362	14300	46200	360.36	735.13	WWF400-157	6190	20100	156.78	319.83	W150-22	339	2840	22.15	45.19
WWF500-343	13700	43800	341.64	696.95	WWF350-155	5980	19800	154.44	315.06					

[1] Factored Resistance, [2] Cross Section Area, [3] vertical linear meter,[4] Prices shown are calculated based on US national average costs (for specific locations, the cost of steel is adjusted accordingly).

Table 3.A.5: Concrete Column Costs

Width (m)	[1]CSA (m^2)	[2]L (m)	[3]FA (m^2)	[4]LS %	[5]EXS %	Reinforcement (1000kg)/[6] vlm	Concrete Volume (m^3 / vlm)	[7]FR (kN)	[8]Form Cost $/vlm	[8]Concrete Cost $/vlm	[8]Steel Cost $/vlm
0.25	0.065	15.50	1.02	4.3	1	0.027	0.065	1517	46.84	9.26	38.22
0.3	0.093	10.76	1.22	3.9	1	0.035	0.093	2072	56.21	13.33	50.67
0.36	0.126	7.91	1.42	3.8	1	0.047	0.126	2795	65.57	18.15	67.84
0.41	0.165	6.05	1.63	4.0	1	0.064	0.165	3736	74.94	23.70	92.30
0.46	0.209	4.78	1.83	4.0	1	0.082	0.209	4741	84.31	30.00	117.06
0.51	0.258	3.88	2.03	3.9	1	0.098	0.258	5756	93.68	37.04	140.76
0.56	0.312	3.20	2.24	4.0	1	0.122	0.312	6939	103.04	44.82	174.51
0.61	0.372	2.69	2.44	4.0	1	0.146	0.372	8445	112.41	53.33	209.34
0.66	0.436	2.29	2.64	3.9	1	0.167	0.436	9754	121.78	62.59	238.86
0.71	0.506	1.98	2.84	4.0	1	0.195	0.506	11386	131.15	72.59	279.85
0.76	0.581	1.72	3.05	3.9	1	0.220	0.581	12926	140.51	83.33	315.42
0.81	0.661	1.51	3.25	4.0	1	0.259	0.661	15000	149.88	94.82	370.69
0.86	0.746	1.34	3.45	4.0	1	0.292	0.746	16930	159.25	107.04	418.48
0.91	0.836	1.20	3.66	4.1	1	0.331	0.836	19057	168.62	120.00	473.83
0.97	0.932	1.07	3.86	4.1	1	0.369	0.932	21269	177.98	133.70	528.98
1.02	1.032	0.97	4.06	4.1	1	0.408	1.032	23545	187.35	148.15	584.97
1.07	1.138	0.88	4.27	4.0	1	0.439	1.138	25624	196.72	163.33	629.67
1.12	1.249	0.80	4.47	4.1	1	0.495	1.249	28547	206.09	179.26	709.21
1.17	1.365	0.73	4.67	4.0	1	0.528	1.365	30725	215.45	195.93	756.84
1.22	1.486	0.67	4.88	4.0	1	0.584	1.486	33774	224.82	213.34	837.38
1.27	1.613	0.62	5.08	4.0	1	0.633	1.613	36626	234.19	231.48	906.81
1.32	1.745	0.57	5.28	4.0	1	0.682	1.745	39543	243.56	250.37	976.91
1.37	1.881	0.53	5.49	4.0	1	0.731	1.881	42522	252.92	270.00	1047.19
1.42	2.023	0.49	5.69	4.0	1	0.783	2.023	45531	262.29	290.37	1121.67
1.47	2.17	0.46	5.89	3.9	1	0.833	2.170	48637	271.66	311.48	1193.52
1.52	2.323	0.43	6.10	4.1	1	0.922	2.323	53106	281.03	333.34	1321.38

[1]Cross Section Area, [2]Length of a column for one cubic meter volume, [3] Forming Area for one meter of the column, [4] Longitudinal Steel percentage, [5] EXtra Steel percentage for ties and over laps, [6] vertical linear meter, [7] Factored Resistance, [8] Prices shown are calculated based on US national average costs(for specific locations, the costs of reinforcement, concrete and forming are adjusted accordingly).

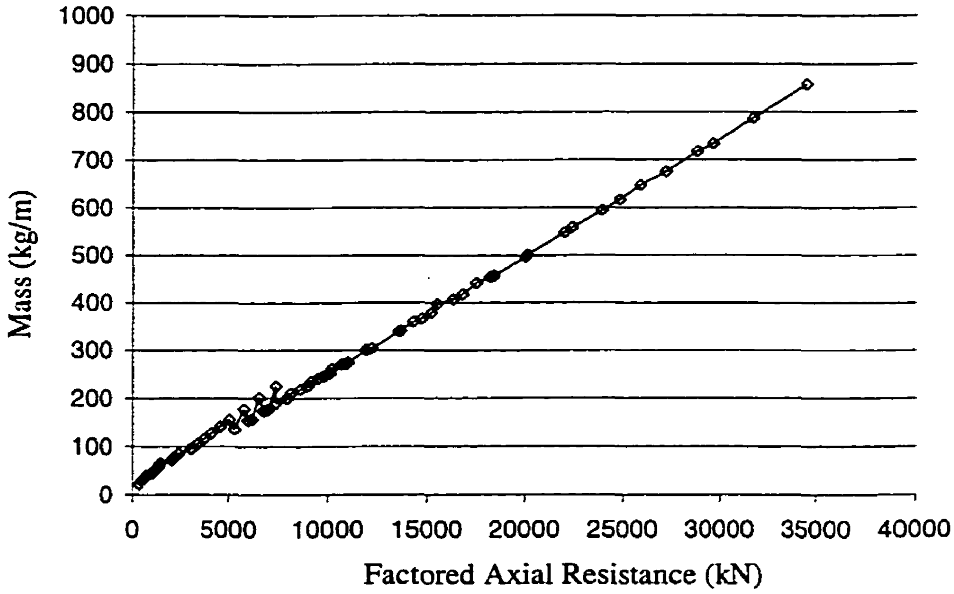


Figure 3.A.1: Mass of Steel Columns vs. Axial Load Capacity

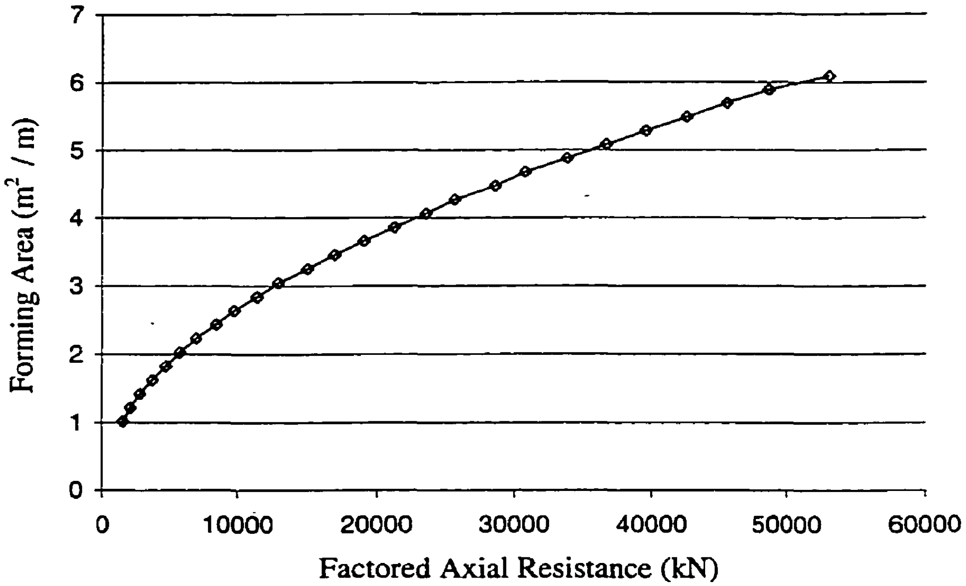


Figure 3.A.2: Forming Area vs. Axial Load Capacity of Concrete Columns

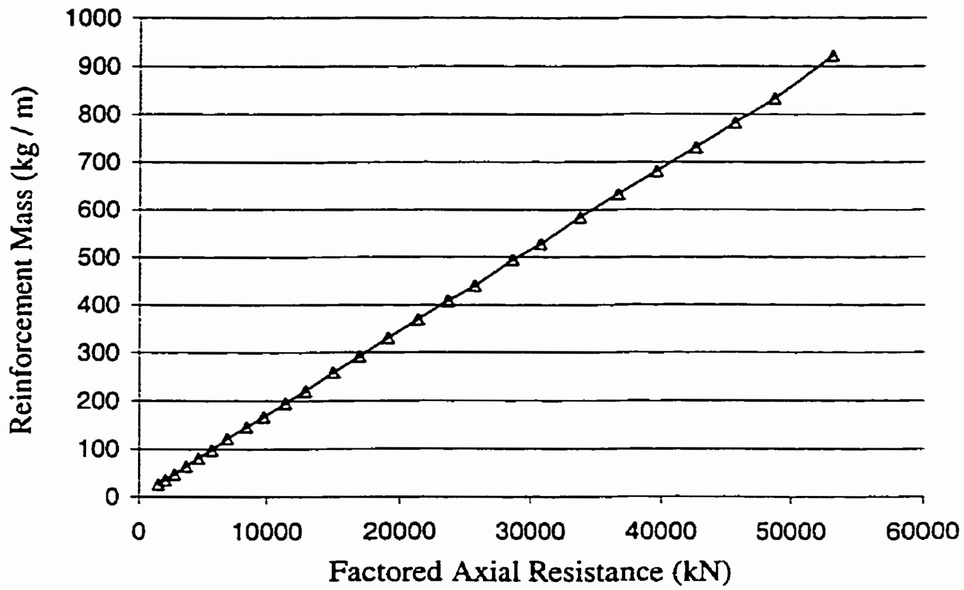


Figure 3.A.3: Reinforcement Mass vs. Axial Load Capacity of Concrete Columns

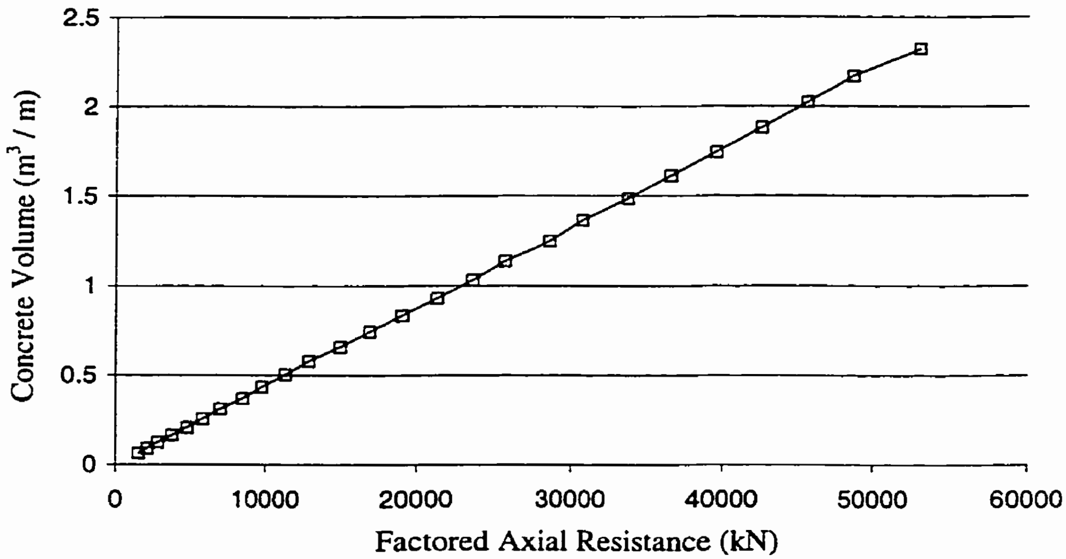


Figure 3.A.4: Concrete Volume vs. Axial Load Capacity of Concrete Columns

Having the cost of each column per vertical linear meter, the cost of columns is found as,

$$Cost_{\text{Steel columns}} = HF \times \sum_{j=1}^{NF} \sum_{i=1}^{nc} Col_{mass/vlm} \times SCLF \times Steel_{unit\,cost} \tag{3.A.4}$$

$$\begin{aligned} Cost_{\text{Concrete columns}} = HF \times \sum_{j=1}^{NF} \sum_{i=1}^{nc} \; & (Rebar_{mass/vlm} \times RCLF \times Rebar_{unit\,cost} + \\ & Concrete_{volume\,vlm} \times CCLF \times Concrete_{unit\,cost} + \\ & Forming_{area\,vlm} \times FCLF \times Forming_{unit\,cost}) \end{aligned} \tag{3.A.5}$$

where $Col_{mass/vlm}$ is the mass of each column per vertical linear meter, nc is the number of columns, NF and HF are the number and height of floors, and $SCLF$, $CCLF$, $RCLF$ and $FCLF$ are the cost location factors for steel, concrete, reinforcement and forming, respectively. The mass of reinforcement, volume of the concrete and area of forming in Eq. (3.A.5) are each for one meter length of the column. While forces in columns change from one story to the next, columns sizes are held constant over four stories by designing for the column forces in the lowest of the four stories.

3.A.4 Cost of Lateral Load Resisting System

For structural systems that carry lateral forces using only column and floor systems, the cost of the lateral load resisting system is already accounted for since choosing appropriate column and floor sizes accounts for worse-case gravity and lateral load combinations. However, for systems relying on additional means for lateral stability, such as shear walls, bracings, outrigger trusses and tubes, approximate indeterminate analysis (e.g., Portal Method) for lateral loads is used to estimate forces and, hence, sizes for these additional structural elements. Having the sizes, the cost of the lateral load resisting system is then found.

To achieve a feasible structural layout of lateral load resisting systems, it is assumed that there are at least two columns within the core area in both the *a* and *b* directions for systems that involve shear walls or bracings. The shear walls and bracings are placed in the core area in a symmetrical arrangement aligned with the axes of the column rows (Figure 3.A.5). When bracing is used, K-bracing is placed in the direction having the largest span distance between columns so as to provide appropriate openings for access to the area within the structural core, and either K or X-bracing is used in the other direction. To ensure access to the entire floor plan on floors that contain outrigger trusses, only K-trusses are used to transfer load to the exterior columns.

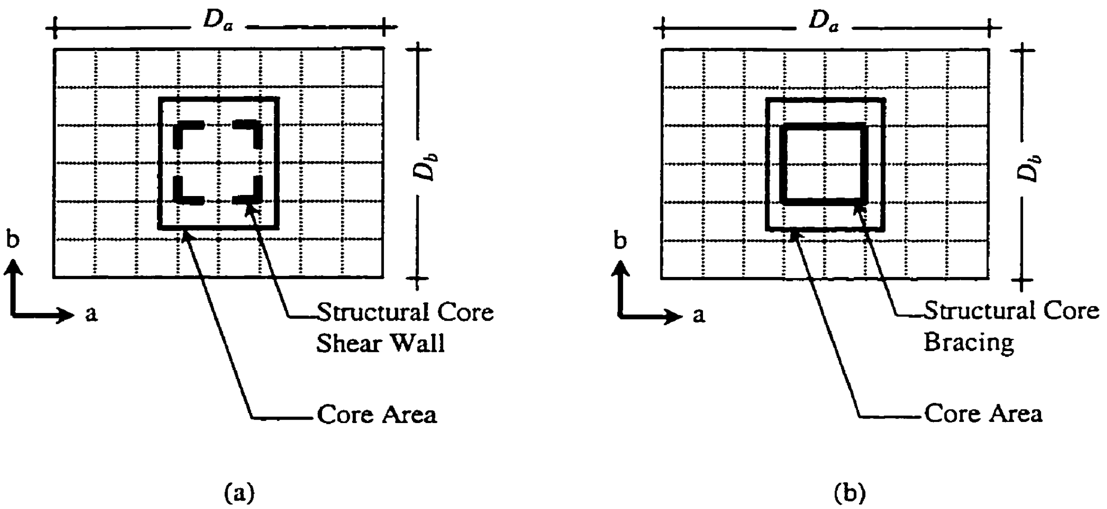


Figure 3.A.5: Schematic of Location of Structural Core within the Core Area

Hollow tubular sections are considered for both compression and tension members (Table 3.6) in designing vertical and outrigger truss systems. Having the size and mass of individual members, the cost of the bracing system is given by the sum of the costs of all members as,

$$Cost_{Bracing} = \sum_{j=1}^{NF} \sum_{i=1}^{nbm} BraM_{ij} \times lbm_{ij} \times SCLF \times Steel_{unit\ cost} \tag{3.A.6}$$

where $BraM_{ij}$ is the mass per meter of bracing member i in story j, lbm_{ij} is the length of bracing member i in story j, nbm is the number of bracing members for each story and $SCLF$ is the steel cost location factor. The sizes of bracing members are changed every four stories to account for changes in their induced axial forces over the height of the building.

Figure 3.A.6 shows the resistance vs. mass relationship for a tension bracing member, while Figure 3.A.7 shows the same for a compression bracing member that is 7.21m long. Figure 3.A.8 demonstrates the relationship between cross-sectional area and radius of gyration for the steel sections in Table 3.A.6 (this relationship is used to express the equation for compression resistance of bracing members solely in terms of cross-section area A).

To facilitate access to the area within the structural core, shear wall openings of three meters (3m) width are introduced on each side of the core. Furthermore, the structural shear-wall box is designed as a vertical cantilever beam column that carries, in addition to lateral loads, gravity loads corresponding to the tributary area of the columns that are replaced by the structural core.

Table 3.A.6: Steel Sections used for Bracing Members

Designation	Mass (kg/m)	CSA[1] (mm²)	Designation	Mass (kg/m)	CSA[1] (mm²)	Designation	Mass (kg/m)	CSA[1] (mm²)
HSS 610-13	187	23800	HSS 324-9.5	73.9	9410	HSS 114-8	20.9	2660
HSS 610-11	164	20900	HSS 324-8	61.9	7890	HSS 114-6.4	16.9	2150
HSS 610-9.5	141	18000	HSS 324-6.4	49.7	6330	HSS 114-4.8	12.9	1640
HSS 559-13	171	21800	HSS 273-13	81.6	10400	HSS 102-8	18.4	2340
HSS 559-11	150	19100	HSS 273-11	71.9	9160	HSS 102-6.4	14.9	1900
HSS 559-9.5	129	16400	HSS 273-9.5	61.9	7890	HSS 102-4.8	11.4	1450
HSS 508-13	155	19800	HSS 273-8	52	6620	HSS 102-3.8	9.19	1170
HSS 508-11	136	17400	HSS 273-6.4	41.8	5320	HSS 89-8	15.9	2020
HSS 508-9.5	117	14900	HSS 219-13	64.6	8230	HSS 89-6.4	12.9	1650
HSS 508-8	98	12500	HSS 219-11	57.1	7270	HSS 89-4.3	9.92	1260
HSS 406-13	123	15700	HSS 219-9.5	49.3	6270	HSS 89-3.8	8	1020
HSS 406-11	108	13800	HSS 219-8	41.4	5270	HSS 73-6.4	10.4	1330
HSS 406-9.5	93.3	11900	HSS 219-6.4	33.3	4240	HSS 73-4.8	8.04	1020
HSS 406-8.0	78.1	9950	HSS 219-4.8	25.3	3220	HSS 73-3.8	6.5	828
HSS 406-6.4	62.6	7980	HSS 168-9.5	37.3	4750	HSS 73-3.2	5.48	698
HSS 356-13	107	13700	HSS 168-8	31.4	4000	HSS 60-6.4	8.45	1080
HSS 356-11	94.6	12000	HSS 168-6.4	25.4	3230	HSS 60-4.8	6.54	834
HSS 356-9.5	81.3	10400	HSS 168-4.8	19.3	2460	HSS 60-3.8	5.31	676
HSS 356-8	68.2	8680	HSS 141-9.5	31	3950	HSS 60-3.2	4.48	571
HSS 356-6.4	54.7	6970	HSS 141-8	26.1	3330	HSS 48-1.8	5.13	654
HSS 324-13	97.5	12400	HSS 141-6.4	21.1	2690	HSS 48-3.8	4.18	533
HSS 324-11	85.8	10900	HSS 141-4.8	16.1	2050	HSS 48-3.2	3.54	451

