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**Development of a System Architecture and
Applications for an Integrated Computer Software
System for the Analysis and Design of Steel
Structures**

by

Douglas Allan Raby

**A thesis submitted to the
Faculty of Graduate Studies and Research
in partial fulfilment of the requirements
for the degree of
Master of Engineering**

**Department of Civil and Environmental Engineering
Carleton University
Ottawa, Ontario
September, 1999**

**The Master of Engineering in the Civil and Environmental Engineering
is a joint program with the University of Ottawa,
administered by the Ottawa-Carleton Institute for Civil Engineering**

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Department of Civil and Environmental Engineering

**The Undersigned recommend to the Faculty of Graduate Studies
and Research acceptance of the thesis**

**Development of a System Architecture and Applications for an Integrated
Computer Software System for the Analysis and Design of Steel Structures**

**submitted by
Douglas Allan Raby, B.Eng.**

**in partial fulfilment of the requirements for
the degree of Master of Engineering**

**Dr. J.L. Humar, Department of Civil and Environmental
Engineering**

Dr. S. J. Kennedy, Thesis Supervisor

**Carleton University
Ottawa, Ontario, Canada**

September, 1999

Abstract

A series of research projects have been conducted to develop a prototype modular Integrated System for Steel Structures. The prototype modules reflect the technological evolution associated with the extended period of development, and are not readily integratable. A new object oriented system architecture has been proposed using the Java programming language. The new architecture would allow the system to be platform independent, expandable, and easily updateable in terms of computing and engineering technology. To validate the architecture and to demonstrate it's versatility, a prototype Object Oriented Handbook (OOH) and numerical method module have been developed.

The OOH represents clauses of CAN/CSA-S16.1 and additional tools with packages of formatted software classes, and tables with formatted databases. The provisions for the design of beam-columns within CAN/CSA-S16.1 were reviewed. The Standard permits the use of rational design and analysis based on theory and it was determined that an advanced numerical method for the analysis and design of beam-columns was required.

A prototype numerical method module has been developed within the framework of the system architecture. Four finite element classes were constructed, a triangular shell element and three beam-column elements for I-shaped, rectangular, and square hollow cross-sections. Models constructed using the I-shaped beam-column element were compared to test data and they were found to predict the inelastic behaviour of beam-column members and frames accurately.

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I will miss my friends and family, as I now depart to start my new life and career in Washington D.C. It is even harder to leave considering I have not seen most of you in months. When I return, you will know where to find me, at the end of my dock listening to the loons, waiting for you.

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Ian Fuller's Excellent Adventure has come to an end and "It's a good thing" (Stewart, Martha, 1996). We make a good team there are not too many people who can handle me in the morning as many times as you did and remain friends. Thanks for all the coffee, good luck, and you can stop sending me all those emails now. Cheers...

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List of Symbols

A	=	area, intermediate factored used in the determination of ω_1
B	=	intermediate factored used in the determination of ω_1
C	=	unfactored axial compressive force
C_{1,2,3,4}	=	constants of integration
C_e	=	Euler buckling strength $\left(\frac{\pi^2 EI}{L^2} \right)$
C_{ey}	=	Euler buckling strength considering only buckling about the y axis
C_f	=	factored axial compression
C_r	=	axial compressive resistance
C_{rx}	=	axial compressive resistance considering the x axis
C_{ry}	=	axial compressive resistance considering the y axis
C_T	=	GJ, torsional constant
C_w	=	warping torsional constant
C_y	=	axial compressive load at yield stress
d	=	depth of the cross section
E	=	elastic modulus of steel
E_{st}	=	strain hardening modulus
F_x	=	internal axial force
F_y	=	nominal yield stress
G	=	shear modulus of steel
G_{U,L}	=	dimensionless end restraint parameter for the upper and lower ends of the member
h	=	storey height, clear depth of web
I	=	moment of Inertia
I_w	=	warping torsional constant
I_x	=	moment of inertia about the x axis
I_y	=	moment of inertia about the y axis
J	=	St. Venant torsional constant
K	=	Effective length factor
k_v	=	$\pm \sqrt{P/EI}$
L	=	length
M	=	bending moment in a member
M₁	=	bending moment in a member at first quarter point
M₂	=	bending moment in a member at second quarter point
M₃	=	bending moment in a member at third quarter point
M_{Bx}	=	applied moment at point B about the x axis
M_{By}	=	applied moment at point B about the y axis
M_f	=	factored bending moment
M_{fg}	=	first-order moment under factored gravity loads

M_R	=	first-order translational moment under factored lateral loads, or moment resulting from lateral translation of an unsymmetrical frame, or the moment resulting from an unsymmetrically loaded frame under factored gravity loads
M_{fx}	=	factored moment about the x axis
M_{fy}	=	factored moment about the y axis
M_m	=	maximum bending moment in a member
M_o	=	maximum end moment
M_{ocr}	=	critical elastic moment
M_p	=	plastic moment capacity of the cross-section
M_{px}	=	plastic moment capacity of the cross-section about the x axis
M_{py}	=	plastic moment capacity of the cross-section about the y axis
M'_r	=	factored moment resistance of a member of a given unbraced length
M_{rx}	=	factored member bending resistance about the x axis
M_{ry}	=	factored member bending resistance about the y axis
M_{Tx}	=	applied moment at point T about the x axis
M_{Ty}	=	applied moment at point T about the y axis
M_u	=	critical elastic moment
M_x	=	bending moment about the x axis
M_y	=	bending moment about the y axis, yield moment
n	=	is a compressive resistance parameter that accounts for varying residual stress patterns and out-of-straightness associated with the fabrication of different sections
P	=	Axial force
P_θ	=	critical load for pure torsional buckling
P_e	=	Euler buckling load
P_y	=	Euler buckling load about the y axis
P_{yeild}	=	axial compressive load at yield
r_x	=	radius of gyration with respect to x axis
r_y	=	radius of gyration with respect to y axis
u	=	translational displacement with respect to shear centre
U_1	=	factor to account for moment gradient and for second-order effects of the axial force acting on the deformed member
U_{1x}	=	factor to account for moment gradient and for second-order effects of the axial force acting on the deformed member considering the x axis
U_{1y}	=	factor to account for moment gradient and for second-order effects of the axial force acting on the deformed member considering the y axis
U_2	=	amplification factor to account for second-order effects of gravity loads acting on the laterally displaced storey
v	=	translational displacement with respect to shear centre
x_o	=	the principle co-ordinate of shear centre with respect to the centroid of the cross-section
y	=	distance from centroid to a point on the cross-section

y_o	=	the principle co-ordinate of shear centre with respect to the centroid of the cross-section
z	=	position along longitudinal axis
$[EI_y]_{\text{eff}}$	=	effective stiffness about the y axis
$[K]$	=	structural stiffness matrix
$[K_g]$	=	geometric stiffness matrix
ΣC_f	=	sum of factored axial compressive loads of all columns in a storey
$\Sigma(I_o/L_c)$	=	summation of the ratio of moment of inertia about the weak axis to unsupported length columns framing into the joint
$\Sigma(I_g/I_g)$	=	summation of the ratio of moment of inertia to the unsupported length for girders framing into the joint
ΣV_f	=	sum of factored lateral loads above the story (first order storey shear)
$\bar{K}$	=	Wagner coefficient
$\bar{k}$	=	$-P\bar{r}_o^2 + M_x\beta_x + M_y\beta_y$
$\bar{r}_o$	=	$\pm\sqrt{x_o^2 + y_o^2 + r_x^2 + r_y^2}$
$\bar{z}$	=	location of maximum moment along z axis
λ	=	$\frac{KL}{r}\sqrt{\frac{F_y}{\pi^2 E}}$ (nondimensional slenderness parameter)
ψ	=	$\frac{1}{\cos k_y z}$
Δ	=	translational displacement
ϕ	=	resistance factor, rotational displacement with respect to the shear centre
κ	=	ratio of the smaller factored moment to the large factored moment at opposite ends of the unbraced length, positive for double curvature and negative for single
ω_1	=	coefficient to determine equivalent uniform bending effect
ω_{1x}	=	coefficient to determine equivalent uniform bending effect considering the x axis
ω_{1y}	=	coefficient to determine equivalent uniform bending effect considering the y axis
ω_2	=	coefficient to account for increased moment resistance of a laterally unsupported beam segment when subjected to a moment gradient
Δ_f	=	relative first order translational displacement of the storey due to factored loads
Δ_i	=	translational displacement of storey i
β_x	=	cross-sectional constant
β_y	=	cross-sectional constant
σ	=	stress
σ_y	=	yield stress

Chapter 1

Introduction

1.1 Integrated System Requirements

An integrated system for steel structures implies both the integration of all the steps required in the design process plus the development of a computing system architecture which has a common basis for describing the structure in each process (data structure). A series of research projects, summarized in Kennedy (1996), have been conducted to develop a prototype modular system for the layout, analysis, design, fabrication and erection of steel buildings and to investigate computing methods required to simulate the design process, including data requirements, manipulation and representation. These modules were designed to be integrated into a singular application to be entitled “An Integrated System for Steel Structures”, which would have the ability to be expanded to include modules yet to be developed.

As an integrated system the application should:

- Provide a common database for all users
- Emulate the design process so that the use of the application is natural to the user and reduces complexity
- Minimize the required input provided by the designer
- Provide a high quality graphical user interface

- Provide access to intermediate calculations through reports on design procedure or design checks to avoid the "black box design" phenomena
- Create an easily updateable system to reflect changes in computing technology and advances in standards and codes
- Consolidate the design modules into one "all inclusive" standardized environment

The complete integrated system should include the design process described in Table 1.1 and involves a series of professionals including architects, structural engineers, fabricators and erectors. The architect is responsible for the definition of the structure's general layout and form described in step 1- preprocessing.

Table 1.1: Description of the Design Process (after Martino (1997))

Design Step	Description
1 – Preprocessing	Definition of structural geometry, determination of loads and preliminary sizing of members
2- Structural Analysis	Determination of displacements and internal forces, including geometric and material nonlinear effects
3 - Design of Members	Selection and verification of suitability of members
4 – Reanalysis and Redesign	Iteration of steps 2 and 3 until all design requirements are satisfied in a consistent manner
5 – Description of Structure	Preparation of structural drawings and specifications
6 - Design of Connections	Selection and verification of suitability of connections and joints
7 - Erection Considerations	Investigation of partial structures under construction loading and bracing conditions
8 – Fabrication	Manufacturing of all elements of the structure for field erection
9 – Construction	Erection and completion of finished building

The structural engineer is responsible for the determination of loads and preliminary sizing of members in step 1, all of steps 2 to 5, and the verification, review and approval in steps 5 through 9. The fabricator and/or erector are responsible for steps 6 to 9.

A description of the design module applications that have been developed for the Integrated System corresponding to the steps outlined in Table 1.1 are listed in Table 1.2. In summary the prototype modules reflect the technological evolution and task fragmentation associated with the extended period of development. In particular:

- The evolution in programming languages resulted in modules that were programmed in Fortran, C, and C++. This multiplicity of languages complicates the integration process and the languages used are not readily portable between operating systems.
- The Hoops Graphics System implemented in the Graphical User Interface is proprietary, and has versions that are dependent upon the operating system of the intended platform of the application (not readily portable). Independent interfaces were developed for each module and the graphical functions were imbedded into main functions of the code making integration and updating difficult.
- The development of computationally intensive programs, at the time, required the use of workstation with a Unix based operating system. Advances in computing technology now permit the development and operation of such programs on the relatively less expensive personal computer (currently the preferred platform of the end user).

- **Design Standards evolve to reflect advances in engineering knowledge and generally result in new versions once every 5 years. The current design modules are dated as many of the design requirements in the application have been revised in subsequent versions of the standard. These requirements are embedded into main functions of the individual modules and are not readily updateable.**
- **The modular system is inherently dependent on the knowledge and skill level of each developer and their interpretation of how the particular module is to be integrated into the system.**

As part of the modular development of the Integrated System it was explicitly recognized that a readily available help system which describes the design process, logic and basis of the material standard CAN/CSA-S16.1 was necessary. An appropriate medium for the distribution of this information was through web technology and a hypertext document (Fratarolli, 1999). For the help system to be effective tools such as equation calculators, table generators, section browsers and dynamic design examples were developed to emulate the static components of the handbook. Furthermore, it was envisaged that an “Object Oriented Handbook” (OOH) could be developed to serve as a comprehensive electronic document from which the Integrated System could implement methods to fulfill the requirements of the standard and from which other forms of publication could be derived. The OOH, in conjunction with the corresponding Integrated System architecture, should address the difficulties realized during the development period of the prototype modules.

