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MODELLING AND DESIGN OF UNBRACED REINFORCED CONCRETE FRAMES

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Abstract

The structural design of reinforced concrete structures involves analysis which requires a mathematical model for the structure being considered. The modelling process involves many aspects that affect the final design in different ways. The different aspects of modelling and their effect on the design are the focus of this study.

To implement the study, a tool that is capable of performing structural design of frames is needed. As the design process involves a considerable amount of heuristics and judgmental decisions, the potential application of expert systems in structural design is explored. A structural design expert system (ESURF) was developed using the programming environment OPS83 to evaluate the effect of different modelling methods and analytical techniques on the final design. The system overview and the design knowledge incorporated in the system are described.

To represent different aspects of modelling, five design models are defined. Three test frames are chosen to represent different frame geometries. Member effective stiffnesses and joint size effects are studied. The effect of some construction rules on the final design is considered. Load-related issues such as load combinations and live load reduction factors are discussed.

Considering slenderness effect in unbraced frames in an "exact" way (second-order analysis) and an approximate way (moment magnifier method) are compared. A new expression for the member

stiffness to be used in the momment magnifier method to enhance its performance is proposed. The code expression for EI is critically examined against the proposed expression.

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Table of Contents

List of Tables

List of Figures

List of Symbols

A_g	area of gross concrete cross section
A_s	area of reinforcement
A_v	area of shear reinforcement
A_{vmin}	minimum area of shear reinforcement
b	beam width
b_w	beam web width
B	tributary area of a structural member, square meters
c_1	column total depth
c_2	column width
c	distance from extreme compression fibre to neutral axis
CI	cost index
C_b	ratio of effective stiffness to gross stiffness of beams
C_c	ratio of effective stiffness to gross stiffness of columns
C_m	factor relating actual moment diagram to an equivalent uniform moment diagram
d	distance from extreme compression fibre to centroid of tension reinforcement
d_{bar}	nominal diameter of bar
D	dead loads
e	column eccentricity (P/M)
EI	column effective stiffness
E_c	modulus of elasticity of concrete
E_s	modulus of elasticity of reinforcement
f'_c	specified compressive strength of concrete
f_y	specified yield strength of reinforcement
h	beam total depth
INCR	concrete dimension increment
I_g	moment of inertia of gross concrete section about centroidal axis neglecting reinforcement
I_{cr}	moment of inertia of cracked concrete section about its centroidal axis
I_{se}	moment of inertia of reinforcement about centroidal axis of member
k	effective length factor

l_u	unsupported length of compression member
L	live loads
M	applied bending moment
M_f	factored bending moment in beams
M_{cr}	value of moment that produces first cracking
M_{max}	maximum moment in column
M_1	value of smaller end moment on compression member
M_2	value of larger end moment on compression member
MINS	minimum number of storeys to have the same column dimensions
M_{des}	beam design moment in a span
M_{desfl}	beam design moment in a floor
M_{desab}	beam design moment in the whole frame
n	modular ratio
NSP	number of spans
NST	number of storeys
N_f	factored axial load
N_r	factored axial tensile resistance
P	factored axial load
P_c	critical load
P_{eq}	equivalent axial load with a minimum eccentricity that a column section, carrying axial load P and moment M, can carry
Q,W	live loads due to wind
Q_c	volume of concrete
Q_f	surface area of form
Q_s	mass of reinforcement
$1/r$	mean curvature
S	spacing of shear reinforcement
t_o	width (or depth) of a square column carrying axial load P with minimum eccentricity
U_c	unit price of concrete
U_f	unit price of formwork
U_s	unit price of reinforcement
V	shear force

V_c	factored shear resistance provided by tensile stresses in concrete
V_{co}	factored shear resistance provided by tensile stresses in concrete with no axial load
Vl_1, Vl_2	limiting shear stress values
Vl_3, Vl_4	limiting shear stress values
V_{smin}	shear stress associated with minimum area of shear reinforcement
X1	eccentricity ratio (e/c_1)
X2	reinforcement ratio
Y	ratio of effective stiffness to gross stiffness of columns
Y_{code}	ratio of effective stiffness to gross stiffness of columns calculated according to the code equation
Y_{prop}	ratio of effective stiffness to gross stiffness of columns calculated according to the proposed equation
α	slenderness parameter (Pl_u^2/EI)
α_D	load factor on dead load
α_L	load factor on live load
α_W, α_Q	load factor on wind load
β_1	ratio of depth of rectangular compression block to depth to the neutral axis
β_d	absolute value of the ratio of the maximum factored dead load moment to the maximum factored total load moment
γ	dimensionless parameter for determining reinforcement position in columns
	importance factor
δ_b	braced magnification factor
δ'_b	theoretical braced magnification factor
δ_s	sway magnification factor
ϕ_c	resistance factor for concrete
ϕ_m	member resistance factor
ϕ_s	resistance factor for reinforcing bars
ρ	reinforcement ratio
ρ_{max}, ρ_{min}	maximum and minimum reinforcement ratio

ρ_{maxu}	maximum reinforcement ratio in columns specified by the user
ρ_{req}	required reinforcement ratio
λ	factor to account for low density concrete
ψ	combination factor

1. Introduction

1.1 Structural design

Design, in its broad sense, is the problem of specifying an object or objects to satisfy certain goals and constraints. In structural design, objects represent structural elements that need to satisfy strength and serviceability requirements and imposed constraints. Generally, strength and serviceability requirements are dictated by required conformance to a certain design code. Constraints, which may be job-specific, are implied by non-structural requirements. Examples of such constraints are architectural or mechanical requirements, availability of materials, and the ability to construct the design economically (constructibility).

At first glance, design appears to be simply the classical problem of synthesis and satisfying constraints. However, for structural design, merely satisfying constraints does not necessarily result in an acceptable solution. Moreover, for many applications, there may be more than one acceptable solution.

The structural design process can be viewed as a series of interrelated sub-processes that aims at providing complete and detailed description of the structure being designed. The intimate interrelation among those sub-processes makes it necessary to consider how decisions taken at any stage will affect the design at other stages.

An example of this interrelation is the mutual dependence between the analysis and proportioning processes. Since analysis is the evaluation of a known structure, member properties that are the outcome of the proportioning process need to be known. On the other hand, to proportion a member, one needs to know the stress resultants acting on that member which are not known until the structure is analyzed. Since we have to start somewhere, it is usual to initially guess at member sizes from which relative stiffnesses are estimated. The stress resultants obtained from the analysis based on the guessed sizes are then used to proportion the different members and new member properties are calculated. These member properties are then used with another analysis run. The analysis-proportioning cycle should continue until the member sections are the same as for the previous cycle. This convergence does not mean that the obtained design is unique or optimum. It simply means that the stress resultants used to proportion the different members were obtained from stiffnesses related to the respective member sections.

1.2 Analytical modelling

Analyzing a structure means computing the displacements and stress resultants for every point in the structure. To accomplish this, the engineer needs to represent the behaviour of the structure as well as the construction material under the effect of different loading conditions. It is usual to construct a mathematical model to represent geometry and other parameters involved, hence, this representation is known as modelling.

Modelling can be relatively simple or complex. As the complexity of the model increases, the amount of numerical computations also increases dramatically. For this reason, engineers traditionally used only simple models. For example, a simple model may assume that material response is linear at all stages of loading and equilibrium can always be formulated using the undeformed structure geometry (first-oredr linear analysis). For structures with slender members or non-linear material behaviour such as reinforced concrete, such modelling may not properly represent the actual structure and hence the results from the analysis may not always be adequate. In certain cases, modification factors obtained from experience are used to account for this inadequacy.

A more complex model would account for slenderness effect in structures by performing an iterative first-order analysis updating each time the structure geometry and stiffness matrices (second-order analysis). This type of analysis requires more numerical computation capabilities compared to first-order analysis.

However, with the widespread availability of electronic computers, numerical computation is no longer a limitation and more complex modelling can be considered in practice. The engineer must now decide what level of modelling is proper for a particular design situation.

1.3 Objective

The objective of this study is to examine the different modelling methods and analytical techniques and to evaluate their effect in a meaningful and practical way.

Previous investigations (Lai 1981) compared different analytical techniques on the basis of differences in member stress resultants. For example, comparing different values of bending moments at the same location obtained with different analytical techniques. However, significant differences in stress resultants may not have any effect on the final member sizes. This is because, in practice, members are constructed in certain increments, similar members are merged, reinforcing bars come in discrete sizes, and many other similar requirements.

To accomplish the objective of this study, it is necessary to fully proportion (i.e. to determine the concrete dimensions and reinforcement for) the structure using different modelling techniques. This raises the need to examine the different tools to build an automated design system that will proportion a structure satisfying all imposed constraints. Having such a system should facilitate evaluating, in a practical sense, the effect of different modelling methods on the final design of the structure.

1.4 Scope and outline

The scope of this study is limited to reinforced concrete unbraced frames subject to gravity and/or lateral (wind) loading. This type of structure was selected as it provides a considerable opportunity to examine a multitude of modelling techniques.

Chapter 2 contains a discussion of possible analytical models for unbraced frames. The notion of expert systems and rule-based programming and how they may be used in structural design are given in chapter 3. Chapter 4 summarizes the design knowledge incorporated in the ESURF system and how it is implemented to produce a complete design. To study the effect of some modelling parameters, a number of test frames are designed and compared in chapter 5. An evaluation of slenderness effects using an "exact" method and the code approximate method is presented in chapter 6. A proposed modification to the member stiffness to be used in the code method is also presented. Chapter 7 summarizes the conclusions of this study.

2. Structural Modelling

2.1 Introduction

Modelling, in structural design terms, is the way a structure is represented to assess its response under the effect of different loading conditions. To obtain a reasonable assessment of the structural response, the model must reflect the actual behaviour of the structure. However, introduction of approximations in the modelling of any structure is inevitable. Different modelling techniques for unbraced building frames are presented in this chapter.

The mathematical idealization or modelling of a structure such as a building frame can be grouped into three main categories; structural geometry, material response, and loading. Each category is discussed separately in the following sections.

2.2 Structural geometry

The simplest way to model the geometry of a reinforced concrete frame is to represent members by their centrelines where the points of intersection of these lines represent joints. However, this model does not necessarily adequately represent the interconnectivity between different members. The finite size of the joint affects both the magnitude and location of the design moments. The effect of joint size can be taken into account by either modifying member stiffness matrices to include infinitely stiff ends with finite lengths or by modelling the portion of the member in the joint region as a separate element with very large stiffness. The latter method is simpler since it does not require modifying prismatic member stiffness matrices built into most analysis programs. Therefore, the method was chosen to model the joint size effect in this study.

The geometrically non-linear behaviour of reinforced concrete unbraced frames significantly affects their response to loads especially in the ultimate stage. This non-linearity is mainly caused by the deviation of the loaded structure configuration from the original one giving rise to two effects: lateral drift effect and member stability effect (Fig. 2.1).

A traditional first-order elastic analysis does not account for non-linear effects directly. The moment magnifier method is the most commonly used way to approximate the effect of geometric non-linearity by magnifying the first-order analysis results. The method as adopted by the CSA A23.3 M-84 design code is summarized in appendix B. The design that uses this approximate way to consider geometric non-linearity is referred to in this study as a *first-order design*. A second-order analysis will consider the geometrically non-linear behaviour of frames in an exact way by an iterative procedure that will update the structure geometry and member stiffness matrices for each iteration until convergence. The design that adopts this analytical model to consider second-order effects (i.e. geometric non-linearity) is referred to in this study as a *second-order design*.

2.3 Material response

Reinforced concrete members starts to crack as the applied tensile stresses exceed the concrete tensile strength. The stiffness of reinforced concrete members decreases as cracks progress. The extent of cracking varies along the member which in turn causes the member stiffness to also vary along the member. Ideally, the member should be modelled as a multi-element member, possibly with a different value of stiffness for each element, to best simulate this variance in stiffness. However, this will result in structures with thousands of elements which is impractical from a design point of view. Simplified expressions for equivalent member stiffnesses are needed. Those expressions, however, should account for the varying stiffness of the member along its longitudinal axis and should not merely reflect the properties of the most stressed section in that member.

The simplest expression for the effective stiffness of a reinforced concrete member is to take it as a fraction of the gross stiffness neglecting reinforcement. Several suggestions for this expression are presented in chapter 5 along with a study of the effect of modelling member stiffnesses on the final design.

For a reinforced concrete section, the relationship between axial force (P), bending moment (M) and curvature (1/r) can be calculated based on given stress-strain relationships for concrete and steel. Bernoulli's hypothesis of plane sections remaining plane after loading for cracked sections is also assumed to be valid. Fig. 2.2

shows a typical moment-curvature relationship for a constant axial force. The shape of the load-moment-curvature diagram can be simplified by two or three straight lines.

For structural analysis purposes, assuming elastic behaviour, a single value of stiffness for each member can be used. This value is the slope of a radial line on a P-M-1/r diagram (EI = M/(1/r)) where P, M, and 1/r correspond to the loading condition of this cross section. For design purposes, this loading condition should represent the stage of the section just before reaching ultimate.

Several models for estimating the effective stiffness of a cracked section are available. Branson's formula, as adopted by the ACI code (ACI Committee 435 1966), suggests:

$$I_e = \left(\frac{M_{cr}}{M}\right)^3 I_g + \left(1 - \left(\frac{M_{cr}}{M}\right)^3\right) I_{cr} \qquad (2.1)$$

in which I_e, I_g, and I_{cr} are the effective, gross (uncracked), and fully cracked moments of inertia of the section, respectively; M_{cr} is the cracking moment and M is the applied moment. However, this formula does not consider the axial load effect on the effective stiffness of a cracked section.

A more elaborate model that considers the effect of axial load on the member stiffness and accounts for the contribution of concrete in tension to the effective stiffness (tension stiffening) is the CEB model (CEB-FIP Model Code 1978) described in appendix A. The model estimates the mean curvature (and hence the stiffness) of a cracked reinforced concrete section as a linear interpolation between the curvature of an uncracked section and that of a fully-cracked section.

Time-dependent deformations (creep and shrinkage) increase strains in concrete with time. Creep reduces concrete member stiffnesses as they resist sustained loads. Many mathematical models are suggested to estimate the creep effect, the simplest of which is to reduce the effective stiffness of columns by a certain ratio. The importance of this reduction to the final design is doubted by Ferguson (Ferguson 1979) since the estimate of column stiffnesses involves a typical error of at least 30%. The implications of reducing column stiffnesses to account for creep effect on the final design are discussed in chapter 5.

2.4 Loads

Structures are designed to carry and safely transfer to the foundation all applied loads and associated overturning moments. As part of the design process, these loads should be properly modelled to represent their actual effect. Loads on structures can be divided into two main types, gravity loads and lateral loads.

Gravity loads include dead loads from self-weight, partitions, ceiling and floor finishes, and live loads due to use and occupancy. Loads from partitions and finishes are represented as uniformly distributed loads. Live loads are variable in nature but are generally modelled as uniformly distributed.

Lateral loads include wind loads and earthquake loads. Wind pressure on a surface of a structure depends mainly on the wind velocity which varies with height. Fluctuations in wind speed cause gusts at peak points. Although wind forces on a structure are dynamic, it has been the usual practice for moderate size buildings to represent these forces as equivalent static forces expressed in pressure units. Cladding transfers wind forces to the supporting frame as either a uniform load on exterior columns or concentrated loads at floor levels, depending on how panels are supported.

Modelling of loads on structures requires determining the nominal value and the distribution of these loads. However, other related aspects need to be considered too. These aspects include load combinations, live load patterns (load cases), and live load reductions that are discussed herein.

Load factors are statistically determined to reflect the potential overloads and uncertainties associated with the determination of the nominal effect of these loads. Variable load combinations are defined by most design codes to account for the fact that live loads and lateral loads (wind or earthquake) are not likely to have their designated values all at the same time. The CSA A23.3 design code requires the following load combinations be considered with the corresponding load factors:

Load combination #1 : $\alpha_D\ D + 0.7\ (\alpha_L\ L + \alpha_Q\ Q)$

Load combination #2 : $\alpha_D\ D + \alpha_L\ L$

Load combination #3 : $\alpha_D \ D + \alpha_Q \ Q$

where

α_D = 1.25 for Load combinations #1 and #2

= 0.85 for Load combination #3

α_L = 1.50

α_Q = 1.50

and D, L and Q are dead, live and wind load intensities, respectively.

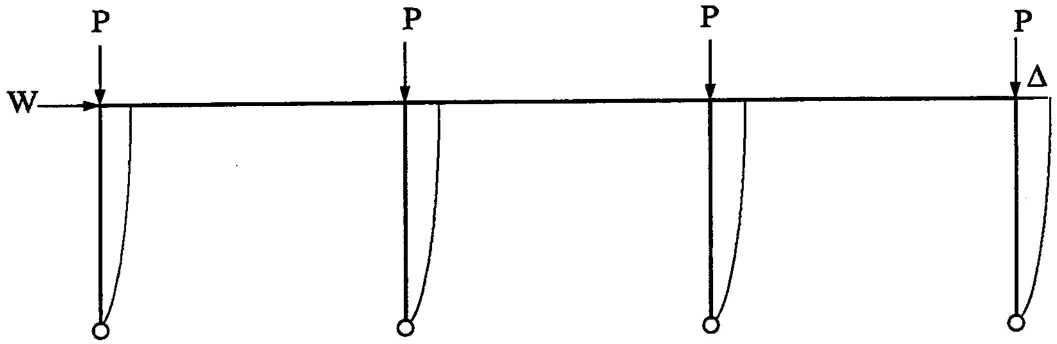
Load patterns are live load arrangements on selected areas that may produce larger stress resultants at a certain location than when all areas are loaded. This requires the designer to check different load cases for each member in the structure which increases the number of analyses required when pattern loading is to be accounted for. Obviously the designer has to take many judgmental decisions as to how to account for these factors without performing many analyses. The effect of load patterns depends mainly on the ratio of live load to dead load. As this ratio gets smaller, the effect of load patterns becomes insignificant.

Live load reduction factors (LLRF) are introduced to reduce the live load portion on members with big tributary areas. Statistically, as the tributary area of a structural member gets bigger, the probability of this whole area being loaded with the full live load gets slimmer. The National Building Code of Canada allows reducing the specified live load due to use and occupancy on structural members supporting a tributary area greater than 20 m^2 by:

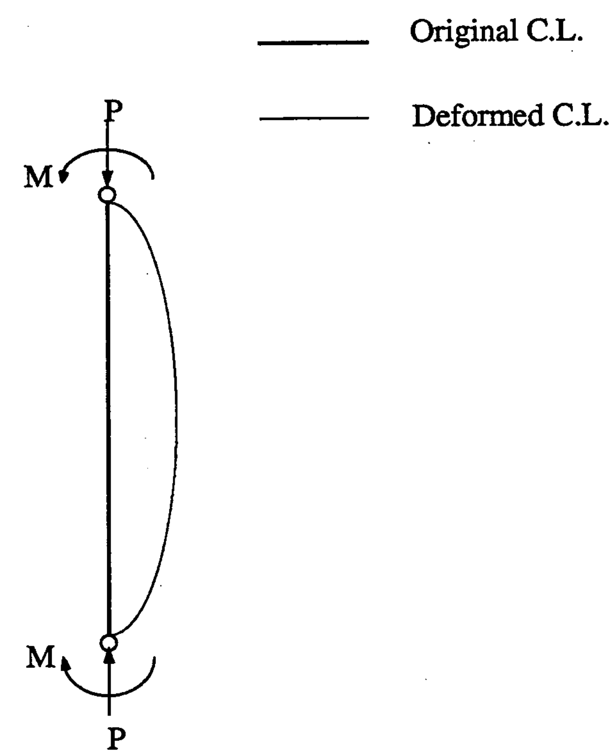
$$0.3 + \sqrt{9.8/B}$$

with a maximum reduction of 50%, where B is the tributary area in square meters. This formula is suitable for individual members rather than building frames. Each member in a frame will ultimately have a different factor which requires numerous analyses with different live load intensities on different areas. This makes the application of the above formula in the analysis of structural frames impractical. An appropriate formula for LLRF is needed to reduce live load for structural frames rather than for individual members in these frames.

To consider all of the above load combinations and cases for each member in a building frame requires running a great number of analyses. Some judgment must be exercised by the designer to select the load combinations and/or cases to be considered. The designer must also decide which load combination to design for and which one(s) to check.



Lateral drift effect



Member stability effect

Fig. 2.1 Slenderness effect in unbraced frames

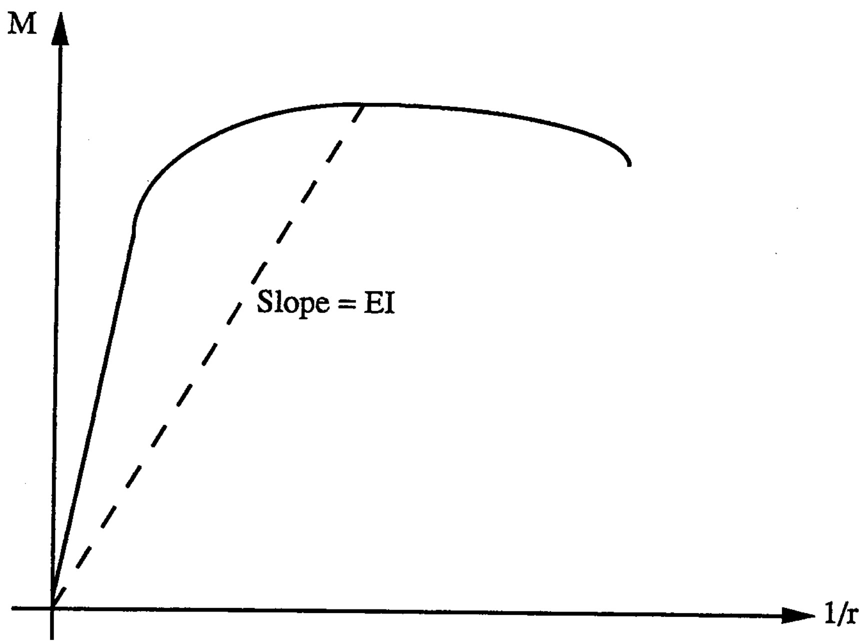


Fig. 2.2 Moment-curvature relationship for a constant axial load

3. Knowledge-Based Expert Systems

3.1 Background

Professionals in their every-day work accumulate a great deal of knowledge that pertains to their domain. After some time those professionals become so knowledgeable that they are categorized as "experts." It is believed that knowledge and the ability to exploit this knowledge are the main reasons behind the superior performance attributed to those experts relative to less-experienced professionals in the same domain. The desire to duplicate this superior performance with computers motivated computer scientists to investigate not only how professionals acquire experiential knowledge but also the way in which they apply such knowledge. Eventually, this led to the evolution of knowledge-based expert systems (KBES). As their name implies, these systems are problem-solving programs that rely heavily on knowledge and reasoning to perform tasks in a specific domain in a way similar to persons who are experts in that domain.

KBES have been written for many different applications such as medical diagnosis (MYCIN, Shortliffe 1976), planning (TATR, Callero 1984), design (R1, McDermott 1980) and monitoring (REACTOR, Nelson 1982). In each case, expert systems provide professional consultation or expert advice with the final decision still in the hand of the practitioner. Since the domain of this study is the structural design of reinforced concrete unbraced frames, further discussion relates to this domain.

3.2 Knowledge representation

Knowledge, in a general sense, is the ability to describe objects in a specific domain, how these objects are related and what operations can be done on them to reach a preset goal (Hayes-Roth et al. 1983). Knowledge is frequently classified in two forms. The first is knowledge that is clearly understood and founded on well established theories. This form is usually referred to as text-book or public knowledge. However, in many domains, knowledge often comes in a fuzzy way for which no well defined basis exists. This form of knowledge is the outcome of long-time practice in a specific domain and is usually referred to as experience or heuristic knowledge. Some examples would include the knowledge of possible actions to begin the solution to a problem, an evaluation of the progress, and revision of procedure until a satisfactory answer is achieved. It is the incorporation of this kind of knowledge and reasoning that distinguishes expert systems from other traditional computer systems.

Implementing expert systems in a certain domain involves encoding the knowledge pertaining to that domain. This will make knowledge available for other practitioners in the same domain to exploit (Adeli 1988). It will also open the door for criticism of this knowledge and hence improving and updating it. In this context, the term "knowledge engineering" has been adopted to describe the process of extracting, articulating, and representing experts' knowledge.

Choosing a set of conventions to describe objects, the relations between them and the processes that can be performed on them is referred to as knowledge representation. Among the different approaches to represent knowledge, production systems are the most commonly used in engineering design applications and hence is described briefly.

A production system consists of a collection of rules each of which has an IF part that, if satisfied, a THEN part of the rule is executed. The rule structure provides a simple way to capture knowledge and represent how to apply it. This feature makes production rules fairly easy to code and understand. Casting knowledge in modules (rules) facilitates adding new rules, removing old rules, and modifying existing rules without, in general, affecting other rules in the system.

In engineering design applications, where intensive numerical computations are involved together with a considerable amount of engineering heuristics, rule-based programming must be coupled with algorithmic implementation of some design procedures.

3.3 Tools for building expert systems

To build an expert system requires building two major components: the knowledge base and the inference mechanism that manipulates it. There is a variety of software tools available for writers of expert systems to choose from. The choice should be based primarily on the problem characteristics. Some consideration should also be given to the system portability, the hardware availability and desired running speed.

Expert systems building tools can be classified in three main groups as follows:

1) Programming languages:

While KBES can be written in any programming language, certain languages are more suited for writing such systems. LISP and PROLOG are the most common languages that can conveniently manipulate symbols and their relations. C language has less capability for symbolic manipulation than LISP but it is less demanding in memory and processing power. Writing an expert system from scratch is a slow process. On the other hand, it gives the programmer the ultimate control on the way the system works.

2) Shells:

Shells, or skeletal systems, are derivatives from previously built expert systems. They have a ready-made inference mechanism and an empty knowledge base that can be filled with domain-specific knowledge. In theory, using shells should simplify the process of building an expert system; in practice this is rarely true. A serious problem associated with the use of expert system shells is that the original framework may not be appropriate to the current problem being considered. Moreover, some task-specific knowledge may be hidden in the original system in a way that it cannot be detected. EMYCIN (van Melle 1979), a domain-independent version of MYCIN and EXPERT (Weiss 1979), derived from CASNET are two examples of expert system shells.

3) **Programming environments:**

A programming environment usually consists of some procedures and functions written in a programming language and a user-controlled inference mechanism to adopt to a wide variety of problems. Programming environments are less constrained than shells but yet not as primitive as programming languages. They can be used for a broader range of problems than shells although the building process may be more difficult. OPS83 (Forgy C. L. 1983), HEARSAY-III (Erman et al. 1980), and ROSIE (Hayes-Roth et al. 1981) are examples of programming environments.

3.4 Selection of building tool

In selecting a tool to build an expert system for structural design of reinforced concrete frames, some features are desirable. These are summarized as follows:

1. The ability to utilize programs written in other languages (especially FORTRAN) is an essential requirement in structural design applications.

2. The ability to represent knowledge in the form of production rules as this is a clear and efficient way to cast some of the knowledge.

3. Support for algorithmic programming paradigm facilitates implementing arithmetic-intensive design routines.

After a short study of possible tools at the time this project was conducted, OPS83 programming environment (Production System Technology) seemed to satisfy all above requirements and fall within the available funds and hardware range. Accordingly, OPS83 was chosen to build the system **ESURF** described in chapter 4.

3.5 OPS83

OPS83 is a programming environment that supports both rule-based and algorithmic programming paradigms. It also permits using routines written in other languages. This is a very important feature for structural design applications since most analysis programs are already written in FORTRAN. OPS83 comes in different versions to run on many computer systems. In this study, it was mounted on a VAX 11/780 computer and a SUN3 workstation under the UNIX operating system.

An OPS83 program can be contained in a single file or a number of files depending on the size of the program. This facilitates the incremental growth and fast debugging of the system during development. If multiple files are used, some declarations must be made for the compiler to indicate how those file are related.

A system implemented in OPS83 consists of two components: a collection of IF-THEN production rules (knowledge base) together with some functions and procedures, and a global data base referred to in OPS83 terminology as *working memory*. A function or a procedure is a group of sequential instructions to perform a specific task. A rule is basically composed of a conditional expression (LHS) and an unconditional sequence of actions (RHS). The LHS of a rule consists of one or more patterns each generally includes one element. An element has one or more attributes that can take either a numerical or a symbolic value. Elements are stored in working memory and they represent the current state of the problem. Elements can be altered from the RHS of a rule. A pattern is said to be satisfied if all element attributes in this pattern are satisfied. A rule is considered to be satisfied if all patterns in its LHS are satisfied.

While functions and procedures are invoked by simply writing their names at the point in the code where they are needed to be executed, rules are invoked by a rule interpreter that performs a sequence of operations known as the recognize-act cycle. This cycle includes the following steps:

1. Match: Evaluate the LHSs of rules to determine which ones are satisfied according to the current working memory.

2. Conflict Resolution: Select one rule from those with satisfied LHSs. If no rule is satisfied, stop.

3. Act: Execute the operations in the RHS of the selected rule.

4. Go to step 1.

The typical sequence of events as a system runs is: the RHS of a satisfied rule makes changes to working memory, another rule becomes satisfied, and on the next cycle this rule makes further changes to working memory which causes a third rule to be satisfied. This process continues driving the system closer to the solution of the problem. The bottleneck of the recognize-act cycle is the matching step. However, the OPS83 matching algorithm is very efficient which makes the cycle very fast. Execution time of a production system decreases as the number of pattern matches decreases. To minimize the number of matches, the following guide lines are suggested:

1. Use the first element of each LHS as a goal element which quickly eliminates a great number of rules that are not applicable.

2. Let more restrictive elements precede less restrictive ones.

3. Let more restrictive element attributes precede less restrictive ones.

4. Use more elements to clearly distinguish similar rules.

In OPS83, each rule is given a unique name. Rules in ESURF were each given a self explanatory name that appears on the screen (or any output channel) to indicate the firing of this rule. This feature allows the user to monitor the execution of different design steps as they take place.

4. ESURF Detailed Description

4.1 Introduction

E S U R F is an acronym for **E**xpert **S**ystem for design of **U**nbraced **R**einforced concrete **F**rames. The system was developed to evaluate the effect of different modelling methods and solution strategies on the final design. This requires the system to be able to produce designs that meet all safety requirements and are economical and constructible.

Design knowledge was obtained from different sources. Strength and serviceability requirements were generally based on code specifications, the writer's experience, and discussions with staff members and other students. Constructibility rules were verified in discussions with several practicing engineers.

With only the essential information (mainly frame geometry and loading), ESURF is capable of producing a complete design that satisfies all the applicable requirements and constraints. Moreover, with its interactive capabilities, ESURF can accommodate any requirements or constraints that the user may specify before or during the course of the design process.

4.2 Scope of ESURF

ESURF performs structural design of unbraced building frames under the effect of gravity and/or lateral loads. Certain limitations are incorporated on the scope of application. These limitations are summarized as follows:

1. Frames are assumed to be unbraced.
2. Frame geometry is restricted to vertical columns and horizontal beams.
3. Maximum number of spans is 10 and maximum number of storeys is 20 (a likely upper limit for the number of storeys of unbraced frames).
4. The system can handle any combination of gravity loads (dead and live) and lateral loads (wind).
5. Design is performed in compliance with CSA A23.3 standards.
6. SI unit system is used throughout the design.
7. Columns in lower storey are assumed to be fixed at their base.

4.3 System overview

The flow diagram shown in Fig. 4.1 shows the different components of the system and how they interact to produce a final design. The design process in ESURF includes five main stages:

1. Input.
2. Modelling (member stiffnesses and joint sizes).
3. Analysis (consideration of geometric non-linearity).
4. Selection of concrete dimensions (proportioning).
5. Selection of reinforcement (detailed design).

A general description of the overall system is presented first followed by a detailed description of different stages in the design process.

ESURF embodies the knowledge necessary to produce complete and acceptable designs of unbraced reinforced concrete frames. It also allows the user to implement job-related constraints by changing some design parameters that are otherwise defaulted by the system. Since the design is ultimately used to evaluate different modelling methods for joint size and member stiffness, three models are implemented that cover the practical range of these methods.

Initial sizes for members are obtained from an iterative process as follows. Using preset relative member stiffnesses, a first-order elastic analysis is performed and the results are used to select concrete dimensions for all members (using assumed reinforcement ratios) to satisfy strength and constructibility requirements. Based on these concrete dimensions, new stiffnesses are computed and another analysis is performed. The cycle is repeated until the same set of member dimensions is obtained for two successive iterations.

At this stage, the geometric non-linearity of the structure is taken into account by either modifying first-order moments using the moment magnifier method (first-order design), or performing a second-order analysis (second-order design). The modified stress resultants are then used to reproportion all members. In a second-order design, this process is also iterated until a final set of dimensions is reached. In a first-order design, a partial iteration is used in some cases as described in section 4.6. Longitudinal and transverse reinforcement for each member is then selected.

For a second load combination, an analysis consistent with that of the first load combination is performed to obtain stress resultants for the second load combination using the member properties determined from the first one. Strength adequacy is checked for each member and if inadequate, the member reinforcement is increased until strength is satisfied. If the maximum permissible reinforcement for a member is reached and the member is still inadequate, the system will print a message to this effect and then check the next member. At the present time, when this situation occurs, a complete design can be obtained by reversing the order of the load combinations and restarting the design process.

To compare different designs, concrete volume, area of formwork, and mass of steel are calculated. To facilitate comparisons, unit prices are assigned to these quantities to estimate a cost index using the formula:

$$CI = \frac{Q_c U_c + Q_s U_s + Q_f U_f}{U_c + U_s + U_f}$$

where CI is the cost index, Q is the quantity, and U is the unit price. The subscripts c, s, and f refer to concrete, steel, and formwork, respectively.

An ESURF sample output for a frame design is listed in appendix D.

4.4 Input

ESURF operates in a menu-driven interactive mode. Input data and applicable design constraints can be entered either by loading a pre-edited file or by interacting with the system through a sequence of menus (starting with the main menu). Each menu has a number of items (or submenus) with an integer number before each one. The user enters the integer number before the item he wishes to access. The current value(s) of the parameter(s) in that item are automatically displayed. To alter a parameter value, the user enters the new value which appears instead of the old one. An EXIT option is provided as the last item in each menu.

4.4.1 Main menu

The main menu has three submenus to input design data and seven items to control the design process. These submenus and items as they appear in the main menu are as follows:

MAIN MENU

1) Menu A
2) Menu B
3) Menu C
4) First-order design
5) Second-order design
6) Display column dimensions
7) Display beam dimensions

8) Select column reinforcement
9) Select beam reinforcement
10) Check second load combination
11) Calculate quantities

Items 1, 2, and 3 are submenus for input of data that are discussed later. The rest of the items (4 to 11) are commands to control the design process. Items 4 and 5 are used to specify the way geometric non-linearity is to be considered. Items 6 and 7 are used to display the concrete dimensions before selecting actual reinforcement, should this be desired. Items 8 and 9 are used to select reinforcement for columns and beams, respectively. Checking the second load combination can be done by choosing item 10. Quantities and cost index are calculated in item 11.

4.4.2 Menu A

The essential information needed to design a reinforced concrete frame is grouped under menu A that has the following items:

MENU A

1) Number of spans and number of bays
2) Span lengths (in meters)
3) Storey heights (in meters)
4) Uniformly distributed gravity loads
5) Concentrated gravity loads

6) Wind loads

7) Load factors

8) Yield stress of rebars and concrete strength(s) (in MPa)

The first item defines the frame overall width and height in multiples of spans and storeys, respectively. The information to be provided in items 2 to 6 is ensured to be consistent with the two numbers in the first item. The second and third items define span lengths and storey heights, respectively. Each span length and storey height may have a different value.

Uniformly distributed gravity loads include dead load and live load specified in kN/m for every span. Concentrated gravity loads also include dead load and live load defined in kN on each span. There is a one-load pattern at mid-span point or a two-load pattern at one-third points. Provision is made for similarly loaded storeys. Wind loads are specified as concentrated forces (in kN) acting horizontally at each storey level. Each force may have a different value.

Two sets of load factors are used to specify which load combination to be considered first for design and which one to be used for checking. Three load factors (for dead, live, and wind loads) are input for each load combination.

The yield strength of reinforcing bars is assumed to be the same for all members. Provision is made to input one concrete strength for beams and up to three concrete strengths for columns.

4.4.3 Menu B

Menu B is used to input design parameters and/or constraints on the design process. The menu includes the following items:

MENU B

1)	Dimension increment	(mm)	50
2)	Minimum concrete cover	(mm)	40
3)	Minimum concrete dimension	(mm)	250
4)	Stirrups/ties bar number		10
5)	Maximum ρ in columns		0.04
6)	Minimum ρ in columns		0.01

All parameters have the default values shown on the right. Any of these values may be changed by entering the corresponding item number and providing the new value which will then appear instead of the old one. The first four items apply to both beams and columns while the last two apply to columns only.

4.4.4 Menu C

Menu C permits input of constraints related to constructibility requirements. For ease of use, menu C is divided into three submenus, each relates to a member type:

1. Menu C1 for column related constraints
2. Menu C2 for beam related constraints
3. Menu C3 for beam-column related constraints

MENU C1

1) All columns are identical	off
2) Specify column stacks	off
3) Minimum number of storeys per column cross section	MINS

The first item permits the user to instruct the program to choose one cross section for all columns in the frame. The second item permits the user to specify his own choice of column dimensions. Those two options are set to be off by default, in which case column dimensions are selected by the program. The third item permits the user to specify an upper limit on how often the column dimension(s) may be reduced, that is the minimum number of storeys for which the column dimensions are to remain unchanged (MINS). The default value for MINS is defined according to the following rule:

IF	NST < 4	MINS = NST
ELSEIF	NST < 6	MINS = 2
ELSEIF	NST < 12	MINS = 3
ELSE		MINS = 4

where NST is the number of storeys.

MENU C2

1)	Specify beam groups		off
2)	Specify maximum beam depth	(mm)	0.0
3)	Specify beam width	(mm)	0.0
4)	Same beam cross section for the frame		off

Using this menu, the user can assign different depths for beam groups. Each group includes all the beams in a specific number of storeys. The second item allows the user to specify an upper limit on any beam depth that will be chosen by the system. The user can also specify the beam width which is assumed to be constant for all beams. The user may also indicate that all beams should have the same dimensions. All parameters are defaulted to inactive state (either off or zero).

MENU C3

1)	Beam width must NOT equal any column width	on
2)	Beam width must equal column width	off

This menu allows the user to select the beam-column intersecting criterion that best fits practical requirements. Options include choosing either different widths for beams and columns (item 1) or the same width for both (item 2). In the latter case, only one width will be chosen to all columns. Only one of the two options can be on.

4.5 Modelling

Modelling of joint size and member stiffness of concrete members can be done in different ways. While any number of models for joint size and member stiffness could be incorporated in the system, it is thought that three models cover the range of practical applications. The models are described herein starting with the simplest to the most elaborate.

