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A COST EVALUATION
OF COMPOSITE FLOOR BEAMS
IN HIGH-RISE OFFICE CONSTRUCTION

by

Andrew I. Allan, B.Eng.

A thesis submitted to the Faculty
of Graduate Studies and Research in
partial fulfillment of the require-
ments for the degree of Master of
Engineering.

Faculty of Engineering

Carleton University

Ottawa, Ontario

June 1978

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The undersigned recommend to the Faculty of Graduate Studies and Research acceptance of the thesis "A Cost Evaluation of Composite Floor Beams in High-Rise Office Construction" submitted by Andrew I. Allan, B.Eng. in partial fulfillment of the requirements for the degree of Master of Engineering.

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Abstract

Faced with the complexity of modern building design, the designer encounters difficulties and increased risks in making decisions based purely on his instinct and experience. The designer should supplement experience with quantitative evaluations of individual components of the building where ever possible.

This work develops an approach to the evaluation of floor systems in high-rise steel framed buildings. This procedure is based on construction costs determined from a cost survey of the Ottawa region in February and March of 1978. It accounts for increases in total structural cost due to depth and weight of the floor system as well as the cost of the floor system itself. The evaluation also includes changes in non-structural costs due to variation in storey height. This floor system evaluation procedure is used to evaluate different composite floor constructions in high-rise office buildings.

The study of these different composite floor systems provides a basis from which the approach can be applied to other buildings and floor systems. Several design recommendations for composite floor construction are presented in this study.

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Table of Contents

	<u>Page</u>
1. Introduction	1
1.1 General.....	1
1.2 Objective.....	2
1.3 Scope.....	3
2. Floor System Evaluation Procedure	5
3. Evaluation of Composite Beam Floor Systems.....	15
3.1 Building Scheme.....	16
3.2 Floor Performance Requirements.....	19
3.3 Alternative Floor Designs.....	20
3.4 Cost Information.....	22
3.5 Building-Floor Interaction	27
3.6 Evaluation of Floor Systems.....	39
4. Conclusions and Recommendations	54
4.1 Conclusions.....	54
4.2 Recommendations for Further Study.....	56
Appendix A - Cost Documentation.....	58
Appendix B - CISC Structural Steel Costing Method.....	88
Appendix C - Structural Steel Building Skeleton.....	95
Appendix D - Building Skeleton Costs.....	170
Appendix E - Floor Slab Costs.....	181
Appendix F - Composite Floor Beams.....	193
References.....	213

List of Tables

<u>Table</u>		<u>Page</u>
1	Formed Concrete Slabs.....	23
2	Steel Deck and Cover Slab.....	23
3	Building Designs.....	29
4	Formed Slab Building Skeleton Costs.....	31
5	Steel Deck Building Skeleton Costs.....	31
6	Floor Beam Designs and Costs.....	41
7	Building Skeleton Costs.....	49
8	Total Structural Costs.....	52
9	Openings in Beams B6 and B7.....	53
10	Comparison of Floor Evaluation Methods.....	55
11	Comparison of Beam Costs for 35 ft. and 40 ft. Spans	57

List of Figures

<u>Figure</u>		<u>Page</u>
1	Requirements of a Floor System.....	7
2	Formed Slab Building Scheme	17
3	Steel Deck Building Scheme.....	18
4	Standard Rolled WF.....	21
5	Standard WF and Cover Plate.....	21
6	Standard WF and Shallow Haunch.....	21
7	Plot of C_{BS} vs DL for Formed Slab Scheme.....	32
8	Plot of C_{BS} vs DL for Steel Deck Scheme.....	33
9	Plot of ΔC_{BS} vs S for Formed Slab Scheme.....	34
10	Plot of ΔC_{BS} vs S for Steel Deck Scheme.....	35
11	Depth of Floor System.....	50

Table of Nomenclature

C_{BS}	Building skeleton cost
DL	Floor system dead load
f'_c	Concrete compressive strength
F_y	Yield stress of steel
i	Rate of discount per year
INR	Impact noise rating
l_i	Span length of beam or slab
n	Life span of project in years
R	Correction coefficient
S	Storey height
STC	Sound transmission class rating
u	Thermal coefficient of transmission
w	Unit weight of concrete
ΔC_{BS}	Building skeleton cost adjustment
ΔT	Temperature differential

1 - INTRODUCTION

1.1 General

Today's designer is faced with an ever increasing challenge. He must cope with accelerating technology, changing design philosophies (Limit States Design) and unstable construction costs. Added to this is the Canadian conversion to the metric International System of Units (SI). Faced with all this complexity, a decision maker encounters difficulties and increased risks in making up his mind purely on his instinct and experience.

Recognizing that intuition and experience will always form the basis for conceptual design, the designer must learn to supplement these with quantitative evaluations of individual components where ever possible. The structural system may be divided into two major components, gravity load resisting systems and lateral load resisting systems. The gravity load resisting system may be further subdivided into horizontal components, or floor systems, and vertical components. The lateral load resisting system and the vertical component of the gravity load resisting system combine to control the overall framing scheme for the building. This framing scheme is determined through the complex interaction of the various building disciplines (Architectural, Structural, Geotechnical,

2

Mechanical and Electrical). Generally one framing concept can accommodate many different floor systems, this is especially true in steel frame buildings and to a lesser extent in reinforced concrete buildings.

The initial framing scheme must still be considered within the realm of instinct and experience of the design team. However, the selection of the floor system should be based on a detailed cost evaluation. In multistorey steel framed buildings the floor system represents approximately fifty percent of the total structural cost. Therefore, any savings in the cost of the floor system will result in a significant reduction of the total structural cost.

1.2 Objective

This study will attempt to provide the structural designer with a rational approach to the design of floor systems. The objective of this thesis is twofold:

1. Develop a procedure to evaluate and compare floor systems in high-rise steel construction.
2. Apply this procedure in the evaluation of composite beam floor systems in a high-rise office building.

The term high-rise shall refer to any building four or more storeys high. Below four stories the benefits derived from this detailed evaluation procedure do not justify its

3

use and a simpler evaluation should be performed. However, as the number of storeys increases the importance of an accurate evaluation grows.

1.3 Scope

The structural frame for a commercial high-rise building may represent 15-20% of the project cost. Unlike heavy industrial structures designed as an integral part of processing equipment, where structural cost represents a small part of the total capital cost, the viability of commercial projects is strongly influenced by structural cost. In high-rise steel construction one framing concept can accommodate many different floor systems (1). For these reasons the floor evaluation procedure has been tailored for use in high-rise steel construction.

In recent years, the trend in high-rise office buildings has been to provide larger column free spaces. This allows greater flexibility in office layout and the use of the "open office planning" concept. This design consideration has led to the development of a number of structural floor systems which span directly from the core to the exterior walls.

Composite floor beams are considered most efficient for relatively long spans, with the beams spaced as far apart as practical. Composite beams are also twice as stiff as comparable non-composite beams. These characteristics suit the composite beam for use in floor systems of high-rise office buildings.

The evaluation procedure will be used to determine which composite floor system is the most cost effective. Unfortunately, it is impossible to evaluate these floor systems without reference to a specific building because the evaluation is highly dependent on the building-floor interaction. The building has been chosen to be representative of contemporary office towers in Canada. This study will provide a basis for further applications of the floor system evaluation procedure and insight into economic composite construction.

2 - FLOOR SYSTEM EVALUATION PROCEDURE

The evaluation procedure evolved from the systems approach in building design outlined by Haider and Khachaturian (2). The process described by Haider and Khachaturian is a general approach to the design of buildings. The basic design philosophy they present was used to develop a working evaluation procedure for floor systems, which could be incorporated into the structural design process.

The following six steps form the basis of the evaluation procedure:

1. Identify a building scheme.
2. Specify floor performance requirements.
3. Generate alternative floor designs.
4. Obtain cost information.
5. Determine building-floor interaction.
6. Evaluate floor systems.

Each of these steps will now be explained in detail.

1. Identify a building scheme.

The first task is to identify the owner's requirements, local building and zoning laws, and any constraints the building site may impose. Next, the design team must determine a feasible building scheme that satisfies these requirements.

This will be based on their knowledge of the construction industry in their region. It is during this stage that cooperation within the design team is most essential, because it is here that total building efficiency and much economy can be achieved. Trade-offs between building disciplines may have to be made here in the interest of overall economy.

If more than one preliminary building scheme is selected, each must be evaluated separately. Fortunately, many authors (1,3,4,5) have discussed the relative economies of various framing concepts for high-rise structures. The designer is advised to consult these studies to aid him in his selection of a suitable building scheme.

2. Specify floor performance requirements.

Before alternative floor designs are generated it is important to be aware of the various performance requirements or functions that a ceiling to floor sandwich must fulfill. These should be determined from the owner's requirements with reference to local building codes. Outlined below are the major functions a floor system should be expected to satisfy (Fig. 1).

Loads: The primary structural function of a floor system is to resist vertical loading. A common secondary requirement in most buildings is to provide diaphragm action to transfer

Fig. 1 REQUIREMENTS OF A FLOOR SYSTEM (6)

lateral loading to cores or other stiffening elements of the building.

Ceiling: Light fixtures and other items usually must be suspended from, or attached to, the floor system. Also, an acceptable ceiling appearance must be obtained. To fulfill these requirements, it is necessary to restrict live load deflections and sometimes even deflections under dead and construction loads.

Mechanical Distribution: Mechanical systems in many buildings require vertical ducts or pipes to pass directly through the floors. In others, ventilation ducts are located horizontally and feed downward to diffuser outlets.

Electrical Distribution: Most electrical distribution systems in use today require horizontal runs somewhere between walking surface and the finished ceiling. Upward or downward connections (depending on the system employed) are required to service electrical outlets, telephones etc. and downward connections service lights.

Fire: The system itself must not be combustible and must not allow transmission of smoke or excessive heat through it.

For a steel system, resistive layers (fire proofing) must limit the temperature rise of steel components.

Walking Surface: The top surface is required to be sufficiently rugged to handle construction traffic, yet it must be smooth for application of the final floor finish.

Vibration: Floors of light construction, long spans and little inherent damping have sometimes resulted in problems of objectionable floor vibrations during normal human activity. It is important that floor vibrations do not exceed certain thresholds of annoyance.

Sound: Floor systems for buildings should have adequate resistance to transmission of both airborne and impact sound.

The more easily a system satisfies basic structural requirements while at the same time providing for easy solutions of other, non-structural functions, the more marketable it is likely to be.

These performance requirements should now be translated into design specifications. These will take the form of: design loads, allowable deflections, constraints imposed by mechanical and electrical systems, required fire resistance rating, and acceptable levels for vibration and sound transmission. The floor system should be designed to meet these specifications on a pass-fail basis. The pass-fail method was selected over a rating method where a system might receive "bonus points" for exceeding specifications. Unless the owner will receive increased revenue as a result of performance beyond specifications there is no incentive for him to invest money above that needed to provide a floor system that meets his requirements.

3. Generate alternative floor designs.

The designer must now decide on the floor constructions and layouts to be investigated. Through his experience and knowledge of the industry he can identify the designs, which satisfy floor requirements and suit the framing scheme, that are worthy of further consideration. For steel high-rise construction there will generally be several floor systems which merit further study.

At this stage, preliminary floor designs are sufficient. The arrangement of floor members to be examined in the different designs should be established. The following design parameters to be investigated must also be set:

- I) Type of Construction - non-composite
 - composite - shored
 - composite - unshored
- II) Type of Members - W-shapes
 - steel joists
 - built-up sections
- III) Floor Slab
- IV) Material Properties
- V) Method of Fire Protection
- VI) Mechanical and Electrical Integration with Floor

The designer may wish to select only one floor framing scheme and determine which parameters produce the most economic design. Conversely, he may arbitrarily fix the design parameters

and examine several floor framing schemes, or he could do both. It is important that the designer pay special attention to this phase of the evaluation, for in the selection of these conceptual designs he sets the bounds for the study. The designer should be encouraged to investigate new concepts but he can save time and money by eliminating uneconomic designs here rather than proceeding with their fruitless evaluation for the sake of completeness.

4. Obtain cost information.

The different floor systems will be evaluated by comparing the total structural cost of the building with each of the floor systems. This cost includes the structural framing, foundations and fire protection. There must also be an adjustment made to account for any non-structural costs that vary with the storey height. If the results of the investigation are to be considered valid, it is essential that current and accurate cost data be obtained.

Material, fabrication and construction costs will be required to determine the in-place unit costs necessary for calculation of the total structural cost. The designer must ascertain which unit costs to seek out from the materials and constructions involved in his study. Non-structural costs for vertical mechanical and electrical runs, interior and exterior walls, and life cycle heating and air conditioning of the building will be required to account for variations in storey height.

There are five major sources of cost information and these groups are listed below.

I) Published Data:

- estimating guides
- published papers

II) Government Departments:

- Statistics Canada
- Department of Public Works (DPW)
- Department of Industry, Trade and Commerce

III) Trade Organizations:

- Canadian Institute of Steel Construction (CISC)
- Portland Cement Association (PCA)
- Reinforcing Steel Institute of Ontario (RSIO)

IV) Consultants:

- Structural Engineers
- Mechanical and Electrical Engineers
- Architects
- Quantity Surveyors

V) Industry

The most accurate estimates on unit costs can be obtained from the industry itself (suppliers and contractors). If this proves difficult the other sources can provide valuable information.

5. Determine building-floor interaction.

Now the Building is divided into two parts, the building skeleton (lateral force resisting system, columns, and spandrel beams) and the floor system. The spandrel beams are considered part of the building skeleton because they tie the exterior columns together forming the exterior structural shell as well

as supporting the exterior cladding. The spandrel beams are special elements which conform to requirements different from those outlined as floor performance requirements. The effect of floor parameters on the cost of the building skeleton can now be examined and the interaction between the two defined.

There are only two properties of the floor system which have a significant effect on the cost of the building skeleton. These are the weight and depth of the floor to ceiling sandwich. A good approach is to establish the cost of the building skeleton as a function of the dead load of the floor system. Then adjust the cost of the building skeleton to account for the different storey heights caused by the various floor to ceiling depths of the designs being evaluated. This storey height adjustment function must account for changes in non-structural costs such as vertical mechanical and electrical runs, interior and exterior walls, and life cycle heating and air conditioning of the building as well as structural costs of the skeleton. A base storey height must be selected, say the lowest or the average, so that the changes in cost can be determined relative to this height. The cost of the building skeleton as a function of floor dead load should be calculated using this base storey height. It is unrealistic to compare or evaluate floor systems of different depths and weights without ascertaining their effect on the total structural cost.

6. Evaluate floor systems.

This final stage can be broken down into detailed design and costing of the floor systems, and evaluation of the different designs. The preliminary designs outlined in the generation of alternative floor designs must now be translated into a set of final designs to enable a cost evaluation of the different floor layouts and design parameters. Utilizing the unit costs obtained in step 4 determine the cost of each floor design. The cost of floor systems of similar weight and depth may be compared and the more costly designs eliminated. Now the cost of the building skeleton for each of the floor systems is determined from its depth and weight, using the relationships defined in step 5. The total structural cost of the building for each floor system can now be calculated and compared.

The evaluation phase involves reviewing of the costs derived for each floor design and a final check to ensure that all performance requirements have been met. The designer should also outline any inherent unquantifiable advantages or disadvantages of each floor system. These unquantifiable entities can then be evaluated against cost advantages and a decision made by the designer on which floor to select.

3 - EVALUATION OF COMPOSITE BEAM FLOOR SYSTEMS

There are three basic elements which make up a composite beam: steel section, slab construction, and shear connection. The steel section resists the tensile forces due to bending and the shear forces, as well as supporting the formwork (permanent or temporary) for the slab during construction. The slab construction has two main structural functions to span between beams, and to act as the top flange of the composite beam. The function of the shear connector is to provide adequate horizontal shear capacity and provide uplift resistance to hold the slab down to the steel section.

For the reasons outlined in the introduction, composite floor systems are being used extensively in high-rise office buildings. A floor construction of steel deck with cover slab over beams at 10 ft. spacings which span from core to exterior wall is the current industry standard. If an innovative, efficient forming system utilizing a 15 ft. beam spacing could be developed, a formed concrete slab could provide an economic alternative. An evaluation of these two systems will determine the forming cost which will make the formed slab construction competitive.

If it were possible to start from first principles in conceiving an ideal steel section as a component of a composite floor beam, the answer would certainly not be the W-shapes now in common use. Except as a location for studs, the top flange

of the usual steel section contributes little to the strength or stiffness of the composite beam. To overcome the inefficiency of the W-shape, several other steel sections have been proposed in recent state of the art discussions (6,7).

The economic advantages derived from some of these new concepts will be investigated in this study.

3.1 Building Scheme

The building used in this study (Fig. 2) has been chosen to be representative of contemporary office towers in Canada. The framing scheme has been selected to be compatible with long span composite beams. The dimensions of the building are consistent with the 5 ft. module currently used in office buildings. The ceiling grids and partition layouts of most offices are being calculated on this module.

The building scheme for the formed concrete slab with beams at 15 ft. centers is shown in Fig. 2. The building scheme must be modified slightly to accommodate the 10 ft. spacing of beams with steel deck. The core of the building must be shifted 5 ft. as shown in Fig. 3.

The example building is 20 storeys high, having plan dimensions 115 ft. by 150 ft., yielding: a gross floor area of 17,250 sf., a total area of 345,000 sf., and a ratio of core area to gross area of 12 per cent.

Fig. 2 Formed Slab Building Scheme

Fig. 3 Steel Deck Building Scheme

3.2 Floor Performance Requirements

Presented below are the design specifications for the floor system of the office building being considered. These were determined through the identification of performance requirements as outlined in chapter 2.

Loads:

Dead Loads -	Mechanical & Electrical	5 psf.
	Ceiling	5 psf.
	Flooring	<u>2.5 psf.</u>
		12.5 psf.
	+ dead load of floor system	

Live Loads -	Use and Occupancy	50 psf.
	Movable Partitions	<u>20 psf.</u>
		70 psf.

Construction -	Formwork (formed slab)	10 psf.
Loads	Workman	10 psf.

Deflections:

Allowable immediate deflection due to live load = $\ell/360$
(ℓ = span length of beam or slab).

Allowable long-term deflection, the sum of the long-term deflection due to all sustained loads and the immediate deflection due to live load = $\ell/240$ (sustained live load accounts for 40% of the total live load).

If unshored construction is used the steel section must be cambered to ensure uniform thickness of concrete slab.

Mechanical and Electrical Distribution:

The mechanical system for the example building will be a variable volume system, utilizing a 10 ft. grid along the 150 ft. dimension with three rows of units per bay. Provisions must be made for a 20 in. deep variable volume unit with a 10 in. diameter duct (12 in. with insulation) running between units, located 2 in. below the top of the unit. The bottom of this unit is located flush with the underside of the finished ceiling.

A 4 in. clear space from the underside of the ceiling must be provided to accommodate the light fixtures.

Fire:

The floor assembly shall be of noncombustible construction and requires a 2 hour fire resistance rating.

Vibration:

The floor vibrations should fall below the threshold of annoyance outlined in CSA S16.1-1974, Appendix G, Fig. G3 (8). The floor system is assumed to have 6% damping.

Sound:

The floor construction should provide a sound transmission class rating of not less than "STC 45" (9) and a FHA impact noise rating greater than "INR 0" (9).

3.3 Alternative Floor Designs

This study will investigate the relative economies of the formed concrete slab scheme (Fig. 2) vs. the steel deck scheme (Fig. 3). It will also determine if any savings can be achieved through modifications to the standard W-shape (44W grade steel). Concrete slab design parameters such as unit weight of concrete, strength of concrete, and slab thickness will also be examined.

The cost of the composite floor system using W-shapes and G40.21-44W steel will be calculated and compared with the cost of steel sections utilizing the following modifications (Fig. 4,5,6):

- I) High-strength steel ($F_y = 50 \text{ \& } 60 \text{ ksi}$)
- II) Cover plates
- III) Shallow haunch in concrete slab (3 in. deep)
- IV) Openings in steel section for mechanical ducts.
- V) Construction methods (shored vs. unshored)

Fig. 4 STANDARD ROLLED WF: a) opening, no opening
b) shored, unshored

Fig. 5 STANDARD WF & COVER PLATE

Fig. 6 STANDARD WF & SHALLOW HAUNCH

To investigate the effects of slab design parameters on cost, the floor slabs listed in Tables 1 and 2 will be combined with the steel sections outlined previously. These slabs were chosen to represent the various combinations of unit weight of concrete, concrete strength and thickness of slab.

The method of shear connection will be limited to 3/4 in. diameter shear studs. The welded shear stud connector is the best and most economical available to date and for this reason it will be the only shear connection considered. Allowance will be made for partial shear connection, the percentage connection required will be determined in the design process.

3.4 Cost Information

Presented in this section are the results of a survey of construction costs in the Ottawa region, conducted during February and March 1978. The costs summarized here are documented in Appendix A.

Structural Steel Costs

Structural Steel Members:

The CISC piece by piece costing method (10) was used to determine the cost of the structural steel frame. This method is reproduced in Appendix B. The following cost coefficient values necessary for the method were obtained from the CISC.

B - Modified base prices

- B_1 (for wide flanges) = 12.69 \$/100 lb.
- B_2 (for WWF > 140 plf.) = 17.24 \$/100 lb.
- B_3 (for WWF < 140 plf.) = 16.72 \$/100 lb.

Table 1 Formed Concrete Slabs

Slab No.	Slab Thickness (in.)		Weight of Concrete (pcf.)	Concrete Strength (ksi)
	Exterior Span	Interior Span		
1	5.0	4.5	115	4
2	5.0	5.0	150	3
3	5.5	5.0	115	3
4	5.0	5.0	150	4
5	5.0	5.0	115	4
6	5.0	5.0	150	5
7	5.5	5.5	150	3
8	5.5	5.5	115	3
9	5.5	5.5	150	4
10	5.5	5.5	115	4
11	5.5	5.5	150	5

Table 2 Steel Deck* and Cover Slab

Slab No.	Cover Slab Thickness (in.)	Weight of Concrete (pcf.)	Concrete Strength (ksi.)	Spray on Fire Protection
12	2.5	150	3	Y
13	2.5	150	4	Y
14	2.5	150	5	Y
15	3.25	115	3	N
16	3.25	115	4	N

* .036 in. thick, Westeel-Rosco T-30V Hi-Bond, 3" composite steel deck was used with all the cover slabs.

F - Freight charges mill to fabricator

- F_1 (for W and WWF sections) = .82 \$/100 lb.

FST (Federal Sales Tax) = 5%

GE (Grade extra) - see Appendix A

GP (Sales overhead and gross profit) = 6%

PST (Provincial Sales Tax) = 7%

Q_D (Engineering and drafting coefficient) = 1.45

Q_E (Erection cost coefficient) = 2.4

Q_M (Material cost coefficient) = 1.08

Q_S (Shop fabrication coefficient) = 2.4

R_D (Hourly drafting rate) = 15.00 \$/hr.

R_E (Basic erection crew rate) = 863 \$/day

R_S (Shop fabrication hourly rate) = 8.71 \$/hr.

S (Shipping rate from fabricator to site) = .40 \$/100 lb.

SE (Section extra) - see Appendix A

ρ (Productivity factor) = 1.0

Cover Plates:

Material costs - see Appendix A

Fabrication of light beam cover plate = .2 hr./ft.
($< .75$ in. thick)

Fabrication of heavy column cover plate = .3 hr./ft.

Openings in Beams:

Reinforced, 16 in. diameter = 6 hr./opening

Reinforced, 12 in. diameter = 4.5 hr./opening

Cambering of Beams:

W14 = 50 \$/beam
 W16 = 65 \$/beam
 W18 = 90 \$/beam
 W21 = 115 \$/beam
 W24 = 140 \$/beam
 WWF28 = 165 \$/beam

Composite Construction:

Shear Studs - shop welded = .80 \$/stud
 (3/4" dia. x 4.5")

- field welded = 1.10 \$/stud

Temporary shoring to = 5 \$/jack
 composite beam

Base Plates:

A simplified CISC method of estimating the cost of structural steel (11) was used to determine the cost of base plates.

Cost = Net Weight x Connection Factor x Cost Factor x Cost Index

Connection Factor = 1.02

Cost Factor = 1.2

Cost Index = 800 \$/Ton

Foundation Costs

The following unit costs for foundations include the cost of concrete in place, forms, and reinforcing steel. The costs were taken from "Means Building Construction Cost Data 1978" (12).

Footings, spread, over 5 cy. = 90 \$/cy.

Foundation, mat over 20 cy. = 68 \$/cy.

Fire Protection Costs

Spray on Fire Protection = .25 \$/sf. of contact area
for beams and deck

Encase columns in Drywall = 11.00 \$/lf.

Extra layer of drywall each = .80 \$/sf.
 side of core wall to fire
 proof interior braced frames

Concrete Slab Costs

Normal Weight Concrete (W = 150 pcf.):

3000 psi. concrete = 34.75 \$/cy.

4000 psi. concrete = 37.10 \$/cy.

5000 psi. concrete = 41.10 \$/cy.

Semi-Lightweight Concrete (W = 115 pcf.):

3000 psi. concrete = 63.50 \$/cy.

4000 psi. concrete = 66.40 \$/cy.

Hoist and place concrete = 8.00 \$/cy.

Finishing of concrete = .12 \$/sf.

Form slab between beams = 1.50 \$/sf.

Premium to form 3 in. haunch = .05 \$/sf.
along beams

Supply and place slab reinforcing bars = 385 \$/Ton

Wire mesh supply cost:

6" x 6" - 10 x 10 = 56 \$/1000 sf.

6" x 6" - 8 x 8 = 80 \$/1000 sf.

6" x 6" - 6 x 6 = 102 \$/1000 sf.

Wire mesh placing cost = .06 \$/sf.

Steel Deck Costs

3" composite deck, .036" thick = .94 \$/sf.