Table 1.2: Modular Design of an Integrated System for Steel Structures

Reference	Description
Preprocessing	
Chmelicek(1993,1996), Determination of Loads Acting on Surfaces of Structural Steel Buildings in Canada	<ul style="list-style-type: none"> Developed Loads 95 a geometric modeler for steel structures and which determines the loads (every load) and load combinations (including live load reduction factor) acting on the members according to the National Building Code of Canada (1990) Geometric modeler permits curved and irregular surfaces with any orientation
Preprocessing and Structural Analysis	
Martino (1997), Development of an Integrated System for the Geometry Definition, Load Determination and Analysis of Steel Structures	<ul style="list-style-type: none"> Integrated Loads 95 (Chmelicek, 1993 and Chmelicek, 1996) and CU-STAND 4.0 CU-STAND is a second order analysis program developed at Cornell University
Design of Members	
He (1993), Interactive Design of Flexural Steel Members Subjected to Bending, and Combined Bending and Torsion	<ul style="list-style-type: none"> Development of a program for the analysis and design of flexural steel members using resistances specified in CAN/CSA-S16.1, guidelines specified in the Steel Structural Research Council (SSRC) Guide (reference) or rational analysis (numerical, published research) Implements optimization algorithm for member selection based on weight
Srna (1995), Development of an Interactive Design and Analysis Program for Steel Beam-Columns	<ul style="list-style-type: none"> Development of a program entitled BeCol for the analysis and design of steel beam-columns Designs any doubly symmetric, prismatic steel beam-column in accordance with CAN/CSA S16.1 or CAN/CSA S136 Developed an educational module which includes: moment-thrust curvature curves, numerical integration techniques to determine in-plane stability, illustrations of equivalent moment factors and column curves Incorporates FEA program T-Shell (Albert, 1995) for advanced analysis of beam-columns

Table 1.2: Modular Design of an Integrated System for Steel Structures (cont.)

Reference	Description
Design of Members and Design of Connections	
Jiang (1995), Interactive Design of Continuous Flexural Steel Members	<ul style="list-style-type: none"> • Development of a finite element for modeling in-plane linear elastic loads • Permits various loading conditions (uniform, trapezoidal and triangular distributed loads and point loads) and variable cross-sections along the length • Developed an algorithm to determine the resistance of eccentrically loaded bolt or weld groups (symmetric or unsymmetric) with in-plane loads at any orientation • Incorporated by He (1993)
Design of Connections	
Devitt (1992), The Design, Analysis and Detailing of Bolted and Welded Simple Steel Connections	<ul style="list-style-type: none"> • Development of a program which designs and details single and double angle simple connections for beam to beam and beam to column connections • Integrates equations for resistance and geometric constraints specified in CAN/CSA-S16.1 with the general rules of practice for fabrication given in the Handbook • Emphases was placed on the development of a graphic user interface (GUI) which would minimize and simplify input and generates a 3-D representation of the designed connection, design notes, and material quantities
Sumajouw (1995), The Computer Aided Analysis, Design and Detailing of Column Base Plate Steel Connections	<ul style="list-style-type: none"> • Development of a program that analyzes, designs and details column base plates for eleven specific geometric configurations using resistances specified in CAN/CSA-S16.1 and CAN/CSA-A23.3 • Incorporated work by Devitt (1992) with similar input/output properties and GUI.
Bao (1996), Analysis, Design, and Detailing of Flexible and Extended End Plate Connections	<ul style="list-style-type: none"> • Development of a program to analyze and design flexible and extended end plate connections using the detail and resistance specifications in CAN/CSA-S16.1 • Internal forces and moment rotation curves were calculated from analytical models • Incorporated work by Sumajouw (1995) and implement a cost algorithm based on user provided data file

1.2 Objectives and Scope

The objectives and scope of this thesis are:

- To describe the system architecture required for an Integrated System for Steel Structures.
- To determine the appropriate programming language that reflects current trends in software developments (longevity), suits the requirements of the system, and is portable.
- To develop a prototype section for the Object Oriented Handbook that will illustrate the functionality of the architecture. The prototype will be limited to the Clauses, associated Clauses and appendices related to the design of beams and beam-columns as specified in CAN/CSA-S16.1
- To develop a prototype advanced analysis tool (finite element analysis program) that exploits the intent of clauses like 8.6.1. This clause specifies that rational analysis can be used to determine the resistance of members or the behaviour of frames not specifically described by the standard.
- To demonstrate the functionality of the Object Oriented Handbook within the Integrated System's architecture.

1.3 Summary

An assessment of the effects of changing computing technology and advances in engineering knowledge on the development of the Integrated System for Steel Structures is given in Chapter 1, and the requirements for an Integrated System are defined. Chapters 2 presents a description of a system architecture that should address the identified deficiencies and provide the attributes listed in Chapter 1. A description of the attributes, functionality, and integration of the OOH within the Integrated System and of other beneficial applications (such as the production of the publication and hypertext help module) are given. The theoretical background, limitations, and corresponding resistance formulations for beam-column design given in CAN/CSA-S16.1 that correspond to the prototype OOH and the FEA developments are outlined in Chapter 3. Chapter 4 presents an appropriate FEA program that was developed to validate the system architecture and to provide the capacity for the system to exercise advanced rational methods of analysis as referred to in CAN/CSA-S16.1. The summary, conclusions, recommendations, and future work are presented in Chapter 5.

Chapter 2

Integrated System for Steel Structures

2.1 Introduction

The development of a series of analysis and design modules for an integrated system for steel structures was instructive in that:

- it clearly demonstrated that computing, advanced analysis and Graphical User Interface (GUI) tools are capable of supporting the envisaged system; and
- it defined data requirements in terms of necessity, manipulation, and representation to suit the system; and
- it outlined the problems associated with the development of a large computer application over time in which both structural engineering knowledge and computing technology are evolving.

The latter point was recognised and addressed by the computing industry through the development of programming languages and philosophies like object oriented programming which allows for the independent development of objects (modules) once the system data requirements and class hierarchy are defined. Object based and object oriented programming allow the developer to create their own data types called objects. These objects contain properties that define the characteristic of an object and methods that service the object. The characteristics define the state of an object. Methods define the actions or reactions of an object and usually fall into

one of four broad categories (Wagner, et. al., 1997) as listed in Table 2.1. Object oriented programming languages, unlike object based languages, allow an object class to inherit the methods and characteristics of a super class. A class is the map of an object that defines the variables and methods that an object possesses when instantiated (Lewis and Loftus, 1998). Other computing terms are defined intermittently through the text and all terms are defined in the glossary.

The first application within the Integrated System to use these computing developments was the Hyper Handbook, which used JavaScript, an object based programming language. The application defines the Clause objects which are dynamically created from data files that are used to display the contents of the Hyper Handbook (similar to the printed version of the *"Handbook of Steel Construction"*, CISC (1994)). The Clause object has the following characteristics: title, clause number, parent clause, module, application, equation, tables and images (Frattaroli, 1999).

Table 2.1: Object Method Categories

Method Category	Description
Constructor	Creates a new object and initializes it's characteristics based on the parameters that it is passed.
Modifier	Changes the state/characteristics of the object. These methods may be called from outside or within the object
Selector	Accesses and returns a characteristic of the object
Action	Performs an action. The action may call modifier methods to change the state of the object

The next logical step in the development of an Integrated System was to define the system properties, which can be categorised as either:

- **System Characteristics:** Defines the technical framework in which the development will occur and the general form of the end product, or
- **System Architecture:** Defines how the modules interact and scope of each individual module within the context of the system.

The System allows for independent modular development and ensures compatibility while minimising redundancy. Previously developed algorithms could be easily incorporated into the System.

2.2 System Properties

2.2.1 System Characteristics

The system characteristics define the 'physical' attributes of the system. The three main characteristics that should be defined prior to software development are:

- programming language
- type of interface (system communication with the user and inter module communications)
- platform delivery

An investigation to determine the best-suited programming language for the system was performed. Exploratory applications using Perl, C, JavaScript and Java were developed. It was concluded that Java was the best-suited programming language as it possess all the attributes required by the system architecture including

platform independence, object oriented software structure, inclusion of graphics and windowing toolkits, file input/output capabilities, mathematical functions library, and garbage collection. Detailed descriptions and the corresponding definitions and benefits for these attributes are listed in Table 2.2. In addition, Java is a progressive language that is widely adopted, and seems to be becoming an industry standard, increasing the probability of longevity.

Level 1 (Figure 2.1), the main application of the integrated system, is an interface module which contains a Graphical User Interface which allows the user to define the building components and review the design, and functions which provide communications between modules. A GUI module using core Java libraries and toolkits eliminates the use of proprietary graphical packages and achieves platform independence. The system application should have the capability of maintaining a history on a given project so that it can be reinstated for future modification or review. In addition, the system should provide interface capabilities with the Internet, which will allow common updates and data to be automatically downloaded to multiple users.

The application will be deliverable over most operating systems and packaged in a CD format. The CD contains the entire Integrated System, the Hyper Handbook, the appropriate object packages, installation instructions for most platforms and the Java Virtual Machine version for each platform with the corresponding installation instructions. Note the Hyper-Handbook will also be delivered over the World Wide Web and contains Java Applets, which give it a portion of the Integrated System's functionality.

Table 2.2: Relevant Java Attributes

Attribute	Description, Definition, Benefits
Platform Independent	<ul style="list-style-type: none"> • Java applications can run on all major platforms (Unix, Windows, Linux, Mac etc.) as long as that platform contains a Java Virtual Machine • Makes program portable • Multiple platform versions not required thus requiring only one version to be up-dated in the event of changes to standards or design methodology
Object Oriented	<ul style="list-style-type: none"> • Java has an excellent Object Oriented Model, which is very versatile for the intended applications • Objects encapsulate data and methods • Allows for both class inheritance and interfaces which ensures conformity to the defined model and consistency • Encapsulation, inheritance and interfaces are beneficial for independent modular development
Graphics and Abstract Windowing Toolkit	<ul style="list-style-type: none"> • The language contains a large (and expanding) library of graphics objects and functions • The AWT (Abstract Windowing Toolkit) package contains objects, which will allow the programmer to develop Java Applets and Windows • Java Applets are Java applications that can be executed by an Internet Browser and are very useful for adding computationally intensive operations to web pages • These toolkits with some manipulation will enable a developer to create a professional graphical user interface
File Read/Write Capabilities	<ul style="list-style-type: none"> • Easy to use objects and methods that efficiently read or write data or text files
Math Library	<ul style="list-style-type: none"> • The Java language contains a math library that provides methods to perform numerous mathematical operations • The precision of Java primitive variable types is suitable for high order numerical computations
Garbage Collection	<ul style="list-style-type: none"> • Java garbage collection is defined by Lewis and Loftus (1998) as: <ul style="list-style-type: none"> • <i>"The process of reclaiming unneeded dynamically allocated memory."</i> • Java reclaims the allocated memory of an object once that object ceases to have valid references • This property is extremely beneficial to systems that dynamically create numerous objects for temporary use • Programs will slow down as allocated memory space becomes full thus freeing up allocated memory space will speed up the program

2.2.2 System Architecture

The specifications for the system architecture have been developed to meet the following objectives:

- to be easily updateable for changes to design standards, and to integrate developments in computing and engineering technologies, and
- to allow for staged modular development of independent modules, and
- to provide a standardised separate interface which will allow for the flow of information between modules and between the system and the user.

A schematic diagram of the proposed hierarchical architecture for the integrated system is illustrated in Figure 2.1. It contains four levels of applications, Objects and databases entitled System (interface), System Programs (design step modules), Elements (member objects) and Constraints and Properties (requirement objects, tools objects and databases).

Level 4, **Constraints and Properties**, contains all databases, libraries, objects and methods associated with the Object Oriented Standard, and can be accessed by all upper levels. The aqua coloured area that includes what is considered the Object Oriented Standard forms the Object Oriented Handbook and is discussed in detail in Section 2.3. The databases with a consistent structure and mark-up contain the following kinds of information: shapes and products (standard or user defined), corresponding geometric properties, pricing information, loading data, and building layout and member data. These may be classified as static or standardised (established and generally constant with respect to time), industry data (changes

periodically and a function of availability and current market pricing), and dynamic databases that are created by the user through the interface.

Level 3, **Elements**, contains objects that are defined by class as either structural, design, or geometric. Structural elements contain the methods to construct the global structural and geometric stiffness matrices and to calculate the stresses, strains and displacements for a given structural element. These are implemented in analysis programs that calculate member forces and global deflections. Design elements describe member or connection objects and contain the method (design algorithm) for the design according to the appropriate material standard, specification or code. Member or connection elements include: beam, beam-column, column, open web steel joist, composite beam, end plate, extended end plate, back to back angle connections etc., and may implement other design elements such as bolt or weld. Design elements construct dynamic database objects such as `SectionProperties`, to facilitate data transfer, by directly accessing the databases and the methods contained in the tools libraries in Level 4. The design algorithm method implements the Clause objects of the Object Oriented Standard by passing a reference to the dynamic database. This ensures encapsulation of Level 4, since the design element is not required to identify the geometric parameters required by the Clause objects. The geometric class of elements includes surface and line objects such as roofs, walls, floors and members that are used by the system's load definition program to determine the loads acting on the surfaces. The surface objects define the relationship of the member and connection objects contained within the surface and attribute loads to them in accordance with the National Building Code of Canada.