[1] *Cross Sectional Area.*

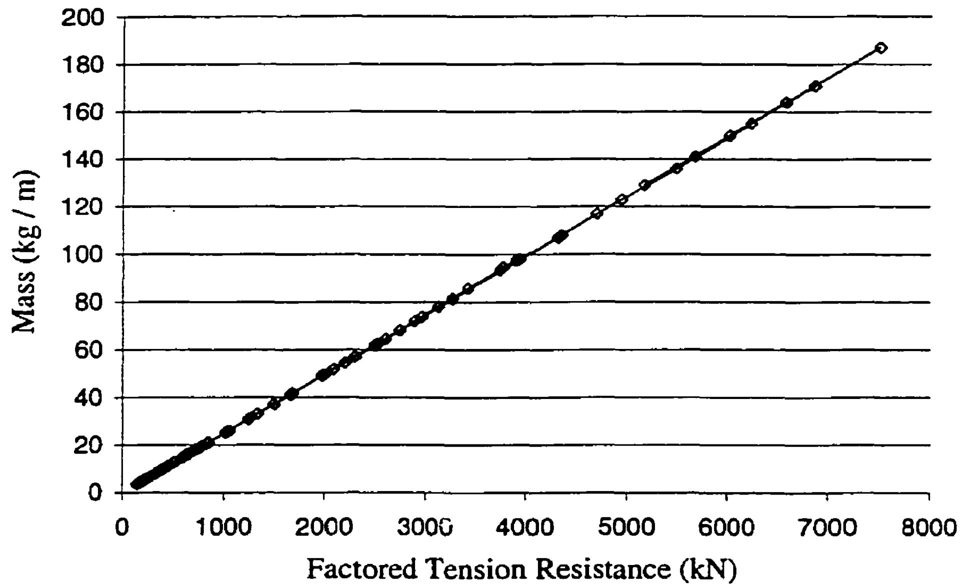


Figure 3.A.6: Factored Tension Resistance vs. Mass for Bracing Members

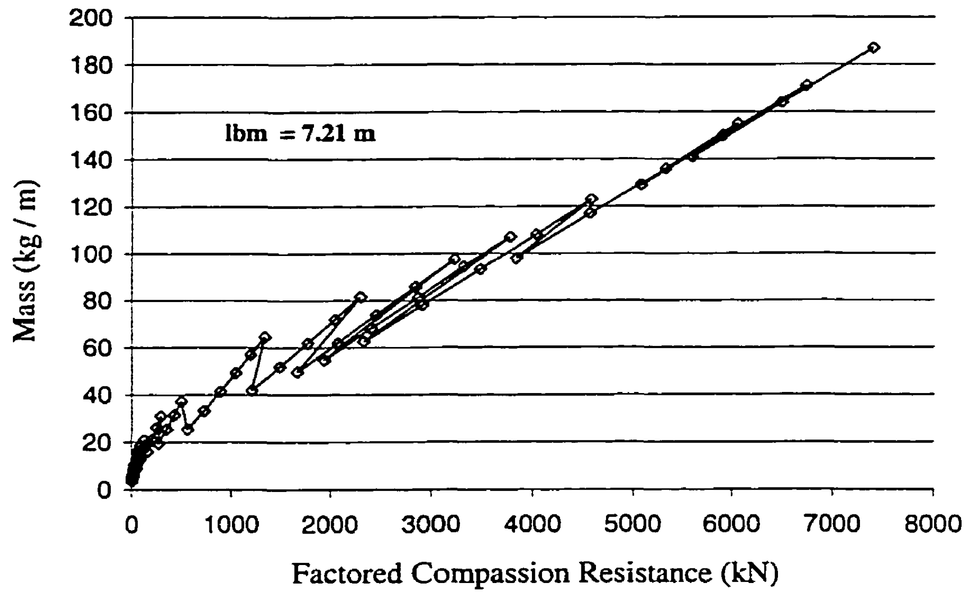


Figure 3.A.7: Factored Compression Resistance vs. Mass for Bracing Members

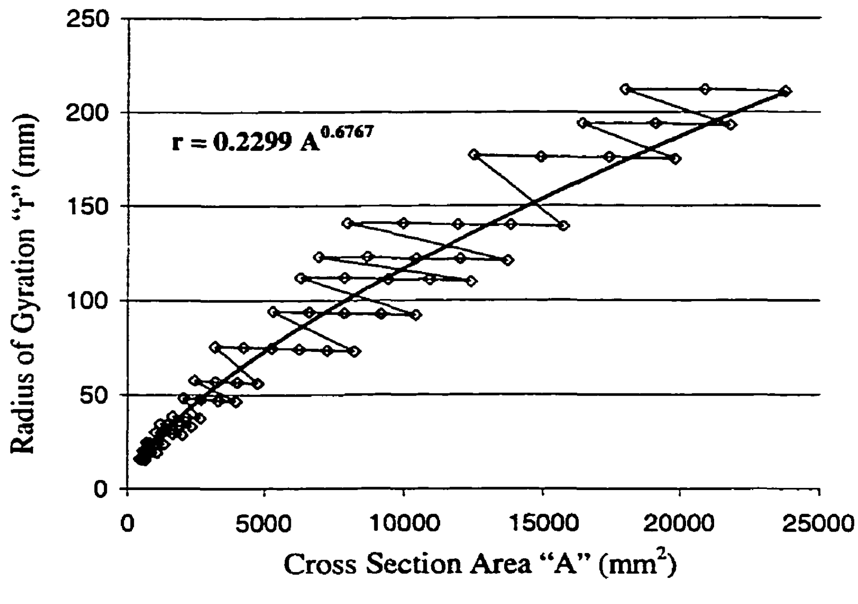


Figure 3.A.8: Cross Sectional Area vs. Radius of Gyration for Bracing Sections

To simplify the approximate analysis of the lateral load resisting system, it is assumed that the coupling beams connecting the four corners of the structural core together are infinitely rigid and that shear flow analysis can be used to estimate their shear forces and bending moments. Finally, it is assumed that the design of each shear wall prevails constant over four stories of the building. The cost of shear walls and coupling beams is found as,

$$Cost_{\text{Shear wall and coupling beams}} = \sum_{i=1}^{NF} (Rebar_{mass} \times RCLF \times Rebar_{unit\ cost} + Concrete_{volume} \times CCLF \times Concrete_{unit\ cost} + Forming_{area} \times FCLF \times Forming_{unit\ cost})_i \quad (3.A.7)$$

where NF is the number of floors and, as previously defined, *RCLF, CCLF and FCLF* are cost location factors (Table 2.1).

The Portal Method of approximate analysis is the basis for determining the size and, hence, the cost of structural elements in tubular systems. Only lateral loads are considered in the design of spandrel beams. The steel sections in Table 3.A.7 are used in the design of spandrel beams in steel tubular systems. Figure 3.A.9a demonstrates the mass vs. factored moment of resistance relationship for all sections in Table 3.A.7, while Figure 3.A.9b demonstrates that for only those sections in Table 3.A.7 that are the most economical to carry bending moments. In the same manner, Figure 3.A.10a represents the relationship between mass and factored shear resistance for all sections in Table 3.A.7, while Figure 3.A.10b only refers to those sections in Table 3.A.7 that are the best for resisting shear force. Each spandrel beam is designed for both shear force and bending moment, and the appropriate mass is assigned to the beam in accordance with the governing shear or bending case. In concrete tubular systems, it is assumed that the height of the beam is twice its width (for a minimum width of 250mm). Concrete spandrel beams are designed for both shear and bending and their costs are defined by their concrete volume, mass of reinforcement and area of forming. In stories where the floor system alone can overcome forces induced by combined gravity and lateral loads, no extra cost for spandrel beams is considered. The costs of spandrel beams for steel and concrete frame tube systems are found as,

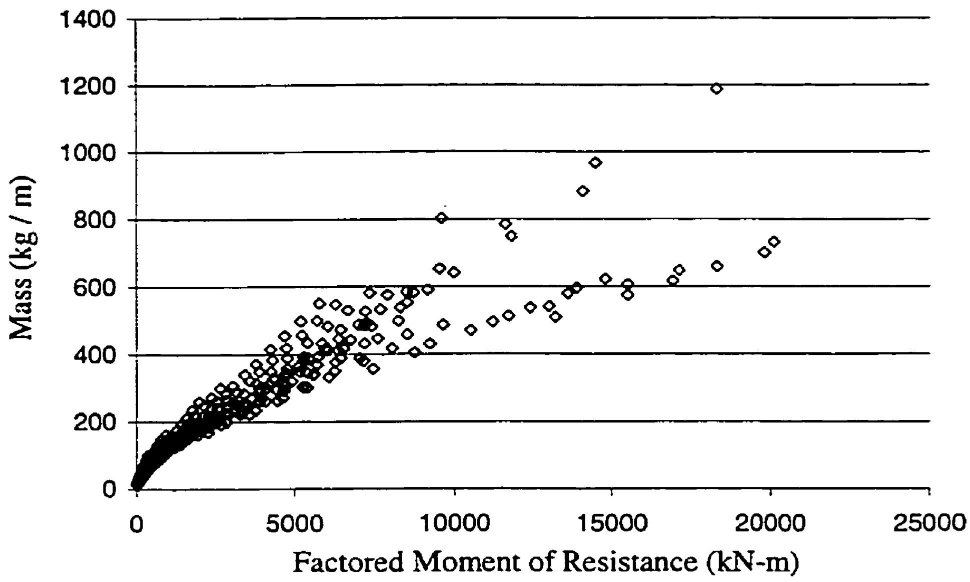
$$Cost_{\text{Steel spandrel beam}} = \sum_{i=1}^{NF} (SBeaM \times SCLF \times Steel_{\text{unit cost}})_i \tag{3.A.8}$$

$$\begin{aligned} Cost_{\text{Concrete spandrel beams}} = \sum_{i=1}^{NF} (&Rebar_{\text{mass}} \times RCLF \times Rebar_{\text{unit cost}} + \\ &Concrete_{\text{volume}} \times CCLF \times Concrete_{\text{unit cost}} + \\ &Forming_{\text{area}} \times FCLF \times Forming_{\text{unit cost}})_i \end{aligned} \tag{3.A.9}$$

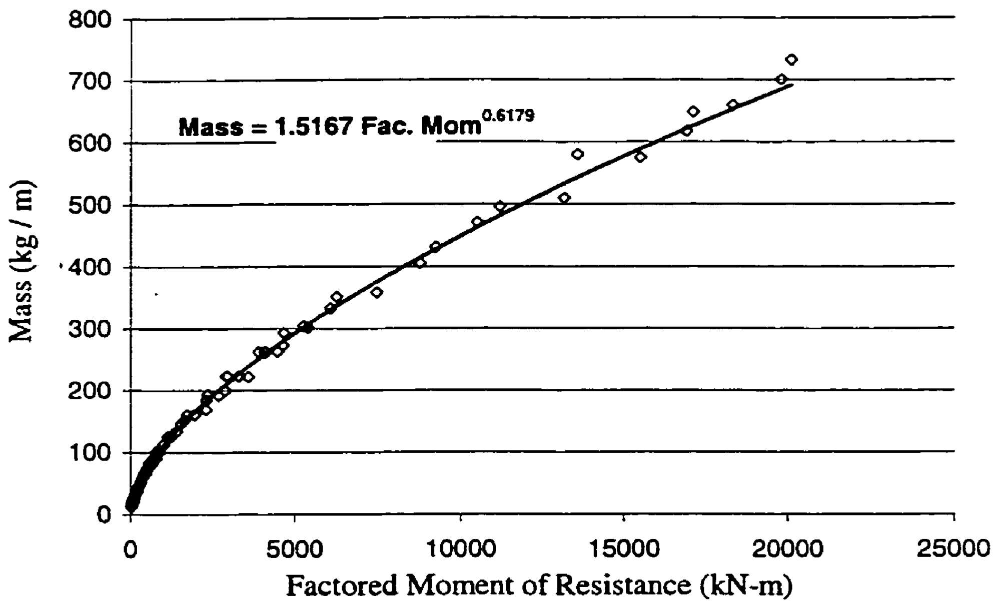
where: *SBeaM* is the total mass of steel spandrel beams around each floor; the mass of reinforcement bars, volume of concrete and area of forming are for the entire length of concrete spandrel beams around each floor; and *SCLF*, *RCLF*, *CCLF* and *FCLF* are the corresponding cost location factors. The design of spandrel beams is changed every four stories to account for the changes in their induced forces over the height of the building.

Table 3.A.7: Sections for Steel Spandrel Beams in Tubular Systems

W 920-1188	WWF 1400-471	WWF 1200-333	W 920-223	W 610-140	W 460-74	W 310-28
W 920-967	WWF 1100-458	W 840-329	WWF 800-223	W 530-138	W 410-74	W 250-28
W 1000-883	W 690-457	W 690-323	W 1000-222	W 760-134	W 310-74	W 200-27
W 690-802	W 610-455	W 1000-321	W 760-220	W 360-134	W 250-73	W 250-25
W 920-784	WWF 1000-447	W 1000-314	W 530-219	W 410-132	W 530-72	W 310-24
W 1000-749	W 920-446	W 760-314	W 690-217	W 310-129	W 360-72	W 250-24
WWF 2000-732	W 1000-443	W 920-313	W 610-217	W 460-128	W 200-71	W 150-24
WWF 1800-700	W 760-434	WWF 900-309	WWF 700-214	W 690-125	W 460-68	W 250-22
WWF 1800-659	W 840-433	W 610-307	W 460-213	W 610-125	W 460-67	W 200-22
W 920-653	W 1100-432	WWF 1100-304	W 840-210	W 530-123	W 410-67	W 150-22
WWF 2000-648	WWF 1600-431	WWF 1200-302	W 920-201	W 360-122	W 310-67	W 310-21
W 1000-641	W 690-419	WWF 800-300	WWF 1000-200	W 310-118	W 250-67	W 200-21
WWF 1600-626	WWF 1200-418	W 530-300	W 760-196	W 410-114	W 530-66	W 200-19
WWF 1800-617	WWF 900-417	W 840-299	WWF 700-196	W 610-113	W 360-64	W 250-18
WWF 2000-607	W 920-417	W 1000-296	W 530-196	W 460-113	W 460-61	W 150-18
WWF 1400-597	W 610-415	WWF 1000-293	W 610-195	W 360-110	W 460-60	W 200-15
W 1000-591	W 1000-414	W 920-289	W 840-193	W 530-109	W 410-60	W 150-14
W 920-585	W 1000-412	W 690-289	W 460-193	W 310-107	W 310-60	W 150-13
W 1000-583	WWF 1400-405	W 610-285	WWF 900-192	W 460-106	W 200-59	
W 760-582	W 1000-393	W 760-284	W 690-192	W 610-101	W 250-58	
WWF 1600-580	W 840-392	WWF 1100-273	W 760-185	W 530-101	W 360-57	
W 840-576	W 1100-390	W 1000-272	WWF 800-184	W 360-101	W 410-54	
WWF 1800-575	W 760-389	W 530-272	W 530-182	W 250-101	W 460-52	
W 1000-554	WWF 1100-388	W 920-271	W 460-177	W 410-100	W 310-52	
W 610-551	W 920-387	W 690-265	W 840-176	W 200-100	W 200-52	
W 690-548	W 690-384	WWF 1200-263	WWF 700-175	W 460-97	W 360-51	
WWF 2000-542	W 920-381	WWF 1000-262	W 610-174	W 310-97	W 250-49	
W 1000-539	WWF 1200-380	WWF 900-262	W 760-173	W 610-92	W 410-46	
WWF 1600-538	WWF 1000-377	W 610-262	W 690-170	W 530-92	W 200-46	
W 920-534	W 610-372	W 460-260	WWF 900-169	W 610-91	W 360-45	
W 760-531	W 1000-371	W 760-257	W 530-165	W 360-91	W 310-45	
W 840-527	W 920-365	W 920-253	W 360-162	W 460-89	W 250-45	
WWF 1400-513	W 840-359	WWF 800-253	WWF 800-161	W 250-89	W 200-42	
WWF 1800-510	WWF 1400-358	W 840-251	W 760-161	W 310-86	W 410-39	
W 690-500	WWF 1100-351	W 1000-249	W 460-158	W 200-86	W 360-39	
W 1100-499	W 1000-350	W 530-248	W 610-155	W 530-85	W 310-39	
W 610-498	W 760-350	WWF 700-245	W 610-153	W 410-85	W 250-39	
WWF 1600-496	W 690-350	W 610-241	WWF 700-152	W 610-84	W 150-37	
W 1000-493	WWF 900-347	W 690-240	W 690-152	W 610-82	W 200-36	
W 920-488	W 920-345	W 920-238	W 530-150	W 530-82	W 360-33	
WWF 1200-487	W 1100-342	W 460-235	W 410-149	W 460-82	W 310-33	
W 1000-486	W 920-342	WWF 1100-234	W 760-147	W 250-80	W 250-33	
W 760-484	W 610-341	WWF 900-231	W 360-147	W 360-79	W 310-31	
W 1000-483	WWF 1000-340	W 840-226	W 460-144	W 310-79	W 200-31	
W 840-473	WWF 800-339	WWF 1000-223	W 690-140	W 530-74	W 150-30	

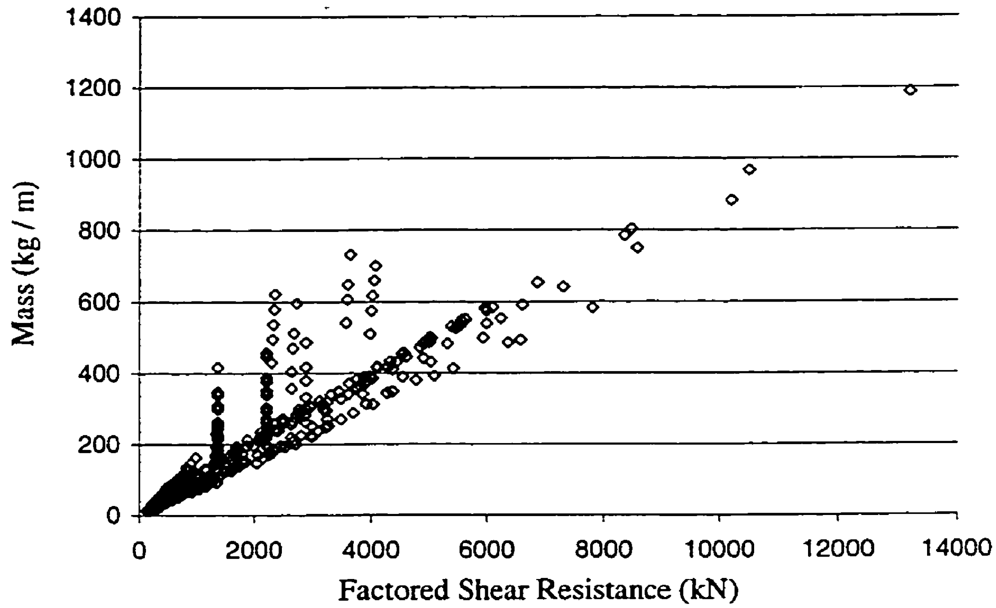


(a) All sections in Table 3.A.7

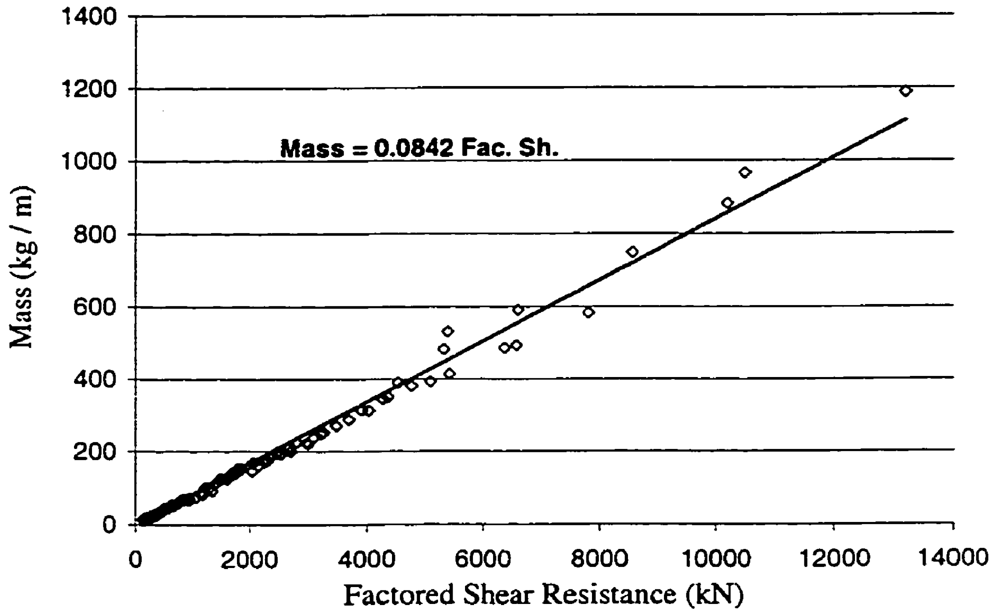


(b) Economical moment sections in Table 3.A.7

Figure 3.A.9: Mass vs. Factored Moment Resistance in Steel Spandrel Beams



(a) All sections in Table 3.A.7



(b) Economical shear sections in Table 3.A.7

Figure 3.A.10: Mass vs. Factored Shear Resistance in Steel Spandrel Beams

3.A.5 Cost of Stairs

The width and number of risers for stairs are functions of the story height and the floor plan dimensions. Since the cost of steel and concrete stair cases are almost equal (Mean's manual 1999), and since steel stair cases have the advantage of being easily constructible for both steel and concrete structures, this study only considers steel stair cases. Figure 3.A.11 represents the relation between the cost of a 1.2 meter wide steel stair case and the number of risers. The cost of the stair cases for all *NF* floors of a building is found as (Mean's manual 1999),

$$Cost_{\,Stair\,case} = NSC \times \frac{WSC}{1.2} \times NF \times SCLF \times (181.88 \times NRSC + 1320) \qquad (3.A.10)$$

where *NSC* and *WSC* are the number and width of stair cases, respectively, *NRSC* is the number of risers between floors, and *SCLF* is the steel cost location factor.

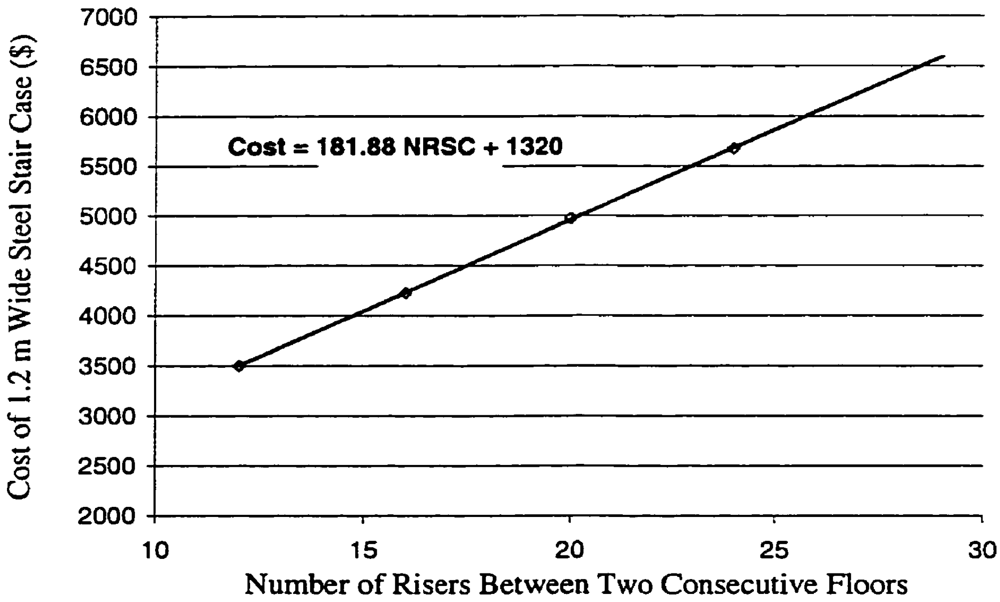


Figure 3.A.11: Cost of 1.2 m Wide Stair Case vs. Number of Risers

3.A.6 Cost of Façade and Roofing

The cost of the façade comprises the cost of windows and cladding, assuming that mechanical floors do not have any windows. The roofing cost comprises the cost of material and workmanship involved in insulating the roof of the building. Table 3.A.8 presents unit costs and some properties of window, cladding and roofing elements. Corresponding costs are represented by the products of the areas of windows, cladding and floor plan times the unit costs of windows, cladding and roofing, respectively. These areas and costs are found as,

$$Area_{Window} = WIR \times (D_a + D_b) \times 2 \times RNF \times (h_{cle} - 1) \tag{3.A.11a}$$

$$Area_{Cladding} = (D_a + D_b) \times 2 \times H - Area_{Window} \tag{3.A.11b}$$

$$Area_{Roof} = D_a \times D_b \tag{3.A.11c}$$

$$Cost_{Window} = Area_{Window} \times WCLF \times Window_{Unit\ cost} \tag{3.A.12a}$$

$$Cost_{Cladding} = Area_{Cladding} \times C_LCLF \times Cladding_{Unit\ cost} \tag{3.A.12b}$$

$$Cost_{Roofing} = Area_{Roof} \times R_OCLF \times Roofing_{Unit\ cost} \tag{3.A.12c}$$

where WIR and RNF are window ratio and rentable number of floors, respectively, $WCLF$, C_LCLF and R_OCLF are window, cladding and roofing cost location factors, respectively, and h_{cle} is the floor-to-ceiling clearance distance.