Cut and flash at columns = 3.50 \$/column

Concrete retaining angle = .83 \$/ft.
around perimeter

Non-Structural Costs

Cladding:

Precast concrete panels = 7.90 \$/sf.

12" cavity wall = 5.45 \$/sf.

Face brick and stud wall = 4.35 \$/sf.

Aluminium curtain wall = 6.60 \$/sf.

Partitions:

Drywall on studs = 2.00 \$/sf.
(1 hr. fire rating)

6" block wall = 2.35 \$/sf.

Vertical Access:

Cost for a steel stair per foot increase in storey height = 30 \$/ft.

Cost for an elevator per foot increase in storey height = 150 \$/ft.
(78% material, 22% labour)

Mechanical and Electrical:

Mechanical and electrical vertical elements = 500 \$/ft.

The basis for calculation of increased life cycle heating, ventilating, and air conditioning cost are given in Appendix A. The calculations are carried out in section 3.5.

3.5 Building - Floor Interaction

The first task is to determine the cost of the building skeleton (including foundations and fire protection) as a function of the dead load of the floor system. To accomplish this, three versions of the building scheme must be analyzed and designed. An average floor to floor height is selected as the base storey height for the designs but three different floor dead loads are used: an average, an upper bound, and a lower bound. A quantity take-off and cost analysis of these three designs will yield three points on a cost vs. weight of floor system plot. A line representing the cost as a function of the dead load can now be fit to the points using the method of least squares.

The storey height of the building is the sum of the required architectural height (floor surface to underside of ceiling) which is fixed and the depth of the floor system which is variable. The building skeleton cost calculated using the floor dead load must now be modified to account for the effect of storey height (depth of floor system). In addition to the building with the base storey height and the average dead load two more versions must be designed, both with the average dead load but one with an upper bound storey height and the other with a lower bound storey height. From a quantity take-off and cost analysis of these designs, the change in cost of the building skeleton due to storey height can be determined. The change in cost for the base storey height is zero and cost adjustments for other storey heights can be calculated using linear interpolation between the three points. Non-structural cost changes due to the depth of the floor system must also be calculated and accounted for in the modification of the building skeleton cost.

The five different buildings shown in Table 3 were analyzed, designed, and costed, for the formed concrete slab building scheme, in Appendix C and Appendix D. Appendix C presents a design brief for Building No. 1 outlining the design assumptions and methods. Also included in Appendix C are summaries of the building skeleton designs and estimates of the structural steel costs. In Appendix D the cost of base plates, foundation, and fire protection are determined and added to the structural

Table 3 Building Designs

Building No.	Storey Height (S) (ft.)	Floor System Dead Load (DL) (psf.)
1	12.5	80
2	11.5	80
3	13.5	80
4	12.5	100
5	12.5	60

steel costs yielding the building skeleton costs which are summarized in Table 4. For the steel deck building scheme the designs were altered and the costs adjusted as shown in Appendix D and summarized in Table 5.

The line defining the relationship between dead load and building skeleton cost can now be calculated for both schemes (Fig. 7 and 8). The cost adjustment to account for storey height can also be determined as shown in Fig. 9 and 10.

Storey height also affects the cost of certain non-structural elements such as: cladding, partitions, vertical access, mechanical and electrical vertical runs, and heating ventilating and air conditioning loads. Each of these elements will now be examined.

Cladding: For any increase in floor to floor height the wall panel will increase and the window size remain constant. An average cost of the various wall types will be used in this study. For simplicity it is assumed that all costs increase in a direct basis, this is true of masonry, but with precast concrete panels or curtain wall installed on a per piece basis, field labour and erection costs per square foot may actually decrease. The perimeter of the building is 530 ft.

Precast concrete panels	=	7.90 \$/sf.
12" cavity wall	=	5.45 \$/sf.
Face brick and stud wall	=	4.35 \$/sf.
Aluminium curtain wall	=	<u>6.60 \$/sf.</u>
Average	=	6.08 \$/sf.

Cladding cost = $530 \times 6.08 = 3220$ \$/ft. of height

Table 4 Formed Slab Building Skeleton Costs

Building No.	Structural Steel (\$1000)	Foundations (\$1000)	Base Plates (\$1000)	Fire Protection (\$1000)	Building Skeleton (\$1000)	Building* Skeleton (\$/sf.)
1	956.5	90.1	32.4	116.8	1196	3.47
2	900.7	85.2	30.1	109.8	1126	3.26
3	1038.6	91.3	34.6	123.8	1288	3.73
4	1030.2	102.4	39.8	116.8	1289	3.74
5	882.4	77.7	25.7	115.9	1102	3.19

Table 5 Steel Deck Building Skeleton Costs

Building No.	Structural Steel (\$1000)	Foundations (\$1000)	Base Plates (\$1000)	Fire Protection (\$1000)	Building Skeleton (\$1000)	Building* Skeleton (\$/sf.)
1	929.5	90.1	32.4	115.4	1167	3.38
2	873.7	85.2	30.1	108.3	1097	3.18
3	1016.7	91.3	34.6	122.4	1265	3.67
4	1013.1	102.4	39.8	115.5	1271	3.68
5	856.3	77.7	25.7	115.3	1075	3.12

* based on a gross area for the building of 345,000 sf.

Building No.	Floor System Dead Load (psf.)	Building Skeleton Cost (\$/sf.)
5	60	3.19
1	80	3.47
4	100	3.74

Fig. 7 Plot of C_{BS} vs DL for Formed Slab Scheme

Building No.	Floor System Dead Load (psf.)	Building Skeleton Cost (\$/sf.)
5	60	3.12
1	80	3.38
4	100	3.68

Fig. 8 Plot of C_{BS} vs DL for Steel Deck Scheme

Building No.	Storey Height (ft.)	Building Skeleton Cost (\$/sf.)	Building Skeleton Cost Adjustment (\$/sf.)
2	11.5	3.26	-.21
1	12.5	3.47	0
3	13.5	3.73	+.26

Fig. 9 Plot of ΔC_{BS} vs S for Formed Slab Scheme

Building No.	Storey Height (ft.)	Building Skeleton Cost (\$/sf.)	Building Skeleton Cost Adjustment (\$/sf.)
2	11.5	3.18	-.20
1	12.5	3.38	0
3	13.5	3.67	+.29

Fig. 10 Plot of ΔC_{BS} vs S for Steel Deck Scheme

Partitions: Movable partitions in the office areas are not considered. These generally do not extend above the ceiling. The example building has 100 ft. of 6" block wall around the elevator shafts and 400 ft. of drywall-stud partitions enclosing the core braced frames and forming the permanent partitions in the core. The extra layer of drywall required for fire rating the core braced frames was included in fire protection costs.

$$6" \text{ block wall} = 100 \times 2.35 \text{ \$/sf.} = 235 \text{ \$/ft.}$$

$$\text{Drywall partitions} = 400 \times 2.00 \text{ \$/sf.} = \underline{800 \text{ \$/ft.}}$$

$$\text{Partition Cost} = 1035 \text{ \$/ft. of height}$$

Vertical Access: A building of this size requires 2 stairs and 6 elevators. Only material costs will be included for elevators as most of the field labour is required at the ends of components and does not increase with length.

$$\text{Stairs} = 2 \times 30 \text{ \$/ft.} = 60 \text{ \$/ft.}$$

$$\text{Elevators} = 6 \times .78 \times 150 \text{ \$/ft.} = \underline{702 \text{ \$/ft.}}$$

$$\text{Vertical Access Cost} = 762 \text{ \$/ft. of height}$$

Mechanical and Electrical Vertical Runs: The list below includes the major vertical elements for an office building of this size.

$$\text{Vertical air ducts} = 4 \times 60 \text{ \$/ft.} = 240 \text{ \$/ft.}$$

$$\text{Toilet exhaust} = 2 \times 10 \text{ \$/ft.} = 20 \text{ \$/ft.}$$

$$\text{Chilled water \& return} = 2 \times 10 \text{ \$/ft.} = 20 \text{ \$/ft.}$$

$$\text{Heating water \& returns} = 16 \times 8 \text{ \$/ft.} = 128 \text{ \$/ft.}$$

$$\text{Domestic water supply} = 5 \times 6 \text{ \$/ft.} = 30 \text{ \$/ft.}$$

$$\text{Domestic drainage} = 2 \times 4 \text{ \$/ft.} = 8 \text{ \$/ft.}$$

Storm water drainage = 1 x 4 \$/ft. = 4 \$/ft.

Fire hose & sprinkler riser = 2 x 5 \$/ft. = 10 \$/ft.

Bus duct = 1 x 40 \$/ft. = 40 \$/ft.

Mech. & Elec. Vertical Runs Cost = 500 \$/ft. of height

Heating, Ventilating, and Air Conditioning (HVAC): The

increase in storey height will result in additional HVAC loads.

These must be calculated and the yearly costs reduced to an equivalent capital cost (net present value). The following HVAC costs are calculated for a height increase of 1 ft. The additional heating load will be caused by increased transmission loss. Assume a coefficient of transmission (u) of .1, the temperature differential (ΔT) for Ottawa is 96°F.

$$\text{Heat loss} = \text{Area} \times u \times \Delta T$$

$$= 530 \times .1 \times 96 = 5088 \text{ BTU/hr.}$$

$$\text{Heating cost} = 5088 \text{ BTU/hr.} \times \frac{\$500}{80000 \text{ BTU/hr.}} = 31.80 \text{ \$/yr.}$$

Internal cooling characteristics will be unchanged leaving only the change in skin area which is affected by solar radiation and temperature differential. The first acts primarily and instantaneously through the glazing while on well constructed walls solar effect will be delayed 3 to 5 hours so that heat gain in a zoned system will make itself felt in an off peak period. For this reason we will consider only temperature difference as contributing to additional cooling load. There are approximately 1000 equivalent full load hours per year for cooling systems in Ottawa.

$$\text{Cooling loss} = \text{Area} \times u \times \Delta T$$

$$= 530 \times .1 \times 15 = 795 \text{ BTU/hr.}$$

$$\text{Cooling Cost} = \frac{795 \text{ BTU/hr.} \times 1000 \text{ hr.}}{12000 \text{ BTU/Kwh}} \times .025 \text{ \$/Kwh} = 1.66 \text{ \$/yr.}$$

The ventilation fans operate an average of 2500 hours per year.

Due to friction losses, the output requirement for these fans will increase .16 HP (1 HP = 1 running Kw) per foot of supply duct (2 ducts required for the building).

$$\text{Ventilation Cost} = .32 \text{ Kw} \times 2500 \text{ hr.} \times .025 \text{ \$/Kwh} = 20.00 \text{ \$/yr.}$$

The sum of these annual costs must be reduced to an equivalent capital cost. A discount rate of 10% will be used but energy costs are predicted to rise at a rate of 6% above inflation which yields a net discount rate of 4%. Inflation effects can be ignored if income and costs rise equally due to inflation. The net present value of the annual payments will now be calculated using a net discount rate (i) of 4% for a 50 year life span (n).

$$\begin{aligned} \text{HVAC life cycle cost} &= \text{annual cost} \times \frac{(1+i)^n - 1}{i(1+i)^n} \\ &= 53.46 \times \frac{1.04^{50} - 1}{.04 \times 1.04^{50}} \\ &= 1150 \text{ \$/ft. of height} \end{aligned}$$

These non-structural costs can now be totalled and the cost per gross square foot (17250 sf. per floor) per foot of storey height determined.

Cladding	= 3220 \$/ft.
Partitions	= 1035 \$/ft.
Vertical Access	= 762 \$/ft.
Mechanical & Electrical Vertical Runs	= 500 \$/ft.
Life Cycle HVAC	= <u>1150 \$/ft.</u>
Total Non-Structural Costs	= 6667 \$/ft.
	= .39 \$/sf. per ft. of storey height

This change in non-structural cost can now be included with the building skeleton cost adjustment to fully describe the effect of storey height on the building cost.

3.6 Evaluation of Floor Systems

The detailed design and costing of the sixteen floor slabs is carried out in Appendix E. The composite floor beam designs and costs which were investigated are presented in Appendix F. Upon examining the results for beam B6, it was found that little or no saving in the steel section was achieved by thickening the slabs or increasing the concrete strength. Therefore, the slab cost premiums for thicker slabs and higher concrete strengths are not justified and only slabs number 1, 2, 12 and 15 are considered in the final evaluation of the floor system.

From the design of these different floor beams the following economic alternatives to shored and unshored construction with W-shapes (44W grade steel) were determined.

Shored Construction:

Standard W-Shape

 $F_y = 44 \text{ ksi.}$

vs.

Standard W-Shape

 $F_y = 44 \text{ ksi.}$

Unshored Construction:

Standard W-Shape

 $F_y = 44 \text{ ksi.}$

vs.

Standard W-Shape

 $F_y = 60 \text{ ksi.}$

In shored construction the governing design requirement is long-term deflection. The shallow haunch acts to increase the moment of inertia allowing the use of smaller steel sections. In unshored construction the governing design requirement is the bottom fiber stress. The high strength steel increases the allowable stress and the cover plate increases the bottom fiber section modulus which reduces the bottom fiber stress. The cost of these four composite beam constructions when used with slabs 1, 2, 12, and 15 is presented in Table 6.

Knowing the composite beam designs the dead load and depth of the floor system can be determined. The dead load of the floor system is governed by the slab design. Since the steel beam dead load is small and changes in this value do not substantially affect the total dead load of the floor, an allowance of 3 psf. was made for structural steel floor beam dead load in all the designs. The dead load of each of the floor designs and

Table 6 Floor Beam Designs and Costs

Slab #1 - Shored & $F_y = 44$ ksi

Beam	Section	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W18x45	70	18	765	13770	.91*
B7	W16x36	80	4	564	2256	.15*
B8	W24x94	80	4	1716	6864	.40
						1.46 \$/sf.

Storey Height (ducts under beams) = 12.25'

Storey Height (ducts through beams) = 11.75'

Slab #1 - Shored & Haunch & $F_y = 44$ ksi

Beam	Section	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W16x36	100	18	675	12150	.80*
B7	W16x26	100	4	493	1972	.13*
B8	W24x94	80	4	1716	6864	.40
						1.33 \$/sf.

Storey Height (ducts under beams) = 12.33'

Storey Height (ducts through beams) = 11.75'

* based on gross area - core area = 15150 sf./floor

Table 6 Floor Beam Designs and Costs (continued)

Slab #1 - Unshored & $F_y = 44$ ksi

Beam	Section	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W18x50	50	18	902	16236	1.07*
B7	W16x40	50	4	655	2620	.17*
B8	W24x100	50	4	1910	7640	.44
						1.68* \$/sf.

Storey Height (ducts under beams) = 12.25'

Storey Height (ducts through beams) = 11.75'

Slab #1 - Unshored & Cover Plate (when economic) & $F_y = 60$ ksi

Beam	Section	Cover Plate	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W16x31	4.5"x1/4"x19'	50	18	804	14472	.96*
B7	W14x34	-	50	4	612	2448	.16*
B8	W21x55	7"x1/2"x27'	60	4	1583	6332	.37
							1.49 \$/sf.

Storey Height (ducts under beams) = 12.08'

Storey Height (ducts through beams) = 11.54'

* based on gross area - core area = 15150 sf./floor

Table 6 Floor Beam Designs and Costs (continued)

Slab #2 - Shored & $F_y = 44$ ksi

Beam	Section	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W18x45	100	18	778	14004	.93*
B7	W16x36	100	4	571	2284	.15*
B8	W24x110	60	4	1928	7712	.45
						1.52 \$/sf.

Storey Height (ducts under beams) = 12.29'

Storey Height (ducts through beams) = 11.79'

Slab #2 - Shored & Haunch & $F_y = 44$ ksi

Beam	Section	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W16x40	100	18	729	13122	.86*
B7	W16x31	100	4	555	2220	.15*
B8	W24x110	60	4	1958	7712	.45
						1.46 \$/sf.

Storey Height (ducts under beams) = 12.38'

Storey Height (ducts through beams) = 11.79'

* based on gross area - core area = 15150 sf./floor

Table 6 Floor Beam Designs and Costs (continued)

Slab #2 - Unshored & $F_y = 44$ ksi

Beam	Section	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W18x55	50	18	968	17424	1.15*
B7	W18x45	50	4	754	3016	.20*
B8	W24x120	50	4	2192	8768	.51
						1.86 \$/sf.

Storey Height (ducts under beams) = 12.29'

Storey Height (ducts through beams) = 11.79'

Slab #2 - Unshored & Cover Plate (when economic) & $F_y = 60$ ksi

Beam	Section	Cover Plate	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W16x31	4.5"x1/2"x24"	50	18	866	15588	1.03*
B7	W14x38	-	50	4	665	2660	.17*
B8	W21x68	7"x5/8"x26"	50	4	1804	7216	.42
						1.62 \$/sf.	

Storey Height (ducts under beams) = 12.17'

Storey Height (ducts through beams) = 11.58'

* based on gross area - core area = 15150 sf./floor

Table 6 Floor Beam Designs and Costs (continued)

Slab #12 - Shored & $F_y = 44$ ksi

Beam	Section	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W16x31	100	28	594	16632	1.10*
B7	W14x26	100	7	449	3143	.21*
B8(40')	W24x84	100	2	1409	2818	.16
B8(50')	WWF28x110	100	2	2456	4912	.29
						<u>1.76 \$/sf.</u>

Storey Height (ducts under beams) = 12.17'

Storey Height (ducts through beams) = 12.17'

Slab #12 - Unshored & $F_y = 44$ ksi

Beam	Section	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W16x40	50	28	736	20608	1.36*
B7	W16x31	50	7	569	3983	.26*
B8(40')	W24x94	60	2	1641	3282	.19
B8(50')	WWF28x110	100	2	2611	5222	.30
						<u>2.11 \$/sf.</u>

Storey Height (ducts under beams) = 12.17'

Storey Height (ducts through beams) = 12.17'

* base on gross area - core area = 15150 sf./floor

Table 6 - Floor Beam Designs and Costs (continued)

Slab #12 - Unshored & Cover Plate (when economic) & $F_y = 60$ ksi

Beam	Section	Cover Plate	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W14x30	6"x1/4"x6'	50	28	674	18872	1.25*
B7	W14x26	-	50	7	519	3633	.24*
B8(40')	W21x55	7"x3/8"x22'	100	2	1408	2816	.16
B8(50')	W24x76	8"x1/2"x26'	100	2	2151	4302	.25
							1.90 \$/sf.

Storey Height (ducts under beams) = 12.00'

Storey Height (ducts through beams) = 11.88'

Slab #15 - Shored & $F_y = 44$ ksi

Beam	Section	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W16x31	80	28	593	16604	1.10*
B7	W14x26	90	7	453	3171	.21*
B8(40')	W24x74	80	2	1335	2670	.16
B8(50')	WWF28x101	100	2	2341	4682	.27
						1.74 \$/sf.

Storey Height (ducts under beams) = 12.23'

Storey Height (ducts through beams) = 12.2'

* base on gross area - core area = 15.50 sf./floor

Table 6 Floor Beam Designs and Costs (continued)

Slab #15 - Unshored & $F_y = 44$ ksi

Beam	Section	Shear, Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W16x36	50	28	688	19264	1.27*
B7	W16x31	50	7	573	4011	.27*
B8(40')	W24x94	50	2	1659	3318	.19
B8(50')	WWF28x110	50	2	2603	5206	.30
						2.03 \$/sf.

Storey Height (ducts under beams) = 12.23'

Storey Height (ducts through beams) = 12.23'

Slab #15 - Unshored & Cover Plate (when economic) & $F_y = 60$ ksi

Beam	Section	Cover Plate	Shear Connection (%)	Quantity	Cost per Beam (\$)	Total Cost (\$)	Total Cost (\$/sf.)
B6	W14x30	-	50	28	637	17836	1.18*
B7	W14x26	-	50	7	524	3668	.24*
B8(40')	W21x55, 7"x1/4"x20"		60	2	1397	2794	.16
B8(50')	W24x68, 8"x1/2"x27"		70	2	2028	4056	.24
							1.82 \$/sf.

Storey Height (ducts under beams) = 12.06'

Storey Height (ducts through beams) = 11.94'

* based on gross area - core area = 15150 sf./floor

the resulting building skeleton cost, calculated using the equations determined in section 3.5 Building-Floor Interaction, are shown in Table 7.

The storey height is equal to the sum of the architectural height requirement of 9 ft. and the depth of the floor system. The mechanical system requires three 16" diameter openings in beams B8 but the designer has the option of running the three smaller 10" diameter ducts (12" diameter with insulation) under or through beams B6 and B7. The additional cost of openings in these beams must be compared with the saving due to reduced storey height. In the case of no openings in beams B6 and B7 the depth of B6 governs the floor depth (Fig. 11a), with openings the depth of B8 governs floor depth (Fig. 11b). The floor systems will be evaluated with and without openings in beams B6 and B7. The storey height and its effect on the building skeleton cost for each of the floor designs is included in Table 7.

The shop fabrication time for a 16" diameter and a 12" diameter reinforced opening is 6 hours and 4.5 hours respectively, the costs per opening are as follows:

16" diameter opening = 140 \$/opening

12" diameter opening = 105 \$/opening

From these unit costs the total cost of openings per floor can be calculated.

Table 7 Building Skeleton Costs.

Floor Construction	Slab No.	Floor System Dead Load DL (psf.)	Building Skeleton Cost CBS (\$/sf.)	Ducts Under Beams		Ducts Through Beams	
				Storey Height S (ft.)	Bldg. Skeleton* Cost Adjustment Δ CBS (\$/sf.)	Storey Height S (ft.)	Bldg. Skeleton* Cost Adjustment Δ CBS (\$/sf.)
Shored & $F_y = 44$ ksi	1	60	3.19	12.25	-.15	11.75	-.45
	2	78	3.44	12.29	-.13	11.79	-.43
	12	68	3.23	12.17	-.19	12.17	-.19
	15	64	3.17	12.23	-.16	12.23	-.16
Shored & Haunch & $F_y = 44$ ksi	1	60	3.19	12.33	-.10	11.75	-.45
	2	78	3.44	12.38	-.07	11.79	-.43
	1	60	3.19	12.25	-.15	11.75	-.45
	2	78	3.44	12.29	-.13	11.79	-.43
Unshored & $F_y = 44$ ksi	12	68	3.23	12.17	-.19	12.17	-.19
	15	64	3.17	12.23	-.16	12.23	-.16
	1	60	3.19	12.08	-.25	11.54	-.58
	2	78	3.44	12.17	-.20	11.58	-.56
Unshored & Cover Plate (when economic) & $F_y = 60$ ksi	12	68	3.23	12.00	-.30	11.88	-.37
	15	64	3.17	12.06	-.26	11.94	-.33

*Includes adjustment for non-structural costs.

a) Beam B6 and B7

b) Beam B8

Fig. 11 Depth of Floor System

Now that all the costs have been determined, the total structural cost of each of the floor systems can be calculated and compared (Table 8). From these results, the steel deck scheme using 2-1/2" thick normal weight concrete, spray on fire protection to deck, standard W-shape, shored construction and 44W grade steel which costs 6.96 \$/sf. would be the floor system selected. However, if the forming costs (including the premium for shallow haunch) for the formed concrete slab scheme could be reduced to 1.35 \$/sf., then it would become an economic alternative. It is also interesting to note that there is little difference in cost between shored and unshored construction if cover plates and high strength steel are used to meet bottom fiber stress requirements in unshored construction.

The saving in total structural cost is compared with the additional cost of openings in Table 9. This comparison clearly shows that openings in beam B6 and B7 are not feasible for this floor system. In other floor systems where the decrease in floor depth is not restricted by a transfer beam such as B8, openings for mechanical ducts may prove economically feasible. These openings are more likely to be economic the wider the beams are spaced apart as this reduces the number of openings in beams.

Table 8 Total Structural Costs

Floor, Construction	Slab No.	Cost of Slab (\$/sf.)	Cost of Beams (\$/sf.)	Cost of Openings* (\$/sf.)	Unadjusted Skeleton (\$/sf.)	Bldg. Skeleton Cost Adjustment* (\$/sf.)	Total Structural Cost (\$/sf.)
Shored & $F_y = 44$ ksi	1	2.63	1.46	.10	3.19	-.15	7.23
	2	2.25	1.53	.10	3.44	-.13	7.19
	12	2.06	1.76	.10	3.23	-.19	6.96
	15	2.19	1.74	.10	3.17	-.16	7.04
Shored & Haunch & $F_y = 44$ ksi	1	2.63	1.33	.10	3.19	-.10	7.15
	2	2.25	1.46	.10	3.44	-.07	7.18
	12	2.06	1.86	.10	3.44	-.13	7.52
	15	2.19	2.03	.10	3.17	-.16	7.33
Unshored & $F_y = 44$ ksi	1	2.63	1.68	.10	3.19	-.15	7.45
	2	2.25	1.86	.10	3.44	-.13	7.52
	12	2.06	2.11	.10	3.23	-.19	7.31
	15	2.19	2.03	.10	3.17	-.16	7.33
Unshored & Cover Plate (when economic) & $F_y = 60$ ksi	1	2.63	1.49	.10	3.19	-.25	7.16
	2	2.25	1.62	.10	3.44	-.20	7.21
	12	2.06	1.90	.10	3.23	-.30	6.99
	15	2.19	1.82	.10	3.17	-.26	7.02

* based on ducts running under beams B6 and B7

Table 9 Openings in Beams B6 and B7

Floor Construction	Slab No.	Reduction in Storey Height (ducts through beams B6 & B7) (ft.)	Savings in Total Structural Cost (\$/sf.)	Cost of Openings in beams B6 & B7 (\$/sf.)
Shored & $F_y = 44$ ksi	1	.50	.30	.50
	2	.50	.30	.50
	12	0	0	.64
	15	0	0	.64
Shored & Haunch & $F_y = 44$ ksi	1	.58	.35	.50
	2	.59	.36	.50
	12	0	0	.64
	15	0	0	.64
Unshored & $F_y = 44$ ksi	1	.50	.30	.50
	2	.50	.30	.50
	12	0	0	.64
	15	0	0	.64
Unshored & Cover Plate (when economic) & $F_y = 60$ ksi	1	.54	.33	.50
	2	.59	.36	.50
	12	.12	.07	.64
	15	.12	.07	.64

4 - CONCLUSIONS AND RECOMMENDATIONS

4.1 Conclusions

In this study a rational approach to the design of floor systems in high-rise steel frame buildings was presented. The floor evaluation procedure was used to investigate various composite floor constructions in high-rise office buildings.