4.5.1 Model A

In this model, beams and columns are modelled as one-element prismatic members (Fig. 4.2). The effective stiffness for all beams is taken to be C_b times the stiffness based on gross concrete dimensions and neglecting reinforcement. The effective stiffness for columns is taken to be C_c times their gross stiffness neglecting reinforcement. The selection of the values of C_b and C_c is discussed in section 5.4. To account implicitly for the joint size effect, beam end moments are taken to be the centreline moments minus a value of $Vc_1/3$, where V is the centreline shear force and c_1 is the column depth. Column end moments are taken to be the centreline moments without modification.

4.5.2 Model B

This model accounts explicitly for finite joint sizes by modelling members as 3-element members (Fig. 4.3). For each member, the end elements are assigned a large value for stiffness to simulate the rigid joint effect. The middle element is assigned an effective stiffness value as described in model A. Beams and columns end moments and shears are taken to be those of the corresponding middle elements.

4.5.3 Model C

This model is considered to be the most sophisticated among the three models. Beams and columns are modelled as 5-element members (Fig. 4.4). The end elements, as in model B, are assigned a large stiffness value to account for joint size. The middle three elements represent the difference in effective stiffness along the member due to varying degree of cracking and reinforcement. For each one of those three elements, the effective stiffness is calculated as a function of the loading condition and reinforcement level in the element according to the CEB model (CEB-FIP 1978) described in appendix A.

4.6 Analysis

Analysis is performed in ESURF using a finite element program initially developed by EL-Zanaty and Murray (1980) and further modified by Lai (1982). The system, through this program, is capable of analyzing unbraced frames either in a first-order or a second-order mode.

For a first-order design, the system will perform a first-order analysis and then magnify the first-order moments using the moment magnifier method. While implementing the moment magnifier method, it was noticed that when the member properties from an unmodified first-order analysis were used to calculate the sway magnifier δ_s, the results in some cases were not acceptable (i.e. the value of δ_s was so large that resulting member sizes were unreasonable). This was treated by partially iterating whenever δ_s is found to be unreasonable (more than 2.0, MacGregor 1988). In each partial iteration, column depths or beam depths are increased alternatively.

For a second-order design, the system will perform a second-order analysis which takes into account the geometric non-linearity (i.e. lateral drift effect). The member stability effect is considered by applying the closed form solution (Chen and Lui, 1987) to each column:

$$M_{max} = |M_2| \sqrt{\frac{\left(M_1/M_2\right)^2 + 2\left(M_1/M_2\right)\cos\alpha + 1}{\sin^2\alpha}} \qquad (4.1)$$

where $\alpha = l_u \sqrt{\frac{P}{EI}}$

and P is the axial load in the column, l_u is the column unsupported length and EI is the column stiffness. M_1 and M_2 are the smaller and larger end moments of the member, respectively. The term M_1/M_2 is positive for members bent in double curvature and negative for members bent in single curvature. To determine the location of the maximum moment x, Eqns. (4.2) and (4.3) can be used for double and single curvatures, respectively. For Eqn. (4.1) to be applicable, the location of maximum moment must be between the member ends (i.e. the value of x must fall between 0.0 and l_u).

$$\tan \bar{\alpha} = \frac{-(M_1 \cos \alpha + M_2)}{M_1 \sin \alpha} \qquad (4.2)$$

$$\tan \bar{\alpha} = \frac{-(M_1 \cos \alpha - M_2)}{M_1 \sin \alpha} \qquad (4.3)$$

where

$$\bar{\alpha} = x \sqrt{\frac{P}{EI}}$$

4.7 Selecting concrete dimensions

In each design cycle, selecting concrete dimensions for members takes place after the structure is analyzed and the stress resultants acting on each member are determined. The reasoning involved in the selecting process is similar to that used when selecting concrete dimensions manually. ESURF selects concrete dimensions to satisfy strength and serviceability requirements as well as constructibility constraints. The nature of these requirements and constraints and the knowledge incorporated in ESURF to satisfy them are described herein.

4.7.1 Constructibility requirements

To facilitate the construction of a building frame, the design must satisfy some practical requirements. Specifying concrete dimensions in discrete increments, utilizing the same concrete cross section for as many members as possible, and considering the criterion by which beams and columns should intersect are examples of these requirements. ESURF possesses the knowledge to account for many constructibility requirements as well as the ability to accommodate any other requirements that are imposed by the user.

Because of constructibility requirements, it was noted during their implementation in ESURF that the iteration process used to determine acceptable concrete dimensions oscillated infinitely between two sets of member dimensions. In other words, using a set of member dimensions for determining member properties for analysis resulted in stress resultants that, when used to proportion members, produced a second set of dimensions. This second set, when used to calculate member properties, resulted in stress resultants that suggested exactly the first set of member dimensions. This was overcome by stopping the iteration process when this phenomenon was detected and choosing the more appropriate dimension set.

4.7.1.1 Intersecting beams with columns

The common practice in construction of building frames is that the width of columns be different from the width of beams to avoid conflict of beam and column reinforcement. Rules that select concrete dimensions in ESURF consider relation between beam and column widths using the following logic:

1. If the beam width is specified:

1a. And the column in conflict with the beam is an exterior column, the column width c_2 is reduced by one dimension increment and the column cross section is reproportioned to calculate the corresponding column depth c_1.

1b. And the column in conflict with the beam is an interior column, the column width is increased by one dimension increment. This is justified because the design of interior columns is often controlled by axial load rather than bending moment.

2. If the beam width is NOT specified:

2a. And the beam depth is less than the maximum beam depth specified by the user (if any), the beam width is reduced by one dimension increment and the beam depth is recalculated based on the reduced width.

2b. And if the beam depth is equal to the maximum beam depth specified by the user, the beam width is increased by one dimension increment and the beam depth is recalculated according to the new width.

When the user specifies (in menu C3) that the width of columns must be equal to the width of beams, other rules are invoked to accommodate this requirement.

4.7.1.2 Beam groups

A beam group is a number of beams having the same concrete dimensions. The selection of beam groups is performed after the bottom floor beam is proportioned. The chosen width of this beam is kept constant for all the beams. This will reduce the versatility in formwork and simplify the reinforcement details. It will also reduce the chance of having conflict in beam-column widths in a case where such an occurrence is to be avoided. The depth of the bottom floor beam will be altered in upper floors only when a significant change in design moment is detected. To test the change in moments, a scanning process is performed for all beams. The process starts by determining the maximum (design) moment for each span in each floor. This moment is designated as $M_{des}(i,j)$ where i refers to the storey number and j refers to the span number. The design moment for each storey $M_{desfl}(i)$ is determined as the maximum moment of $M_{des}(i,j)$ and j is scanned from 1 to NSP (number of spans). The rules for determining the number of beam groups according to the variation in $M_{desfl}(i)$ are based on the following logic:

1. Assume the number of beam groups = 1 and the absolute maximum design moment in all storeys M_{desab} is the design moment for the bottom floor ($M_{des}(1)$).

2. Compare M_{desab} to the design moment of the next floor $M_{des}(i)$. If they differ by more than 40% of M_{desab}, define a new beam group (add one to the number of beam groups).

3. Repeat 2. until roof.

Upon arriving at the number of beam groups, a similar process is done at the group level. This process is designed to check if the depth of a beam group can be reduced for short or lightly loaded spans. For each group, the design moments $M_{des}(i,j)$ are compared to the maximum design moment of that group. If any design moment is found to be less than 60% of the maximum design moment of the group, the beam depth in this span will be reduced.

The procedure described above is a compromise between keeping the same beam dimensions as much as possible to simplify formwork and to keep the concrete volume a minimum.

4.7.1.3 Column stacks

The selection of column stacks follows after the bottom floor columns and all beams are proportioned and any beam-column intersection problem is resolved. The group of rules that perform the selection is activated only if the user did not provide the column dimensions. The minimum number of storeys per column cross section (MINS) plays an important role in selecting column stacks. This number, if not provided by the user, is determined by the system according to the rule in section 4.4.4.

Interior column stack design is controlled by a group of rules based on the following philosophy:

1. Keep the same cross section as long as the required reinforcement ratio is greater than the minimum one.

2. Each cross section should continue for MINS stories or more.

3. For each cross section, reduce ρ as you go up the stack.

4. For any cross section, the required reinforcement ratio should fall between ρ_{maxu} and ρ_{min} except when item 2 needs to be satisfied.

5. When item 2 is satisfied and the required reinforcement is less than ρ_{min}, reducing the cross section is considered. If the cross section is square, both the depth and the width will be decreased to preserve the column shape. If the cross section is not square, the depth will be reduced more than the width (if at all). This is mainly because any rectangularity in the column cross section will not be needed in the upper floors where moments decrease. The column width is reduced such that it matches the beam-column intersecting criterion set by the user. Column dimensions are reduced by either 50 or 100 mm depending on the local circumstances. The reduction is done in a way such that the centroid of the column is almost constant for all floors (Fig. 4.5).

Exterior column stack design is controlled by almost the same rules that control the design of an interior column stack except for item 5 which determines **when** and **how** column dimensions are reduced. The when part is the same, but the how part is different. The width of an exterior column is kept constant to simplify architectural details. Only the column depth c_1 is reduced when indicative.

When implementing the procedure for exterior column stack design, it was observed that the moment in the top exterior column was so great that in many cases it dominated the design of the whole stack. For this special location, the reinforcement ratio was limited by the maximum ratio of 8.0% (CSA A23.3). This can be justified considering the fact that the column steel in this location need not overlap with other column steel and can be anchored by bending into the merging roof beam.

4.7.2 Column dimensions

The proportioning of a column cross section subject to a combined action of axial force (P) and bending moment (M) requires the designer to determine the column depth and width (c_1 and c_2, respectively) and the steel reinforcement that will adequately resist the stress resultants acting on the cross section in an efficient way. This frequently requires the designer to perform a trial and error procedure.

Bezzina and Simmonds (1987) showed that the eccentricity of the applied load (e=M/P) is considered to be a major factor in proportioning an efficient column cross section. In a substudy, many trials of proportioning for different combinations of M and P were performed. A correlation between e/t_o and the ratio c_1/c_2 was established, where t_o is the side length of a square column carrying P with minimum moment. This correlation was based on picking the most efficient section to resist a given M and P. The relationship was cast as follows:

$$\left.\begin{array}{lll} \text{For} & e/t_o \le 0.15 & c_1/c_2 = 1.0 \\ \text{For} & 0.15 < e/t_o < 1.0 & c_1/c_2 = 1.0 + e/t_o \\ \text{For} & e/t_o \ge 1.0 & c_1/c_2 = 2.0 \end{array}\right\} \qquad (4.4)$$

It should be noted that this relationship suggests only the rectangularity ratio of a cross section that will best resist the given M and P. To determine the actual concrete dimensions, the total required area is needed. To estimate the required cross sectional area, the concept of "equivalent load" was employed. The equivalent load is defined, for a given M and P acting on a cross section, as the maximum axial load combined with minimum moment that this cross section can carry. After calculating the equivalent loads for different combinations of M and P, a correlation between e/t_o and P_{eq}/P was established and cast in the form:

$$\left.\begin{array}{lll} \text{For} & e/t_o \le 0.15 & P_{eq}/P = 1.0 \\ \text{For} & e/t_o > 0.15 & P_{eq}/P = 1.0 + e/t_o \end{array}\right\} \qquad (4.5)$$

Once P_{eq} is determined, the total required area of the column cross section A_g can be calculated from the CSA A23.3 equation (10-3):

$$P_{r(max)} = 0.80[0.85\ \phi_c f'_c(A_g - A_{st}) + \phi_s f_y A_{st}] \qquad (4.6)$$

Substituting in Eqn. (4.6) with $P_{r(max)} = P_{eq}$ and $A_{st} = \rho\ A_g$ and rearranging we get:

$$A_g = \frac{P_{eq}}{0.8\,[\,0.85\,\phi_c f'_c + \rho\,(\phi_s f_y - 0.85\,\phi_c f'_c)\,]} \qquad (4.7)$$

Applying Eqn. (4.7) requires specifying the values of reinforcement ratio ρ and the concrete compressive strength f'_c. For ρ, the system uses the maximum reinforcement ratio specified by the user ρ_{maxu}. For f'_c, the system considers the available range of values for this parameter. If more than one value of f'_c is provided for columns, the system decides which value to use. Good engineering practice suggests that only one value of f'_c is to be used for all columns in a given storey. The selection of the value of f'_c to use at each storey is based on interior columns since they contribute more to the concrete volume than exterior columns.

When column dimensions are provided by the user, the system tries to use the minimum value of f'_c specified. If using this value results in reinforcement ratios within the allowable range, the system will choose that value. If not, it will try a higher value of f'_c until reinforcement ratios are satisfactory or the maximum specified f'_c is used, whichever comes first. If the maximum specified f'_c is used and the reinforcement ratios are still beyond the maximum allowable values, the system will convey that to the user indicating the column(s) with $\rho > \rho_{maxu}$.

When the system is to choose the column dimensions, it will proportion the bottom interior column, assuming maximum reinforcement ρ_{maxu}, using the maximum value of f'_c to obtain minimum concrete dimensions. The system will then check if using the minimum value of f'_c will result in the same concrete dimensions, and if true, will use this value of f'_c throughout the column stack. Otherwise, the maximum value of f'_c will be used for the column in bottom storey but lower values of f'_c may be used for columns in higher storeys.

The system determines c_1 and c_2 using Eqn. (4.7), the ratio c_1/c_2, and the constructibility requirements. The system then checks the cross section to ensure strength adequacy. This is done by calculating the resisting moment M_r that the section can withstand at the given axial load level assuming maximum reinforcement. If the resisting moment is greater than the applied one, the section is adequate. Otherwise, the section dimension(s) need to be increased. This is more likely to happen for high e/t_o ratios where it is usually more efficient to increase the column dimension c_1. Hence, the system first considers increasing c_1 by one dimension increment. Strength adequacy is again checked and the process is repeated until the section is adequate. Generally only one increment of increase (if any) is required.

4.7.3 Beam dimensions

The task of proportioning a beam cross section is relatively easier than that of proportioning a column cross section. If no dimensions are specified, assuming a rectangular beam action and no compression reinforcement, most beam dimensions are governed by magnitude of negative moments. The beam effective depth and width (d and b) are related according to Eqn. (4.8) (CPCA, 1985):

$$b\,d^2 = \frac{M_f \times 10^6}{\rho \phi_s f_y \left(1.0 - \dfrac{\rho\,\phi_s f_y}{1.7\,\phi_c f'_c}\right)} \tag{4.8}$$

Where M_f is in kN.m, b and d are in mm, f_y and f'_c are in MPa.

To determine b and d using the above equation, we need to assume the reinforcement ratio ρ and either one of b and d or the ratio between them. A reasonable value for ρ is 0.75 ρ_{max} where ρ_{max} is determined from the code requirement to prevent compression failure:

$$\frac{c}{d} \le \frac{600}{600 + f_y} \tag{4.9}$$

where d is the beam effective depth and c is the neutral axis depth that can be determined from the equation:

$$c = \frac{\phi_s f_y A_s}{\beta_1 (0.85\,\phi_c f'_c)} \tag{4.10}$$

Substituting Eqn. (4.10) in Eqn. (4.9) we get:

$$\frac{\phi_s}{\phi_c} \frac{f_y}{f'_c} \frac{A_s}{0.85\,\beta_1 (b\,d)} \le \frac{600}{600 + f_y} \tag{4.11}$$

Substituting in Eqn. (4.11) for ϕ_s=0.85, ϕ_c=0.6 and $\rho = \frac{A_s}{b\,d}$:

$$\rho_{max} = 0.6\,\beta_1 \frac{f'_c}{f_y} \left(\frac{600}{600 + f_y}\right) \tag{4.12}$$

where

$\beta_1 = 1.09 - (0.008\ f'_c)$ but not greater than 0.85

The common practice in structural design suggests that the d/b ratio should lie in the neighbourhood of 2.0. Substituting in Eqn. (4.8) for $b = d/2.0$ and $\rho = 0.75\ \rho_{max}$ where ρ_{max} can be determined from Eqn. (4.12), the depth d can be determined. The width b is then determined and rounded to the nearest dimension increment. Using this value of b in Eqn. (4.8), the effective depth d is obtained. This value is used in Eqn. (4.13) (assuming bars #25) to determine the total depth h:

$$h = d + \text{cover} + \frac{d_{bar}}{2.0} \qquad (4.13)$$

The total depth h obtained from this equation is then rounded to the nearest dimension increment.

If one of the two beam dimensions (h or b) is given by the user, the other dimension can be easily obtained from Eqn. (4.8) assuming the same reinforcement ratio as before. Upon determining the section dimensions, the maximum factored shear stress in the beam is calculated and compared to the allowable shear stress by CSA A23.3 (see section 4.8.2). If the shear stress in the beam is excessive, although rare, the beam width is increased.

The system can also handle T-shaped beam cross sections that are encountered in mid-span regions where positive bending moments usually act.

4.8 Selecting reinforcement

The determination of the longitudinal and transverse reinforcement for each member in the frame is referred to in the rules as detailed design. Determining reinforcement includes specifying the number, diameter(s) and arrangement of the reinforcing bars. The process takes place after all member dimensions have been selected and the required reinforcement ratios determined. The rules that choose reinforcement for beams and columns are based on the following considerations.

4.8.1 Column reinforcement

Since exterior columns are often subject to significant bending moments while interior columns carry mainly axial loads, it was decided to place longitudinal reinforcement on two faces for exterior columns and on four faces for interior columns. Having decided that, the number and diameters of bars are still to be determined. The traditional way of doing this is to assume one of the two unknowns (the number or diameter of bars) and calculate the other one. This is usually repeated in a trial and error procedure until a choice that satisfies strength and spacing requirements is obtained. However, this approach may not be the most suitable for automation. Another approach suggested here is to store all the plausible combinations of bar numbers and diameters. Those combinations were chosen based on the following criteria (assuming bar #10 is ranked #1 and bar #35 is ranked #6):

1. Maximum bar size allowed is bar #35 (rank 6).
2. Minimum bar size allowed is bar #15 (rank 2).
3. Bars are arranged in double symmetry.
4. A maximum of two successive bar diameters are allowed in the same column cross section.

The choices of reinforcement that satisfy the above requirements and give an area of reinforcing steel between 800 and 16,000 mm^2 are tabulated in Tables 4.1 and 4.2 for interior and exterior columns, respectively. The 800 mm^2 area of steel is the absolute minimum reinforcement (4#15) that has the minimum number of bars (4) and the minimum bar diameter (bar #15). The 16,000 mm^2 upper limit is roughly the maximum reinforcement ratio (4.44%) of a 600x600 mm column which is thought of as the largest column for the frames in the domain of this study. There are 32 reinforcement choices for interior columns (four-face arrangement) and 60 choices for exterior columns (two-face arrangement). However, those choices are stored in electronic files for which more choices can be easily added. For some areas of steel, two choices of reinforcement were available of which the one with the fewer reinforcing bars is considered first.

The system will search the appropriate table of choices for the reinforcement combination with the closest area of steel to the required one. This is done by dividing the required area of steel by 400 for interior columns and by 200 for exterior columns to get a reinforcement index. This index is used to choose the reinforcement with the closest index (i.e. area of steel) to the required one. Once a choice has been selected, it will be checked against minimum and maximum bar spacings. Strength adequacy of the cross section will then be checked. If the section proves to be inadequate, the system will consider the next choice of reinforcement on the corresponding table and checks if spacing and strength requirements are satisfied.

Ties for both exterior and interior columns are chosen as #10 bars spaced at the smallest of:

1. 16 times the smallest longitudinal bar diameter.
2. 48 times the tie diameter (540 mm).
3. The smaller dimension of the column.

4.8.2 Beam reinforcement

The choice of longitudinal reinforcement for beams is a relatively easier task than that for columns. The same approach for choosing longitudinal reinforcement for columns described in section 4.8.1 was adopted for beams. Table 4.3 shows the different reinforcement choices for beams based on similar criteria as for columns. The reinforcement index in this table represents the area of reinforcement (in mm^2) divided by 100. The system searches the table with an index of the required reinforcement for the closest reinforcement index to that one. The chosen reinforcement will be checked to ensure strength adequacy, bar spacing and minimum number of bars for crack width control (CSA A23.3 clause 10.6.4). If the reinforcement choice does not satisfy all of the above requirements, the system will try the next choice in line until all requirements are satisfied.

The choice of transverse reinforcement for beams requires determining the number and spacing of stirrups along each span. The bars used as stirrups are #10 bars. The system starts the process by calculating four shear stress values for each span to determine the shear envelope on that span. The four values are determined assuming symmetry of gravity loads and reversibility of lateral loads as shown in Fig. 4.6.

Once the shear envelope is determined, four limiting shear stress values are calculated:

$$Vl_1 = 0.5\ V_c \tag{4.14}$$

$$Vl_2 = V_c + V_{s\ min} \tag{4.15}$$

$$Vl_3 = V_c + 2.0\ V_{co} \tag{4.16}$$

$$Vl_4 = V_c + 4.0\ V_{co} \tag{4.17}$$

where

$$V_{co} = 0.2\,\lambda\,\phi_c\sqrt{f_c'} \tag{4.18}$$

For members subject to axial tension:

$$V_c = V_{co}\left(1 - \frac{N_f}{N_r}\right) \tag{4.19}$$

For members subject to axial compression:

$$V_c = V_{co}(1 - \frac{3N_f}{A_g f'_c}) \qquad (4.20)$$

where

$$N_r = 0.6\lambda\,\phi_c \sqrt{f'_c}\,A_g \qquad (4.21)$$

and N_f is the factored axial force (positive in tension).

The shear stress associated with minimum area of stirrups is:

$$V_{smin} = \frac{\phi_s A_{vmin} f_y}{b_w S} \qquad (4.22)$$

where

$$A_{vmin} = 0.35 \frac{b_w S}{f_y} \qquad (4.23)$$

Substituting Eqn. (4.23) in Eqn. (4.22):

$$V_{smin} = 0.35\,\phi_s \qquad (4.24)$$

The value of Vl_4 is the maximum allowable shear stress along the beam. During beam proportioning, the maximum applied shear stress was ensured to be lower than this value.

Depending on the shear stress distribution along the span each half span will have from one to three shear zones. A shear zone is a distance along the beam longitudinal axis in which the stirrups spacing is constant. The limits of these zones are illustrated in Fig. 4.7.

In zone Z_4 (heavy reinforcement), the spacing S is governed by:

$$S \leq \frac{\phi_s A_v f_y}{b_w (V_f - V_c)}$$

$$S \leq \frac{d}{4.0}$$

$$S \leq 300 \text{ mm}$$

In zone Z_3 (normal reinforcement) :

$$S \leq \frac{\phi_s A_v f_y}{b_w (V_f - V_c)}$$

$$S \leq \frac{d}{2.0}$$

$$S \leq 600 \text{ mm}$$

In zone Z_2 (minimum reinforcement):

$$S \leq \frac{A_v f_y}{0.35\, b_w}$$

$$S \leq \frac{d}{2.0}$$

$$S \leq 600 \text{ mm}$$

In zone Z_1, stirrups are not required.

For each zone, the spacing is rounded to a specified increment. The length of each zone is calculated accordingly before moving to the next zone (if needed).

4.9 Organization of rules

In ESURF, to facilitate organization of the knowledge base, rules are grouped in five rule modules. The first module contains thirty-nine rules for member proportioning. The second, third and fourth modules contain twenty-four rules for selecting transverse and longitudinal reinforcement of beams and columns. The fifth module contains six rules for the user interface.

Examples of ESURF rules in OPS83 format and their English meaning are listed in appendix C. All ESURF rules are listed in appendix E in English form.

Rt. index	First choice					Second choice				
	a	b	c	d	Reinf.	a	b	c	d	Reinf.
2	4	2	1	4	4#15	-	-	-	-	-
3	4	3	1	4	4#20	-	-	-	-	-
4	8	2	1	8	8#15	-	-	-	-	-
5	4	4	1	4	4#25	8	3	2	4	4#20+4#15
6	8	4	1	8	8#20	12	2	1	12	12#15
7	4	5	1	4	4#30	12	3	2	4	4#20+8#15
8	8	5	2	4	4#25+4#20	16	2	1	16	16#15
9	12	3	1	12	12#20	16	3	2	4	4#20+12#15
10	4	6	1	4	4#35	8	4	1	8	8#25
11	12	4	2	4	4#25+8#20	-	-	-	-	-
12	8	5	2	4	4#30+4#25	16	3	1	16	16#20
14	8	5	1	8	8#30	16	4	2	4	4#25+12#20
15	12	4	1	12	12#25	-	-	-	-	-
17	8	6	2	4	4#35+4#30	12	5	2	4	4#30+8#25
20	8	6	1	8	8#35	16	4	1	16	16#25
21	12	5	1	12	12#30	-	-	-	-	-
22	16	5	2	4	4#30+12#25	-	-	-	-	-
24	12	6	2	4	4#35+8#30	-	-	-	-	-
28	16	5	1	16	16#30	-	-	-	-	-
30	12	6	1	12	12#35	-	-	-	-	-
31	16	6	2	4	4#35+12#30	-	-	-	-	-
40	16	6	1	16	16#35	-	-	-	-	-

a = total number of bars　　b = rank of bigger diameter bar
c = number of bar diameters　d = number of bigger diameter bars
Rt. index = reinf. area (sq. mm) / 400

Table 4.1 Interior column reinforcements

Rt. index	First choice					Second choice				
	a	b	c	d	Reinf.	a	b	c	d	Reinf.
4	4	2	1	4	4#15	-	-	-	-	-
6	4	3	1	4	4#20	6	2	1	6	6#15
7	6	3	2	2	4#15+2#20	-	-	-	-	-
8	6	3	2	4	4#20+2#15	8	2	1	8	8#15
10	4	4	1	4	4#25	8	3	2	4	4#20+4#15
11	6	4	2	2	4#20+2#25	-	-	-	-	-
12	8	4	1	8	8#20	12	2	1	12	12#15
13	6	4	2	4	4#25+2#20	-	-	-	-	-
14	4	5	1	4	4#30	12	3	2	4	4#20+8#15
15	6	4	1	6	6#25	10	3	1	10	10#20
16	8	5	2	4	4#25+4#20	16	2	1	16	16#15
17	6	5	2	2	4#25+2#30	-	-	-	-	-
18	12	3	1	12	12#20	16	3	2	4	4#20+12#15
19	6	5	2	4	4#30+2#25	10	4	2	4	4#25+6#20
20	4	6	1	4	4#35	8	4	1	8	8#25
21	6	5	1	6	6#30	14	3	1	14	14#20
22	12	4	2	4	4#25+8#20	-	-	-	-	-
24	8	5	2	4	4#30+4#25	16	3	1	16	16#20
25	10	4	1	10	10#25	-	-	-	-	-
27	6	6	2	4	4#35+2#30	-	-	-	-	-
28	8	5	1	8	8#30	16	4	2	4	4#25+12#20
29	10	5	2	4	4#30+6#25	-	-	-	-	-
30	6	6	1	6	6#35	12	4	1	12	12#25
34	8	6	2	4	4#35+4#30	12	5	2	4	4#30+8#25
35	10	5	1	10	10#30	14	4	1	14	14#25
38	12	5	2	8	8#30+4#25	-	-	-	-	-
40	8	6	1	8	8#35	16	4	1	16	16#25
41	10	6	2	4	4#35+6#30	-	-	-	-	-
42	12	5	1	12	12#30	-	-	-	-	-
43	14	5	2	8	8#30+6#25	-	-	-	-	-
44	16	5	2	4	4#30+12#25	-	-	-	-	-
48	12	6	2	4	4#35+8#30	16	5	2	8	8#30+8#25
49	14	5	1	14	14#30	-	-	-	-	-
50	10	6	1	10	10#35	-	-	-	-	-
54	12	6	2	8	8#35+4#30	-	-	-	-	-
56	16	5	1	16	16#30	-	-	-	-	-
60	12	6	1	12	12#35	-	-	-	-	-
61	14	6	2	8	8#35+6#30	-	-	-	-	-
62	16	6	2	4	4#35+12#30	-	-	-	-	-
68	16	6	2	8	8#35+8#30	-	-	-	-	-
70	14	6	1	14	14#35	-	-	-	-	-
80	16	6	1	16	16#35	-	-	-	-	-

a = total number of bars b = rank of bigger diameter bar
c = number of bar diameters d = number of bigger diameter bars
Rt. index = reinf. area (sq. mm) / 200

Table 4.2 Exterior column reinforcements

Rt. index	First choice					Second choice				
	a	b	c	d	Reinf.	a	b	c	d	Reinf.
4	2	2	1	2	2#15	-	-	-	-	-
6	2	3	1	2	2#20	3	2	1	3	3#15
8	3	3	2	2	2#20+1#15	4	2	1	4	4#15
9	3	3	1	3	3#20	-	-	-	-	-
10	2	4	1	2	2#25	4	3	2	2	2#20+2#15
11	3	4	2	2	2#20+1#25	5	3	2	1	4#15+1#20
12	4	3	1	4	4#20	6	2	1	6	6#15
13	3	4	2	2	2#25+1#20	-	-	-	-	-
14	2	5	1	2	2#30	-	-	-	-	-
15	3	4	1	3	3#25	5	3	1	5	5#20
16	4	4	2	2	2#25+2#20	-	-	-	-	-
17	3	4	2	1	2#25+1#30	5	4	2	1	4#20+1#25
18	6	3	1	6	6#20	-	-	-	-	-
19	3	5	2	2	2#30+1#25	5	4	2	2	2#25+3#20
20	4	4	1	4	4#25	2	6	1	2	2#35
21	3	5	1	3	3#30	5	4	2	3	3#25+2#20
22	6	4	2	2	4#20+2#25	-	-	-	-	-
24	4	5	2	2	2#30+2#25	3	6	2	1	2#30+1#35
25	5	4	1	5	5#25	-	-	-	-	-
26	6	4	2	4	4#25+2#20	-	-	-	-	-
27	3	6	2	2	2#35+1#30	5	5	2	1	4#25+1#30
28	4	5	1	4	4#30	-	-	-	-	-
29	5	5	1	2	3#25+2#30	-	-	-	-	-
30	3	6	1	3	3#35	6	4	1	6	6#25
31	5	5	2	3	3#30+2#25	-	-	-	-	-
34	4	6	2	2	2#35+2#30	6	5	2	2	4#25+2#30
35	5	5	1	5	5#30	16	4	1	16	16#25
36	6	5	2	3	3#30+3#25	-	-	-	-	-
38	6	5	2	4	4#30+2#25	-	-	-	-	-
40	4	6	1	4	4#35	-	-	-	-	-
41	5	6	2	2	3#30+2#35	-	-	-	-	-
42	6	5	1	6	6#30	-	-	-	-	-
43	7	5	2	4	4#30+3#25	-	-	-	-	-
44	5	6	2	3	3#35+2#30	-	-	-	-	-
48	8	6	2	4	4#35+4#30	-	-	-	-	-
49	7	5	1	7	7#30	-	-	-	-	-
50	5	6	1	5	5#35	-	-	-	-	-
51	6	6	2	3	3#35+3#30	-	-	-	-	-
54	6	6	2	4	4#35+2#30	-	-	-	-	-
60	6	6	1	6	6#35	-	-	-	-	-
61	7	6	2	4	4#35+3#30	-	-	-	-	-
70	7	6	1	7	7#35	-	-	-	-	-
80	8	6	1	8	8#35	-	-	-	-	-

a = total number of bars   b = rank of bigger diameter bar
c = number of bar diameters d = number of bigger diameter bars
Rt. index = reinf. area (sq. mm) / 100