Figure 2.1: Hierarchical Architecture of the Integrated System

Level 2, **System Programs**, contains application modules that perform design tasks such as load definition, structural analysis, member design, and connection design. The Load Definition program instantiates the surfaces and line (member) elements, determines the loads and load combinations acting on the surfaces and attributes the loads acting on the line elements. The Structural Analysis program

instantiates the structural elements, defines the boundary conditions of the system, and conducts the appropriate structural analysis to determine the internal member forces and deflection data. The design programs instantiate the corresponding member and connection elements that in turn apply the methods that describe the design algorithm.

Level 1, **System**, is the communication centre that provides user access to define the building structure and control the design process. It also provides methods to accommodate interaction between system program modules and exterior applications (World Wide Web), which may be transparent to the user. Note, communication between applications, objects, and methods between levels is achieved through the use of dynamic databases which are objects in themselves designed for each type of communication process. The graphical user interface associated with level 1 would minimise input and eliminate subsequent transfer of data by the user as output from one module to be input into another module.

Changes to the Object Oriented Standard are made only to the clause objects associated with changed clauses in the standard. Changes in clause hierarchy (numbering of the clauses) would require a utility program to make the corresponding changes to the references to those objects at other levels. Additional or new clauses or sub-clauses would require direct modification to the higher order objects. New databases are constructed using the same standardised format and replace out-dated versions.

The architecture allows for expansion of the system as new knowledge is developed and is best illustrated through an example related to the class hierarchy for level 3, **Elements**, as shown in Figure 2.2. New elements, indicated by the shaded

Figure 2.2: Class Hierarchy Diagram for the Class Element

area, can be directly added to this level. For instance, if a tubular beam-column element (Tube_BeCol) is to be introduced, the new element would extend FEAElement and inherit all of its attributes and methods required by the Level 2 analysis application. The only change required to the higher level applications is to add the new element in the selection interface algorithm. The same principles hold true if additional elements are added in parallel on any branch of the class hierarchy.

2.3 Object Oriented Handbook (OOH)

The Object Oriented Handbook houses the Object Oriented Standard and all of the tools libraries and databases that are required to use the standard. The Object Oriented Standard represents CAN/CSA-S16.1 with a package of formatted software classes that encapsulate the methods, variables, and data contained within the clauses.

As described in Figure 2.3, the Object Oriented Handbook serves the integrated system by providing all relevant design information to the design element (Level 3) including structural steel section tables, geometric tools library, design requirements and resistances (Object Oriented Standard), and extensions to the Standard contained in the extended library.

Current Methods for updating and publishing the Handbook of Steel Construction (CISC, 1994) are inefficient as the text, equations, tables and figures reside in multiple and sometimes obsolete formats. As a result, the Object Oriented Handbook is being constructed to provide XML (Extensible Mark-up Language) files

Figure 2.3: Schematic Diagram of the Object Oriented Handbook within the Integrated System, after Fuller (1999)

and JPEG (Joint Photographic Experts Group) images both of which are well suited for web and printed publishing. Each clause object has the following attributes:

- parent clauses
- child clauses
- title
- Clause number
- XML formatted source text and tabular databases files
- references to JPEG image files

The source text contains XML tags to locate and to format the corresponding equations, tables and figures. Both forms of publication require an assembler program to interpret the source files and the proper published format. An assembler application has been developed for the Object Oriented Hypertext Handbook (Fratarolli, 1999) and common publication tools can be used to produce the printed version.

With the proposed system architecture revisions the CAN/CSA-S16.1 standard can be more readily incorporated into the design algorithms, and into a file structure that can accommodate multiple publication platforms. The work completed on the OOH was done in conjunction with Fuller and includes the development of the classes associated with the clauses and appendices related to the design of beams and beam-columns as listed in Table 2.3. A thorough description of the details of this work can be found in Fuller (1999).

Table 2.3: Clauses, Referenced Clauses and Appendices, and Externally Referenced Clauses Incorporated into the Prototype OOH, after Fuller (1999)

Applicability	Clause and Title
CAN/CSA-S16.1-94, Primary Prototype Clauses	Clause 13: Member and Connection Resistance
	Clause 15: Beams and Girders
CAN/CSA-S16.1-94, Referenced Clauses	Clause 10: Slenderness Ratios
	Clause 11: Width-Thickness Ratios: Elements in Compression
	Clause 12: Gross and Net Areas
CAN/CSA-S16.1-94, Referenced Appendices	Appendix D: Torsional-Flexural Buckling of Compression Members
	Appendix F: Columns Subject to Biaxial Bending
CAN/CSA-S136-94, External Referenced Clauses	Clause 5.6: Properties of Sections
	Clause 6.2: Resistance Factors for Strength Analysis
	Clause 6.4: Members in Bending

Chapter 3

Derivation and Limitations of Current Beam-Column Design Formulations

3.1 Introduction

The current design formulations for beam-columns are based on research that has focused on compact and semi-compact sections (Galambos, 1988). CAN/CSA-S16.1-M94 (S16.1) makes use of interaction equations and differentiates between Class 1 I-shaped sections and any other doubly symmetric section. These interaction equations have been calibrated against test data and provide reasonably accurate strength predictions (White and Clarke, 1997b). The specifications for beam-columns in S16.1 were not intended to be applied to non-doubly symmetric shapes or class 4 sections, although not explicitly stated. The limitations and restrictions are known given the derivations of the equations and the limitations of the research from which they are derived. Other design standards follow similar approaches, some with slightly modified interaction equations.

Beams are members subjected only to bending and shear action, whereas columns are subjected to only axial compressive forces. Most members in rigid, semi-rigid and braced frames should be designed as beam-columns if $C_b/C_r > 0.15$ (Kennedy, 1990). Exceptions to this general rule exist when it can be shown that second-order geometric effects are negligible, there is no in-span transverse load

acting on the element, and all adjoining members are connected to the element by simple connections with negligible moment capacity such as bracing elements, truss elements, and columns in very rigidly braced frames. Normally in braced frames the axial loads induced in the beams are small and can be ignored and the horizontal members can be designed as beams. Common beam to column connections in braced frames such as single and double web angles do not have sufficient moment capacity to transfer a significant moment to the column due to the beam's end rotations and thus these moments can normally be ignored.

This chapter details the derivation and limitations of current Canadian beam-column design formulations and emphasises the need for integration of advanced analysis into the design process and the development of an Object Oriented Standard to direct structural engineers in the application of these methods.

3.2 Second-Order Differential Equations for Beam-Columns

The governing second-order differential equations for a prismatic member loaded only at it's ends, given by Equations 3.1 a, b, and c for strong axis bending, weak axis bending, and twist buckling, are based on the following sets of assumptions:

- small deflections
- elastic behaviour
- initially straight
- no translation of member ends (simple supports)
- constant axial load

Strong axis bending

$$EI_x v'' + Pv - \phi \left[M_{By} - \frac{z}{L} (M_{Ty} + M_{By}) + Px_o \right] = M_{Bx} - \frac{z}{L} (M_{Tx} + M_{Bx}) \quad (3.1a)$$

Weak axis bending

$$EI_y u'' + Pu - \phi \left[M_{Bx} - \frac{z}{L} (M_{Tx} + M_{Bx}) - Py_o \right] = M_{By} - \frac{z}{L} (M_{Ty} + M_{By}) \quad (3.1b)$$

Twist buckling

$$\begin{aligned} EC_w \phi'' - (C_T + \bar{k}) \phi' + u \left[-M_{Bx} + \frac{z}{L} (M_{Bx} + M_{Tx}) + Py_o \right] - \\ v \left[M_{By} - \frac{z}{L} (M_{By} + M_{Ty}) + Px_o \right] - \frac{v}{L} (M_{Ty} + M_{By}) \\ - \frac{u}{L} (M_{Bx} + M_{Tx}) = 0 \end{aligned} \quad (3.1c)$$

- where,
- I_y = moment of inertia about the y axis
 - I_x = moment of inertia about the x axis
 - L = length
 - C_w = warping torsional constant
 - C_T = GJ, torsional constant
 - E = elastic modulus
 - v, u = translational displacement with respect to shear centre (see Figure 3.1)
 - ϕ = rotational displacement with respect to the shear centre (see Figure 3.1)
 - z = position along longitudinal axis (see Figure 3.1)

Figure 3.1: Beam-Column with Arbitrary Cross-Section Subjected to Bending and Axial Load

x_o, y_o	=	the principal co-ordinates of shear centre with respect to the centroid of the cross-section (see Figure 3.1)
$M_{Bx,y}, M_{Tx,y}$	=	applied moment at points B and T, respectively, about the x and y axis, respectively (see Figure 3.1)
P	=	axial force
$\bar{k}$	=	$-Pr_o^2 + M_x\beta_x + M_y\beta_y$
$\bar{r}_o$	=	$\pm\sqrt{x_o^2 + y_o^2 + r_x^2 + r_y^2}$
$M_{x,y}$	=	bending moment about the x axis and y axis respectively
$\beta_{x,y}$	=	cross-sectional constant
$r_{x,y}$	=	radius of gyration with respect to the x and y axis, respectively
G	=	shear modulus
J	=	St. Venant's torsional constant

The solutions to the differential equations for specific cross-sections and boundary conditions provide a description of the elastic behaviour and elastic buckling capacities (about either principal axis, lateral torsional buckling, and flexural torsional buckling), for columns, beams and beam-columns. Applying these differential equations to a beam-column, subjected to uniaxial bending about the strong axis, with symmetry in the weak axis, gives a similar set of equations.

Strong axis bending

$$EI_x v'' + Pv = M_o \left[-1 + \frac{z}{L} (1 + \kappa) \right] \quad (3.2a)$$

Weak axis bending

$$EI_y u'' + Pu - M_o \left[1 - \frac{z}{L} (1 + \kappa) \right] \phi + P y_o \phi = 0 \quad (3.2b)$$

Twist buckling

$$EC_w \phi'' - (C_T + \bar{k}) \phi' + M_o \left[1 - \frac{z}{L} (1 + \kappa) \right] u' + P y_o u' + \frac{M_o}{L} (1 + \kappa) u = 0 \quad (3.2c)$$

M_o = maximum end moment

κ = ratio of the smaller factored moment to the large factored moment at opposite ends of the unbraced length, positive for double curvature and negative for single

a) Elastic inplane behaviour

Equation 3.2a is independent of the others (v displacements only) and describes the inplane behaviour of a beam-column. The solution gives the maximum moment, $M_{\max}$, along the length or the relationship between the maximum and the end moment, and accounts for the axial loading acting on the deformed configuration for a linear moment field.

Differentiating equation 3.2a twice and substituting $k_v^2 = P/EI_x$ yields the corresponding homogenous characteristic differential equation,

$$v^{iv} + k_v^2 v'' = 0 \quad (3.3)$$

which has the following solution for the displacement when the appropriate boundary conditions are applied:

$$v = C_1 \sin k_v z + C_2 \cos k_v z + C_3 z + C_4 \quad (3.4)$$

$$\text{where, } k_v = \sqrt{P/EI}$$

$$C_{1,2,3,4} = \text{constants of integration}$$

After solving for the constants of integration based on the boundary conditions and differentiating twice, the moment at any location z along the length of the member can be found by substituting the well known moment curvature relationship (Equation 3.5) for the curvature, and is given by Equation 3.6.

$$v'' = \frac{M}{EI_x} \quad (3.5)$$

$$M = M_o \left[- \left(\frac{\kappa + \cos k_v L}{\sin k_v L} \right) \sin k_v z + \cos k_v z \right] \quad (3.6)$$

From this expression, the maximum moment can be determined by setting the first derivative of M with respect to z equal to zero and solving for $\bar{z}$, the location along the length of the member where the maximum moment occurs. The maximum moment is obtained by substituting $\bar{z}$ into Equation 3.6 and is expressed by,

$$M_{\max} = \frac{M_o}{\cos k_v \bar{z}} = M_o \psi \quad (3.7)$$

$$\text{where, } \bar{z} = \text{location of maximum moment along } z \text{ axis}$$

$$\psi = \frac{1}{\cos k_v \bar{z}} = \sqrt{\frac{1 + \kappa^2 + 2\kappa \cos k_v L}{\sin k_v L}}$$

One possible solution yields a maximum moment that lies outside the beam-column's ends and occurs when $\cos kL > -\kappa$. For this case the maximum value of Ψ is 1.0.

b) Elastic lateral torsional buckling

Another limit of elastic beam-column strength to be considered is lateral torsional buckling. The critical end moment, expressed as Equations 3.8 and 3.9, can be determined from derivatives of equations 3.2 b and c with simply supported boundary conditions.

$$EI_y u'''' + Pu'' + (M_o + Py_o)\phi'' = 0 \quad (3.8)$$

$$EC_w \phi'''' - (C_T - \bar{k})\phi'' + (M_o + Py_o)u'' = 0 \quad (3.9)$$

Trial and error must be used to determine the combination of P and M_o , which will cause lateral torsional buckling. For doubly symmetric sections a closed form solution can be derived. This solution, as presented in Equation 3.10, is expressed as the ratio of the critical end moment to the yield moment.

$$\frac{M_{ocr}}{M_y} = \sqrt{\frac{d^2 \left(1 + \frac{I_y}{I_x}\right)}{4r_x^2} \left(\frac{P_y - P}{P_{yield}}\right) \left(\frac{P_\theta - P}{P_{yield}}\right)} \quad (3.10)$$

where, d = depth of the cross section

P_{yield} = axial compressive load at yield

P_y = Euler buckling load about the y axis

P_θ = critical load for pure torsional buckling

M_{ocr} = critical elastic moment

M_y = yield moment

3.3 Beam-Column Interaction Equations

Currently CAN/CSA-S16.1 defines the resistance of a beam-column, having defined the class of the section, as the lesser value determined from the applicable interaction equations for one of the three probable modes of failure, cross-sectional strength, overall member stability, and lateral torsional strength. The interaction equations are generally limited to doubly symmetric I-shaped sections and are described in detail by 13.8.1 and 13.8.2 for all classes of sections and class 1 sections, respectively.