As a result of this study the following conclusions can be made.

1. If a meaningful evaluation of the floor system is to be achieved, the effect of the floor system on the total structural cost must be determined. This is demonstrated in a comparison of a ranking of floor systems based on the floor system cost alone vs. a ranking of floor systems based on the total structural cost (Table 10).
2. The following recommendations can be made based on the evaluation of composite floor systems in one, high-rise, steel framed office building.
 - I) The slab parameters of thickness and concrete strength should be selected to minimize slab cost (i.e. low strength concrete and as thin a slab as possible).
 - II) The alternatives of normal weight concrete or semi-lightweight concrete (115 pcf.) should be investigated.
 - III) If forming costs reduce to 1.35 \$/sf. the formed concrete slab can become an economic alternative to the industry standard of beams at 10 ft. centers spanned with steel deck.
 - IV) Openings in beams, for mechanical ducts, to reduce storey height may be worth investigating if the full depth occupied by the duct can be recovered and the beams are spaced at 15 ft. centers or greater.
 - V) In shored construction the shallow 3 in. haunch provides a saving over standard shored construction.
 - VI) In unshored construction the use of high-strength steel and cover plates is essential for an economic design.

Table 10 Comparison of Floor Evaluation Methods

Floor Construction		Floor System		Building		Total Structural	
Slab No.		Cost* (\$/sf.)	Cost Ranking	Skeleton Cost (\$/sf.)	Cost* (\$/sf.)	Cost Ranking	
Shored & F _y = 44 ksi	1	4.19	8	3.04	7.23	10	
	2	3.88	2	3.31	7.19	8	
	12	3.92	3	3.04	6.96	1	
	15	4.03	5	3.01	7.04	4	
Shored & Haunch & F _y = 44 ksi	1	4.06	6	3.09	7.15	5	
	2	3.81	1	3.37	7.18	7	
Unshored & F _y = 44 ksi	1	4.41	13	3.04	7.45	13	
	2	4.21	9	3.31	7.52	14	
	12	4.27	11	3.04	7.31	11	
	15	4.32	12	3.01	7.33	12	
Unshored & Cover Plate (when economic) & F _y = 60 ksi	1	4.22	10	2.94	7.16	6	
	2	3.97	4	3.24	7.21	9	
	12	4.06	6	2.93	6.99	2	
	15	4.11	7	2.91	7.02	3	

*Based on duct running under B6 and B7

- VII) The use of standard unshored construction should be avoided as this construction is over .30 \$/sf. more costly than other economic constructions.

4.2 Recommendations for Further Study

During the course of the present study the following areas of research were identified for further study.

1. Establish a method for quickly and accurately assessing the cost of the vertical gravity systems and the lateral force resisting systems to facilitate the determination of building skeleton costs. In this study the entire building had to be designed to determine this cost. If a simple costing method for different lateral force resisting systems could be developed it would be possible to investigate many more framing schemes.
2. Develop a forming system to form between beams spaced at 15 ft. centers that will reduce forming costs below 1.35 \$/sf. If shored construction is used the forming system should incorporate a shallow haunch.
3. Investigate different lengths of beams spanning from core to exterior wall to determine the most economic span. Table 11 indicates that beams spanning 40 ft. cost approximately .20 \$/sf. more than beams spanning 35 ft. This indicates that the beam span is a sensitive parameter and needs to be examined more fully taking into account changes in the building skeleton cost.

Table 11 Comparison of Beam Costs for 35 ft. and 40 ft. Spans

Floor Construction	Slab No.	Beam B6 (40' span)		Beam B7 (35' span)		Reduction in Beam Cost for 35' Span Over 40' Span (\$/sf.)	
		Cost per Beam (\$)	Beam Cost* (\$/sf.)	Cost per Beam (\$)	Beam Cost** (\$/sf.)		
Shored & F _y = 44 ksi	1	765	1.28	564	1.07		.21
	2	778	1.30	571	1.09		.21
	12	594	1.49	449	1.28		.21
	15	593	1.48	453	1.29		.19
Shored & Haunch & F _y = 44 ksi	1	675	1.13	493	.94		.19
	2	729	1.22	555	1.06		.16
Unshored & F _y = 44 ksi	1	902	1.50	655	1.25		.25
	2	968	1.61	754	1.44		.17
	12	736	1.84	569	1.63		.21
	15	688	1.72	573	1.64		.08
Unshored & Cover Plate (when economic) F _y = 60 ksi	1	804	1.34	612	1.17		.17
	2	866	1.44	665	1.27		.17
	12	674	1.69	519	1.48		.21
	15	637	1.59	524	1.50		.09

* based on areas of 600 sf. for slabs 1 & 2 and 400 sf. for slabs 12 & 15

** based on areas of 575 sf. for slabs 1 & 2, and 350 sf. for slabs 12 & 15

Appendix A

Cost Documentation

This appendix documents the costs presented in section 3.4 Cost Information. Letters from the following firms serve to summarize the cost information obtained in the cost survey for the Ottawa region (March 1978).

Canadian Institute of Steel Construction

The Algoma Steel Corporation Ltd. (Grade and Sections Extras)

M. Zagerman and Company Ltd.

The Steel Company of Canada Ltd. (STELCO)

Stelco (Grade and Size Extras)

Francon Division of Canfarge Ltd.

Right Forming Ltd.

Westeel-Rosco Ltd.

Tristan Inc.

Vandenbelt Metal Works Ltd.

Otis Elevator Company Ltd.

Clemann, Large, Patterson and Associates Ltd.

Canadian Steel Industries Construction Council

201 Consumers Road, Suite 300,
Willowdale, Ontario M2J 4G8
Telephone (416) 491-9898

CSICC

March 17th, 1978.

Mr. A. I. Allan,
Department of Civil Engineering,
Carleton University,
Ottawa, Ontario
K1S 5B6

Dear Mr. Allan:

Enclosed with this letter is a summary of the structural steel construction costs we discussed on February 21, 1978. I hope they will be of use in the preliminary cost evaluation of your buildings.

Yours very truly,

E. Y. Chien,
Assistant Director Project Analysis Division.

encl.

61

SUMMARY OF STRUCTURAL STEEL CONSTRUCTION COSTS FOR COST ESTIMATION
COST VARIABLES FOR THE ESTIMATION OF STEEL OFFICE STRUCTURES

The items below are for use in conjunction with the "Piece by Piece Costing Method" outlined in the proceedings of the "CSCE Specialty Conference on Computers in Structural Engineering Practice". All cost variables are for Ottawa region as of February 1978.

Material Costs

Modified Base Prices:

$$B_1 \text{ (WF)} = 12.69 \text{ \$/100 lb.}$$

$$B_2 \text{ (WWF > 140 plf)} = 17.24 \text{ \$/100 lb.}$$

$$B_3 \text{ (WWF < 140 plf)} = 16.72 \text{ \$/100 lb.}$$

Freight Charges: (mill to fabricator)

$$F_1 = .82 \text{ \$/100 lb.}$$

Material Cost Coeff:

$$Q_m: \text{ use } 1.08$$

Fabrication Costs

Shop Worker Rate: $R_s = 8.71 \text{ \$/hr.}$

Shop Fabricator Coeff: $Q_s: \text{ use } 2.4$

Shipping Rate: $S = .40 \text{ \$/100 lb.}$
(fabricator to site; assumed site)

Engineering and Drafting Costs

Drafting Dept. Rate: $R_D = 15.00 \text{ \$/hr.}$

Engineering and Drafting Coeff: $Q_D: \text{ use } 1.45$

Erection Costs

Erection Crew Rate: $R_E =$ ~~800~~
863 $\text{\$/day}$

(Based on a 6 man crew including crane operator and crane rental costs and power)

Erection Cost Coeff: $Q_E: \text{ use } 2.4$

Taxes and Gross Profit

Federal Sales Tax: $FST = 5\%$

Provincial Sales Tax: $PST = 7\%$

Gross Profit and Sales Overhead: $GP = 6\%$

continued...

General

Productivity Factor: ϕ : use 1.0

Other Costs

Shear Studs: Shop Installed: $.7 \times 1.10 = .80$ \$/stud

Field Installed: 1.10 \$/stud

Fire Proofing: Sprayed on Beams: .25 \$/sf C.A.

Drywall Around Col.: 11 \$/lf

(\$11 x 50%) Structural

(\$11 x 50%) Architectural)

Steel Deck: 20 ga. V-Pan = .45 \$/sf

(inplace costs)

Wire Mesh : .15 - .20 \$/sf

Edge Form Angles: 1.50 \$/lf

///

Algoma Grade and Section Extras

This section presents parts of "Hot Rolled Structural Shapes - Classification of Extras" (13) published by Algoma, specifically the grade section extras for W and WWF(M) shapes. All extras are given in dollars per hundred pounds and are to be added to base prices.

First the section extras for W-shapes are presented. There are no section extras for WWF(M) shapes, just a base price for sections over 140 plf. and another for those under 140 plf. The grade extras for W-shapes are broken down into groups of sections as shown. In the July 31, 1977 Classification of Extras no extra for 60 Ksi. grade steel was published. Algoma-Montreal suggested that this cost would have increased by the same percentage as the 50 ksi grade. Using the published February 1974, 60 Ksi., grade extra of 1.55 \$/100 lb. (Group I), a 1977 extra of 3.30 \$/100 lb. was calculated. The grade extras used for WWF(M) shapes are shown below:

WWF - Shapes

Grade (Ksi.)	Extra \$/100 lb.
44	-
50	2.65
60	3.50

HOT ROLLED STEEL STRUCTURAL SHAPES

DIMENSIONS SIZE AND SECTION

Wide Flange Shapes

W 6 x 15 5 lb.	1.45
W 6 x 20 and 25 lb.	1.30
W 8 x 17 and 20 lb.	1.45
W 8 x 24 and 28 lb.	1.00
W 8 x 31 to 67 lb.	.80
W 10 x 21, 25 and 29 lb.	1.20
W 10 x 33, 39 and 45 lb.	1.00
W 10 x 49 to 112 lb.	.75
W 12 x 27, 31 and 36 lb.	1.00
W 12 x 40 to 58 lb.	.75
W 12 x 65 to 190 lb.	.45
W 14 x 22 and 26 lb.	1.55
W 14 x 30 to 38 lb.	.80
W 14 x 43 to 53 lb.	.65
W 16 x 26 and 31 lb.	1.75
W 16 x 36 to 50 lb.	.80
W 18 x 40 lb.	1.10
W 18 x 45 to 60 lb.	.75
W 21 x 55 to 73 lb.	1.00
W 24 x 68 to 94 lb.	.85
W 24 x 100 to 120 lb.	.75

Standard Beams

S 3 x 5.7 and 7.5 lb.	3.50
S 4 x 7.7 lb.	2.45
S 5 x 10 and 14 75 lb.	2.25
S 6 x 12.5 and 17.25 lb.	1.75
S 8 x 18.4 and 23 lb.	1.45
S 10 x 25 4 and 35 lb.	1.10
S 12 x 31 8 and 35 lb.	1.00

H - Beams

M 4 x 13 lb.	2.10
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HOT ROLLED STEEL STRUCTURAL SHAPES

SPECIFICATIONS AND PROCESSING

CLASSIFICATION OF STRUCTURAL SHAPES (ALGOMA'S RANGE)

	GROUP 1	GROUP 11	GROUP 111
Wide Flange Shapes (W)	6 x 15.5 to 25 lb. 8 x 17 to 48 lb. 10 x 11.5 to 45 lb. 12 x 14 to 58 lb. 14 x 22 to 53 lb. 16 x 26 to 50 lb. 18 x 40 to 60 lb. — —	— 8 x 58 to 67 lb. 10 x 49 to 112 lb. 12 x 65 to 106 lb. — — — 21 x 55 to 73 lb. 24 x 68 to 120 lb.	— — — 12 x 120 to 190 lb. — — — — —
Bearing Pile (HP)	—	To 102 lb.	Over 102 lb.
Standard Beams (S)	To 35 lb.	—	—
Standard Channels (C)	To 20.7 lb.	Over 20.7 lb.	—
Shipbuilding Channels (MC)	To 28.3 lb.	—	—
Angles & Zees (L)(Z)(CZ)	To 1/2 in.	Over 1/2 to 3/4 in.	Over 3/4 in.

HOT ROLLED STEEL STRUCTURAL SHAPES

SPECIFICATIONS AND PROCESSING

SPECIFICATIONS

The following extras for Association Specifications or their equivalents include Classification, Chemistry, Mechanical Properties and Quality Extras but no other extras unless indicated.

CSA G40.21

Grade - Type	GROUP 1	GROUP 11	GROUP 111
* 44W	Base	Base	Base
44T	1.75	1.75	1.75
50W	1.80 (a)	2.65	—
50A	4.30	4.30	4.30

* Standard Grade - For other grades, please enquire for availability.

(a) If vanadium specified - higher extras apply.

STRUCTURAL STEEL DIVISION

March 31, 1978

Mr. A. I. Allan
Department of Civil Engineering,
Carleton University,
Ottawa, Ontario.
K1S 5B6.

Dear Mr. Allan:

We are enclosing herewith a summary of the structural steel and reinforcing steel costs - discussed on March 27, 1978. We hope these will be of use in the cost analysis of your buildings, and should you have any further questions, please contact us.

Yours very truly,

M. Zagerman & Co, Limited

R. P. Whalen, P. Eng.
Chief Engineer

RPW/gt
Encl.

STRUCTURAL STEEL

Shipping : for an Ottawa Site = 8\$/Ton

Reinforcing Openings in Beams: 16 in. square = 6-8 Hr./opening
or 16 in. diameter

Cover Plates: Light beam cover plate = .2 hr./Lineal foot
($\leq$.75 in. thick)

Heavy column cover plate = .3 Hr./Lineal foot

REINFORCING STEEL

Mesh

Supply cost 6X6 - 10X10 52 \$/1000 square ft.
6X6 - 8 X8 75 \$/1000 square ft.
6X6 - 6 X6 95 \$/1000 square ft.
(Plus Provincial sales tax)

Placing cost .06 \$/square ft.

Reinforcing Bars

Supply cost 310 \$/Ton
Placing cost 85 \$/Ton

Cambering

In a phone conversation Dick Whalen of Zagerman advised that cambering costs would be \$90 for a W18 beam and \$140 for a W24 beam, costs for other sections could be estimated from these values.

stelco

The Steel Company
of Canada, Limited

General Office
P O Box 460
Case Postale
Montreal 101, Quebec
(514) 933 8686

March 29, 1978.

Mr. Drew Allan
c/o: Civil Engineering Department
Carleton University
OTTAWA, Ontario.
K1S 5B6

Dear Mr. Allan:

The following will answer your questions brought up during my visit last week.

MESH

- a) Our maximum width limitation for construction mesh is between 12 ft. and 14 ft. However, the practical limit for transportation is 8 ft. since larger widths require escorts during transport.
- b) The list price for construction fabrics ranges between \$20. and \$21. per 100 lbs. However, with the current lull in the market it is being sold for \$15. to \$16. per 100 lbs. to fabricators and contractors. You must add 5% Federal + 7% Provincial tax.
- c) The shipping cost from Montreal to Ottawa is an average \$250. per 25,000 lb. truck load.
- d) Odd sizes such as 6 x 6 x 1/0 and 6 x 6 x 4/A are as readily available as standard sizes when ordering a minimum 20 tons per size.

REBAR

- a) #4 Rebar is currently selling for about \$250./ton for Grade 50 and \$260./ton for Grade 60. However, in normal market conditions these would be priced at \$290 and \$300. respectively, excluding sales tax.
- b) Freight from Montreal to Ottawa is approximately \$12./ton.

.../2

Page 2
Civil Engineering Dept.
Carleton University
March 29, 1978

PLATE & BAR PRODUCTS

I have enclosed our latest price lists for Plate and Bar products (flats & angles).
Also enclosed "Selection & Uses of Structural Steels".

If you have any further questions do not hesitate to call.

THE STEEL COMPANY OF CANADA, LIMITED

P. C. Girard
Field Sales Engineer
Eastern Region

PCG:cc

Stelco Grade and Size Extras

This section presents parts of the "Hot Rolled Plates - Classification of Extras" (14) published by Stelco, specifically the grade and size extras. All extras are given in dollars per hundred pounds and are to be added to base prices.

Since the light cover plates for beams (<.75" thick) range from bar to plate product classifications, a complex calculation to determine small differences in cost would be required. To simplify the procedure the following average material costs which include base price, size and grade extras will be used.

<u>Light Beam Cover Plates</u>	
<u>Grade</u> <u>(Ksi.)</u>	<u>Material Cost</u> <u>(\$/100 lb.)</u>
44	15.60
50	16.45
60	17.15

The heavy beam cover plate required in the lower column tiers will be calculated using the size and grade extras for hot rolled plates reproduced in this section.

- Hot Rolled Carbon Steel Plates — Metric (Imperial) ⁷²

Base Price — Dollars per One Hundred Pounds

Plates are invoiced on the theoretical mass of the ordered size, using density of 7850 kg/m³ or .2836 pounds per cubic inch, rounded to nearest full pound per piece.

Product	Base Price	Effective Date
Hot Rolled Plates	\$14.45	78 04 16

The term "Hot Rolled Carbon Steel Plates" encompasses both Carbon (C) and High Strength Low Alloy (HSLA) flat hot rolled finished steel products as classified by size below.

Product Classification by Size of Flat Hot Rolled Carbon Steel

Width, millimetres (Inches)	Thickness, millimetres (Inches)			
	Over 6.0mm (.2362")	Over 5.0mm (.1968") to 6.0mm (.2362")	Over 4.5mm (.1771") to, 5.0mm (.1968")	1.2mm and over (.0472") to 4.5mm (.1771")
To 150mm incl. (To 5.90" incl.)	Bar	Bar	Strip	Strip
Over 150mm to 200mm incl. (Over 5.90" to 7.87" incl.)	Bar	Strip	Strip	Strip
Over 200mm to 300mm incl. (Over 7.87" to 11.81" incl.)	PLATE	Strip	Strip	Strip
Over 300mm to 1200mm incl. (Over 11.81" to 47.24" incl.)	PLATE	Sheet	Sheet	Sheet
Over 1200mm (Over 47.24")	PLATE	PLATE	PLATE	Sheet

Classification

Hot rolled carbon steel Plates fall into one of the following classifications:

(1) Carbon .33 per cent maximum limit only — check analysis tolerances not applicable [Reg. Q]	Nil
(2) Chemical composition limits or ranges other than carbon .33 per cent max. [Chem Req.]	.10
(3) Mechanical properties only [Mech Req.] (See Note)	.20
(4) Chemistry (carbon and/or manganese) and mechanical properties [Chem and Mech Req.]	.30

Note: When structural quality Plates are ordered to mechanical properties only and the specified minimum tensile strength is over 413.7 MPa, chemistry extras for the necessary maximums of carbon and/or manganese and for necessary silicon shall apply and such material falls into Classification 4.

stelco

The Steel Company of Canada Limited

General Office: Hamilton, Ontario, Canada

EFFECTIVE 78 04 16

Hot Rolled Carbon Steel Plates — Metric (Imperial)

Size Extras

Width and Thickness — Sheared and Gas Cut Edges

When not specified the greater dimension is considered to be the length although not necessarily parallel to the direction of rolling. When Plates are specified with the greater dimension at right angles to the final direction of rolling, this dimension shall be considered as the width, and the extra calculated accordingly.

Width — Millimetres (Inches)	Under 6mm (.2362")	6mm (.2362") to 8mm (.3150") excl.	8mm (.3150") to 9mm (.3543") excl.	9mm (.3543") to 12mm (.4724") incl.	Over 12mm (.4724") to 25mm (.9843") incl.	Over 25mm (.9843") to 38mm (1.50") incl.	Over 38mm (1.50") to 80mm (3.1496") incl.	Over 80mm (3.1496") to 150mm (5.9055") incl.	Over 150mm (5.9055")
600mm to 750mm excl. (23.6" to 29.5" excl.)	—	1.50	1.30	1.05	1.00	1.75	1.85	2.10	2.25
750mm to 900mm excl. (29.5" to 35.4" excl.)	—	1.35	1.15	.90	.85	1.50	1.65	1.90	2.05
900mm to 1200mm excl. (35.4" to 47.2" excl.)	1.55	1.30	1.05	.80	.75	1.30	1.45	1.75	1.95
1200mm to 1500mm excl. (47.2" to 59.1" excl.)	1.40	1.15	.90	.65	.60	1.15	1.25	1.60	1.75
1500mm to 2000mm excl. (59.1" to 78.7" excl.)	1.40	1.00	.80	.55	.40	.85	1.05	1.45	1.55
2000mm to 2300mm incl. (78.7" to 90.6" incl.)	1.45	1.00	.75	.50	.30	.70	.90	1.30	1.40
Over 2300mm to 2500mm incl. (Over 90.6" to 98.4" incl.)	1.65	1.25	.90	.70	.50	.85	1.05	1.40	1.50
Over 2500" to 2800" incl. (Over 98.4" to 110.2" incl.)	1.85	1.45	1.15	.90	.70	.95	1.10	1.45	1.55
Over 2800mm to 3000mm incl. (Over 110.2" to 118.1" incl.)	2.05	1.60	1.35	1.10	.90	1.10	1.25	1.60	1.70
Over 3000mm to 3300mm incl. (Over 118.1" to 129.9" incl.)	2.25	1.80	1.55	1.30	1.10	1.30	1.45	1.80	1.90
Over 3300mm to 3500mm incl. (Over 129.9" to 137.8" incl.)	2.60	2.15	1.85	1.60	1.40	1.60	1.75	2.05	2.15
Over 3500mm (Over 137.8")	2.95	2.40	2.15	1.90	1.70	1.90	2.05	2.35	2.45

Plates over 38mm (1.50") thick

For Plates over 38mm (1.50") thick, unless Killed Steel specified or implied, add **1.00**

Plates 38mm (1.50") and under in thickness

Plates of individual pattern mass (theoretical) in excess of 6 tonnes (13228 lbs) ordered in thickness 38mm (1.50") and under unless Killed Steel is specified or required **1.00**

Gas Cutting Extra [Gas Cut]

12mm (.4724") to 25mm (.9843") incl. for widths under 1150mm (45.21"), or if specified by customer **.35**

Edge Treatment Extra [Edge Treat]

25mm (.9843") and over, if specified 40% max. Carbon or over, or .25% min. Carbon with 1.00% min. Manganese **.50**

stelco

The Steel Company of Canada Limited

General Office: Hamilton, Ontario, Canada

EFFECTIVE 78 04 15

Hot Rolled Carbon Steel Plates — Metric (Imperial) ⁷⁴

Structural Specifications

The Extras for specifications listed below include applicable extras for classification, chemical requirements, mechanical requirements and test requirements but no other extras unless otherwise indicated.

—Stelcoloy [Stelcoloy ()]

50-thickness 38mm (1.5") and under	4.45
50-thickness over 38mm (1.5") to 100mm (3.937") incl.	4.55
60-thickness max. 25mm (.9843")	4.80
70-thickness max. 20mm (.7874")	5.20

—ASTM A36 [ASTM A36-(YR)]

20mm (.7874") and under	.30
over 20mm (.7874") to 38mm (1.5") incl.	.60
over 38mm (1.5")	1.70
over 38mm (1.5") Bearing Plates other than bridge application no Mechanical tests [Brg O]	1.40

—ASTM A242 [ASTM A242-(YR) Type ()]

Type 1 thickness max. 14mm (.5512)	4.45
Type 2 thickness 38mm (1.5") and under	4.45
thickness over 38mm (1.5") to 100mm (3.937")	4.55

—ASTM A283 Grades A, B, C, and D

[ASTM A283-(YR) Grd ()]

38mm (1.5") and under	.20
over 38mm (1.5")	.30†

† Killed Steel not specified or implied over 38mm (1.5") — refer to page 4, Plates over 38mm (1.5") thick.

—ASTM A441 [ASTM A441-(YR)]

38mm (1.5") and under	3.55
over 38mm (1.5")	3.65

—ASTM A573 [ASTM A573-(YR) Grd ()]

Grade 58 — 38mm (1.5") and under	1.80
over 38mm (1.5") to 50mm (1.9685") max.	1.90
Grade 65 — thickness max. 38mm (1.5")	1.90
Grade 70 — thickness max. 38mm (1.5")	1.90

—ASTM A588 Grade C [ASTM A588 (YR) Grd C]

refer to "Stelcoloy 50"

—(ASTM [A572-(YR) Grd () Type ()]

	TYPE 1 Columbium 2 Vanadium	Minimum Yield MPa	to 10mm (.3937") incl.	over 10mm (.3937") to 20mm (.7874") incl.	over 20mm (.7874") to 38mm (1.5") incl.	over 38mm (1.5") to 100mm (3.937") incl.
A572 Grade 42	1	289.6	1.10	—	—	—
A572 Grade 42	2		1.60	1.70	2.00	3.00*
A572 Grade 45	1	310.3	1.30	—	—	—
A572 Grade 45	2		1.80	2.05	2.40	—
A572 Grade 50	1	344.7	1.45	—	—	—
A572 Grade 50	2		2.00	2.30	3.45*	—
A572 Grade 55	1	379.2	1.80	—	—	—
A572 Grade 55	2		2.45	2.75	4.00*	—
A572 Grade 60	1	413.7	2.15	—	—	—
A572 Grade 60	2		—	—	—	—
A572 Grade 65	1	448.2	2.65	—	—	—
A572 Grade 65	2		—	—	—	—

*Includes Fully Killed extra

stelco

The Steel Company of Canada, Limited

General Office : Hamilton, Ontario, Canada

EFFECTIVE 78 04 16

Hot Rolled Carbon Steel Plates — Metric (Imperial)

Structural Specifications (continued)

CSA G40.21 [CSA G40.21-(YR) Grd ()]

► Grade 33

Type G	38mm (1.5") and under	.20
	Over 38mm (1.5")	.30†

† Killed Steel not specified or implied over 38mm (1.5") refer to page 4, Plates over 38mm (1.5") thick.