Table 4.3 Beam reinforcements

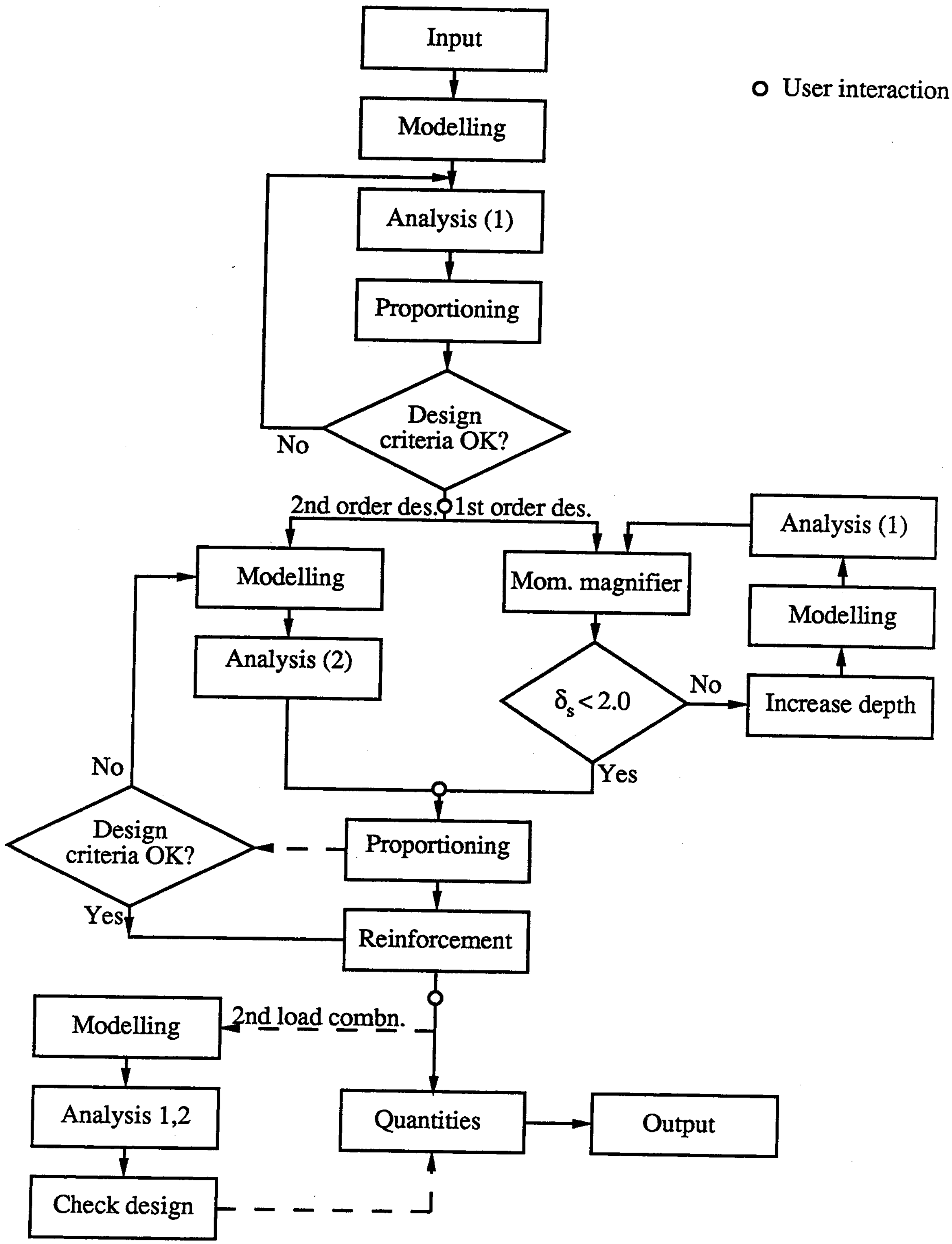


Fig. 4.1 System Overview

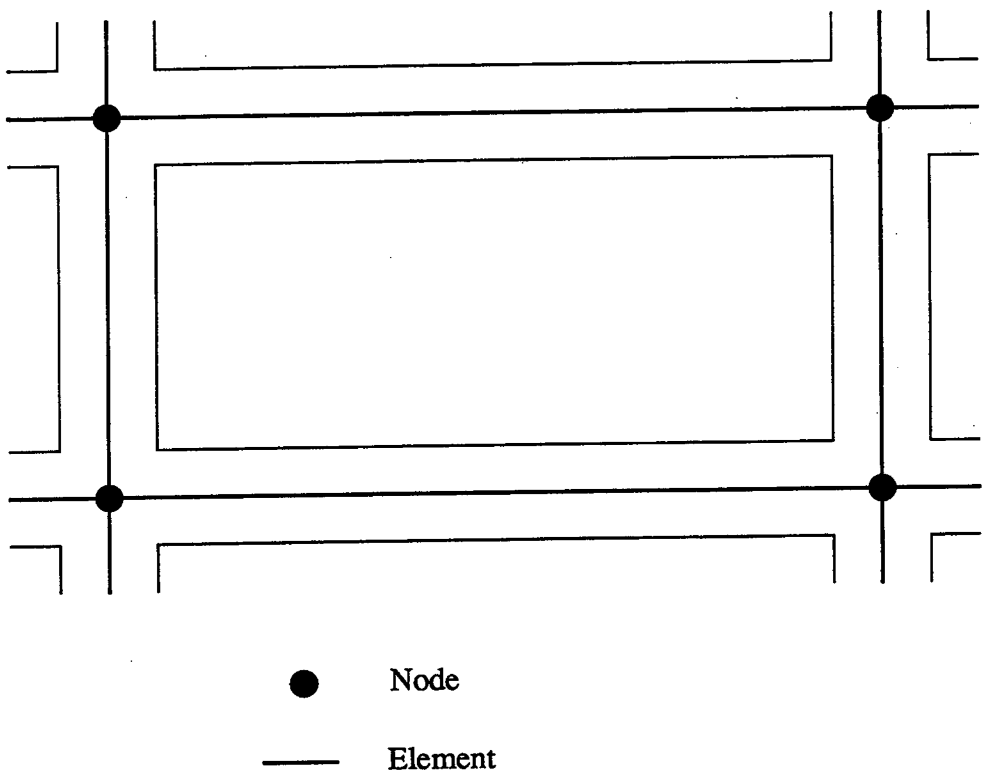


Fig. 4.2 Model A

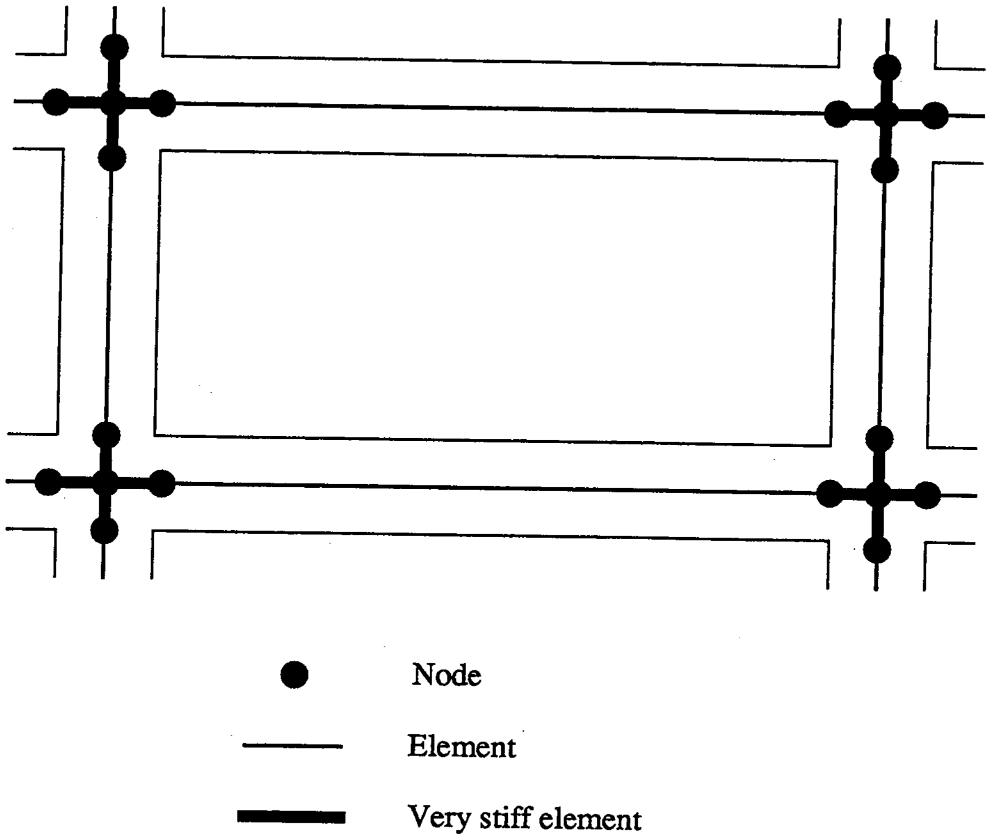


Fig. 4.3 Model B

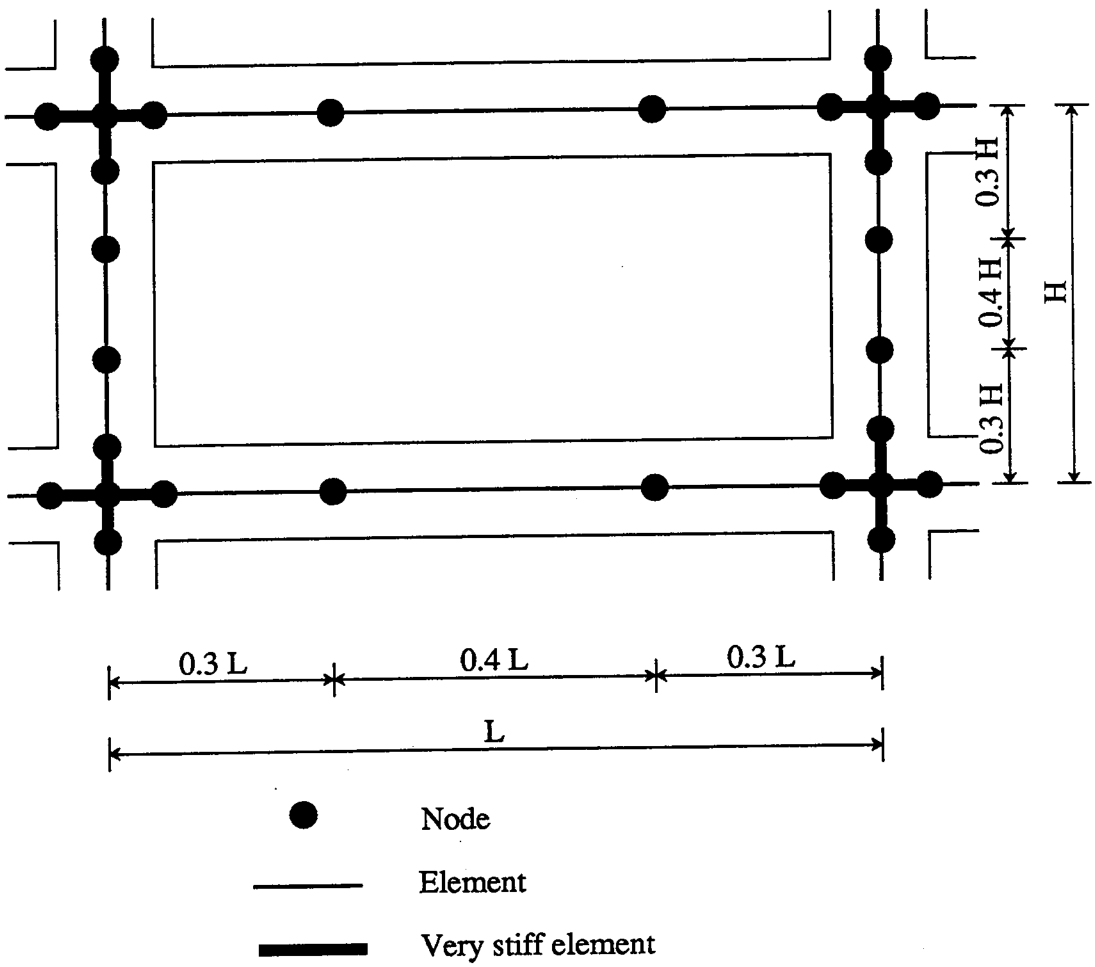


Fig. 4.4 Model C

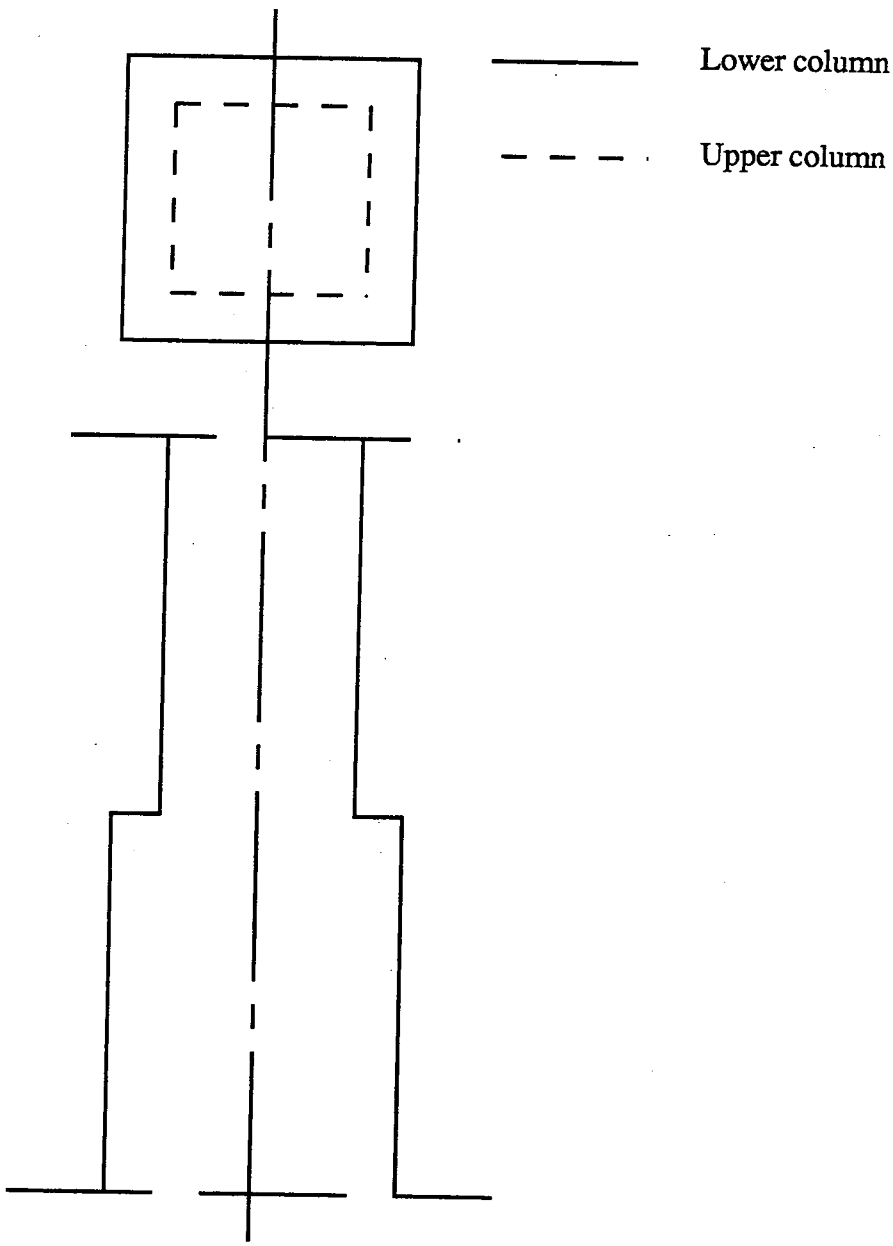
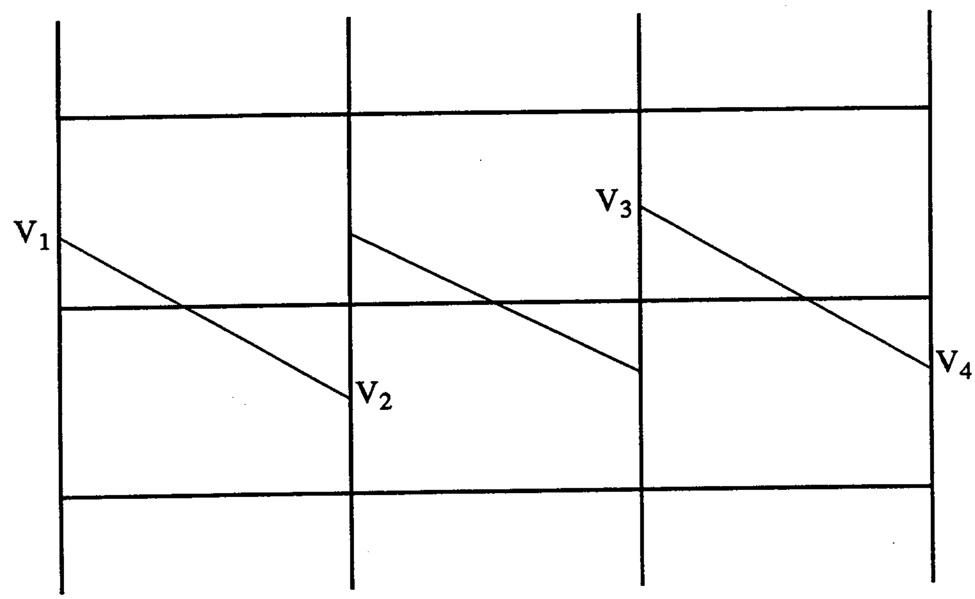
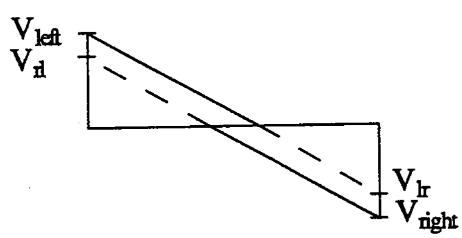


Fig. 4.5 Reducing interior column cross section





V_{left} = larger of V_1 and V_4
$V_{lr} = V_2$ if $V_{left} = V_1$
$V_{lr} = V_3$ if $V_{left} = V_4$

V_{right} = larger of V_2 and V_3
$V_{rl} = V_1$ if $V_{right} = V_2$
$V_{rl} = V_4$ if $V_{right} = V_3$

Fig. 4.6 Shear envelope determination

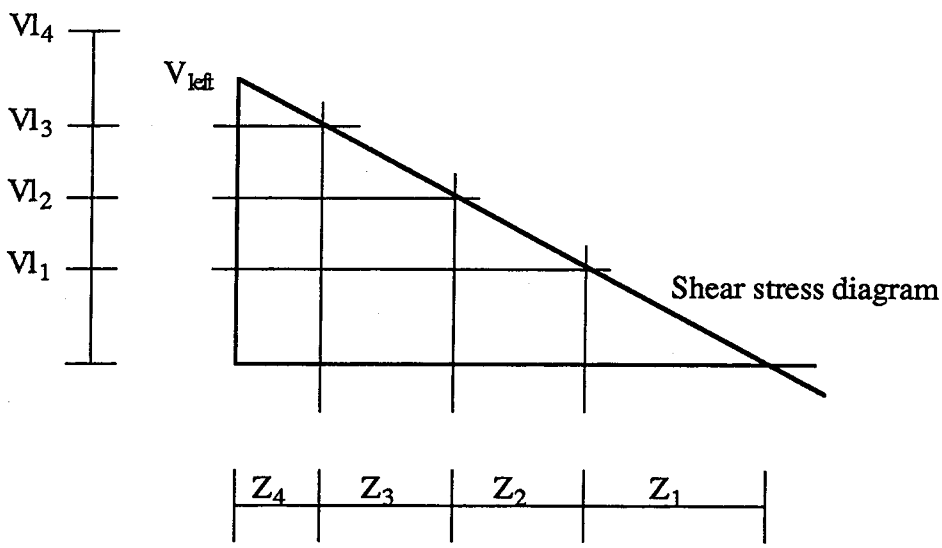


Fig. 4.7 Shear zones

5. Evaluation of Modelling Techniques

5.1 Introduction and objectives

When analyzing a structure, the principles of statics must be satisfied regardless of the way the structure is modelled. However, the differences in the manner in which the model considers various effects may lead to differences in the final design. The objective of this study is to determine the magnitude of these differences and their significance on the final design of unbraced building frames.

The study described in this chapter has the following objectives:

1. to study the effect of modelling joint sizes.
2. to study the impact of using different sets of effective member stiffnesses.
3. to study the effect of selected construction constraints on the final design.
4. to evaluate some design aspects regarding loading (load combinations and live load reduction factors (LLRF)).

In undertaking these objectives, five design models incorporating different combinations of modelling and analytical techniques were considered. While not exhaustive, these models were selected to facilitate evaluating the effects of different aspects of modelling.

Each design model is designated by a letter and a number. The letter refers to the geometry model (section 4.5) and the number refers to the order of design (first or second). The designations of the five selected models and their meanings are as follows:

1. Design model **A1** is a first-order design using model A (section 4.5.1).
2. Design model **A2** is a second-order design using model A.
3. Design model **B1** is a first-order design with finite joint sizes and effective member stiffnesses based on model B (section 4.5.2).
4. Design model **B2** is a second-order design using model B.
5. Design model **C2** is a second-order design with finite joint sizes and uses the CEB model to determine member stiffnesses (model C, section 4.5.3).

In the following studies, designs obtained from different modelling procedures are compared on the basis of bending moments in members and material quantities and cost indices for the overall frame.

5.2 Test frame designations and variables

To facilitate comparing different designs, all frames considered in the following studies used the same gravity loading represented by uniformly distributed loads having a nominal value of 25.0 kN/m for dead load and 17.0 kN/m for live load. Lateral loading was represented as a concentrated load at each floor level with a magnitude of W kN (0.5W at roof level). Values of W were selected to represent different wind intensities and taken to be 0.0 kN for no wind load (W0), 14.0 kN for moderate wind load (W1), and 28.0 kN for heavy wind load (W2).

Two load combinations are implemented in ESURF as specified by the National Building Code of Canada and designated as:

Load combination #1 : $\alpha_D \; D + \gamma\psi \, (\alpha_L \; L + \alpha_W \; W)$

Load combination #2 : $\alpha_D \; D + \gamma\psi \, (\alpha_L \; L)$

where D, L, and W are dead load, live load, and wind load intensities, respectively and:

$\alpha_D = 1.25$, $\alpha_L = 1.50$ and $\alpha_W = 1.50$

$\gamma = 1.0$ (importance factor)

$\psi = 1.0$ for load combination #2

$= 0.7$ for load combination #1

Frames are designed according to load combination #1 (W1 or W2) and the design is checked for load combination #2 (W0).

Each test frame is designated by the notation:

<name>.<loading>.<design model>

where <name> indicates the number of spans and storeys in the frame, <loading> indicates wind load intensity (W0, W1 or W2), and <design model> is the design model used as defined in the previous section. Thus a frame having 3 bays and 12 storeys, resisting moderate wind and designed using model A2 would be designated as 3B12S.W1.A2.

Three different frame geometries representative of typical building types were selected as follows:

1. High-rise frame with equal spans (3B12S shown in Fig. 5.1).
2. Low-rise frame with equal spans (3B4S shown in Fig. 5.2).
3. Institutional building frame with unequal spans (3B5S shown in Fig. 5.3).

Cost indices were calculated as described in section 4.3 based on the following unit prices which are considered to be representative of construction cost in the City of Edmonton during the summer of 1991:

For concrete	$121.00/m^3$
For forms	$40.50/m^2$
For steel	$0.95/kg

5.3 Modelling of joint sizes

The finite size of joints affects the stress resultants in members of a building frame. Two procedures are used by design engineers to consider this effect. The first is to perform the frame analysis based on the centreline dimensions and then to reduce the stress resultants in members to account for the joint size effect. This procedure is implemented in design model A described in section 4.5.1. The second procedure is to explicitly consider the joint size effect in the analysis by modelling the joint regions as infinitely stiff members. This procedure is implemented in design model B described in section 4.5.2.

When using the first procedure, there is a variety of methods in the literature for reducing moments to the face of the joint. A single reduction based on the area under the shear force diagram does not account for the joint stiffening effect. The method implemented in model A is based on the PCA (1959) recommendation that reduces centreline beam negative moments by $Vc_1/3$ where V is the centreline shear force and c_1 is the column depth. This value accounts for the increased negative moment due to the haunching effect of the stiff column region as well as the reduction due to the decreased span. For columns, no similar reduction was implemented as explained at the end of this section.

When using the second procedure (model B), a stiffness value for the members representing the joint regions must be selected. Since an infinite value of stiffness is not practical, a suitable value needs to be assessed. For this purpose, the frame design 3B4S.W1 was performed for both first-order and second-order modes using three values of stiffness corresponding to 10, 100, and 10,000 times the stiffness of the member outside the joint region. Design moments and material quantities were scanned to estimate the effect of the numerical way of representing infinite stiffness. Designs based on a multiplier value of 100 and 10,000 were identical in design moments, and hence in material quantities. The design based on a multiplier value of 10 gave slightly different design moments (less than 1%) and less than 0.5% difference in cost indices. This suggested that to represent infinite stiffness for joint regions, it is sufficient to use the section properties of a neighbouring element multiplied by a factor of 100 or greater.

To evaluate the joint size effect, designs based on the two models (model A and model B) are compared for each of the three test frames defined in the previous section. For both models, based on the recommendation of the explanatory notes of the CPCA (1985), member effective stiffnesses were taken to be half of their gross stiffnesses. The effect of this factor on the final design is discussed in the next section.

Each test frame was designed using models A and B in a first-order and a second-order design mode for two load combinations (load combination #1 and #2 defined in section 5.2). Selected design moments in beams and columns were recorded and compared with corresponding values from other designs. Also final designs were compared in terms of material quantities and cost indices.

Figs. 5.4 to 5.12 show design moments in beams, exterior columns and interior columns of each of the three test frames with design models A and B using a first-order design. Figs. 5.13 to 5.21 show design moments using a second-order design. Tables 5.1 to 5.3 show the concrete dimensions for beams, exterior columns and interior columns for design models A and B with first-order design and Tables 5.5 to 5.7 show that with second-order design. Table 5.4

shows the concrete volume, the steel mass, the form surface area and the cost index for each first-order design and Table 5.8 shows that for each second-order design.

For test frame 3B12S, Figs. 5.4(a) to 5.6(a) and 5.13(a) to 5.15(a) show that design moments in exterior columns using model A were greater than those using model B, except for the top story columns, by 5% to 15%. This difference was reflected in some cases (design 3B12S.W2.A2) by a one-increment larger column depth (Table 5.5). For interior columns, Figs. 5.4(b) to 5.6(b) and 5.13(b) to 5.15(b) show that interior column design moments from model A were generally greater than those from model B by 15% to 35% for first order design and 10% to 25% for second-order design. The difference was not significant enough to affect the interior column dimensions (except for 3B12S.W2.A2 design where a 600x500 mm cross section was selected compared to a 550x550 mm cross section selected in B2 design). Figs. 5.4(c) to 5.6(c) and 5.13(c) to 5.15(c) show that design moments in beams with the A model were different by less than 4% from those with the B model. All beam concrete dimensions were identical in both cases. However, the impact of the above differences on the total cost index of the frame (Tables 5.4 and 5.8) was less than 0.75%.

For test frame 3B4S, Figs. 5.7(a) to 5.9(a) and 5.16(a) to 5.18(a) show that design moments in exterior columns from model A were generally lower than those from model B by 10%, except for the W2.A2 case where moments from model A were greater than moments from model B by 10%. For this case, the exterior column cross section was one increment larger in the A2 design than that in the B2 design (Table 5.6). For interior columns, Figs. 5.7(b) to 5.9(b) and 5.16(b) to 5.18(b) show that design moments obtained using model A were 15% to 50% greater than moments obtained using model B. The difference caused a change in concrete dimensions only in the W2.A2 design where a 350x300 mm column was selected based on an A2 design compared to a 300x300 mm column based on a B2 design. Figs. 5.7(c) to 5.9(c) and 5.16(c) to 5.18(c) show a difference of less than 4% in beam moments from models A and B. This difference did not cause any beam dimension in model A to differ from that in model B.

The above differences in moments and concrete dimensions caused the cost index to differ only by less than two thirds of a percent (Tables 5.4 and 5.8).

For test frame 3B5S, Figs. 5.10(a) to 5.12(a) and 5.19(a) to 5.21(a) show that design moments in exterior columns from model A were lower than those from model B by as much as 15% to 20%, except for 3B5S.W2 where design moments from model A were greater than those from model B by as much as 17%. This was reflected by a 50 mm larger exterior column depth for wind intensity W1 using model B than that using model A. For wind intensity W2, exterior column was 50 mm larger in depth using model A than that using model B. For interior columns, Figs. 5.10(b) to 5.12(b) and 5.19(b) to 5.21(b) show that design moments using model B were 10% greater than those using model A, except for W2.A1 design where design moments were greater by up to 30% than those from model B. The 10% difference in design moment was not enough to change the interior column dimensions while the 30% difference in moment caused the interior column depth in model B1 to be 50 mm larger than that in model A1. Figs. 5.10(c) to 5.12(c) and 5.19(c) to 5.21(c) show that differences in beam design moments from design models A and B were less than 4%.

The above differences were reflected in the total cost index by less than two thirds of a percent difference (Tables 5.4 and 5.8).

From the above discussion, we can see that considering the joint size effect for beams when using model A by reducing the centreline moments by $Vc_1/3$ is satisfactory. However, the centreline moments for columns obtained using model A were in some cases smaller than the moments obtained at the beam face using model B. This is illustrated in Fig. 5.22 where the increased stiffness of the joint area resulted in significantly larger column centreline moments when using model B. This indicated that reducing the centreline column moments to account for end stiffening effect is, in some cases, on the unsafe side. It is therefore recommended that centreline moments for columns obtained using model A should be used in design without reductions.

It is concluded that the effect of joint size for unbraced frames can be considered either implicitly by reducing the beam moments (model A) or explicitly by modelling the joint regions as very stiff elements (model B). The differences in beam design moments were insignificant. Column design moments using model A were in some cases greater than those using model B which led to a one increment difference in column dimensions. The impact of these differences on the cost index was trivial. Model A is simpler than model B in that it requires fewer number of joints and members. On the other hand, member stress resultants from model A need to be modified to account for joint size effect.

5.4 Modelling of member stiffnesses

Representation of member stiffnesses when analyzing concrete structures is considered to be a major problem (Mathews and MacGregor 1977). From a practical point of view, a single value for each member stiffness is desirable. Ideally, this value should reflect the effect of cracking, reinforcement, creep, axial load and inelastic behaviour of concrete and steel. It should also account for the variation of cracking and amount of reinforcement along the entire length of the member.

Most design codes require that, for a second-order analysis, member stiffnesses must represent the state of the structure just prior to reaching ultimate. Different sets of values are suggested in the literature:

1. The explanatory notes of the CPCA suggest using $0.5E_cI_g$ for both beams and columns.

2. The ACI Commentary 1989 (clause R10.10.1) suggests using $0.5E_cI_g$ for beams and $(0.2+1.2\ \rho\ E_s/E_c)\ E_cI_g$ for columns. For an average modular ratio (E_s/E_c) of 7.0 and a reinforcement ratio of 4.0%, the above expression for column stiffnesses is very close to $0.5E_cI_g$. For smaller values of column reinforcement ratios, the expression gives smaller stiffness values such that for minimum reinforcement ratio (1.0%), the column stiffness approaches $0.3E_cI_g$.

3. Kordina (1973) recommended the fraction of E_cI_g to be used as an effective stiffness is $(0.2+15\rho)$ for columns and $(0.3+10\rho)$ for beams.

4. Hage (1974) suggested using $0.4E_cI_g$ for beams and $0.8E_cI_g$ for columns.

The implications of using one set of member stiffnesses or another on the final design is the objective of the following study. In undertaking this objective, a decision is required as to which of the design models more closely represents the behaviour of the frame being modelled. It is assumed that the solution obtained using design model C2, as it incorporates the most accurate evaluation of stiffness, is the best solution and is used as the reference design. The design model B2 is used with a moderate wind load (W1) to design different frames. In each frame design, the effective stiffness is taken to be $C_bE_cI_g$ for all beams and $C_cE_cI_g$ for all columns where E_cI_g is the gross stiffness of the member. Considering the above guidelines, the parameters C_b and C_c were assigned different values (0.4 and 0.5 for C_b and 0.3, 0.5, 0.8, and 1.0 for C_c). For each set of C_b and C_c values, a complete second-order design was carried out using design model B2 and the same moderate wind load (W1).

To compare different designs, the following information was recorded for each design:

1. Concrete dimensions for beams, exterior columns and interior columns.

2. Design moments for beams, exterior columns, and interior columns for both load combinations (M_I for load combination #1 and M_{II} for load combination #2). These moments were taken at the bottom floor level for interior columns and beams and at the second floor level for exterior columns as they have maximum values at these locations. The design moments for the reference design were taken as unity and all the moments from other designs were referenced to this set of moments.

3. Concrete volume, steel mass, forms surface area, and cost index.

Tables 5.9 to 5.11 show the results of different designs. In these tables, the <frame> part of the designation of all designs is omitted and can be obtained from the table heading. The reference design was hence referred to as W1C2. Since all other designs have the designation <frame>.W1.B2, this part is also omitted and replaced by the designation CxCy where x and y refer to the values (without the decimal point) of C_b and C_c, respectively. For example, C5C8 refers to a design in which C_b has a value of 0.5 and C_c has a value of 0.8.

The results show that design moments differed by as much as 15% for beams and 45% for columns. However, differences in concrete dimensions were at most one dimension increment (50 mm) for beams and two dimension increments (100 mm) for columns. Cost indices differed by less than 5%.

Keeping that in mind, let us now have a closer look at different designs as compared to the reference design. Tables 5.9 to 5.11 show that, for all frames, the C5C3 design underestimates the design moments in exterior columns by as much as 45%. It also underestimates the design moments in interior columns for frame 3B5S by 45%. The C5C10 design gave significantly smaller design moments in interior columns (50% less) and also in beams (11% less). This indicates that neither 0.3 nor 1.0 is a good estimate for C_c.

Looking at design C4C5 vs. C5C5 (or C4C8 vs. C5C8) we find very slight difference in designs, if at all. This suggests that a 0.5 value for C_b is as good as 0.4. On the other hand, if we look at design C5C5 vs. C5C8 (or C4C5 vs. C4C8) we find that a 0.5 value for C_c gives more realistic design moments in beams (except for 3B5S) while it slightly underestimates design moments in exterior columns. A coefficient of 0.8 for C_c gives quite the opposite. Obviously there is no single set of C_b and C_c that will exactly fit the reference design in all cases. However, there is no significant difference in design when C_b and C_c take reasonable values (0.4-0.5 for C_b and 0.5-0.8 for C_c).

This comparison also sheds some light on the effect of considering creep in design by reducing the effective stiffness of columns. Comparing designs C5C5 and C5C8, we can see that reducing the effective stiffness of columns from 0.8 to 0.5 their gross stiffness (37.5% reduction) was reflected by a very slight difference in the final design (one dimension increment in some members and less than 1% difference in cost index).

In the above study, when the CEB model was used to evaluate member stiffness, the value of β_b was taken to be 1.0. Reference design W1C2 for the three test frames was rerun using a more realistic value of 0.5 for β_b. This did not have any impact on column concrete dimensions in all three frames. The same was observed for beam concrete dimensions except for frame 3B4S where the same depth was used and the width was increased by one increment. This shows that the final reference design (W1C2) is not sensitive to the value of β_b used in evaluating member stiffnesses in the CEB model when this value is in the range of 0.5 to 1.0.

It is therefore concluded that different values for effective member stiffnesses do not result in significantly different designs as long as these values are within a reasonable range. A value of $0.5E_cI_g$ is recommended for both beams and columns which resulted in reasonable estimates of moments. This recommendation agrees with those of the CPCA explanatory notes and the ACI commentary. This value of effective stiffness is used for all designs in the remainder of this study except for C2 designs.

To verify this conclusion, the test frames were designed for heavy wind load (W2) using design model B2 with member effective stiffnesses being 0.5 times their gross stiffnesses and again using design model C2 as the reference design. The results of these designs for wind load W2 as well as wind load W1 from the previous study are presented in Figs. 5.13 to 5.21 and Tables 5.5 to 5.8.

Figs. 5.13 to 5.21 show design moments in beams, exterior columns and interior columns for each of the three test frames using design models A2, B2, and C2. Tables 5.5 to 5.7 show the concrete dimensions for beams, exterior columns, and interior columns for each design model. Table 5.8 shows the concrete volume, the mass of steel, form surface area, and cost index for each design.

For test frame 3B12S, Figs. 5.13(a) to 5.15(a) show that design moments in exterior columns using model B2 are smaller than those obtained using model C2. The difference ranges from 5% to up to 50% for the bottom floor exterior column in load combination #2 (Fig. 5.15(a)). This underestimation of moments is reflected by slightly smaller exterior column sections using design models B2 than those using design model C2. Figs. 5.13(b), to 5.15(b) show that design moments in interior columns from model B2 are smaller than those from model C2 by less than 20% except for load combination #2 where moments are mostly less than minimum moments. Figs. 5.13(c) to 5.15(c) show that moments in beams from model B2 compare well with those from model C2 with less than 5% difference.

However, the impact of those differences in design moments on the final design, in terms of material quantities and cost, was minimal (Table 5.8). Concrete dimensions from models B2 or C2 differed, if any, by one dimension increment. Moreover, the cost indices differed by less than 1% except for 3B12S.W2.C2 where the cost index was less than that for 3B12S.W2.B2 by 2%.

For test frame 3B4S, Figs. 5.16(a) to 5.18(a) show that design moments in exterior columns from model B2 are lower than those from model C2 by as much as 30%. Again, this resulted in sometimes a difference of one dimension increment in concrete dimensions. Figs. 5.16(b) to 5.18(b) show that design moments in interior columns from model B2 are mostly smaller than those from model C2 by as much as 15%. However, all interior column cross sections from different designs were identical except for 3B4S.W2.B2 where the column depth was 50 mm smaller than that in the C2 design (Table 5.6). Figs. 5.16(c) to 5.18(c) show that design moments in beams from models B2 and C2 are different by less than 5%.

Again, those differences in design moments were reflected by differences in cost indices of 1.5% or less (Table 5.8).

For test frame 3B5S, Figs. 5.19(a) to 5.21(a) show that design moments in exterior columns from model B2 are lower than those from model C2 by as much as 35%. This caused, in some cases, a difference of one increment in exterior column dimensions. The same can be said about interior columns (Figs. 5.19(b) to 5.21(b)). Design moments in beams from model B2 are lower than those from model C2 by less than 10% (Figs. 5.19(c), to 5.21(c)). However, the same beam cross section is chosen in each case. Cost indices had a difference of less than 1% (Table 5.8).

This verifies the previous conclusion that modelling member effective stiffnesses to have a value of 0.5 times their gross stiffnesses results in less than 2% differences in total cost indices in spite of significant differences in design moments.

5.5 Effect of selected construction rules

Some constructibility rules are believed to play a significant role in the final design of a structure. Other rules do not seem to make much difference. To examine this, the frame design 3B12S.W1.B2 is repeated changing two parameters that were believed to be significant. The first parameter is the minimum number of storeys using the same column dimensions in a column stack, MINS, where it was changed from the default value of 4 to the new value of 2. The second parameter is the concrete dimension increment, INCR, where it was changed from the default value of 50 mm to 25 mm.

Table 5.12 shows the material quantities and cost indices for frame design 3B12S.W1.B2 using three combinations of MINS and INCR. The first design has the default values for MINS and INCR (4 and 50 mm respectively). The second design has a value of 2 for MINS and the default value for INCR. The third design has the default value for MINS and a value of 25 mm for INCR. From the numbers in this table we can see that reducing the dimension increment INCR had more significant effect on the final design (10% less concrete volume and 4% less cost index) than reducing the parameter MINS (3% less concrete volume and 1% less cost index). Moreover, the effect of reducing the dimension increment on the cost index is more than four times the effect of modelling the joint size and two times the effect of modelling member stiffnesses.

5.6 Load combinations

Load combinations are specified by design codes to define the range of unfavourable effects to which a structure may be subject. The two load combinations implemented in ESURF (load combinations #1 and #2) were defined in section 5.2.

While both load combinations #1 and #2 must be considered in proportioning members, it is desirable for the designer to have a feeling of which load combination is likely to govern the design of a particular group of members. It is obvious that this will depend on the magnitude of the factored dead, live and wind loads. However, in this study, only the load values specified in section 5.2 were used to design the three test frames with the design model B2 for both wind intensities (W1 and W2). For each test frame, stress resultants for each member were obtained for both load combinations (#1 and #2). The results were compared and the governing load combination is indicated in Table 5.13.

For test frame 3B12S, we can see that interior columns design is governed by load combination #2. This agrees with engineering logic since interior columns design is mostly governed by the magnitude of their axial load which is usually higher in load combination #2. However, columns in upper storeys were governed by minimum reinforcement requirement. On the other hand, load combination #1 gives larger moments and slightly smaller axial forces in exterior columns and hence governs their design. Beam dimensions in lower storeys were governed by load combination #1 where negative moments were larger. For beams in upper storeys, load combination #2 gave larger moments. Positive reinforcements in beams were always governed by load combination #2.

For test frame 3B4S, the design of interior columns was mostly governed by load combination #2 except for W2 case where load combination #1 governed in the lower two storeys. Design of exterior columns in the bottom storey was governed by load combination #1 while columns in upper storeys were governed by load combination #2. Design of beams was always governed by load combination #2.

For test frame 3B5S, the design of interior columns in lower storeys was governed by load combination #1 while columns in upper storeys were governed by load combination #2. Exterior columns followed almost the same pattern. The design of beams was always governed by load combination #2.

For the frames and load values considered in this study, design of exterior columns in high-rise frames was generally governed by the gravity and lateral load combination. However, for low-rise frames, exterior columns in upper storeys were governed by the gravity load combination. Design of interior columns in high-rise frames was governed by the gravity load combination. For low-rise frames, interior columns design in lower storeys was governed by the gravity and lateral load combination whereas columns in upper storeys were governed by the gravity load combination. Design of beams was generally governed by the gravity load combination for low-rise frames while it was governed by the gravity and lateral load combination, except for upper storeys beams, for high-rise frames. Positive reinforcement in beams was always determined by the gravity load combination. In all of these designs, member cross sections based on stress resultants from the gravity and lateral load combination were adequate for stress resultants from the gravity load combination, however, in some cases, the reinforcement had to be increased.

5.7 Live load reduction factor (LLRF)

The National Building Code of Canada allows reducing the specified live load due to use and occupancy on structural members supporting a tributary area greater than 20 m^2 by:

$$0.3 + \sqrt{9.8/B}$$

where B is the tributary area in square meters. This formula is suitable for individual members rather than building frames. Each member in a frame will have, as a general case, a different reduction factor. In all previous studies, the effect of LLRF was not considered in order not to mask the effect of the parameters being studied.

Since LLRF effect is more significant in column design than in beam design, this effect was considered in ESURF by reducing each column load before proportioning according to the above formula. The intention here was to assess the effect of LLRF in columns on the final design.

The test frames 3B12S, 3B4S, and 3B5S were redesigned using design model B2 with LLRF in columns being considered. Table 5.14 shows the concrete dimensions for each test frame design. Table 5.15 shows the material quantities and cost indices for these designs. As expected, the effect of LLRF on the final design of test frame 3B12S was the greatest since it has columns with large tributary areas. Concrete volume has decreased by 5% and the cost index by 2.5%. For test frames 3B4S and 3B5S, saving in cost index was in the range of 1%.