Table 3.A.8: Unit Costs and Properties of Building Envelope Components

Cladding and Window types	Cost $/m^2	Thermal Transmittance W/m^2 K	Shading Coefficient (Unit less)
Pre-cast concrete	215	0.44	-
Metal siding panel	90	0.71	-
Stucco wall	105	0.69	-
Glazing panel	235	0.75	-
Standard glass	285	6.3	0.95
Insulated glass	310	3.5	0.82
Standard HA	305	6.3	0.71
Insulated HA	330	3.5	0.56
Roofing	63	0.7	-

3.A.7 Cost of Finishing and Partitioning

Tenants in office buildings usually pay for their own interior office partitions and finishes, while the owner/developer pays for the exterior shell of the building and the main interior walls, including toilet partitions and elevator walls, floor and ceiling finishes, and finishes required for interior surfaces of exterior walls (The Toronto Real Estate Board 1999). As such, the cost function adopted by this study for finishing and partitioning pertains only to that paid for by the owner/developer (i.e., rental area finishing costs are not considered). The finishing cost, then, is the product of the floor and wall surface areas times finishing unit costs, with account for the prevailing cost location factor, i.e.,

$$Cost_{\,Finishing} = (NF \times D_a \times D_b \times Finishing_{\,unit\,cost} + NF \times ISA \times Wall\ Finishing_{\,unit\,cost}) \times F_l CLF \qquad (3.A.13)$$

where ISA is the interior surface area of exterior walls and $F_l CLF$ is the cost location factor for finishing.

3.A.8 Cost of HVAC System

The cost of the HVAC system for a building includes a cost that is directly based on the size of the floor area (i.e. costs for plumbing, ducts, fan units, and water sprinkler), i.e.,

$$Cost_{\,plumbing,\,ducts,\,fan\,units,\,water\,sprinkler} = NF \times D_a \times D_b \times MCLF \times Mechanical_{\,unit\,cost} \qquad (3.A.14)$$

where *MCLF* is the mechanical cost location factor. Other HVAC costs for boilers, chillers and related components involve more detailed calculation, as described in the following.

To establish an accurate estimate of the cost of the HVAC system for a building it is necessary to calculate its heating and cooling loads, which are defined by the amount of energy per unit time that must be given to or removed from the building in order for its environment to be acceptable to the occupants. The HVAC heating and cooling loads are functions of the building dimensions, exterior walls, window material and area, the external environmental conditions, the desired inside temperature and humidity, and the geographical location and orientation of the building. In lieu of an exact analysis to establish the heating and cooling loads (which involves considering every day of the year), this study only focuses on twelve representative days corresponding to the twelve months of a year, which results in an acceptable estimation of maximum cooling and heating loads.

The first step taken to calculate HVAC heating and cooling loads involves finding the outside temperatures and the energy given to the building from sun radiation at any hour of the twelve sample days. To this end, this study proposes the use of sinusoidal functions in conjunction with American Society of Heating, Refrigerating and Air-Conditioning Engineers guidelines (ASHRAE, 1989) to estimate the maximum and minimum temperatures $TMAX_m$ and $TMIN_m$ for any given sample day, and the temperature for any given hour of the day, knowing only the annual maximum and minimum temperatures and their daily ranges. The sinusoidal functions used to estimate $TMAX_m$ and $TMIN_m$ for any of the twelve sample days are,

$$TMAX_m = \frac{AHDT_{max} + ACDT_{max}}{2} + \left|\frac{AHDT_{max} - ACDT_{max}}{2}\right| \times COS\left(\pi \frac{m-7}{6}\right) \qquad (3.A.15a)$$

$$TMIN_m = \frac{AHDT_{min} + ACDT_{min}}{2} + \left|\frac{AHDT_{min} - ACDT_{min}}{2}\right| \times COS\left(\pi \frac{m-7}{6}\right) \qquad (3.A.15b)$$

where $AHDT_{max}$ and $AHDT_{min}$ are the average maximum and minimum temperatures for a hot day in July, and $ACDT_{max}$ and $ACDT_{min}$ are the average maximum and minimum temperatures for a cold day in January, and subscript m = 1,....,12 where 1= January and 12= December.

Since the earth moves around the sun in an almost circular motion, Eqs. (3.A.15) estimate the changes in temperature over the year very well. Figure 3.A.12 illustrates the close proximity between the temperatures found from Eqs. (3.A.15) and the actual change in outside temperature for New York City; $TMAX_m$ and $TMIN_m$ are shown by continuous lines superimposed on the actual air temperatures in broken lines (Olgyay 1992).

The sinusoidal function used to estimate hourly temperatures for any given day of the twelve sample days is,

$$T_{mh} = \frac{TMAX_m + TMIN_m}{2} + \left|\frac{TMAX_m - TMIN_m}{2}\right| \times COS\left(\pi \frac{h-15}{12}\right) \qquad (3.A.16)$$

where T_{mh} is the outside temperature for any $h = 1^{st},\ldots,24^{th}$ hour of the day in any given month ***m***. Eq. (3.A.16) ensures that the maximum and minimum daily temperatures occur at 3pm and 3am, respectively, which is very close to reality. The ASHRAE (1989) hand book of HVAC fundamentals suggests the use of a set of constants in order to estimate the change of air temperature within a day. Figure 3.A.13 shows that the hourly changes in temperature found using the ASHRAE constants compare well with those found using Eq. (3.A.16).

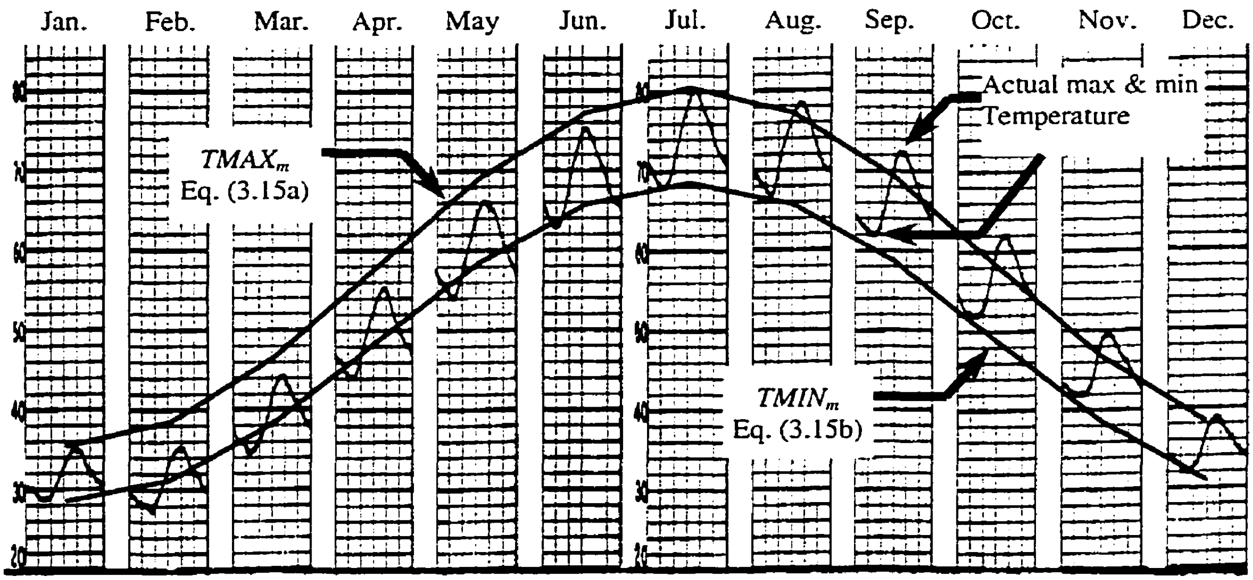


Figure 3.A.12: Comparison Between Sinusoidal Function and Actual Climate Change

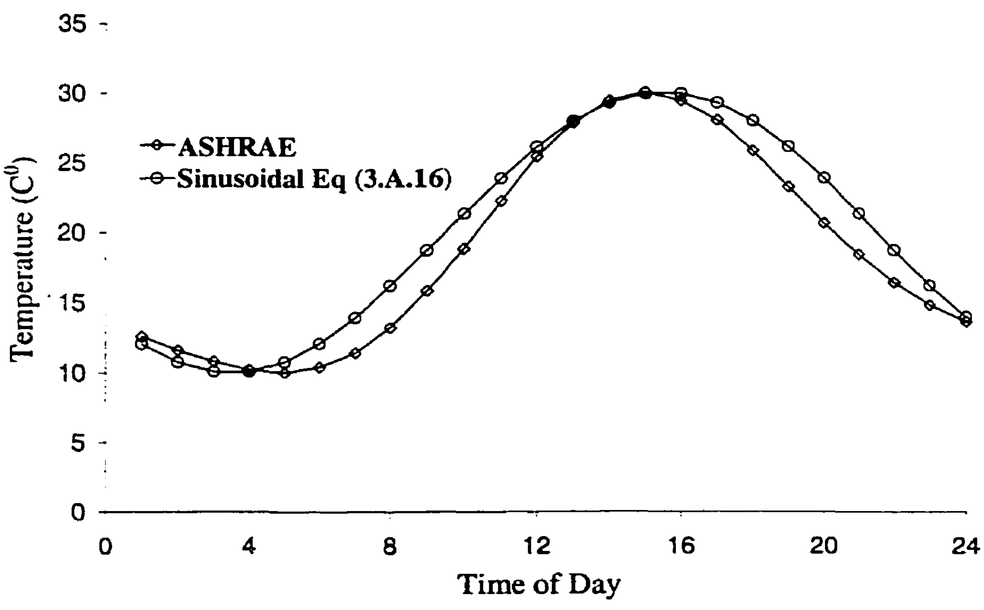


Figure 3.A.13: Change of Temperature During 24 Hours of a Day

Temperatures found using Eqs. (3.A.15) and (3.A.16) are directly used to calculate heat gain or loss through ventilation and conduction of windows. The additional gains and losses of energy due to solar radiation are estimated by increasing or decreasing the temperature on the surface of the cladding and roof and calculating the solar heat gain through the windows (ASHREA 1989). Solar heat gain or loss caused by radiation of the sun and radiation of the building at night to the clear sky are functions of the geographical location and orientation of the building (ASHREA 1989). In lieu of a rigorous method to calculate the heating and cooling loads for an office building, this study uses an approximate method that divides the building into four zones (Figure 3.A.14) and then conducts an analysis of each zone to find out if it needs to be heated or cooled at any given time over the day. After establishing heating and cooling loads in this way for all 24 hrs of the twelve sample days, the maximum heating and cooling loads for the building are then found by combining the loads of the four zones.

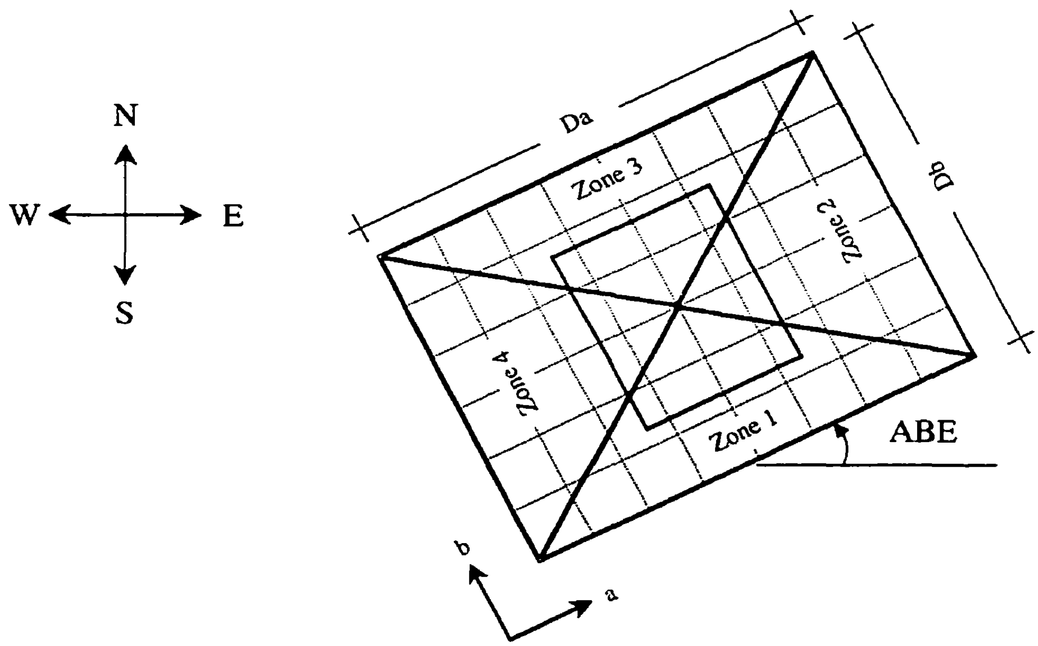


Figure 3.A.14: Air Conditioning Zoning for a Typical Building

The loads imposed on the building by the occupants, lighting system and equipment should also be considered in addition to the heating energy injected into the building by ventilation, conduction of the building envelope and solar heat gain through the windows and walls. Albeit, to conservatively establish the heating load on a cold day, this additional heat is typically neglected to arrive at the worse-case scenario.

This study employs the ASHRAE (1989) guidelines to establish the annual heating and cooling loads for a typical building having the following properties:

$D_a = 50$ m
$D_b = 40$ m
$H = 124$ m
$RNF = 30$
WIT= Insulated HA
$WIR = 50\%$
WAT= precast concrete
$AOT_{max} = 30\ ^{0}C$
$HDTR = 20\ ^{0}C$
$AOT_{min} = -15\ ^{0}C$
$HDTR = 10\ ^{0}C$
$DIT = 20\ ^{0}C$
$IRH = 50\%$
$LA = 40\ ^{0}$ N
$ABE = 0\ ^{0}$

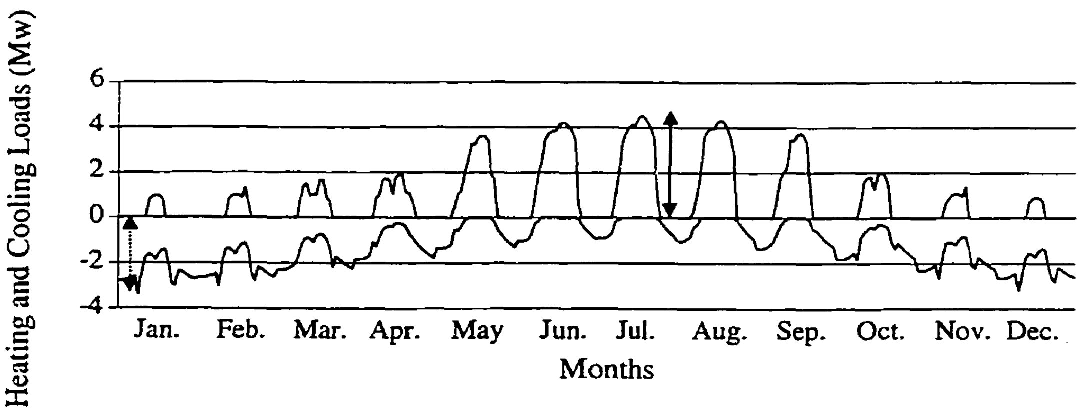


Figure 3.A.15: Heating & Cooling Loads vs. Time for a Typical Building

The annual heating and cooling loads are illustrated in Figure 3.A.15, where the vertical dashed and continuous double arrows indicate the magnitudes of the maximum heating and cooling loads, respectively. Having the annual heating and cooling loads for a building, such as in Figure 3.A.15, the costs of boilers and chiller/cooling towers are estimated as,

$$Cost_{Boilers} = Annual\ Heating_{Load} \times MCLF \times Boilers_{unit\ cost} \tag{3.17a}$$

$$Cost_{Chillers\ \&\ cooling\ towers} = Annual\ Cooling_{Load} \times MCLF \times Chillers\ \&\ Cooling\ tower_{unit\ cost} \tag{3.17b}$$

where $MCLF$ is the mechanical cost location factor.

3.A.9 Cost of Elevators

The number of elevators is a function of the total building floor area rather than the number of stories or floor plan area. This is because the number of elevators is kept constant, while their speed is increased, as the building height increases. This results in higher cost for elevators in taller buildings, mainly because of higher costs for motors and gears required to accommodate faster speeds over longer distances. As such, the cost of each elevator is a function of the number of stories and the type of elevator. In this study, only one type and size of elevator is considered; the US national average cost of one such elevator vs. the number of stories is shown in Figure 3.A.16. The approximate calculation of the cost of elevators for a building with NF floors is given by (Mean's manual 1999),

$$Cost_{Elevators} = NE \times E_L CLF \times (8962.5 \times NF + 119625) \tag{3.A.18}$$

where NE is the number of elevators and $E_L CLF$ is the elevator cost location factor.

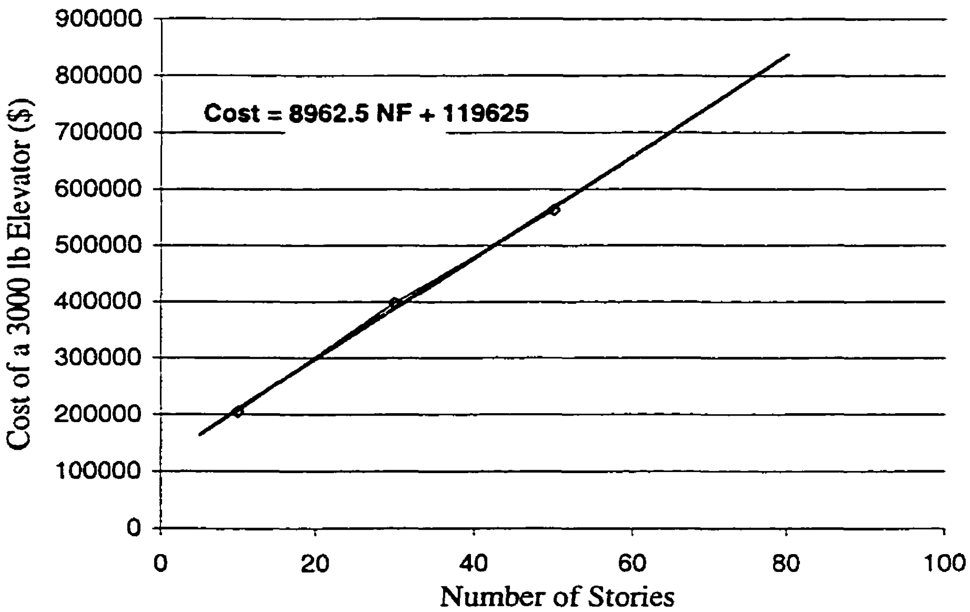


Figure 3.A.16: Cost of a 3000 lb Elevator vs. Number of Stories

3.A.10 Cost of Lighting System

The cost of the lighting system, including electrical outlets, is calculated as the product of the floor area times the electrical system unit cost with account for the prevailing cost location factor, i.e.,

$$Cost_{Lighting} = NF \times D_a \times D_b \times ECLF \times Electrical_{unit\ cost} \tag{3.A.19}$$

where *ECLF* is the cost location factor for the electrical system.

Appendix 3.B - Annual Operating Cost (Eq. 3.3)

3.B.1 Annual Cost of Energy

The first step to approximate the annual cost of energy is to estimate the energy consumed by the HVAC, lighting, elevator and equipment systems. To establish the amount of energy consumed by the HVAC system, it is necessary to find the energy needed to heat up or cool down the building at any given time over the year. The sum of these heating and cooling energies represent the energy that is input to and removed from the building in one year. It is recommended to include all heat gain from the sun and internal sources to arrive at an accurate estimation of annual heating energy (ASHRAE 1989). Since it is not realistic to assume a clear sky at all times, which causes over-estimation of heat gains and losses, this study employs a clear sky factor found from local environmental information to reduce both the temperature increase of cladding and the incoming energy through windows due to solar radiation. Figure 3.B.1 demonstrates the added heating and removed cooling energies for a typical building example at any hour of the twelve sample days. The area above the solid line represents the heating energy, while the area beneath the broken line signifies the cooling energy. Heating and cooling energies found from Figure 3.B.1 are multiplied by the average number of days in a month to arrive at the energy consumed for the entire year. The energy operating cost for a HVAC system is found as,

$$Operating\ Cost_{HVAC\ energy} = Annual_{heating\ energy} \times Gas\ energy_{unit\ cost} + Annual_{cooling\ energy} \times Electrical\ energy_{unit\ cost} \quad (3.B.1)$$

where the heating and cooling energy unit costs are defined by the unit costs of gas and electricity, respectively.

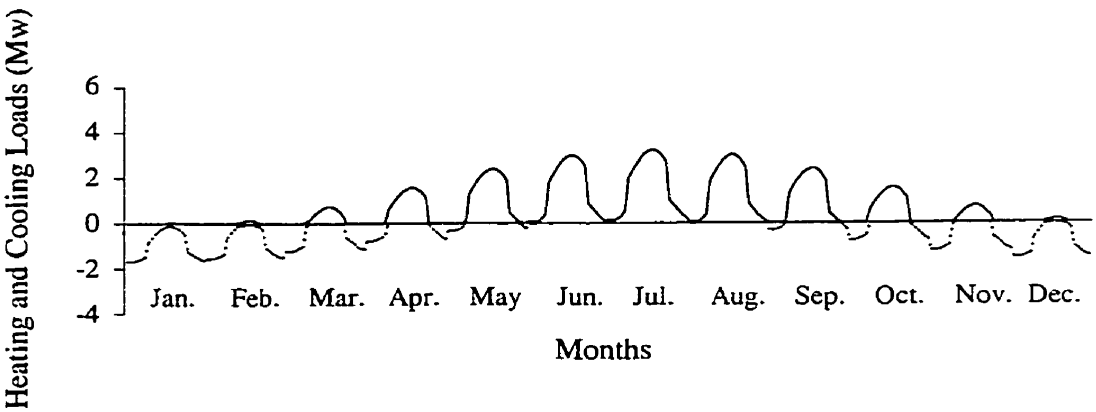


Figure 3.B.1: Heating & Cooling Energies vs. Time for a Typical Building

To arrive at the total annual cost of energy, the cost of energy consumed by lighting and elevator systems and office equipment must be added to the HVAC energy cost. To this end, Table 3.B.1 represents the energy load for these systems during working and non-working hours. Hence, the operating energy cost for the lighting, and elevator equipment systems is found as,

$$Operating\ Cost_{Lighting,\ elevators\ and\ equipment\ energy} = Annual\ Energy_{Lighting,\ elevators\ and\ equipment} \times Electrical\ energy_{unit\ cost} \quad (3.B.2)$$

where the annual energy for lighting, elevators and equipment is the sum of the energy they individually consume in working and non-working hours over a year.

Table 3.B.1: Energy Loads for Lighting, Elevator and Equipment Systems

	Lighting		Elevators and Equipment	
	Working hrs.	Non-working hrs.	Working hrs.	Non-working hrs.
Energy Load (w/m^2)	20	10	15	7.5

3.B.2 Annual Maintenance Cost.