► Grade 38

Type W	20mm (.7874") and under	.50
	Over 20mm (.7874") to 38mm (1.5") incl.	.60
Type T	20mm (.7874") and under	1.80
	Over 20mm (.7874") to 38mm (1.5") incl.	1.90
	Over 38mm (1.5") to 100mm (3.937") incl.	2.00

► Grade 44

Type W	20mm (.7874") and under	.60
	Over 20mm (.7874") to 38mm (1.5") incl.	1.25
Type T	38mm (1.5") and under	1.90
	Over 38mm (1.5") to 100mm (3.937") incl.	2.00

► Grade 50

Type W	14mm (.5512") and under	1.45
	Over 14mm (.5512") to 20mm (.7874") incl.	2.30
	Over 20mm (.7874") to 38mm (1.5") incl.	3.10
Type T	14mm (.5512") and under	2.75
	Over 14mm (.5512") to 20mm (.7874") incl.	3.60
	Over 20mm (.7874") to 38mm (1.5") incl.	3.75
	Over 38mm (1.5") to 100mm (3.937") incl.	3.85
Type R	14mm (.5512") and under	4.45
Type A	38mm (1.5") and under	4.45
	Over 38mm (1.5") to 100mm (3.937") incl.	4.55

► Grade 60

Type W	14mm (.5512") and under	2.15
Type T	14mm (.5512") and under	3.45
	Over 14mm (.5512") to 20mm (.7874") incl.	4.05
Type A	25mm (.9843") and under	4.80

► Grade 70

Type W	10mm (.3937") and under	3.25
Type T	14mm (.5512") and under	5.10
	Over 14mm (.5512") to 20mm (.7874") incl.	5.20

Abrasion Resisting Specification

Stelco Wearwell [Stelco Wearwell]

To 25mm (.9843") excl.	1.90
25mm (.9843") to 50mm (1.9685") incl. Including Edge Treatment extra refer to Page 4.	2.40

stelco

The Steel Company of Canada, Limited

General Office: Hamilton, Ontario, Canada

EFFECTIVE 78 04 16

F R A N C O N

DIVISION of CANFARGE LTD

1575 RUSSELL ROAD
OTTAWA, ONT K1G 0N1
TELEPHONE 733-7910PRICE LIST FOR READY MIXED CONCRETEEFFECTIVE JAN. 1ST TO DEC. 31/78.OTTAWA AREAPRICE PER CUBIC YARD

<u>Strength</u> <u>Resistance</u>	<u>3/4"</u>	<u>3/8"</u>
1500 p.s.i.	\$31.75	\$33.10
2000 p.s.i.	32.75	34.10
2500 p.s.i.	33.75	35.10
3000 p.s.i.	34.75	36.10
3500 p.s.i.	35.85	37.20
4000 p.s.i.	37.10	38.45
4500 p.s.i.	38.70	40.05
5000 p.s.i.	41.10	42.45

LISTE DE PRIX POUR BETONEFFECTIF LE 1ER JAN. AU 31 DEC./78.HULL AREAPRIX PAR VERGE CUBE

<u>3/4"</u>	<u>3/8"</u>
\$31.40	\$32.75
32.30	33.65
33.25	34.60
34.20	35.55
35.25	36.60
36.45	37.80
38.00	39.35
40.35	41.70

EXTRAS - SUPPLEMENTS

Air Entraining Agent \$0.90 per cu. yd.
Béton Air Entraîné

Calcium Chloride - Per 1% \$0.55 per cu. yd.
Chlorure de Calcium - Par 1%

Winter Handling - November 1st to April 15th incl. \$2.25 per cu. yd.
Bétonnage d'Hiver - du 1er nov. au 15 avril inclus.

Slump - per 1 inch over 4 inches. \$0.40 per cu. yd.
Affaissement - par 1 pouce au-dessus de 4 pouces.

Underload: \$5.00 per 1/2 cu. yd. from 5 cu. yds. to actual delivery.

Voyage Partiel: \$5.00 par 1/2 v.c. entre 5 v.c. et le nombre de v.c. livrés sera applicable.

TAX: Ontario Provincial Sales Tax Included.

TAXE: La taxe de vente provinciale d'Ontario est incluse.

TAX: Québec Provincial Sales Tax Extra.

TAXE: La taxe de vente provinciale du Québec en plus.

TERMS: Net 15th of the month following.

TERMES: Net le 15 du mois suivant.

(10/11/77)

MAIL ADDRESS P.O. BOX 8464, OTTAWA, ONT K1G 3K2
OFFICE AND PLANT / PRESTRESSED CONCRETE / 8300 PIERRE BOULEVARD, MONTREAL, QUE H1Z 3T6

OTTAWA - CONCRETECUSTOMER PRICESLIGHTWEIGHT CONCRETEP R I C E

<u>VERMICULITE</u>	<u>ONTARIO</u>	<u>QUEBEC</u>	<u>APPROX. STRENGTH</u>	<u>APPROX. WEIGHT</u>
1 - 3	\$95.35	\$95.80	500	35-45
1 - 4	88.85	87.70		
1 - 4½	86.65	85.60		
1 - 5	85.35	84.45	250	30-35
1 - 6	82.05	81.25		
1 - 8	80.00	79.45		
1 - 12	77.40	77.00	125	20-25
1 - 3' - 2	68.40	67.55		
			620	80

HAYDITESEMI-LIGHTWEIGHT

110 - 118 lbs./cu. ft.

ONTARIOQUEBECLIGHTWEIGHT

95 - 105 lbs./cu. ft.

ONTARIOQUEBEC

2000	\$61.35	\$61.00	\$75.30	\$74.95
2500	62.30	61.85	76.85	76.40
3000	63.50	63.00	78.35	77.90
3500	64.70	64.15	79.65	79.15
4000	66.40	65.80	81.05	80.45

- 1) Add \$2.00 per cubic yard for C.O.D.
- 2) Add \$3.00 for Dry Batch used in pile work.
- 3) Winter Handling Charge \$2.25 per cubic yard from November 1st to April 15th inclusively.
- 4) Terms: Net - 15th of month following.
- 5) Ontario Sales Tax of 7% is included in the Ontario Prices.
Quebec Sales Tax of 8% is not included in the Quebec Prices and must be charged extra.

EFFECTIVE JANUARY 1, 1978

✓
RIGHT FORMING LIMITED

CONCRETE FORMING SPECIALISTS

199 CAMELIN BLVD.
HULL, QUEBEC
J8Y 1W2

March 9, 1978

Mr. A. I. Allan,
Department of Civil Engineering,
Carleton University,
Ottawa, Ontario
K1S 5B6

Dear Mr. Allan:

Enclosed with this letter is a summary of the concrete construction costs we discussed on March 3, 1978. I hope they will be useful in your cost evaluation of your building..

Yours truly,
RIGHT FORMING LIMITED.

✓

Claudio Gnani
President

CG/ws
enc.

Forming Cost:

A forming cost of \$1.50 per sq. ft. could be achieved with large flying forms utilizing the "Aluma" forming system. This price is based on the assumption that the steel erection and forming can be coordinated efficiently.

Concrete Costs:

Hoisting & Placing :	\$8.00 per cu. yd.
Finishing of floors:	.12 per sq. ft.
Shoring of Steel Beam:	5.00 per jack

Shallow Haunch

In a phone conversation Claudio Gnani of Right Forming advised that the premium for a shallow 3" haunch along the composite beams would be 0 to 5 cents per square foot depending on the forming system used.

WESTEEL-ROSCO LIMITED

1601 MICHAEL ST MOBILE ROUTE NO 1, POSTAL STN E
K1P 5K0

OTTAWA, ONT.
745-6831

28 March 1978

Mr. Drew Allan
Dept of Civil Engineering
Carleton University
Ottawa, Ontario

Dear Drew:

As requested, listed below are the various cost breakdown for floor deck profiles and associated flashings. These prices are based on a twenty store structure and include installation costs, all applicable taxes and freight charges to the job site.

Material Thickness	Profile	Price
.032	T-15 Hi-Bond	.77 per square foot
.038	T-15 Hi-Bond	.82 per square foot
.050	T-15 Hi-Bond	.98 per square foot
.032	T-30-6 HiBond	1.04 per square foot
.038	T-30-6 HiBond	1.12 per square foot
.050	T-30-6 HiBond	1.26 per square foot
.032	T-30-8 HiBond	.96 per square foot
.038	T-30-8 HiBond	1.02 per square foot
.050	T-30-8 HiBond	1.19 per square foot
.032	T-30-V HiBond	.87 per square foot
.038	T-30-V HiBond	.94 per square foot
.050	T-30-V HiBond	1.10 per square foot
.038-.038	T-30-VF HiBond	1.98 per square foot
.050-.050	T-30-VF HiBond	2.42 per square foot

.050 angle closure to close cells of deck - .69 per lineal foot
.062 combination closure and concrete retainer flashing - .83 per lineal foot
cut and flash at columns - 3.50 per column

The finish of the above material would be the standard G-90 1.25 oz galvanized.

Should you have any queries regarding the above or if we may be of further assistance, please do not hesitate to contact the writer.

Yours truly,

WESTEEL-ROSCO LIMITED,

J.D. Dehler,
Building Products Department.

JDD/at

TRISTAN INCORPORATED

75 ALBERT STREET

SUITE 206

OTTAWA, K1P 5E7

March 31, 1978

Department of Civil Engineering,
Carleton University,
Ottawa, Ontario
K1S 5B6

Attention: Mr. A. I. Allan

Dear Drew:

Enclosed please find a summary of cladding and partition costs as discussed on March 23, 1978. We would caution you that the prices are a guide and the unit prices could fluctuate up or down depending on quantities and or heights in excess of 10'. We note also that there is no allowance in the unit prices for the General Contractors mark up, should that be a concern.

We trust this is the information you require.

Yours very truly,
TRISTAN INCORPORATED

E.W. Ruiter, P. ENG. . .

EWR::ma

c.c. J. Adjeleian

CLADDING AND PARTITION COSTS1. MASONRY

BLOCK: 6" block	.45 / block
laying block	1.10 / block
reinforcing steel and grout	.55 / block

Total	2.10 / block
-------	--------------

6" block wall - 2.35/sq.ft.

BRICK: 4" brick	1.50 / 1000 bricks
lay bricks	3.00 / 1000 bricks

Total	4.50 / 1000 brick
-------	-------------------

4" face brick - 2.05/sq.ft.

2. EXTERIOR WALLSPRECAST CONCRETE PANELS:

5" panel	6.50 / sq.ft.
2" rigid insulation	.45 / sq.ft.
5/8" drywall	.60 / sq.ft.
caulking	.35 / sq.ft.

Total	7.90 / sq.ft.
-------	---------------

Note: No cost included
for fancy exterior
finish.

12" CAVITY WALL:

6" block	2.35 / sq.ft.
4" brick	2.05 / sq.ft.
2" rigid insulation	.45 / sq.ft.
5/8" drywall	.60 / sq.ft.

Total	5.45 / sq.ft.
-------	---------------

FACE BRICK AND STUD WALL:

5/8" drywall on studs	2.00 / sq.ft.
3-1/2" Fiberglass insulation	.25 / sq.ft.
Asphalt felt	.05 / sq.ft.
4" face brick	2.05 / sq.ft.

Total	4.35 / sq.ft.
-------	---------------

3. INTERIOR PARTITION

1 hr. fire-rated
1/2" F.C. drywall on studs 2.00 / sq.ft.

1 hr. fire-rated insulated
1/2" F.C. drywall on studs 2.25 / sq.ft.

2 hr. fire-rated
double 1/2" F.C. drywall on studs 2.80 / sq.ft.

6" block wall 2.35 / sq.ft.

VANDENBELT METAL WORKS LTD.

1090 Cummings Avenue, Ottawa, Ontario, Canada K1J 7R8 Telephone (613) 745-9475

STEEL FABRICATORS

May 4, 1978

Mr. Drew Allan
Civil Engineering Department
Carleton University
OTTAWA, Ontario

Dear Sir:

The following are the estimated costs as you requested during our discussion:

- (A) Estimated cost for steel stair per foot increase in floor-to-floor elevation
\$25 - \$30
- (B) Estimated cost per reinforced duct opening in steel beam
\$30 - \$40

I hope that this is sufficient information.

Yours truly,

H. Burt Lilley
H. Burt Lilley,
Estimator

HBL:ar

Head Office
VANDENBELT METAL WORKS LTD.
1090 Cummings Avenue
Ottawa, Ontario K1J 7R8
Telephone (613) 745-9475

HILKRON STEEL
Division of Vandenberg Metal Works Ltd.
5815 Dixie Road North
Mississauga, Ontario L4W 1E8
Telephone (416) 677-3950

VANDENBELT CRANES
Division of Vandenberg Metal Works Ltd.
1090 Cummings Avenue
Ottawa, Ontario K1J 7R8
Telephone (613) 745-9475

VANDENBELT CRANES
Branch Office
5815 Dixie Road North
Mississauga, Ontario L4W 1E8
Telephone (416) 677-3064

OTIS ELEVATOR COMPANY LIMITED

2480 Lancaster Rd.,
Ottawa, Ontario

Mar. 30, 1978

Dept. of Civil Engineering,
Carleton University,
Ottawa, Ontario

Att: Mr. Drew Allan

Subject: Multi-Purpose Office Building

Dear Sir:

We wish to confirm our meeting of March 27, 1978, in which you were advised that, the increase in costs for an elevator designed for a multi-purpose office building, to increase the total rise would be approximately \$150.00 per foot.

This price would consist of approximately 78% material and 22% labour.

The materials that would be affected would include the hoist ropes, compensating ropes, travelling cable, main and counterweight rails, hoistway troughing and wiring, entrance fascias and struts, as well as additional support brackets for the rails.

Trusting the above is the information you require, we remain,

Yours very truly,

OTIS ELEVATOR COMPANY LIMITED

R.J. Bowker

Sales Representative

March 27, 1978

**clemann
large
patterson**

and associates ltd.

Mark Clemann, Eng. S.W.K.I.
President

A. J. Large
Executive Vice-President

G. B. Patterson, B. Eng., P. Eng.
Secretary-Treasurer

D. E. Henselwood, B.Sc., P. Eng.
Vice-President

K. J. Hazell, P. Eng.
Vice-President

Noel Brown
Vice-President

consulting engineers

Mr. A. I. Allan;
Department of Civil Engineering
Carleton University;
Ottawa, Ont.
K1S 5B6

Dear Mr. Allan:

Enclosed is a summary of the mechanical and electrical requirements and costs we discussed on March 6, 1978.

We hope that this information will help you evaluate the mechanical and electrical costs for your buildings.

Yours very truly,

Mark Clemann, Eng., S.W.K.I.
MC/ch
Encl.

2

MECHANICAL SYSTEM

The most appropriate mechanical system for your example building would be a variable volume system, utilizing a 10 foot grid along the 150 ft. dimension with three rows of units per bay.

CEILING-TO-FLOOR DEPTH

Provisions must be made for a 20 inch deep variable volume unit with a 10 inch diameter duct (12" with insulation) running between units, located 2 inches below the top of the unit. The bottom of this unit is located flush with the underside of the finished ceiling.

HEATING

For heating costs, use the equivalency for Ottawa that a heat loss of 80,000 BTU/hr costs approximately \$500.00 per year, (1978 cost).

COOLING

For an air conditioning system in Ottawa, there are approximately 1,000 equivalent full load hours. Use a cost of 2.5 cents per kilowatt.

VENTILATION

The ventilation fans operate at an average of 2,500 hours per year. Due to friction losses, the output requirement for these fans will increase approximately .8 HP for every 5 feet of supply duct.

ENERGY COSTS

It is predicted that energy costs will rise 5% to 8% above the rate of inflation per year.

MECHANICAL AND ELECTRICAL VERTICAL ELEMENTS

Vertical Air Supply Ducts:	4 @ \$60/ft.
Toilet Exhaust:	2 @ \$10/ft.
Chilled Water & Returns:	2 @ \$10/ft.
Heating Water & Returns:	16 @ \$ 8/ft.
Domestic Water Supply:	5 @ \$ 6/ft.
Domestic Drainage:	2 @ \$ 4/ft.
Storm Water Drainage:	1 @ \$ 4/ft.
Fire Hose Cabinet & Sprinkler Riser:	2 @ \$ 5/ft.
Bus Duct:	1 @ \$40/ft.

Appendix B

CISC Structural Steel

Costing Method

CISC Piece by Piece Costing Method for Structural Steel (10)

A "piece by piece costing method" was developed in the Project Analysis Division of the CISC and with the assistance of member fabricators, test verified for accuracy. The method is presented below for consideration of inclusion in structural steel estimating procedures.

Basic Equations for Piece by Piece Costing

$$M_I = A_1 + A_2 + A_3 + A_4 + A_5 + A_6 + A_7$$

$$M_S = M_I \times \alpha_S$$

$$M_{CI} = M_I \times \alpha_{CI}$$

$$M_{CS} = M_I \times \alpha_{CI} \times \alpha_S$$

Where:

$$A_1 = W \times (SE + GE + B + F) \times L_M \times Q_M \div 100$$

$$A_2 = (T_S \times R_S \times Q_S \times L'_S + W \times S \times L_{SH} \div 100) \div \rho$$

$$A_3 = T_D \times R_D \times Q_D \div \rho$$

$$A_4 = R_E \div 8 \div E \times H \times Q_E \times L_E \div \rho$$

$$A_5 = (A_1 \div Q_M - W \times F \div 100) \times FST \div 100$$

$$A_6 = (A_1 + A_2 + A_3 + A_4 + A_5) \times PST \div 100$$

$$A_7 = (A_1 + A_2 + A_3 + A_4) \times GP \div 100$$

Symbols

- A_1 - Material cost with capital finance cost (\$)
- A_2 - Shop cost with overhead plus delivery to site (\$)
- A_3 - Engineering and drafting with overhead (\$)
- A_4 - Erection with overhead (\$)
- A_5 - Federal Sales Tax (\$)
- A_6 - Provincial Sales Tax (\$)
- A_7 - Sales overhead and gross profit (\$)
- B - Modified base prices
 - B_1 (for wide flanges) = $1.015 \times C_1$
 - B_2 (for WWF > 140 PLF) = $1.008 \times C_2$
 - B_3 (for WWF < 140 PLF) = $1.007 \times C_3$
 - B_4 (for HSS) = $1.035 \times C_4$
- C_1 - Base price for Hot Rolled Structural Shapes (\$/100 lbs.)
- C_2 - Base price for WWF > 140 PLF (\$/100 lbs.)
- C_3 - Base price for WWF < 140 PLF (\$/100 lbs.)
- C_4 - Base price for HSS (\$/100 lbs.)
- E - Erection speed (pieces per hour) (see chart 2)
- F - Freight charges mill to fabricator
 - F_1 - for W and WWF sections (\$/100 lbs.)
 - F_2 - for HSS (\$/100 lbs.)
- FST - Federal Sales Tax (%)
- GE - Grade extra (\$/100 lbs.)
- GP - Sales overhead and gross profit (%)
- H - Height correction factor (see chart 3)

Symbols (continued)

- L_E - Erection member length factor (see chart 1)
 L_M - Material length factor (for use with W and WWF only, see chart 1)
 L_S - Shop fabrication length factor (see chart 1)
 L_{SH} - Shipping length factor (see chart 1)
 M_{CI} - Cost of interior members of continuous construction (\$)
 M_{CS} - Cost of spandrel members of continuous construction (\$)
 M_I - Cost of interior members of simple connection, (\$)
 M_S - Cost of spandrel members of simple connection (\$)
PST - Provincial Sales Tax (%)
 Q_D - Engineering and Drafting coefficient to reflect the cost of overhead. Q_D varies from 1.3 to 1.6
 Q_E - Erection cost coefficient to reflect the cost of unloading, sorting, bolting, welding and painting, clean up, preparation, plumbing, etc. Erection overhead* cost are also included.
 Q_E varies from 2.2 to 3.0
 Q_M - Material cost coefficient to reflect:
- cost of capital financing
- cost of material storage
- waste factor
 Q_M varies from 1.05 to 1.08
 Q_S - Shop fabrication coefficient to reflect additional cost of work preparation, sorting, painting, marking,*loading, etc., and overhead* (see note below for appropriate items)
 Q_S varies from 2.0 to 2.8 depending on individual operators.

*Note: Overhead includes: supervisory salaries, travel and personnel expenses, watchman, light, heat, power, telephone, water, insurance, sanitary facilities, office supplies and furniture, small tools, storage sheds, safety barricades, photographs, progress reports, public relations, dewatering, first aid and medical facilities, etc.

Symbols (continued)

- $\bar{R}_D$ - Hourly drafting rate (\$/hr.)
 R_E - Basic erection crew rate + crane rental (\$/day)
 Crew rate = (hourly rate of forman or pusher
 + number of ironworkers x hourly rate of
 ironworkers
 + crane operator(s) x hourly rate of crane
 operator
 + crane cost, fuel charge) x 8 hour day
 R_S - Shop fabrication hourly rate (\$/hour)
 S - Shipping rate (from fabricator to site) (\$/100 lbs.)
 SE - Section extra (\$/100 lbs.)
 T_D - Actual drafting time per member (hours) (see charts 4 and 5)
 T_S - Time spent on member fabrication (hours) (see charts 4 and 5)
 W - Gross member weight = net weight + connection weight (lbs.)
 α_{CI} - Continuous member cost factor (1.3)
 α_S - Spandrel cost factor (1.15)
 ρ - Productivity factor, to reflect individual and regional
 variations

CHART 2

CHART 3

CHART 1

* T VALUES PLOTTED ARE BASED ON SINGLE STOREY LISTS. FOR DOUBLE STOREY LISTS, T IS TO BE MULTIPLIED BY 1.8 AND FOR 3 STOREY LISTS, T IS TO BE MULTIPLIED BY 2.6

Appendix C

Structural Steel Building Skeleton

Five different structural steel office buildings were designed to investigate the effect of storey height and dead load of the floor system on the cost of the building skeleton.

Building No.	Storey Height (S) (ft.)	Floor System Dead Load (psf.)
1	12.5	80
2	11.5	80
3	13.5	80
4	12.5	100
5	12.5	60

In this appendix a detailed design brief for Building No. 1 is presented as well as the final designs for the remaining buildings. The buildings have been designed in accordance with CSA S16.1 - Limit States Design (8). The steel grade used was G40.21-44W. A quantity take-off and cost estimate utilizing the CISC "piece by piece" costing method is performed for each building.

EXAMPLE BUILDING

Building No. 1

Storey Height (S) = 12.5 ft

Dead Load (DL) = 80 psf.

Specified Loads (psf.)

The building is assumed to be located in Ottawa.