<table>
<tr><th rowspan="2">Wind</th><th rowspan="2">Storey</th><th colspan="2">Exterior Column $c_1 \times c_2$ mm</th><th colspan="2">Interior Column $c_1 \times c_2$ mm</th><th colspan="2">Beam b x h mm</th></tr>
<tr><th>A1</th><th>B1</th><th>A1</th><th>B1</th><th>A1</th><th>B1</th></tr>
<tr><td rowspan="12">W1</td><td>1</td><td rowspan="4">500x350</td><td rowspan="4">500x350</td><td rowspan="4">600x500</td><td rowspan="4">600x500</td><td rowspan="12">400x700</td><td rowspan="12">400x700</td></tr>
<tr><td>2</td></tr>
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<tr><td>4</td></tr>
<tr><td>5</td><td rowspan="4">450x350</td><td rowspan="4">450x350</td><td rowspan="4">550x450</td><td rowspan="4">550x450</td></tr>
<tr><td>6</td></tr>
<tr><td>7</td></tr>
<tr><td>8</td></tr>
<tr><td>9</td><td rowspan="4">400x350</td><td rowspan="4">400x350</td><td rowspan="4">450x350</td><td rowspan="4">450x350</td></tr>
<tr><td>10</td></tr>
<tr><td>11</td></tr>
<tr><td>12</td></tr>
<tr><td rowspan="12">W2</td><td>1</td><td rowspan="5">450x350</td><td rowspan="5">450x350</td><td rowspan="4">600x500</td><td rowspan="4">600x500</td><td rowspan="12">400x700</td><td rowspan="12">400x700</td></tr>
<tr><td>2</td></tr>
<tr><td>3</td></tr>
<tr><td>4</td></tr>
<tr><td>5</td><td rowspan="4">500x450</td><td rowspan="4">500x450</td></tr>
<tr><td>6</td><td rowspan="7">400x350</td><td rowspan="7">400x350</td></tr>
<tr><td>7</td></tr>
<tr><td>8</td></tr>
<tr><td>9</td><td rowspan="4">400x350</td><td rowspan="4">400x350</td></tr>
<tr><td>10</td></tr>
<tr><td>11</td></tr>
<tr><td>12</td></tr>
</table>

Table 5.1 Concrete dimensions of frame 3B12S for models A1 and B1

Wind	Storey	Exterior Column $c_1 \times c_2$ mm		Interior Column $c_1 \times c_2$ mm		Beam b x h mm	
		A1	B1	A1	B1	A1	B1
W1	1 2 3 4	450x250	450x250	400x300	400x300	350x650	350x650
W2	1 2 3 4	450x250	450x250	400x300	400x300	350x650	350x650

Table 5.2 Concrete dimensions of frame 3B4S for models A1 and B1

Wind	Storey	Exterior Column $c_1 \times c_2$		Interior Column $c_1 \times c_2$		Beam b x h	
		A1	B1	A1	B1	A1	B1
W1	1 2 3 4 5	400x250	450x250	450x250	450x250	400x650	400x650
W2	1 2 3 4 5	450x250	450x250	350x300	400x300	400x650	400x650

Table 5.3 Concrete dimensions of frame 3B5S for models A1 and B1

		3B12S.W1	3B12S.W2	3B4S.W1	3B4S.W2	3B5S.W1	3B5S.W2
Concrete Volume cu. m	A1	134.7	132.7	34.2	34.2	41.7	42.9
	B1	134.7	132.7	34.2	34.2	41.9	43.4
Steel Mass kg	A1	14,901	17,232	4,360	4,649	9,489	8,915
	B1	14,646	17,212	4,519	4,680	8,852	8,939
Forms sq. m	A1	1,102	1,092	327	327	381	387
	B1	1,102	1,092	327	327	384	390
Cost Index	A1	462.1	471.8	132.5	134.2	181.6	180.5
	B1	460.6	471.7	133.4	134.4	178.6	181.8

Table 5.4 Material quantities and cost indices for models A1 and B1

Wind	Storey	Exterior Column $c_1 \times c_2$ mm			Interior Column $c_1 \times c_2$ mm			Beam b x h mm		
		A2	B2	C2	A2	B2	C2	A2	B2	C2
W1	1									
	2	400x350	400x350	450x350	500x500	500x500	500x500			
	3									
	4	—	—	—	—	—	—			
	5							400x650	400x650	400x650
	6	350x350	350x350	400x350	450x450	450x450	450x450			
	7									
	8	—	—	—	—	—	—			
	9									
	10	300x350	300x350	350x350	350x350	350x350	350x350			
	11									
	12									
W2	1									
	2	450x350	400x350	450x350	600x500	550x550	500x500			
	3									
	4				—	—	—			
	5	—	—	—						
	6				500x450	450x450	450x450	400x700	400x700	400x650
	7									
	8				—	—	—			
	9	400x350	350x350	400x350						
	10				400x350	350x350	350x350			
	11									
	12									

Table 5.5 Concrete dimensions of frame 3B12S for models A2, B2, and C2

Wind	Storey	Exterior Column $c_1 x c_2$ mm			Interior Column $c_1 x c_2$ mm			Beam b x h mm		
		A2	B2	C2	A2	B2	C2	A2	B2	C2
W1	1 2 3 4	350x250	350x250	350x250	300x300	300x300	300x300	350x650	350x600	350x600
W2	1 2 3 4	400x250	350x250	350x250	350x300	300x300	350x300	400x600	400x600	400x600

Table 5.6 Concrete dimensions of frame 3B4S for models A2, B2, and C2

Wind	Storey	Exterior Column $c_1 x c_2$ mm			Interior Column $c_1 x c_2$ mm			Beam b x h mm		
		A2	B2	C2	A2	B2	C2	A2	B2	C2
W1	1 2 3 4 5	400x250	450x250	450x250	450x250	450x250	450x250	400x650	400x650	400x650
W2	1 2 3 4 5	450x250	400x250	450x250	350x300	350x300	350x300	400x650	400x650	400x650

Table 5.7 Concrete dimensions of frame 3B5S for models A2, B2, and C2

		3B12S.W1	3B12S.W2	3B4S.W1	3B4S.W2	3B5S.W1	3B5S.W2
Concrete Volume cu. m	A2	121.0	132.7	30.5	34.8	41.7	42.9
	B2	121.0	130.27	30.50	34.0	41.9	42.4
	C2	122.6	123.2	30.5	34.5	41.9	42.9
Steel Mass kg	A2	16,970	16,121	5,151	4,920	9,013	8,751
	B2	16,391	17,110	5,269	5,299	9,026	9,191
	C2	16,246	17,622	5,427	5,471	9,218	9,078
Forms sq. m	A2	1,035	1,091	304	321	381	387
	B2	1,035	1,079	304	318	384	383
	C2	1,044	1,047	304	318	384	389
Cost Index	A2	447.5	465.3	128.6	134.8	178.8	179.5
	B2	444.1	466.3	129.3	135.0	179.7	180.8
	C2	446.6	456.0	130.2	137.0	180.8	182.0

Table 5.8 Material quantities and cost indices for models A2, B2, and C2

Frame	Exterior Column			Interior Column			Beam			Quantities		
	c_1 x c_2 mm	M_I	M_{II}	c_1 x c_2 mm	M_I	M_{II}	b x h mm	M_I	M_{II}	Conc. m^3	Steel kg	Index
W1C2	450x350	1.00	1.00	500x500	1.00	1.00	400x650	1.00	1.00	122.6	16,246	446.6
C5C3	400x350	0.70	0.72	500x500	1.17	1.26	400x650	1.08	1.01	121.0	15,956	441.6
C5C5	400x350	0.84	0.86	500x500	1.13	1.07	400x650	1.01	0.97	121.0	16,391	444.1
C5C8	450x350	0.90	1.09	500x500	1.12	0.57	400x600	0.95	0.90	115.7	17,105	437.8
C5C10	450x350	1.01	1.12	500x500	1.14	0.51	400x600	0.89	0.89	115.7	17,348	439.2
C4C5	400x350	0.89	0.92	500x500	1.19	1.00	400x650	1.00	0.96	121.0	16,706	446.0
C4C8	450x300	1.03	1.12	500x500	1.20	0.51	400x600	0.91	0.89	115.7	17,298	438.9

Table 5.9 Frame design 3B12S.W1 for different sets of stiffness

Frame	Exterior Column			Interior Column			Beam			Quantities		
	$c_1 \times c_2$ mm	M_I	M_{II}	$c_1 \times c_2$ mm	M_I	M_{II}	b x h mm	M_I	M_{II}	Conc. m^3	Steel kg	Index
W1C2	350x250	1.00	1.00	300x300	1.00	1.00	350x600	1.00	1.00	30.5	5,427	130.2
C5C3	300x250	0.74	0.55	300x300	1.07	0.86	350x600	0.96	0.98	30.2	5,571	130.0
C5C5	350x250	0.89	0.91	300x300	0.91	0.71	350x600	0.93	0.93	30.5	5,269	129.3
C5C8	350x250	1.02	1.06	300x300	0.84	0.65	350x600	0.90	0.91	30.5	5,520	130.8
C5C10	350x250	1.07	1.12	300x300	0.81	0.61	350x600	0.89	0.90	30.5	5,210	129.0
C4C5	350x250	1.03	0.98	300x300	0.94	0.71	350x600	0.92	0.92	30.5	5,504	130.7
C4C8	350x250	1.07	1.12	300x300	0.84	0.63	350x600	0.89	0.90	30.5	5,341	129.7

Table 5.10 Frame design 3B4S.W1 for different sets of stiffness

Frame	Exterior Column			Interior Column			Beam			Quantities		
	$c_1 \times c_2$ mm	M_I	M_{II}	$c_1 \times c_2$ mm	M_I	M_{II}	b x h mm	M_I	M_{II}	Conc. m^3	Steel kg	Index
W1C2	450x250	1.00	1.00	450x250	1.00	1.00	400x650	1.00	1.00	41.9	9,218	180.8
C5C3	400x250	0.82	0.82	350x300	0.56	0.53	350x650	0.85	0.86	37.5	8,853	170.9
C5C5	450x250	0.87	1.00	450x250	0.87	0.87	400x650	0.91	0.91	41.9	9,026	179.7
C5C8	450x250	0.91	1.05	500x250	1.04	1.07	400x650	0.95	0.96	42.3	9,148	181.5
C5C10	450x250	0.93	1.06	500x300	1.07	1.14	400x650	0.96	0.98	43.4	9,133	183.3
C4C5	450x250	0.90	1.04	450x250	0.92	0.93	400x650	0.92	0.92	41.9	9,246	180.9
C4C8	450x250	0.92	1.07	450x250	1.10	1.14	400x650	0.97	0.98	41.9	9,170	183.5

Table 5.11 Frame design 3B5S.W1 for different sets of stiffness

MINS	4	2	4
INCR (mm)	50	50	25
Concrete Volume cu. m.	121.0	117.5	109.7
Steel Mass kg	16,391	16,930	16,973
Forms sq. m.	1,035	1,016	988
Cost Index	444.1	439.8	427.3

Table 5.12 Frame design 3B12S.W1.B2 for different construction rules

Frame	Storey	W1				W2			
		Columns		Beams		Columns		Beams	
		Ext.	Int.	-ve	+ve	Ext.	Int.	-ve	+ve
3B12S	1	1	2	1	2	1	2	1	2
	2	1	2	1	2	1	2	1	2
	3	1	2	1	2	1	2	1	2
	4	1	2*	1	2	1	2*	1	2
	5	1	2	1	2	1	2	1	2
	6	1	2	2	2	1	2	1	2
	7	1	2*	2	2	1	2*	1	2
	8	1	2*	2	2	1	2*	1	2
	9	1	2	2	2	1	2	2	2
	10	1	2*	2	2	1	2*	2	2
	11	1	2*	2	2	1	2*	2	2
	12	1	2*	2	2	1	2*	2	2
3B4S	1	1	2	2	2	1	1	2	2
	2	2	2	2	2	2	1	2	2
	3	2	2*	2	2	2	2*	2	2
	4	2	2*	2	2	2	2*	2	2
3B5S	1	1	1	2	2	1	1	2	2
	2	2	2	2	2	2	1	2	2
	3	2	2	2	2	2	1	2	2
	4	2	2	2	2	2	2	2	2
	5	2	2	2	2	2	2	2	2

* Minimum reinforcement governs

Table 5.13 Governing load combinations

Frame	Storey	Exterior Column $c_1 \times c_2$ mm		Interior Column $c_1 \times c_2$ mm		Beam b x h mm	
		LLRF	No LLRF	LLRF	No LLRF	LLRF	No LLRF
3B12S	1	400x300	400x350	450x450	500x500	400x650	400x650
	2						
	3						
	4						
	5		350x350	350x350	450x450		
	6						
	7	350x300					
	8						
	9		300x350	250x250	350x350		
	10						
	11						
	12						
3B4S	1	350x250	350x250	250x250	300x300	350x600	350x600
	2						
	3						
	4						
3B5S	1	450x250	450x250	450x250	450x250	400x650	400x650
	2						
	3						
	4						
	5						

Table 5.14 Concrete dimensions of test frames with and without LLRF

Frame		3B12S	3B4S	3B5S
Conc. vol. cu. m	LLRF	114.3	29.7	41.9
	No LLRF	121.0	30.5	41.9
Steel mass kg	LLRF	16,861	5,404	8,665
	No LLRF	16,391	5,269	9,026
Forms sq. m	LLRF	1,001	298	384
	No LLRF	1,035	304	384
Cost Index	LLRF	433.4	127.9	177.5
	No LLRF	444.1	129.3	179.9

Table 5.15 Material quantities and cost indices of test frames with and without LLRF