The corresponding form of the equations from these clauses are reproduced and are given by Equations 3.11 a and b with specific definition of the variables given in Table 3.1.

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{0.6U_{1y}M_{fy}}{M_{ry}} \leq 1.0 \quad (3.11a)$$

$$\frac{C_f}{C_r} + \frac{U_{1x}M_{fx}}{M_{rx}} + \frac{U_{1y}M_{fy}}{M_{ry}} \leq 1.0 \quad (3.11b)$$

Where, C_f = factored axial compression

C_r = factored member compressive resistance

M_{fx}, M_{fy} = maximum factored bending moment for strong and weak axis respectively, including second-order effects as discussed in section 3.2.1

M_{rx}, M_{ry} = factored member bending resistance about the x and y axis, respectively

U_{1x}, U_{1y} = factor to account for moment gradient and for second-order effects of the axial force acting on the deformed member considering the x and y axis, respectively

The following sections describe the derivation of the interaction equations and identify their limitations. They also illustrate the need to incorporate standardised numerical methods into either the standard or into an integrated system or both.

Table 3.1: Variable Definitions for Interaction Equations

Cross-Sectional Strength, Clause 13.8.1a and 13.8.2a
<ul style="list-style-type: none"> • C_r is as defined by Clause 13.3 with the slenderness parameter, λ, value equal to zero • M_r is as defined by Clause 13.5 for the appropriate class of section • U_{1x} and U_{1y}, are taken as one
Overall Member Strength, Clause 13.8.1b and 13.8.2b
<ul style="list-style-type: none"> • C_r is defined in Clause 13.3 with the effective length value, K, equal to one, and is based on the maximum slenderness ratio for biaxial bending. For uniaxial strong axis bending, $C_r = C_{rx}$ (see also Clause 9.3.2). • M_r is as defined by Clause 13.5 for the appropriate class of section • U_{1x} and U_{1y} are as defined in Clause 13.8.3
Lateral Torsional Buckling Strength, 13.8.1c and 13.8.2c
<ul style="list-style-type: none"> • C_r is defined in Clause 13.3, and is based on weak axis buckling with an effective length factor, K, as defined in Clause 9.3.3 and Appendix B and C • M_{rx} is as defined by Clause 13.6 for the appropriate class of section • M_{ry} is as defined by Clause 13.5 for the appropriate class of section • U_{1x} as defined in Clause 13.8.3, but not less than one • U_{1y} is as defined in Clause 13.8.3

3.3.1 Derivations

a) Cross-sectional strength

The interaction equations for cross-sectional strength were developed for the full plastic capacity of I-shaped sections with a typical flange to web area ratio for axial load and bending moments about either principal axis. The maximum factored moment that can be applied in combination with any factored axial load, expressed as the ratio of the axial load to compressive yield strength for strong axis and weak axis bending, are given by Equations 3.12 and 3.13 respectively, are illustrated in Figure 3.2.

$$M_{fx} = 1.18\phi M_{px} \left(1 - \frac{C_f}{\phi C_y} \right) \leq \phi M_{px} \quad (3.12)$$

$$M_{fy} = 1.19\phi M_{py} \left[1 - \left(\frac{C_f}{\phi C_y} \right)^2 \right] \leq \phi M_{py} \quad (3.13)$$

where, C_y = axial compressive load at yield stress

M_{px}, M_{py} = plastic moment capacity of the cross-section about the x and y axis respectively

ϕ = resistance factor

These equations can be expressed non-dimensionally in the form of a simplified interaction equation given in Clause 13.8.2a (Equation 3.11a) which combines the effects of bending about both axes linearly. As illustrated in Figure 3.2 the weak axis bending term is a lower bound linear approximation of the parabolic function given by Equation 3.13. This approximation in conjunction with the linear

combination of the strong and weak axis formulation is considered conservative (Kennedy, 1990). The maximum moment that can be applied is equal to the plastic section capacity and is illustrated by the limit shown in Figure 3.2.

Dawe and Kulak (1984) have determined theoretically, and subsequently verified experimentally by Dawe (Kennedy, 1990), that the cross-sectional strength is appropriately described (3.11b) for class 2 sections, with the moment resistance taken as the factored plastic moment capacity. Similar derivations for hollow structural sections have been developed and can be found in Appendix F for square hollow structural sections. Also given are the coefficients for the moment term in the interaction equation for uniaxial and biaxial bending. Interaction Equations 3.11 a and b give conservative results for rectangular hollow structural sections since these sections have a more favourable distribution of area (flange to web area ratio) over I-shaped sections. The cross sectional strength interaction equations given in S16.1 are dependent upon an assumed geometry (I shape with a given flange to web area ratio). Obviously a singular expression does not adequately describe the cross-sectional strength for all shapes.

Figure 3.2: Interaction Expression For Class 1 and 2 I-Shaped members

b) Overall member strength

The general form of the interaction equation for overall member strength can be derived from a simplified mechanics of materials formulation in which the maximum normal stress anywhere within the member due to axial load, bending, and the axial load acting on the deformed member, is limited to the yield stress as given by Equation 3.14.

$$\frac{P}{A} + \frac{M_o y}{I} + \frac{P v y}{I} \geq \sigma_y \quad (3.14)$$

where, A = area

y = distance from centroid to a point on the cross-section

σ_y = yield stress

The displacement, v , anywhere along the length obtained by solving the governing differential equation for beam-column bending about the strong axis is given by Equation 3.15, with a transverse sinusoidal load, $q = q_o \sin \pi z/L$, that produces the same displaced shape as a constant moment field.

$$EI_z v'''' + P v'' = q_o \sin \frac{\pi z}{L} \quad (3.15)$$

The displacement in terms of M_o is given as,

$$v = \frac{M_o / P_c}{\left(1 - P/P_c\right)} \quad (3.16)$$

$$\text{where, } M_o = \frac{L^2}{\pi^2} q_o \sin \frac{\pi z}{L}$$

$$P_e = \text{Euler buckling load}$$

Substituting into Equation 3.14, dividing through by the yield stress, and making the appropriate simplifications gives the basic form of the interaction.

$$\frac{P}{P_y} + \frac{M_o}{M_y} \left[\frac{1}{1 - P/P_e} \right] \leq 1.0 \quad (3.17)$$

This solution is extended to inelastic behaviour by replacing the elastic capacities with maximum axial load and the maximum bending resistance, C_r and M_r (defined in Table 3.1). The coefficients of 0.85 and 0.6 are included to give the correct limit associated with cross sectional strength as illustrated in Figure 3.3 for an applied axial load of $0.35C_y$ and a slenderness ratio equal to zero. To account for varying moment fields an equivalent moment factor is included. The interaction equations for class 1 I-shaped members for strong axis and weak axis bending, with terms consistent with those given in CAN/CSA-S16.1, become Equations 3.18 and 3.19, respectively.

$$\frac{C_f}{C_x} + \frac{0.85\omega_{1x}Mf_x}{M_x \left(1 - C/C_x \right)} \leq 1.0 \quad (3.18)$$

$$\frac{C_f}{C_y} + \frac{0.6\omega_{1y}Mf_y}{M_y \left(1 - C/C_y \right)} \leq 1.0 \quad (3.19)$$

where, $C_{rx}, C_{ry} =$ axial compressive resistance considering the x and y axis respectively

ω_{1x}, ω_{1y} = coefficient to determine equivalent uniform bending effect considering the x and y axis respectively

A similar interaction equation is used for class 2 and 3 sections with appropriate terms as defined in Table 3.1 and with coefficients for the moment term consistent with the cross-sectional strength interaction equations.

These interaction equations are based on a multiplicity of assumptions and simplifications and are applied, in practice, much more broadly than the original formulation was intended. Uncertainty related to determining a beam-column's strength (not readily quantifiable) and in interpreting physical behaviour can be essentially eliminated through the application of well defined and calibrated numerical methods.

c) Lateral torsional buckling strength

The interaction equation for lateral torsional buckling is formulated in a similar fashion to that for overall member strength and can be expressed non-dimensionally in terms of the ratio of the maximum elastic normal stress to yield stress ratio, given by Equation 3.20.

$$\frac{P}{P_y} + \left[\frac{M_{ocr}}{M_y} \right] \Psi \leq \frac{\sigma_e}{\sigma_y} \quad (3.20)$$

where, σ_e = maximum elastic normal stress

Figure 3.3: Typical Variation of the Moment Resistance With the Slenderness Ratio for Class 1 and 2 Sections With an Applied Axial Load of $0.35C_y$ (after the Canadian Institute for Steel Construction, 1994)

Two possible approaches to approximate the inelastic lateral torsional buckling strength include the reduced stress method (reduces the elastic critical moment in an analogous manner to the reduction of axial load in the inelastic region in columns (Galambos, 1968) or by providing the proper definitions of terms to the interaction Equations 3.11 a and b. For class 1 I-shaped sections with bending about the principle axes the interaction equation becomes,

$$\frac{C_r}{C_r} + \frac{0.85\omega_{lx}M_r}{M'_r\left(1 - C_r/C_r\right)} \leq 1.0 \quad (3.21)$$

where, M'_r = factored moment resistance of a member of a given unbraced length

The effect of the lateral torsional buckling as compared to inplane strength on the moment capacity for a typical class 1 section with a compressive load of $0.35C_y$ is illustrated in Figure 3.3 along with the corresponding critical elastic lateral torsional capacity as described by Equation 3.10. Applying the appropriate interaction equations with the same conditions to a class 2 section gives the other two curves in this figure.

I-shaped sections subject to weak axis bending only are not susceptible to lateral torsional buckling. The interaction equation for lateral torsional buckling with biaxial bending gives the appropriate interpretation of the terms at the end points when considering bending about either of the principle axes. The capacity of intermediate values based on a simple linear interaction is questionable.

Any approximations and uncertainties in the calculation of the compressive resistance, the reduced moment resistance, and the moment amplification factor,

which are described in the following sections would be inherited into the calculation for lateral torsional buckling capacity. Once again application of an appropriate numerical method to determine the beam-column's strength would eliminate some of the uncertainty and would give a better estimate of the member's behaviour.

3.3.2 Compressive Resistance

The compressive resistance for class 1, 2 and 3 sections is defined in Clause 13.3.1 as,

$$C_r = \phi A F_y (1 + \lambda^{2n})^{-1/n} \quad (3.22)$$

Where, n = is a compressive resistance parameter that accounts for varying residual stress patterns and out-of-straightness associated with the fabrication of different sections, defined in Table 3.2

ϕ = resistance factor to be taken as, 0.9

F_y = nominal yield stress

λ = $\frac{KL}{r} \sqrt{\frac{F_y}{\pi^2 E}}$ (nondimensional slenderness parameter)

K = effective length factor

The compressive resistances given are curve fits to a large series of numerical analyses, which account for varying out-of-straightness and stress patterns associated with the fabrication of different sections. Other codes, standards, and specifications have as many as five column curves to define the compressive resistances of available

sections. The use of two curves obviously leads to a certain amount of approximation.

Table 3.2: Compressive Resistance Parameter

n	Applicable Sections
1.34	<ul style="list-style-type: none"> • W shape of Group 1, 2, and 3 of Table 1 of CSA Standard G40.20 • Fabricated I-shapes, box shapes, Hollow Structural Sections (HSS), manufactured according to CSA Standard G40.20 cold-formed, non-stress-relieved (Class C)
2.24	<ul style="list-style-type: none"> • Welded Wide Flange (WWF) shapes with flame cut edges, manufactured according to CSA Standard G40.20 • hot-formed or cold-formed stress-relieved (Class H) HSS sections, manufactured according to CSA Standard G40.20

Note: Use a value of $n=1.34$ for sections not specifically defined in the table.