Live Load:	Office Floor	= 50
	Mechanical Floor	= 75
	Roof (.8 x ground snow)	= 48
	Core Area (office floors)	= 100
Dead Load:	Office Floor	= DL = 80
	Mechanical Floor*	= DL + 120 = 200
	Roof	= DL = 80
	Allowance for weight of columns	= 5
Partition and Wall Load:	Movable Office Partitions (assumed reducible live load)	= 20
	Core Walls	= 50
	Exterior Wall	= 60 lbs./sf. of walls
	Glazing in Exterior Wall**	= 10 lbs./sf. of glazing

* 120 psf. is to account for a heavier structural floor and mechanical equipment

** 0-1 66% of the floor to ceiling space of glazing
 1-2 to 18-19 50% of the floor to ceiling space of glazing
 19-20 0% of the floor to ceiling space of glazing

Column Design for Gravity Loads

- a Department of Public Works (DPW) program "COLO" was used to calculate the column loads due to gravity
- the results are presented on pages 104 to 108
- this program requires that the wall loads be represented as an equivalent distributed dead load
- similarly different live loadings over portions of the tributary area must be represented by an average loading
- the columns were designed in 2-storey tiers to reduce splicing costs
- since P- Δ effects are included in the analysis of the braced frames and only simple connections are used, $K = 1$ for all columns
- moments due to eccentric connections will be considered only for columns C4 and C5 as these are exterior columns without a member framing into the opposite side to balance the eccentric load

Column Designations

• COLUMN C1

$$\text{TRIBUTARY AREA (TA)} = 37.5 \times 22.5 = 844 \text{ FT}^2$$

$$\text{DIST. D.L. (psf): } 1-18 = 85 + \frac{37.5 \times 525}{844} = 108$$

$$19 = 205 + \frac{37.5 \times 1110}{844} = 254$$

$$20 = 80$$

$$\text{DIST. L.L. (psf): } 1-18 = 70$$

$$19 = 75$$

$$20 = 0$$

$$\text{DIST. S.L. (psf): } 1-19 = 0$$

$$20 = 48$$

• COLUMN C2

$$\text{TA} = 37.5 \times 37.5 = 1406 \text{ FT}^2 \quad \left(\begin{array}{l} 1144 \text{ FT}^2 - \text{OFFICE} \\ 262 \text{ FT}^2 - \text{CORE} \end{array} \right)$$

$$\text{DIST. D.L. (psf): } 1-18 = \frac{1144 \times 85 + 262 \times 135}{1406} = 94$$

$$19 = 205$$

$$20 = 80$$

$$\text{DIST. L.L. (psf): } 1-18 = \frac{1144 \times 70 + 262 \times 100}{1406} = 76$$

$$19 = 75$$

$$20 = 0$$

$$\text{DIST. S.L. (psf): } 1-19 = 0$$

$$20 = 48$$

• COLUMN C3

$$TA = 30 \times 37.5 = 1125 \text{ FT}^2 \begin{pmatrix} 600 \text{ FT}^2 - \text{OFFICE} \\ 525 \text{ FT}^2 - \text{CORE} \end{pmatrix}$$

$$\text{DIST. D.L. (psf): } 1-18 = \frac{600 \times 85 + 525 \times 135}{1125} = 108$$

$$19 = 205$$

$$20 = 80$$

$$\text{DIST. L.L. (psf): } 1-18 = \frac{600 \times 70 + 525 \times 100}{1125} = 84$$

$$19 = 75$$

$$20 = 0$$

$$\text{DIST. S.L. (psf): } 1-19 = 0$$

$$20 = 48$$

• COLUMN C4

$$TA = 15 \times 20 = 300 \text{ FT}^2$$

$$\text{DIST. D.L. (psf): } 1-18 = 85 + \frac{35 \times 525}{300} = 146$$

$$19 = 205 + \frac{35 \times 1110}{300} = 335$$

$$20 = 80$$

$$\text{DIST. L.L. (psf): } 1-18 = 70$$

$$19 = 75$$

$$20 = 0$$

$$\text{DIST. S.L. (psf): } 1-19 = 0$$

$$20 = 48$$

MOMENT DUE TO ECCENTRIC CONNECTION: (ASSUME $e = 10"$)

$$1-18 \rightarrow R_D = (20 \times 525 + 80 \times 150) 1.25 \\ = 28.1^k$$

$$R_L = 70 \times 150 \times 1.5 \\ = 15.8^k$$

$$M_{D+L} = 36.6^{1k} \quad M_D = 23.4^{1k}$$

$$19 \rightarrow R_D = (20 \times 1110 + 200 \times 150) 1.25 \\ = 65.3^k$$

$$R_L = 75 \times 150 \times 1.5 \\ = 16.9^k$$

$$M_{D+L} = 68.5^{1k} \quad M_D = 54.4^{1k}$$

$$20 \rightarrow R_D = 80 \times 150 \times 1.25 \\ = 15.0^k$$

$$R_L = 48 \times 150 \times 1.5 \\ = 10.8^k$$

$$M_{D+L} = 21.5^{1k} \quad M_D = 12.5^{1k}$$

• COLUMN C5

$$TA = 30 \times 20 = 600 \text{ FT}^2$$

$$\text{DIST. D.L. (psf): } 1-18 = 85 + \frac{30 \times 525}{600} = 111$$

$$19 = 205 + \frac{30 \times 1110}{600} = 261$$

$$20 = 80$$

$$\begin{aligned} \text{DIST. LL (psf): } 1-18 &= 70 \\ 19 &= 75 \\ 20 &= 0 \end{aligned}$$

$$\begin{aligned} \text{DIST. SL (psf): } 1-19 &= 0 \\ 20 &= 48 \end{aligned}$$

MOMENT DUE TO ECCENTRIC CONNECTION: (ASSUME $C=10"$)

$$1-18 \rightarrow R_D = 80 \times 300 \times 1.25 = 30.0^k$$

$$R_L = 70 \times 300 \times 1.5 = 31.5^k$$

$$M_{D+L} = 51.3^{1k} \quad M_D = 25.0^{1k}$$

$$\begin{aligned} 19 \rightarrow R_D &= 200 \times 300 \times 1.25 \\ &= 75.0^k \end{aligned}$$

$$\begin{aligned} R_L &= 75 \times 300 \times 1.5 \\ &= 33.8^k \end{aligned}$$

$$M_{D+L} = 90.6^{1k} \quad M_D = 62.5^{1k}$$

$$\begin{aligned} 20 \rightarrow R_D &= 80 \times 300 \times 1.25 \\ &= 30.0^k \end{aligned}$$

$$\begin{aligned} R_L &= 48 \times 300 \times 1.5 \\ &= 21.6^k \end{aligned}$$

$$M_{D+L} = 43.0^k \quad M_D = 25.0^{1k}$$

DREW ALLAN

3/1/78

PAGE

1

PUBLIC WORKS CANADA ----- STRUCT COLD 77 09 01

EX BLDG (S=12.5 & DL=80)

COLUMN LOADS DUE TO GRAVITY, NBC 1977 AND CSA S16.1 1973

COLUMN MK C1

LOAD INCREMENTS ARE SHOWN AT STORY LEVELS, TOTALS ARE SHOWN
BETWEEN STORIESFACTORED LOAD = 1.25 X DEAD LOAD + 1.5 X SNOW LOAD
+ 1.5 X LIVE LOAD

SL = SNOW LOAD OR LIVE LOAD NOT SUBJECT TO REDUCTION

LL = LIVE LOAD SUBJECTED TO REDUCTION

LEVEL	TRIB AREA SQ. FT	DIST DL PSF	CONC DL KIPS	DIST SL PSF	CONC SL KIPS	DIST LL PSF	CONC LL KIPS	LL REDUC FACT	DEAD LOAD KIPS	SNOW LOAD KIPS	UNRED LL KIPS	RED LL KIPS	FACT LOAD KIPS
20	844.	80	0.0	48.	0.0	0	0.0		68	41.	0		
	0.							1 000	68.	41.	0	0	145
19	844.	254.	0.0	0	0.0	75.	0.0		214.	0	63		
	844.							644	282.	41.	63	41	474
18	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	1688.							543	373.	41.	122.	67	627
17	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	2532.							499	464.	41.	181.	90	777
16	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	3376.							472	555.	41.	241.	114	925
15	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	4220.							454	647.	41.	300.	136	1073
14	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	5064.							441	738.	41.	359.	158	1220
13	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	5908.							430	829.	41.	418.	180	1366
12	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	6752.							422	920.	41.	477.	201	1512
11	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	7596.							415	1011.	41.	536.	222	1658
10	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	8440.							409	1102.	41.	595.	243	1804
9	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	9284.							404	1193.	41.	654.	264	1949
8	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	10128.							399	1285.	41.	713.	285	2094
7	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	10972.							395	1376.	41.	772.	305	2239
6	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	11816.							392	1467.	41.	831.	326	2383
5	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	12660.							389	1558.	41.	890.	346	2528
4	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	13504.							386	1649.	41.	949.	367	2672
3	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	14348.							383	1740.	41.	1009.	387	2816
2	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	15192.							381	1831.	41.	1068.	407	2960
1	844.	108.	0.0	0.	0.0	70.	0.0		91.	0	59		
	16036.							379	1923.	41.	1127.	427	3105

DREN ALLAN

3/1/78

PAGE

1

PUBLIC WORKS CANADA - - - - - STRUCT. COLO 77 09 01

EX BLDG (S=12.5 & DL=80)

COLUMN LOADS DUE TO GRAVITY, NBC 1977 AND CSA S16.1 1973

COLUMN MK C2

LOAD INCREMENTS ARE SHOWN AT STORY LEVELS. TOTALS ARE SHOWN
BETWEEN STORIESFACTORED LOAD = 1.25 X DEAD LOAD + 1.5 X SNOW LOAD
+ 1.5 X LIVE LOAD

SL = SNOW LOAD OR LIVE LOAD NOT SUBJECT TO REDUCTION

LL = LIVE LOAD SUBJECTED TO REDUCTION

	TRIB	DIST	CONC	DIST	CONC	DIST	CONC	LL	DEAD	SNOW	UNRED	RED	FACT
LEVEL	AREA	DL	DL	SL	SL	LL	LL	LL	LOAD	LOAD	LL	LL	LOAD
	SQ. FT	PSF	KIPS	PSF	KIPS	PSF	KIPS	FACT	KIPS	KIPS	KIPS	KIPS	KIPS
20	1406.	80.	0.0	48.	0.0	0.	0.0	112	67	0.	0	242	
19	1406.	205.	0.0	0.	0.0	75.	0.0	1000	112	67	0	0	242
	1406.							567	401	67	105	60	692
18	1406.	94	0.0	0.	0.0	76.	0.0	132	0	107.			
	2812							489	533	67	212	104	923
17	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107			
	4218							454	665	67	319.	145.	1150
16	1406.	94	0.0	0.	0.0	76.	0.0	132	0	107			
	5624							433	797.	67	426	185.	1375
15	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	7030							419	929	67	533.	223.	1598.
14	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	8436							409	1062.	67	640.	262.	1820.
13	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	9842							401	1194.	67	747	299.	2042
12	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	11248							394	1326.	67	853	337.	2263.
11	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	12654							389	1458.	67	960	373.	2484.
10	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107			
	14060							384	1590.	67	1067.	410.	2704.
9	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	15466							380	1722.	67	1174.	447	2924.
8	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107			
	16872							377	1855.	67	1281.	483.	3144.
7	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	18278							374	1987.	67	1388	519.	3363.
6	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	19684							371	2119.	67	1495.	555.	3582.
5	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	21090							369	2251.	67	1601.	591.	3801.
4	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	22496							367	2383.	67	1708.	626.	4020.
3	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	23902							365	2515.	67	1815.	662.	4238.
2	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	25308							363	2647.	67	1922.	697.	4457.
1	1406.	94.	0.0	0.	0.0	76.	0.0	132	0	107.			
	26714							361	2780.	67	2029.	733.	4675.

DREW ALLAN

3/1/78

PAGE

1

PUBLIC WORKS CANADA ----- STRUCT: COLO-77 09 01.

EX BLDG (S=12.5 & DL=80)

COLUMN LOADS DUE TO GRAVITY, NBC 1977 AND CSA S16.1 1973

COLUMN NO: C3

LOAD INCREMENTS ARE SHOWN AT STORY LEVELS, TOTALS ARE SHOWN
BETWEEN STORIESFACTORED LOAD = 1.25 X DEAD LOAD + 1.5 X SNOW LOAD
+ 1.5 X LIVE LOAD

SL = SNOW LOAD OR LIVE LOAD NOT SUBJECT TO REDUCTION

LL = LIVE LOAD SUBJECTED TO REDUCTION

LEVEL	TRIB AREA SQ. FT	DIST DL PSF	CONC DL KIPS	DIST SL PSF	CONC SL KIPS	DIST LL PSF	CONC LL KIPS	LL REDUC FACT	DEAD LOAD KIPS	SNOW LOAD KIPS	UNRED LL KIPS	RED LL KIPS	FACT LOAD KIPS	SECTION
20	1125 0.	80.	0.0	48.	0.0	0.	0.0	1.000	90	54	0	0	193	
19	1125 1125.	205	0.0	0.	0.0	75.	0.0	231	0	84.	0	84.	557	W12x65
18	1125 2250.	108.	0.0	0.	0.0	84.	0.0	121	0	94	0	94	771	
17	1125 3375.	108.	0.0	0.	0.0	84.	0.0	511	442	54	179	91	979	WWF(M)16x105
16	1125 4500.	108.	0.0	0.	0.0	84.	0.0	121	0	94	0	94	1185	
15	1125 5625.	108.	0.0	0.	0.0	84.	0.0	472	564	54	273	129	979	WWF(M)16x105
14	1125 6750.	108.	0.0	0.	0.0	84.	0.0	449	685	54	368	165	1185	
13	1125 7875.	108.	0.0	0.	0.0	84.	0.0	121	0	94	0	94	1390	WWF(M)18x134
12	1125 9000.	108.	0.0	0.	0.0	84.	0.0	433	807	54	462	200	1390	WWF(M)18x134
11	1125 10125.	108.	0.0	0.	0.0	84.	0.0	121	0	94	0	94	1593	
10	1125 11250.	108.	0.0	0.	0.0	84.	0.0	422	928	54	557	235	1593	
9	1125 12375.	108.	0.0	0.	0.0	84.	0.0	121	0	94	0	94	1796	WWF(M)20x170
8	1125 13500.	108.	0.0	0.	0.0	84.	0.0	413	1050	54	651	269	1796	WWF(M)20x170
7	1125 14625.	108.	0.0	0.	0.0	84.	0.0	121	0	94	0	94	1998	
6	1125 15750.	108.	0.0	0.	0.0	84.	0.0	405	1171	54	746	302	1998	
5	1125 16875.	108.	0.0	0.	0.0	84.	0.0	121	0	94	0	94	2200	WWF(M)20x205
4	1125 18000.	108.	0.0	0.	0.0	84.	0.0	399	1293	54	840	336	2200	WWF(M)20x205
3	1125 19125.	108.	0.0	0.	0.0	84.	0.0	121	0	94	0	94	2402	
2	1125 20250.	108.	0.0	0.	0.0	84.	0.0	394	1414	54	935	369	2402	
1	1125 21375.	108.	0.0	0.	0.0	84.	0.0	121	0	94	0	94	2603	WWF(M)20x256
								390	1536	54	1029	401	2603	WWF(M)20x256
								121	0	94	0	94	2803	
								386	1657	54	1124	434	2803	
								121	0	94	0	94	3004	WWF(M)20x306
								383	1779	54	1218	466	3004	WWF(M)20x306
								121	0	94	0	94	3204	
								380	1900	54	1313	498	3204	
								121	0	94	0	94	3404	WWF(M)20x377
								377	2022	54	1407	531	3404	WWF(M)20x377
								121	0	94	0	94	3604	
								375	2143	54	1502	563	3604	
								121	0	94	0	94	3803	WWF(M)20x377
								372	2265	54	1596	594	3803	WWF(M)20x377
								121	0	94	0	94	4003	
								370	2386	54	1691	626	4003	
								121	0	94	0	94	4202	WWF(M)20x437
								368	2508	54	1785	658	4202	WWF(M)20x437

DREN ALLAN

3/1/78

PAGE

1

PUBLIC WORKS CANADA ----- STRUCT. COLO. 77.09.01
 EX BLDG (S=12.5 & DL=80)
 COLUMN LOADS DUE TO GRAVITY, NBC 1977 AND CSA S16.1 1973
 COLUMN MK C4

LOAD INCREMENTS ARE SHOWN AT STORY LEVELS. TOTALS ARE SHOWN
 BETWEEN STORIES

FACTORED LOAD = 1.25 X DEAD LOAD + 1.5 X SNOW LOAD
 + 1.5 X LIVE LOAD

SL = SNOW LOAD OR LIVE LOAD NOT SUBJECT TO REDUCTION

LL = LIVE LOAD SUBJECTED TO REDUCTION

LEVEL	TRIB AREA SQ FT	DIST DL PSF	CONC DL KIPS	DIST SL PSF	CONC SL KIPS	DIST LL PSF	CONC LL KIPS	LL REDUC FACT	DEAD LOAD KIPS	SNOW LOAD KIPS	UNRED LL KIPS	RED LL KIPS	FACT LOAD KIPS	BENDING ABOUT X-AXIS		SECTION
														M ₁	M ₂	
20	300	80	0.0	48	0.0	0	0.0	24	14	0	0	52	0	36.6		
19	300	335	0.0	0	0.0	75	0.0	1.000	24	14	0	0	52	0	36.6	
	300							877	124	14	22	20	207	23.4	36.6	WBx40
18	300	146	0.0	0	0.0	70	0.0	44	0	21	44	0	21	278	23.4	36.6
17	300	146	0.0	0	0.0	70	0.0	708	168	14	43	31	348	23.4	36.6	WBx43
16	300	146	0.0	0	0.0	70	0.0	633	212	14	64	41	417	23.4	36.6	WBx43
15	300	146	0.0	0	0.0	70	0.0	589	256	14	85	50	417	23.4	36.6	
14	300	146	0.0	0	0.0	70	0.0	558	300	14	106	59	485	23.4	36.6	WBx60
13	300	146	0.0	0	0.0	70	0.0	536	343	14	127	68	553	23.4	36.6	
12	300	146	0.0	0	0.0	70	0.0	518	387	14	148	77	621	23.4	36.6	WBx72
11	300	146	0.0	0	0.0	70	0.0	504	431	14	169	85	689	23.4	36.6	
10	300	146	0.0	0	0.0	70	0.0	492	475	14	190	94	756	23.4	36.6	WBx85
9	300	146	0.0	0	0.0	70	0.0	483	519	14	211	102	823	23.4	36.6	
8	300	146	0.0	0	0.0	70	0.0	474	562	14	232	110	890	23.4	36.6	WWF(M)16x92
7	300	146	0.0	0	0.0	70	0.0	467	606	14	253	118	957	23.4	36.6	
6	300	146	0.0	0	0.0	70	0.0	460	650	14	274	126	1024	23.4	36.6	WWF(M)16x105
5	300	146	0.0	0	0.0	70	0.0	454	694	14	295	134	1090	23.4	36.6	
4	300	146	0.0	0	0.0	70	0.0	449	738	14	316	142	1157	23.4	36.6	WWF(M)18x119
3	300	146	0.0	0	0.0	70	0.0	444	781	14	337	150	1223	23.4	36.6	
2	300	146	0.0	0	0.0	70	0.0	440	825	14	358	158	1290	23.4	36.6	WWF(M)18x134
1	300	146	0.0	0	0.0	70	0.0	436	869	14	379	163	1356	23.4	68.5	
	5700							432	913	14	400	173	1422	18.5	68.5	WWF(M)20x150

DREW ALLAN

3/1/78

PAGE

1

PUBLIC WORKS CANADA ----- STRUCT. COLD 77 09.01

EX BLDG (S=12.5 & DL=80)

COLUMN LOADS DUE TO GRAVITY, NBC 1977 AND CSA S16.1 1973

COLUMN MK CS

LOAD INCREMENTS ARE SHOWN AT STORY LEVELS. TOTALS ARE SHOWN
BETWEEN STORIESFACTORED LOAD = 1.25 X DEAD LOAD + 1.5 X SNOW LOAD
+ 1.5 X LIVE LOAD

SL = SNOW LOAD OR LIVE LOAD NOT SUBJECT TO REDUCTION

LL = LIVE LOAD SUBJECTED TO REDUCTION

LEVEL	TRIB AREA SQ FT	DIST DL PSF	CONC DL KIPS	DIST SL PSF	CONC SL KIPS	DIST LL PSF	CONC LL KIPS	LL REDUC FACT	DEAD LOAD KIPS	SNOW LOAD KIPS	UNRED LL KIPS	RED LL KIPS	FACT LOAD KIPS	BENDING ABOUT X-AXIS		SECTION
														M ₁	M ₂	
20	600	80	0.0	48	0.0	0	0.0		48	29	0	0	103	0	51.3	
	0							1.000	48	29	0	0				
19	600	261.	0.0	0	0.0	75	0.0		157	0	45					
	600							708	205	29	45	32	347	25.0	51.3	W10x54
18	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	1200							589	271	29	87	51	459	25.0	51.3	
17	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	1800							536	338	29	129	69	569	25.0	51.3	W12x65
16	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	2400							504	404	29	171	86	678	25.0	51.3	
15	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	3000							483	471	29	213	103	786	25.0	51.3	W12x85
14	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	3600							467	538	29	255	119	894	25.0	51.3	
13	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	4200							454	604	29	297	135	1001	25.0	51.3	WWF(M)16x105
12	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	4800							444	671	29	339	151	1108	25.0	51.3	
11	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	5400							436	737	29	381	166	1214	25.0	51.3	WWF(M)18x113
10	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	6000							429	804	29	423	182	1320	25.0	51.3	
9	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	6600							423	871	29	465	197	1427	25.0	51.3	WWF(M)20x150
8	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	7200							418	937	29	507	212	1532	25.0	51.3	
7	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	7800							413	1004	29	549	227	1638	25.0	51.3	WWF(M)20x170
6	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	8400							409	1070	29	591	242	1744	25.0	51.3	
5	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	9000							405	1137	29	633	257	1849	25.0	51.3	WWF(M)20x185
4	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	9600							402	1204	29	675	271	1955	25.0	51.3	
3	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	10200							399	1270	29	717	286	2060	25.0	51.3	WWF(M)20x205
2	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	10800							396	1337	29	759	301	2165	25.0	90.6	
1	600	111.	0.0	0	0.0	70	0.0		67	0	42					
	11400							394	1403	29	801	315	2270	25.0	90.6	WWF(M)20x230

Spandrel Design

- the spandrel beams and beams in the braced frames will be designed non-compositely but will be considered fully laterally supported by the slab
- the economy of composite construction for the spandrel beams is reduced because the slab projects from only one side
- the braced frame beams have short spans continuous over a center support and carry axial loads, therefore composite construction will not be used
- these beams can be designed non-compositely without a significant reduction in economy

Beam Designations

• BEAM 4

$$TA = 300 \text{ FT}^2$$

$$\text{LIVE LOAD REDUCTION FACTOR (LLRF)} = .3 + \frac{10}{\sqrt{300}} = 0.88$$

$$1-18 \rightarrow M = \frac{300 (80 \times 1.25 + 70 \times 1.5 \times .88) \times 30}{4000} + \frac{1.25 \times 525 \times 30^2}{8000}$$

$$= 432.9 + 73.8 = 507 \text{ }^{\text{K}}$$

$$V = \frac{300 (80 \times 1.25 + 70 \times 1.5 \times .88) + 525 \times 1.25 \times 30}{2000}$$

$$= 38.7 \text{ }^{\text{K}}$$

$$19 \rightarrow M = \frac{300 (200 \times 1.25 + 75 \times 1.5 \times .88) \times 30}{4000} + \frac{1110 \times 1.25 \times 30^2}{8000}$$

$$= 785.3 + 156.1 = 941 \text{ }^{\text{K}}$$

$$V = \frac{300 (200 \times 1.25 + 75 \times 1.5 \times .88) + 1110 \times 1.25 \times 30}{2000}$$

$$= 73.2 \text{ }^{\text{K}}$$

$$20 \rightarrow M = \frac{300 (80 \times 1.25 + 48 \times 1.5) \times 30}{4000}$$

$$= 387 \text{ }^{\text{K}}$$

$$V = \frac{300 (80 \times 1.25 + 48 \times 1.5)}{2000}$$

$$= 25.8 \text{ }^{\text{K}}$$

B4	M_f (K)	V_f (K)	SECTION
1-18	507	38.7	W24x68
19	941	73.2	W24x110
20	387	25.8	W21x55

CHECK DEFLECTION:

B4	I (IN ⁴)	P_L (K)	$\Delta_L = \frac{P_L^3}{48EI}$ (IN)	$\Delta_{ALL} = \frac{L}{360}$ (IN)
1-18	1,820	18.5	0.34	1.00
19	3,330	19.8	0.20	1.00
20	1,140	14.4	0.42	1.00

• BEAM 5

$$TA = 300 \text{ FT}^2 \quad LLRF = .3 + 10/\sqrt{300} = 0.88$$

$$1-18 \rightarrow M = \frac{[(80 \times 1.25 + 70 \times 1.5 \times .88) 7.5 + 1.25 \times 525] 40^2}{8000}$$

$$= 420 \text{ K}$$

$$V = \frac{[(80 \times 1.25 + 70 \times 1.5 \times .88) 7.5 + 1.25 \times 525] 40}{2000}$$

$$= 42.0 \text{ K}$$

$$19 \rightarrow M = \frac{[(200 \times 1.25 + 75 \times 1.5 \times .88) 7.5 + 1.25 \times 1110] 40^2}{8000}$$

$$= 801 \text{ K}$$

$$V = \frac{[(200 \times 1.25 + 75 \times 1.5 \times .88) 7.5 + 1.25 \times 1110] 40}{2000}$$

$$= 80.1 \text{ K}$$

$$20 \rightarrow M = \frac{(80 \times 1.25 + 48 \times 1.5) 7.5 \times 40^2}{8000}$$

$$= 258 \text{ k}$$

$$V = \frac{(80 \times 1.25 + 48 \times 1.5) 7.5 \times 40}{2000}$$

$$= 25.8 \text{ k}$$

B5	M_f (k)	V_f (k)	SECTION
1-18	420	42.0	W21 x 55
19	801	80.1	W24 x 94
20	258	25.8	W18 x 40

CHECK DEFLECTION :

B5	I (in ⁴)	w_L (k/ft)	$\Delta_m = \frac{5w_L l^4}{384EI}$ (in)	$\Delta_{ML} = \frac{L}{360}$ (in)
1-18	1,140	0.462	0.80	1.33
19	2,690	0.495	0.37	1.33
20	618	0.360	1.16	1.33

Wind Loads

- the assumed location is Ottawa, $q = 7.8$ psf. (1/30)
- a DPW program "WIF" was used to calculate wind forces on the building
- the results are presented on page 115

Earthquake Loads

- the assumed location is Ottawa, $A = .04$
- a DPW program "EQF" was used to calculate earthquake forces on the building
- the results are presented on page 116
- building parameters:

Perimeter	=	530 ft.
Gross Area	=	17,250 sf./floor
Core Area	=	2,100 sf./floor
Office Area	=	15,150 sf./floor
Snow Load Roof	=	48 psf.
Dead Load Roof	=	80 psf.
Dead Load Mechanical	=	205 psf.
Dead Load Office	=	85 psf.
Dead Load Core	=	135 psf.
Exterior Wall* Roof Level	=	555 plf.
Exterior Wall* Mechanical Level	=	818 plf.
Exterior Wall* Office Level	=	525 plf.
Exterior Wall* Level 1	=	674 plf.