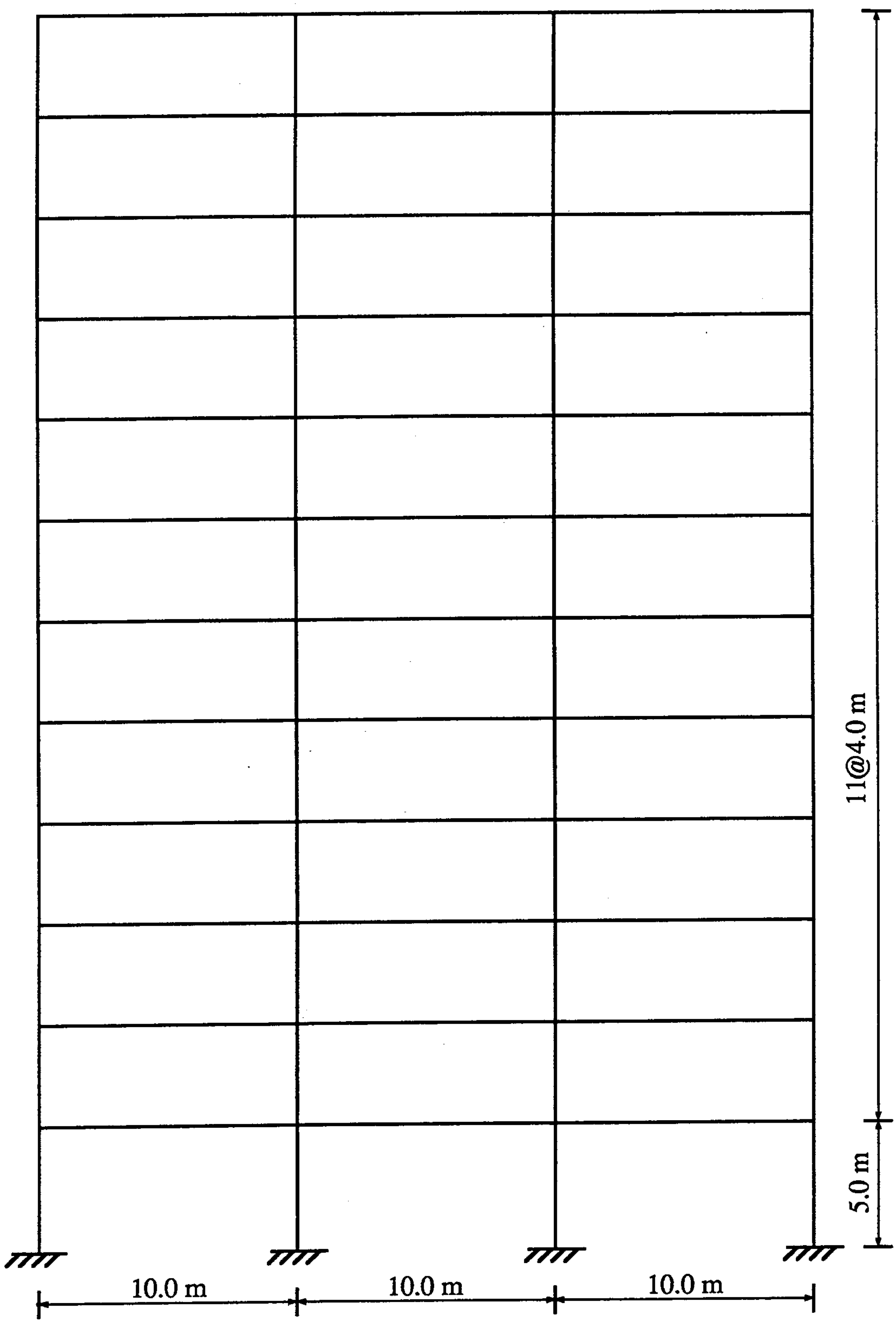

Fig. 5.1 Geometry of frame 3B12S

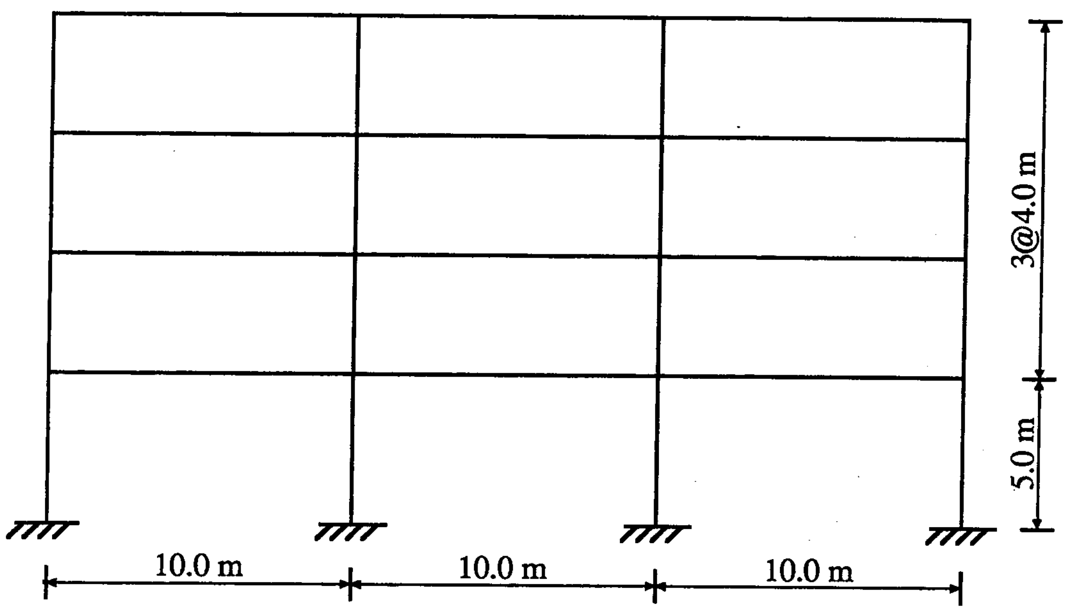

Fig. 5.2 Geometry of frame 3B4S

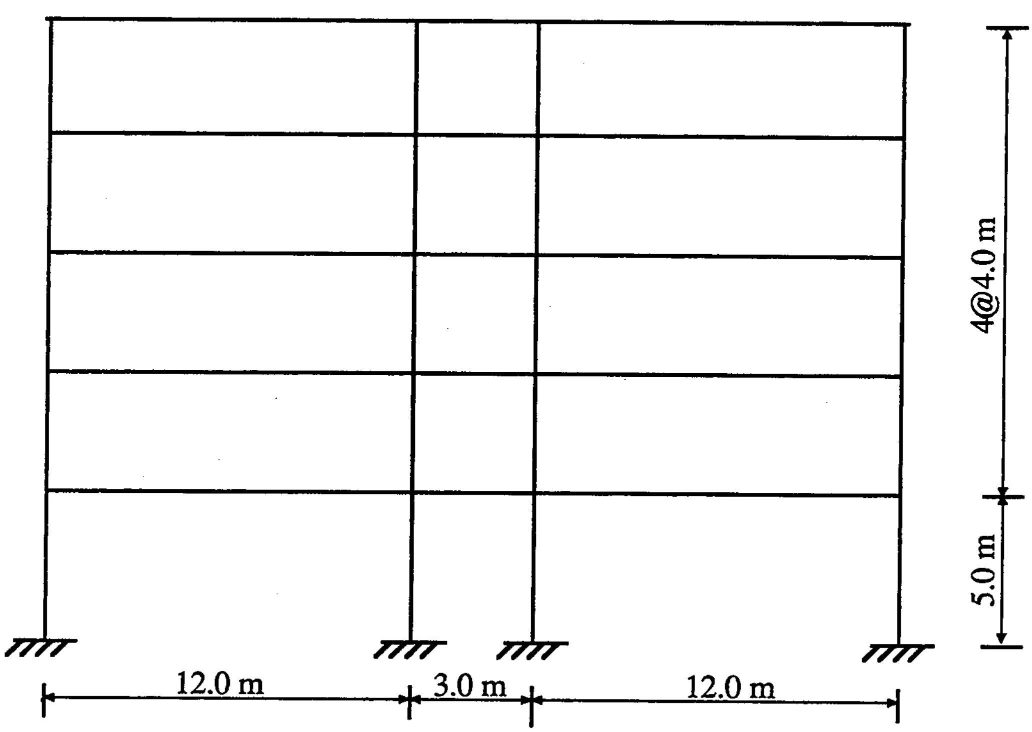

Fig. 5.3 Geometry of frame 3B5S

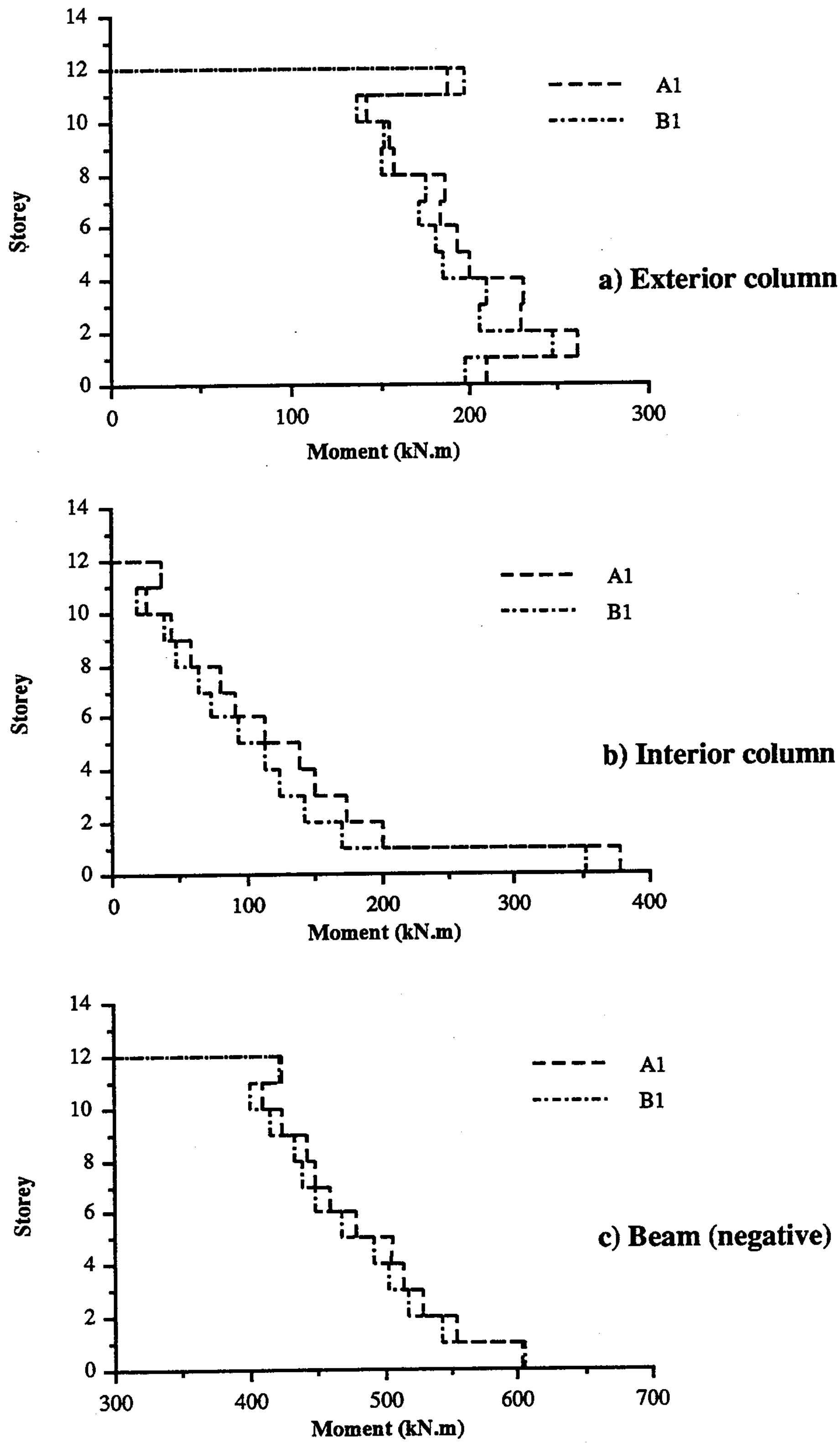

Fig. 5.4 Moments in frame 3B12S.W1 for models A1 and B1

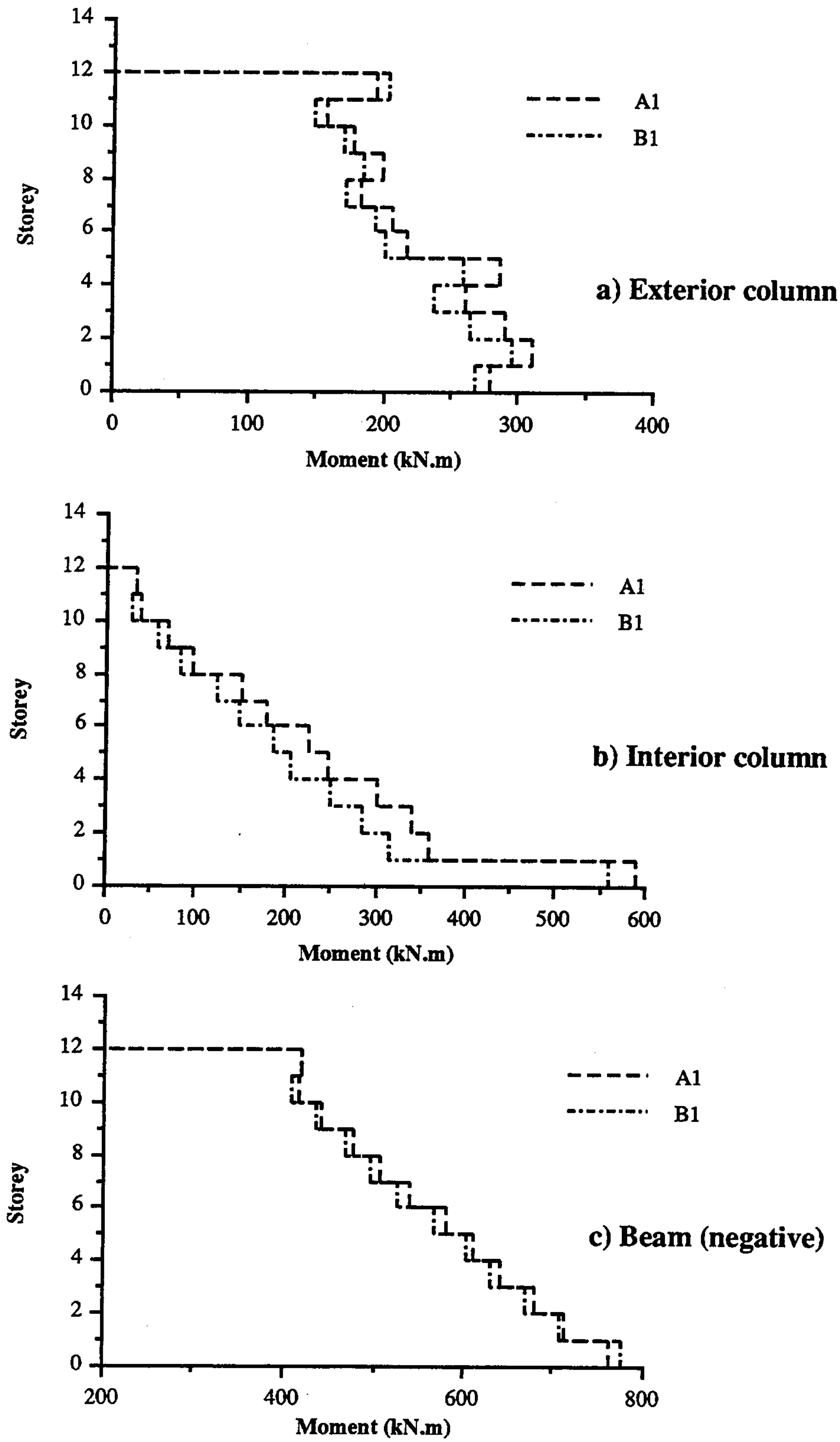

Fig. 5.5 Moments in frame 3B12S.W2 for models A1 and B1

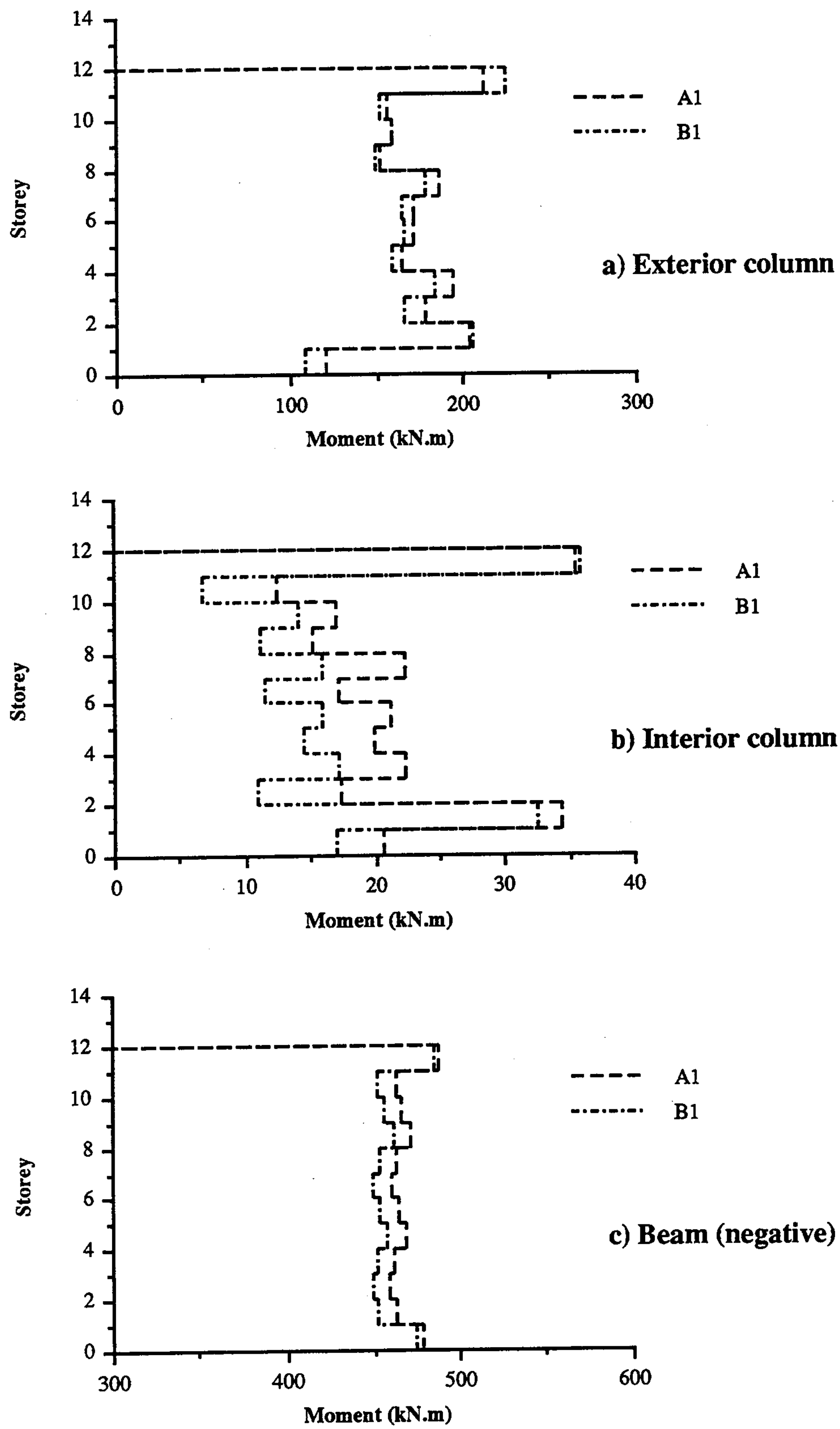

Fig. 5.6 Moments in frame 3B12S.W0 for models A1 and B1

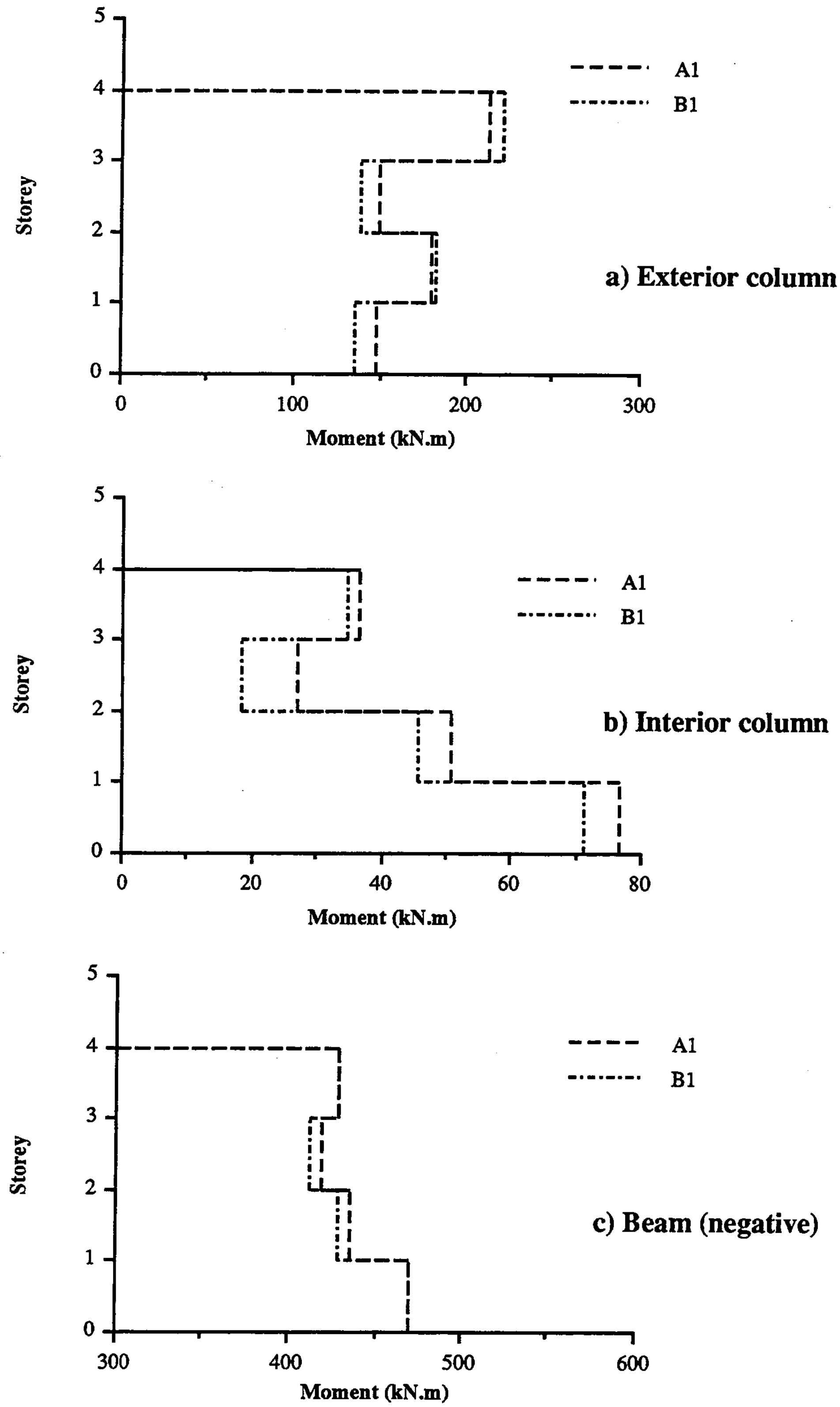

Fig. 5.7 Moments in frame 3B4S.W1 for models A1 and B1

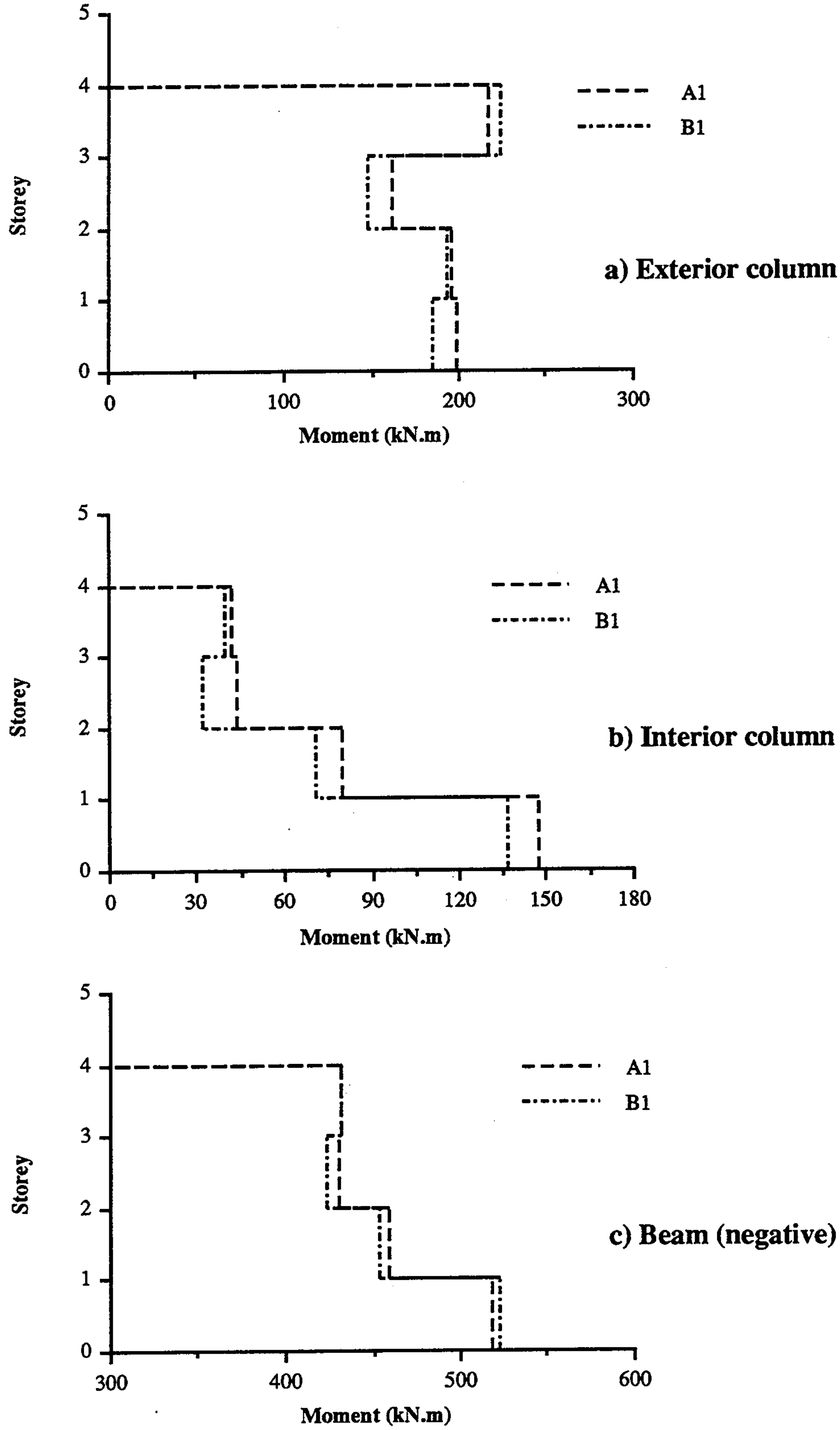

Fig. 5.8 Moments in frame 3B4S.W2 for models A1 and B1

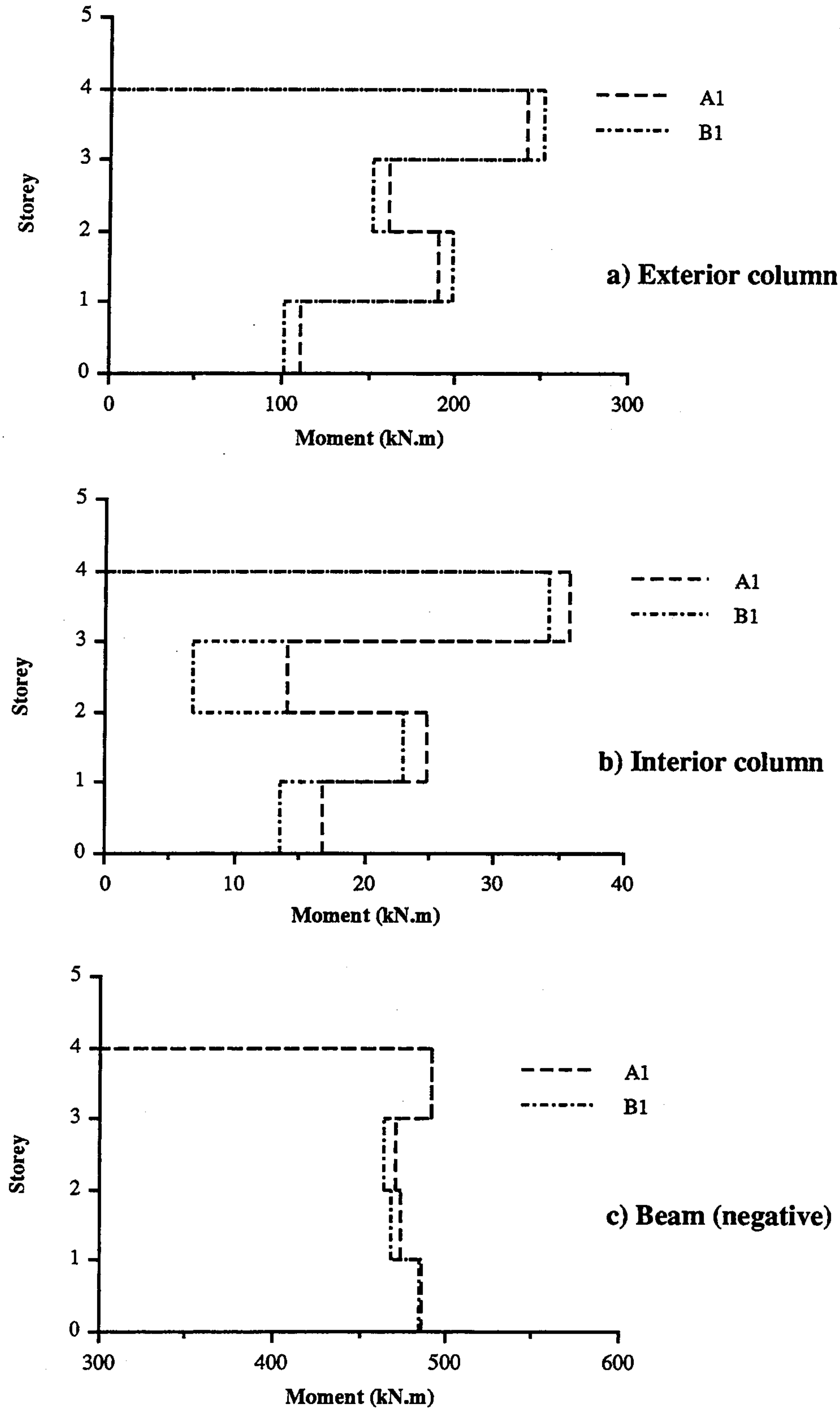

Fig. 5.9 Moments in frame 3B4S.W0 for models A1 and B1

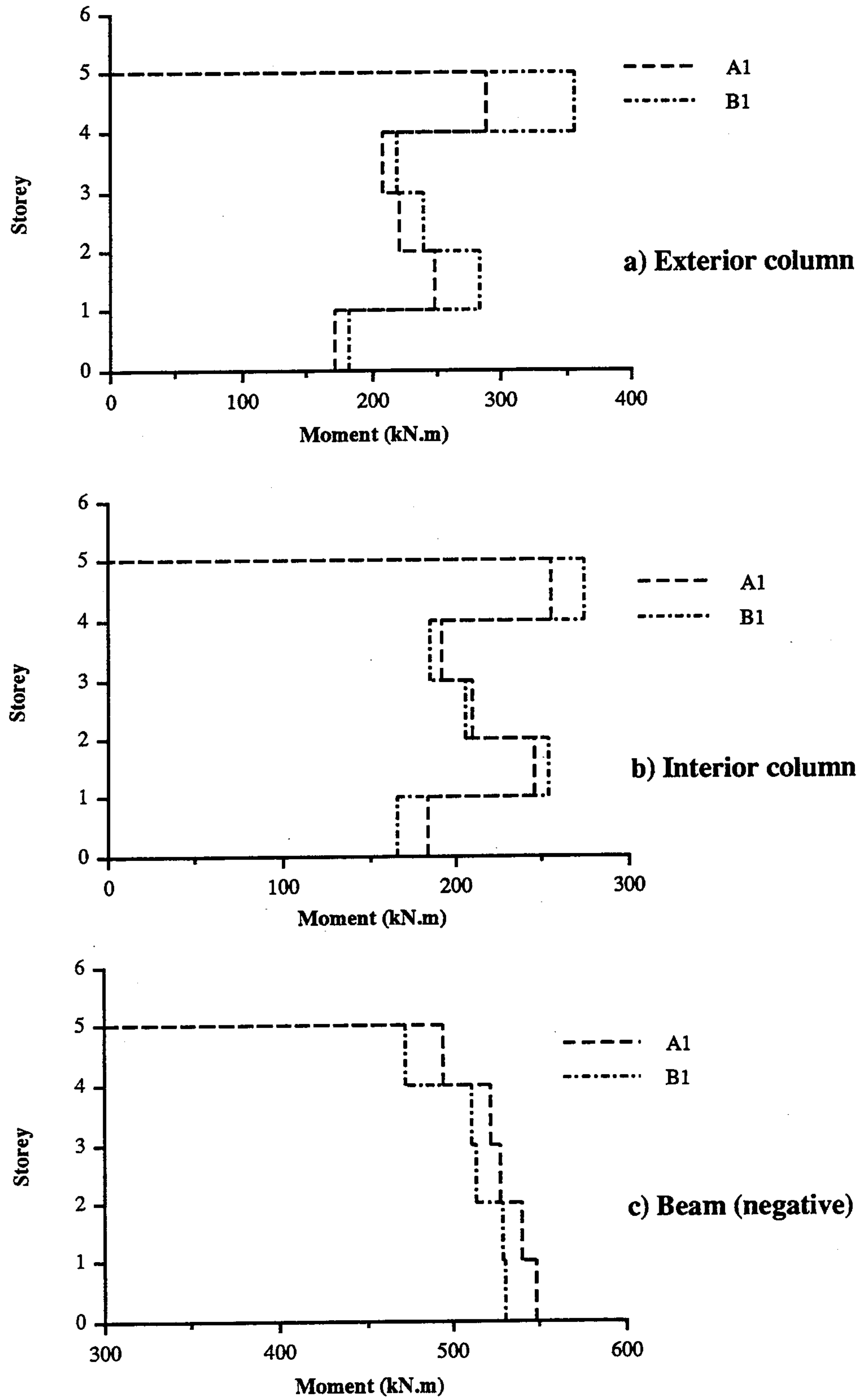

Fig. 5.10 Moments in frame 3B5S.W1 for models A1 and B1

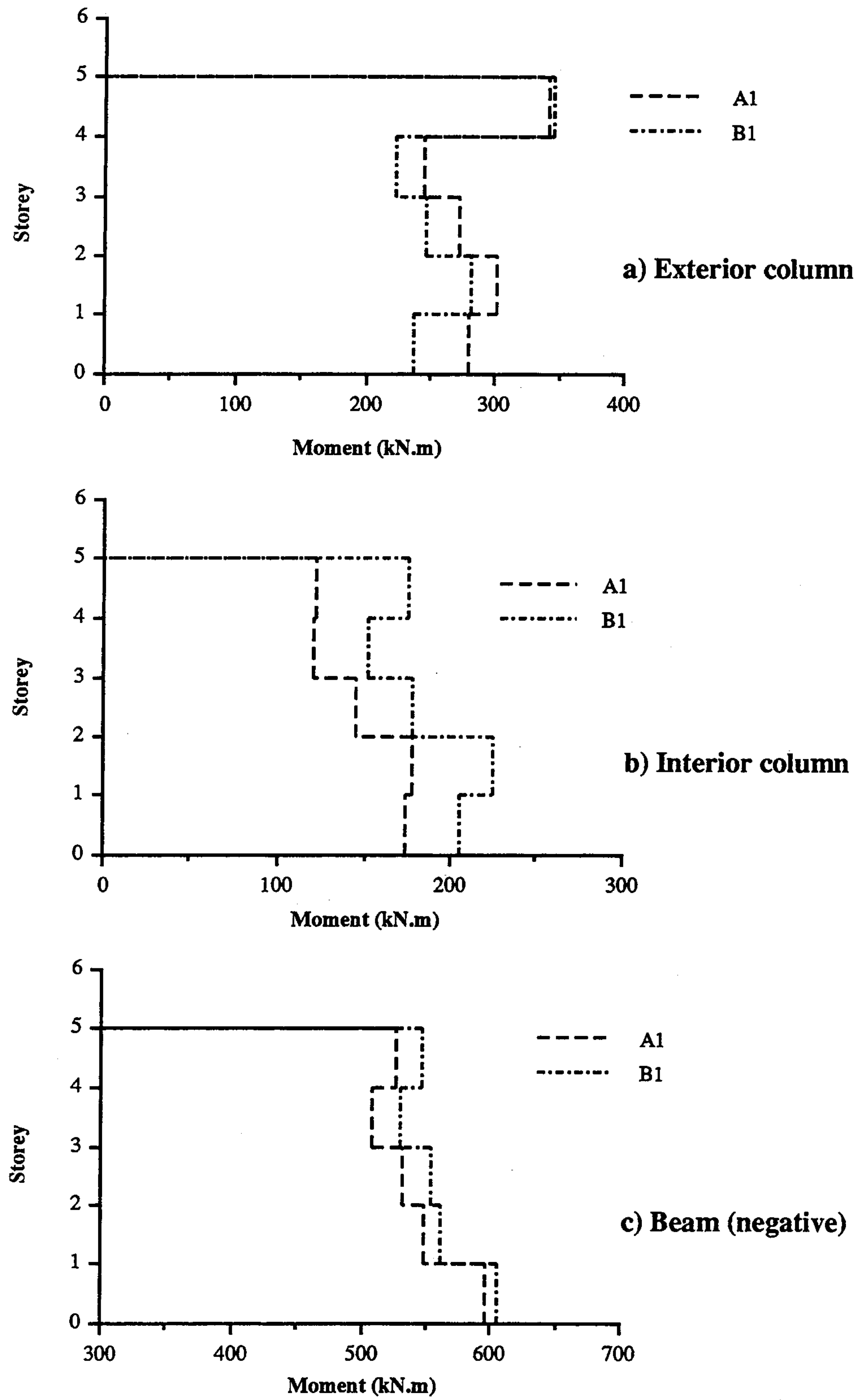

Fig. 5.11 Moments in frame 3B5S.W2 for models A1 and B1

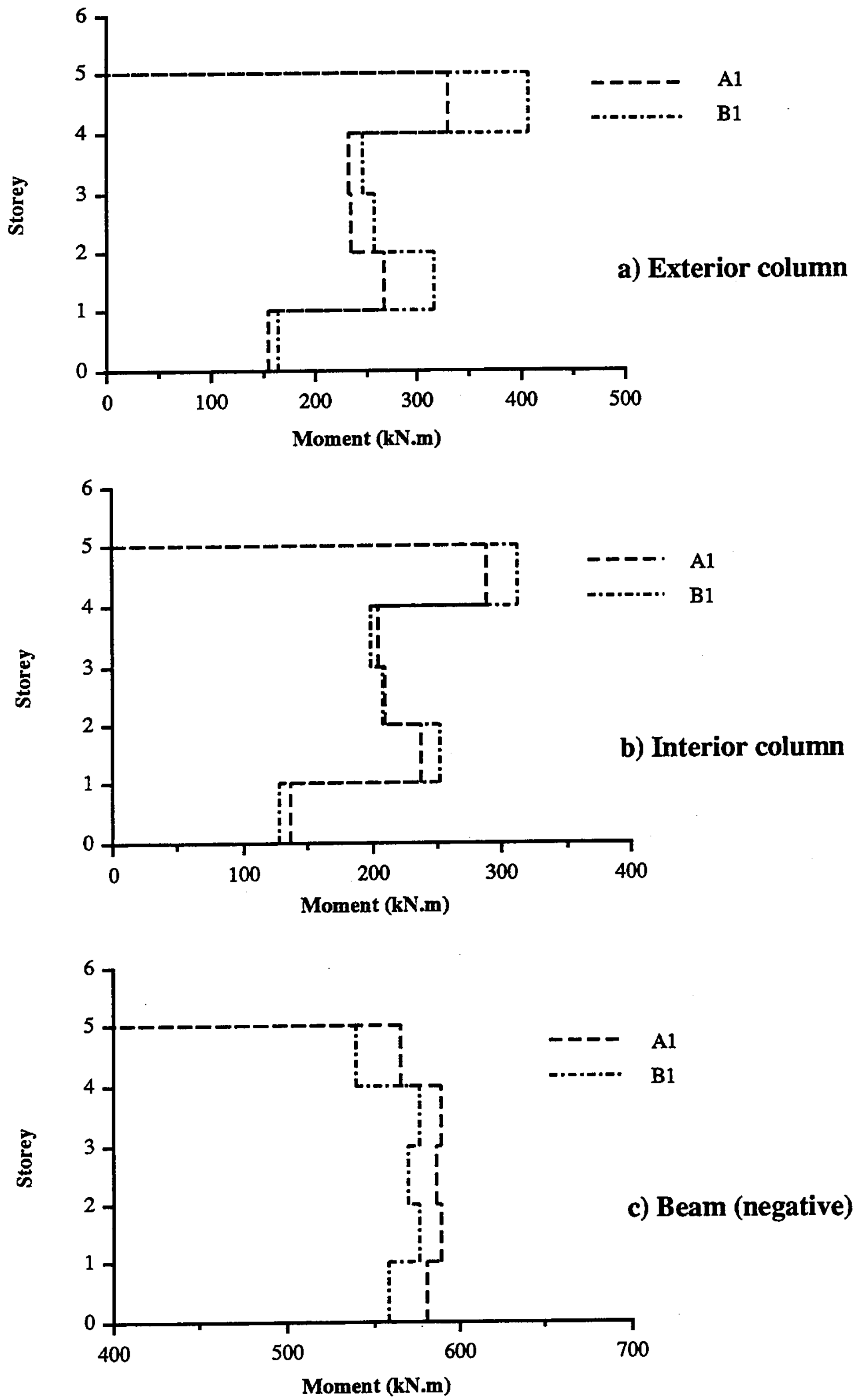

Fig. 5.12 Moments in frame 3B5S.W0 for models A1 and B1

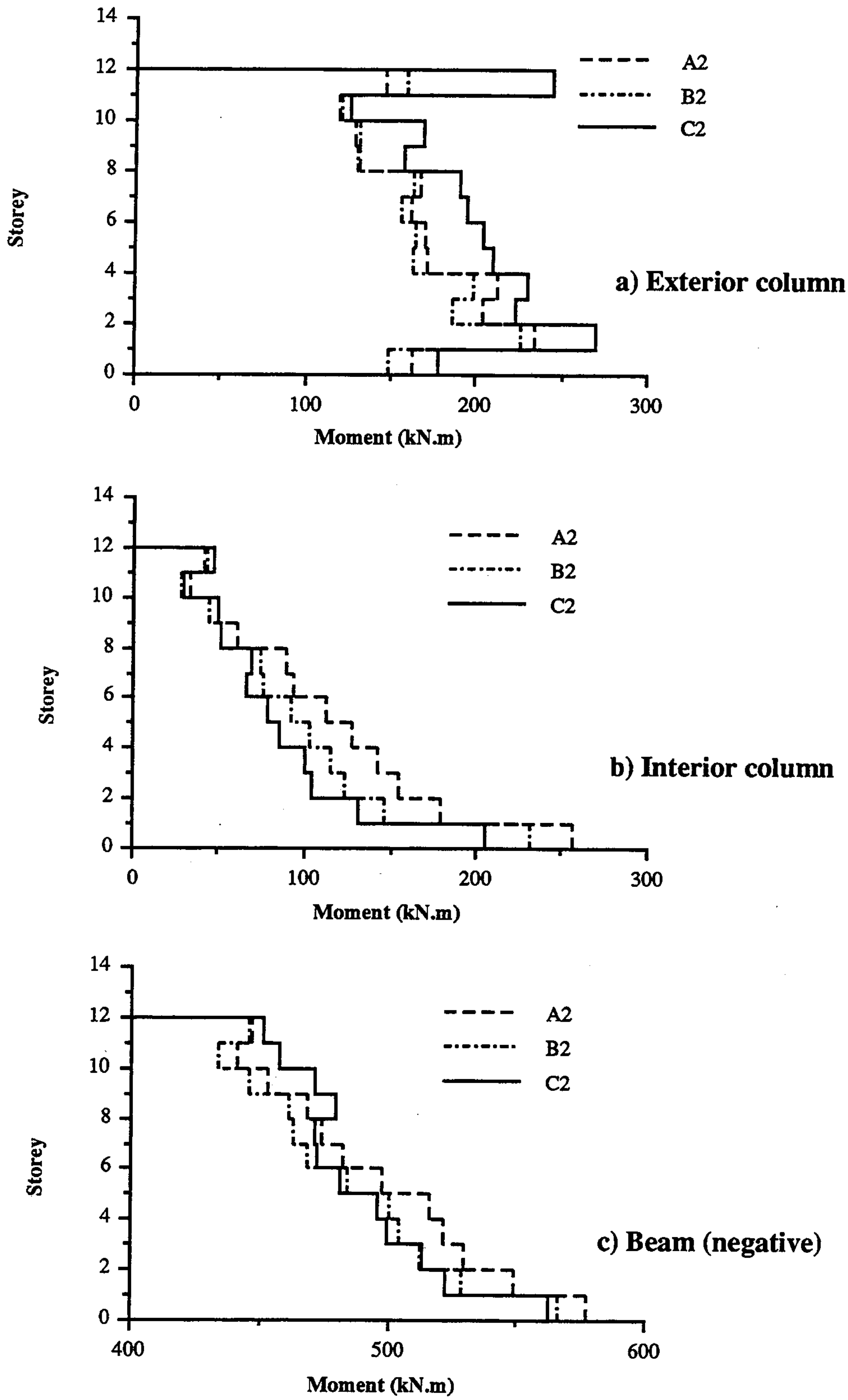

Fig. 5.13 Moments in frame 3B12S.W1 for models A2, B2, and C2

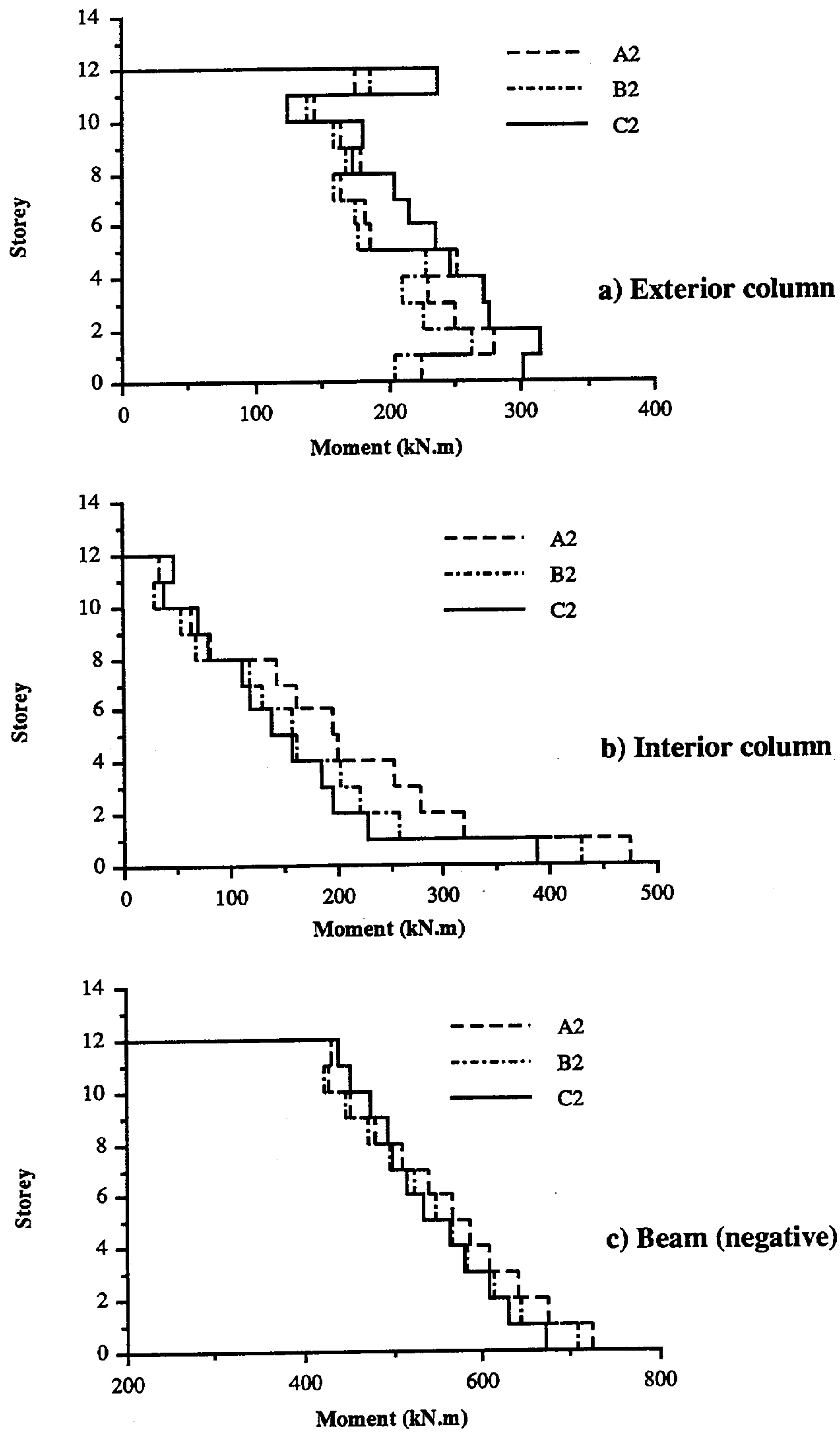

Fig. 5.14 Moments in frame 3B12S.W2 for models A2, B2, and C2

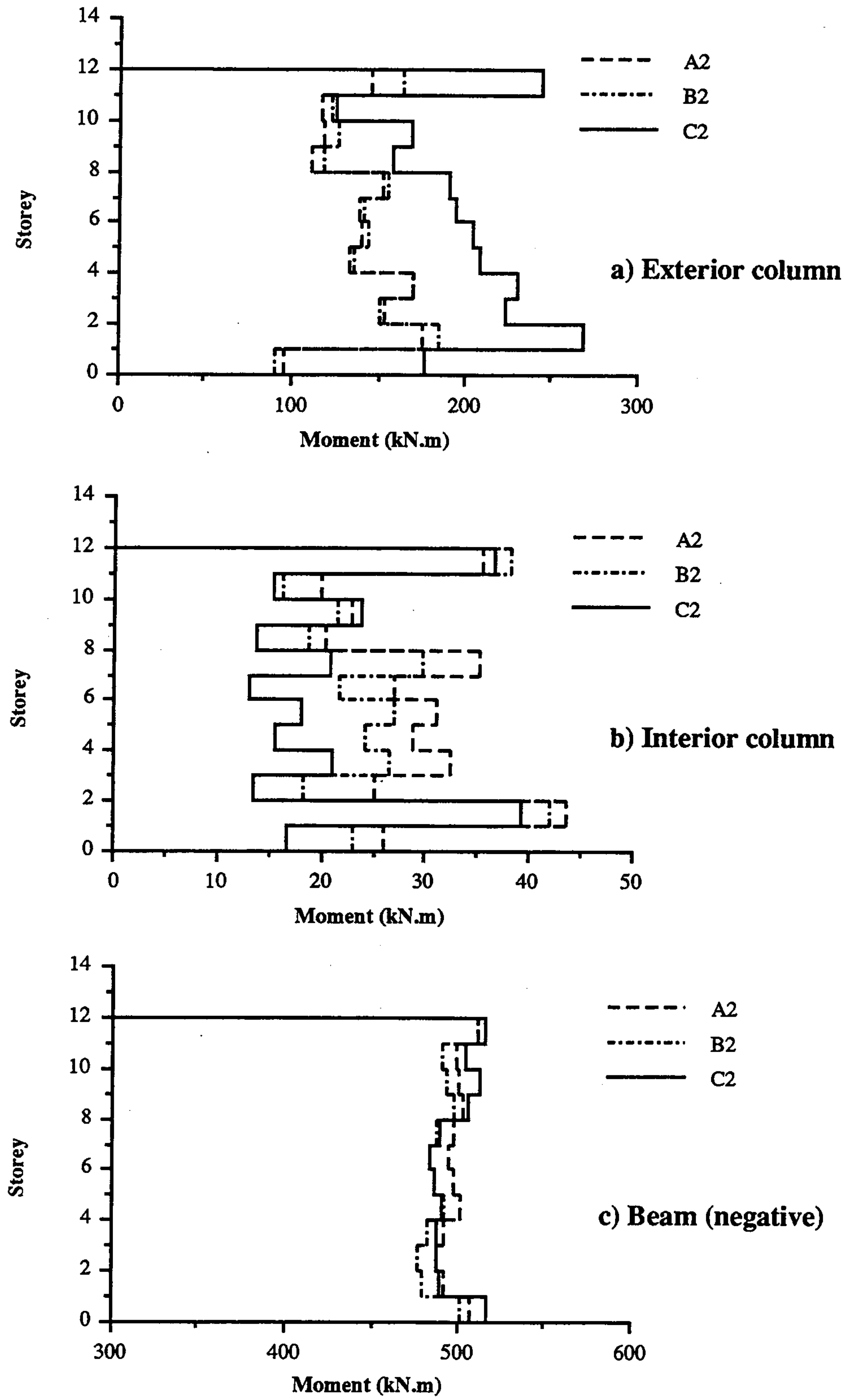

Fig. 5.15 Moments in frame 3B12S.W0 for models A2, B2, and C2

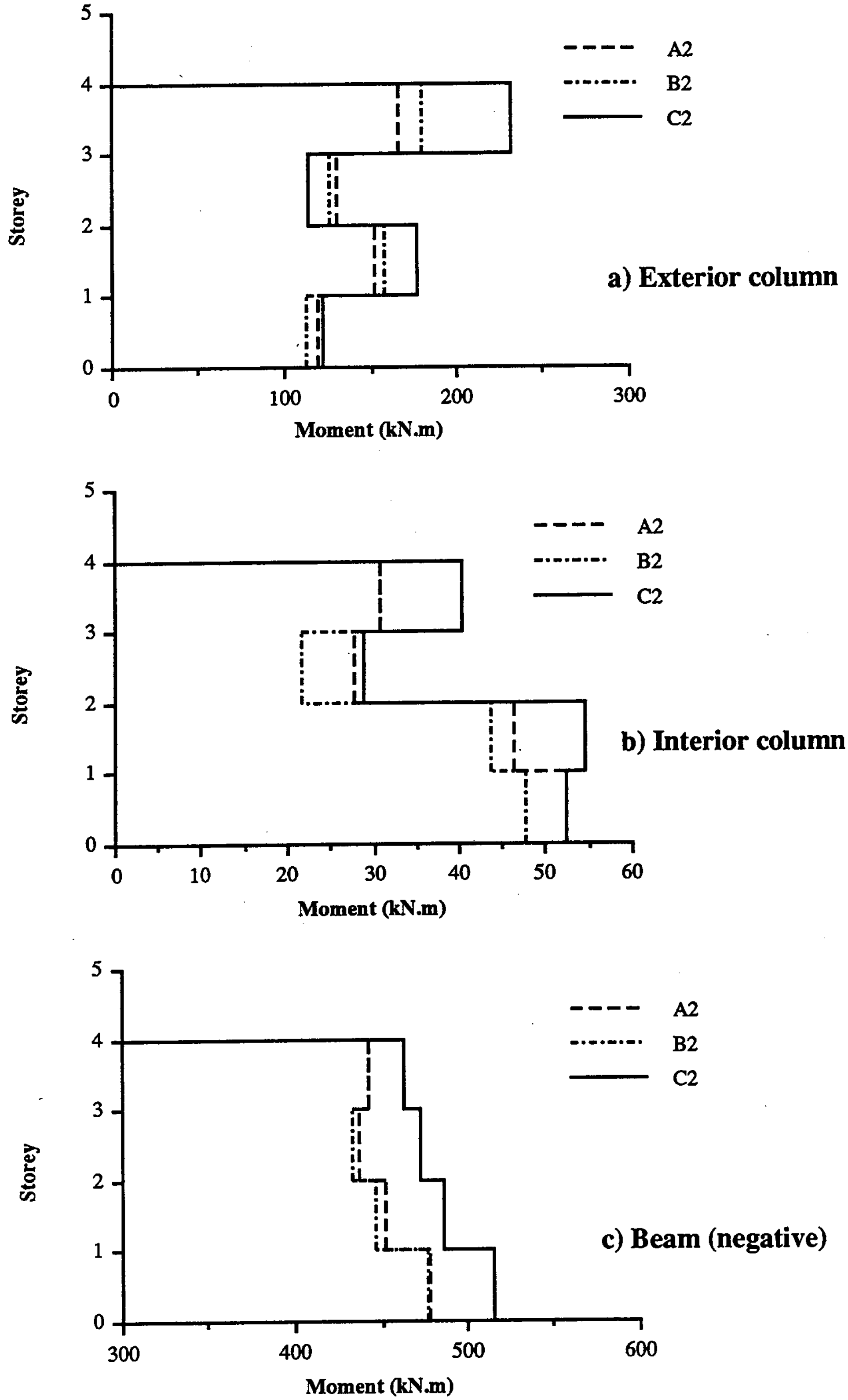

Fig. 5.16 Moments in frame 3B4S.W1 for models A2, B2, and C2

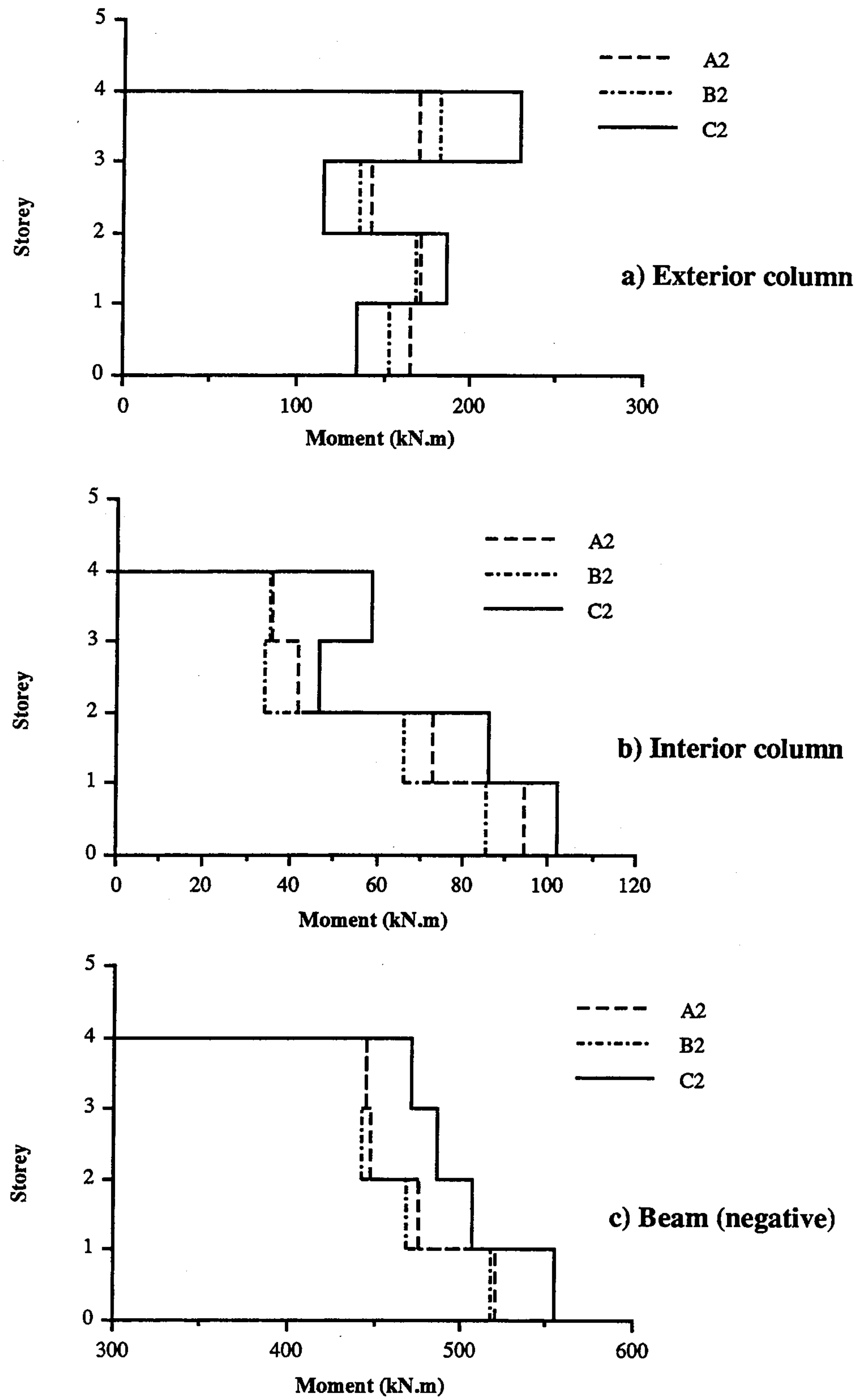

Fig. 5.17 Moments in frame 3B4S.W2 for models A2, B2, and C2

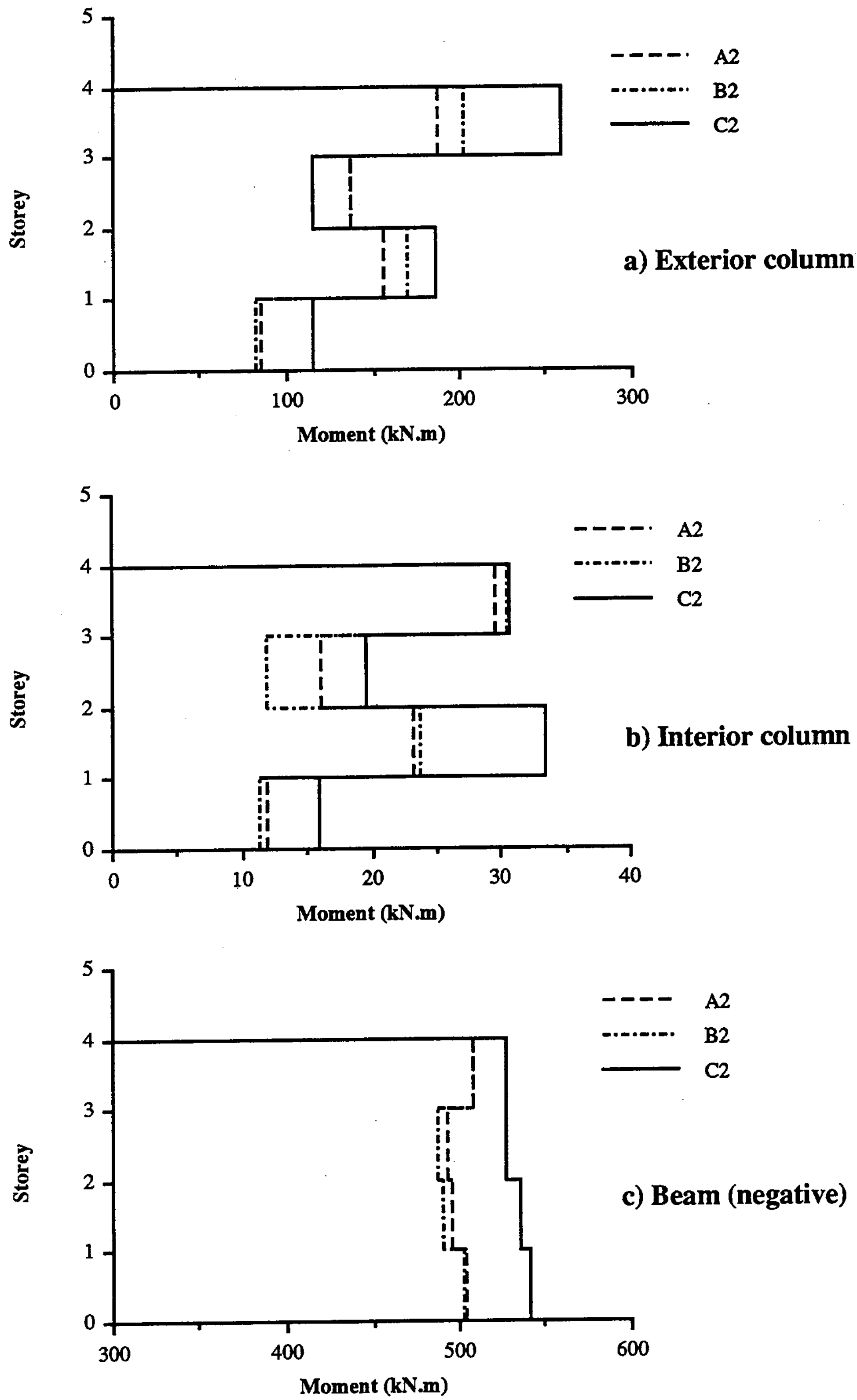

Fig. 5.18 Moments in frame 3B4S.W0 for models A2, B2, and C2

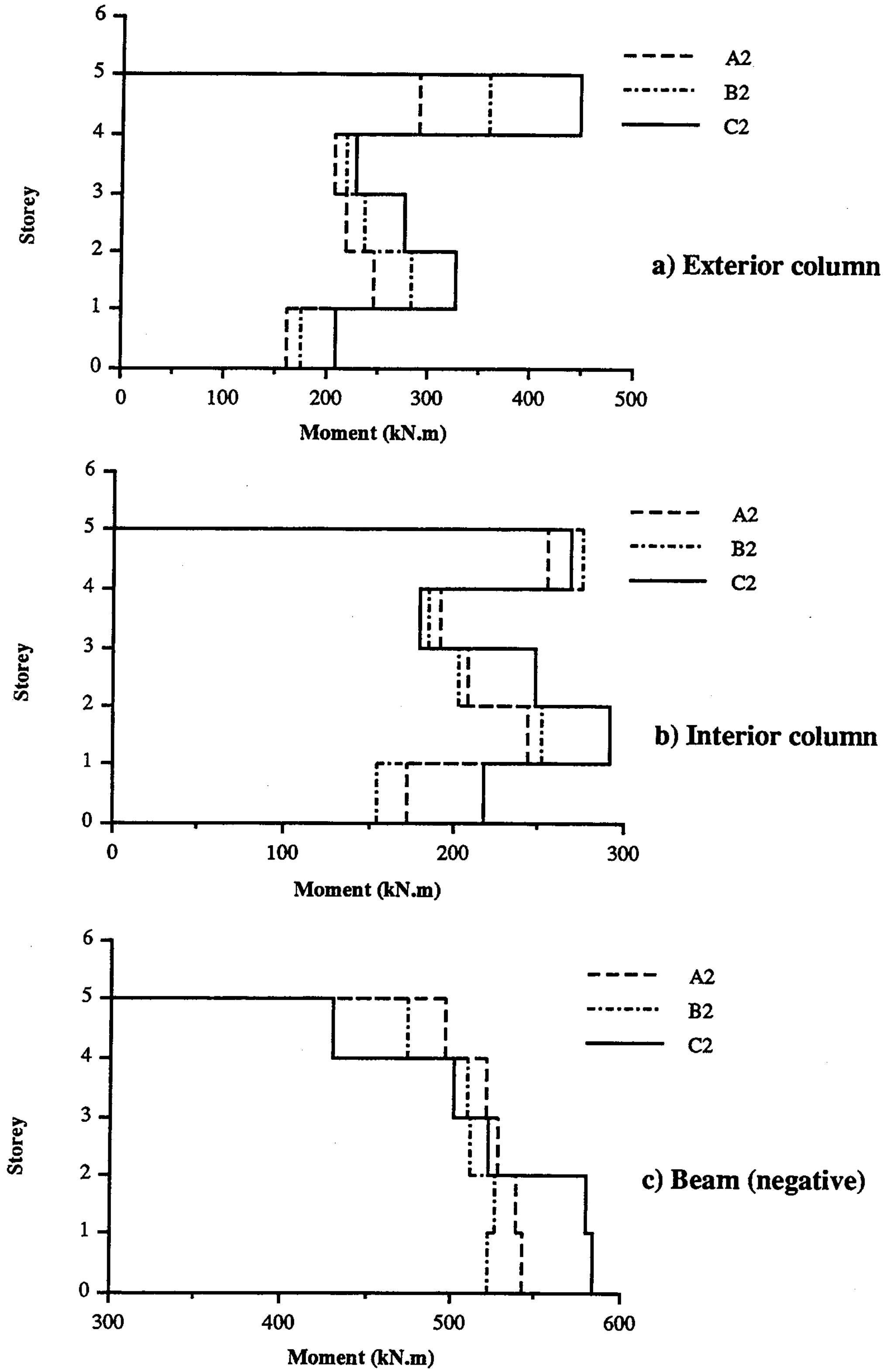

Fig. 5.19 Moments in frame 3B5S.W1 for models A2, B2, and C2

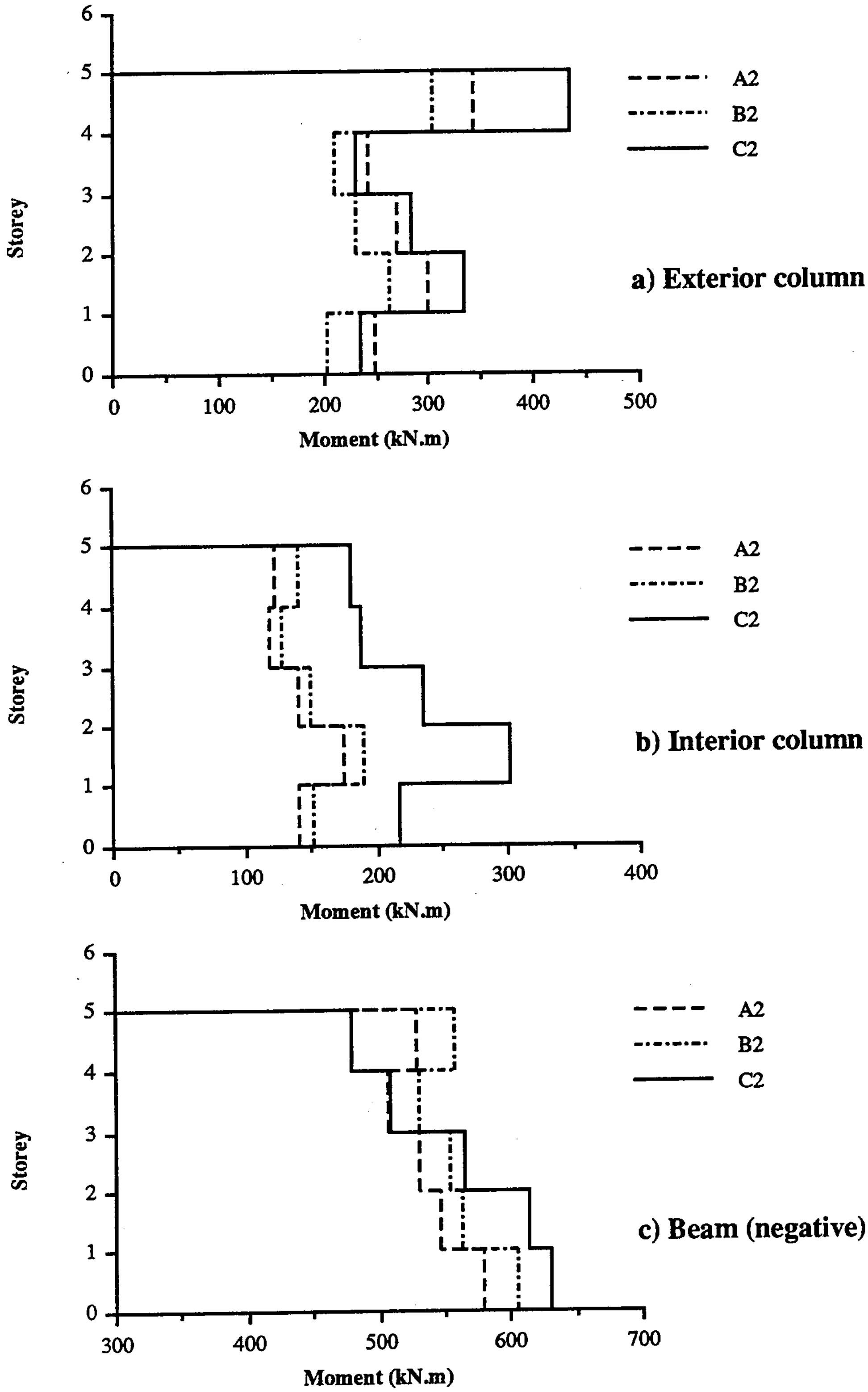

Fig. 5.20 Moments in frame 3B5S.W2 for models A2, B2, and C2

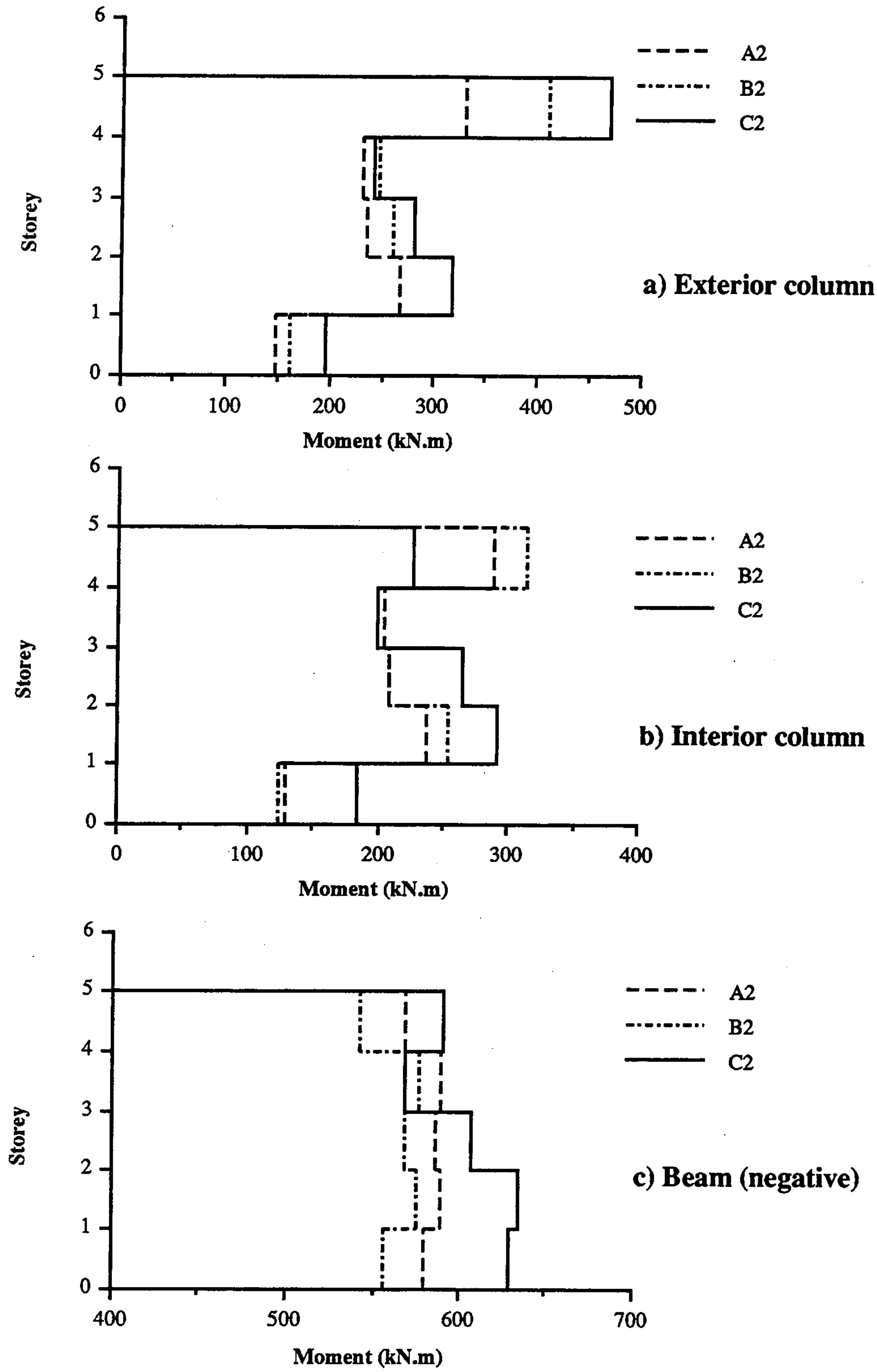

Fig. 5.21 Moments in frame 3B5S.W0 for models A2, B2, and C2

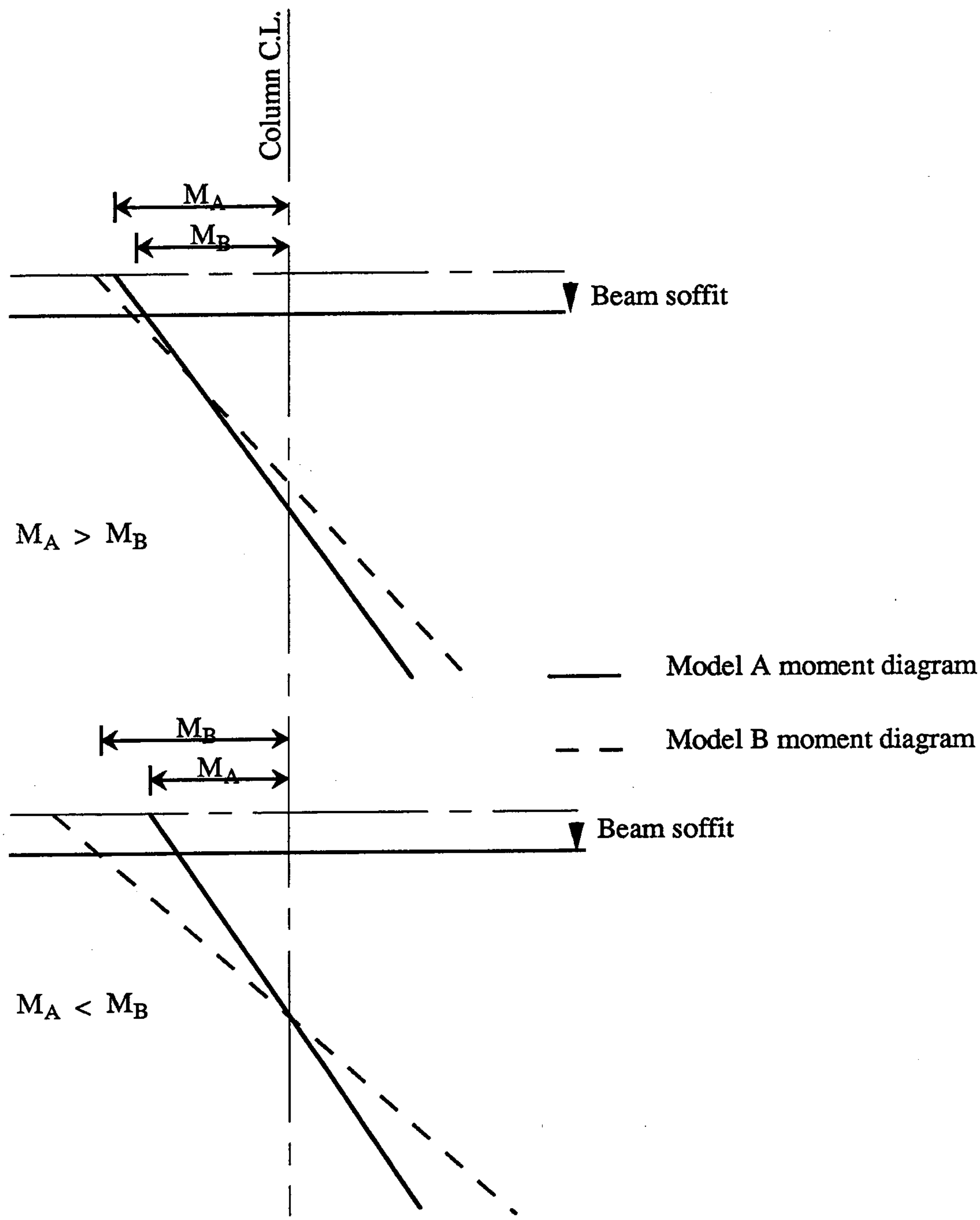

Fig. 5.22 Column moments from models A and B

6. Slenderness Effect in Unbraced Frames

6.1 Introduction

The term slenderness effect is used to denote the changes in the internal forces and stability of a structural system or its individual members due to the effect of deformation of this system. Slenderness effect in unbraced frames is attributed to two main causes:

1. Lateral drift effect which results from the fact that each storey will be laterally shifted from the one below it.

2. Member stability effect which results from the deviation of the deformed member centreline from the original (undeformed) centreline.

The "exact" way to account for lateral drift effect is to consider geometric non-linearity when analyzing the structure in question. This is usually done by iterating first-order analyses where the structural geometry and member stiffnesses are updated each time until the process converges. This is referred to as a second-order analysis.

To evaluate member stability effect, it is necessary to determine whether the maximum moment in a member is greater than its maximum end moment. A closed form solution for this problem is available based on Eqn. (4.1).

Approximate methods have been introduced to evaluate slenderness effect. The most commonly used method is the moment magnifier method, described in appendix B, which magnifies the maximum end moment of a slender member, obtained from a first-order analysis. This magnification is applied in two stages, a sway magnifier δ_s to account for lateral drift effect and a braced magnifier δ_b to account for member stability effect.

In this chapter, an "exact" second-order design and the first-order design using the moment magnifier method are compared. Based on this comparison, a modification to the column stiffness to be used when applying the moment magnifier method is proposed.

6.2 Evaluation of the moment magnifier method

To evaluate the moment magnifier method, test frames 3B12S, 3B4S and 3B5S were designed using design models B1 and B2 for both wind loads W1 and W2. It should be noted that the concrete dimensions in B1 design were selected to limit the sway magnifier (δ_s) to be less than 2.0 so that the magnified moments are reasonable. The design moments in beams, interior columns and exterior columns are shown in Figs. 6.1 to 6.6. The concrete dimensions for all members are listed in Tables 6.1 to 6.3. Table 6.4 shows the material quantities and cost indices for different designs.

For frame design 3B12S.W1, design moments in interior columns using design model B1 were not significantly different than those using design model B2 except for the bottom floor column where the design moment in B1 design was 50% more than that in B2 design. For exterior columns, design moments in B1 design were greater than those in B2 design in all floors. The difference was greatest for the bottom floor column where it reached 30% while it reached only 10% for the second floor column where the maximum design moment occurs. The difference in beam design moments was less than 10% in all floors.

Beam and column concrete dimensions from B1 design were significantly larger than those from B2 design (100 mm and 50 mm larger for column depths and beam depths, respectively). For the frame, a saving of 12% in concrete volume and 4% in overall cost was credited to B2 design compared to B1 design.

The same trend was also observed for frame design 3B12S.W2, except that less difference between B1 and B2 designs was recorded. Design moments in columns from design B1 were up to 25% greater than those from design B2. Design moments in beams were less than 10% different. Column depths were 50 mm larger in design B1 while beam dimensions were the same in both designs. Concrete volume in B2 design was less than that in B1 design by 3%. The overall cost was reduced by 1% using the B2 design.

For frame 3B4S, the column design moments for B1 design were generally greater than those for B2 design by as much as 50% in bottom interior column and 20% in bottom exterior column. Negative moments in beams were less than 5% different. The B1 design gave larger member dimensions (100 mm for columns and 50 mm for beams) than the B2 design for wind intensity W1. This was mainly due to the preset condition that the value of the sway magnifier δ_s should not exceed 2.0. For heavy wind W2, the columns in B1 design were 100 mm larger in depth than in B2 design and beam dimensions were 350x650 mm in B1 design and 400x600 mm in B2 design. For wind intensity W1, using model B2 resulted in 12% less concrete volume and 3% less cost compared to using model B1. For wind intensity W2, no saving was recorded for B2 design since the saving in column concrete volume was offset by larger beam concrete volume.

For frame 3B5S.W1, models B1 and B2 gave essentially the same design as design moments in all members differed by less than 10%. For wind intensity W2, B1 design gave 50 mm deeper columns than B2 design. The maximum difference in column moments was 35% for interior columns and 20% for exterior columns. However, the difference in concrete volume and cost index was less than 0.7% for both wind intensities. This can be attributed to the fact that unbalanced moments in columns due to gravity loads were as dominant as moments due to lateral load. As lateral load is doubled, moments due to lateral load become more dominant but not enough to significantly affect the total cost.

It can be concluded that, in general, the moment magnifier method gives a more conservative design compared to one based on a second-order analysis. Furthermore, the method does not seem to be consistent in its conservatism. A proposed modification to the method to give more consistent results is introduced.

6.3 Proposed expression for EI

While developing the part of ESURF that performs the moment magnifier method, it was noticed that the resulting moments are sensitive to the value of EI used in evaluating the value of the critical load P_c for calculating the sway magnifier δ_s and the braced magnifier δ_b. It was thought that if a better value of EI could be determined, the resulting moments, and hence the final design, would be closer to the second-order design.

Since the CEB model (appendix A) gives the most realistic estimate for the effective stiffness EI, it was considered reasonable to try to develop an expression for EI based on this model. The objective here is to improve the moment magnifier method by introducing a more realistic expression for the effective stiffness EI rather than to accurately predict the value given by the CEB model. As described in appendix A, the EI value obtained using the CEB model for any cross section depends mainly on the cross sectional dimensions (c_1 and c_2), the loading conditions (axial load P and bending moment M), and the amount of reinforcement in the section. For columns, those factors can be represented by the following three parameters:

E_cI_g = the gross stiffness of the concrete cross section.

e/c_1 = the ratio of eccentricity to the section depth.

ρ = the reinforcement ratio.

The desired equation will then take the form:

$$EI = \{f(\frac{e}{c_1}, \rho)\} E_cI_g \tag{6.1a}$$

To determine the function f, a non-linear regression analysis was undertaken. The subroutine RNLIN from the IMSL library (IMSL 1987) which fits a set of data using the least squares method was used for this purpose. The dependent variable is chosen to be the ratio of the effective to the gross stiffness (Y). The independent variables are chosen to be as follows:

X1 = e/c_1

X2 = reinforcement ratio ρ

Thus Eqn. (6.1(a)) can be expressed as:

$$Y = f(X1,X2) \tag{6.1b}$$

To determine the function implied in Eqn. (6.1(b)), a general expression was assumed of the form:

$$Y = a\,X1^{b} + c\,X2^{d} + e \tag{6.2}$$

where a, b, c, d, and e are constants.

The data required for the regression analysis were obtained from ESURF by printing out the values of Y, X1, and X2 for each column in frame designs 3B12S.W1.C2, 3B12S.W2.C2, and 3B4S.W1.C2. The results are listed in Table 6.5. Using this data, the resulting equation was:

$$Y = 0.28\,X1^{-0.73} + 17.49\,X2^{1.09} + 0.046 \qquad (6.3)$$

with a coefficient of determination (Welsolowsky 1976) $r^2 = 0.967$.

To simplify the equation, the constants b and d were given the values -0.5 and 1.0, respectively. The analysis was then repeated to evaluate a, c, and e. The resulting equation became:

$$Y = \frac{0.49}{\sqrt{X1}} + 15.48\,X2 - 0.24 \qquad (6.4)$$

with a coefficient of determination $r^2 = 0.962$

For more simplicity, the constants were rounded off and the equation in its final form was:

$$\frac{EI}{E_c I_g} = \frac{0.5}{\sqrt{\frac{e}{c_1}}} + 15.0\,\rho - 0.25 \qquad (6.5a)$$

The EI value from Eqn. (6.5(a)) should not exceed the stiffness of the uncracked transformed section. This usually occurs when the ratio e/c_1 is very small (less than 0.15). From a study of a number of cross sections under different loading conditions, the uncracked transformed stiffness was found to be approximately as follows:

$$\left(\frac{EI}{E_c I_g}\right)_{max} = 1.0 + 8.0\,\rho \tag{6.5b}$$

The proposed expression for column effective stiffness takes the form:

$$EI = \left(\frac{0.5}{\sqrt{\frac{e}{c_1}}} + 15.0\,\rho - 0.25\right) E_c I_g \leq (1.0 + 8.0\,\rho)\, E_c I_g \tag{6.6}$$

6.4 Verification of the proposed expression

To examine the proposed expression, the three test frames 3B12S, 3B4S, and 3B5S were redesigned using the proposed expression to calculate EI in the moment magnifier method. The resulting moments are also plotted in Figs. 6.1 to 6.6 and designated as modified moment magnifier design (B1M). In some cases, the difference in moments between B2 design and B1M design was so small that the two lines representing these two design moments overlapped. The concrete dimensions for each design are listed in Tables 6.1 to 6.3. Material quantities and cost indices are shown in Table 6.4.

In all cases, the first-order design using the moment magnifier method with the proposed expression for EI gave the same concrete dimensions as the second-order design. Moreover, the moments in beams and columns from the two methods were at most 5% different. This resulted in no difference in concrete volume and very trivial difference in cost index.

It is therefore concluded that the proposed expression for evaluating the effective stiffness of columns EI to be used in the moment magnifier method results in designs that are very close to the second-order designs.

6.5 Critical examination of the code expression for EI

Expressions for the effective stiffness to be used in the moment magnifier method contained in CSA A23.3 design code are reproduced in appendix B as Eqns. (B.2) and (B.3). To facilitate examining these expressions against the proposed expression, they need to be recast as follows:

Eqn. (B.2) will be:

$$\frac{EI}{E_c I_g} = 0.25 \qquad (6.7)$$

For Eqn. (B.3), setting creep coefficient β_d=0 :

$$EI = (0.2\, E_c I_g + E_s I_{se}) \qquad (6.8a)$$

$$EI = (0.2 + n \frac{I_{se}}{I_g})\, E_c I_g \qquad (6.8b)$$

where I_{se} is the moment of inertia of reinforcement about centroidal axis of cross section and n is the modular ratio. For a 2-face arrangement of reinforcement, this moment of inertia can be expressed as:

$$I_{se} = 2\left(\frac{A_s}{2}\left(\frac{\gamma c_1}{2}\right)^2\right) \qquad (6.9a)$$

where γ is a dimensionless parameter indicating the location of reinforcement relative to the column depth.

With $\quad A_s = \rho\, c_1 c_2$

and $\quad I_g = c_1^3 c_2/12.0,$

$$I_{se} = 3\gamma^2 \rho\ I_g \tag{6.9b}$$

Substituting Eqn. (6.9(b)) in Eqn. (6.8(b)):

$$EI = (0.2 + 3n\gamma^2 \rho)\, E_c I_g \tag{6.10}$$

Assuming a value of 0.7 for γ and 7.0 for n:

$$EI = (0.2 + 10\,\rho)\, E_c I_g \tag{6.10a}$$

or

$$\frac{EI}{E_c I_g} = 0.2 + 10\,\rho \tag{6.10b}$$

Eqns. (6.6), (6.7) and (6.10) are plotted on Fig. 6.7 for values of ρ=1.0%, 2.0%, 3.0%, and 4.0%. The figure shows that the code expression (Eqn. (6.7)) is an absolutely conservative estimate for EI. The other code expression (Eqn. (6.10)) significantly underestimates EI for small values of e/c_1 (mostly uncracked sections). For values of e/c_1 between 1.0 and 2.0, Eqn. (6.10) gives reasonably conservative values for EI. For the rare cases of e/c_1 greater than 2.0, values from Eqn. (6.10) are close to the values from the proposed expression for high reinforcement ratios and slightly larger for low reinforcement ratios. For such high values of eccentricities, it is highly unusual to have low ratios of reinforcement if the dimensions are selected according to strength requirements.

It would appear that the code expression for EI was selected to give reasonable results for high ratios of eccentricity and for all values of reinforcement ratios but to give very conservative results for low ratios of eccentricity especially for low reinforcement ratios.

To demonstrate the above observations, a numerical example to calculate the braced magnification factor δ_b for an isolated column is introduced. The factor calculated using the theoretical solution given in Eqn. (4.1) is denoted by δ'_b and when using the code equation (Eqn. (B.8)) is denoted by δ_b. For each equation, both the proposed expression (Eqn. (6.6)) and the code expression (Eqn. (6.10)) for EI are used. The braced magnification factor is calculated for small eccentricity ratios ($e/c_1 \leq 0.15$) and large eccentricity ratios ($e/c_1 \geq 1.0$). In each case, a high reinforcement ratio (4.0%) and a low reinforcement ratio (1.0%) are used.

The theoretical solution gives the magnification factor δ'_b as:

$$\delta'_b = \sqrt{\frac{\left(\frac{M_1}{M_2}\right)^2 + 2\left(\frac{M_1}{M_2}\right)\cos\alpha + 1}{\sin^2\alpha}} \qquad (4.1)$$

where

$$\alpha^2 = \frac{Pl_u^2}{EI} = \frac{Pl_u^2}{YE_cI_g}$$

The code equation (Eqn. (B.8)) estimates δ_b as:

$$\delta_b = \frac{C_m}{1 - \frac{P}{\phi_m P_c}} \tag{6.11}$$

where

$$\phi_m = 0.65$$

$$C_m = 0.6 + 0.4\left(\frac{M_1}{M_2}\right) \tag{6.12}$$

and

$$P_c = \frac{\pi^2 EI}{(kl_u)^2} \tag{6.13}$$

Assuming an effective length factor k=1.0 and substituting for EI as $Y(E_c I_g)$, we can write:

$$\frac{P}{\phi_m P_c} = \frac{P\, l_u^2}{\phi_m \pi^2 Y E_c I_g} = \frac{\alpha^2}{\phi_m \pi^2} \tag{6.14}$$

Substituting for Eqn. (6.14) in Eqn. (6.11) with ϕ_m=0.65 and π^2=9.87 we can see that:

$$\delta_b = \frac{C_m}{1 - \frac{\alpha^2}{6.415}} \tag{6.15}$$

The value of Y is calculated using the code expression (Eqn. (6.10)), denoted by Y_{code}, and using the proposed expression (Eqn.

(6.6)) denoted by Y_{prop}. The value of α^2 is then calculated to evaluate δ_b and δ'_b.

The column being considered has the following information:

c_1	= 300	mm	c_2	= 300	mm
f'_c	= 40	MPa	f_y	= 400	MPa
E_c	= 31,623	MPa	E_s	= 200,000	MPa
l_u	= 4400	mm	γ	= (300-125)/300 = 0.583	
n	= 6.325				

Four cases are considered:

Case (a): Small eccentricity ratio with high reinforcement ratio where

M_2	= 50.0	kN.m	M_1	= -50.0 kN.m to 50.0 kN.m	
P	= 2200	kN	ρ	= 4.0%	(on two faces)
e/c_1	= 0.08				

$$\alpha^2 = \frac{Pl_u^2}{YE_cI_g} = \frac{2200\text{x}10^3(4400)^2}{31{,}623\,\frac{300(300)^3}{12}\,Y} = \frac{2.0}{Y}$$

where Y is calculated in two ways:

$$Y_{code} = 0.2 + 3\ (6.325)\ (0.583)^2\ (0.04) = 0.458$$

$$Y_{prop} = 1.0 + 8.0\ (0.04) = 1.32$$

Case (b): Small eccentricity ratio with low reinforcement ratio where

M_2	= 50.0	kN.m	M_1	= -50.0 kN.m to 50.0 kN.m	
P	= 1600	kN	ρ	= 1.0%	(on two faces)
e/c_1	= 0.10				

$$\alpha^2 = \frac{Pl_u^2}{YE_cI_g} = \frac{1600x10^3(4400)^2}{31{,}623\,\frac{300(300)^3}{12}\,Y} = \frac{1.455}{Y}$$

and Y is either:

$$Y_{code} = 0.2 + 3\ (6.325)\ (0.583)^2\ (0.01) = 0.265$$

or:

$$Y_{prop} = 1.0 + 8.0\ (0.01) = 1.08$$

Case (c): Large eccentricity ratio with high reinforcement ratio where

M_2	= 150.0	kN.m	M_1	= -150.0 kN.m to 150.0 kN.m	
P	= 400	kN	ρ	= 4.0%	(on two faces)
e/c_1	= 1.25				

$$\alpha^2 = \frac{Pl_u^2}{YE_cI_g} = \frac{400x10^3(4400)^2}{31{,}623\,\frac{300(300)^3}{12}\,Y} = \frac{0.364}{Y}$$

and Y is either:

$$Y_{code} = 0.2 + 3\ (6.325)\ (0.583)^2\ (0.04) = 0.458$$

or:

$$Y_{prop} = \frac{0.5}{\sqrt{1.25}} + 15.0\,(0.04) - 0.25 = 0.80$$

Case (d): Large eccentricity ratio with low reinforcement ratio where

M_2	= 54.0	kN.m	M_1	= -54.0 kN.m to 54.0 kN.m	
P	= 180	kN	ρ	= 1.0%	(on two faces)
e/c_1	= 1.0				

$$\alpha^2 = \frac{Pl_u^2}{YE_cI_g} = \frac{180\times10^3(4400)^2}{31{,}623\,\frac{300(300)^3}{12}\,Y} = \frac{0.164}{Y}$$

and Y is either:

$$Y_{code} = 0.2 + 3\ (6.325)\ (0.583)^2\ (0.01) = 0.265$$

or:

$$Y_{prop} = \frac{0.5}{\sqrt{1.0}} + 15.0\,(0.01) - 0.25 = 0.40$$

Table 6.6 shows the values of δ_b and δ'_b , using the values of EI computed from both Eqns. (6.10) and (6.6), for different values of end moments ratio (M_1/M_2) for all four cases. Fig. 6.8 shows the ratio (δ_b/δ'_b) with different (M_1/M_2) ratios for low eccentricity ratios (cases (a) and (b)).

For low eccentricity ratios (cases (a) and (b)), it is seen that the values of δ_b and δ'_b computed using the code value of EI given in Eqn. (6.10) differ significantly and for low values of reinforcement ratios by a factor of 3. However, when using the proposed value of EI given in Eqn. (6.6) the values of the braced magnification factor δ_b and δ'_b are essentially identical for all ratios of reinforcement.

For high eccentricity ratios (cases (c) and (d)), Table 6.6 shows that the code equation and the theoretical equation gave almost the same values of the braced magnifier using either the code expression or the proposed expression for EI.

In summary, it is concluded that the code equation for calculating the braced magnifier, using the code expression for EI, significantly overestimates its value compared to the one given by the theoretical equation for low eccentricity ratios. For high eccentricity ratios, the code equation gives good agreement with the theoretical equation. This confirms the previously observed inconsistency in the conservatism of the code expression for EI. On the other hand, when the proposed expression for EI was used, both the code and theoretical equation for calculating the braced magnification factor were in good agreement for both low and high eccentricity ratios.

An interesting observation about the expression given in CSA A23.3 design code for determining the effective stiffness EI (Eqn. (6.10)) is the weight it gives to the position of reinforcing bars. To investigate this, cases (a) through (d) were recalculated using the code expression for EI assuming a 4-face arrangement of reinforcement. The number of reinforcing bars is assumed to be eight distributed as shown in Fig. 6.9. In Table 6.6, the numbers between brackets indicate the values of δ_b (or δ'_b) for 4-face reinforcement.

The numbers show that, for low eccentricity ratios, the code expression for EI is so sensitive to the reinforcement arrangement that it resulted in, for $M_1/M_2 = 1.0$, 54% larger value of δ_b than the 2-face arrangement case whereas it gave only 17% larger value of δ'_b using the theoretical equation. For high eccentricity ratios, the code expression is not much affected by the reinforcement arrangement giving almost the same values as for the 2-face reinforcement arrangement. Thus it is concluded that the code expression for EI places too much weight on the effect of reinforcement arrangement on the column effective stiffness for cases of low eccentricity ratios.