For inplane strength the specified value of K equal to one as given in clause 9.3.2 is consistent with the boundary conditions assumed in the derivation and subsequent numerical approximation of the corresponding interaction equation. For the lateral torsional buckling strength the effective length factor is associated with the out-of-plane restraint provided to the member ends by the framing elements in this plane and is defined by clause 9.3.3. K values may be approximated from the solution under ideal theoretical boundary conditions as illustrated by Figure 3.4a or evaluated from an exact solution for varying elastic end restraints as given by equation 3.23.

$$\frac{G_U G_L}{4} \left(\frac{\pi}{K} \right)^2 + \left(\frac{G_U + G_L}{2} \right) \left(1 - \frac{\pi/K}{\tan \pi/K} \right) + 2 \frac{\tan \pi/K}{\pi/K} = 1 \quad (3.23)$$

where,

$$G_{U,L} = \frac{\sum(I_c/L_c)}{\sum(I_g/L_g)}, \text{ dimensionless end restraint parameter for the upper and lower ends of the member}$$

$$\Sigma(I_c/L_c) = \text{summation of the ratio of moment of inertia about the weak axis to unsupported length columns framing into the joint}$$

$$\Sigma(I_g/L_g) = \text{summation of the ratio of moment of inertia to the unsupported length for girders framing into the joint}$$

The nomograph given in Figure 3.4b can be used to determine approximately K in lieu of solving Equation 3.23. The transverse restraint provided to the ends of the beam-column is difficult to assess, as it should include the torsional restraint provided by the inplane girder and any restraint provided by the flooring system.

Clause 13.3.2 and 13.3.3 provide guidance for the calculation of the compressive resistance of singly symmetric, asymmetric, cruciform, and class 4 columns that should not be used in the beam-column interaction equations.

3.3.3 Reduced Moment Resistance

The reduced moment resistance of laterally unsupported members is determined according to clause 13.6 which describes the elastic and inelastic lateral torsional buckling capacities and has an upper limit equal to the cross sectional strength.

Buckled shape of column is shown by dashed line	(a)	(b)	(c)	(d)	(e)	(f)
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.0	2.0
End condition code		Rotation fixed Rotation free Rotation fixed Rotation free		Translation fixed Translation fixed Translation free Translation free		

a) idealised end conditions

Sidesway Prevented

b) nomograph for column in continuous frames

Figure 3.4: Effective Length Factor (after CISC (1994), and Kulak and Gilmor (1998))

The corresponding set of equations are:

- elastic lateral torsional buckling strength if $M_u \leq 0.67M_{y,p}$

$$M_r = \phi M_u \quad (3.24)$$

- inelastic lateral torsional buckling strength if $M_u > 0.67M_{y,p}$

$$M_r = 1.15\phi M_r \left(1 - \frac{0.28M_p}{M_u} \right) \leq \phi M_p \quad (3.25)$$

where,

$$M_u = \frac{\omega_2 \pi}{L} \sqrt{EI, GJ + \left(\frac{\pi E}{L} \right)^2 I, C_w}, \text{critical elastic moment}$$

and where, ω_2 = equivalent moment factor and is,

- $1.75 + 1.05\kappa + 0.3\kappa^2 \leq 2.5$ when the member is subjected to end moments
- 1.0 when the maximum moment occurs between supports or when lateral support is not provided to the compression flange at one of the member ends
- evaluated according to the Guide to Structural Stability Design Criteria for Metal Structures (Galambos, 1988)

$M_{y,p}$ = Is equal to the elastic or plastic moment capacity for class 3 and 4, and class 1 and 2 sections, respectively, as defined in Clause 13.5

The elastic lateral torsional buckling formula was derived for torsionally and flexurally pinned end conditions, doubly symmetric shapes, and constant end moments. The equivalent moment factor properly accounts for the likely moment field (for beam-columns) but does not account for the possibility of interactive continuous buckling of continuous columns. Approximation in the inelastic lateral torsional buckling equation may lead to greater uncertainties in the prediction of the capacity using the beam-column interaction formula.

3.3.4 Moment Amplification and Equivalent Moment Factors

The solution of the governing differential equations for beam-columns with pinned boundary conditions, no relative lateral displacements of the ends, symmetric in the plane of bending section, and axial end forces, describes the elastic behaviour and accounts for both varying moment fields and the effect of the axial load acting on the deformed structure. The derivation of the corresponding interaction equations in Clause 13.8 of CAN/CSA-S.16.1 is based on solid mechanics and a series of assumptions and approximations, one of which, the moment amplification factor, U_1 , was introduced to capture these effects.

U_1 as given in Equation 3.26 approximates the value ψ described in Section 3.2 which modifies the largest end moment to give the maximum moment along the member as given in Equation 3.7.

$$U_1 = \frac{\omega_1}{1 - \frac{C_f}{C_c}} \approx \psi \quad (3.26)$$

Comparison of the maximum moment within the member as given by the solution for the differential equations for the case in which the member is subjected to varying moments or a transverse sinusoidal load (producing the same deflected shape as a constant moment field) gives the equivalent moment factor ω_1 .

$$\omega_1 = \psi \left(1 - \frac{C_f}{C_c} \right) \quad (3.27)$$

Ketter et al. (1961) derived an empirical expression that approximates the equivalent moment factor, ω_1 , which was based on the solution to a series of numerical analyses on beam-columns with varying end moments and ratios of applied load.

$$\omega_1 = 0.6 - 0.4\kappa \geq 0.4 \quad (\text{Ketter et al., 1961}) \quad (3.28)$$

The limit of 0.4 tends to be conservative for higher values of C_r/C_e (Kennedy, 1990). CAN/CSA-S16.1 uses this approximation for ω_1 and further defines values for selected load cases. The corresponding commentary (CISC, 1994) to Clause 13.8.4 provides additional cases and refers to the Structural Stability Research Guide for Metal Structures (Galambos, 1988) for the definition of ω_1 for additional instances. A selection of these is illustrated in Figure 3.5.

Galambos (1988) further suggested that ω_1 may be taken as the inverse of ω_2 , which is defined as the coefficient to account for increased moment resistance of a laterally unsupported beam segment to a moment gradient. Semi-empirical expressions have been developed for ω_2 to describe a broad range of load application cases. One series of cases describes the effect of the point of application with respect to the shear centre for three different load cases. The equivalent moment factor, ω_2 , with the load applied to the top flange, shear centre and bottom flange is described by Equations 3.29 a, b, and c respectively, with terms illustrated in Figure 3.6.

$$\omega_2 = \frac{A}{B} \quad (3.29a)$$

$$\omega_2 = A \quad (3.29b)$$

$$\omega_2 = AB \quad (3.29c)$$

For an arbitrary moment field, as illustrated in Figure 3.7, Kirby and Nethercot (1979) suggest that ω_2 can be calculated by equation 3.30.

$$\frac{1}{\omega_2} = \frac{3M_1 + 4M_2 + 3M_3 + 2M_m}{12M_m} \quad (3.30)$$

where, moments M_1 , M_2 , M_3 , and M_m are defined in Figure 3.7

Derivations of ω_2 values, which apparently increase the scope of application of ω_1 , have been derived in the absence of axial load. The correctness of this application is unknown.

Load Case	Formula for ω_1	Limits
$0.6 - 0.4\kappa$	$C_1/C_c \geq 0.4$
	$1 - \frac{1 - (0.6 - 0.4\kappa)C_1}{0.4C_c}$	$C_1/C_c < 0.4$
or	$0.5 + 0.7 \frac{M_m}{M_o}$	$C_1/C_c \geq 0.5$
	$1 - 2[1 - (0.5 + 0.7 M_m/M_o)] \frac{C_1}{C_c}$	$C_1/C_c < 0.5$

Note: M_m is the average moment on the member
 M_o is the maximum moment on the member

Figure 3.5: Equivalent Moment Factor, ω_1 , for Additional Load Cases

Loading	BMD	M	A	B
$\frac{PL}{4}$	1.35	$1 - 0.180W^2 + 0.649W$
$\frac{wL^2}{8}$	1.12	$1 - 0.154W^2 + 0.535W$
PL_1	$1 + \left(\frac{L_1}{2L_1 + L_2}\right)^2$	$1 - 0.465W^2 + 1.636W$

Note: $W = \frac{\pi}{L} \sqrt{\frac{EC_v}{GJ}}$

Figure 3.6: Parameters to Determine the Equivalent Moment Factor, ω_2 (after Galambos, 1988)

Figure 3.7: Definition of ω_2 for an Arbitrary Bending Moment Field

3.4 Analysis of Structure and Stability Effects

To determine the maximum load effects on a structure CAN/CSA-S16.1 requires that a second-order elastic analysis be conducted or that the second-order effects be accounted for by amplifying the transitional load effects obtained from a first-order analysis by the amplification factor U_2 in accordance with clause 8.6.1.

$$U_2 = \frac{1}{1 - \left[\frac{\sum C_f \Delta_f}{\sum V_f h} \right]} \quad (3.31)$$

where, $\sum C_f$ = sum of factored axial compressive loads of all columns in a storey

$\sum V_f$ = sum of factored lateral loads above the story (first-order storey shear)

h = storey height

Δ_f = relative first-order lateral translation of the storey due to factored loads (coincident with M_R)

The corresponding maximum factored moment is determined by Equation 3.32.

$$M_f = M_{fg} + U_2 M_R \quad (3.32)$$

where, M_{fg} = first-order moment under factored gravity loads (assuming no lateral translation or imperfections)

M_R = first-order transitional moment under factored loads or the moment resulting from transitional or gravity loads acting on an unsymmetrical frame

Note the amplification factor U_2 is the limit of the iterative procedure described in Appendix J of the standard and referred to in Clause 8.6.1b.

Clause 8.6.2 specifies a notional lateral load, applied to each storey, equal to 0.005 times the factored gravity loads acting on that storey, to provide a minimum translational load effect for load combinations considering gravity loads only. The application of this clause in combination with the relevant member resistance clauses implies that the capacity of the frame is limited to development of the first plastic hinge associated with the load effects determined by a second-order elastic analysis. This is unconservative as it underestimates the lateral deflection (particularly for flexible structures) and is conservative in that it underestimates the frames ultimate capacity. The most accurate form of analysis to determine second-order effects is that which performs a second-order elastic-plastic analysis (White and Clarke, 1997a).

Elastic-plastic analysis programs such as these are not yet readily available and would require a comprehensive knowledge of non-linear behaviour by the user (Kennedy, 1990). If the second-order elastic-plastic analysis is sufficiently accurate to include out-of-plane stability effects like lateral torsional buckling and if the application of the analysis is simple then the advanced analysis tool could be used for analysis and design. This is already permitted by the standard in Clause 1.4 in that rational design and analysis based on theory, and engineering practice acceptable to the regulatory authorities may be used in lieu of the formulations provided in the standard.

The requirements to account for load amplification due to second-order geometric effects contained within the standard are intuitive and lend themselves to automated design. The standard separates the structural analysis and member design requirements through load amplification factors and effective lengths. Chen and Lui

(1991) noted that the overall frame behaviour should be considered when designing a beam-column. It was indicated in previous sections that stability effects such as interactive member buckling could not be captured by consideration of the member in isolation. The standard encourages the designer to implement advanced analysis to determine the second-order effects. The next logical step is to incorporate structural analysis and member design through the introduction of guidelines for advanced numerical methods capable of simultaneously considering the member's capacity and the frame's stability.

Chapter 4

Finite Element Analysis Application

The current design requirements in CAN/CSA-S16.1 for frames specify that either a second-order elastic analysis or a first-order elastic analysis with translational moment amplification to account for P- Δ effects be conducted to determine the member end forces. The interaction resistance equations for beam-columns are limited to Class 1, 2 and 3 doubly symmetric sections with a small set of specified boundary and loading conditions. Reference is made to the Structural Stability Research Council Guide for Metal Structures (Galambos, 1988) to extend the application of these equations.

The inconsistencies between the specified analysis and resistance requirements coupled with limitations of application and increasing complexity can be resolved through the application of an appropriate numerical method for steel structures. Within the proposed Integrated System the numerical method program should serve as both an analysis and design tool and be developed in a format consistent with the system architecture. As a result a prototype object oriented finite element analysis application was developed.

The main objectives of this work were to determine the suitability of object oriented programming to perform advanced analysis, to demonstrate the merits of the

system architecture, and to identify the advantages and limitations of the elements and analysis technique implemented.

4.1 Software Algorithm and Architecture

The prototype object oriented finite element analysis application is based on the finite element formulations and principles used in an existing application called T-Shell (Albert, 1998) that employs a three-noded triangular shell elements and three two-noded line elements. The general algorithm implements the direct displacement (stiffness) method and the incremental load control solution strategy. The latter ignores the unbalanced forces at the end of each load step and does not guarantee equilibrium. Unlike other control methods like the Newton-Raphson, modified Newton, and the arc length methods, the solution will always converge and remains stable throughout limit points on the load deflection curve if adequately small load increments are used (Chen and Lui, 1991).