The annual cost of building maintenance work is a function of the upkeep costs for the mechanical and electrical systems, facade and roofing, and the building's finishing. No annual maintenance cost is associated with structural components since they are protected by the building shell and theoretically designed to last indefinitely (CSA, Canadian Standards Association, 1995). This study finds the annual maintenance cost for a building as,

$$\textit{Operating Cost}_{\textit{Maintenance}} = \textit{Partial Building Capital Cost} \times \textit{Maintenance Factor} \quad (3.B.3)$$

where the partial building capital cost refers only to those systems or components of the building that are in need of maintenance, and the maintenance factor is a fixed percentage of that capital cost (e.g., see Table 4.1).

3.B.3 Annual Property Tax

The operating cost associated with annual property tax is usually a function of the value of the building and the municipality tax rate (The Toronto Real Estate Board 1999), i.e.,

$$\textit{Operating Cost}_{\textit{Tax}} = \textit{Building Value} \times \textit{Tax Rate} \quad (3.B.4)$$

where tax rates are defined by municipal authorities and value of the building is a function of the locality and leaseable area of the building.

Appendix 3.C - Genetic Algorithms

Genetic algorithms (GA's) are search algorithms based on the principles of natural selection (survival of the fittest) and genetics. They involve structured yet randomized exchanges of information among candidates of a population of solutions that progressively improve the average fitness of the population until convergence occurs to a 'best' solution. While the operations of genetic algorithms are randomized they are by no means simple random walks. They efficiently exploit historical information to speculate on new search directions for which improved fitness is expected to occur for candidate solutions. Since the development of GA's by Holland and his colleagues (1975), they have been applied to commerce, engineering, mathematics, medicine, and pattern recognition with promising results (Goldberg 1989). A number of studies have efficiently applied GA's to optimum structural design (e.g., Adeli et al., 1993; Grierson and Pak, 1993; Jenkins, 1994; etc), conceptual design (Goldberg, 1991; Mathews et al.,1994; Grierson, 1997), and multi-criteria optimization (Gero et al.,1995; Kundu, 1996; Park and Grierson, 1997).

Genetic algorithms work with a coding (e.g., binary) of the variables, not the actual variables themselves. This makes them computationally well suited for treating discrete variables. However, they can treat continuous variables when the required precision is specified. Moreover, as GA's work simultaneously with a population of solutions, they are able to operate in multi-modal solution spaces without the need for gradient information. In essence, they use directed random choice as a tool to guide the search toward regions of the space having more desirable values for the prevailing objective function(s) for the problem.

For binary coding, each solution in the population of solutions is represented by a bit string, the length of which depends on the cardinal number of the bit, the number of variables, and the number of discrete values that each variable can assume. For example, consider a conceptual design problem for a high-rise office building having primary design variables whose base-10 and binary (base-2) values are as shown in Tables 3.1, and 3.2, respectively. From Table 3.1, variable ST has 10 possible choices, variables S_a, S_b and WIR each have 16 choices, variables NS_a, NS_b and CDF each have 8 choices, variables CFT, SFT, TS_a, TS_b, WIT, and WAT each have 4 choices values and, finally, variables BT and $DCDD$ each have two possible choices. Therefore, the variables ST, S_a, S_b, and WIR can each be represented by a 4-bit binary code, the variables NS_a, NS_b and CDF by a 3-bit binary code, the variables CFT, SFT, TS_a, TS_b, WIT and WAT by a 2-bit binary code and, finally, the variables BT and $DCDD$ can each be represented by a 1-bit binary code. Hence, any one design is represented by a 4x4+3x3+6x2+2x1= 39-bit string. Note that only some parts of this binary code is applicable for any given design in that some information is not applicable for certain structure types (as indicated by NA below). For example, the binary code for a particular conceptual design of an office building may be:

ST	BT	CFT	SFT	S_a	S_b	NS_a	NS_b	NTS_a	NTS_b	$DCDD$	CDF	WIT	WIR	WAT
0011	0	10 NA	11	1010	0111	101	011	01 NA	10 NA	1	100	10	1001	11

which, from Tables 3.1 and 3.2, decodes as a steel rigid frame with a composite steel beam and deck and concrete slab floor system having 8 and 6 spans of 9.5 m and 8.0 m length in the *a* and *b* directions, respectively, a core having a *b* dimension that is 0.564 times the *b* dimension of the building foot print, standard heat absorbing windows, a 70% window ratio and a glazed panel cladding system.

The GA commences by randomly selecting an initial population of arbitrary solutions (e.g., a population of 39-bit strings). The relative fitness F of each solution is assessed through its performance fitness function,

$$F = F_{max} - \Phi(x) \tag{3.C.1}$$

where F_{max} is an arbitrarily large positive number that ensures the fitness is always positive, $\Phi(x)$ is an objective function with built-in penalties reflecting any constraint violations for a solution, and $x = [x_1 \; x_2 \; \ldots . x_n]$ represents the variable vector for the problem. Having the fitness of all designs, genetic operators are then applied to create a new population of solutions having better average fitness. The three most commonly used operators are: selection (parents); crossover (simulated mating); and mutation (random diversity).

The reproduction cycle consists of the selection and crossover operations and is the heart of the genetic algorithm that creates new and, probably, fitter solutions. Selection is the process of choosing parents from the current population for subsequent mating to create offspring for the next generation. There are several selection techniques, such as pure-random, fit-fit, fit-weak and roulette wheel methods (Chambers, 1995). In the pure random method, the parents are selected from the population at random. Fit-fit selection pairs an individual with the next fittest individual in the population by simply searching through the list of individuals. In fit-weak selection, the fittest individual is paired with the least fit, the next fittest is paired with the next least fit, and so on. The weighted roulette wheel method, which is a traditional GA selection technique, operates such that each solution occupies a portion of the weighted roulette wheel in proportion to its relative fitness. A random number is then generated and used to select a parent solution from the roulette wheel. Those solutions of high proportional fitness have a high probability of being selected as parent solutions, while those of medium and low proportional fitness have average and lower selection probability, respectively. That is, individuals of high fitness may be selected (reproduced) a number of times, those of medium fitness may be reproduced singly, and those of low fitness may not be reproduced at all in the selection process.

After the selection procedure is complete, the crossover operator is applied to create a random interchange of information between randomly paired parents. This operator carries out a structured data exchange that recombines the parent solutions according to a specified probability, using either one-site or multi-site crossover. For example, two-site crossover involves randomly selecting two splicing sites for a pair of parent solution strings, and then exchanging the information located between the two sites between the two parents. The two new strings so formed are called "children" solutions and become members of the next population. For example, for the conceptual design of the previously described high-rise building example, if a pair of parent designs and the splice (exchange) sites are as follows:

	ST		BT	CFT	SFT	S_a	S_b	NS_a	NS_b	NTS_a	NTS_b	DCDD	CDF		WIT	WIR	WAT
Parent A:	*00*	*11*	*0*	*10*	*11*	*1010*	*0111*	*101*	*011*	*01*	*10*	*1*	*10*	*0*	*10*	*1001*	*11*
Parent B:	01	00	1	11	00	1100	0011	001	110	11	00	0	00	1	01	1010	00

Exchange Site (ST) ... Exchange Site (CDF)

Then, after crossover the two child designs are:

	ST		BT	CFT	SFT	S_a	S_b	NS_a	NS_b	NTS_a	NTS_b	DCDD	CDF		WIT	WIR	WAT
Child A:	*00*	00	1	11	00	1100	0011	001	110	11	00	0	00	*0*	*10*	*1001*	*11*
Child B:	01	*11*	*0*	*10*	*11*	*1010*	*0111*	*101*	*011*	*01*	*10*	*1*	*10*	1	01	1010	00

Exchange Site (ST) ... Exchange Site (CDF)

That is, after exchanging genes, from Tables 3.1 and 3.2, Parent A (*ST* = steel rigid frame, *SFT* = composite beam, deck and slab, S_a = 9.5m, S_b = 8.0m, NS_a = 8, NS_b = 6, C_b = $0.564 \times S_b \times NS_b$, *WIT* = standard heat absorbing glass, *WIR* = 70% and *WAT* = glazed panel) and Parent B (*ST* = steel frame and bracing, *BT* = K&X, *SFT* = steel joist & beam, deck & slab, S_a = 10.5 m, S_b = 6.0 m, NS_a = 4, NS_b = 9, Ca = $0.329 \times S_a \times NS_a$, *WIT* = insulated glass, *WIR* = 75% and *WAT* = precast concrete) are replaced by Child A (*ST* = concrete rigid frame, *CFT* = waffle slab, S_a = 10.5m, S_b = 6.0m, NS_a = 4, NS_b = 9, C_a = $0.25 \times S_a \times NS_a$, *WIT* = standard heat absorbing glass, *WIR* = 70% and *WAT* = glazed panel) and Child B (*ST* = steel rigid frame and shear wall, *SFT* = composite beam and deck and slab , S_a = 9.5m, S_b = 8.0m, NS_a = 8, NS_b = 6, C_b = $0.643 \times S_b \times NS_b$, *WIT* = insulated glass, *WIR*= 75% and *WAT* = precast concrete). Note that the values of the two design variables *ST* and *CDF* are changed, the values of the three design variables *WIT*, *WIR*, and *WAT* remain constant, and that the values of the rest of the variables are simply exchanged.

Even though the selection and crossover operators effectively search the solution space, they may occasionally miss some useful genetic features. To prevent such a loss and to avoid premature convergence to a local optimum, the mutation operator is applied to each bit position of each child solution string according to a preset probability of occurrence. In the case of binary coded genes, mutation is performed by flipping the value of a gene from 0 to 1 or vice versa. Typically, the mutation probability is set quite low.

After application of the selection, crossover and mutation operators to create the next generation of new solutions, the possible convergence of the GA to an optimum solution is checked. Three convergence criteria that are often adopted are described in the following. The first criterion checks to see if there is no improvement in the maximum solution fitness for the population for a specified number of consecutive generations, at which point the GA is terminated. A second criterion terminates the search if the same number of solutions have the same maximum fitness for the population for a specified number of consecutive generations. A third criterion is sensitive to the computational time required to generate the optimal solution, and causes the GA to stop running after a pre-assigned number of generations. As a GA does not embody any formal mechanism that guarantees finding the global optimum, it is generally run several to many times for a number of different randomly generated initial populations, with the expectation that most if not all runs will converge to almost the same optimum solution.

Appendix 3.D - Pareto Optimization

It is generally considered that multi-criteria optimization originated towards the end of the nineteen century when Pareto (1848-1923) presented a qualitative definition of non-preferential optimality for multiple competing criteria (Pareto, 1896). The basic concepts of multi-criteria Pareto optimization are briefly explained in the following.

The multi-criteria optimization problem may be stated as:

Minimize: $f(x) = [f_1(x), f_2(x), \ldots f_Q(x)]^T$ (3.D.1)

Subject to : $g(x) \leq 0$, $h(x)=0$ (3.D.2)

where $x = [x_1 \; x_2 \; \ldots . \; x_n]^T$ is the vector of n variables for the problem, $f(x)$ is the vector of $i=1,2, \; ,Q$ objective functions $f_i(x)$ that are each to be minimized, and the functions $g(x) \leq 0$ and $h(x)=0$ define the inequality and equality constraints for the problem. A solution x^0 is Pareto optimal for the problem defined by Eqs. (3.D.1) and (3.D.2) if there exists no other solution x satisfying Eqs. (3.D.2) for which $f_i(x) \leq f_i(x^0)$ for $i=1,2, \; ,Q$, with $f_i(x) < f_i(x^0)$ for at least one objective criterion. In words, the solution x^0 is Pareto optimal if there exists no other feasible solution x which dominates it for all objective criteria.

The Pareto-optimal solution set is the set of solutions distributed along the Pareto-optimal surface defining the trade-off between the different objective criteria. From among a population of N solutions, the number P of solutions belonging to the Pareto-optimal solution set depends on the specific nature of the problem posed by Eqs. (3.D.1) and (3.D.2), and theoretically can be anywhere in the range of $1 \leq P \leq N$.

Chapter 4

Conceptual Design Examples

4.1 INTRODUCTION

The optimal cost-revenue conceptual designs of four office buildings are presented in this chapter to illustrate the applicability, efficiency and practicality of the computer-based multi-criteria optimization capability developed by this study. Table 4.1 lists the parameter values governing the design of the four example buildings, which differ only in their geographic locations and, thus, in their land costs, lease and tax rates and material costs. It is assumed that locations with lower land cost and tax rates have lower lease rates (e.g., see Table 4.1, where the rangers of low to high annual lease rates correspond to building that have poor to good quality of office space-see Figure 3.3).

Example 1 concerns the design of an office building that has U.S. national average unit costs for concrete and steel construction, and which is located in a city having expensive land and high lease and tax rates. Example 2 modifies the Example 1 design case by locating the building in another city that has cheaper land and lower lease and tax rates. Example 3 considers yet another design case by locating the building in a city that has a relatively high cost for steel construction compared to that for concrete construction. Conversely, Example 4 differs from Example 3 in that the building is located in a city that has a high cost for concrete construction compared to that for steel construction.

Table 4.1: Governing Parameters for Design Examples

Design Parameter	Design Example 1	2	3	4
Location Information				
Land Unit Cost ($/m^2)	12000	1000	12000	12000
Annual Lease Rates ($/m^2/yr)	300-540	100-360	300-540	300-540
Maintenance (%capital cost)	2	2	2	2
Taxes (%building value)	5	2	5	5
Mortgage Rate (%)	10	10	10	10
Inflation Rate (%)	3	3	3	3
Cost Location Factors ($/USavg$)				
Structural steel	1	1	1	0.88
Concrete	1	1	0.74	1
Reinforcement	1	1	0.79	1
Forming	1	1	0.51	1
Cladding	1	1	1	1
Windows	1	1	1	1
Roofing	1	1	1	1
Finishing	1	1	1	1
Electrical	1	1	1	1
Mechanical	1	1	1	1
Elevators	1	1	1	1
Geographical & Orientation Information				
Latitude (Degree North)	40	40	40	40
Angle of building with East (Degree)	0	0	0	0
Environmental Information				
Clear Sky Percentage (%)	75	75	75	75
Hot Day Relative Humidity (%)	80	80	80	80
Cold Day Relative Humidity (%)	50	50	50	50
Inside Temperature (C^0)	22	22	22	22
Ave. Max. Outside Temp.(C^0)	31	31	31	31
Ave. Min. Outside Temp.(C^0)	-20	-20	-20	-20
Hot Day Temp. Range (C^0)	10	10	10	10
Cold Day Temp. Range (C^0)	10	10	10	10
Load Information				
Applied Dead Load (kN/m^2)	1.45	1.45	1.45	1.45
Gravity Live Load (kN/m^2)	2.80	2.80	2.80	2.80
Wind Load Pressure (kPa)	0.48	0.48	0.48	0.48
Seismic Load	N/A	N/A	N/A	N/A
Building Limits				
Max Footprint Width (m)	70	70	70	70
Max Footprint Length (m)	70	70	70	70
Max. Building Height (m)	300	300	300	300
Min. Lease Office Space (m^2)	60,000	60,000	60,000	60,000
Fixed Core/ Footprint Area (%)	20	20	20	20
Min. Core/ Perimeter Distance(m)	7	7	7	7
Min. Aspect Ratio	0.5	0.5	0.5	0.5
Max. Slenderness Ratio	9	9	9	9
Min. Floor/ Ceiling Clearance (m)	3	3	3	3

From Table 4.1, note that: all four buildings have 60,000m^2 of lease office space; the cost of maintenance work required to maintain and upkeep the building components is taken as 2% of the capital cost of HVAC, elevator and lighting systems, finishes, facade and roofing; the annual cost of property taxes is taken as 5% of the building value for Examples 1, 3 and 4, while it is taken as 2% for Example 2; the unit dead load accounts for the weight of wall partitions, ceilings and fixtures, floor finishing, plumbing and ducting (NBCC 1990); the unit live load accounts for the weight of office equipment, furnishings and occupants (NBCC 1990); all gravity dead and live loads are applied as uniformly distributed loads over the entire building footprint area at each story level, including the roof; lateral wind loads are calculated as a function of the building surface area and the specified wind pressure; both direct and suction wind loading are applied at each story level as equivalent concentrated loads; seismic loading is assumed to be not applicable for the building designs; and that all four design examples are controlled by the same building limitations, i.e.,

- Maximum building footprint width a_{max} = 70m
- Maximum building footprint length b_{max} = 70m
- Maximum building height H_{max} = 300m
- Minimum lease office space A_{req} = 60,000m^2
- Core area $Percentage(D_a \times D_b)$ = 20% of footprint area
- Minimum distance between building core and perimeter CPD_{min} = 7m
- Minimum building aspect ratio $(D_a / D_b)_{Lower} = 0.5$ (assuming $D_a < D_b$)
- Maximum building slenderness ratio $(H / D_a)^{Upper} = 9.0$ (assuming $D_a < D_b$)

These limitations restrict the buildings to have from 15 to 80 stories which, for practical design purposes, limits the structure types that may be considered for their conceptual design to the ten choices listed in Table 3.1 (also listed are the possible choices for the floors, cladding, windows, window ratio, number of bays and corresponding span distances-see Section 3.2.3). It is assumed that each building is in a downtown city location, with zero property clearance, such that the land cost is defined by the area of the building footprint.

The basic unit costs listed in Table 4.2 are U.S. national averages (Mean's Manuals 1999). It is noted that (see Chapter 3 for full details): the finishing unit cost accounts for the cost of painting, carpets and other trim for the building in addition to the cost of the main partitions; the electrical unit cost accounts for the cost of florescent lighting required to provide an illumination level of 20 Watts/m^2, in addition to the cost of associated wiring, outlets and transformers (Mean's Manuals 1999); the HVAC unit costs account for the cost of boilers, chillers, ducts and fan rooms required to accommodate the heating and cooling loads imposed on the building by occupants, lighting, equipment, ventilation, thermal conduction through exterior walls, and thermal conduction and solar radiation through windows (the ventilation, conduction and radiation loads are defined by the clear sky, humidity and temperature factors listed in Table 4.1, and by the thermal and shading coefficients for the types of cladding and windows for the building listed in Table 3.A.8); the plumbing unit cost accounts for the cost of toilets and service fixtures, in addition to the cost of plumbing required for the HVAC and fire extinguisher systems; the energy unit cost accounts for the cost of the energy consumed by office equipment and by the HVAC, elevator and lighting systems.

The computer-based computational procedure outlined in Figure 3.4, and described in Section 3.2.6, is applied to find Pareto-optimal conceptual designs for the four example office buildings that minimize capital and operating costs and maximize revenue income. To facilitate application of the multi-criteria genetic algorithm (MGA), the primary design variable values listed in Table 3.1 are represented by their binary equivalents given in Table 3.2, and the following genetic operators and data are adopted:

- Genetic population size = 1000 conceptual designs
- Reproduction = Weighted roulette wheel simulation
- Crossover = Two-point, with 100% probability
- Mutation = Single-bit, with initial probability of 5% that gradually decreases to 2% as the genetic search progresses so as to avoid significant random changes in the genetic pool at the final stages of the search.

Convergence at the final stages of the genetic search is taken to occur when 1) the number of Pareto-optimal designs, 2) the optimum values for the three objective criteria and 3) the design located at the knee of the Pareto surface (i.e., the design closest to the point in the Pareto space having the optimum values of the three objective criteria as its coordinates) all remain relatively unchanged for 20 consecutive generations. For each of the four building examples, the MGA is run for three different initial genetic populations and the Pareto designs found at convergence of the three runs are combined together to form the corresponding overall Pareto-optimal design set.

Table 4.2: Basic Building Costs

Materials, Components and Energy	Cost
Steel Cost ($/ton)	2039
Concrete Cost ($/m^3)	143
Reinforcement Cost ($/ton)	1400
Formwork Cost ($/m^2)	45
Finishing Cost($/m^2)	130
Roofing Cost($/m^2)	63
Plumbing Cost ($/m^2)	45
HVAC Boiler Cost ($/kW)	225
HVAC Chillers Cost ($/kW)	715
Electrical System Cost ($/m^2)	121
Energy Cost Elec. ($/mWhr)	100
Energy Cost Gas. ($/mWhr)	40

All unit costs are US national averages and include account for the costs of materials, shipping, unloading, accessories and installation.

4.2 DESIGN EXAMPLE 1

One purpose of this example is to study the effect of relatively expensive land cost on the design of an office building. Upon applying the multi-criteria optimization procedure (Figure 3.4), the three different runs of the MGA converged after 147, 149 and 140 generations to find 779, 766 and 752 Pareto designs, respectively. The Pareto designs found from the three runs were then combined together to form the overall set of 815 Pareto-optimal conceptual designs for the office building indicated (by grey dots) in Figure 4.1. From among all Pareto designs for the building, the minimum and maximum lease office spaces are 60,000 m^2 and 61,740 m^2, respectively, a difference of less than 3%. The shortest Pareto design is 19 stories high and has a plan footprint that measures 70m x 60m. The tallest Pareto design is 52 stories high with a 50m x 30m plan footprint.

The 815 individual Pareto-optimal designs plotted in Figure 4.1 collectively form a three-dimensional (3-D) convex surface that represents the Pareto trade-off relationships between the objective criteria to minimize capital and operating costs and maximize income revenue (i.e., minimize 1/income revenue). Figure 4.1 is not very informative as it is, but its wealth of information becomes immediately evident when computer color filtering is used to highlight zones of the Pareto surface occupied by different architectural and structural parameters for the building. These Pareto zones identify cost-revenue trends and relationships in a graphical format that can be readily understood by architects and design engineers, as shown in the following.

The computer-generated color filtering of the 3-D Pareto surface shown in Figures 4.2, 4.3, 4.4, and 4.5 highlights the Pareto zones corresponding to the different structural types, number of stories, bay areas, and window ratios possible for the building. These colour graphs yield the interesting observation that the Pareto zones are grouped with little or no overlap (which is a direct consequence of the cost-revenue interplay occurring between the different types of Pareto-optimal conceptual designs for the building). Figures 4.2 and 4.3 indicate that among the ten structural types considered for the design (Table 3.1), only eight are suitable for this example; namely, steel frame with bracing & outriggers and concrete rigid frame with shear walls, which are the tallest Pareto-optimal designs at about 35 to 52 stories, followed by steel frame/ rigid frame with concrete shear walls at 28 to 36 stories, steel frame/ rigid frame with bracing at 21 to 29 stories, unbraced steel rigid frame at 19 to 23 stories, and unbraced concrete rigid frame at 20 stories and below. Figures 4.6, 4.7, 4.8 and 4.9 present 2-D plots of Figures 4.2, 4.3, 4.4 and 4.5, respectively, and readily provide the following cost-revenue information concerning the Pareto-optimal conceptual designs for the building.