Wind	Storey	Exterior Column $c_1 \times c_2$ mm			Interior Column $c_1 \times c_2$ mm			Beam $b \times h$ mm		
		B1	B2	B1M	B1	B2	B1M	B1	B2	B1M
W1	1–4	500x350	400x350	400x350	600x500	500x500	500x500	400x700	400x650	400x650
	5–8	450x350	350x350	350x350	550x450	450x450	450x450			
	9–12	400x350	300x350	300x350	450x350	350x350	350x350			
W2	1–4	450x350	400x350	400x350	600x500	550x550	550x550	400x700	400x700	400x700
	5–8	400x350	350x350	350x350	500x450	450x450	450x450			
	9–12				400x350	350x350	350x350			

Table 6.1 Concrete dimensions of frame 3B12S for models B1, B2, and B1M

Wind	Storey	Exterior Column $c_1 \times c_2$ mm			Interior Column $c_1 \times c_2$ mm			Beam b x h mm		
		B1	B2	B1M	B1	B2	B1M	B1	B2	B1M
W1	1 2 3 4	450x250	350x250	350x250	400x300	300x300	300x300	350x650	350x600	350x600
W2	1 2 3 4	450x250	350x250	350x250	400x300	300x300	300x300	350x650	400x600	400x600

Table 6.2 Concrete dimensions of frame 3B4S for models B1, B2, and B1M

Wind	Storey	Exterior Column $c_1 \times c_2$ mm			Interior Column $c_1 \times c_2$ mm			Beam b x h mm		
		B1	B2	B1M	B1	B2	B1M	B1	B2	B1M
W1	1 2 3 4 5	450x250	450x250	450x250	450x250	450x250	450x250	400x650	400x650	400x650
W2	1 2 3 4 5	450x250	400x250	400x250	400x300	350x300	350x300	400x650	400x650	400x650

Table 6.3 Concrete dimensions of frame 3B5S for models B1, B2, and B1M

	Model	3B12S.W1	3B12S.W2	3B4S.W1	3B4S.W2	3B5S.W1	3B5S.W2
Concrete Volume cu. m	B1	134.67	132.68	34.20	34.20	41.92	42.88
	B2	121.0	130.27	30.50	34.0	41.90	42.40
	B1M	121.0	130.27	30.50	34.0	41.90	42.40
Steel Mass kg	B1	14,646	17,212	4,519	4,680	8,852	8,751
	B2	16,391	17,110	5,269	5,299	9,026	9,191
	B1M	16,461	17,192	5,306	5,326	9,067	9,289
Forms sq. m	B1	1,102	1,092	327	327	384	387
	B2	1,035	1,079	304	318	384	383
	B1M	1,035	1,079	304	318	384	383
Cost Index	B1	460.6	471.7	133.4	134.4	178.6	179.5
	B2	444.1	466.3	129.3	135.0	179.7	180.8
	B1M	444.5	466.8	129.5	135.2	179.9	181.4

Table 6.4 Material quantities and cost indices for models B1, B2, and B1M

Frame	Y	X1	X2	Y	X1	X2
3B12S.W1	1.484	0.224	0.050	0.815	0.424	0.021
	1.355	0.214	0.037	0.821	0.402	0.019
	1.305	0.243	0.034	0.691	0.502	0.019
	1.197	0.267	0.028	0.792	0.520	0.025
	1.220	0.199	0.023	0.669	0.724	0.025
	1.092	0.289	0.023	0.660	0.894	0.027
	1.230	0.218	0.024	0.629	1.045	0.027
	0.963	0.361	0.024	0.770	2.578	0.042
	1.056	0.299	0.021	0.767	2.706	0.042
3B12S.W2	1.473	0.247	0.046	1.076	0.322	0.025
	1.345	0.236	0.034	0.741	0.576	0.025
	1.272	0.280	0.033	0.842	0.488	0.026
	1.145	0.297	0.026	0.683	0.735	0.026
	1.018	0.357	0.025	0.678	0.827	0.027
	1.230	0.183	0.024	0.630	1.041	0.027
	0.969	0.358	0.024	0.690	2.317	0.036
	1.280	0.193	0.029	0.686	2.458	0.036
	0.873	0.491	0.029			
3B4S.W1	1.153	0.213	0.016	0.715	0.860	0.030
	0.745	0.410	0.016	0.688	0.980	0.030
	0.810	0.664	0.032	0.852	2.610	0.048
	0.775	0.749	0.032	0.850	2.710	0.048

Table 6.5 Regression analysis data

EI Exprn.	Case	Factor	$\frac{M_1}{M_2}$								
			1.00	0.75	0.50	0.25	0.00	-0.25	-0.50	-0.75	-1.00
Proposed (Eqn. (6.6))	(a)	δ'_b	1.23	1.09	1.02	1.00	1.00	1.00	1.00	1.00	1.00
		δ_b	1.31	1.18	1.05	1.00	1.00	1.00	1.00	1.00	1.00
	(b)	δ'_b	1.20	1.07	1.01	1.00	1.00	1.00	1.00	1.00	1.00
		δ_b	1.27	1.14	1.01	1.00	1.00	1.00	1.00	1.00	1.00
	(c)	δ'_b	1.06	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
		δ_b	1.08	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	(d)	δ'_b	1.05	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
		δ_b	1.07	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Code (Eqn. (6.10))	(a)	δ'_b	1.99 (2.33)	1.75 (2.04)	1.52 (1.77)	1.32 (1.51)	1.15 (1.29)	1.04 (1.11)	1.00 (1.01)	1.00 (1.00)	1.00 (1.00)
		δ_b	3.13 (4.82)	2.82 (4.34)	2.51 (3.86)	2.19 (3.37)	1.88 (2.89)	1.57 (2.41)	1.25 (1.93)	1.25 (1.93)	1.25 (1.93)
	(b)	δ'_b	2.58 (3.20)	2.26 (2.47)	1.95 (2.13)	1.66 (1.81)	1.40 (1.51)	1.18 (1.25)	1.04 (1.07)	1.04 (1.07)	1.04 (1.07)
		δ_b	7.01 (11.16)	6.31 (10.04)	5.61 (8.93)	4.91 (7.81)	4.21 (6.70)	3.51 (5.58)	2.80 (4.46)	2.80 (4.46)	2.80 (4.46)
	(c)	δ'_b	1.11 (1.12)	1.01 (1.02)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)
		δ_b	1.14 (1.17)	1.03 (1.05)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)
	(d)	δ'_b	1.08 (1.09)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)
		δ_b	1.11 (1.12)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)

Values in brackets are for 4-face reinforcement arrangement

Table 6.6 Theoretical and code values of braced magnification factor

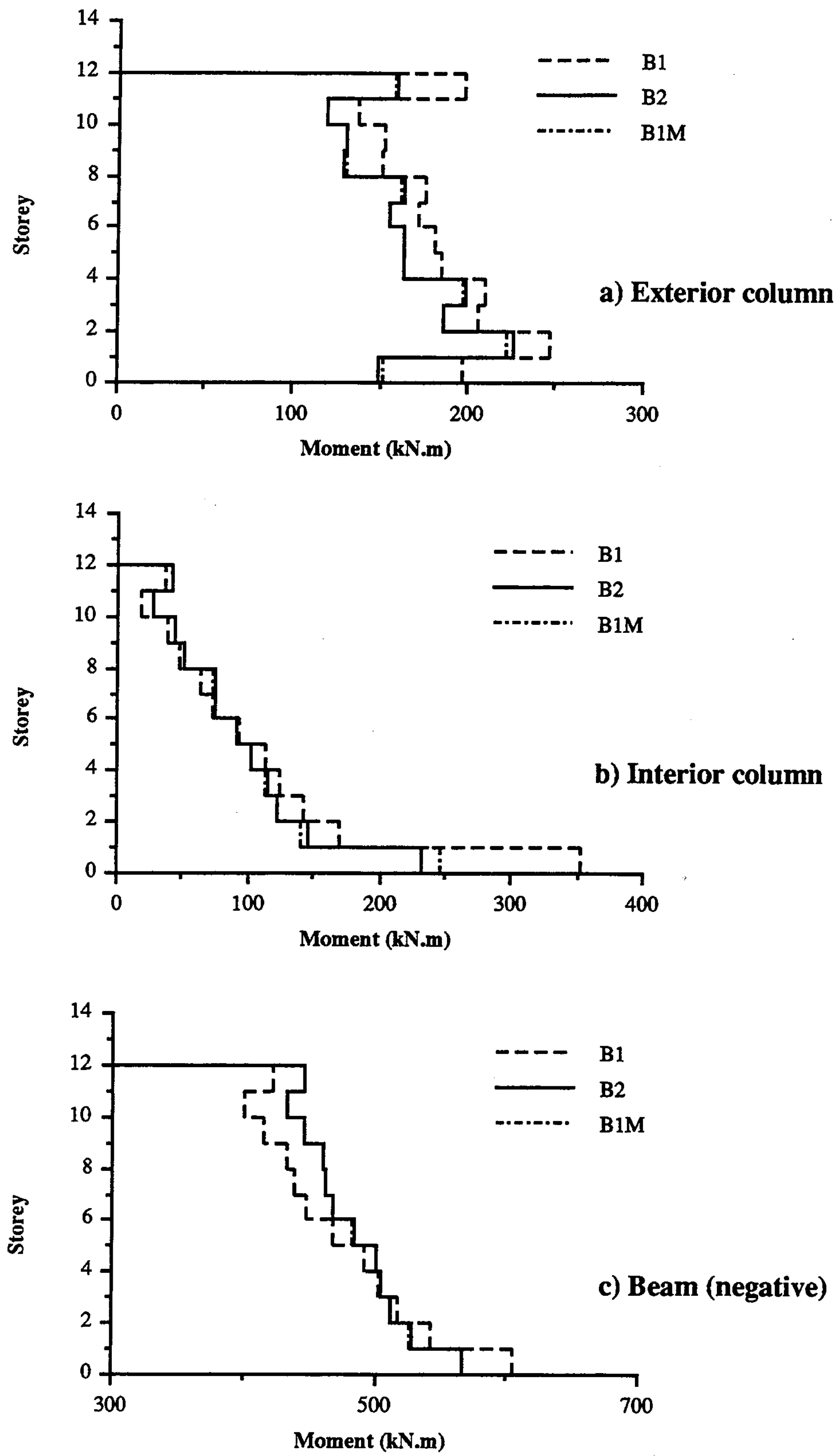

Fig. 6.1 Moments in frame 3B12S.W1 for models B1, B2, and B1M

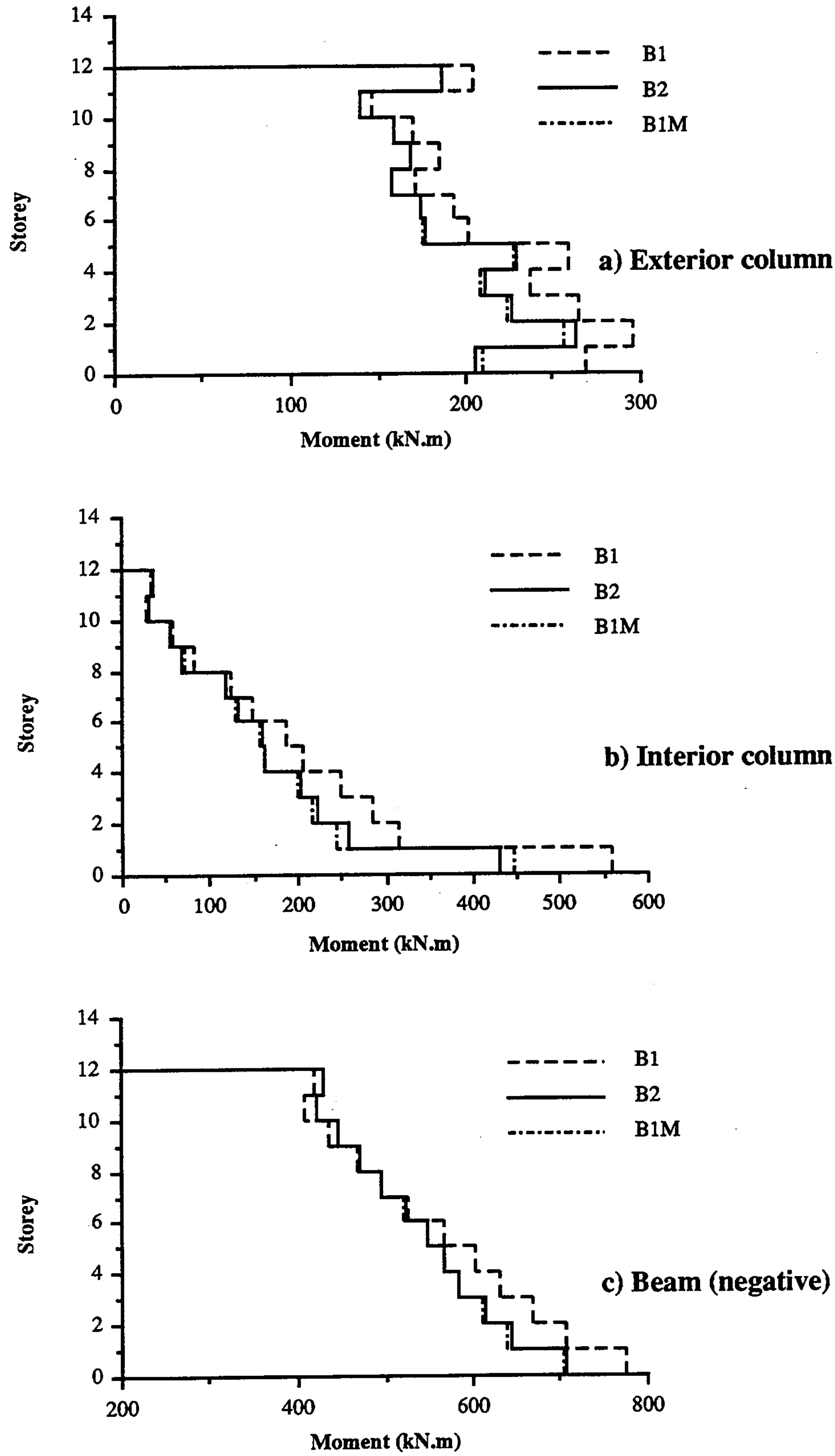

Fig. 6.2 Moments in frame 3B12S.W2 for models B1, B2, and B1M

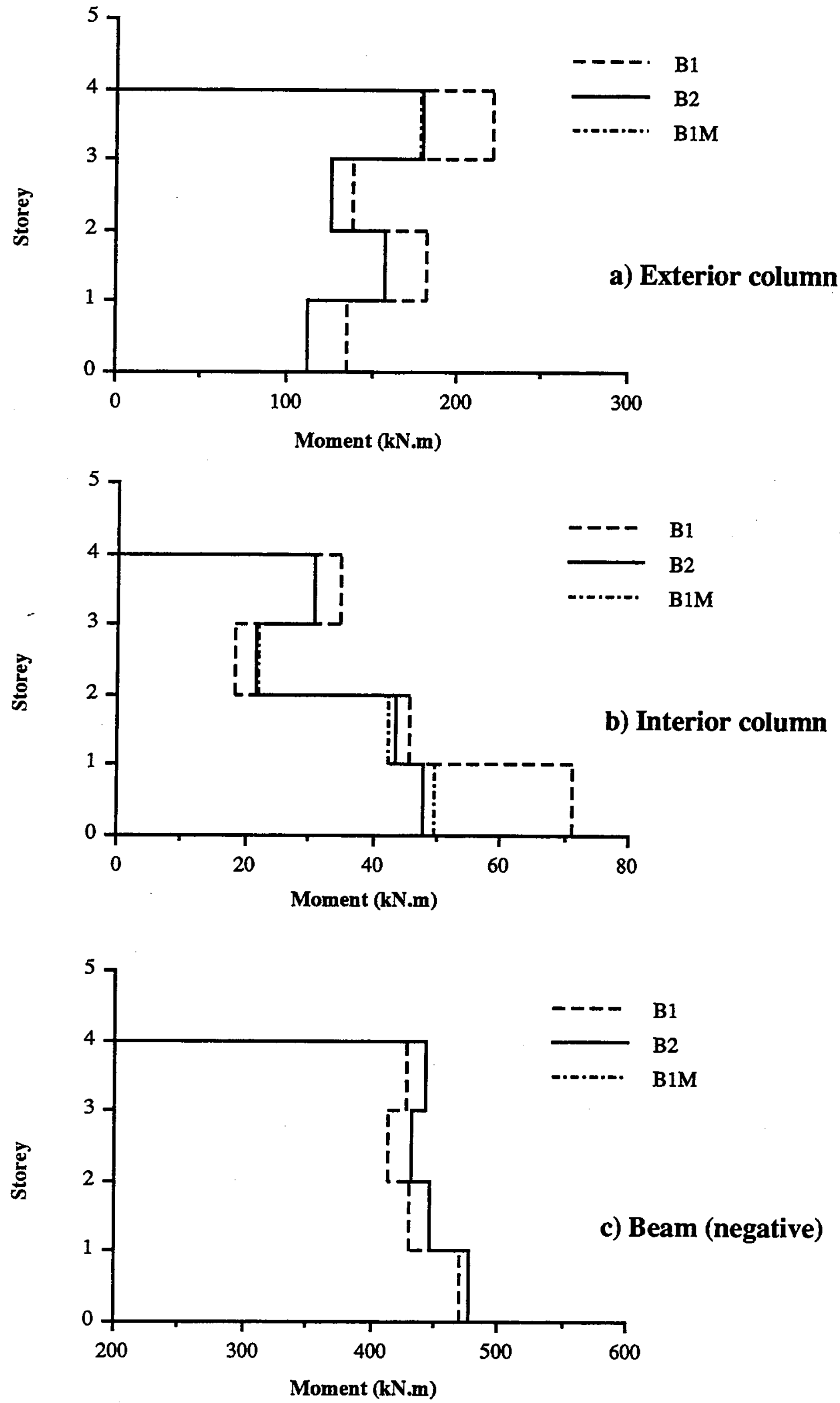

Fig. 6.3 Moments in frame 3B4S.W1 for models B1, B2, and B1M

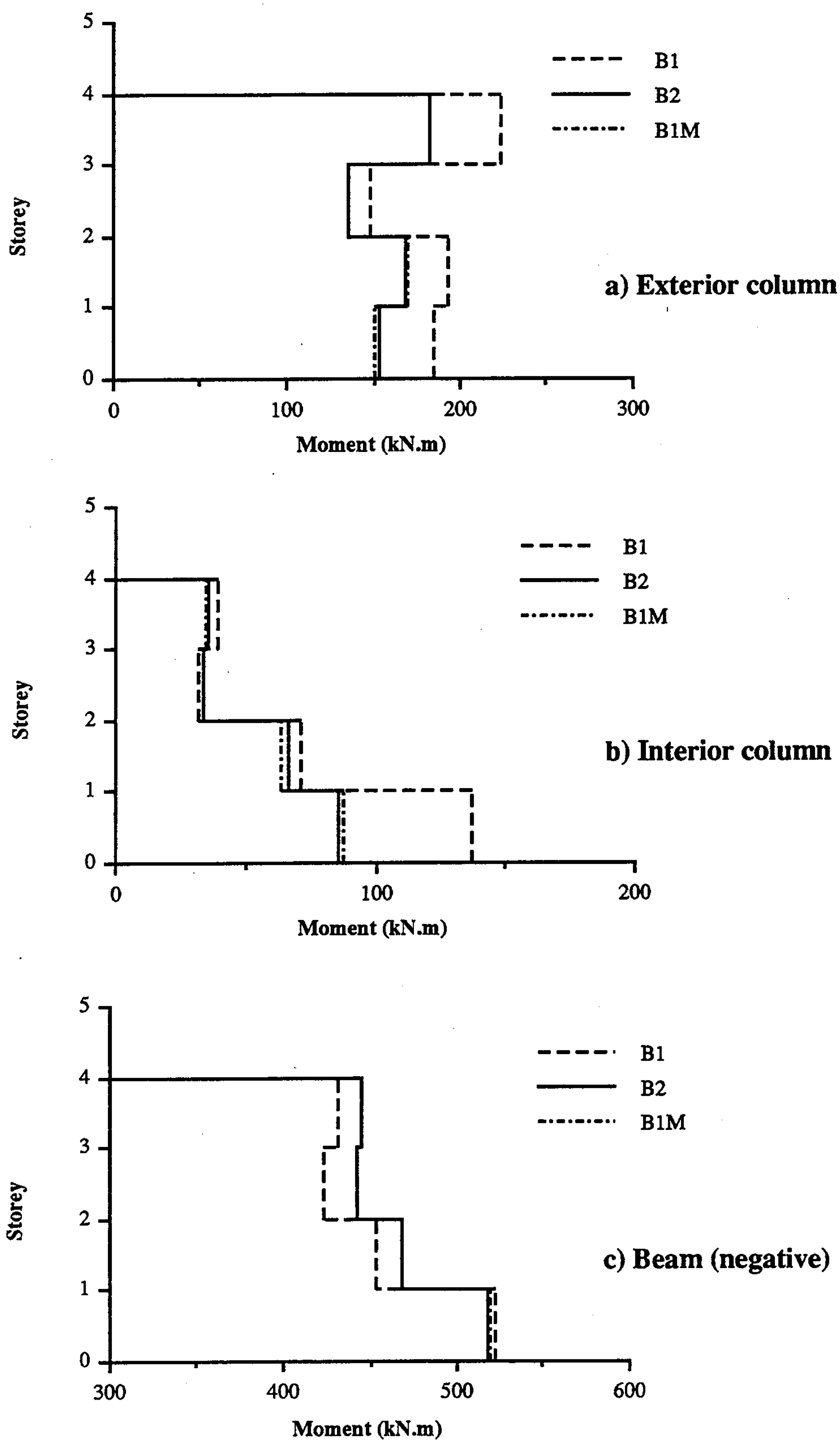

Fig. 6.4 Moments in frame 3B4S.W2 for models B1, B2, and B1M

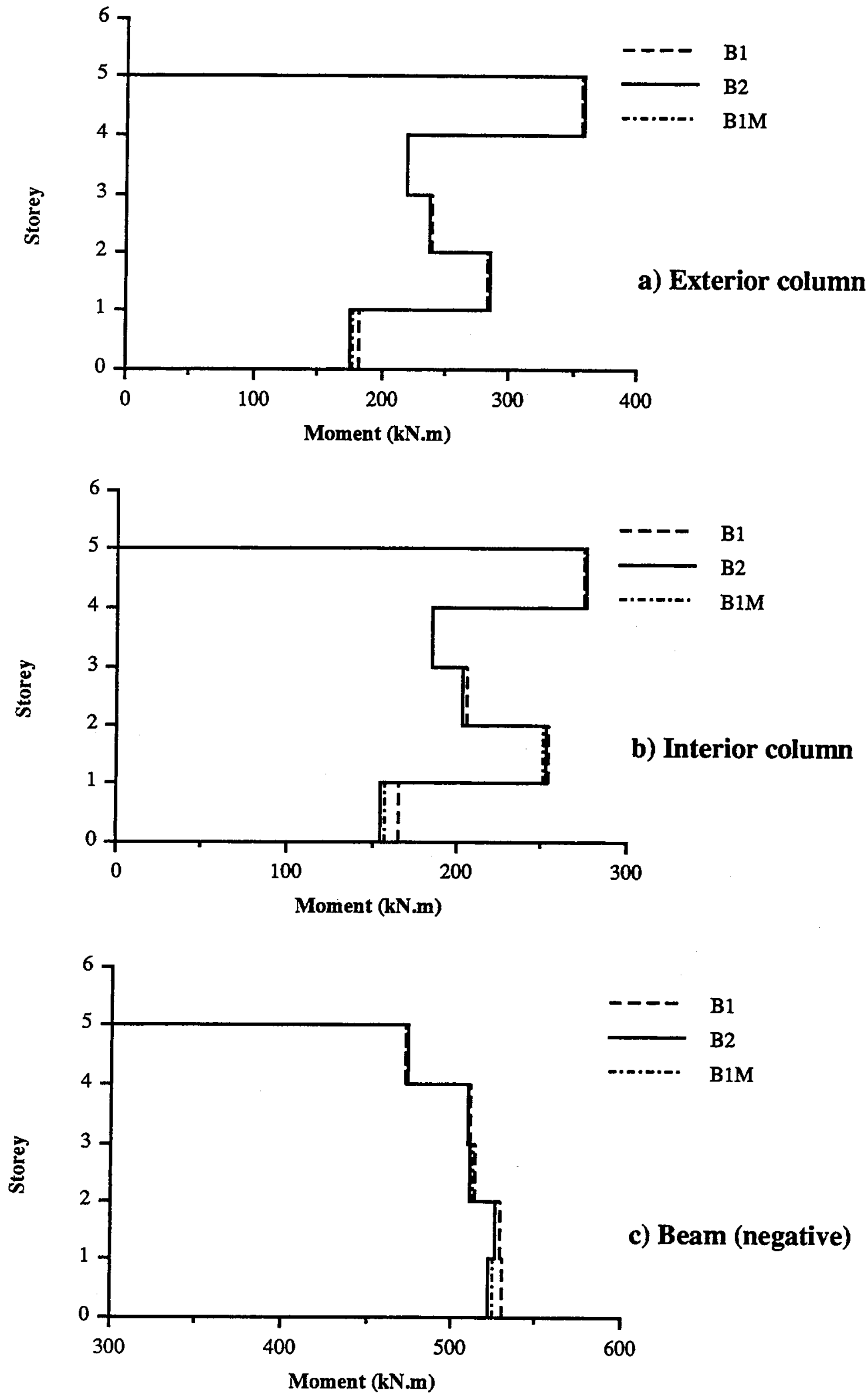

Fig. 6.5 Moments in frame 3B5S.W1 for models B1, B2, and B1M

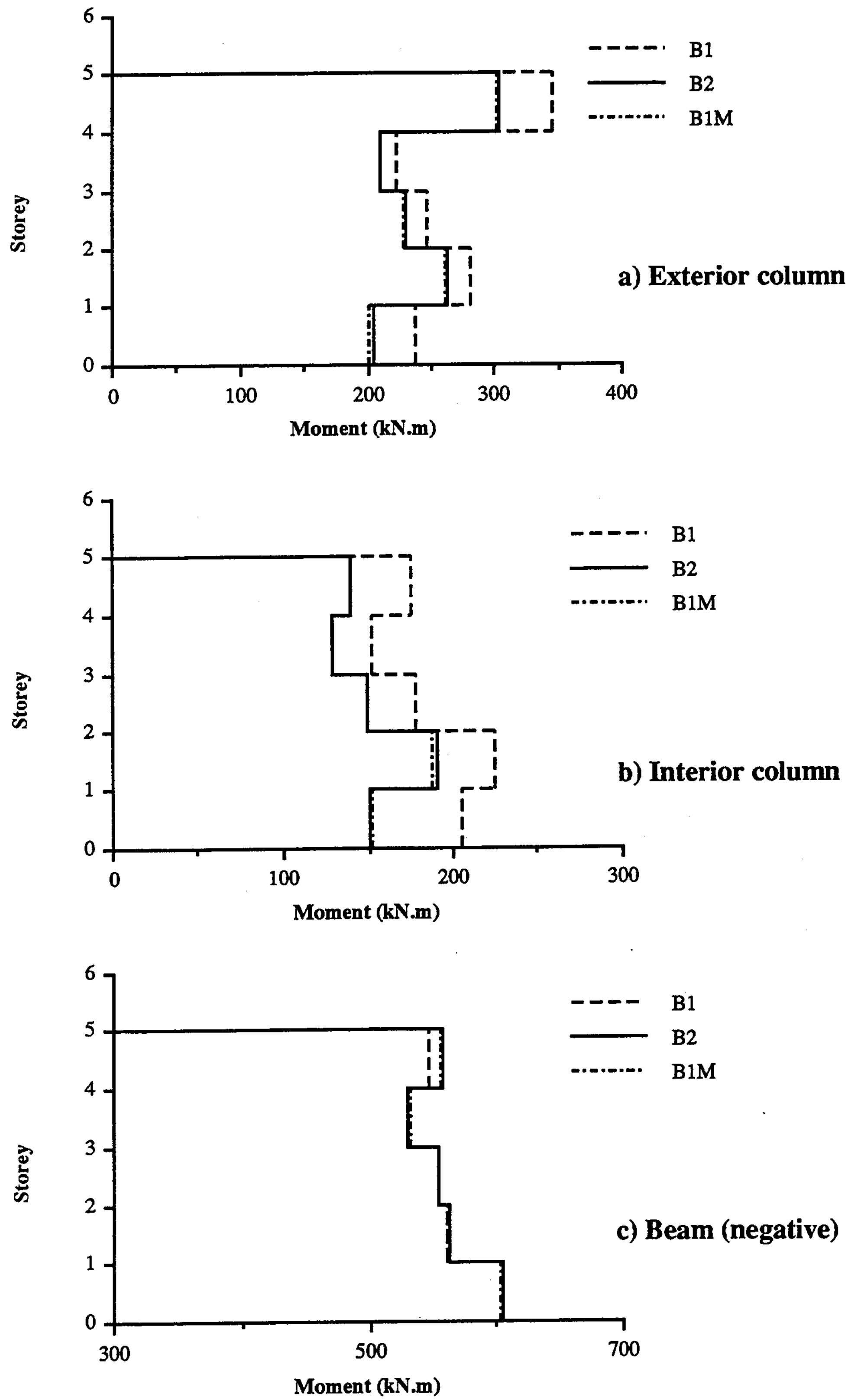

Fig. 6.6 Moments in frame 3B5S.W2 for models B1, B2, and B1M

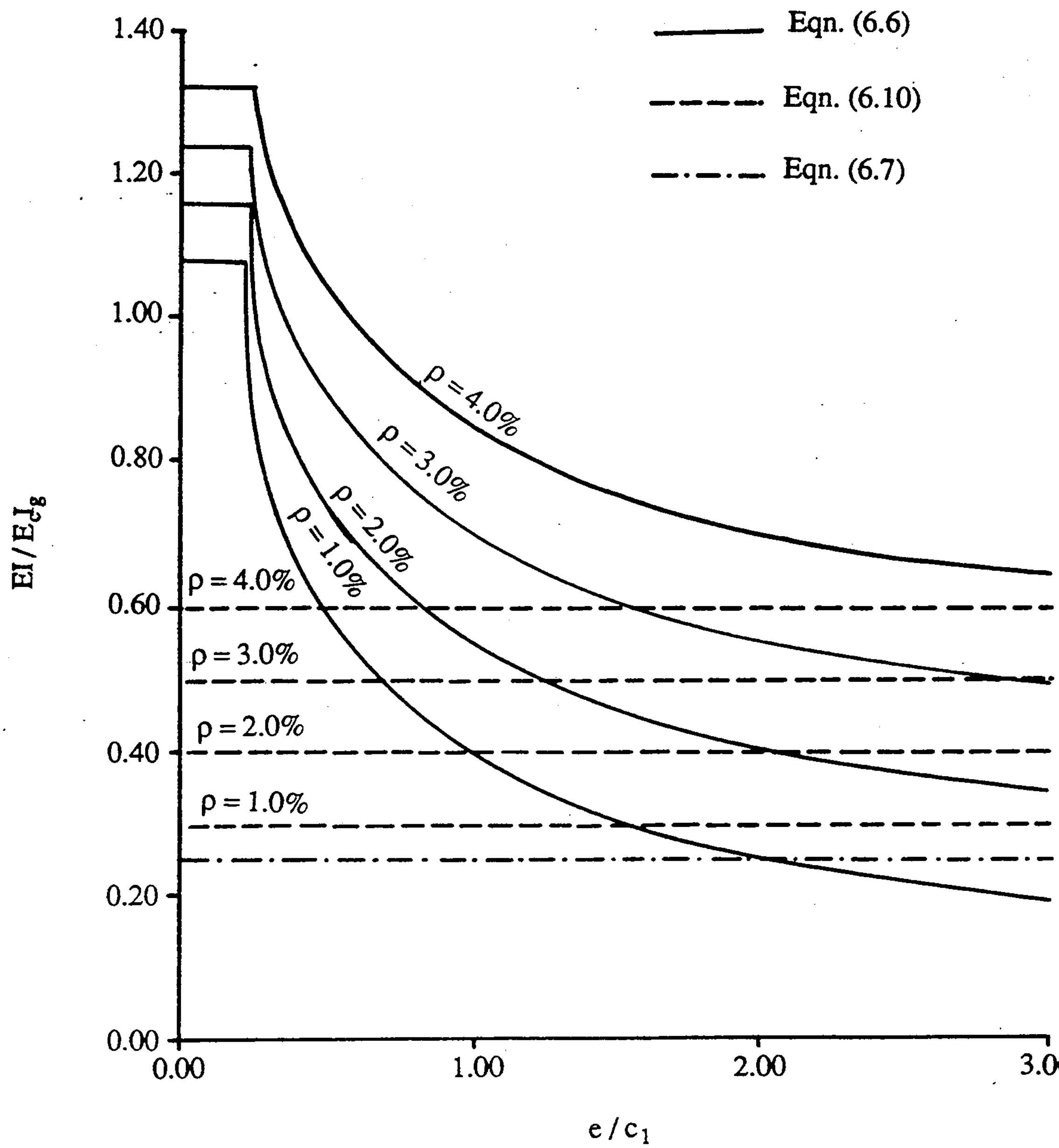

Fig. 6.7 Comparison of code and proposed expressions for EI

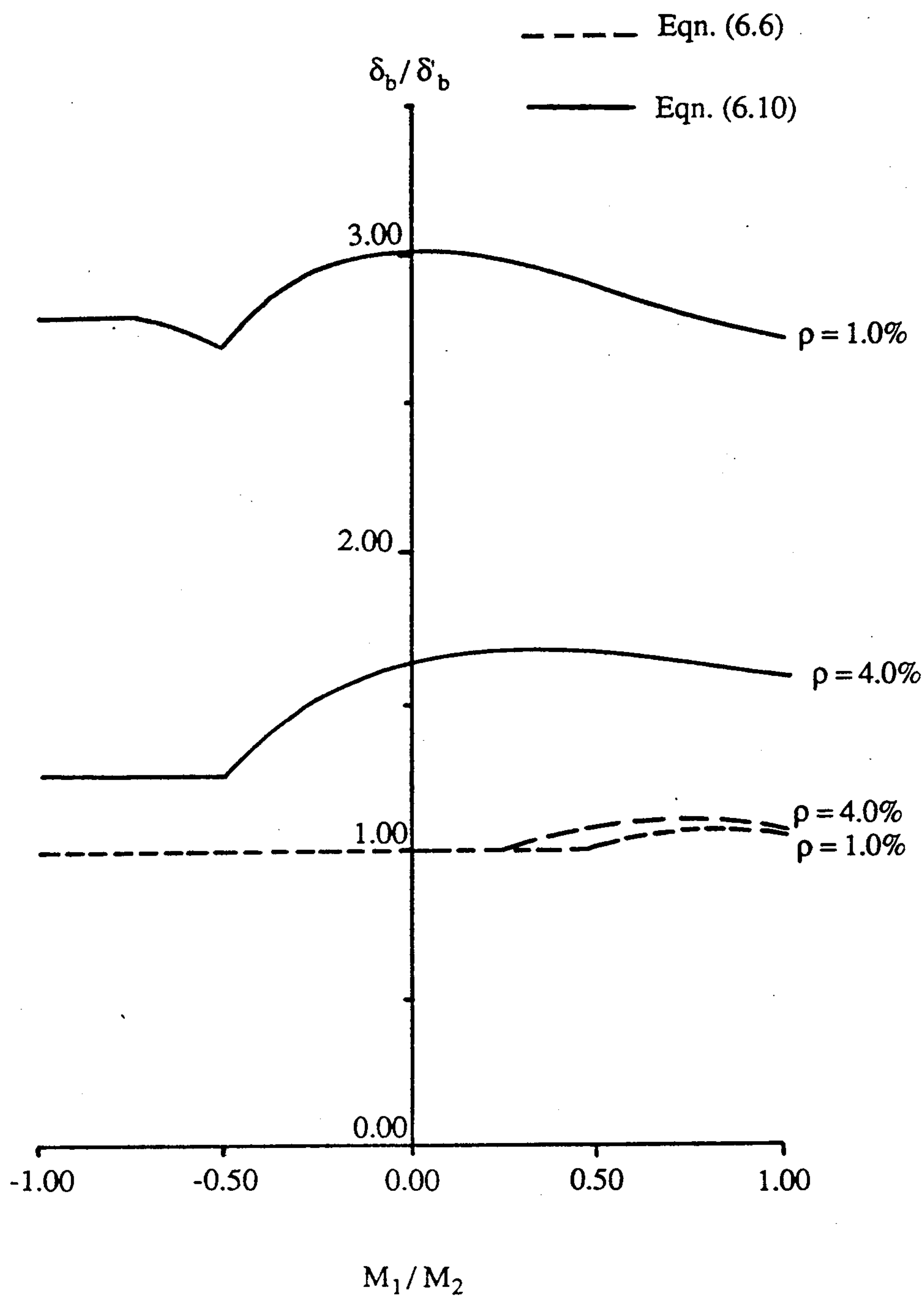

Fig. 6.8 Comparison of code and theoretical values of braced magnification factor for small eccentricity ratios

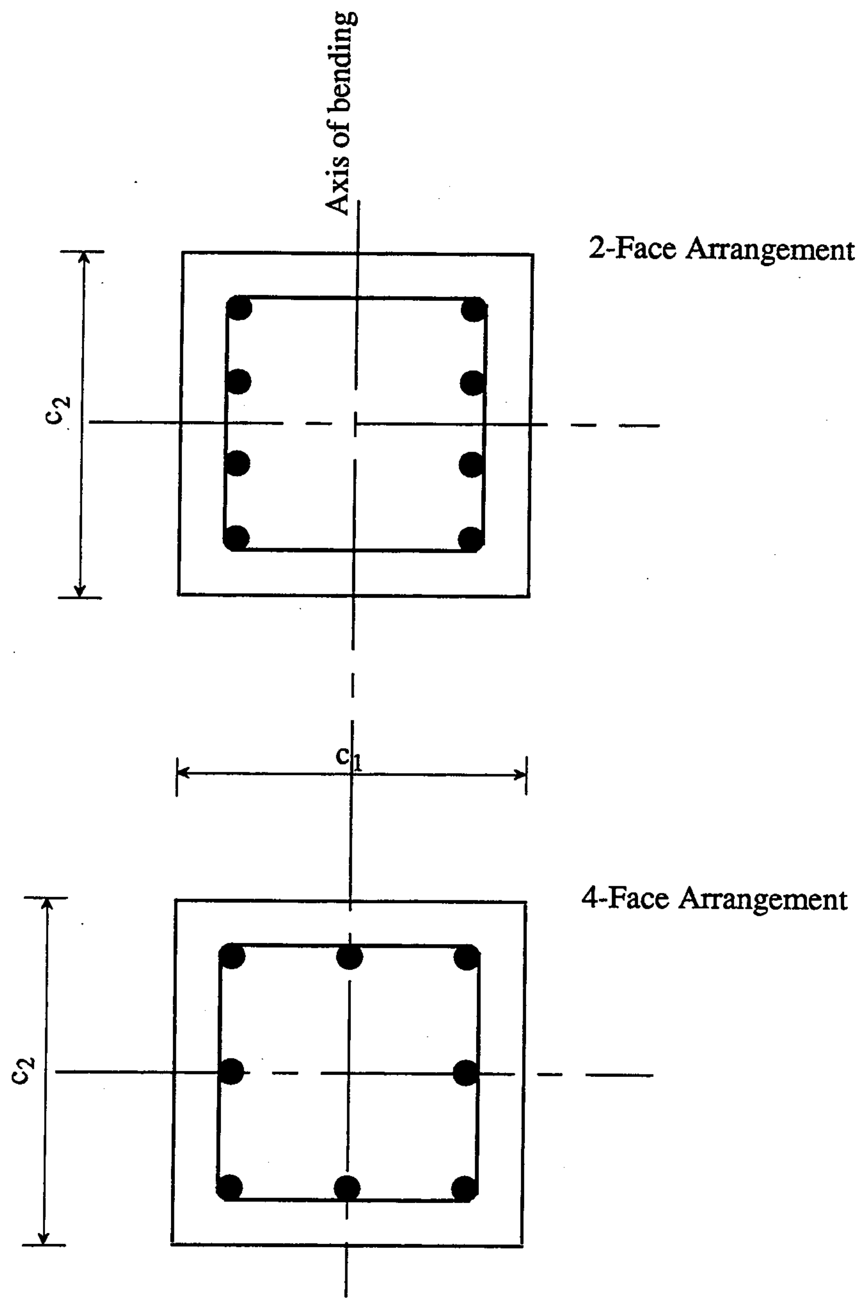

Fig. 6.9 Reinforcement arrangements for column example

7. Summary and Conclusions

The different aspects of modelling of reinforced concrete unbraced frames for design purposes were studied. To facilitate this study, a knowledge-based expert system (ESURF) was developed using the programming environment OPS83. The notion of expert systems as it pertains to structural design was discussed. Rule structure in OPS83 and its implementation in ESURF were also discussed.

Implementing the process of structural design in a computer system that incorporates design experience and adopts to job-specific constraints was explained. ESURF was presented as an expert system for structural design of reinforced concrete unbraced frames. Different aspects of design and problems encountered while developing the system were analyzed.

A few studies were conducted to evaluate the effect of modelling on the final design. Five design models and three test frames were considered to perform these studies. Gravity loading was kept as constant while wind loading was varied.

The effect of modelling joint sizes was studied by comparing designs with implicit consideration of joint size effect (model A) and those with explicit consideration of joint size effect (model B) in first-order and second-order modes. The effect of modelling member effective stiffnesses as a ratio of their gross stiffnesses was studied by comparing designs with different sets of effective stiffnesses using model B2 to the reference design using model C2 where the CEB model was used to calculate member stiffnesses. The effect of some construction rules on the final design was studied. Some loading aspects such as load combinations and live load reduction factors were also studied.

Slenderness effect in unbraced frames was considered in an exact way by performing a second-order analysis and in an approximate way by using a first-order analysis with the moment magnifier method. A new expression for EI based on a non-linear regression analysis for effective stiffnesses evaluated using the CEB model was proposed and evaluated.

The main conclusions of this study can be summarized as follows:

1) Knowledge-based expert systems are an efficient way for automating structural design.
2) The programming environment OPS83 provides a suitable tool for building a structural design expert system.
3) The developed system ESURF generates acceptable designs considering strength, serviceability, and constructibility requirements.
4) The effect of joint size on the design of members is essentially the same when considered implicitly by reducing centreline moments to the face of the support (accounting for stiffened ends) or when considered explicitly by representing joint regions as separate stiff elements.

5) For second-order analysis, using an effective stiffness of half the gross stiffness for both beams and columns gives comparable results to analysis that accounts for the variable member stiffness due to varying reinforcement and degree of cracks using the CEB model.

6) Significant differences in bending moments in columns using different modelling techniques may result in no difference in column sections when constructibility requirements are to be satisfied.

7) The concrete dimension increment is a very significant construction parameter. Reducing the increment from 50 mm to 25 mm resulted in 4% saving in cost.

8) Saving in cost of up to 4% can be obtained by using second-order analysis to consider slenderness effect in unbraced frames compared to the moment magnifier method.

9) The expression for the effective flexural stiffness given by the CSA A23.3 to be used in the moment magnifier method is very conservative for columns with small eccentricity ratios.

10) A proposed expression for the effective flexural stiffness (as a function of section concrete dimensions, eccentricity ratio, and amount of reinforcement) when used with the moment magnifier method gives designs in good and consistent agreement with designs based on a second-order analysis.

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Appendix A: The CEB-FIP Model for Effective Stiffness

The CEB model is a procedure to predict the curvature (and hence the stiffness) of a reinforced concrete cracked section as an interpolation between two curvatures (Fig. A.1). The first curvature is that of the cross section when the tensile stress is less than the modulus of rupture of concrete and hence no cracking has occurred. This state is designated as state I. The second curvature is that of the cross section when the tensile stress is big enough to cause the concrete in the whole tensile zone to be fully-cracked. This state is designated as state II. The interpolation factor ζ is an estimate for the contribution of the concrete in the tensile zone to the rigidity of the cross section. It also takes into account the bond properties of the reinforcing bars and the effect of the applied load duration and repetition. The theoretical basis of the CEB model is explained by Ghali, 1987 where the formula for the mean strain in an axially loaded member was derived and applied to members loaded with combined axial load and bending moment.