*The exterior wall inertia weights were determined using half of the wall above and below the level

$$\text{BLDG. PERIOD N-S} = \frac{.05 \times 262}{\sqrt{115}} = 1.22$$

$$\text{BLDG. PERIOD E-W} = \frac{.05 \times 262}{\sqrt{150}} = 1.07$$

• INERTIA WEIGHTS :

$$\begin{aligned} 1 \rightarrow W &= \frac{15,150 \times 85 + 2100 \times 135 + 530 \times 674}{1000} \\ &= 1929^k \end{aligned}$$

$$\begin{aligned} 2-18 \rightarrow W &= \frac{15150 \times 85 + 2100 \times 135 + 530 \times 525}{1000} \\ &= 1850^k \end{aligned}$$

$$\begin{aligned} 19 \rightarrow W &= \frac{17250 \times 205 + 530 \times 818}{1000} \\ &= 3970^k \end{aligned}$$

$$\begin{aligned} 20 \rightarrow W &= \frac{17250 \times .25 \times 48 + 17250 \times 80 + 530 \times 555}{1000} \\ &= 1881^k \end{aligned}$$

PUBLIC WORKS CANADA ----- STRUCT WIF 77 08 19

WIND LOAD EFFECTS ON MULTISTORY BUILDINGS

NATIONAL BUILDING CODE OF CANADA 1977

EX BLDG (S=12.5 & DL=80) N-S

SIMPLE PROCEDURE USED FOR COMPUTATION OF
EXPOSURE FACTOR AND GUST FACTOR

WIND VELOCITY PRESSURE q FOR 30 YEAR RETURN PERIOD = 7.80 PSF
 EXTERNAL PRESSURE COEFFICIENT (WINDWARD+LEEWARD) C_p = 1.30
 GUST EFFECT FACTOR C_g = 2.00
 EXPOSURE CONDITION = A
 ELEVATION OF GRADE = 0.00 FT
 HEIGHT TO AVERAGE WIDTH RATIO OF THE BUILDING = 1.75

METHOD OF FIGURE B-6 OF COMMENTARIES ON PART 4 OF NBC 1975 USED

ALL UNITS ARE FEET, KIPS, AND SECONDS, EXCEPT AS NOTED

PRESSURE FACTOR = EXTERNAL PRESSURE COEFFICIENT
 X EXPOSURE FACTOR FOR WINDWARD WALL + THE SAME FOR
 LEEWARD WALL

STORY LEVEL	STORY ELEVATION	INTER- STORY HEIGHT	STORY WIDTH	HORI- ZONTAL FORCE	INTER- STORY SHEAR	MOMENT	PRESSURE FACTOR
PROFET							
TOP	262.00			0.0		0	2.20
		0.00	0.0		0		
20	262.00	18.50	150.0	47.7	48	0	2.20
19	243.50	12.50	150.0	79.8	127	882	2.20
18	231.00	12.50	150.0	64.4	192	2475	2.20
17	218.50	12.50	150.0	64.4	256	4824	2.20
16	206.00	12.50	150.0	64.4	321	8877	2.20
15	193.50	12.50	150.0	64.4	385	12086	2.20
14	181.00	12.50	150.0	64.4	449	16899	2.20
13	168.50	12.50	150.0	64.4	514	22518	2.20
12	156.00	12.50	150.0	64.4	578	28941	2.20
11	143.50	12.50	150.0	64.4	643	36169	2.20
10	131.00	12.50	150.0	64.4	707	44202	2.20
9	118.50	12.50	150.0	64.4	771	53048	2.20
8	106.00	12.50	150.0	64.4	836	62882	2.20
7	93.50	12.50	150.0	64.4	900	73128	2.20
6	81.00	12.50	150.0	64.4	965	84388	2.20
5	68.50	12.50	150.0	64.4	1029	96468	2.20
4	56.00	12.50	150.0	64.4	1093	109388	2.20
3	43.50	12.50	150.0	64.4	1158	122978	2.20
2	31.00	12.50	150.0	64.4	1222	137462	2.20
1	18.50	18.50	150.0	79.8	1388	152728	2.20
0	0.00			47.7		176887	

PUBLIC WORKS CANADA ----- STRUCT WIF 77 08 19

WIND LOAD EFFECTS ON MULTISTORY BUILDINGS

NATIONAL BUILDING CODE OF CANADA 1977

EX BLDG (S=12.5 & DL=80) E-W

SIMPLE PROCEDURE USED FOR COMPUTATION OF
EXPOSURE FACTOR AND GUST FACTOR

WIND VELOCITY PRESSURE q FOR 30 YEAR RETURN PERIOD = 7.80 PSF
 EXTERNAL PRESSURE COEFFICIENT (WINDWARD+LEEWARD) C_p = 1.30
 GUST EFFECT FACTOR C_g = 2.00
 EXPOSURE CONDITION = A
 ELEVATION OF GRADE = 0.00 FT
 HEIGHT TO AVERAGE WIDTH RATIO OF THE BUILDING = 2.28

METHOD OF FIGURE B-7 OF COMMENTARIES ON PART 4 OF NBC 1975 USED

ALL UNITS ARE FEET, KIPS, AND SECONDS, EXCEPT AS NOTED

PRESSURE FACTOR = EXTERNAL PRESSURE COEFFICIENT
 X EXPOSURE FACTOR FOR WINDWARD WALL + THE SAME FOR
 LEEWARD WALL

STORY LEVEL	STORY ELEVATION	INTER- STORY HEIGHT	STORY WIDTH	HORI- ZONTAL FORCE	INTER- STORY SHEAR	MOMENT	PRESSURE FACTOR
PROFET							
TOP	262.00			0.0		0	2.22
		0.00	0.0		0		
20	262.00	18.50	115.0	36.9	37	0	2.22
19	243.50	12.50	115.0	61.5	98	682	2.19
18	231.00	12.50	115.0	48.9	147	1912	2.17
17	218.50	12.50	115.0	48.5	194	3754	2.15
16	206.00	12.50	115.0	48.0	244	6201	2.13
15	193.50	12.50	115.0	47.4	291	9248	2.10
14	181.00	12.50	115.0	46.9	338	12888	2.08
13	168.50	12.50	115.0	46.3	384	17114	2.05
12	156.00	12.50	115.0	45.7	430	21919	2.02
11	143.50	12.50	115.0	45.1	475	27296	2.00
10	131.00	12.50	115.0	44.4	520	33228	1.96
9	118.50	12.50	115.0	43.7	563	39731	1.93
8	106.00	12.50	115.0	42.9	606	46772	1.89
7	93.50	12.50	115.0	42.0	648	54349	1.86
6	81.00	12.50	115.0	41.1	689	62452	1.81
5	68.50	12.50	115.0	40.1	729	71087	1.76
4	56.00	12.50	115.0	38.9	768	80187	1.71
3	43.50	12.50	115.0	37.4	806	89791	1.64
2	31.00	12.50	115.0	35.9	842	99845	1.56
1	18.50	18.50	115.0	42.3	885	110889	1.56
0	0.00			25.8		126745	

PUBLIC WORKS CANADA' ----- STRUCT EBF 77 08 05

PUBLIC WORKS CANADA' ----- STRUCT EBF 77 08 05

EARTHQUAKE EFFECTS ON MULTISTORY BUILDINGS
NATIONAL BUILDING CODE OF CANADA 1977EARTHQUAKE EFFECTS ON MULTISTORY BUILDINGS
NATIONAL BUILDING CODE OF CANADA 1977

EX BLDG (S=12.5 & DL=80) M-5

EX BLDG (S=12.5 & DL=80) E-W

ACCELERATION RATIO, A = 0.40
 FORCE REDUCTION FACTOR, K = 1.00
 IMPORTANCE FACTOR, I = 1.00
 FOUNDATION FACTOR, F = 1.00
 ESTIMATED NATURAL PERIOD OF BUILDING = 1.22 SEC
 WIDTH OF LATERAL FORCE RESISTING SYSTEM, DS = 35.00 FT
 SEISMIC INERTIA WEIGHT OF STRUCTURE = 39230 KIPS
 RATIO OF BASE SHEAR TO SEISMIC INERTIA WEIGHT = 0.187
 BASE SHEAR, V = 735 KIPS
 EXTRA TOP FORCE, FT = 110.2 KIPS

ACCELERATION RATIO, A = 0.40
 FORCE REDUCTION FACTOR, K = 1.00
 IMPORTANCE FACTOR, I = 1.00
 FOUNDATION FACTOR, F = 1.00
 ESTIMATED NATURAL PERIOD OF BUILDING = 1.07 SEC
 WIDTH OF LATERAL FORCE RESISTING SYSTEM, DS = 60.00 FT
 SEISMIC INERTIA WEIGHT OF STRUCTURE = 39230 KIPS
 RATIO OF BASE SHEAR TO SEISMIC INERTIA WEIGHT = 0.196
 BASE SHEAR, V = 767 KIPS
 EXTRA TOP FORCE, FT = 58.5 KIPS

ALL UNITS ARE FEET, KIPS AND SECONDS

ALL UNITS ARE FEET, KIPS AND SECONDS

STORY	STORY	INTER-	STORY	STORY	INTER-				STORY	STORY	INTER-	STORY	STORY	INTER-				STORY	STORY	INTER-	STORY	STORY	INTER-				
LEVEL	ELEVATION	STORY	WEIGHT	FORCE	STORY	MOMENT	JK		LEVEL	ELEVATION	STORY	WEIGHT	FORCE	STORY	MOMENT	JK		LEVEL	ELEVATION	STORY	WEIGHT	FORCE	STORY	MOMENT	JK		
20	262.00		1881	165.0		0	1.00		20	262.00		1881	120.7		0	1.00		20	262.00		1881	120.7		0	1.00		
		18.50			165.						18.50			121						18.50			121				
19	243.50		3970	107.5		2966	.97		19	243.50		3970	122.0		2183	.98				19	243.50		3970	122.0		2183	.98
		12.50			273.						12.50			243.							12.50			243.			
18	231.00		1850	47.5		6167	.95		18	231.00		1850	53.9		5079	.96				18	231.00		1850	53.9		5079	.96
		12.50			320						12.50			297							12.50			297			
17	218.50		1850	45.0		9828	.94		17	218.50		1850	51.0		8547	.95				17	218.50		1850	51.0		8547	.95
		12.50			365						12.50			348							12.50			348			
16	206.00		1850	42.4		13912	.93		16	206.00		1850	48.1		12542	.94				16	206.00		1850	48.1		12542	.94
		12.50			407						12.50			396							12.50			396			
15	193.50		1850	39.8		18387	.91		15	193.50		1850	45.2		17026	.93				15	193.50		1850	45.2		17026	.93
		12.50			447						12.50			441							12.50			441			
14	181.00		1850	37.2		23226	.90		14	181.00		1850	42.3		21965	.92				14	181.00		1850	42.3		21965	.92
		12.50			486						12.50			483							12.50			483			
13	168.50		1850	34.7		28406	.89		13	168.50		1850	39.3		27328	.92				13	168.50		1850	39.3		27328	.92
		12.50			519						12.50			523							12.50			523			
12	156.00		1850	32.1		33908	.89		12	156.00		1850	36.4		33086	.91				12	156.00		1850	36.4		33086	.91
		12.50			351						12.50			359							12.50			359			
11	143.50		1850	29.5		39711	.88		11	143.50		1850	33.5		39215	.90				11	143.50		1850	33.5		39215	.90
		12.50			381						12.50			392							12.50			392			
10	131.00		1850	27.0		45801	.87		10	131.00		1850	30.6		45688	.90				10	131.00		1850	30.6		45688	.90
		12.50			408						12.50			423							12.50			423			
9	118.50		1850	24.4		52160	.87		9	118.50		1850	27.7		52484	.90				9	118.50		1850	27.7		52484	.90
		12.50			432						12.50			451							12.50			451			
8	106.00		1850	21.8		58772	.87		8	106.00		1850	24.8		59577	.89				8	106.00		1850	24.8		59577	.89
		12.50			454						12.50			476							12.50			476			
7	93.50		1850	19.2		65620	.86		7	93.50		1850	21.8		66945	.89				7	93.50		1850	21.8		66945	.89
		12.50			473						12.50			497							12.50			497			
6	81.00		1850	16.7		72604	.86		6	81.00		1850	18.9		74562	.89				6	81.00		1850	18.9		74562	.89
		12.50			490						12.50			716							12.50			716			
5	68.50		1850	14.1		79946	.86		5	68.50		1850	16.0		82401	.89				5	68.50		1850	16.0		82401	.89
		12.50			704						12.50			732							12.50			732			
4	56.00		1850	11.5		87382	.86		4	56.00		1850	13.1		90436	.89				4	56.00		1850	13.1		90436	.89
		12.50			715						12.50			745							12.50			745			
3	43.50		1850	9.0		94967	.86		3	43.50		1850	10.2		98636	.89				3	43.50		1850	10.2		98636	.89
		12.50			724						12.50			756							12.50			756			
2	31.00		1850	6.4		102674	.86		2	31.00		1850	7.2		106968	.89				2	31.00		1850	7.2		106968	.89
		12.50			731						12.50			763							12.50			763			
1	18.50		1929	4.0		110471	.86		1	18.50		1929	4.5		115398	.89				1	18.50		1929	4.5		115398	.89
		18.50			735						18.50			767							18.50			767			
0	0.00		0	0.0		122100	.86		0	0.00		0	0.0		127969	.89				0	0.00		0	0.0		127969	.89

Braced Frame Design

- the braced frame was analysed for lateral loads assuming pinned connections at all joints
- P- Δ effects were included in the analysis using the maximum allowable drift due to wind $(1/30) = L/318$
- the load combination factor for wind and live load of .7 was used yielding a net live load factor = $1.5 \times .7 = 1.05$
- a summary of the final designs is presented on pages 122 to 130
- as the stiffnesses of the frames cannot be determined until the frames are designed the distribution of lateral forces is assumed and checked later

Braced Frame Designations

• P-Δ LOADS

• ASSUME : 1-18 → LLRF = 0.5
19 → LLRF = 0.7

$$\begin{aligned}
 1-18 : \sum P_o &= 1.25 \times 0.085 \times 15150 = 1610 \\
 &+ 1.25 \times .135 \times 2100 = 354 \\
 &+ 1.25 \times .525 \times 530 = \underline{348} \\
 &2312^k
 \end{aligned}$$

$$\begin{aligned}
 \sum P_L &= .5 \times 1.05 \times .070 \times 15150 = 557 \\
 &+ .5 \times 1.05 \times .100 \times 2100 = \underline{110} \\
 &667^k
 \end{aligned}$$

$$\sum P_{o+L} = 2312 + 667 = 2979$$

$$\begin{aligned}
 19 : \sum P_o &= 1.25 \times .205 \times 17250 = 4420 \\
 &+ 1.25 \times 1.110 \times 530 = \underline{735} \\
 &5155^k
 \end{aligned}$$

$$\sum P_L = .7 \times 1.05 \times 0.075 \times 17250 = 951^k$$

$$\sum P_{o+L} = 5155 + 951 = 6106^k$$

$$20 : \sum P_o = 1.25 \times .080 \times 17250 = 1725^k$$

$$\sum P_L = 1.05 \times .048 \times 17250 = 869^k$$

$$\sum P_{o+L} = 1725 + 869 = 2594^k$$

ARTIFICIAL P-Δ LATERAL LOADS :

$$H = \frac{\sum P (\Delta_{i+1} - \Delta_i)}{1} = \frac{\sum P \frac{1}{318}}{1} = \frac{\sum P}{318}$$

LEVEL	$H_D = \frac{\sum P_D}{318} (k)$	$H_{DL} = \frac{\sum P_{DL}}{318} (k)$
1-18	7.3	9.4
19	16.2	19.2
20	5.4	8.2

• AXIAL GRAVITY FORCES IN K-BRACING

$$D = \sqrt{S^2 + L^2}$$

$$KC = \frac{.625 \times W \times L \times D}{2 \times S}$$

BRACED FRAME 1:

LEVEL	L (FT)	S (FT)	D (FT)	W_D (K/FT)	W_L (K/FT)	KC_D (K)	KC_L (K)
0-1	17.5	18.5	25.5	1.125	0.525	8.5	4.0
1-2 TO 17-18	17.5	12.5	21.5	1.125	0.525	10.6	4.9
18-19	17.5	12.5	21.5	2.610	0.563	24.6	5.3
19-20	17.5	18.5	25.5	0.600	0.360	4.5	4.0

BRACED FRAME 2 :

LEVEL	L (FT)	S (FT)	D (FT)	W_D (K/FT)	W_L (K/FT)	KC_D (K)	KC_L (K)
0-1	17.5	18.5	25.5	1.575	1.084	11.9	8.2
1-2 TO 17-18	17.5	12.5	21.5	1.575	1.084	14.8	10.2
18-19	17.5	12.5	21.5	3.000	0.956	28.2	9.0
19-20	17.5	18.5	25.5	1.200	0.720	9.0	5.4

BRACED FRAME 3 :

$$D = \sqrt{S^2 + L^2}$$

$$KC = \frac{P \Delta}{2 \times S}$$

LEVEL	L (FT)	S (FT)	D (FT)	P_D (K)	P_L (K)	KC_D (K)	KC_L (K)
0-1	15	18.5	23.8	58.1	18.9	37.4	12.2
1-2 TO 17-18	15	12.5	19.5	58.1	18.9	45.4	14.8
18-19	15	12.5	19.5	112.5	30.4	87.9	23.7
19-20	15	18.5	23.8	45.0	27.0	29.0	17.4

• BENDING MOMENTS IN BEAMS

$$M_B = -\frac{WL^2}{8}$$

BRACED FRAME 1 :

LEVEL	L (FT)	W_D (K/FT)	W_L (K/FT)	M_D (IK)	M_L (IK)
1-18	17.5	1.125	0.525	43.1	20.1
19	17.5	2.610	0.563	99.9	21.6
20	17.5	0.600	0.360	23.0	20.1

BRACED FRAME 2:

LEVEL	L (FT)	W_D (K/FT)	W_L (K/FT)	M_D (IK)	M_L (IK)
1-18	17.5	1.575	1.173	60.3	44.9
19	17.5	3.000	1.035	114.8	39.6
20	17.5	1.200	0.720	45.9	27.6

BRACED FRAME 3:

- DUE TO THE ONE-WAY ACTION OF THE SLAB
THE BENDING MOMENT WILL BE CONSIDERED
TO BE ZERO

$$1-20 : M_D = 0 \quad \& \quad M_L = 0$$

CONSTANTS: BF-1

2.1 NO. OF BAYS 2
 5.1 BAY WIDTH = 17.50 FT.

DISTRIBUTION FACTOR FOR WIND LOADS = .24

DISTRIBUTION FACTOR FOR DEAD LOADS = .29

UNITS: FEET AND KIPS

LOADS BF-1

LEVEL	STORY HEIGHT	JE	EL	WIND LOADS	P-0.01 LOADS	P-0.04 LOADS
20	18.50	97	145.0	47.7	5.4	8.2
19	12.50	95	107.5	79.8	16.2	19.2
18	12.50	94	47.5	44.4	7.3	9.4
17	12.50	93	45.0	44.4	7.3	9.4
16	12.50	91	42.0	44.4	7.3	9.4
15	12.50	90	39.8	44.4	7.3	9.4
14	12.50	89	37.2	44.4	7.3	9.4
13	12.50	89	34.7	44.4	7.3	9.4
12	12.50	88	32.1	44.4	7.3	9.4
11	12.50	87	29.5	44.4	7.3	9.4
10	12.50	87	27.0	44.4	7.3	9.4
9	12.50	87	24.4	44.4	7.3	9.4
8	12.50	86	21.8	44.4	7.3	9.4
7	12.50	86	19.2	44.4	7.3	9.4
6	12.50	86	16.7	44.4	7.3	9.4
5	12.50	86	14.1	44.4	7.3	9.4
4	12.50	86	11.5	44.4	7.3	9.4
3	12.50	86	9.0	44.4	7.3	9.4
2	12.50	86	6.4	44.4	7.3	9.4
1	18.50	86	6.0	79.8	7.3	9.4
0						

LOADS BF-1

LEVEL	COLUMN DL	COLUMN LL	K-RACE DL	K-RACE LL	BEAM DL	BEAM LL
20	48	41	4.5	2.7	23.0	13.8
19	282	82	24.4	5.3	99.9	21.6
18	373	108	10.6	4.9	43.1	20.1
17	444	131	10.6	4.9	43.1	20.1
16	535	155	10.6	4.9	43.1	20.1
15	647	177	10.6	4.9	43.1	20.1
14	738	199	10.6	4.9	43.1	20.1
13	829	221	10.6	4.9	43.1	20.1
12	920	242	10.6	4.9	43.1	20.1
11	1011	263	10.6	4.9	43.1	20.1
10	1102	284	10.6	4.9	43.1	20.1
9	1193	305	10.6	4.9	43.1	20.1
8	1285	326	10.6	4.9	43.1	20.1
7	1376	346	10.6	4.9	43.1	20.1
6	1467	367	10.6	4.9	43.1	20.1
5	1558	387	10.6	4.9	43.1	20.1
4	1649	408	10.6	4.9	43.1	20.1
3	1740	428	10.6	4.9	43.1	20.1
2	1831	448	10.6	4.9	43.1	20.1
1	1923	468	8.5	4.0	43.1	20.1
0						

[illegible]

CONSTANTS: BF-2

2.1 NO. OF BAYS = 2

3.1 BAY WIDTH = 17.50 FT.

REDISTRIBUTION FACTOR FOR WIND LOADS = .28

REDISTRIBUTION FACTOR FOR EEL LOADS = .29

UNITS: FEET AND KIPS

LOADS BF-2

LOADS BF-2

LEVEL	STORY HEIGHT	JL	EEL LOADS	WIND LOADS	P-9(D) LOADS	P-9(D+J) LOADS	LEVEL	COLUMN DL	COLUMN LL	K-BRACE DL	K-BRACE LL	BEAM DL	BEAM LL
20	18.50	97	145.0	47.7	5.4	8.2	20	112	67	9.0	5.4	45.9	27.4
19	12.50	95	107.5	79.8	16.2	15.2	19	401	127	28.2	9.0	114.8	39.4
18	12.50	94	47.5	44.4	7.3	9.4	18	533	171	14.8	10.2	60.3	44.9
17	12.50	93	45.0	44.4	7.3	9.4	17	645	212	14.8	10.2	60.3	44.9
16	12.50	91	42.0	44.4	7.3	9.4	16	797	252	14.8	10.2	60.3	44.9
15	12.50	90	39.8	44.4	7.3	9.4	15	929	290	14.8	10.2	60.3	44.9
14	12.50	89	37.2	44.4	7.3	9.4	14	1062	329	14.8	10.2	60.3	44.9
13	12.50	89	34.7	44.4	7.3	9.4	13	1194	366	14.8	10.2	60.3	44.9
12	12.50	88	32.1	44.4	7.3	9.4	12	1326	404	14.8	10.2	60.3	44.9
11	12.50	87	29.5	44.4	7.3	9.4	11	1458	440	14.8	10.2	60.3	44.9
10	12.50	87	27.0	44.4	7.3	9.4	10	1590	477	14.8	10.2	60.3	44.9
9	12.50	87	24.4	44.4	7.3	9.4	9	1722	514	14.8	10.2	60.3	44.9
8	12.50	86	21.8	44.4	7.3	9.4	8	1855	550	14.8	10.2	60.3	44.9
7	12.50	86	19.2	44.4	7.3	9.4	7	1987	586	14.8	10.2	60.3	44.9
6	12.50	86	16.7	44.4	7.3	9.4	6	2119	622	14.8	10.2	60.3	44.9
5	12.50	86	14.1	44.4	7.3	9.4	5	2251	658	14.8	10.2	60.3	44.9
4	12.50	86	11.5	44.4	7.3	9.4	4	2383	693	14.8	10.2	60.3	44.9
3	12.50	86	9.0	44.4	7.3	9.4	3	2515	729	14.8	10.2	60.3	44.9
2	12.50	86	6.4	44.4	7.3	9.4	2	2647	764	14.8	10.2	60.3	44.9
1	18.50	86	4.8	79.8	7.3	9.4	1	2780	800	11.9	8.2	60.3	44.9
0							0						

COLUMN FORCES: BF-2										COLUMN FORCES: BF-2										K-RANGE FORCES: BF-2									
LEVEL	WIND	EQ	ATIAL	P-0(0)	P-0(0)	ATIAL	EQ	ATIAL	WIND	LEVEL	WIND	EQ	ATIAL	P-0(0)	P-0(0)	ATIAL	EQ	ATIAL	WIND	LEVEL	WIND	EQ	ATIAL	P-0(0)	P-0(0)	ATIAL	EQ	ATIAL	WIND
20	0	0	0	0	0	0	0	0	0	20	0	0	0	0	0	0	0	0	0	20	0	0	0	0	0	0	0	0	0
19	7	25	0	0	0	0	0	0	0	19	7	25	0	0	0	0	0	0	0	19	7	25	0	0	0	0	0	0	0
18	19	59	2	2	2	2	2	2	2	18	19	59	2	2	2	2	2	2	2	18	19	59	2	2	2	2	2	2	2
17	28	86	5	5	5	5	5	5	5	17	28	86	5	5	5	5	5	5	5	17	28	86	5	5	5	5	5	5	5
16	44	124	9	12	12	12	12	12	12	16	44	124	9	12	12	12	12	12	12	16	44	124	9	12	12	12	12	12	12
15	64	166	13	17	17	17	17	17	17	15	64	166	13	17	17	17	17	17	17	15	64	166	13	17	17	17	17	17	17
14	86	232	18	24	24	24	24	24	24	14	86	232	18	24	24	24	24	24	24	14	86	232	18	24	24	24	24	24	24
13	108	263	24	31	31	31	31	31	31	13	108	263	24	31	31	31	31	31	31	13	108	263	24	31	31	31	31	31	31
12	128	281	31	40	40	40	40	40	40	12	128	281	31	40	40	40	40	40	40	12	128	281	31	40	40	40	40	40	40
11	146	297	38	49	49	49	49	49	49	11	146	297	38	49	49	49	49	49	49	11	146	297	38	49	49	49	49	49	49
10	162	312	46	59	59	59	59	59	59	10	162	312	46	59	59	59	59	59	59	10	162	312	46	59	59	59	59	59	59
9	176	326	54	69	69	69	69	69	69	9	176	326	54	69	69	69	69	69	69	9	176	326	54	69	69	69	69	69	69
8	188	339	63	79	79	79	79	79	79	8	188	339	63	79	79	79	79	79	79	8	188	339	63	79	79	79	79	79	79
7	198	351	74	89	89	89	89	89	89	7	198	351	74	89	89	89	89	89	89	7	198	351	74	89	89	89	89	89	89
6	207	362	84	99	99	99	99	99	99	6	207	362	84	99	99	99	99	99	99	6	207	362	84	99	99	99	99	99	99
5	215	372	95	109	109	109	109	109	109	5	215	372	95	109	109	109	109	109	109	5	215	372	95	109	109	109	109	109	109
4	222	381	106	119	119	119	119	119	119	4	222	381	106	119	119	119	119	119	119	4	222	381	106	119	119	119	119	119	119
3	228	389	117	129	129	129	129	129	129	3	228	389	117	129	129	129	129	129	129	3	228	389	117	129	129	129	129	129	129
2	233	396	128	139	139	139	139	139	139	2	233	396	128	139	139	139	139	139	139	2	233	396	128	139	139	139	139	139	139
1	237	402	139	149	149	149	149	149	149	1	237	402	139	149	149	149	149	149	149	1	237	402	139	149	149	149	149	149	149
0	240	407	149	159	159	159	159	159	159	0	240	407	149	159	159	159	159	159	159	0	240	407	149	159	159	159	159	159	159