1. Steel frame with bracing & outriggers and concrete rigid frame & shear wall structural systems result in the lowest capital cost for the building compared to that for braced steel frames and unbraced steel and concrete frames (Figures 4.2, and 4.6a,b). The reason for this is that the land cost is relatively expensive and is a major component of the overall capital cost for the building. From among the eight structural types found in the Pareto-optimal set for this example, steel frame with bracing & outriggers and concrete rigid frame & shear walls, for US national average construction costs, are the most capital cost-effective for taller buildings which, for a fixed total amount of floor space, have smaller footprint dimensions and therefore require the purchase of the least amount of land.

2. Unbraced concrete rigid frame structural systems result in the highest capital cost for the building compared to that for unbraced and braced steel frames, steel frame with shear walls, concrete rigid frame with shear walls and steel frame with bracing & outriggers (Figures 4.2 and 4.6a,b). The reason for this is that the land cost is relatively expensive and is a major component of the overall capital cost for the building. From among the eight Pareto-optimal structural types found for this example, unbraced concrete rigid frame construction is the most capital cost-effective for shorter buildings which, for a fixed total amount of floor space, have larger footprint dimensions and therefore require the purchase of the most amount of land.

3. For fixed annual revenue income, taller buildings have higher annual operating cost (Figures 4.3 and 4.7c). The reason for this is that two important components of the annual operating cost for a building are the cost of the energy required to operate the HVAC system and the maintenance cost for the HVAC system, elevators and facade. For a fixed total amount of floor space, the surface on the perimeter of the building increases as the building height increases, which increases the HVAC energy cost. In the same manner, the maintenance costs of the HVAC system, elevators and facade increase when the number of stories increases due to the increase in construction costs for these building components.

4. For fixed annual operating cost, shorter buildings have higher annual income revenue (i.e., smaller 1/ income revenue -Figure 4.7c). The reason for this is that larger bay

areas increase the flexibility of floor space usage, which increases the lease rate for office space (see Figure 3.3) and, hence, the annual income revenue for the building. For a fixed total amount of floor space, as the building height decreases the foot print area of the buildings increases, which allows for larger bay areas.

5. Buildings with smaller bay areas have smaller capital cost (Figures 4.8a,b). The reason for this is that a major component of the capital cost of the building superstructure is the cost of the floor system, which decreases as the bay area decreases.

6. Buildings with larger bays areas have bigger annual income revenue (i.e., smaller 1/revenue income -Figure 4.8c). The reason for this is that larger bay areas increase the flexibility of floor space usage, which increases the lease rate for office space and, hence, the annual income revenue for the building.

7. Buildings with lower window ratios have smaller annual operating cost (Figures 4.9a,c). The reason for this is that a major component of the annual operating cost for a building is the cost of the energy required to operate the HVAC system which, for any given structural system and number of stories, decreases as the window ratio decreases.

8. Buildings with higher window ratios have bigger annual income revenue (i.e., smaller 1/ income revenue -Figure 4.9b). The reason for this is that larger window

ratios increase the amount of natural daylight experienced indoors, which increases the space quality and the lease rate for office space (see Figure 3.3) and, hence, the annual income revenue for the building.

Depending on architectural-structural and cost-revenue preferences for the building, the foregoing information can serve to guide the design team's selection of a small subset of the Pareto-optimal conceptual designs for further detailed consideration. One such selection is those designs that first become profitable over time taking into account occupancy levels and life-cycle costing. To that end, for annual revenue income calculated over time for the occupancy levels listed in Table 3.3, for annual operating cost calculated for the entire building area regardless of the occupancy level, and assuming that the entire capital cost of the building is mortgaged, Eq. (3.12) is applied using the annual mortgage and inflation rates given in Table 4.1 to find the subset of designs identified in Figure 4.10 as first becoming profitable in the 11th year after completion of building construction. Observe from Figures 4.2, 4.3 and 4.10 that all of the profitable designs are taller buildings in the range of 32 to 36 stories having steel frame/ rigid frame with concrete shear wall and concrete rigid frame with shear wall structural systems. The design team may select the first profitable design indicated (by a black dot) in Figure 4.10 and shown in Figure 4.11 as the basis for further preliminary/final design calculations. It is noted that the design shown is Figure 4.11 need not be the only design so considered, but that any of the first-profitable designs indicated in Figure 4.10 may be studied further, as may be any other Pareto-optimal design in Figure 4.10 depending on the preference of the design team.