The mean curvature ($1/r_m$) of a cross section subject to axial force N and moment M is calculated from the equation:

$$1/r_m = 1/r_2 - (1/r_{2r} - 1/r_{1r}) \cdot \beta_b \cdot (M_r/M) \qquad (A.1)$$

where

$1/r_1$	is the curvature in state I corresponding to M, N
$1/r_{1r}$	is the curvature in state I corresponding to M_r, N
$1/r_2$	is the curvature in state II corresponding to M, N

$1/r_{2r}$	is the curvature in state II corresponding to M_r, N
M_r	is the cracking moment = $W_1 \cdot (f_{ct} - N/A_1)$
f_{ct}	is the concrete modulus of rupture
W_1	is the section modulus in state I (including steel)
A_1	is the section area in state I (including steel)
β_b	$= \beta_1 \cdot \beta_2$
β_1	is a coefficient for bond properties of reinforcing bars = 1.0 for high bond bars = 0.5 for plain bars
β_2	is a coefficient to account for the duration of application or the repetition of loading = 1.0 for first loading = 0.5 for sustained or repeated loading

In the CEB-FIP model code 1990, the value of β_2 for first loading has been changed from 1.0 to 0.8. However, this change is not believed to have any impact on the conclusions drawn in this study.

Eqn. (A.1) can be simplified to:

$$1/r_m = 1/r_2 - (1/r_2 - 1/r_1) \cdot \beta_b \cdot (M_r/M)^2$$
$$1/r_m = 1/r_1 (\beta_b \cdot (M_r/M)^2) - 1/r_2 (\beta_b \cdot (M_r/M)^2 - 1)$$

Substituting with $\zeta = 1 - \beta_b \cdot (M_r/M)^2$ we get:

$$1/r_m = 1/r_1 (1 - \zeta) - 1/r_2 (\zeta) \qquad \text{(A.2)}$$

Substituting in Eqn. (A.2) with the moment-curvature relation

$$1/r = M/EI \tag{A.3}$$

we get:

$$M/EI_m = (M_1/EI_1)(1 - \zeta) - (M_2/EI_2)(\zeta) \tag{A.4}$$

where

EI_m	is the mean stiffness
EI_1	is the stiffness of the transformed section in state I
EI_2	is the stiffness of the transformed section in state II
M_1	is the applied moment M
M_2	is the equivalent applied moment with the axial load being acting in the centroid of the transformed section in state II

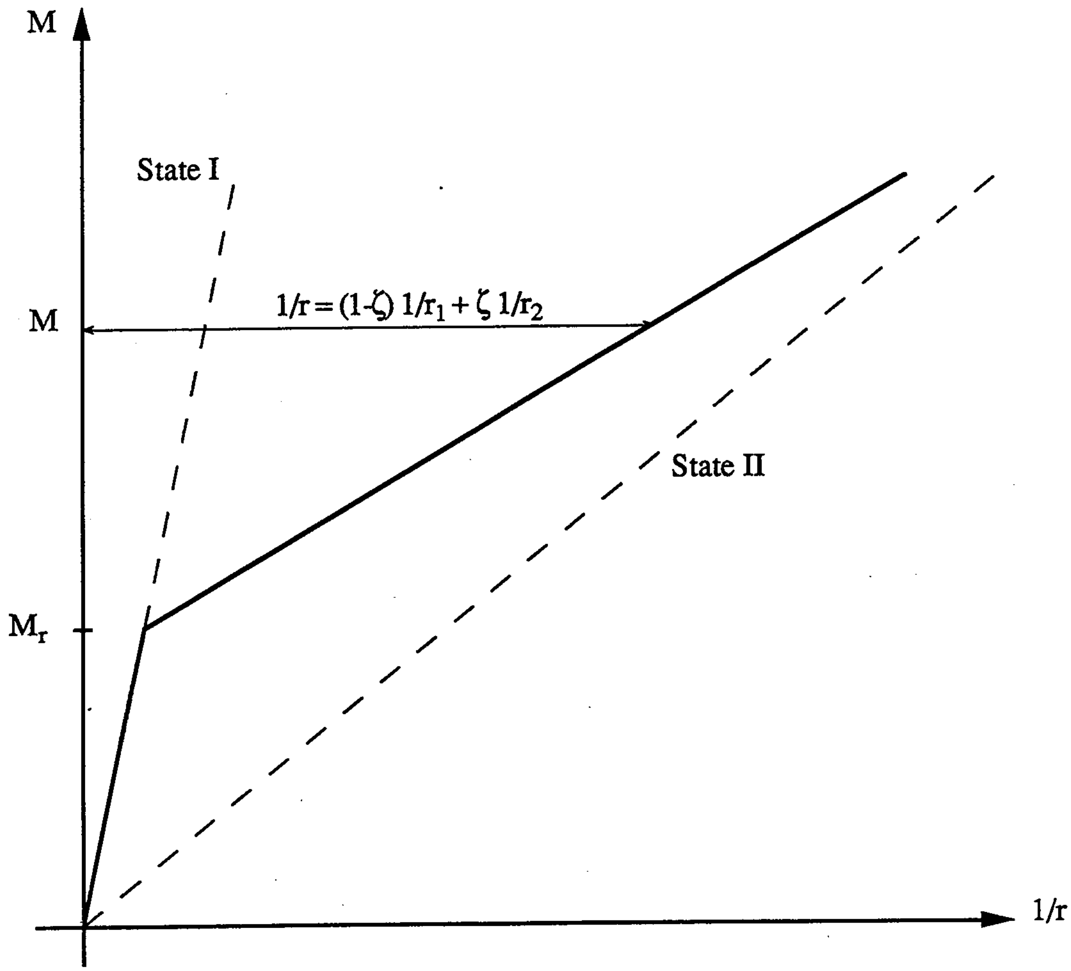

Fig. A.1 Moment-curvature diagram

Appendix B: Moment Magnifier Method

Based on the requirements of CSA A23.3 design code, the moment magnifier method as implemented in ESURF includes the following calculations:

1. For the frame in question, calculate first-order moments in members due to factored loads that produce non-appreciable sway, namely gravity loads, (M_{ns}) and those due to factored loads that produce appreciable sway, namely lateral loads, (M_s). Also calculate axial loads in columns due to total factored loads (P_f).

2. For each column, calculate the critical load from the equation:

$$P_c = \frac{\pi^2 EI}{(kl_u)^2} \tag{B.1}$$

where

$$EI = \frac{0.2\,E_c I_g + E_s I_{se}}{1+\beta_d} \tag{B.2}$$

or conservatively

$$EI = 0.25\,E_c I_g \tag{B.3}$$

and I_{se} is the moment of inertia of reinforcement about centroidal axis of the column cross section and β_d is the absolute value of the ratio of maximum factored dead load moment to the total factored load moment (taken to be 0.0 for transient wind load).

The effective length factor k can be determined from:

$$k = \frac{20 - \psi_m}{20}\sqrt{1 + \psi_m} \qquad \text{for } \psi_m < 2.0 \tag{B.4a}$$

$$k = 0.9\sqrt{1 + \psi_m} \qquad \text{for } \psi_m \geq 2.0 \tag{B.4b}$$

where ψ_m is the average of the ψ values at the two column ends and ψ is the ratio of the sum of stiffnesses of compression elements to the sum of stiffnesses of flexural elements at one column end.

3. For each storey, calculate the sway magnifier δ_s from the equation:

$$\delta_s = \frac{1}{1 - \dfrac{\Sigma P_f}{\phi_m \Sigma P_c}} \tag{B.5}$$

where ϕ_m is a member resistance factor taken to be 0.65.

4. The end moments M_1 and M_2 for each column are calculated as follows:

$$M_1 = M_{1ns} + \delta_s\,(M_{1s}) \tag{B.6a}$$

$$M_2 = M_{2ns} + \delta_s\,(M_{2s}) \tag{B.6b}$$

5. The maximum moment for each column is calculated as:

$$M_{max} = \delta_b\, M_2 \tag{B.7}$$

where

$$\delta_b = \frac{C_m}{1 - \dfrac{P_f}{\phi_m P_c}} \tag{B.8}$$

and

$$C_m = 0.6 + 0.4 \frac{M_1}{M_2} \geq 0.4 \tag{B.9}$$

The term M_1/M_2 is considered to be positive for single curvature cases and negative for double curvature cases.

6. Negative moments in beams are also magnified by calculating a magnification factor δ_{sb} at each joint. The factor is calculated by assuming that the ratio between the sum of column moments and the sum of beam moments at a joint is the same before and after the magnification. Thus for a typical joint with two beams (left and right) and two column (upper and lower), the sway magnification factor for the beam moments at this joint is determined from the relationship:

$$\frac{M_{cu} + M_{cl}}{M_{bl} + M_{br}} = \frac{\delta_{su} M_{cu} + \delta_{sl} M_{cl}}{\delta_{sb} (M_{bl} + M_{br})} \tag{B.10}$$

where the subscripts c denotes column, b denotes beam, u denotes upper, r denotes right, and l denotes lower for column and left for beam.

From Eqn. (B.10), the value of the sway magnifier for beams δ_{sb} can be expressed as:

$$\delta_{sb} = \frac{\delta_{su} M_{cu} + \delta_{sl} M_{cl}}{M_{cu} + M_{cl}} \tag{B.11}$$

The beam negative moments at each joint are then calculated from the equation:

$$M_b = M_{bns} + \delta_{sb} M_{bs} \tag{B.12}$$

where M_b is the magnified beam negative moment, M_{bns} is the beam negative moment due to factored loads that produce non-appreciable sway and M_{bs} is the beam negative moment due to factored loads that produce appreciable sway.

Appendix C: Rule Examples

The following two rules are samples to demonstrate how rules are cast in and manipulated by OPS83. The first rule is related to the proportioning stage in which concrete dimensions of all members are selected to satisfy strength requirements and any other applicable constraints. The second rule is related to the detailed design stage where longitudinal and transverse reinforcement for all members are chosen and strength adequacy is checked to satisfy applicable requirements. Each rule is written in all-English form then followed by OPS83 format.

Example rule (1)

rule reduce reinforcement for interior column

IF

stage is proportioning interior column

& storey is above first

& f'_c is determined

THEN

calculate ρ_{req}

IF (ρ_{req} is greater than ρ_{min}) OR (MINS in not satisfied)

keep the same cross section

use ρ_{req}

move to the next storey

ELSE

consider reducing column dimensions

(where MINS refers to the minimum number of stories to have the same column dimensions and ρ_{req} is the required reinforcement ratio for strength).

The OPS83 format of this rule is:

```
rule  reduce_rhoi
{     &1 (goal stage=intcol);
      &2 (icolst (&2.storey > 1 ∧ &2.status=old));
      &3 (fpce currfpc <> 0.0);
-->
      &sto=&2.storey;
      &agt=&nhint[&sto-1]*&nbint[&sto-1];
      &a=maxi2(&mcu[&sto-1],&mcl[&sto-1]);
      &rhoi[&sto]=getrhoc(&p[&sto],&a,&nhint[&sto-1],&nbint[&sto-1]
      ,&3.currfpc,interior);
      if (&rhoi[&sto] >  &rhomin ∨ (&sto-&last) <= &mins)
      {     &nhint[&sto]=&nhint[&sto-1];
            &nbint[&sto]=&nbint[&sto-1];
            modify &2 (storey=@.storey+1);
      }
      else
      modify &2 (status=new);
};
```

This rule is one of many rules that control the design of column stacks. The rule is to decide whether to keep the concrete dimensions of an interior column in a storey the same as those of the column in the storey below it or to reduce the concrete dimensions (by making another rule satisfied). The LHS of the rule will be matched against the current content of working memory. The first pattern is matched against an element *goal* in working memory with the attribute *stage* taking the symbolic value 'intcol'. If successful, the element *goal* is bound to the variable &1 for further reference to that element within the same rule. The second pattern is then matched against an element *icolst* (interior column stack) with its attributes *status* being 'old' and *storey* being greater than 1. If successful, the element *cstack* is bound to the variable &2. The third pattern is matched against an element *fpce* with its attribute *currfpc* being not equal to zero (i.e. already decided by another rule). The rule is then declared to be satisfied and the RHS is executed. The required reinforcement is then calculated and if greater than the minimum reinforcement or MINS is not satisfied yet, the same column dimensions (as the storey below) are assigned to the column in that storey with the reinforcement ratio ρ_{req}. The current storey is then incremented by one to consider the next storey (if any). If not, the system will consider reducing the concrete dimensions by invoking the appropriate rule.

<u>**Example rule (2)**</u>

rule choose exterior column reinforcement
IF

stage is column reinforcement

& column is exterior

& column reinforcement is to be determined

THEN

calculate required reinforcement index

choose column reinforcement from table

store data of chosen reinforcement for further evaluation

calculate actual reinforcement ratio

activate the rule to evaluate the chosen reinforcement

The rule is written in OPS83 format as follows:

```
rule   get_extc_reinforcement
{      &1 (goal stage=creinf);
       &2 (cscan column=exterior; cstorey>=1);
       &3 (colrein extrein=get);
-->
       &ij=&2.cstorey;
       &ia=rounbrc(&rhox[&ij]*&nhext[&ij]*&nbext[&ij]*10e6,2);
       &noc=&nocx1[&ia];            &ndc=&ndcx1[&ia];
       &nbigc=&nbigcx1[&ia];        &dbigc=&bdiam[&brank];
       &ascx[&ij]=&charcx1[&ia];    &iacx[&ij]=&ia;
       &rhox[&ij]=200.0*reali(&ia)/(&nhext[&ij]*&nbext[&ij]*10e6);
       modify &3 (extrein=evaluate);
};
```

This rule is satisfied when the current *stage* of the *goal* element is 'creinf' (column reinforcement), the *column* of the *cscan* element is 'exterior', and the *extrein* (exterior reinforcement) of the *colrein* element is 'get'. The RHS includes calculating the required reinforcement ratio and accordingly choosing actual reinforcement by invoking the appropriate procedures. A subgoal is set to evaluate this reinforcement by modifying the *extrein* attribute to 'evaluate' which makes the LHS of another rule satisfied.

Appendix D: Sample Output

Output from ESURF for the design of frame 3B4S under moderate wind intensity (W1) and using design model B2 is listed here.

PROBLEM INFORMATION

****

Problem ID : 3Bay_4Storey.W1

Modelling method : MB

Analysis Type : Second_Order

Run Mode : Design

Design Code : CSA

Min. No. of Storeys : 3 (per col. x-sec.)

Dimension Incr. : 50 mm

Load Case : 1.25*D + 1.05*L + 1.05*W

COLUMNS DETAILED DESIGN RESULTS

****

FLOOR NO. 1 Lat. drift= 15.32 mm

Exterior Column :

C1=.35 m C2=.25 m fpc= 40 MPa

Reinf. =|4#20| (0.0137)

Pmax= 918.18 kN Mcor= 112.00 kN.m Check=OK

Interior Column :

C1=.3 m C2=.3 m fpc= 40 MPa

Reinf. =|4#25| (0.0222)

Pmax=2054.27 kN Mcor= 47.73 kN.m Check=OK

FLOOR NO. 2 Lat. drift= 21.32 mm

Exterior Column :

C1=.35 m C2=.25 m fpc= 40 MPa

Reinf. =|8#20| (0.0274)

Pmax= 685.32 kN Mcor= 157.57 kN.m Check=OK

Interior Column :

C1=.3 m C2=.3 m fpc= 40 MPa

Reinf. =|4#20| (0.0133)

Pmax=1535.22 kN Mcor= 43.47 kN.m Check=OK

FLOOR NO. 3 Lat. drift= 24.28 mm

Exterior Column :

C1=.35 m C2=.25 m fpc= 40 MPa

Reinf. =|4#25| (0.0229)

Pmax= 451.33 kN Mcor= 125.77 kN.m Check=OK

Interior Column :

C1=.3 m C2=.3 m fpc= 40 MPa

Reinf. =|4#20| (0.0133)

Pmax=1025.85 kN Mcor= 21.72 kN.m Check=OK

FLOOR NO. 4 Lat. drift= 25.67 mm

Exterior Column :

C1=.35 m C2=.25 m fpc= 40 MPa

Reinf. =|4#35| (0.0457)

Pmax= 218.45 kN Mcor= 179.13 kN.m Check=OK

Interior Column :

C1=.3 m C2=.3 m fpc= 40 MPa

Reinf. =|4#20| (0.0133)

Pmax= 518.90 kN Mcor= 30.82 kN.m Check=OK

BEAMS DETAILED DESIGN RESULTS

****

FLOOR NO. 1 :

Span no. 1

b=.35 m & h=.6 m

Ml=151.091 kN.m Mf=260.603 kN.m Mr=-476.713 kN.m

Rl=|2#25+1#30| (0.009) Rf=|2#25+2#20| (0.009)

Rr=|2#35+2#30| (0.018)

Sl= 11 @ 275 Sr= 5 @ 150 15 @ 275

Span no. 2

b=.35 m & h=.6 m

Ml=381.461 kN.m Mf=171.856 kN.m Mr=-429.782 kN.m

Rl=|2#35+2#30| (0.018) Rf=|2#20+1#25| (0.006)

Rr=|2#35+2#30| (0.018)

Sl= 12 @ 275 Sr= 12 @ 275

Span no. 3

b=.35 m & h=.6 m

Ml=394.486 kN.m Mf=260.603 kN.m Mr=-265.124 kN.m

Rl=|2#35+2#30| (0.018) Rf=|2#25+2#20| (0.009)

Rr=|2#25+1#30| (0.009)

Sl= 5 @ 150 15 @ 275 Sr= 11 @ 275

FLOOR NO. 2 :

\-\-\-\-\-\-\-\-\-\-\-\-\-\-

Span no. 1

b=.35 m & h=.6 m

Ml=229.582 kN.m Mf=236.31 kN.m Mr=-446.808 kN.m

Rl=|6#20| (0.010) Rf=|3#25| (0.008) Rr=|3#35| (0.016)

Sl= 11 @ 275 Sr= 13 @ 275

Span no. 2

b=.35 m & h=.6 m

Ml=378.001 kN.m Mf=182.587 kN.m Mr=-411.779 kN.m

Rl=|3#35| (0.0160) Rf=|2#20+1#25| (0.0059) Rr=|3#35| (0.016)

Sl= 12 @ 275 Sr= 12 @ 275

Span no. 3

b=.35 m & h=.6 m

Ml=402.298 kN.m Mf=236.31 kN.m Mr=-283.835 kN.m

Rl=|3#35| (0.0160) Rf=|3#25| (0.0080) Rr=|6#20| (0.0096)

Sl= 13 @ 275 Sr= 11 @ 275

FLOOR NO. 3 :

Span no. 1

b=.35 m & h=.6 m

Ml=256.593 kN.m Mf=230.061 kN.m Mr=-432.293 kN.m

Rl=|6#20| (0.010) Rf=|2#30| (0.007) Rr=|3#25+2#30| (0.015)

Sl= 11 @ 275 Sr= 13 @ 275

Span no. 2

b=.35 m & h=.6 m

Ml=383.315 kN.m Mf=186.451 kN.m Mr=-398.737 kN.m

Rl=|3#25+2#30| (0.015) Rf=|2#20+1#25| (0.006)

Rr=|3#25+2#30| (0.015)

Sl= 12 @ 275 Sr= 12 @ 275

Span no. 3

b=.35 m & h=.6 m

Ml=411.492 kN.m Mf=230.061 kN.m Mr=-282.301 kN.m

Rl=|3#25+2#30| (0.015) Rf=|2#30| (0.007) Rr=|6#20| (0.010)

Sl= 13 @ 275 Sr= 11 @ 275

FLOOR NO. 4 :

Span no. 1

b=.35 m & h=.6 m

Ml=162.566 kN.m Mf=271.885 kN.m Mr=-442.672 kN.m

Rl=|2#25| (0.005) Rf=|2#25+1#30| (0.009) Rr=|3#35| (0.016)

Sl= 9 @ 275 Sr= 13 @ 275

Span no. 2

b=.35 m & h=.6 m

Ml=411.858 kN.m   Mf=163.357 kN.m   Mr=-416.383 kN.m

Rl=|3#35| (0.0160)   Rf=|2#25| (0.0053)   Rr=|3#35| (0.0160)

Sl= 11 @ 275   Sr= 11 @ 275

Span no. 3

b=.35 m & h=.6 m

Ml=437.214 kN.m   Mf=271.885 kN.m   Mr=-168.79 kN.m

Rl=|3#35| (0.016)   Rf=|2#25+1#30| (0.009)   Rr=|2#25| (0.005)

Sl= 13 @ 275   Sr= 9 @ 275

SUMMARY OF QUANTITIES

****

Concrete volume	=	30.54350 m^3
Steel mass	=	5269.35654 kg
Form sur. area	=	303.82000 m^2
Cost index	=	129.30971 Qu

Appendix E: Listing of ESURF Rules

Module 1

rule perform analysis

IF

stage is analysis

& run mode is design

THEN

prepare input for analysis

call analysis program

rule assign given member dimensions

IF

stage is analysis

& member dimensions are given

THEN

store member dimensions in corresponding arrays

prepare input for analysis

call analysis program

rule determine MINS

IF

stage is get MINS

THEN

IF number of storeys < 4	MINS = NST
ELSEIF number of storeys < 6	MINS = 2
ELSEIF number of storeys < 12	MINS = 3
ELSE	MINS = 4

<u>rule choose f'_c for columns</u>

IF

stage is proportioning interior column

& storey is first

& f'_c is not determined

& column stack is not given

THEN

proportion interior column using maximum f'_c

IF (minimum f'c not equal maximum f'_c)

proportion interior column using minimum f'_c

IF (there is a saving in concrete volume)

use minimum f'_c

ELSE

use maximum f'_c

<u>rule choose f'_c for a given column stack</u>

IF

stage is proportioning interior column

& storey is first

& f'_c is not determined

& column stack is given

THEN

calculate ρ_{req} using minimum f'_c

IF (ρ_{req} is less than ρ_{maxu})

use minimum f'_c

ELSEIF (medium f'_c not equal minimum f'_c)

calculate ρ_{req} using medium f'_c

IF (ρ_{req} is less than ρ_{maxu})

use medium f'_c

ELSE

check maximum f'_c

rule check maximum f'_c for given column stacks

IF

stage is proportioning interior column

& storey is first

& f'_c is not determined and maximum f'_c is to be checked

& column stack is given

THEN

calculate ρ_{req} using maximum f'_c

IF (ρ_{req} is less than ρ_{maxu})

column check is no good

rule dimension interior bottom column

IF

stage is proportioning interior column

& storey is first

& f'_c is determined

& column stack is not given

THEN

store column concrete dimensions corresponding to chosen f'c

calculate ρ_{req}

rule assign given interior column stack

IF

stage is proportioning interior column

& column stack is given

THEN

store column dimensions in corresponding arrays

rule same dimensions for whole interior column stack

IF

stage is proportioning interior column

& storey is above first

& same column dimensions for whole stack

THEN

assign bottom interior column dimensions for columns in all storeys

rule use maximum f'_c

IF

stage is proportioning interior column

& storey is above first

& maximum f'_c is to be considered

& beam-column intersection is not a problem

& column stack is not given

THEN

calculate ρ_{req} using maximum f'_c

IF (ρ_{req} is greater than ρ_{min})

use maximum f'_c

move to the next storey

ELSE

consider using medium f'_c

rule use medium f'_c

IF

stage is proportioning interior column

& storey is above first

& medium f'_c is to be considered

& beam-column intersection is not a problem

& column stack is not given

THEN

calculate ρ_{req} using minimum f'_c

IF (ρ_{req} is less than ρ_{maxu})

consider using minimum f'_c

ELSE

calculate ρ_{req} using medium f'_c

IF (ρ_{req} is greater than ρ_{min})

use medium f'_c

move to the next storey

ELSE

consider using minimum f'_c

rule use minimum f'_c

IF

stage is proportioning interior column

& storey is above first

& minimum f'_c is to be considered

& beam-column intersection is not a problem

& column stack is not given

THEN

calculate ρ_{req} using minimum f'_c

IF (ρ_{req} is greater than ρ_{min} OR MINS is not satisfied)

use minimum f'_c

move to the next storey

ELSE

consider reducing column concrete dimensions

rule use maximum f'_c for given column stacks

IF

stage is choosing f'_c

& storey is above first

& maximum f'_c is to be considered

& beam-column intersection is not a problem

& column stack is given

THEN

calculate ρ_{req} using maximum f'_c

IF (ρ_{req} is greater than ρ_{min})

use maximum f'_c

move to the next storey

ELSE

consider using medium f'_c

rule use medium f'_c for given column stacks

IF

stage is choosing f'_c

& storey is above first

& medium f'_c is to be considered

& beam-column intersection is not a problem

& column stack is given

THEN

calculate ρ_{req} using medium f'_c

IF (ρ_{req} is greater than ρ_{min})

use medium f'_c

move to the next storey

ELSE

consider using minimum f'_c

rule use minimum f'_c for given column stacks

IF

stage is choosing f'_c

& storey is above first

& minimum f'_c is to be considered

& beam-column intersection is not a problem

& column stack is given

THEN

calculate ρ_{req} using minimum f'_c

use maximum f'_c

move to the next storey

rule consider reducing interior column dimensions

IF

stage is proportioning interior column

& storey is above first AND MINS is satisfied

& reducing f'_c has been considered

THEN

IF (column cross-section is square)

reduce each column dimension by two increments

calculate ρ_{req}

IF (ρ_{req} is less than ρ_{maxu} AND the reduced column width satisfies beam-column intersection requirements)

use the reduced column dimensions

ELSE

reduce each column dimension by one increment

IF (the reduced column width satisfies beam-column intersection requirements)

use the reduced column dimensions

ELSEIF (difference between column dimensions > 2 increments)

reduce depth by 2 increments

calculate ρ_{req}

IF (ρ_{req} is less than ρ_{maxu})

use the reduced column dimensions

ELSE

reduce depth by 2 increments and width by an increment

calculate ρ_{req}

IF (ρ_{req} is less than ρ_{maxu} AND the reduced column width satisfies beam-column intersection requirements) use the reduced column dimensions

IF (all attempts to reduce column dimensions fail)

use the same column dimensions

move to the next storey

rule reduce reinforcement for interior column

IF

stage is proportioning interior column

& storey is above first

& f'_c is determined

THEN

calculate ρ_{req}

IF (ρ_{maxu} is greater than ρ_{min} OR MINS is not satisfied)

keep the same column dimensions

use ρ_{req}

move to the next storey

ELSE

consider reducing column dimensions

rule complete interior column stack

IF

stage is proportioning interior column stack

& remaining number of storeys is less than MINS

THEN

use the last column dimensions for the remaining storeys

calculate ρ_{req} for each storey

rule dimension bottom exterior column

IF

stage is proportioning exterior column

& storey is first

& column stack is not given

THEN

call routine to choose column dimensions

calculate ρ_{req}

rule same dimensions for the whole exterior column stack

IF

stage is proportioning exterior columns

storey is above first

same column dimensions for the whole stack

THEN

assign bottom exterior column dimensions to columns in upper storeys

rule reduce reinforcement for exterior column

IF

stage is proportioning exterior column

& storey is above first

& f'_c is determined

THEN

calculate ρ_{req}

IF (ρ_{req} is greater than ρ_{min} OR MINS is not satisfied)

keep the same column dimensions

use ρ_{req}

move to the next storey

ELSE

consider reducing column dimensions

rule same exterior-interior column stack

IF

stage is proportioning exterior column

& same exterior column stack as the interior one

& f'_c is determined

THEN

assign interior column dimensions to exterior column

calculate ρ_{req} in each storey

rule reduce exterior column dimensions

IF

stage is proportioning exterior column

& storey is above first AND MINS is satisfied

& f'_c is determined

THEN

reduce column depth by one increment

calculate ρ_{req}

IF (ρ_{req} is less than ρ_{maxu})

use the reduced column depth

ELSE

use the unreduced column depth

move to the next storey

rule complete exterior column stack

IF

stage is proportioning exterior column stack

& remaining number of storeys is less than MINS

THEN

use the last column dimensions for the remaining storeys

calculate ρ_{req} for each storey

rule get beam design moments

IF

stage is proportioning beams

design moments are not determined

THEN

determine design moment in each span

determine design moment in each storey

determine design moment in the frame

rule determine number of beam groups in the frame

IF

stage is proportioning beams

& beam groups are not determined

THEN

compare each storey design moment to frame design moment

IF (difference in design moments > 40%)

define a new beam group

rule determine number of beam cross sections in each beam group

IF

stage is proportioning beams

& beam groups are determined

& no. of cross sections in each beam group is to be determined

THEN

Compare design moment in each span to group design moment

IF (difference in design moment > 40%)

reduce depth of beam

rule proportion bottom storey beam

IF

stage is proportioning beams

& beam groups are determined

& bottom storey beam dimensions are to be determined

THEN

IF (neither depth nor width is specified)

call routine to determine beam depth and width

ELSEIF (only depth is specified)

call routine to determine beam width

ELSEIF (only width is specified)

call routine to determine beam depth

ELSE

calculate ρ_{req} for specified width and depth

<u>*rule proportion bottom storey beam given maximum depth*</u>

IF

    stage is proportioning beams
&   beam groups are determined
&   maximum beam depth is specified

THEN

```
IF (beam width is specified)
    call routine to calculate beam depth
    IF (calculated beam depth > maximum beam depth)
        use maximum beam depth
        calculate ρ_req for specified width and depth
ELSE
    call routine to calculate beam width and depth
    IF (calculated beam depth > maximum beam depth)
        use maximum beam depth
        call routine to calculate beam width
```

<u>*rule check beam-column intersection*</u>

IF

    stage is beam-column intersection
&   beam and column widths must not be equal

THEN

```
IF (beam width = exterior column width)
    declare conflict in beam-exterior column
ELSEIF (beam width = interior column width)
    declare conflict in beam-interior column
ELSE
```

no conflict is detected

rule resolve beam-exterior column width conflict

IF

stage is beam-column intersection

& beam-exterior column width conflict is declared

THEN

IF (beam depth < maximum beam depth)

decrease beam width by one increment

call routine to calculate beam depth

ELSE

increase beam width by one increment

call routine to calculate beam depth

rule resolve beam-exterior column width conflict for a specified beam width

IF

stage is beam-column intersection

& beam-exterior column width conflict is declared

& beam width is specified

THEN

decrease exterior column width by one increment

call routine to calculate exterior column depth

rule resolve beam-interior column width conflict for maximum f'_c

IF

stage is beam-column intersection

& beam-interior column width conflict is declared

& maximum f'_c was used

THEN

call routine to dimension interior column using minimum f'_c

rule resolve beam-interior column width conflict for minimum f'_c

IF

stage is beam-column intersection

& beam-interior column width conflict is declared

& maximum f'_c was not used

THEN

IF (beam depth < maximum beam depth)

decrease beam width by one increment

call routine to calculate beam depth

ELSE

increase beam width by one increment

call routine to calculate beam depth

rule resolve beam-interior column width conflict for specified beam width

IF

stage is beam-column intersection

& beam-interior column width conflict is declared

& beam width is specified

THEN

increase interior column width and depth by one increment

calculate ρ_{req}

rule dimension beam groups

IF

stage is proportioning beams

& beam groups are determined

& beam-column width conflict is resolved

THEN

call routine to dimension each beam group

rule reduce beam depth in beam groups

IF

stage is proportioning beams

& beam groups are determined

& beam-column width conflict is resolved

& beam groups are dimensioned

THEN

call routine to reduce beam depth if indicative

rule assign beam group dimensions to individual beams

IF

stage is proportioning beams

& beam groups are determined

& beam-column width conflict is resolved

& beam groups are dimensioned

& beam depths are reduced in beam groups (if indicative)

THEN

assign beam group dimensions for each individual beam *rule iterate analysis-proportioning cycle*

IF

stage is iterate

THEN

call check routine to compare member dimensions of the current and previous iteration

IF (check is not OK)

repeat analysis-proportioning cycle

ELSE

return to main menu

Module 2

rule determine shear stress design values

IF

stage is shear design

& shear stress design values are to be determined

THEN

calculate four shear stress values for each span to determine the shear envelopes

rule determine shear zone

IF

stage is shear design

& shear stress design values are determined

& shear zones are to be determined

THEN

calculate limiting shear stress values for different shear zones
compare maximum shear stress in a span to limiting shear
stress values and hence determine the shear zone

<u>rule no stirrups needed</u>

IF

stage is shear design

& shear stress design values are determined

& shear zone is no stirrups needed

THEN

move to the next span

<u>rule heavy stirrups at left side</u>

IF

stage is shear design

& shear stress design values are determined

& shear zone is heavy stirrups AND beam side is left

THEN

call routine to calculate distance after which heavy stirrups are
no longer required (i.e. normal stirrups take over)
calculate number and spacing of stirrups to cover this distance

<u>rule normal stirrups at left side</u>

IF

stage is shear design

& shear stress design values are determined

& shear zone is normal stirrups AND beam side is left

THEN

call routine to calculate distance after which normal stirrups are no longer required (i.e. minimum stirrups take over)

calculate number and spacing of stirrups to cover this distance

rule minimum stirrups at left side

IF

stage is shear design

& shear stress design values are determined

& shear zone is minimum stirrups AND beam side is left

THEN

call routine to calculate distance after which minimum stirrups are no longer required

calculate number and spacing of stirrups to cover this distance

rule heavy stirrups at right side

IF

stage is shear design

& shear stress design values are determined

& shear zone is heavy stirrups AND beam side is right

THEN

call routine to calculate distance after which heavy stirrups are no longer required (i.e. normal stirrups take over)

calculate number and spacing of stirrups to cover this distance

rule normal stirrups at right side

IF

stage is shear design

& shear stress design values are determined

& shear zone is normal stirrups AND beam side is right

THEN

call routine to calculate distance after which normal stirrups are no longer required (i.e. minimum stirrups take over)

calculate number and spacing of stirrups to cover this distance

rule minimum stirrups at right side

IF

stage is shear design

& shear stress design values are determined

& shear zone is minimum stirrups AND beam side is right

THEN

call routine to calculate distance after which minimum stirrups are no longer required

calculate number and spacing of stirrups to cover this distance

Module 3

rule get negative design moments in beams

IF

stage is selecting beam reinforcement

& negative design moments are to be determined

THEN

determine the negative design moment at each joint

IF (joint is interior)

design moment of joint is the maximum of left and right beam negative moments

ELSE

design moment of joint is negative moment in beam

rule get positive design moments in beams

IF

stage is selecting beam reinforcement

& negative design moments are determined

& positive design moments are to be determined

THEN

calculate positive moment in each span knowing the two negative moments and loading condition of this span

rule consider symmetry

IF

stage is selecting beam reinforcement

& negative and positive design moments are determined

& frame geometry and loading are symmetric

THEN

compare moments in each location to the moment in the corresponding location at the other side of the frame centreline and assign the larger moment to both locations

rule choose negative reinforcement

IF

stage is selecting beam reinforcement

& negative and positive design moments are determined

& negative reinforcement is to be determined

THEN

calculate required reinforcement index
choose beam negative reinforcement from table
store data of chosen reinforcement for further evaluation
calculate actual reinforcement ratio
activate the rule to evaluate the chosen reinforcement

rule evaluate chosen negative reinforcement

IF

stage is selecting beam reinforcement
& negative and positive design moments are determined
& chosen negative reinforcement is to be evaluated

THEN

check strength adequacy
check bar spacing
IF (all check OK)
move to the next member
ELSE
choose next reinforcement in table

rule choose positive reinforcement

IF

stage is selecting beam reinforcement
& negative and positive design moments are determined
& positive reinforcement is to be determined

THEN

calculate required reinforcement index
choose beam positive reinforcement from table

store data of chosen reinforcement for further evaluation
calculate actual reinforcement ratio
activate the rule to evaluate the chosen reinforcement

rule evaluate chosen positive reinforcement

IF

stage is selecting beam reinforcement
& negative and positive design moments are determined
& chosen positive reinforcement is to be evaluated

THEN

check strength adequacy
check bar spacings
check crack width requirement
IF (all check OK)
move to the next member
ELSE
choose next reinforcement in table

Module 4

rule get design moments for interior columns

IF

stage is column reinforcement
& column is interior
& design moments are to be determined

THEN

IF (design is second-order)

calculate maximum moment in column using Eqn. (4.1)

ELSE

use maximum moment from moment magnifier method

rule get design moments for exterior columns

IF

stage is column reinforcement

& column is exterior

& design moments are to be determined

THEN

IF (design is second-order)

calculate maximum moment in column using Eqn. (4.1)

ELSE

use maximum moment from moment magnifier method

rule choose interior column reinforcement

IF

stage is column reinforcement

& column is interior

& column reinforcement is to be determined

THEN

calculate required reinforcement index

choose interior column reinforcement from table

store data of chosen reinforcement for further evaluation

calculate actual reinforcement ratio

activate the rule to evaluate the chosen reinforcement *rule evaluate chosen reinforcement for interior column*

IF

stage is column reinforcement

& column is interior

& column reinforcement is to be evaluated

THEN

check minimum and maximum bar spacings

IF (bar spacings are OK)

strength adequacy is to be checked

ELSE

try next reinforcement choice in table

rule check strength adequacy for interior column

IF

stage is column reinforcement

& column is interior

& strength adequacy is to be checked

THEN

call routine to check column strength

IF (strength check is OK)

move to the next member

ELSE

try next reinforcement in table

rule choose exterior column reinforcement

IF

stage is column reinforcement

& column is exterior

& column reinforcement is to be determined

THEN

calculate required reinforcement index

choose exterior column reinforcement from table

store data of chosen reinforcement for further evaluation

calculate actual reinforcement ratio

activate the rule to evaluate the chosen reinforcement

rule evaluate chosen reinforcement for exterior column

IF

stage is column reinforcement

& column is exterior

& column reinforcement is to be evaluated

THEN

check minimum and maximum bar spacings

IF (bar spacings are OK)

strength adequacy is to be checked

ELSE

try next reinforcement in table

rule check strength adequacy for exterior column

IF

stage is column reinforcement

& column is exterior

& strength adequacy is to be checked

THEN

call routine to check column strength

IF (strength check is OK)

move to the next member

ELSE

try next reinforcement in table

<u>Module 5</u>

<u>*rule input mode*</u>

IF

stage is input

& input mode is unknown

THEN

ask the user to enter the input mode (interactive or batch)

<u>*rule batch input mode*</u>

IF

stage is input

& input mode is batch

THEN

ask the user to enter old input filename

load the input file

activate main menu

<u>*rule interactive input mode*</u>

IF

stage is input

& input mode is interactive

THEN

ask the user to enter new input filename

activate main menu

<u>*rule display main menu*</u>

IF

stage is input

& main menu is to be activated

THEN

display main menu

ask the user to choose an item from menu

execute the chosen item

<u>*rule display column detailed design*</u>

IF

stage is input

& column detailed design menu is to be displayed

THEN

display column detailed design menu

ask the user to choose an item from menu

execute the chosen item

<u>*rule display beam detailed design*</u>

IF

stage is input

& beam detailed design menu is to be displayed

THEN

display beam detailed design menu
ask the user to choose an item from menu
execute the chosen item