The T-Shell algorithm implemented by Albert (1998) using a top down procedural programming approach is illustrated in Figure 4.1. The first step (highlighted in blue) is to read the input file and format the data into vectors indexed on node and element number and is called Initialise. The first step in the iterative procedure (highlighted in green) is the formation of the constitutive matrices for shell elements or the calculations of properties for beam-column elements based on each element's stress state at the start of each iteration. Subsequently the local geometric

Figure 4.1: T-Shell Algorithm, Albert (1998)

stiffness, local structural stiffness, and transformation matrices, based on the deformed configuration, are formed for each element. The banded tangent stiffness matrix for the structure is constructed by the superposition of the transformed elemental stiffness matrices and the restraints. Restrained degrees of freedom are assumed to be relatively rigid and are accounted for by the addition of a large stiffness value (10^{20}) to the corresponding diagonal term. The system of equations is solved using a procedure, Mondkar and Powell (1974), that first reduces the structure's global stiffness matrix to an equivalent upper triangular matrix (Crout reduction). Subsequently, the reduction operations are applied to the force vector and the displacements are solved using back substitution. Incremental member forces, reactions, strains, and stresses are calculated and their total values maintained. Finally, the nodal co-ordinates are updated and the process repeated for each incremental load.

The object oriented finite element application within the framework of the Integrated System, as illustrated in Figure 4.2, accrues the same benefits as the integrated system. In particular, the benefits include modular structure, hierarchical functionality, and simplicity in that the methods are compartmentalised resulting in easy interpretation. Two independent threads that run in parallel with the main method (FEAMain), which is housed within Level 2, simulate Level 1 of the Integrated System. A thread is an independent process executing within a program (Lewis and Loftus, 1998). The first thread provides a primitive Graphical User Interface, illustrated in Figure 4.3, from which the program is controlled and incremental data is displayed. Figure 4.3a illustrates the main window from which

Figure 4.2: Architecture of Object Oriented Finite Element Application

the tools menu allows the user to view the normalised load vs. Euclidean norm curve (Figure 4.3b) or to modify the load increments (Figure 4.3c) at a predefined stop step. The second thread directly accesses Element and Node objects and outputs the appropriate data to a text file. The output file contains all nodal displacements, element stresses located at the given stress point (von Mises, normal and shear stresses), reactions at restrained nodes, Euclidean norm, Euclidean slope, total load factor, and the determinant of global stiffness matrix for each load increment in a tabular form. FEAMain has a sleep method that is activated by the output thread prior to it passing the displacement information to the SetDisplacement method within the FEAElement objects during the next iteration. This ensures that values required by the output thread are not changed prior to being printed to the text file.

The FEAMain application (Level 2) contains the methods to instantiate the FEAElement objects, to assemble the global structural stiffness matrix, and to solve the system of equations for the displacements. User input is received in a text file format that is read by a method, DataInput, that calls upon a series of static methods contained in a methods library called Initialisation. The input file specifies the location of all nodes, material types, element types and properties, restraints, loads, and load increments. The encapsulation of the Input method, which instantiates the objects required by the main method, readily enables the adaptation of the application to receive a dynamic database object as input in lieu of a text file as envisaged by the Integrated System's architecture. FEAMain instantiates the FEAShield object, which contains references to state information such as the incremental load vector,

Number of Increments	25
Load Increment 20	0.02
Load Increment 21	0.02
Load Increment 22	0.02
Load Increment 23	0.02
Load Increment 24	0.02
Load Increment 25	0.02
Stop Step	19

c) Load Increment Modification

Figure 4.3: Graphical User Interface for Finite Element Analysis Application

Euclidean norm vector, and current incremental step number. It also contains methods to permit access and alteration of state by the threads.

I_BeCol, R_BeCol, HR_BeCol, and T-Shell are child classes (sub classes) extended from the parent class (super class) FEAElement that inherit all of it's methods and characteristics. These classes may be referred to as FEAElement when other objects or applications access the methods, listed in Table 4.1, or characteristics that were inherited. This allows the addition of new element types with no change to FEAMain. The SetDisplacement and Update methods are particular to the child classes and as a consequence are overwritten. Knowledge of the specific object type is required at the time of its construction by the DataInput Method. Level 4 houses physical and auxiliary objects such as Node, illustrative example in Figure 4.2, Restraint, NodalLoad, Material and Matrix. Level 4 also contains a method library for all required operations.

A complete description of the two noded beam-column elements (I_BeCol, R_BeCol, and HR_BeCol I-shaped, rectangular and hollow rectangular cross-section, respectively) and the three noded triangular element (T-Shell) follow.

Table 4.1: Methods Inherited by Child Elements from FEAElement

Method	Description
setNodesDof	<ul style="list-style-type: none"> • Accesses the Node object and sets the number of degrees of freedom for that node
getType	<ul style="list-style-type: none"> • Returns an alphanumeric string depicting the element type
getStructural	<ul style="list-style-type: none"> • Returns the structural stiffness matrix for the element in global co-ordinates
getNodes	<ul style="list-style-type: none"> • Returns a vector containing references to all Nodes associated with the Element
getNumNodes	<ul style="list-style-type: none"> • Returns an array of the Node numbers associated with the Element
getNodealDOFInfo	<ul style="list-style-type: none"> • Returns a two dimensional array including the Node numbers and the number of degrees of freedom per node associated with the element • Implemented to allow for the future development of Elements that do not have a consistent number of degrees of freedom per node
setDisplacements	<ul style="list-style-type: none"> • Sets the incremental displacements in the element • Calculates incremental member forces, strains, and stresses and updates the total values • This method is overwritten by each Element since the method is element type dependent
update	<ul style="list-style-type: none"> • Updates the Element's global positioning variables and recalculates the transformation, structural stiffness, and geometric stiffness matrices • This method is overwritten by each Element since the method is element type dependent
getTransformation	<ul style="list-style-type: none"> • Returns the Element's transformation matrix
getStresses	<ul style="list-style-type: none"> • Returns an array of stresses located at the predetermined stress points
getVonMises	<ul style="list-style-type: none"> • Returns an array of von Mises stresses located at the same predetermined stress points.

4.2 Beam-Column Elements

Three beam-column element classes, with six degrees of freedom (d.o.f.) per node, as illustrated in Figure 4.5a, capable of modelling second-order inelastic behaviour, have been developed as part of the prototype finite element application. A complete description of each requires the definition of:

- Object geometry (cross-sectional properties, orientation, and d.o.f.)
- Residual stress pattern
- Structural stiffness matrix (bilinear material characteristics, failure criterion)
- Geometric Stiffness Matrix
- Initial out-of-straightness and out-of-plumb

To completely initialise each beam-column element the following parameters need to be defined: geometry (t, w, b, Point P, and references to two Node objects), magnitude of the residual stress, and a reference to a Material object.

a) Object Geometry

The general cross-sectional shape is defined by the classes I_BeCol, R_BeCol, and HR_BeCol, for I-shaped, rectangular, and square hollow structural sections, respectively, as illustrated in Figure 4.4a. The parameters passed to the constructor method instantiates the object and defines the size, orientation of the cross-section and the local co-ordinate system as illustrated in 4.4a and 4.5b. The parameter point P is described by three global co-ordinates and is located in the plane perpendicular to the longitudinal axis that contains the first Node (Node 1). Nodes 1 and 2 describe

a) cross-sections

b) residual stresses

Figure 4.4: Beam-Column Elements

the local longitudinal axes. Point P is used to define the depth of the cross-section and in conjunction with the Nodes establishes the relationship between the local and global co-ordinate systems.

a) Residual Stresses

The stress distribution for each cross-section is modelled by an assumed constant stress state over a series of stress zones. The magnitudes of the assumed stresses (for each zone) are equal to the stresses evaluated at a predefined stress point associated with each zone. The location of the stress points are based on a natural co-ordinate system with stress points located at the intersection of each grid line as illustrated in Figure 4.6. The stress points with their corresponding areas for each cross-section are also shown in this figure. Stress points 4 and 6 for I-shaped sections and 5 for square hollow sections have area values of zero assigned to them. Based on this definition of internal stress the residual stress distributions are those illustrated in Figure 4.4b with a maximum value defined as input. Each flange and web component is in equilibrium.

a) degrees of freedom

b) local co-ordinate system

Figure 4.5: Beam-Column Element Definition

a) normalized natural co-ordinate system and location of stress points

b) stress points and areas for I-shaped cross-section

c) stress points and areas for rectangular cross-section

d) stress points and areas for square HSS

Figure 4.6: Description of Nodal Co-Ordinate System, Stress Points, and Areas for Cross-Section

c) Material Definition and Structural Stiffness Matrix

The material behaviour for each stress point on the cross-section of the element is defined by a bilinear stress-strain curve with a von Mises isotropic strain hardening yield surface. The bilinear stress-strain curve, illustrated in Figure 4.7, is defined by the data contained within the Material object that is referenced by FEAElement. To avoid numerical instability the strain hardening modulus, E_{α} , must be specified greater than zero. The Material object calculates the shear modulus, G . Note that although each FEAElement must be defined by a single material, the user may define more than one material for the structure. To account for cross-sectional plasticity, effective cross-sectional stiffnesses (reduced) are calculated for the stresses and moduli associated with each stress zone using a modification of the Simpson's Method to integrate over the cross-sectional area. These stiffnesses include axial,

Figure 4.7: Bilinear Stress Strain Curve

EA, strong axis bending, EI_z , weak axis bending, EI_y , torsional, GJ, and warping, C_w , where C_w is assumed to be zero for rectangular plates and square hollow sections and is approximated for I shapes as,

$$C_w = [EI_y]_{eff} \left(\frac{d-t}{2} \right)^2 \quad (4.5)$$

where $[EI_y]_{eff}$ = effective weak axis bending stiffness

The corresponding structural stiffness matrix, $[K]$, for a three-dimensional beam-column element, derived using the standard unit displacement method (assuming elastic behaviour) is given in Figure 4.8.

The finite element object I_BeCol gives the option to include additional d.o.f. to account for the effects of warping. The original derivation (Reilly, 1972) of the appropriate stiffness terms was extended by Albert (1998) to three-dimensional beam-column elements as highlighted in Figure 4.9.

d) Geometric Stiffness Matrix

The second-order geometric effects, axial load acting on the deformed structure (P- δ , P- Δ), and the effects of the axial load acting on the torsional stiffness of a thin plate subjected to flexural torsional buckling are accounted for through the inclusion of the geometric stiffness matrix, $[K_g]$, illustrated in Figure 4.10. The matrix (Albert, 1998) was derived using the same methods as Yang and McGuire (1986), except that linear displacements between nodes were assumed rather than using a cubic Lagrangian polynomial.

$$\begin{bmatrix}
 \frac{EA}{L} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
 0 & \frac{12EI_z}{L^3} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
 0 & 0 & \frac{12EI_y}{L^3} & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
 0 & 0 & 0 & \frac{12EI_z}{L^3} & 0 & 0 & 0 & 0 & 0 & 0 \\
 0 & 0 & 0 & 0 & \frac{12EI_y}{L^3} & 0 & 0 & 0 & 0 & 0 \\
 0 & 0 & 0 & 0 & 0 & \frac{12EI_z}{L^3} & 0 & 0 & 0 & 0 \\
 0 & 0 & 0 & 0 & 0 & 0 & \frac{12EI_y}{L^3} & 0 & 0 & 0 \\
 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{12EI_z}{L^3} & 0 & 0 \\
 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{12EI_y}{L^3} & 0 \\
 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{12EI_z}{L^3}
 \end{bmatrix}$$

SYM.

$$\begin{bmatrix}
 -\frac{EA}{L} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
 0 & \frac{6EI_z}{L^2} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
 0 & 0 & \frac{6EI_y}{L^2} & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
 0 & 0 & 0 & \frac{6EI_z}{L^2} & 0 & 0 & 0 & 0 & 0 & 0 \\
 0 & 0 & 0 & 0 & \frac{6EI_y}{L^2} & 0 & 0 & 0 & 0 & 0 \\
 0 & 0 & 0 & 0 & 0 & \frac{6EI_z}{L^2} & 0 & 0 & 0 & 0 \\
 0 & 0 & 0 & 0 & 0 & 0 & \frac{6EI_y}{L^2} & 0 & 0 & 0 \\
 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{6EI_z}{L^2} & 0 & 0 \\
 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{6EI_y}{L^2} & 0 \\
 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{6EI_z}{L^2}
 \end{bmatrix}$$

Figure 4.8: Structural Stiffness for Beam-Column Elements

$$\begin{bmatrix}
 \frac{EA}{L} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
 0 & \frac{12EI_z}{L^3} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
 0 & 0 & \frac{12EI_y}{L^3} & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
 0 & 0 & 0 & \frac{6EI_z}{L^2} & 0 & 0 & 0 & 0 & 0 & 0 \\
 0 & 0 & 0 & 0 & \frac{6EI_y}{L^2} & 0 & 0 & 0 & 0 & 0 \\
 0 & 0 & 0 & 0 & 0 & \frac{6C_w}{L^2} & 0 & 0 & 0 & 0 \\
 0 & 0 & 0 & 0 & 0 & 0 & \frac{4EI_z}{L} & 0 & 0 & 0 \\
 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{4EI_y}{L} & 0 & 0 \\
 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{4C_w}{L} & 0 \\
 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0
 \end{bmatrix}$$

SYM.