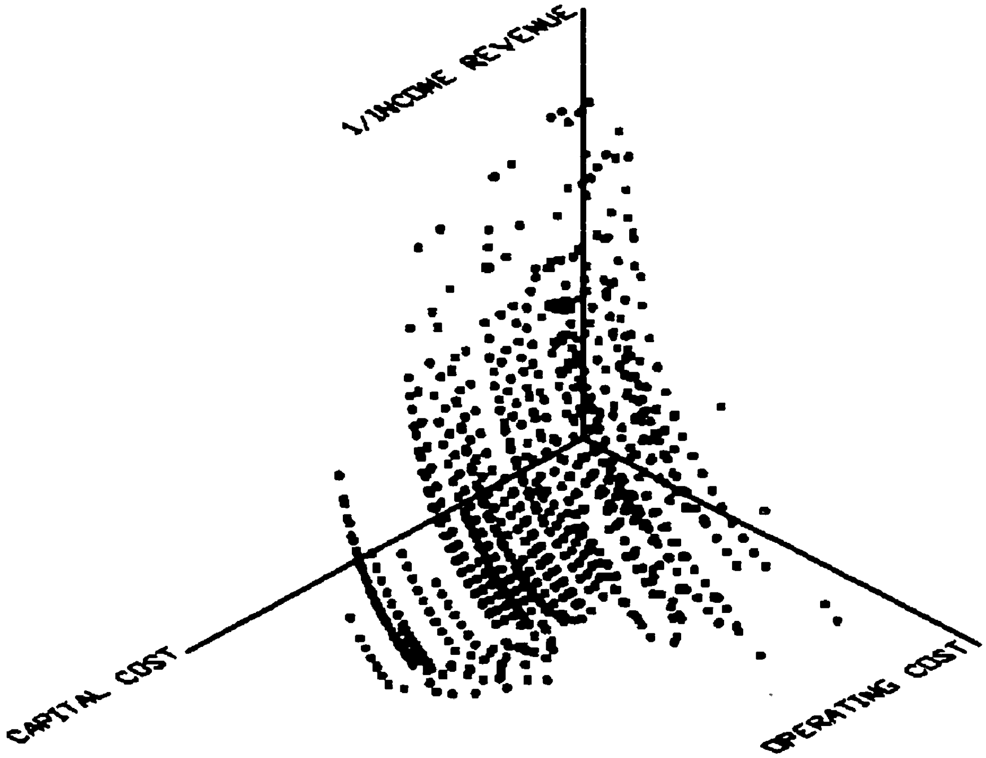


Figure 4.1: Example 1- 3D Pareto Design Space

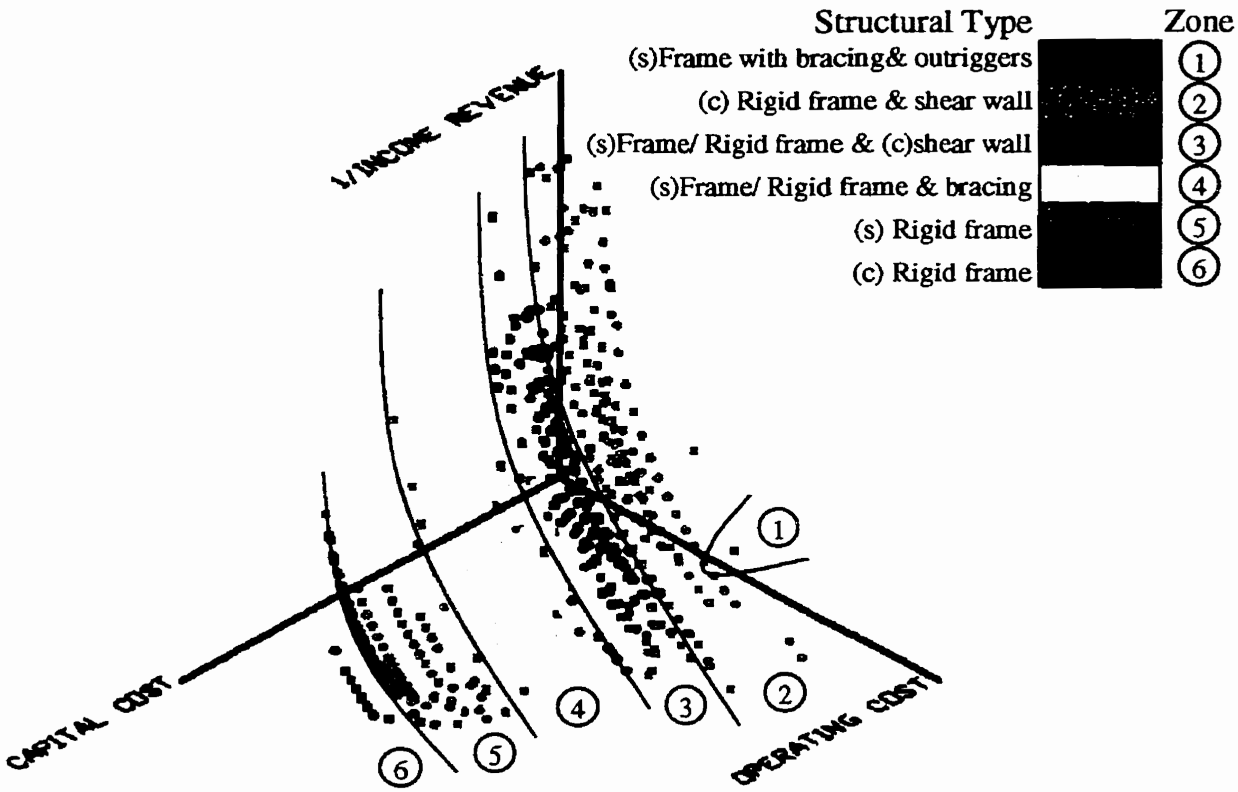


Figure 4.2: Example 1- Structural Type Pareto Zones

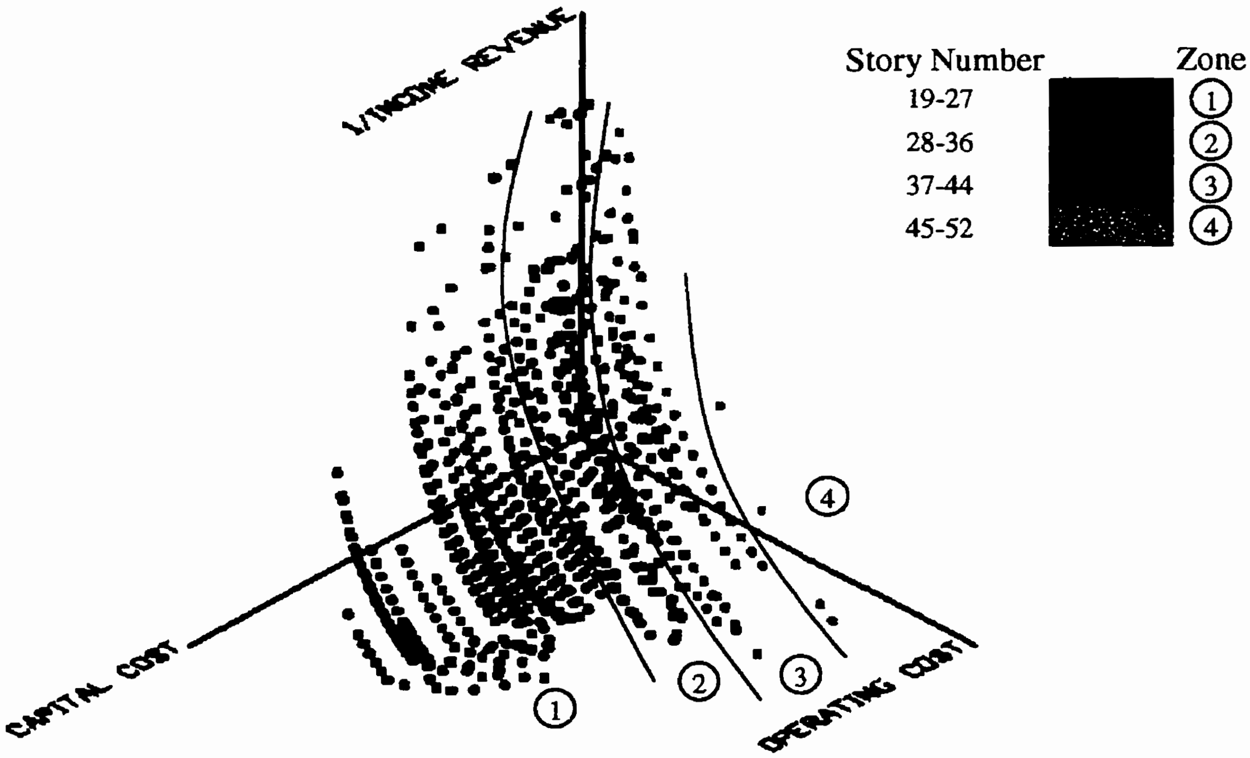


Figure 4.3: Example 1- Story Number Pareto Zones

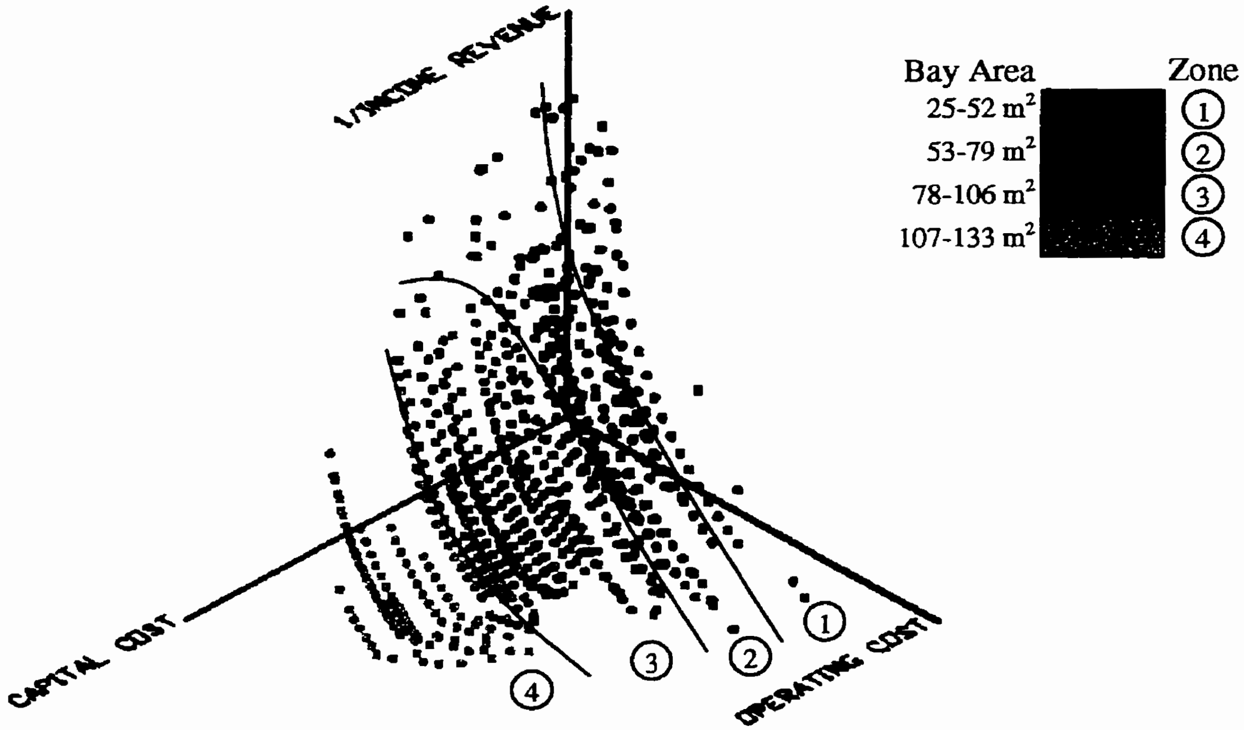


Figure 4.4: Example 1- Bay Area Pareto Zones

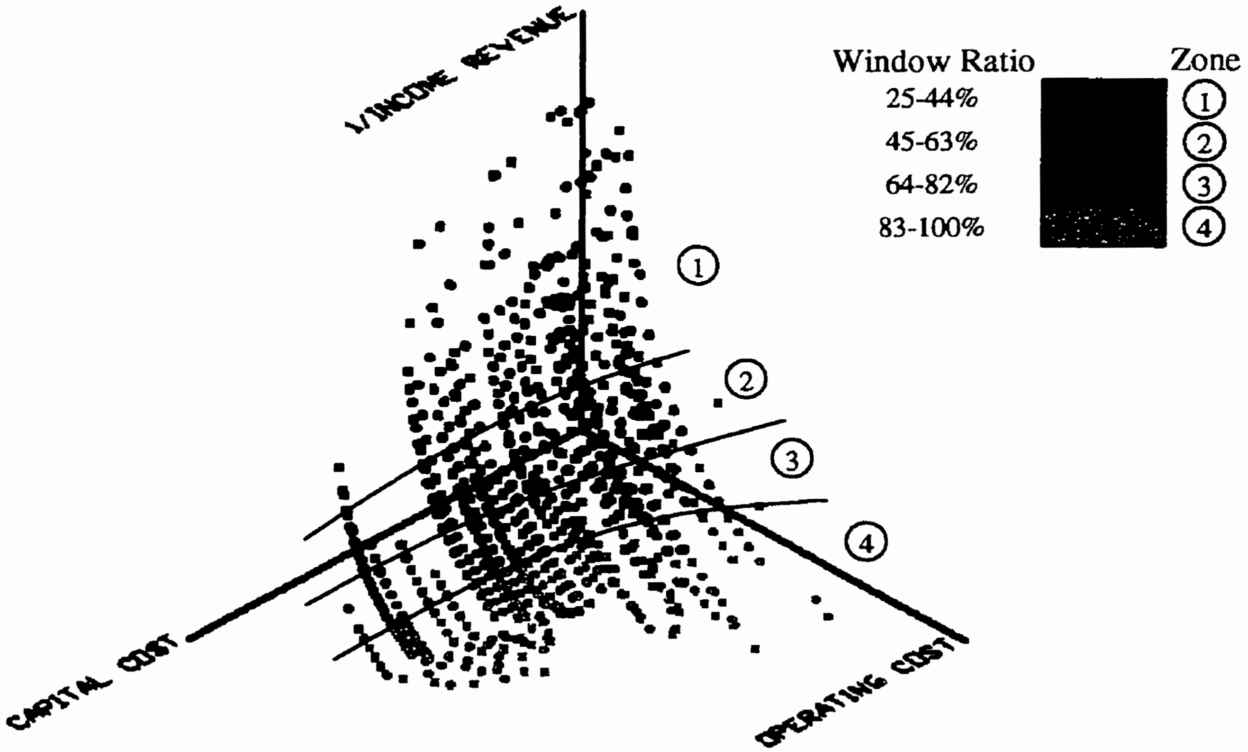


Figure 4.5: Example 1- Window Ratio Pareto Zones

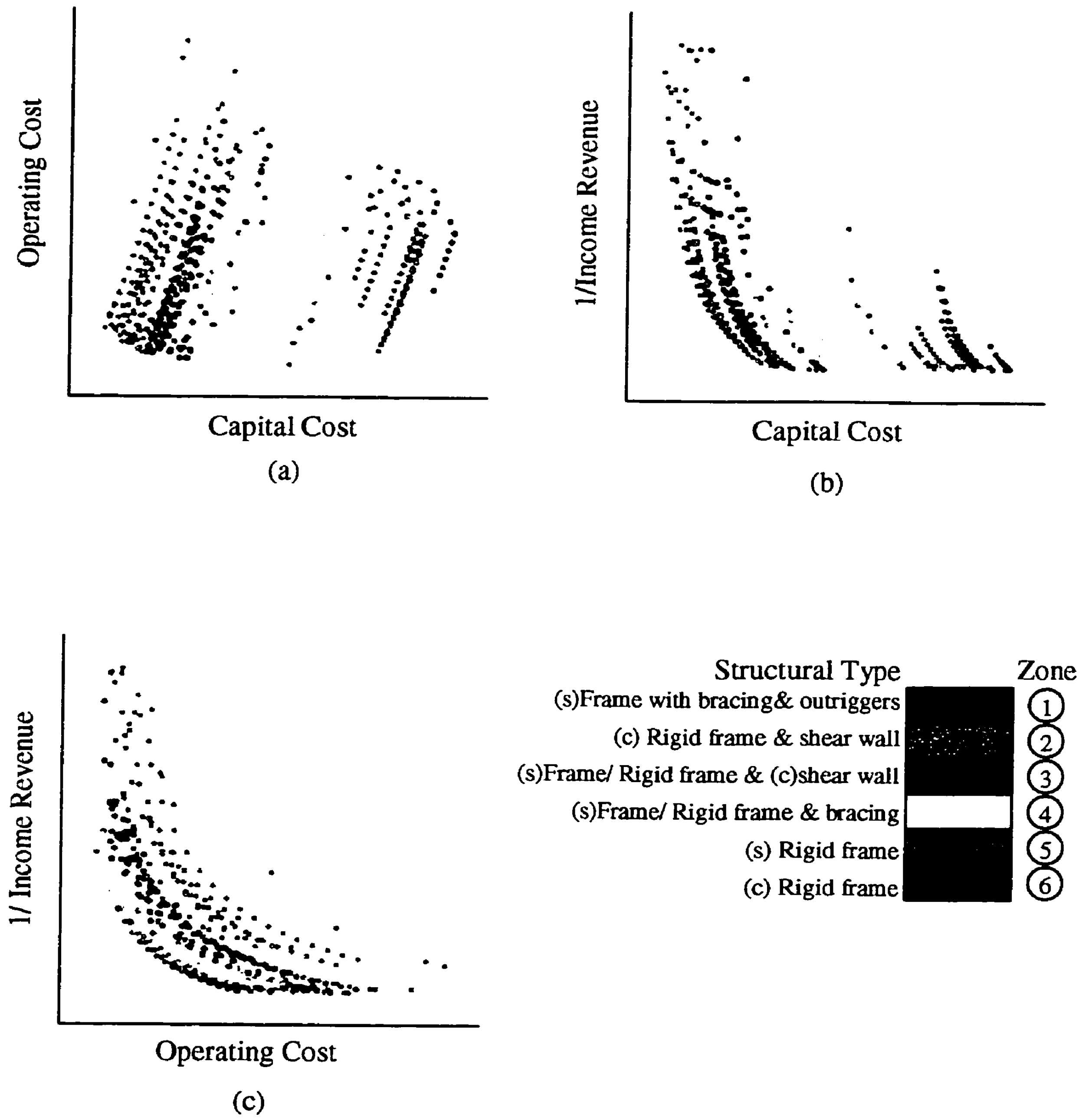


Figure 4.6: Example 1- 2D Plots of Structural Types Pareto Zones

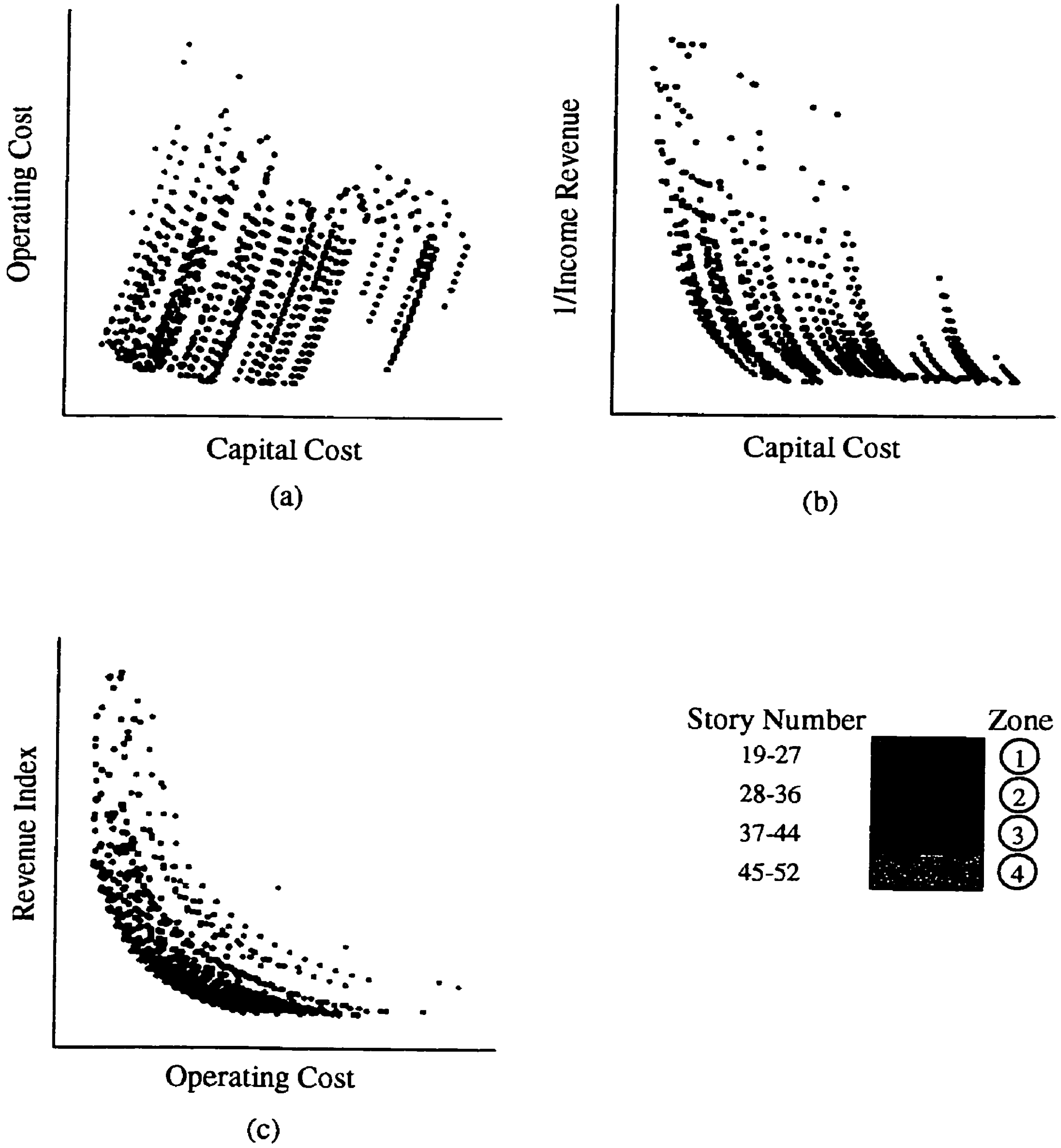


Figure 4.7: Example 1- 2D Plots of Story Number Pareto Zones

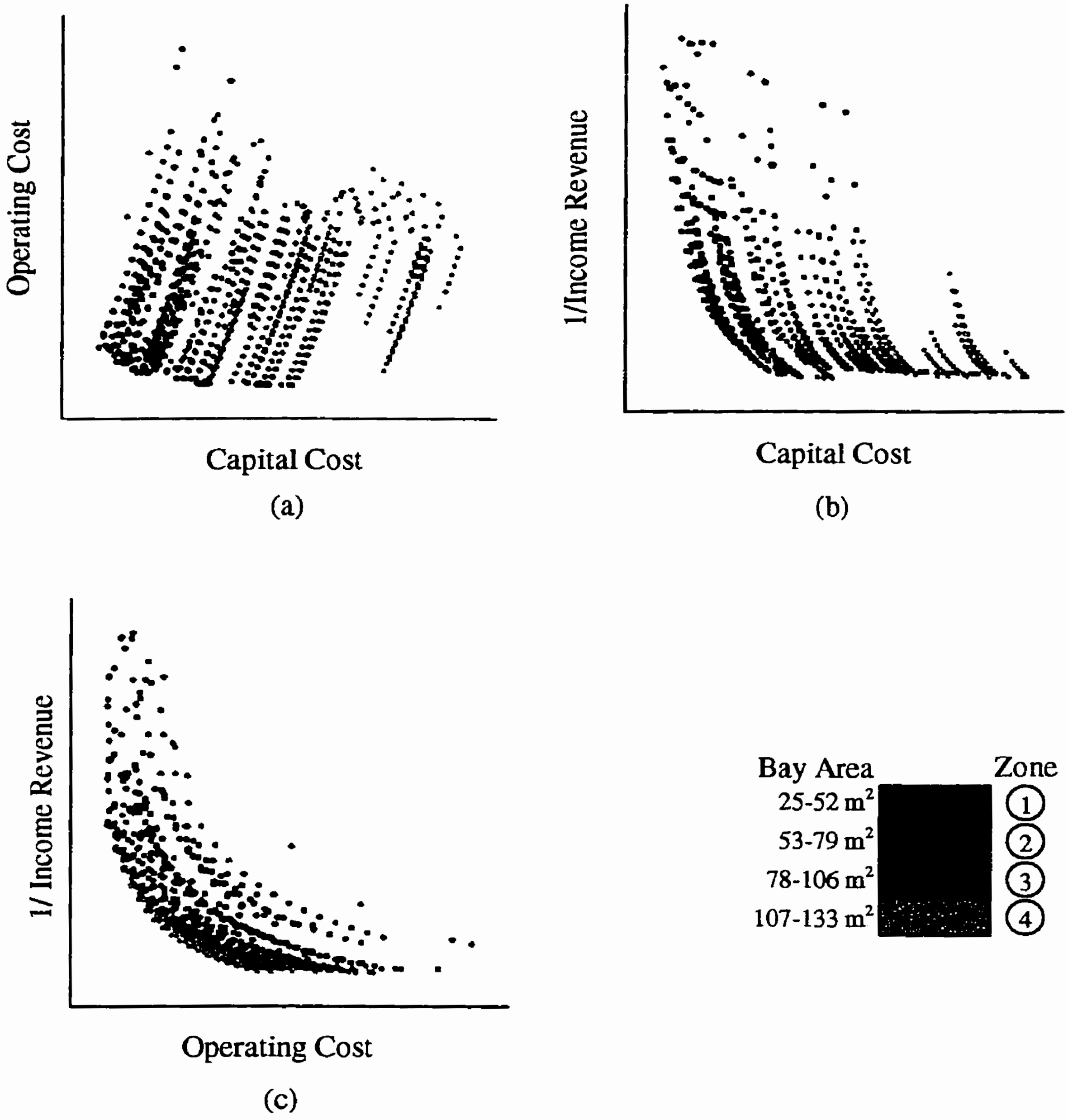


Figure 4.8: Example 1- 2D Plots of Bay Area Pareto Zones

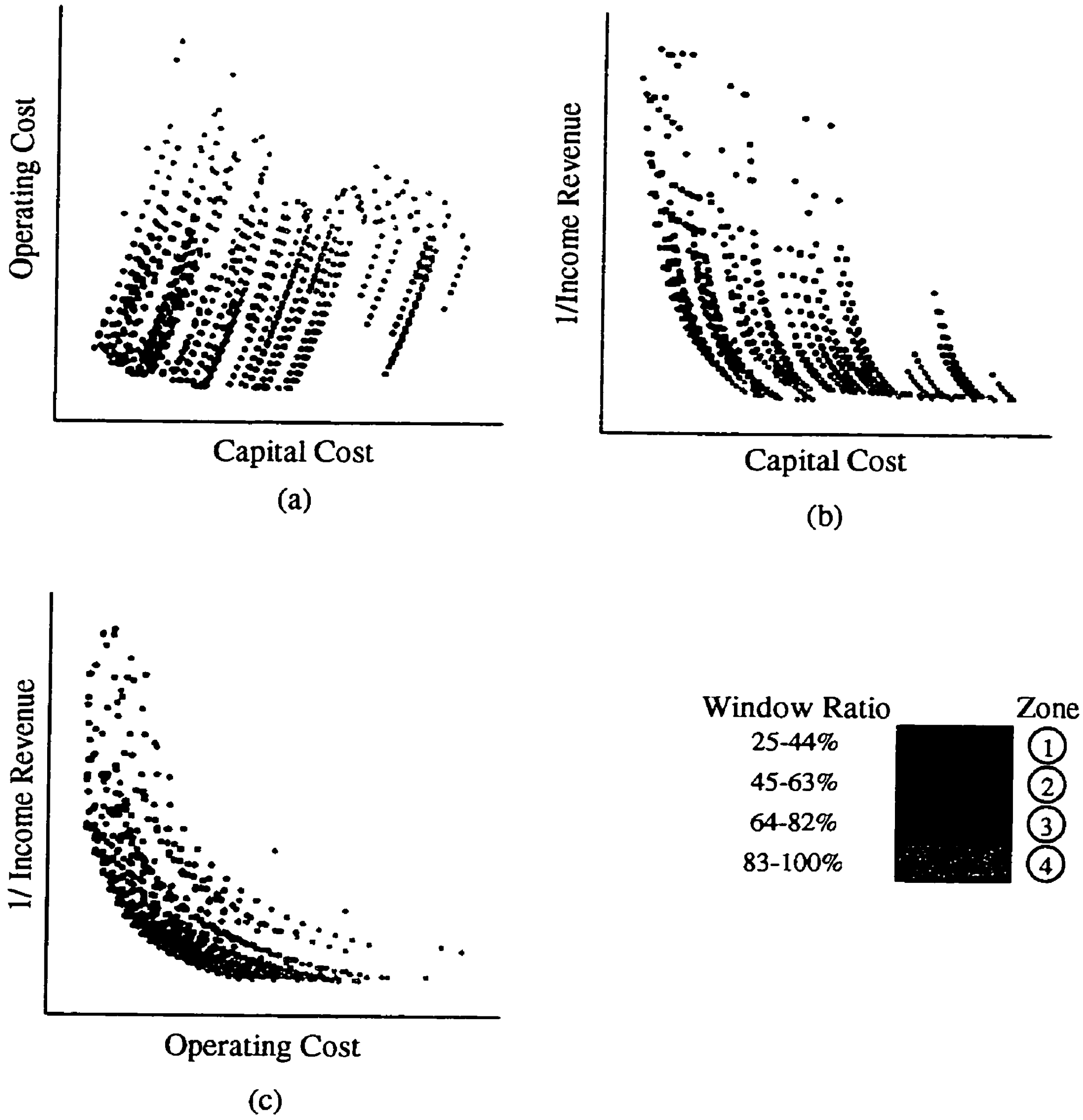


Figure 4.9: Example 1- 2D Plots of Window Ratio Pareto Zones

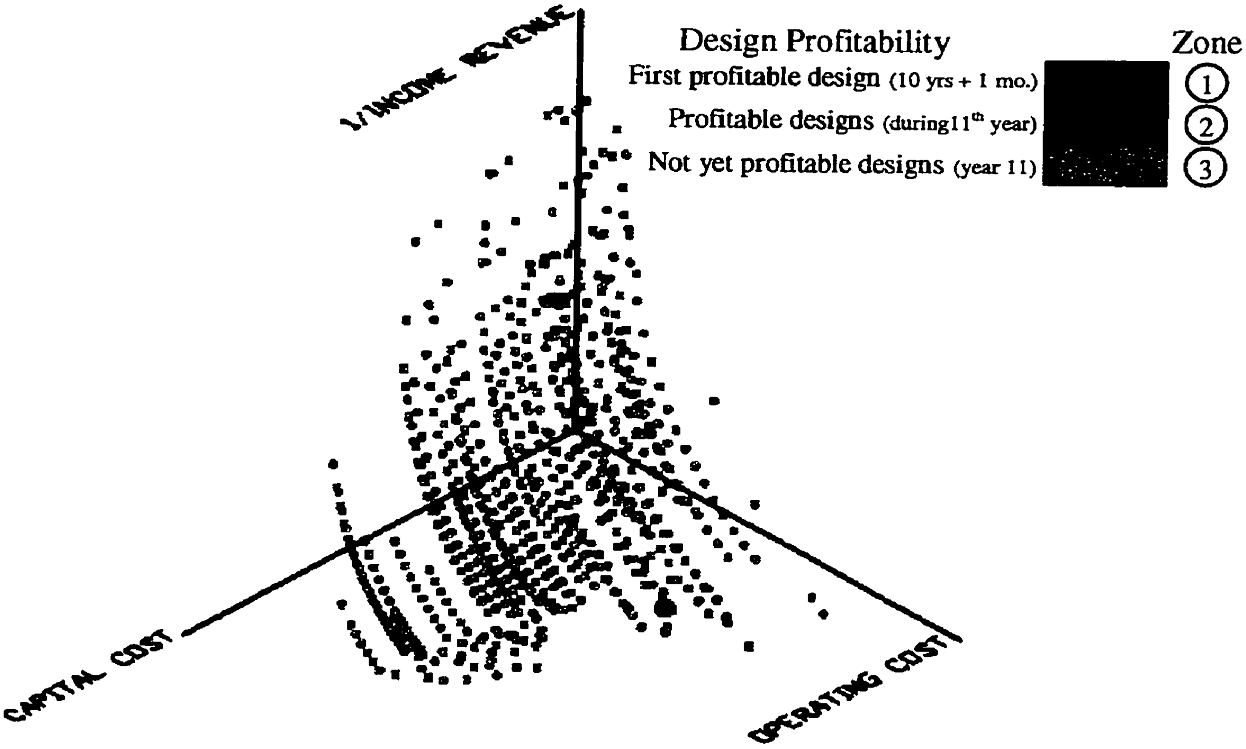


Figure 4.10: Example 1- Design Profitability

Structure: Concrete frame and shear wall
Floor: Two way flat plate
Total Number of floors: 32
Number of rental floors: 31
Number of mechanical floors: 1
Foot print: 68m x 36m
Height: 122m
Bay area: 8.5m x 12m
Core: 38.37m x 12.76m
Floor area: 60710m^2
Window Ratio: 100%
Window : Insulated HA
Cladding: Metal siding panel
Elevators: 21
Stairs: 2
Stair width: 1.1m
Capital cost: $ 107.98 M
Annual operating cost: $ 8.91 M
Annual revenue Income: $ 29.98 M
Lease rate: 497 $/m^2

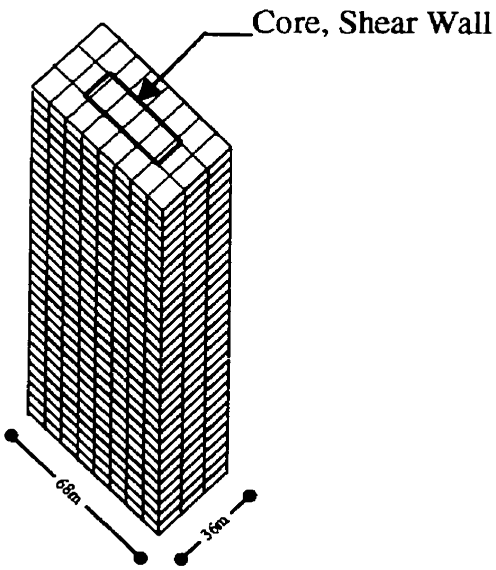


Figure 4.11: Example 1-The First Profitable Design

4.3 DESIGN EXAMPLE 2

This example is the same as Example 1 except that it has smaller land unit cost and office space lease rates in addition to lower tax rates (see Table 4.1), and serves to illustrate that the solution of the conceptual design problem can be quite sensitive to changes in the parameter values prescribed for office buildings. Here, the three different runs of the MGA converged after 147, 151 and 162 generations to find 99, 115 and 122 Pareto designs, respectively, which were then combined together to form the overall set of 139 Pareto-optimal conceptual designs for the office building indicated (by grey dots) in Figures 4.12. From among all Pareto designs for the building, the minimum and maximum lease office spaces are 60,000m^2 and 61,180m^2, respectively, a difference of less than 2%. The shortest Pareto design is 17 stories high with a 69m x 68m plan footprint, while the tallest Pareto design is only 26 stories high with a 60m x 50m footprint.

The computer colour filtering of the 3-D Pareto surface shown in Figures 4.13 4.14, 4.15 and 4.16 highlights the Pareto zones corresponding to the different structural types, number of stories, bay areas and window ratios possible for the building. A comparison of these four colour graphs with those in Figures 4.2, 4.3, 4.4 and 4.5 indicates that the results for this example are significantly different than those for Example 1. Figure 4.13 indicates that steel frame/ rigid frame & bracing, unbraced steel rigid frame and concrete rigid frame are the only viable structural systems for the building; i.e., contrary to Figure 4.2 for Example 1, there are no Pareto-optimal conceptual designs of the building for this example that have a concrete rigid frame with shear wall, steel frame/ rigid frame with shear wall or steel frame with bracing & outriggers structural system. Moreover, Figures 4.13, 4.14 4.17a and 4.18a together indicate that shorter buildings with an unbraced concrete frame structural system have the lowest capital cost; i.e., contrary to that indicated in Figures 4.2 and 4.3 for Example 1, taller buildings with braced and unbraced steel frame structural systems have higher capital cost for this example. The main reason for this reversal is that the cheaper land for this example favours shorter buildings with larger plan foot print areas; i.e., contrary to Example 1, the capital cost trade-off between buying more land or constructing taller structural systems is such that it is cheaper to buy more land (in fact, as implied by Figure 4.14, structural systems that are beyond 26 stories for this example result in uneconomical buildings in the sense that they are not Pareto-optimal because shorter building designs exist that simultaneously have lower capital and operating costs and higher income revenue than they do).

On the other hand, the trends concerning bay areas and window ratios for this example, Figures 4.15 and 4.16, were found to be essentially the same as those previously observed in Figures 4.4 and 4.5 for Example 1. For example, similar to that observed in Figure 4.4, buildings with smaller bay areas have smaller capital cost (because the cost of the floor system decreases as the bay area decreases) and, similar to that observed in Figure 4.5 and 4.9a, buildings with lower window ratios have smaller annual operating cost (because the energy cost for the HVAC system decreases as the window ratio decreases).

For the same occupancy levels and mortgage and inflation rates as previously noted for Example 1, Eq. (3.12) was applied for this example to identify a subset of Pareto designs that first become profitable in the 12^{th} year after completion of building construction, as shown in Figure 4.19. Contrary to Example 1, it was found that all of the profitable designs were shorter buildings with unbraced concrete rigid frame structural systems (see Figures 4.13, 4.14 and 4.19). The building design to first become profitable for this example is shown in Figure 4.20. The lower capital cost, operating cost and revenue income for this design compared that for the first profitable design for Example 1 (Figure 4.11) are the result of the lower land unit cost, lower tax rate and lower lease rate for this example. Note from Figures 4.14 and 4.19 that all of the profitable designs for this example are in the range of only 18 to 20 stories high as compared to the taller profitable buildings for Example 1 that range from 28 to 36 stories. It is interesting to note that the building design that first becomes profitable for this example is only 19 stories high (Figure 4.20), while that for Example 1 is 32 stories tall (Figure 4.11). The design team may select the first profitable design indicated (by a black dot) in Figure 4.19 and shown in Figure 4.20 as the basis for further preliminary/final design calculations. In fact, any number of the Pareto designs in Figure 4.12 could be selected for further study. If profitability is a motivating factor, however, the design team may be advised to concentrate on the first-profitable designs indicated in Figure 4.19, all of which have a concrete frame structural system that is 19 to 20 stories high.