WIND (100' WIDE)
10-10-10
(100' WIDE)

COLUMN FORCES: IF-3										COLUMN FORCES: IF-3										K-RACE FORCES: IF-3														
LEVEL	ACTUAL WIND	ACTUAL EQ	ACTUAL P-DID	ACTUAL P-DID(L)	ACTUAL LL	FACT. WIND	FACT. EQ	FACT. P-DID	FACT. P-DID(L)	FACT. LL	LEVEL	ACTUAL WIND	ACTUAL EQ	ACTUAL P-DID	ACTUAL P-DID(L)	ACTUAL LL	FACT. WIND	FACT. EQ	FACT. P-DID	FACT. P-DID(L)	FACT. LL	LEVEL	ACTUAL WIND	ACTUAL EQ	ACTUAL P-DID	ACTUAL P-DID(L)	ACTUAL LL	FACT. WIND	FACT. EQ	FACT. P-DID	FACT. P-DID(L)	FACT. LL		
20	0	0	0	0	0	261	140	140	140	210	19	7	24	1	1	29	20	7	24	1	1	29	19	16	41	3	4	92	25	16	41	3	4	92
19	5	19	0	1	2401	692	311	329	0	654	18	23	50	4	5	67	18	23	50	4	5	67	17	31	58	5	7	47	15	31	58	5	7	47
18	25	48	3	4	523	923	692	731	847	891	17	31	58	5	7	47	16	39	44	7	9	47	16	39	44	7	9	47	15	39	44	7	9	47
17	31	77	6	7	445	1149	884	941	1095	1131	16	31	58	5	7	47	15	47	74	8	10	47	15	47	74	8	10	47	15	47	74	8	10	47
16	31	77	6	7	445	1149	884	941	1095	1131	15	47	74	8	10	47	14	55	81	9	12	47	14	55	81	9	12	47	15	55	81	9	12	47
15	31	77	6	7	445	1149	884	941	1095	1131	14	62	88	10	13	47	13	62	88	10	13	47	13	62	88	10	13	47	15	62	88	10	13	47
14	77	158	14	16	1428	1291	1360	1612	1811	1872	13	69	94	11	15	47	12	69	94	11	15	47	12	69	94	11	15	47	15	69	94	11	15	47
13	147	246	19	25	1642	2042	1732	1845	2040	2124	12	77	100	13	16	47	11	77	100	13	16	47	11	77	100	13	16	47	15	77	100	13	16	47
12	142	238	25	33	1194	2042	1732	1845	2040	2124	11	84	105	14	16	47	10	84	105	14	16	47	10	84	105	14	16	47	15	84	105	14	16	47
11	182	315	32	41	1326	2264	1944	2062	2315	2379	10	91	110	15	19	47	9	91	110	15	19	47	9	91	110	15	19	47	15	91	110	15	19	47
10	227	375	46	51	1458	2482	2204	2320	2575	2639	9	96	114	16	21	47	8	96	114	16	21	47	8	96	114	16	21	47	15	96	114	16	21	47
9	276	439	46	62	1590	2763	2451	2581	2841	2904	8	105	117	17	22	47	7	105	117	17	22	47	7	105	117	17	22	47	15	105	117	17	22	47
8	301	507	57	73	1732	3144	2767	2820	3114	3166	7	112	121	18	24	47	6	112	121	18	24	47	6	112	121	18	24	47	15	112	121	18	24	47
7	369	577	67	86	1855	3553	3241	3353	3675	3707	6	118	123	20	25	47	5	118	123	20	25	47	5	118	123	20	25	47	15	118	123	20	25	47
6	432	660	76	100	1987	3982	3519	3619	3943	3981	5	125	126	21	27	47	4	125	126	21	27	47	4	125	126	21	27	47	15	125	126	21	27	47
5	509	736	89	114	2119	4268	3801	3804	4257	4256	4	131	127	22	28	47	3	131	127	22	28	47	3	131	127	22	28	47	15	131	127	22	28	47
4	592	844	101	136	2251	4603	4018	4056	4555	4555	3	136	129	23	30	47	2	136	129	23	30	47	2	136	129	23	30	47	15	136	129	23	30	47
3	648	883	114	146	2383	4931	4394	4431	4859	4810	2	175	156	30	39	28	1	175	156	30	39	28	1	175	156	30	39	28	15	175	156	30	39	28
2	748	944	128	164	2515	5287	4653	4700	5148	5088	1						0						0											
1	882	1065	142	182	2647	5673	4673	4962	5463	5370	0																							
0	919	1128	157	202	2780	6000																												

SP-2 STAGS GOVERN

* IF-2 66665 COVERED

Distribution of Lateral Forces to the Braced Frames

- the loads will be distributed to the braced frames according to their stiffnesses
- for this analysis the fraction of the total load carried by each braced frame will be assumed and checked later against the actual value calculated for the final design
- an allowance will also be made for torsional effects under earthquake loading
- assumed distribution factors:

Braced Frame	Number of Frames	Direction of Forces	Wind Dist. Factor	E.Q. Dist. Factor
BF-1	2	N-S	.24	.29
BF-2	2	N-S	.28	.29
BF-3	2	E-W	.50	.52

Lateral Drift

- a DPW version of "SP Stress" was used to determine the deflections of the braced frames due to wind loads
- the results are presented on pages 134 and 135
- the joint and member designations are outlined on page 132
- Braced Frame 3 was modelled as shown because of core memory limitations of the mini-computer (see page 133)
- the relative stiffness of each frame was determined using "SP Stress" by placing a point load of 50 kips at each level and calculating the deflection of the top level

Model of BF-3 Used For Analysis

*** SYSTEMS PROFESSIONAL STRESS

STRUCTURE-EXAMPLE BLDG (S=12.5 & DL=80) M-S

TYPE PLANE TRUSS

NUMBER OF JOINTS 62

NUMBER OF MEMBERS 120

NUMBER OF LOADINGS 1

NUMBER OF SUPPORTS 2

UNITS KIPS FEET

TABULATE DISPLACEMENTS

JOINT COORDINATES

1 0 0 5 2 35 0 0 5 3 0 18.5
 4 17 5 18 5 5 35 0 18 5 5 6 0 31 0
 7 17 5 31 0 8 35 0 31 0 8 9 0 43 5
 10 17 5 43 5 11 35 0 43 5 12 0 54 0
 13 17 5 56 0 14 35 0 56 0 15 0 68 5
 16 17 5 68 5 17 35 0 68 5 18 0 81 0
 19 17 5 81 0 20 35 0 81 0 21 0 93 5
 22 17 5 93 5 23 35 0 93 5 24 0 106 0
 25 17 5 106 0 26 35 0 106 0 27 0 118 5
 28 17 5 118 5 29 35 0 118 5 30 0 131 0
 31 17 5 131 0 32 35 0 131 0 33 0 143 5
 34 17 5 143 5 35 35 0 143 5 36 0 156 0
 37 17 5 156 0 38 35 0 156 0 39 0 168 5
 40 17 5 168 5 41 35 0 168 5 42 0 181 0
 43 17 5 181 0 44 35 0 181 0 45 0 193 5
 46 17 5 193 5 47 35 0 193 5 48 0 206 0
 49 17 5 206 0 50 35 0 206 0 51 0 218 5
 52 17 5 218 5 53 35 0 218 5 54 0 231 0
 55 17 5 231 0 56 35 0 231 0 57 0 243 5
 58 17 5 243 5 59 35 0 243 5 60 0 262 0
 61 17 5 262 0 62 35 0 262 0

UNITS INCHES

*COLUMNS

MEMBER PROP PRIS AX 592 0 81 2 8 61 662
 MEMBER PROP PRIS AX 508 0 8 3 4 63 64
 MEMBER PROP PRIS AX 426 4 8 5 6 65 64
 MEMBER PROP PRIS AX 372 6 7 8 6 67 68
 MEMBER PROP PRIS AX 316 2 8 9 6 10 69 70
 MEMBER PROP PRIS AX 259 8 11 8 12 71 72
 MEMBER PROP PRIS AX 209 0 13 8 14 73 74
 MEMBER PROP PRIS AX 158 4 15 8 16 75 76
 MEMBER PROP PRIS AX 120 0 17 8 18 77 78
 MEMBER PROP PRIS AX 74 2 19 8 20 79 80

*K-BRACING

MEMBER PROP PRIS AX 92 8 41 6 101
 MEMBER PROP PRIS AX 73 6 42 6 102
 MEMBER PROP PRIS AX 73 6 43 6 103
 MEMBER PROP PRIS AX 70 0 44 6 104
 MEMBER PROP PRIS AX 64 2 45 6 105
 MEMBER PROP PRIS AX 60 6 46 6 106
 MEMBER PROP PRIS AX 60 6 47 6 107
 MEMBER PROP PRIS AX 57 6 48 6 108
 MEMBER PROP PRIS AX 57 6 49 6 109
 MEMBER PROP PRIS AX 57 6 50 6 110
 MEMBER PROP PRIS AX 55 2 51 6 111
 MEMBER PROP PRIS AX 55 2 52 6 112
 MEMBER PROP PRIS AX 50 0 53 6 113
 MEMBER PROP PRIS AX 47 2 54 6 114
 MEMBER PROP PRIS AX 44 2 55 6 115
 MEMBER PROP PRIS AX 41 2 56 6 116
 MEMBER PROP PRIS AX 36 5 57 6 117
 MEMBER PROP PRIS AX 36 5 58 6 118
 MEMBER PROP PRIS AX 36 5 59 6 119
 MEMBER PROP PRIS AX 36 5 60 6 120

*BEAMS

MEMBER PROP PRIS AX 47 6 21 8 81
 MEMBER PROP PRIS AX 47 6 22 8 82
 MEMBER PROP PRIS AX 43 6 23 8 83
 MEMBER PROP PRIS AX 43 6 24 8 84
 MEMBER PROP PRIS AX 43 6 25 8 85
 MEMBER PROP PRIS AX 41 3 26 8 86

MEMBER PROP PRIS AX 36 9 27 8 87
 MEMBER PROP PRIS AX 36 9 28 8 88
 MEMBER PROP PRIS AX 36 9 29 8 89
 MEMBER PROP PRIS AX 36 5 30 8 90
 MEMBER PROP PRIS AX 36 5 31 8 91
 MEMBER PROP PRIS AX 36 5 32 8 92
 MEMBER PROP PRIS AX 32 6 33 8 93
 MEMBER PROP PRIS AX 32 6 34 8 94
 MEMBER PROP PRIS AX 32 6 35 8 95
 MEMBER PROP PRIS AX 31 2 36 8 96
 MEMBER PROP PRIS AX 31 2 37 8 97
 MEMBER PROP PRIS AX 31 2 38 8 98
 MEMBER PROP PRIS AX 29 5 39 8 99
 MEMBER PROP PRIS AX 26 0 40 8 100

MEMBER INCIDENCES

1 1 3 6 2 3 6 6 9 6 9 12
 5 12 15 6 15 18 7 18 21 6 21 24
 9 24 27 6 10 27 30 8 11 30 33 6 12 33 36
 13 36 39 6 14 39 42 8 15 42 45 6 16 45 48
 17 48 51 8 18 51 54 8 19 54 57 8 20 57 60
 21 3 4 6 22 6 7 23 9 10 24 12 13
 25 15 16 6 26 18 19 27 21 22 6 28 24 25
 29 27 28 6 30 31 8 31 33 34 6 32 36 37
 33 39 40 6 34 42 43 6 35 45 46 6 36 48 49
 37 51 52 6 38 54 35 6 39 57 58 6 40 60 61
 41 1 4 6 42 3 7 43 6 10 44 9 13
 45 12 16 6 46 15 19 47 18 22 6 48 21 23
 49 24 28 6 50 27 31 8 51 30 34 6 52 33 37
 53 36 40 6 54 39 43 6 55 42 46 6 56 45 49
 57 48 52 6 58 51 35 6 59 54 58 6 60 57 61
 61 2 5 6 62 5 8 63 8 11 64 11 14
 65 14 17 6 66 17 20 6 67 20 23 6 68 23 26
 69 24 29 6 70 27 32 6 71 32 35 6 72 35 38
 73 38 41 6 74 41 44 6 75 44 47 6 76 47 50
 77 50 53 6 78 53 56 6 79 56 59 6 80 59 62
 81 4 5 6 82 7 8 83 10 11 84 13 14
 85 16 17 6 86 19 20 6 87 22 23 6 88 25 26
 89 28 29 6 90 31 32 6 91 34 35 6 92 37 38
 93 40 41 6 94 43 44 6 95 46 47 6 96 49 50
 97 52 53 6 98 55 56 6 99 58 59 6 100 61 62
 101 2 4 6 102 5 7 6 103 8 10 6 104 11 13
 105 14 16 6 106 17 19 6 107 20 22 6 108 23 25
 109 26 28 6 110 29 31 6 111 32 34 6 112 35 37
 113 38 40 6 114 41 43 6 115 44 46 6 116 47 49
 117 50 52 6 118 53 55 6 119 56 58 6 120 59 61

CONSTANTS E 29000. ALL

UNITS FEET

LOADING NO 1 WIND + P-0

JOINT LOADS

3 FORCE 1 26 8 8 5
 4 FORCE 1 42 8 8 8
 9 FORCE 1 35 1 6 11
 12 FORCE 1 35 1 6 14
 15 FORCE 1 35 1 6 17
 18 FORCE 1 35 1 6 20
 21 FORCE 1 35 1 6 23
 24 FORCE 1 35 1 6 26
 27 FORCE 1 35 1 6 29
 30 FORCE 1 35 1 6 32
 33 FORCE 1 35 1 6 35
 36 FORCE 1 35 1 6 38
 39 FORCE 1 35 1 6 41
 42 FORCE 1 35 1 6 44
 45 FORCE 1 35 1 6 47
 48 FORCE 1 35 1 6 50
 51 FORCE 1 35 1 6 53
 54 FORCE 1 35 1 6 56
 57 FORCE 1 38 7 6 59
 60 FORCE 1 42 1 6 62

UNITS KIPS INCHES

SOLVE

PROBLEM CORRECTLY SPECIFIED. EXECUTION TO PROCEED.
 62 7.0118 - 4885

STRUCTURE-EXAMPLE BLDG (S=12.5 & DL=80) M-S

(UNITS IN INCHES AND KIPS)

LOADING NO 1 WIND + P-0

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT
3	2802	0637
4	1719	0 0000
5	2802	- 0637
6	5550	1024
7	4508	- 0000
8	5550	- 1024
9	8521	1429
10	7455	- 0000
11	8521	- 1429
12	1 1716	1790
13	1 0708	- 0000
14	1 1716	- 1790
15	1 5142	- 2170
16	1 4192	- 0000
17	1 5142	- 2170
18	1 0825	2503
19	1 7883	- 0000
20	1 0825	- 2503
21	2 2640	2835
22	2 1726	0000
23	2 2640	- 2835
24	2 6643	3121
25	2 5774	- 0000
26	2 6643	- 3121
27	3 0684	3407
28	2 9880	- 0000
29	3 0684	- 3407
30	3 4830	3847
31	3 4043	0000
32	3 4830	- 3847
33	3 9032	3887
34	3 8315	- 0000
35	3 9032	- 3887
36	4 3251	4082
37	4 2604	- 0000
38	4 3251	- 4082
39	4 7577	4271
40	4 6730	- 0000
41	4 7577	- 4271
42	5 1910	4416
43	5 1288	- 0000
44	5 1910	- 4416
45	5 6213	4555
46	5 5737	- 0000
47	5 6213	- 4555
48	6 0366	4650
49	6 0074	- 0000
50	6 0366	- 4650
51	6 4734	4730
52	6 4383	- 0000
53	6 4734	- 4730
54	6 8808	4774
55	6 8539	- 0000
56	6 8808	- 4774
57	7 2644	4885
58	7 2516	- 0000
59	7 2644	- 4885
60	7 6118	4885
61	7 6081	- 0000
62	7 6118	- 4885

*** SYSTEMS PROFESSIONAL STRESS

STRUCTURE-EXAMPLE BLDG (S=12.5 & DL=80) E-W

TYPE PLANE TRUSS

NUMBER OF JOINTS 62

NUMBER OF MEMBERS 120

NUMBER OF LOADINGS 1

NUMBER OF SUPPORTS 2

UNITS KIPS FEET

TABULATE DISPLACEMENTS

JOINT COORDINATES

1 0 0 5 0 2 30 0 5 0 30 18.5
 4 15 18 5 0 5 30 18 5 0 0 31 0
 7 15 31 0 0 0 30 31 0 0 0 43.5
 10 15 43 5 0 11 30 43 5 0 12 0 56.0
 13 15 56 0 0 14 30 56 0 0 15 0 68.5
 16 15 68 5 0 17 30 68 5 0 18 0 81.0
 19 15 81 0 0 20 30 81 0 0 21 0 93.5
 22 15 93 5 0 23 30 93 5 0 24 0 106.0
 25 15 106 0 0 26 30 106 0 0 27 0 118.5
 28 15 118 5 0 29 30 118 5 0 30 0 131.0
 31 15 131 0 0 32 30 131 0 0 33 0 143.5
 34 15 143 5 0 35 30 143 5 0 36 0 156.0
 37 15 156 0 0 38 30 156 0 0 39 0 168.5
 40 15 168 5 0 41 30 168 5 0 42 0 181.0
 43 15 181 0 0 44 30 181 0 0 45 0 193.5
 46 15 193 5 0 47 30 193 5 0 48 0 206.0
 49 15 206 0 0 50 30 206 0 0 51 0 218.5
 52 15 218 5 0 53 30 218 5 0 54 0 231.0
 55 15 231 0 0 56 30 231 0 0 57 0 243.5
 58 15 243 5 0 59 30 243 5 0 60 0 262.0
 61 15 262 0 0 62 30 262 0

UNITS INCHES

*CALLINGS

MEMBER PROP PRIS AX 346.0 0 1 0 2
 MEMBER PROP PRIS AX 286.0 0 3 0 4
 MEMBER PROP PRIS AX 246.0 0 5 0 6
 MEMBER PROP PRIS AX 222.0 0 7 0 8
 MEMBER PROP PRIS AX 180.0 0 9 0 10
 MEMBER PROP PRIS AX 150.0 0 11 0 12
 MEMBER PROP PRIS AX 121.0 0 13 0 14
 MEMBER PROP PRIS AX 88.0 0 15 0 16
 MEMBER PROP PRIS AX 70.0 0 17 0 18
 MEMBER PROP PRIS AX 42.0 0 19 0 20
 MEMBER PROP PRIS AX 10000.0 0 61 0 62
 MEMBER PROP PRIS AX 10000.0 0 63 0 64
 MEMBER PROP PRIS AX 10000.0 0 65 0 66
 MEMBER PROP PRIS AX 10000.0 0 67 0 68
 MEMBER PROP PRIS AX 10000.0 0 69 0 70
 MEMBER PROP PRIS AX 10000.0 0 71 0 72
 MEMBER PROP PRIS AX 10000.0 0 73 0 74
 MEMBER PROP PRIS AX 10000.0 0 75 0 76
 MEMBER PROP PRIS AX 10000.0 0 77 0 78
 MEMBER PROP PRIS AX 10000.0 0 79 0 80

*K-BRACING

MEMBER PROP PRIS AX 28.2 0 41 0 101
 MEMBER PROP PRIS AX 28.0 0 42 0 102
 MEMBER PROP PRIS AX 28.0 0 43 0 103
 MEMBER PROP PRIS AX 28.0 0 44 0 104
 MEMBER PROP PRIS AX 28.0 0 45 0 105
 MEMBER PROP PRIS AX 28.0 0 46 0 106
 MEMBER PROP PRIS AX 28.0 0 47 0 107
 MEMBER PROP PRIS AX 28.0 0 48 0 108
 MEMBER PROP PRIS AX 28.0 0 49 0 109
 MEMBER PROP PRIS AX 28.0 0 50 0 110
 MEMBER PROP PRIS AX 28.0 0 51 0 111
 MEMBER PROP PRIS AX 28.0 0 52 0 112
 MEMBER PROP PRIS AX 26.0 0 53 0 113
 MEMBER PROP PRIS AX 26.0 0 54 0 114
 MEMBER PROP PRIS AX 22.0 0 55 0 115
 MEMBER PROP PRIS AX 22.0 0 56 0 116
 MEMBER PROP PRIS AX 20.0 0 57 0 117
 MEMBER PROP PRIS AX 18.0 0 58 0 118
 MEMBER PROP PRIS AX 26.0 0 59 0 119
 MEMBER PROP PRIS AX 18.0 0 60 0 120

*MEMBS

MEMBER PROP PRIS AX 15.3 0 21 0 81

MEMBER PROP PRIS AX 15.3 0 22 0 82
 MEMBER PROP PRIS AX 15.3 0 23 0 83
 MEMBER PROP PRIS AX 15.3 0 24 0 84
 MEMBER PROP PRIS AX 15.3 0 25 0 85
 MEMBER PROP PRIS AX 15.3 0 26 0 86
 MEMBER PROP PRIS AX 15.3 0 27 0 87
 MEMBER PROP PRIS AX 15.3 0 28 0 88
 MEMBER PROP PRIS AX 15.3 0 29 0 89
 MEMBER PROP PRIS AX 15.3 0 30 0 90
 MEMBER PROP PRIS AX 15.3 0 31 0 91
 MEMBER PROP PRIS AX 13.0 0 32 0 92
 MEMBER PROP PRIS AX 13.0 0 33 0 93
 MEMBER PROP PRIS AX 13.0 0 34 0 94
 MEMBER PROP PRIS AX 13.0 0 35 0 95
 MEMBER PROP PRIS AX 13.0 0 36 0 96
 MEMBER PROP PRIS AX 13.0 0 37 0 97
 MEMBER PROP PRIS AX 13.0 0 38 0 98
 MEMBER PROP PRIS AX 13.0 0 39 0 99
 MEMBER PROP PRIS AX 13.0 0 40 0 100

MEMBER INCIDENCES

1 1 3 0 2 3 6 0 3 6 9 0 4 9 12
 5 12 15 0 6 15 18 0 7 18 21 0 8 21 24
 9 24 27 0 10 27 30 0 11 30 33 0 12 33 36
 13 36 39 0 14 39 42 0 15 42 45 0 16 45 48
 17 48 51 0 18 51 54 0 19 54 57 0 20 57 60
 21 3 4 0 22 6 7 0 23 9 10 0 24 12 13
 25 15 16 0 26 18 19 0 27 21 22 0 28 24 25
 29 27 28 0 30 30 31 0 31 33 34 0 32 36 37
 33 39 40 0 34 42 43 0 35 45 46 0 36 48 49
 37 51 52 0 38 54 55 0 39 57 58 0 40 60 61
 41 1 4 0 42 3 7 0 43 6 10 0 44 9 13
 45 12 16 0 46 15 19 0 47 18 22 0 48 21 25
 49 24 28 0 50 27 31 0 51 30 34 0 52 33 37
 53 36 40 0 54 39 43 0 55 42 46 0 56 45 49
 57 48 52 0 58 51 55 0 59 54 58 0 60 57 61
 61 2 5 0 62 5 8 0 63 8 11 0 64 11 14
 65 14 17 0 66 17 20 0 67 20 23 0 68 23 26
 69 26 29 0 70 29 32 0 71 32 35 0 72 35 38
 73 38 41 0 74 41 44 0 75 44 47 0 76 47 50
 77 50 53 0 78 53 56 0 79 56 59 0 80 59 62
 81 4 5 0 82 7 8 0 83 10 11 0 84 13 14
 85 16 17 0 86 19 20 0 87 22 23 0 88 25 26
 89 28 29 0 90 31 32 0 91 34 35 0 92 37 38
 93 40 41 0 94 43 44 0 95 46 47 0 96 49 50
 97 52 53 0 98 55 56 0 99 58 59 0 100 61 62
 101 2 4 0 102 5 7 0 103 8 10 0 104 11 13
 105 14 16 0 106 17 19 0 107 20 22 0 108 23 25
 109 26 28 0 110 29 31 0 111 32 34 0 112 35 37
 113 38 40 0 114 41 43 0 115 44 46 0 116 47 49
 117 50 52 0 118 53 55 0 119 56 58 0 120 59 61

CONSTANTS E 29000. ALL

UNITS FEET

LOADING NO 1 WIND + P-D

JOINT LOADS

3 FORCE X 12.3 0 5
 6 FORCE X 10.4 0 8
 9 FORCE X 10.9 0 11
 12 FORCE X 11.2 0 14
 15 FORCE X 11.5 0 17
 18 FORCE X 11.7 0 20
 21 FORCE X 12.0 0 23
 24 FORCE X 12.2 0 26
 27 FORCE X 12.4 0 29
 30 FORCE X 12.6 0 32
 33 FORCE X 12.7 0 35
 36 FORCE X 12.9 0 38
 39 FORCE X 13.0 0 41
 42 FORCE X 13.2 0 44
 45 FORCE X 13.3 0 47
 48 FORCE X 13.5 0 50
 51 FORCE X 13.6 0 53
 54 FORCE X 13.7 0 56
 57 FORCE X 13.8 0 59
 60 FORCE X 14.0 0 62

UNITS KIPS INCHES

0

SOLVE

PROBLEM CORRECTLY SPECIFIED. EXECUTION TO FINISH.