Figure 4.9: Structural Stiffness for I-shape Beam-Column (I_BeCol) Including the Effects of Warping

Note, the effects of the axial load on the torsional stiffness are given as a function of the Wagner coefficient, $\bar{K}$,

$$\bar{K} = \frac{P(I_y + I_z)}{A} \quad (4.9)$$

and were approximated by integrating the stresses over the area using the Simpson's method previously described.

e) Out-of-Straightness and Out-of-Plumb

Member out-of-straightness includes sweep, camber and twist and may be accounted for by specifying the measured shape (nodal and point P co-ordinates) or by applying a series of concentrated loads which causes the member to deform. Out-of-plumb frames may be modelled using minimum notional loads as described for the determination of U_2 in Clause 8.6 of CAN/CSA-S16.1.

4.3 Triangular Element

A three noded triangular shell element class with eighteen d.o.f. as illustrated in Figure 4.11, T_Shell, was constructed and is based on a modified version of the element developed by Lee and Yoo (1988). The element superimposes the nine d.o.f. associated with the Kirchhoff plate bending element with the six d.o.f. of a standard inplane element, with an additional three inplane rotational d.o.f. A detailed

description of the T_Shell class and its parameters which describe geometry and state are given in Fuller (1999).

The element has been developed to model structural steel member stability problems and has the potential to capture local instability if the cross-section is sufficiently discretised. Fuller (1999) verified the behaviour of beams and beam-columns with experimental data of members that are subject to lateral torsional buckling and flexural torsional buckling, by modelling the cross-section with R_BeCol objects for the flanges and T_Shell objects for the webs.

Figure 4.11: Degrees of Freedom for Shell Element (T_Shell)

4.4 Model Verification

Two sets of test data for tests conducted on members and frames that are subject to failure modes associated with cross-sectional strength, beam-column stability, and frame stability were used to verify the results of comparative models constructed with the I_BeCol finite element object. For each reference, Van Kuren and Galambos (1964), and Arnold et al. (1966), a description of the geometry and material properties for the members and frame, the loading and boundary conditions,

test results (loads or characteristic load deflection behaviour), numerical model description, predicted loads and behaviour, and the corresponding test to predicted ratio are presented.

4.4.1 Van Kuren and Galambos Tests

Van Kuren and Galambos (1964) reported the behaviour and failure loads for forty-two tests conducted on class 1 and 2 I-shaped beam-columns subjected to combined strong axis bending and axial load for the set of ideal boundary conditions illustrated in Figure 4.12. Note, the loading apparatus restrained the rotation about

Figure 4.12: Boundary and Load Conditions, Van Kuren and Galambos (1964)

the weak axis at the ends of the member. Based on comparisons with the results of numerical analysis this restraint was found to be less than fixed and corresponded to an effective length factor equal to approximately 0.6. Ten of these (A-series) were laterally supported short members that failed either by local buckling or by reaching the section's moment capacity and are not included in this comparison. Of the remaining thirty-two (T-series) which failed by inelastic instability four were eliminated from the data set as they failed by local instabilities which are not modelled by this element.

The test specimens modelled had the following cross-section designations, 8WF13, 4WF13, and 8WF40 with the nominal geometric properties listed in Table 4.2. All rolled sections were made from ASTM A7 steel with the average yield stresses and moduli of elasticity for each given in Table 4.2.

Table 4.2: Geometric and Material Properties, Van Kuren and Galambos (1964)

Section	8 WF 40	8 WF 31	4 WF 13
Metric Equivalent	W200x59	W200x46	W100x19
B (mm)	205	203	103
T (mm)	14.2	11	8.8
D (mm)	210	203	106
W (mm)	9.1	7.2	7.1
Σ_y (MPa)	258.8	275.3	272.6
E (MPa)	195300	206300	201500
Σ_x (MPa)	77.6	82.6	81.8

Note: The average maximum compressive residual stress was cited to be 30% of yield. No other details were given.

Each beam-column was modelled with fourteen d.o.f. I_BeCol elements (includes warping) for the geometry, material properties, and idealised boundary and loading conditions sited by Van Kuren and Galambos. The members were discretised into between 36 and 97 elements with intermediate elements of equal length, equal to 50 mm, and end elements that varied from 44.2 to 78.4 mm in length.

The first step in the loading sequence was to apply small notional loads to cause initial imperfections equal to or less than the rolling tolerances, providing a sweep and camber, Δ , less than $L/1000$. Subsequently a constant axial load or moment equal to the test value was applied. The last part of the sequence was the application of a series of load increments (the control load, either axial load or bending) to cause failure. Figure 4.13 illustrates typical load deflection curves and the corresponding deflected shape for test T-24. The ultimate predicted load for this test is identified.

The test specimen designation, cross-section, loading case, length, test and predicted loads, and test to predicted ratios for this series of test is given in Table 4.3. Green highlighted values indicate tests in which the applied moment was constant and the predicted values were compared to the applied axial load. The others (uncoloured) correspond to tests in which the applied load was constant and the moment was increased until failure.

The mean test to predicted ratio and coefficient of variation for this series of tests was determined to be 0.924 and 0.113, respectively. Test T-6 was rejected as a statistical outlier using Chauvenet's criteria yielding a net mean and coefficient of variation, for the series, of 0.935 and 0.099, respectively.

Figure 4.13: Load Deflection Curves, Test T-24

Table 4.3: Test and Predicted Results and Test Properties

Test	Cross-section	Loading Case	Length (mm)	Test		Predicted Load	t/p
				Axial Load (kN)	Moment (kN•m)		
T-1	8WF31	d	1829	211.8	143.0	148.7	0.962
T-2	8WF40	d	1829	289.3	190.0	195.7	0.971
T-3	8WF31	b	4877	801.0	80.7	-	-
T-4	8WF31	b	4877	199.8	132.0	126.9	1.04
T-5	8WF31	b	4877	1279.8	21.9	20.6	1.065
T-6	4WF13	b	4879	178.0	16.0	24.9	0.645
T-7	4WF13	b	4879	178.0	18.2	25.1	0.725
T-8	8WF31	c	4877	956.8	21.5		0.735
T-9	4WF13	b	4874	66.8	24.0	28.0	0.855
T-10	4WF13	b	4874	333.8	6.0	6.4	0.943
T-11	8WF31	e	4900	1412.9	0		0.893
T-12	8WF31	c	4877	199.8	106.0	123.0	0.862
T-13	8WF31	d	4877	199.8	142.0	158.9	0.894
T-14	8WF31	a	4877	372.5	120.0	-	-
T-15	8WF31	e	3658	1379.5	0		1.053
T-16	8WF31	c	3658	199.8	104.0	-	-
T-17	4WF13	b	2438	81.9	24.7	27.0	0.917
T-18	8WF31	e	2438	1468.5	0		0.917
T-19	8WF31	c	2438	199.8	109.0	132.0	0.826
T-20	4WF13	c	2438	81.9	22.5	23.9	0.943
T-21	4WF13	b	2438	330.6	12.5	-	-
T-22	4WF13	b	2438	418.3	11.1		0.877
T-23	4WF13	d	3658	81.9	27.7	28.6	0.971
T-24	4WF13	b	3658	81.9	31.2	30.3	1.031
T-25	8WF31	e	3899	1139.2	0		1
T-26	4WF13	c	3658	81.9	20.3	23.6	0.862
T-27	8WF31	c	3658	801.0	13.1	13.4	0.98
T-28	4WF13	e	3658	525.1	0		1.099
T-29	4WF13	a	3658	81.9	31.4	30.5	1.031
T-30	4WF13	a	4877	81.9	27.3	28.2	0.971
T-31	4WF13	d	4877	81.9	23.5	26.8	0.877
T-32	4WF13	c	4877	81.9	18.1	19.3	0.935

Note 1: Green highlighted values indicate test in which the applied moment was constant and the predicted values are for axial load. All others are moment predictions.

Note 2: Yellow highlighted test were not considered as these failed prematurely by local plate buckling.

The analytical model is currently limited to the application of idealised boundary conditions, and the effective boundary conditions (fixed/fixed) applied for rotations about the weak axis yielded an effective length factor equal to 0.5. This value as compared to 0.6 (average effective length factor determined by Van Kuren and Galambos) would partially account for the over prediction of failure (test to predicted ratio less than one) by the analytical model. Following up on this premise test T-6, T-7, and T-8, were reanalysed with fixed/pinned end conditions for weak axis bending ($K=0.7$) and gave test to predicted ratios of 0.952, 1.042, and 0.840, respectively. These ratios indicate that the premise may be correct and the sensitivity of beam-column stability behaviour (failure mode and load) to small variations of the out-of-plane rotational restraint.

A portion of the variation in test to predicted loads might be attributed to assumptions in the magnitude of the initial imperfections. As illustrated in Figure 4.14 (Load vs. Euclidean Norm curve where the Euclidean Norm is a characteristic deflection parameter equal to the square root of the sum of the total deflections squared (rotations and displacements)), test T-24 gave test to predicted ratios of 1.010, 1.042, and 1.087 for initial imperfections (sweep and camber) of $L/2000$, $L/1000$, and $L/500$, respectively. The variation may vary from test to test depending on the restraint and failure mode, which may change due to imperfections.

Notwithstanding the effects of end restraint and imperfections the prediction of both ultimate load and behaviour was in good agreement with the tests indicating the versatility of the simple I_BeCol element for this class of problems.

Figure 4.14: Load vs. Euclidean Norm Curve for Varying Initial Out-of-Straightness (Sweep and Camber, Test T-24)

4.4.2 Arnold, Adams, and Lu Test

The single greatest advantage of the beam-column elements that have been developed is their ability to closely model the inelastic second-order behaviour of frames including out-of-plane behaviour. This can be illustrated by comparison with test data on frames, in which sufficient detail about the test is given so that a representative model can be developed.

Arnold, Adams and Lu (1966) conducted a full-scale test on a single storey single bay frame subject to both vertical and horizontal loads as illustrated in Figure 4.15. The frame consisted of a 10I25.4 beam section and a 5WF18.5 column section, constructed of A36 and A441 steel, respectively. The measured geometric and material properties for these sections are listed in Table 4.4. The residual stress distributions for the beam and column sections were reported and the simple arithmetic mean of the absolute peak values are given in Table 4.4.

Table 4.4: Geometric and Material Properties, Arnold et al. (1966)

Section	10I25.4		5WF18.5	
Units	Imperial	Metric	Imperial	Metric
b	4.66 in	118.4 mm	5.025 in	127.6 mm
d	10.0 in	254.0 mm	5.12 in	130.0 mm
t	0.491 in	12.47 mm	0.420 in	10.67 mm
w	.310 in	7.87 mm	0.265 in	6.73 mm
σ_y	38.57 ksi	265.9 MPa	56.17 ksi	387.3 MPa
σ_r	7 ksi	48.3 MPa	8 ksi	55.2 MPa
E	29440 ksi	203000 MPa	30860 ksi	212800 MPa

Figure 4.15: Frame Geometry and Load Locations, Arnold et al. (1966)

The frame was braced at the flange tips against out-of plane translational displacements at the eight points illustrated in Figure 4.16. Beam to column connections were fully welded and the column panel zone was stiffened. Fully welded moment resistant column base plates were bolted to the floor.

The vertical loads were applied to the frame through gravity load simulators that displace laterally with the frame and maintain the alignment of the load in the vertical direction. The vertical loads were applied simultaneously in two increments to a magnitude of $W=89$ kN. The lateral load was applied incrementally to failure and beyond to illustrate the ductility response for full plastification of the hinges that

Figure 4.16: Bracing Points, Arnold et al. (1966)

developed. The characteristic stability behaviour of the frame is illustrated by the horizontal load deflection curve and is given in Figure 4.17. Point a identifies the load level in which the researchers noticed base plate rotation for column 1. Although not stated, the change in stiffness as illustrated by point b probably coincides with the separation of the base plate from the strong floor and has similar characteristics to bolted connections that are subjected to prying.

Figure 4.17: Load vs Deflection Comparison of Test and Predicted Results, Arnold et al. (1966)

The centreline dimension of the frame was modelled with 27 I_BeCol elements for each column and 43 I_BeCol elements for the beam. The elements varied in length from 85.6 to 127.2 mm and were located symmetrically about the line of symmetry shown in Figure 4.16. Nodal points were selected to coincide with load and bracing points. The frame was restrained for out-of-plane displacements at bracing points and column ends were fully fixed. Measured geometric and material properties were used as listed in Table 4.4. Initial out-of straightness and out-of-plumb values were taken as zero.

The horizontal load deflection curve determined from the analysis of the frame is illustrated in Figure 4.17. The test to predicted ratio for the ultimate load of the frame is equal to 0.995. Variations in the horizontal displacements beyond point b in Figure 4.17 are attributed to column base rotations experienced during testing. The minor variation in initial stiffness is of no real consequence and can be easily attributed to a small amount of out-of-plumb for the test and variations in stiffness of the model with respect to the frame.