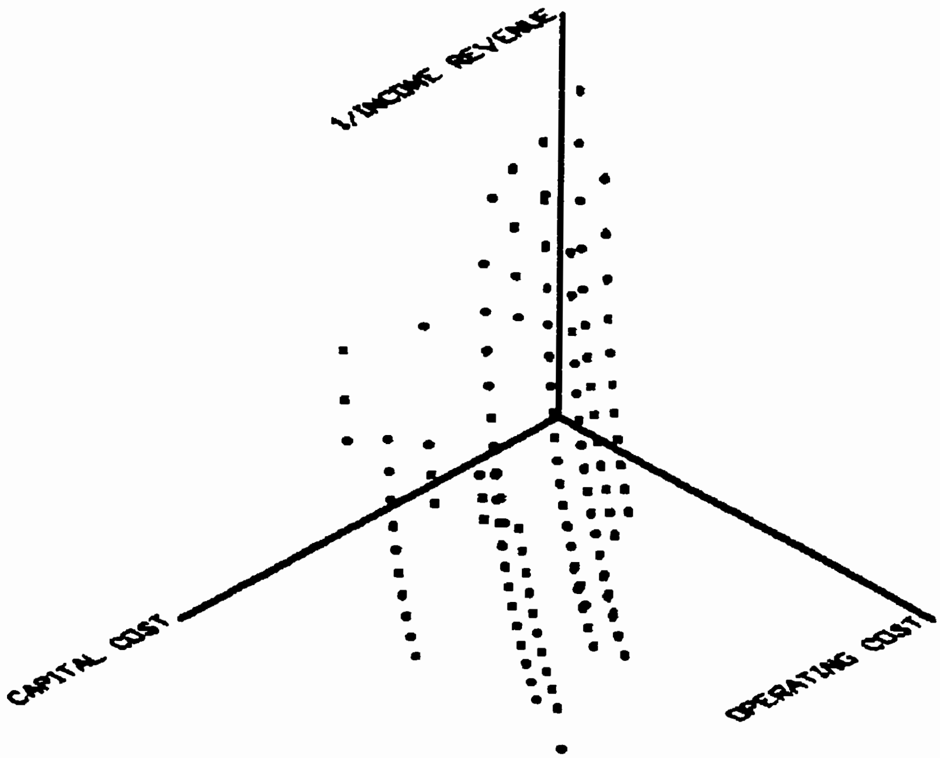


Figure 4.12: Example 2- 3D Pareto Design Space

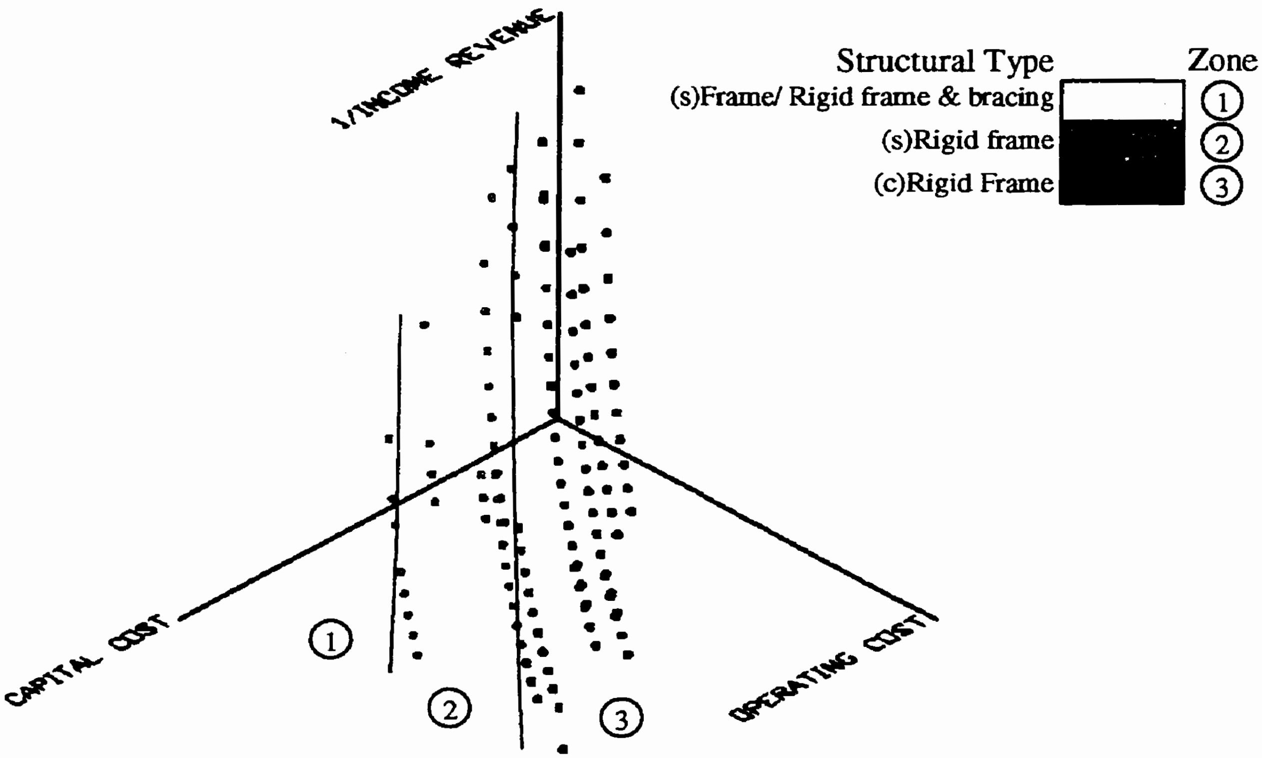


Figure 4.13: Example 2- Structural Type Pareto Zones

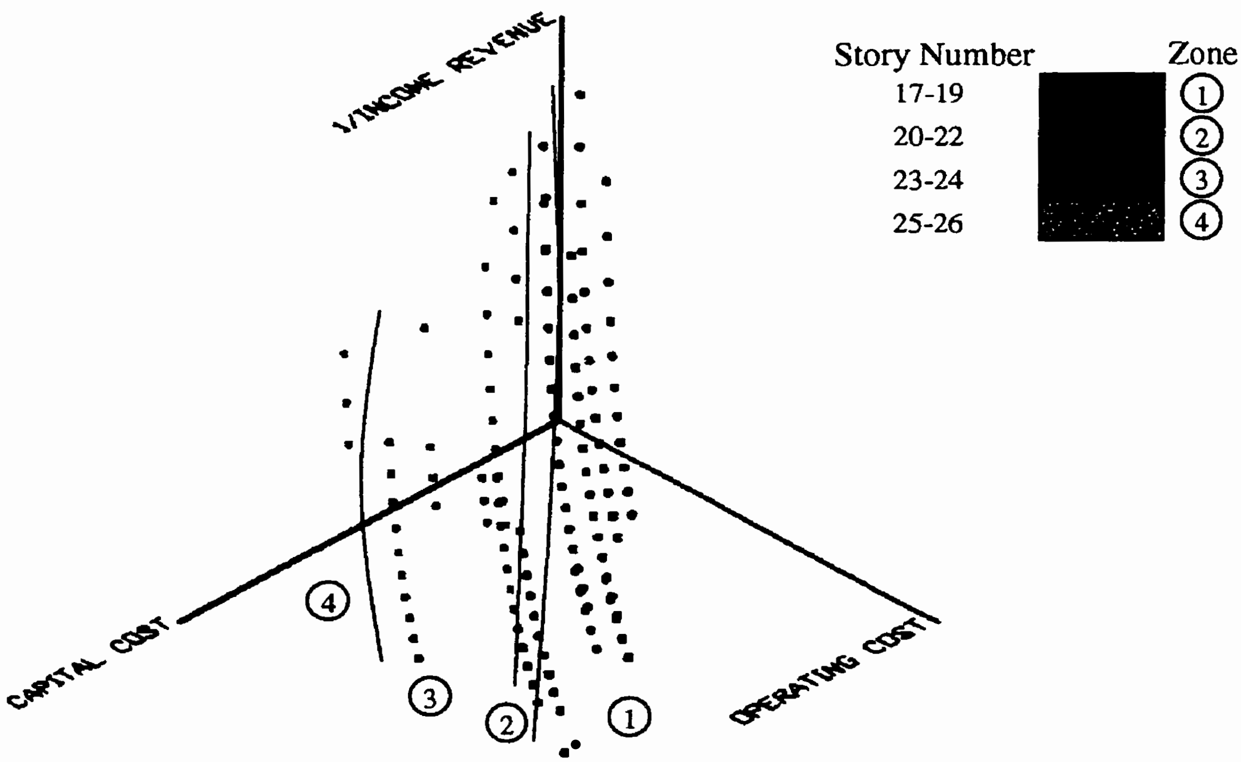


Figure 4.14: Example 2- Story Number Pareto Zones

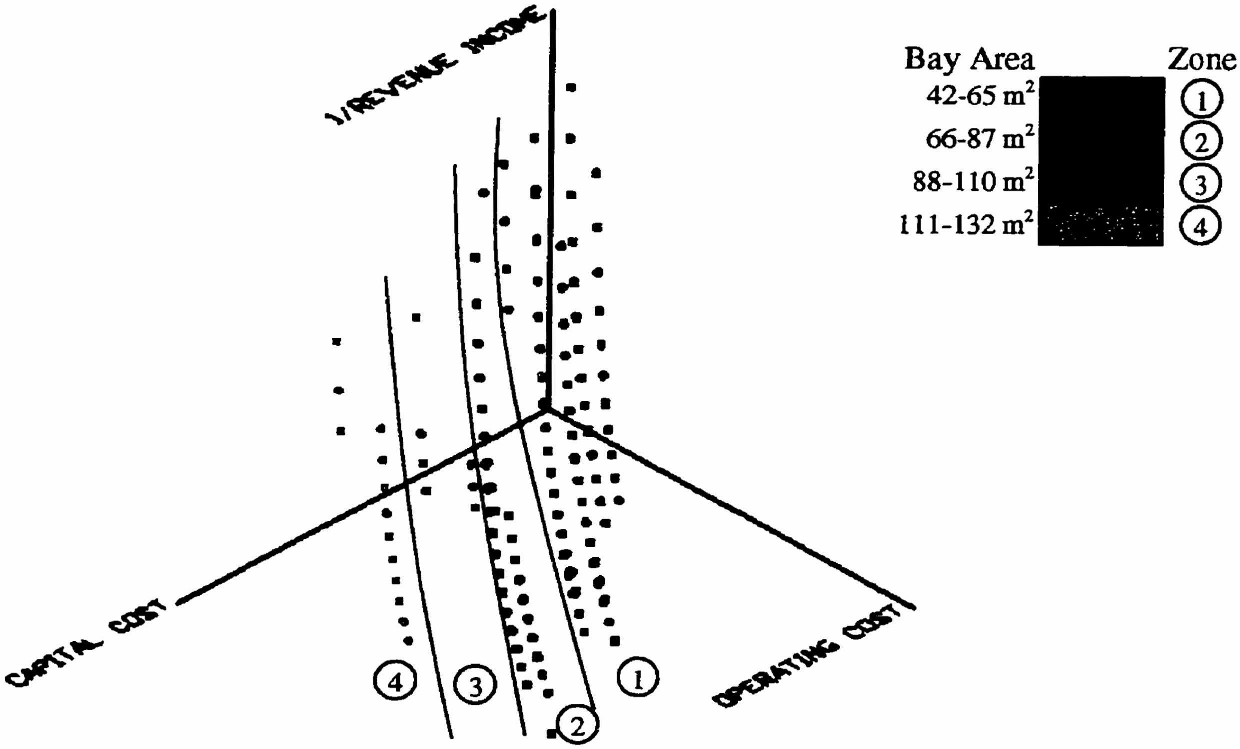


Figure 4.15: Example 2- Bay Area Pareto Zones

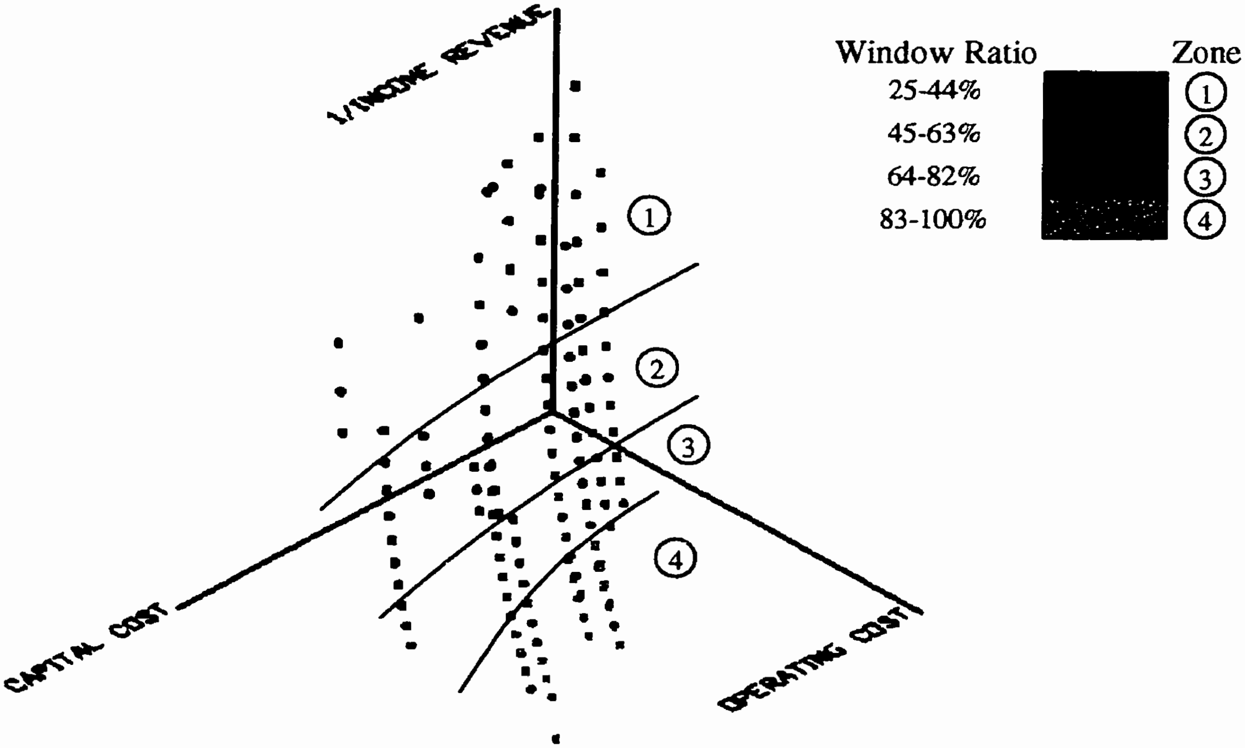


Figure 4.16: Example 2- Window Ratio Pareto Zones

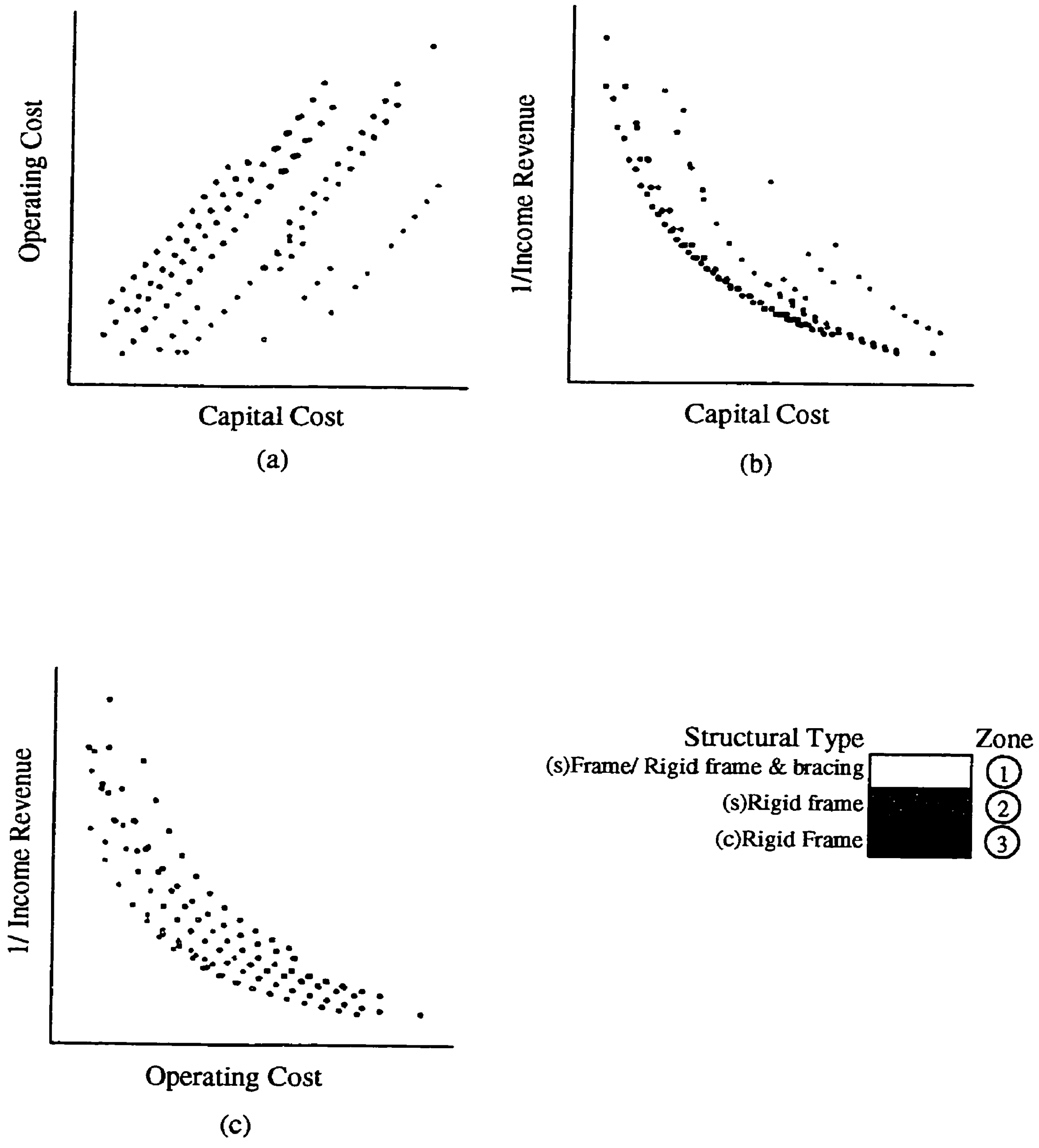


Figure 4.17: Example 2- 2D Plots of Structural Systems Pareto Zones

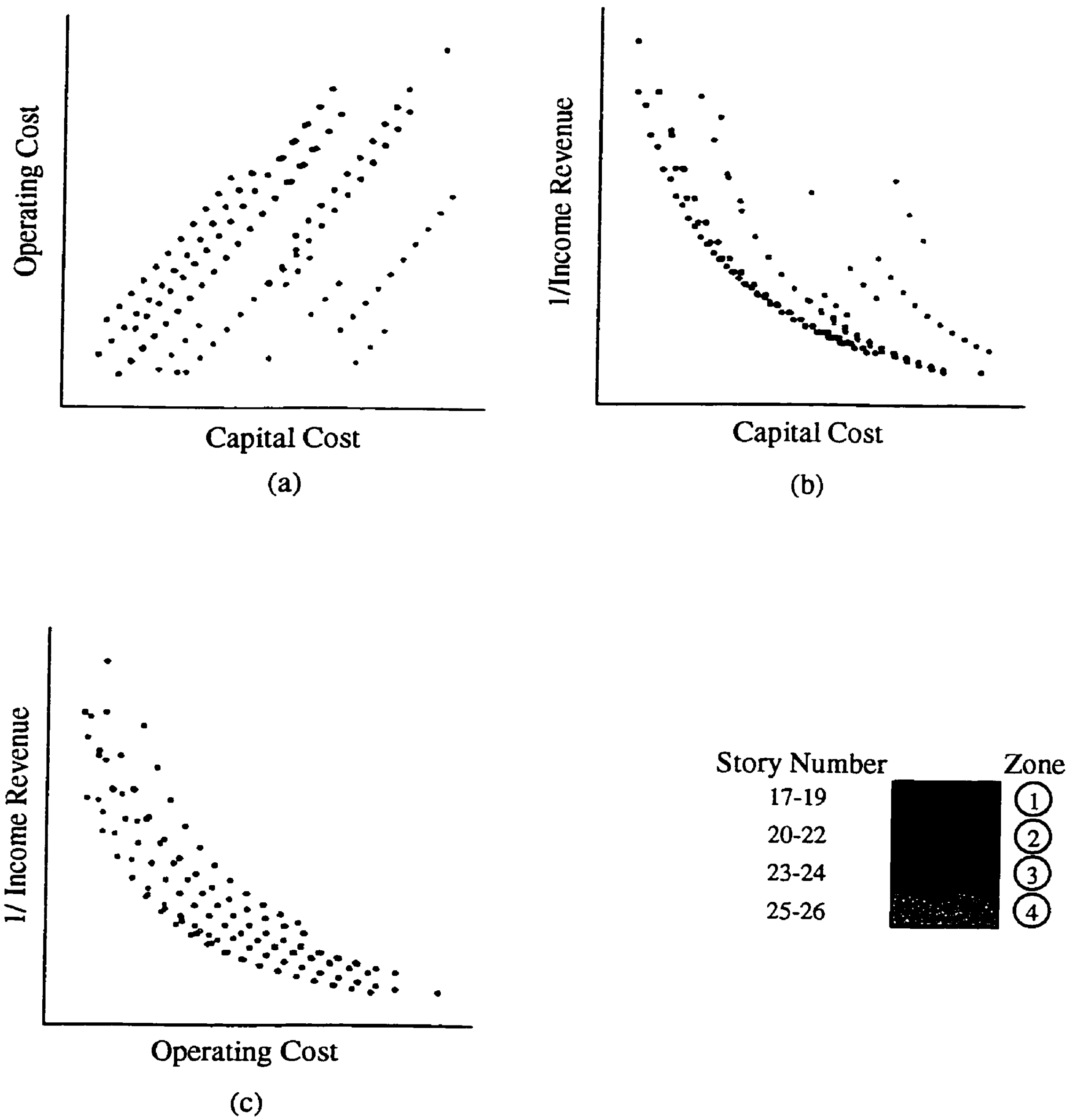


Figure 4.18: Example 2- 2D Plots of Story Number Pareto Zones

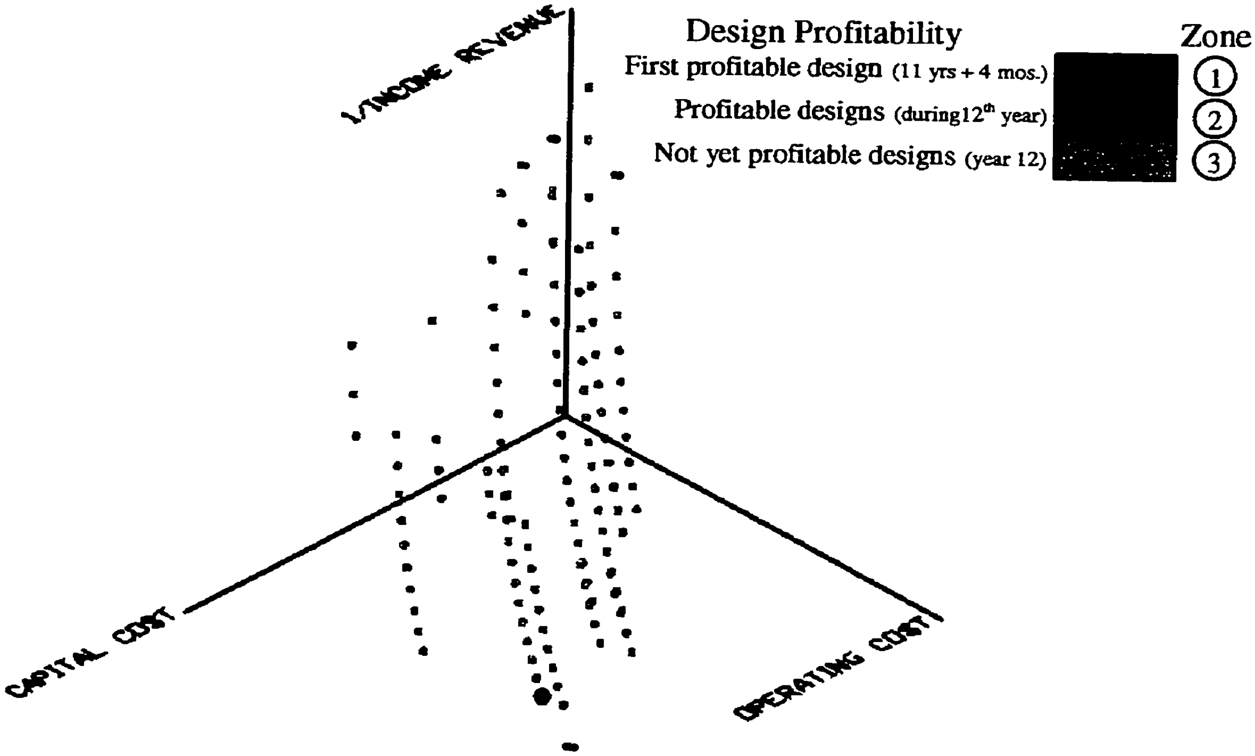


Figure 4.19: Example 2- Design Profitability

Structure: Concrete frame
Floor: Two way beam and slab
Total Number of floors:19
Number of rental floors: 18
Number of mechanical floors: 1
Foot print: 70m x 60m
Height: 76m
Bay area: 7m x 12m
Core: 56m x 15m
Floor area: 60480m^2
Window Ratio: 100%
Window : Insulated HA
Cladding: Metal Siding
Elevators: 21
Stairs:2
Stair width: 1.675m
Capital cost: $ 75.3M
Annual operating cost: $5.2M
Annual revenue Income: $18.6M
Lease rate: $308/m^2

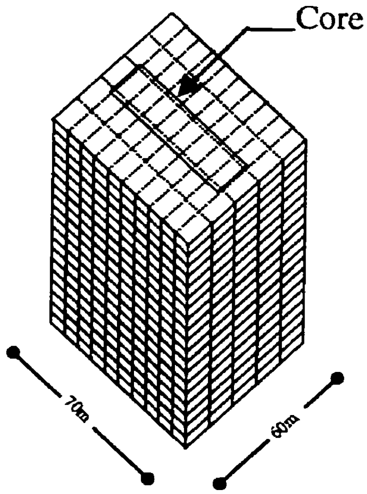


Figure 4.20: Example 2-The First Profitable Design.

4.4 DESIGN EXAMPLES 3 & 4

One purpose of these two examples is to study the effect that different material costs have on the Pareto optimality of building designs. The examples are the same as Example 1, except that Example 3 has (on average) 33% lower unit cost for reinforced concrete construction and Example 4 has 13% lower unit cost for structural steel construction compared to the corresponding U.S. national average unit costs prevailing for Example 1 (see Table 4.2). For Example 3, the three different runs of the MGA converged after 134, 142 and 136 generations to find 675, 652 and 635 Pareto designs, respectively, which combine together to form an overall Pareto set of 804 designs (see Figures 4.21 and 4.22). For Example 4, the three different runs of the MGA converged after 154, 136 and 147 generations to find 820, 852 and 817 Pareto designs, which combine together to form an overall Pareto set of 958 designs (see Figures 4.23 and 4.24).

The conceptual design results presented in Figure 4.21 for Example 3 indicate that concrete rigid frame and concrete rigid frame with shear wall are the only viable structural systems for the building when the cost of reinforced concrete construction is low compared to that for structural steel construction; i.e., contrary to Figure 4.2 for Example 1, there are no Pareto-optimal conceptual designs of the building for Example 3 that have braced or unbraced steel frame structural systems. Furthermore, the Pareto-optimal concrete structural systems for Example 3 are economically viable for a broader range of story numbers than they were for Example 1 (see Figures 4.2, 4.3,4.21 and 4.22). Conversely, the results presented in Figure 4.23 indicate that unbraced and laterally braced steel frames are the only viable structural systems for the building when the cost of structural steel construction is low compared to that for reinforced concrete construction; i.e., contrary to Figure 4.2 for Example 1 and Figure 4.21 for Example 3, there are no Pareto-optimal conceptual designs of the building for Example 4 that have braced or unbraced concrete frame structural systems. Note also that the Pareto-optimal steel structural systems for Example 4 are economically viable for a broader range of story numbers than they were for Example 1 (see Figures 4.2, 4.3,4.23 and 4.24). These two examples serve to illustrate that material costs can have a significant influence on the solution of the conceptual design problem for office buildings.

4.5 COMPUTER EXECUTION TIMES

All results for the foregoing examples were found using a Pentium II computer with 266 MHz CPU (Civil Engineering Department, University of Waterloo). Examples 1, 3 and 4 each took about 14.5 hrs for three runs of the multi-criteria genetic algorithm (MGA), or an average of 4.75 hrs per run, while Example 2 required an average of 4.33 hrs per MGA run. Basically, the computer execution time for a MGA run is comprised of the time required to calculate the values of the cost-revenue objective criteria for all designs in the genetic population, plus the time required to carry out the operations of the MGA. Since the population size is constant for all four examples, the processing time to find the values of the objective criteria for each generation of the genetic search is constant among the four examples. The lower processing time for Example 2 can be attributed to the lower number of Pareto designs found for this example compared to that for Examples 1, 3 and 4.

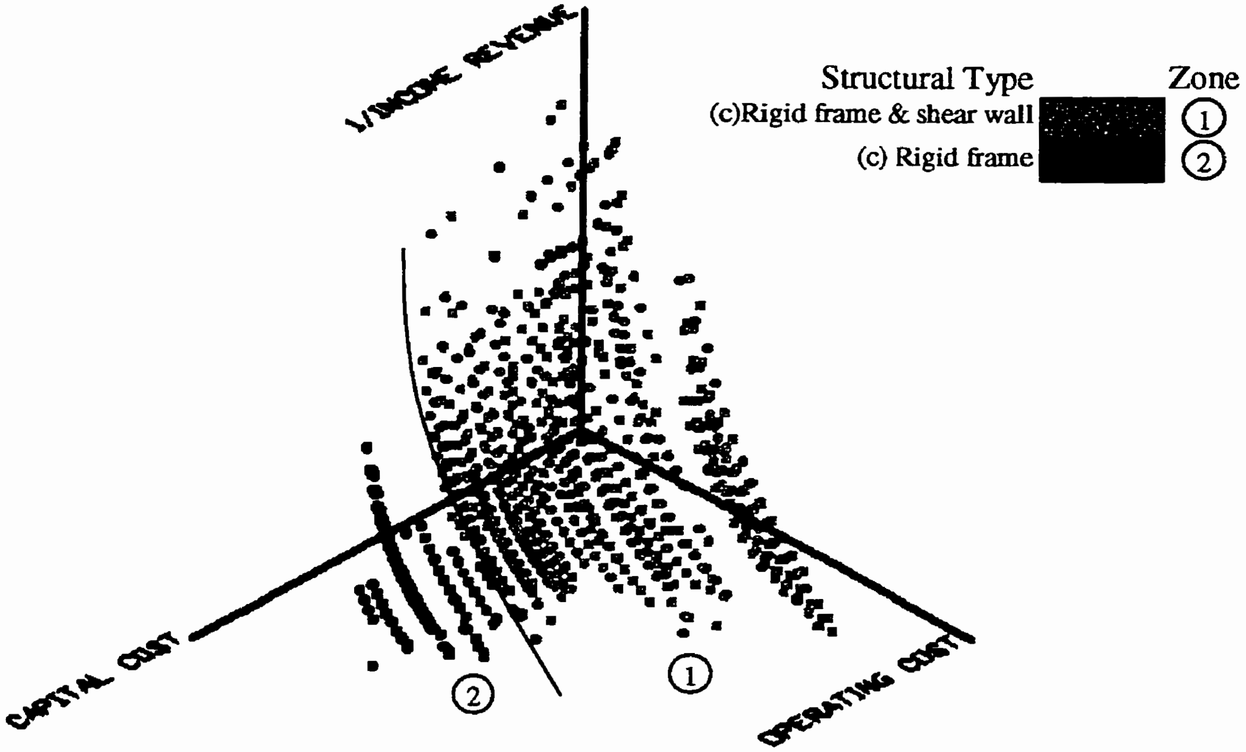


Figure 4.21: Example 3- Structural Type Pareto Zones

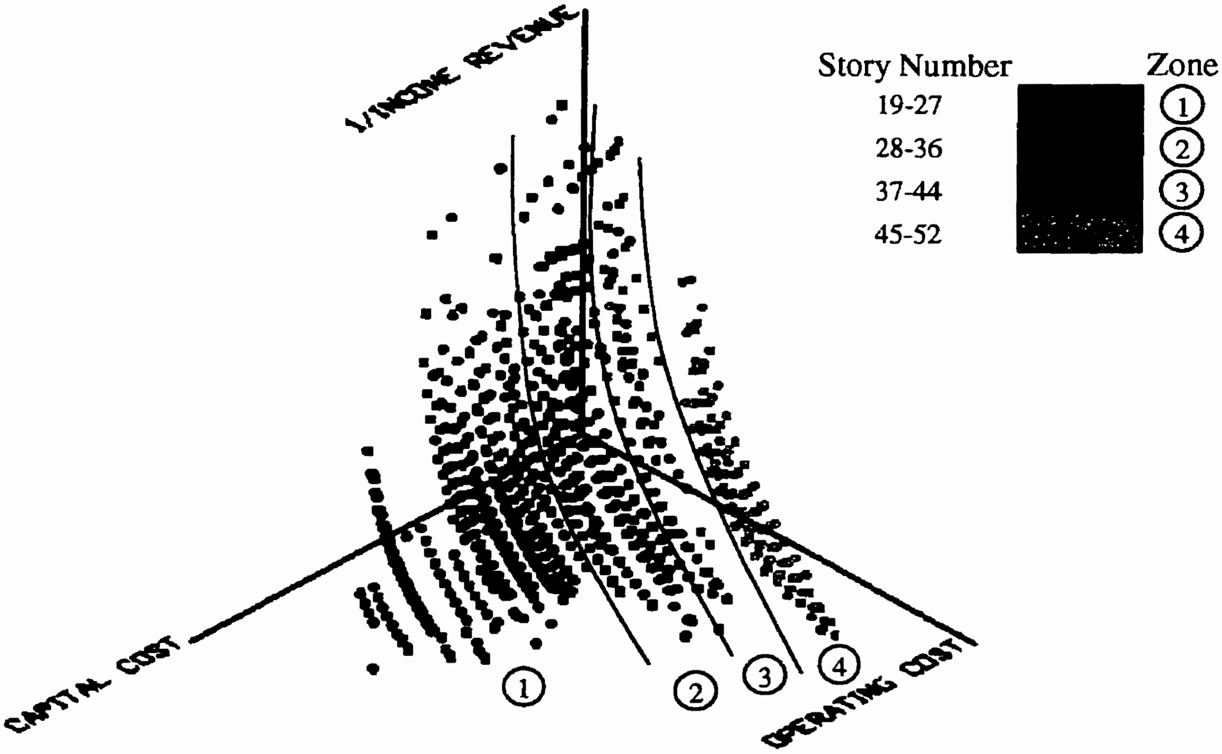


Figure 4.22: Example 3- Story Number Pareto Zones

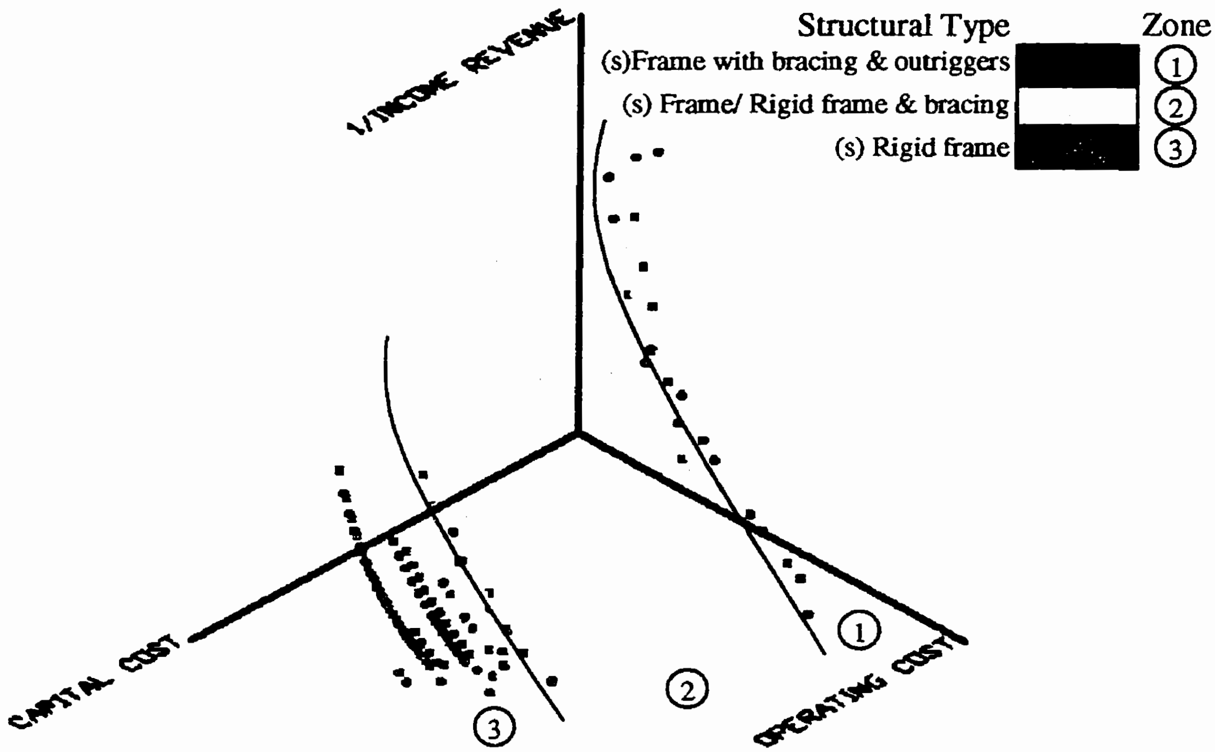


Figure 4.23: Example 4- Structural Type Pareto Zones

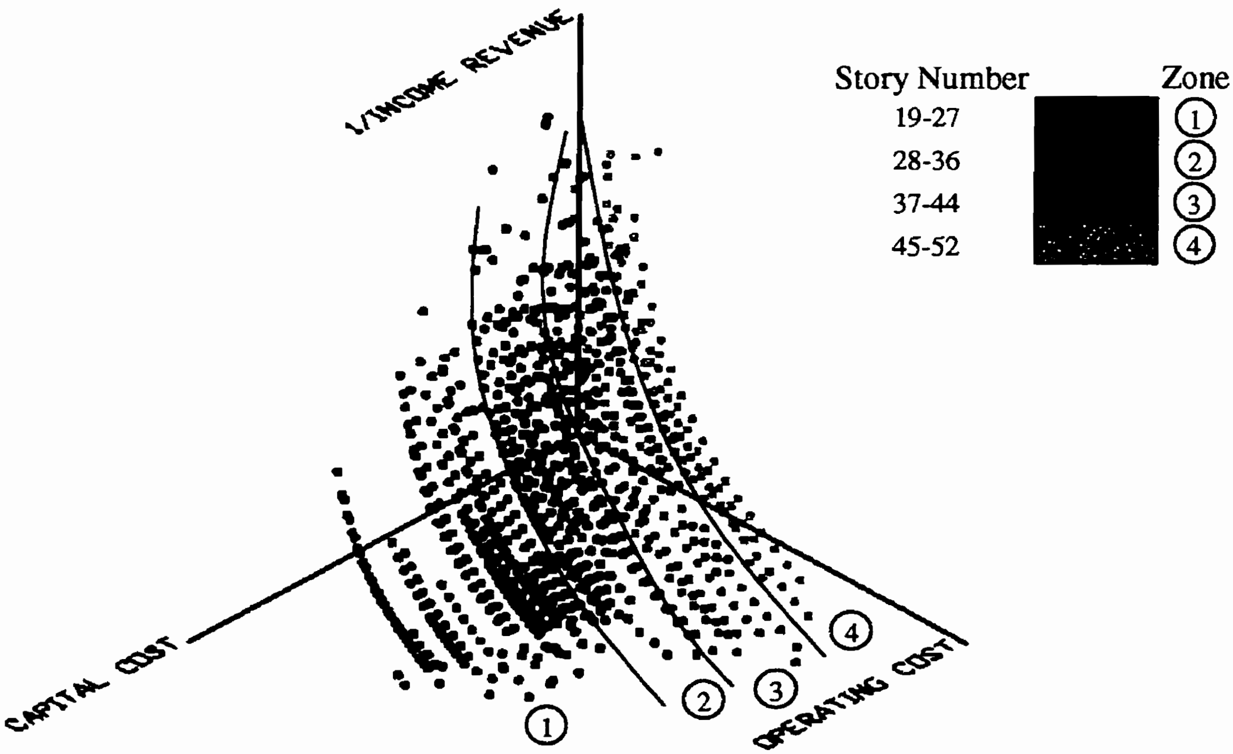


Figure 4.24: Example 4- Story Number Pareto Zones

Chapter 5

Conclusions and Recommendations

5.1 CONCLUDING REMARKS

A major contribution of this study is the development of a practical automatic design tool for the optimal conceptual design of high-rise office buildings subject to given client/user requirements and governing design regulations. Specifically, the computer-based tool has the capability to account for architectural, structural, mechanical and electrical systems and graphically identify optimal trade-off relationships between capital cost, operating cost and income revenue. The information can be used to guide and balance the concerns of the various participants involved in the building design, including the financial concerns of the owner, the enclosure and spatial concerns of the architect, the load-carrying concerns of the structural designer, and the heating, ventilation, air conditioning, elevator and lighting concerns of the mechanical and electrical designers. The computer-based procedure has the additional capability to account for life-cycle costing and predict the potential for different conceptual design scenarios to become profitable over time, which is of particular interest to building owners.

While only buildings with simple rectangular layouts were considered by this study, the developed conceptual design procedure is based on a mathematical model for multi-criteria optimization that is independent of the complexity of the building, and it is readily possible to account for all manner of additional design considerations and features, such as irregular layouts, multiple cores, lobbies, atria, mezzanine floors, foundations and underground parking.

The computer-based multi-criteria genetic algorithm is capable of searching huge design spaces very efficiently; e.g., while the data in Table 3.1 allows for more than 11.5×10^9 different conceptual design scenarios (albeit, many are infeasible), a single run of the MGA for each example office building only needed to consider about 0.0013% of them before converging to a Pareto-optimal design set. For the design examples presented in Chapter 4, one run of the MGA required 4.5 hours on average to execute on a 266 Pentium II computer (i.e., the examples each took approximately 13.5 hours on average to complete three runs of the MGA to find the combined overall Pareto-optimal design set). On average, rapid and steady convergence to the final Pareto optimal design set was achieved in less than 200 design cycles, independent of the different input parameters prevailing for the different design examples. Furthermore, other studies not reported herein have found that the number of Pareto-optiamal designs is apparently independent of the initial population size selected to commence the design process, and that the MGA is only weakly dependent on the number of design variables. It should also be noted that the computer times can be significantly shortened by using parallel-processing technology to exploit the inherently parallel nature of the numerical calculations for Pareto optimization using adaptive genetic search

A major benefit of the computer technique for the optimal conceptual design of office buildings is its capability to identify economical designs. Economy is achieved from two viewpoints. First, conceptual designs are found that each have the characteristic that no other feasible design exists that dominates it in the sense of having lower capital and operating costs and higher revenue income. Second, another meaningful saving for the design team is in the time and cost required to produce a number of conceptual design worthy of further investigation at the subsequent preliminary design stage. Unlike the traditional trial-and error analysis/design method, which typically requires a great amount of time for a competent design team to identify best-concept designs for a high-rise office building project, the automatic conceptual design tool developed by this study is capable of reducing the design time for the same project to a matter of hours.

Once the Pareto-optimal design set for an office building has been found, computer color filtering can be employed to immediately identify the prevailing trade-off relationships existing between cost and revenue for any number of different architectural, structural, mechanical and electrical parameters for the building; e.g., for the structural type, story number, bay area and window ratio parameters considered by this study, and for other parameters such as floor type, window type and, when multiple choices exist, for HVAC, elevator and lighting systems. Moreover, different Pareto-optimal design sets can be found for an office building to investigate the influence that different cost and revenue indices have on the conceptual design solution; e.g., the design examples presented in Chapter 4 demonstrated that changes in land costs and material-dependent construction costs can significantly alter the preferred choices of the architectural and structural systems for a building.

The color graphics identifying optimal trade-off relationships between cost and revenue for an office building are even more comprehensible when they are observed on a computer screen, where they can be readily manipulated for viewing at any angle (e.g., see Figures 4.6 to 4.9 and 4.17 to 4.18). These computer-generated graphics can provide experienced architects and design engineers with comprehensive integrated cost-revenue information that they may otherwise only know and understand as disconnected facts and rules. These results also can serve as an educational tool to augment the knowledge and understanding of novice architects and design engineers.

Finally, while this study has focussed on office buildings and corresponding cost-revenue criteria, the proposed mathematical model for conceptual design is independent of problem type and is readily applicable to any type of artifact and related objective criteria. The computer-based procedure will create viable conceptual design scenarios and informative 2-D and 3-D color graphics identifying optimal trade-off relationships between conflicting objective criteria, even when the number of criteria is greater than three.

5.2 RECOMMENDATIONS FOR FUTURE RESEARCH

While the computer-automated design procedure developed by this study is a useful tool for the conceptual design of regular rectangular high-rise office buildings, it is recommended that the following future research areas be pursued to further enhance the capability of the procedure and to broaden the range of applicability to building design.

1. Building Shapes and Setbacks

   Due to architectural aesthetics and city restrictions, modern high-rise buildings are often found with different shapes and set backs. It is suggested that the developed conceptual design tool be further enhanced to account for setbacks that change the size of the floor plan over the height of the building, as well as to account for floor plan shapes other than rectangular (e.g., circular, triangular, etc.).

2. Design Criteria for Structural System

   The design criteria used in this study were primarily based on strength (stress) concerns, with stiffness (strain) concerns only being met by applying a limitation on the slenderness ratio for a building so as to control lateral sway. It is recommended to develop a formal stiffness check to evaluate each design for lateral deflection, so as to assess more completely the applicability of different structural systems. Furthermore, this study only considered lateral loading due to wind. It is recommended that account also be taken for seismic loading, perhaps through approximate push-over analysis of lateral-load-resisting systems.

3. Materials

   Material strengths in this study were considered to be constant for the entire building. However, as the strength of concrete has substantially improved in recent times (e.g., 28 day cube strength of 60 MPa), more and more tall buildings are being designed with high-strength concrete in the lower story levels and low-strength concrete in the higher story levels of the building. Further improvement of the automatic conceptual design tool is recommended to account for variant material strengths for both concrete and steel construction.

4. Improved Functional Form for Space Quality

   The current study adopted a functional form for space quality that gives equal importance to both floor flexibility and occupant comfort level. Further study is needed to find a function that more completely reflects the relative importance of factors that affect space quality and, hence, lease rates (for example, to include the effect that the luxury and aesthetics of a building have on lease rates).

5. Use of Artificial Neural Networks

   The computational CPU time required to conduct a building conceptual design can be significantly reduced if, trained artificial neural networks are used to carry out repeated design activities for the various building components (e.g., finding the volume of materials, heating and cooling loads, energy consumption, etc.). This will establish the values of the multiple objective functions more quickly and result in overall reduction of the CPU time for the conceptual design process.

6. Parallel Processing

   With the recent advances in parallel computing and its availability for personal computers, it is recommended that parallel processing computing technology be employed to exploit the extensive parallel-compute nature of the developed computational procedure for conceptual design.

7. Pareto Boundary

A method for finding the boundaries of the Pareto design space was discovered in the course of this study. This discovery suggests the possibility to find only the boundaries of the Pareto space without computing the Pareto design set itself. Since the number of designs located on the boundary of the Pareto space is much less than the total number of Pareto designs, this implies that the population size and, hence, the CPU time for the MGA can be reduced, perhaps significantly. Further study in this area is recommended.

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