Currently this prototype element can be used to model frames for second-order inelastic behaviour as long as the connections are fully rigid and the boundary connections are pinned or fixed. Structural engineers could use this numerical model in lieu of a second-order elastic analysis to determine the maximum load effects and the beam-column interaction equations to establish member resistances.

Chapter 5

Summary, Conclusions, Recommendations and Future Work

5.1 Summary and Conclusions

A series of prototype modules for the determination of loads, advanced analysis, and member and connection design for steel structures clearly demonstrated the advantages that would accrue to designers, fabricators, erectors and educators from the development of an Integrated System. Due to the evolutionary nature of standards, codes, engineering knowledge, and computing technologies it is evident that the system architecture for an Integrated System needs to be easily updated and platform independent, which leads to the use of object oriented philosophy and in particular a programming language such as Java. The requirements of the Integrated System that dictate the system architecture are:

- the base structure of the program is founded on an Object Oriented Standard with extended libraries required for basic calculations or section definition
- the associated Graphical User Interface should be common to all design modules and independent of the computational algorithms
- the associated Graphical User Interface should be structured to emulate the design process and reduce complexity to the user

- **dynamic databases be used to ease the transmission of data between system levels and modules**
- **objects be constructed using a class hierarchy that reflects the natural order of building construction and design**

Understanding the requirements of the system, a four level hierarchical system architecture was developed. Level 1, System, contains the Graphical User Interface, defines the building structure, and is responsible for data transfer between system programs. Level 2, System Programs, houses the main applications for the load definition, structural analysis, and member and connection design modules. Level 3, Elements, is a series of element classes that are implemented by the Level 2 system programs. Level 4, Constraints and Properties, services all other levels and contains the Object Oriented Standard, tools libraries, and databases (both those associated with the Object Oriented Handbook and user defined).

To illustrate the versatility and functionality of the system architecture a prototype Object Oriented Handbook (Level 4) and a prototype finite element analysis application (Level 2 and 3) were developed, by Fuller (1999) and the author.

The Object Oriented Handbook houses the Object Oriented Standard, which is a series of Clause Objects that describe the resistances of beams and beam-columns given in CAN/CSA-S16.1 (CSA, 1994) and CAN/CSA-S136 (CSA, 1994), extended libraries, tools libraries, and databases for section properties and selection tables. For new versions of the standard, objects must be created for new Clauses, existing objects must be modified to reflect changed Clauses, and only the modules associated with new, removed, or drastically restructured Clauses would require adjustment.

The requirements of CAN/CSA-S16.1 implemented in the Object Oriented Standard and pertaining to beam-column design are limited in nature and based on a series of assumptions and approximations. In particular, the interaction equations are limited to doubly symmetric class 1, 2 and 3 sections, derived specifically for I-shaped members, and based on plastic theory. The requirements for the determination of member forces, for use in conjunction with the interaction equation, dictate that as a minimum a second-order elastic (or first-order elastic with translational moment amplification) be performed.

The inconsistencies between the specified analysis and resistance and the limitations associated with the interaction equations dictate the need for an advanced analysis tool to be included within the Integrated System. The prototype finite element application was developed to satisfy this need and implements the element objects and methods required to conduct a second-order inelastic analysis that will capture stability failure for class 1, 2, and S sections consisting of plate elements.

To illustrate the versatility and functionality of the simple beam-column element I_BeCol (implemented by the prototype application) for describing beam-column and frame behavior, comparisons were made between the results of the models consisting of these elements with test data on beam-column stability, Van Kuren and Galambos (1964), and frame stability, Arnold et al. (1966). The net test to predicted ratio and coefficient of variation, when compared to tests with varying load and boundary conditions, and all probable failure modes, were 0.935, and 0.099, respectively. The test to predicted ratio for the frame was 0.995.

These results indicate good correlation with the tests and demonstrate this element's ability to capture both member and frame stability and strength. The simultaneous consideration of both member and frame behavior reduces complexity in the design process and eliminates uncertainties associated with assumptions in the current design requirements.

5.2 Recommendations and Future Work

The prototype developments for the object oriented Integrated System clearly establish the advantages of this kind of development. As with any prototype development there are a number of recommendations that can be made which would complete and expand the work.

The Object Oriented Standard, extended libraries, and databases associated with the Object Oriented Handbook should be completed prior to the development of any further design modules. This will permit the development of the independent modules as envisaged by the system architecture and the development of mini applications to illustrate design examples and tables for educational purposes in conjunction with the Hyper Handbook.

The finite element analysis application has demonstrated it's suitability for the intended application and development should be continued to improve its modeling capabilities. Four components have been identified to add value and to improve the

applicability, accuracy, versatility and ease of use of the prototype application. They are:

- a semi-rigid connection element**
- a refined convergence algorithm**
- a more versatile auto-meshing algorithm**
- a method to attribute distributed loads to nodes using the concept of equivalent work**

The first two noted components are directly related to the applicability and accuracy, respectively, of the application and are of primary concern. A literature review was conducted to determine specific recommendations with respect to the requirements of the Integrated System, and is located in Appendix A and Fuller (1999), for semi-rigid connection response and for refined convergence, respectively.

Successful development of a commercially viable integrated system (not prototype exploration) requires substantial development resources and expertise in both engineering and computer science. The developed system architecture will allow for the continued development of the prototype modules in a consistent and integratable format that will facilitate their assembly into a commercial product.

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Glossary

abstract	- A Java reserved word that serves as a modifier for classes, interfaces, and methods. An abstract class cannot be instantiated, and is used to specify bodiless methods that are given definitions by derived methods.
access	- The ability to reference a variable or invoke a method from outside the class in which it is declared. Also called scope or level of encapsulation.
algorithm	- A step by step process for solving a problem.
application	- 1) A generic term for any program. 2) A Java program that can be run without the use of a web browser.
Architectural design	- A high level design that identifies the large portions of a software system.
behaviour	- The functional characteristics of an object, defined by its methods.
black box testing	- Producing and evaluating test cases based on the input and expected output of a software component.
class	- 1) A Java reserved word used to define a class. 2) The blueprint of an object. The model that defines the variables and methods of an object will contain when instantiated.
class diagram	- A diagram that shows the relationship between classes, including inheritance and use relationships.
class hierarchy	- The treelike structure created when classes are derived from other classes through inheritance.
class library	- A set of classes that define useful services for a programmer.
constructor	- A special method in a class that is invoked when an object is instantiated from the class. Used to initialise the object.
data structure	- Any programming construct used to organise data into a format to facilitate access and processing.

encapsulation	-	The characteristic of an object that limits access to the variables and methods contained in it. All interactions with an object occurs through a well defined interface that supports modular design
extends	-	A Java reserved word used to specify the parent class in the definition of a child class.
graphical user interface	-	Software that provides the means to interact with a program or operating system by making use of graphical images and point and click mechanisms.
hierarchy	-	An organisational technique in which items are layered or grouped to reduce complexity.
implements	-	A Java reserved word that is used in a class declaration to specify that the class implements the methods specified in a particular interface.
inheritance	-	The ability to derive a new class from an existing one. Inherited variables and methods of the original (parent) class are available in the new (child) class as if they were declared locally.
instance	-	An object, created from a class. Multiple objects can be instantiated from a single class.
instantiation	-	The act of creating an object from a class.
interface	-	1) A Java reserved word that is used to define a set of bodiless methods that will be implemented by particular classes. 2) The set of messages to which an object responds, defined by the methods that can be invoked from outside of the object. 3) The techniques through which a human user interacts with a program, often graphically.
method	-	A named group of declarations and programming statements that can be invoked when. A method is part of a class.
object	-	1) The primary software construct in the object-oriented paradigm. 2) An encapsulated collection of data, variables and methods. 3) An instance of a class.
object-oriented programming	-	An approach to software design and implementation that is centred around objects and classes.

- package** - **A Java reserved word that is used to specify a group of related classes.**
- reference** - **A variable that hold the address of an object. In Java, a reference can be used to interact with an object, but its address cannot be accessed, set or operated on directly.**
- state** - **The state of being of an object, defined by the values of its data.**

Note: Definitions after Lewis and Loftus (1998)

Appendix A

Semi-Rigid Connection Response

Significant research has been performed in the past to determine the strong axis bending connection responses of various common connections. This work has led to the development of curves fit to test data to represent the moment rotation characteristics of the connections.

Usual design only considers fully rigid or perfectly pinned connections that imply full slope continuity between adjoining members or the beams will behave as simply supported members and the columns will carry no gravity moments (from beams) respectively. These assumptions greatly simplify analysis and design, however, the validity of these assumptions may be questionable since no joint behaves in the prescribed manner. Joint flexibility affects structural response by changing the moment distribution and frame drift. Semi-rigid frames experience larger drift than rigid frames thus increasing the P- Δ effect (Chen and Lui, 1991).

Chen and Lui (1991) have developed a two-noded six d.o.f. frame line element which includes the effect of strong axis joint flexibility at both or either of the member's ends based on the connection's stiffness. They noted that axial and shearing deformations are small and can normally be neglected and that the moment rotation relationship is non-linear over the entire range of loading. The element development process involved the superposition of the standard line element with two rotational spring elements with two rotational d.o.f. each. The outcome of this

process resulted in a three d.o.f. per node hybrid line element, which had compatible d.o.f. with the standard line element. It is proposed that this work be extended to the six d.o.f. beam-column line element implemented in the finite element analysis application discussed in Chapter 4.

The work performed by Chen and Lui was limited to in-plane rotational response only. To capture out-of-plane stability effects torsional and weak axis bending connection response should be considered. However, there does not appear to be adequate test data and research concerning this type of connection response at this time. Ample data exists for the in-plane rotational behaviour of semi-rigid connections and the reader is referred to the following works for a more detailed description: Goverdhan, A.V. (1983), Nethercot, D.A. (1985), Kishi, N. and Chen, W.F. (1986), or Chen, W.F. and Kishi, N. (1989). The abundant test data has been utilised to form empirical expressions for moment-rotations curves. A list of the five basic modelling techniques along with a selected example follows:

- Piecewise linear, Tarpy, T.S. and Cardinal, J.W. (1981)
- Polynomial, Frye, M.J. and Morris, G.A. (1976)
- B-Spline, Jones, S.W., Kirby, P.A. and Nethercot, D.A. (1982)
- Power, Ang, K.M. and Morris, G.A. (1984)
- Exponential, Lui, E.M. and Chen, W.F. (1986)

Linear models are inappropriate since they are not adequately accurate and the sudden change in connection stiffness does not represent the true connection behaviour well. Polynomial models fit the test data very accurately, however they become unstable once outside the range of test data. This suggests that if

implemented in a computer algorithm, once a plastic hinge has developed in the connection, subsequent loading will produce unpredictable response and thus polynomial models not suitable for the intended purpose. The B-spline models fit data extremely well, but require a large data set. This method is also piecewise continuous, which is not desirable.

The exponential model is adequately accurate and predictable in the large deformation range required when doing elastic-plastic analysis, and it becomes asymptotic as deformation increases. The Lui and Chen (1986) model implements a multi-parameter exponential model. The parameters are calculated and measured values that require a significant amount of effort and data to determine and tend to be connection specific.

Of the models presented and discussed in this review, it is recommended that the Ang and Morris (1984) model be applied to determine the connection stiffness parameter to be implemented in the modified hybrid element suggested by Chen and Lui (1991). This model uses a four parameter standardised Ramberg_Osgoode function where the parameters are simple geometric properties of the connections. Similarly to the exponential model, the power model is adequately accurate and predictable in the large deformation range, and becomes asymptotic as deformation increases.

Appendix B

Description of Compact Disk Contents

The source code for the Object Oriented Handbook and the Finite Element Application is stored on the Compact Disc attached to the inside of the back cover. If the compact disk was not provided with your copy and you require the files please contact Prof. S.J. Kennedy at the Department of Civil and Environmental Engineering, Carleton University.

All files are located in the folder named “OOH and Finite Element Application”. Figure B1 illustrates folders and file locations. Files associated with the Object Oriented Handbook and the Finite Element Application are located in the sub folder bearing that name, respectively.

The “Finite Element Application” folder contains the file “Readme.txt” and the “Program” sub folder. The “Readme” file describes the program and provides operating and input file creation instructions. The Program folder includes example input and output files and the “Actual Java Files” sub folder. The “Actual Java Files” folder houses the “.java” files which are the source code for the application.

The “Object Oriented Handbook” folder contains three subfolders.

- tools: code and compiled code of the tools library
- S136: code and compiled code of clauses contained in CAN/CSA-S136
- Clauses: code and compiled code of the Object Oriented Standard (CAN/CSA-S16.1)

The Clause files are named according to clause number. “C_10.java” and “C_5_6.java” are the source code for clauses 10 and 5.6, respectively.

Figure B1: Contents of Compact Disc