STRUCTURE-EXAMPLE BLDG (S=12.5 & DL=80) E-W

(UNITS IN INCHES AND KIPS)

LOADING NO 1 WIND + P-D

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT
3	2661	0463
4	1639	0 0000
5	2661	- 0016
6	4972	0746
7	4000	0223
8	4972	- 0026
9	7314	1054
10	6384	0360
11	7314	- 0035
12	9691	1329
13	8805	0510
14	9691	- 0042
15	1 2088	1612
16	1 1247	0443
17	1 2088	- 0049
18	1 4504	1861
19	1 3710	0781
20	1 4504	- 0056
21	1 6923	2101
22	1 6177	0903
23	1 6923	- 0061
24	1 9339	2308
25	1 8642	1020
26	1 9339	- 0065
27	2 1736	2524
28	2 1088	1121
29	2 1736	- 0069
30	2 4115	2705
31	2 3517	1227
32	2 4115	- 0073
33	2 6440	2804
34	2 5913	1316
35	2 6440	- 0075
36	2 8856	3027
37	2 8273	1404
38	2 8856	- 0078
39	3 1237	3166
40	3 0716	1475
41	3 1237	- 0079
42	3 3547	3272
43	3 3088	1544
44	3 3547	- 0080
45	3 5822	3375
46	3 5426	1596
47	3 5822	- 0081
48	3 7999	3445
49	3 7666	1647
50	3 7999	- 0082
51	4 0111	3499
52	3 9862	1662
53	4 0111	- 0082
54	4 2127	3526
55	4 1923	1708
56	4 2127	- 0083
57	4 3918	3541
58	4 3788	1722
59	4 3918	- 0083
60	4 6362	3591
61	4 6293	1729
62	4 6362	- 0083

• LATERAL DRIFT ($1/30$)

* $1/318$ FOR DISPLACEMENTS CALCULATED FOR A $1/30$ WIND LOAD

LEVEL	N-S		E-W		ALLOWABLE * DRIFT (IN)
	DISPL. (IN)	DRIFT (IN)	DISPL. (IN)	DRIFT (IN)	
20	7.812		4.634		.698
19	7.266	.546	4.392	.242	.472
18	6.881	.385	4.213	.179	.472
17	6.473	.408	4.011	.202	.472
16	6.051	.422	3.800	.211	.472
15	5.621	.430	3.582	.218	.472
14	5.191	.430	3.355	.227	.472
13	4.758	.433	3.124	.231	.472
12	4.325	.433	2.886	.238	.472
11	3.903	.422	2.646	.240	.472
10	3.483	.420	2.412	.234	.472
9	3.068	.415	2.174	.238	.472
8	2.664	.404	1.934	.240	.472
7	2.266	.398	1.692	.242	.472
6	1.883	.383	1.450	.242	.472
5	1.514	.369	1.209	.241	.472
4	1.172	.342	869	.240	.472
3	.852	.320	731	.238	.472
2	.555	.297	497	.234	.472
1	.280	.275	266	.231	.472
0	.000	.280	.000	.266	.698

• LATERAL FORCE DISTRIBUTION FACTORS

- THE DEFLECTION OF LEVEL 20 UNDER A 50^k POINT LOAD AT EACH LEVEL IS GIVEN BELOW FOR EACH BRACED FRAME
- THE RELATIVE STIFFNESS CAN BE REPRESENTED AS THE FORCE PER UNIT OF DEFLECTION.

$$\lambda = P/\Delta_t = 50/\Delta_t$$

FRAME	$\Delta_t (m)$	λ
BF-1	49.00	1.020
BF-2	38.33	1.304
BF-3	17.37	2.879

- EARTHQUAKE LOADS : WHERE F = LOAD ON THE BRACED FRAME
 F_x = LOAD ON THE BUILDING

$$I_p = 2 \times 1.304 \times 30^2 + 2 \times 1.020 \times 75^2 + 2 \times 2.879 \times 17.5^2$$

$$= 2347 + 11475 + 1763 = 15585$$

$$F = \frac{F_x \lambda_i}{\sum \lambda_i} + \frac{T \cdot C_i}{I_p} \lambda_i$$

$$T = F_x c_d \quad c_d = .05 \times D_n$$

BF-.1 :
(N-S)

$$F = \frac{F_x \times 1.020}{2(1.020 + 1.304)} + \frac{(F_x \times 7.5) \times 75 \times 1.020}{15585}$$

$$= (.219 + .037) F_x$$

$$= .26 F_x$$

$$\begin{aligned}
 \text{BF-2 :} \quad F &= \frac{F_x \times 1.304}{2(1.020 + 1.304)} + \frac{(F_x \times 7.5) 30 \times 1.304}{15585} \\
 (\text{N-S}) \quad &= (.281 + .019) F_x \\
 &= .30 F_x
 \end{aligned}$$

$$\begin{aligned}
 \text{BF-3 :} \quad F &= \frac{F_x \times 2.879}{2 \times 2.879} + \frac{(F_x \times 5.75) 17.5 \times 2.879}{15585} \\
 (\text{E-W}) \quad &= (.500 + .019) F_x \\
 &= .52 F_x
 \end{aligned}$$

$$\text{WIND LOADS :} \quad F = \frac{F_x \lambda_i}{\sum \lambda_i}$$

$$\begin{aligned}
 \text{BF-1 :} \quad F &= \frac{F_x \times 1.020}{2(1.020 + 1.304)} = .22 F_x \\
 (\text{E-W}) \quad &
 \end{aligned}$$

$$\begin{aligned}
 \text{BF-2 :} \quad F &= \frac{F_x \times 1.304}{2(1.020 + 1.304)} = .28 F_x \\
 (\text{E-W}) \quad &
 \end{aligned}$$

$$\begin{aligned}
 \text{BF-3 :} \quad F &= \frac{F_x \times 2.879}{2 \times 2.879} = .50 F_x \\
 (\text{N-S}) \quad &
 \end{aligned}$$

SUMMARY :

FRAME	WIND FACTOR		EQ FACTOR	
	CALC.	ASSUMED	CALC.	ASSUMED
BF-1	.22	.24	.26	.29
BF-2	.28	.28	.30	.29
BF-3	.50	.50	.52	.52

• OVERTURNING

- CHECK OVERTURNING MOMENT (OTM) FOR EACH FRAME

BF-1 : $OTM_{WIND} = 176807^{1k}$, $OTM_{EQ} = 121042^{1k}$
(N-S)

$DIST. FACT. W = .22$, $DIST. FACT. E = .26$

$FACTORED\ UPLIFT_W = \frac{1.5 \times 176807 \times .22}{35} = 1667^k$

$FACTORED\ UPLIFT_E = \frac{1.5 \times 121042 \times .26}{35} = 1349^k$

$FACTORED\ DEAD\ LOAD = .85 \times 1923 = 1635^k$

∴ PROVIDE ANCHORAGE OF 32^k

BF-2 $OTM_W = 176807^{1k}$, $OTM_E = 121042^{1k}$
(N-S)

$DIST. FACT. W = .28$, $DIST. FACT. E = .30$

$FACTORED\ UPLIFT_W = \frac{1.5 \times 176807 \times .28}{35} = 2122^k$

$FACTORED\ UPLIFT_E = \frac{1.5 \times 121042 \times .30}{35} = 1556^k$

$FACTORED\ DEAD\ LOAD = 2780 \times .85 = 2363^k$

∴ NO ANCHORAGE REQUIRED

BF-3 $OTM_W = 126765$, $OTM_E = 126808$
(E-W)

$DIST. FACT. W = .50$, $DIST. FACT. E = .52$

$FACTORED\ UPLIFT_W = \frac{1.5 \times 126765 \times .50}{60} = 1585^k$

$FACTORED\ UPLIFT_E = \frac{1.5 \times 126808 \times .52}{60} = 1649^k$

$FACTORED\ DEAD\ LOAD = .85 \times 2780 = 2363^k$

∴ NO ANCHORAGE REQUIRED

BLDG #1COLUMN MEMBERS

TIER	C1	C2	C3	C4	C5
18-20	W10x54	W12x72	W12x65	W8x40	W10x54
16-18	W12x85	WWF18x119*	WWF16x105	W10x49	W12x65
14-16	WWF18x119	WWF20x150	WWF18x134	W10x60	W12x85
12-14	WWF20x150	WWF20x205	WWF20x170	W12x72	WWF16x105
10-12	WWF20x185	WWF20x256	WWF20x205	W12x85	WWF18x119
8-10	WWF20x230	WWF20x306	WWF20x256	WWF14x92	WWF20x150
6-8	WWF20x256	WWF20x377	WWF20x306	WWF16x105	WWF20x170
4-6	WWF20x306	WWF22x416	WWF20x377	WWF18x119	WWF20x185
2-4	WWF20x377	WWF22x484	WWF20x377	WWF18x134	WWF20x205
0-2	WWF22x416	WWF22x484 +2- $\frac{3}{4}$ " @ 5' (20')	WWF20x437	WWF20x150	WWF20x230

*ALL WWF SECTIONS ARE METRIC-WWFM)

BEAM MEMBERS

Level	B1	B2	B3	B4	B5
20	W14x22	W14x22	W14x22	W21x55	W18x40
19	W16x31	W16x36	W14x22	W24x110	W24x94
18	W14x22	W16x31	W14x22	W24x68	W21x55
17	W14x22	W16x31	W14x22	W24x68	W21x55
16	W14x22	W16x31	W14x22	W24x68	W21x55
15	W14x26	W16x31	W14x22	W24x68	W21x55
14	W14x26	W16x31	W14x22	W24x68	W21x55
13	W14x26	W16x31	W14x22	W24x68	W21x55
12	W14x26	W14x34	W14x22	W24x68	W21x55
11	W14x26	W16x36	W14x26	W24x68	W21x55
10	W14x26	W16x36	W14x26	W24x68	W21x55
9	W14x30	W16x36	W14x26	W24x68	W21x55
8	W14x30	W16x36	W14x26	W24x68	W21x55
7	W14x30	W16x36	W14x26	W24x68	W21x55
6	W14x30	W16x40	W14x26	W24x68	W21x55
5	W14x34	W16x40	W14x26	W24x68	W21x55
4	W14x34	W16x40	W14x26	W24x68	W21x55
3	W14x34	W16x40	W14x26	W24x68	W21x55
2	W16x36	W18x45	W14x26	W24x68	W21x55
1	W16x36	W18x45	W14x26	W24x68	W21x55

K-BRACE MEMBERS

TIER	K1	K2	K3
20	W8x31	W8x31	W8x31
19	W8x31	W8x31	W10x45
18	W8x31	W8x31	W8x31
17	W8x31	W8x31	W8x35
16	W8x35	W8x35	W8x40
15	W8x35	W8x40	W8x40
14	W8x40	W8x40	W10x45
13	W8x40	W10x45	W10x45
12	W10x45	W10x49	W10x49
11	W10x45	W10x49	W10x49
10	W10x49	W10x49	W10x49
9	W10x49	W10x49	W10x49
8	W10x49	W10x49	W10x49
7	W10x49	W10x54	W10x49
6	W10x49	W10x54	W10x49
5	W10x49	W10x60	W10x49
4	W10x54	W12x65	W10x49
3	W10x60	W12x65	W10x49
2	W10x60	W12x65	W10x49
1	W12x72	W12x85	W12x65

COLUMN QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W8x40	31.	4	2517	882
W10x54	31.	12	9657	836
W12x65	31.	2	1813	782
W12x72	31.	4	3932	766
W10x49	25.	4	2556	907
W10x60	25.	4	2959	858
W12x65	25.	8	6120	819
W12x72	25.	4	3908	799
W12x85	25.	16	15071	771
WWF(M)14x92	25.	4	4550	860
WWF(M)16x105	25.	14	17801	842
WWF(M)18x119	25.	20	28315	828
WWF(M)18x134	25.	6	9422	815
WWF(M)20x150	31.	4	8558	800
WWF(M)20x230	31.	8	25415	775
WWF(M)22x416	31.	4	22501	759
WWF(M)20x437	31.	2	11811	758
WWF(M)20x150	25.	16	28215	818
WWF(M)20x170	25.	10	19733	807
WWF(M)20x185	25.	12	25568	801
WWF(M)20x205	25.	14	32767	794
WWF(M)20x230	25.	4	10411	787
WWF(M)20x256	25.	10	28747	781
WWF(M)20x306	25.	10	33959	772
WWF(M)20x377	25.	12	49990	769
WWF(M)22x416	25.	4	18355	767
WWF(M)22x484	25.	4	21304	766
WWF(M)22x484 + 2 - 1/2 x 20" R'S	31.	4	32756	784
			<u>Σ = 478122</u>	

Total Cost of Columns = \$ 478100

Total Gross Weight = 1209000 lbs.

Average Cost/Ton = 790 \$/Ton

BEAM QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W14x22	30.	36	11410	915
W14x26	30.	44	15622	867
W21x55	30.	10	8157	942
W24x68	30.	180	173468	900
W24x110	30.	10	14054	811
W14x22	35.	2	660	816
W16x31	35.	12	5382	787
W14x34	35.	2	905	725
W16x36	35.	12	5868	739
W16x40	35.	8	4260	724
W18x45	35.	4	2402	726
W14x22	35.	8	3035	939
W14x26	35.	12	5173	902
W14x30	35.	8	3764	854
W16x31	35.	2	1031	905
W14x34	35.	6	3123	833
W16x36	35.	4	2249	850
W18x40	40.	4	2865	853
W21x55	40.	72	71226	856
W24x94	40.	4	6250	792
			<u>Σ = 340914.</u>	

Total Cost of Beams = *340900

Total Gross Weight = 781200 lbs.

Average Cost/Ton = 873 */Ton

K-BRACE QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W8x31	25.5	8	2934	844
W12x72	25.5	4	2835	702
W12x85	25.5	4	3271	686
W8x31	21.5	24	7766	883
W8x35	21.5	12	4212	848
W8x40	21.5	16	6165	815
W10x45	21.5	12	5211	816
W10x49	21.5	44	20237	794
W10x54	21.5	12	5955	777
W10x60	21.5	12	6477	761
W12x65	21.5	12	6755	732
W8x31	23.8	8	2787	858
W12x65	23.8	8	4904	720
W8x31	19.5	8	2418	909
W8x35	19.5	8	2616	871
W8x40	19.5	16	5729	835
W10x45	19.5	24	9681	836
W10x49	19.5	88	37553	812
			<u>Σ=137516</u>	

Total Cost of K-Bracing = \$137500

Total Gross Weight = 342600 lbs.

Average Cost/Ton = 803 \$/Ton

SUMMARY

Member	Gross Weight (k)	Total Cost (\$)	Average Cost/Ton (\$/Ton)
Columns	1210	478100	790
Beams & Girders	781	340900	856
K-Bracing	343	137500	803
Structural Skeleton	2334	956500	820

BLDG #2COLUMN MEMBERS

TIER	C1	C2	C3	C4	C5
18-20	W10x54	W12x72	W10x6	W8x40	W10x49
16-18	W12x79	WWF18x119	WWF16x105	W10x45	W12x65
14-16	WWF16x105	WWF20x150	WWF18x134	W10x54	W12x85
12-14	WWF18x134	WWF20x205	WWF20x170	W12x65	WWF16x105
10-12	WWF20x170	WWF20x256	WW20x205	W12x79	WWF18x119
8-10	WWF20x205	WWF20x306	WWF20x256	WWF14x92	WWF18x134
6-8	WWF20x256	WWF20x377	WWF22x282	WWF16x105	WWF20x150
4-6	WWF22x282	WWF22x416	WWF22x338	WWF18x119	WWF20x170
2-4	WWF22x338	WWF22x416	WWF22x416	WWF18x119	WWF20x185
0-2	WWF22x416	WWF22x568 12-1/2" R's (20)	WWF22x416	WWF20x150	WWF20x230

*ALL WWF SECTIONS ARE METRIC - WWF(M)

BEAM MEMBERS

Level	B1	B2	B3	B4	B5
20	W14x22	W14x22	W14x22	W21x55	W18x40
19	W16x31	W16x36	W14x22	W24x110	W24x94
18	W14x22	W16x31	W14x22	W24x68	W21x55
17	W14x22	W16x31	W14x22	W24x68	W21x55
16	W14x22	W16x31	W14x22	W24x68	W21x55
15	W14x22	W16x31	W14x22	W24x68	W21x55
14	W14x26	W16x31	W14x22	W24x68	W21x55
13	W14x26	W16x31	W14x22	W24x68	W21x55
12	W14x26	W16x31	W14x22	W24x68	W21x55
11	W14x26	W16x36	W14x26	W24x68	W21x55
10	W14x26	W16x36	W14x26	W24x68	W21x55
9	W14x26	W16x36	W14x26	W24x68	W21x55
8	W14x26	W16x36	W14x26	W24x68	W21x55
7	W14x30	W16x36	W14x26	W24x68	W21x55
6	W14x30	W16x40	W14x26	W24x68	W21x55
5	W14x30	W16x40	W14x26	W24x68	W21x55
4	W14x30	W16x40	W14x26	W24x68	W21x55
3	W14x30	W16x40	W14x26	W24x68	W21x55
2	W14x34	W16x40	W14x26	W24x68	W21x55
1	W14x34	W18x45	W14x26	W24x68	W21x55

K-BRACE MEMBERS

TIER	K1	K2	K3
20	W8x28	W8x31	W8x31
19	W8x31	W8x31	W8x40
18	W8x31	W8x31	W8x31
17	W8x31	W8x31	W8x31
16	W8x31	W8x35	W8x35
15	W8x35	W8x40	W8x40
14	W8x35	W8x40	W8x40
13	W8x40	W8x40	W10x45
12	W8x40	W10x45	W10x45
11	W8x40	W10x49	W10x49
10	W10x45	W10x49	W10x49
9	W10x45	W10x49	W10x49
8	W10x49	W10x49	W10x49
7	W10x49	W10x49	W10x49
6	W10x49	W10x49	W10x49
5	W10x49	W10x54	W10x49
4	W10x49	W10x54	W10x49
3	W10x49	W10x60	W10x49
2	W10x49	W12x65	W10x49
1	W12x65	W12x79	W12x65

COLUMN QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W8x40	29.	4	2396	898
W10x49	29.	8	5692	871
W10x54	29.	4	3058	849
W10x60	29.	2	1656	828
W12x72	29.	4	3722	775
W10x45	23.	4	2291	963
W10x54	23.	4	2581	904
W12x65	23.	12	8622	836
W12x79	23.	8	6661	797
W12x85	23.	8	7053	784
WWF(M)14x92	23.	4	4251	873
WWF(M)16x105	23.	18	21352	854
WWF(M)18x119	23.	20	26385	838
WWF(M)18x134	23.	14	20465	825
WWF(M)20x150	29.	4	8053	805
WWF(M)20x230	29.	8	23873	778
WWF(M)22x416	29.	6	31664	761
WWF(M)20x150	23.	12	19679	827
WWF(M)20x170	23.	14	25668	815
WWF(M)20x185	23.	8	15829	809
WWF(M)20x205	23.	10	21721	801
WWF(M)20x256	23.	10	26649	787
WWF(M)22x282	23.	6	17478	781
WWF(M)20x306	23.	4	12581	777
WWF(M)22x398	23.	6	20797	775
WWF(M)20x377	23.	4	15428	774
WWF(M)22x416	23.	6	25488	772
WWF(M)22x484	23.	4	19719	770
WWF(M)22x484, 12- $\frac{5}{8}$ "x20" B's	29.	4	29936	789
			$\Sigma = 430763$	

Total Cost of Columns = \$430800.

Total Gross Weight = 1077400 lbs.

Average Cost/Ton = 800⁰⁰/Ton.

BEAM QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W14x22	30.	36	11410	915
W14x26	30.	44	15622	867
W21x55	30.	10	8157	942
W24x68	30.	180	173468	900
W24x110	30.	10	14054	811
W14x22	35.	2	660	816
W16x31	35.	14	6279	787
W16x36	35.	12	5868	739
W16x40	35.	10	5325	724
W18x45	35.	2	1201	726
W14x22	35.	10	3794	939
W14x26	35.	14	6035	902
W14x30	35.	10	4705	854
W16x31	35.	2	1031	905
W14x34	35.	4	2082	833
W18x40	40.	4	2865	853
W21x55	40.	72	71226	856
W24x94	40.	4	6250	792
			<u>Σ=340041</u>	

Total Cost of Beams = \$340000

Total Gross Weight = 778600 lbs.

Average Cost/Ton = 874. \$/Ton

K-BRACE QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W8x28	24.8	4	1349	883
W8x31	24.8	4	1437	849
W12x65	24.8	4	2539	716
W12x79	24.8	4	2995	695
W8x31	20.9	28	8881	890
W8x35	20.9	12	4126	855
W8x40	20.9	24	9051	820
W10x45	20.9	12	5100	822
W10x49	20.9	52	23397	799
W10x54	20.9	8	3882	782
W10x60	20.9	4	2110	765
W12x65	20.9	4	2200	736
W8x31	23.1	8	2726	865
W12x65	23.1	8	4781	724
W8x31	18.9	16	4734	918
W8x35	18.9	8	2559	879
W8x40	18.9	24	8398	842
W10x45	18.9	16	6306	843
W10x49	18.9	80	33346	818
			<u>$\Sigma = 129928$</u>	

Total Cost of K-Bracing = \$129900

Total Gross Weight = 318300 lbs.

Average Cost/Ton = 816 \$/Ton

SUMMARY

Member	Gross Weight (k)	Total Cost (\$)	Average Cost/Ton (\$/Ton)
Columns	1077	430800	800
Beams & Girders	779	340000	874
K-Bracing	318	129900	816
Structural Skeleton	2174	900700	829

BLDG #3COLUMN MEMBERS

TIER	C1	C2	C3	C4	C5
18-20	W10x60	W12x79	W12x65	W8x40	W10x54
16-18	WWF14x92*	WWF18x119	WWF16x105	W10x49	W12x72
14-16	WWF18x119	WWF20x170	WWF18x134	W12x65	WWF14x92
12-14	WWF20x150	WWF20x205	WWF20x170	W12x72	WWF16x105
10-12	WWF20x185	WWF20x256	WWF20x205	WWF14x92	WWF18x134
8-10	WWF20x230	WWF22x338	WWF20x256	WWF16x105	WWF20x150
6-8	WWF22x282	WWF22x416	WWF22x282	WWF16x105	WWF20x170
4-6	WWF22x338	WWF22x484	WWF22x338	WWF18x119	WWF20x185
2-4	WWF22x416	WWF22x484 +2-1/2"x21" D's	WWF22x416	WWF18x134	WWF20x205
0-2	WWF22x484	WWF22x484 +2-1"x20" D's	WWF22x484	WWF20x170	WWF20x256

*ALL WWF SECTIONS ARE METRIC-WWF(M)

BEAM MEMBERS

Level	B1	B2	B3	B4	B5
20	W14x22	W14x22	W14x22	W21x55	W18x40
19	W16x31	W16x36	W14x22	W24x110	W24x94
18	W14x22	W16x31	W14x22	W24x68	W21x62
17	W14x22	W16x31	W14x22	W24x68	W21x62
16	W14x26	W16x31	W14x22	W24x68	W21x62
15	W14x26	W16x31	W14x22	W24x68	W21x62
14	W14x26	W16x31	W14x22	W24x68	W21x62
13	W14x26	W16x36	W14x22	W24x68	W21x62
12	W14x26	W16x36	W14x22	W24x68	W21x62
11	W14x26	W16x36	W14x26	W24x68	W21x62
10	W16x31	W16x36	W14x26	W24x68	W21x62
9	W16x31	W16x36	W14x26	W24x68	W21x62
8	W16x31	W16x40	W14x26	W24x68	W21x62
7	W16x31	W16x40	W14x26	W24x68	W21x62
6	W14x34	W16x40	W14x26	W24x68	W21x62
5	W14x34	W16x40	W14x26	W24x68	W21x62
4	W14x34	W18x45	W14x26	W24x68	W21x62
3	W16x36	W18x45	W14x26	W24x68	W21x62
2	W16x36	W18x45	W14x26	W24x68	W21x62
1	W16x36	W18x45	W14x26	W24x68	W21x62

K-BRACE MEMBERS

TIER	K1	K2	K3
20	W8x31	W8x31	W8x31
19	W8x31	W8x31	W10x45
18	W8x31	W8x31	W8x35
17	W8x31	W8x35	W8x35
16	W8x35	W8x40	W8x40
15	W8x35	W8x40	W8x40
14	W8x40	W10x45	W10x45
13	W8x40	W10x49	W10x49
12	W10x45	W10x49	W10x49
11	W10x49	W10x49	W10x49
10	W10x49	W10x49	W10x49
9	W10x49	W10x54	W10x49
8	W10x49	W10x60	W10x49
7	W10x49	W10x60	W10x49
6	W10x54	W12x65	W10x49
5	W10x54	W12x65	W10x49
4	W10x60	W12x65	W10x54
3	W10x60	W12x65	W10x54
2	W12x65	W12x65	W10x54
1	W12x79	W14x92	W12x72

COLUMN QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W8X40	33.	4	2638	869
W10X54	33.	8	6763	825
W10X60	33.	4	3670	806
W12X65	33.	2	1908	774
W12X79	33.	4	4470	746
W10X49	27.	4	2700	887
W12X65	27.	4	3247	804
W12X72	27.	12	10543	786
WWF(M) 14X92	27.	16	19409	849
WWF(M) 16X105	27.	18	24434	833
WWF(M) 18X119	27.	12	18155	819
WWF(M) 18X134	27.	14	23513	807
WWF(M) 20X170	33.	4	10171	788
WWF(M) 20X256	33.	8	29834	768
WWF(M) 22X484	33.	6	41640	756
WWF(M) 20X150	27.	12	22655	811
WWF(M) 20X170	27.	14	29598	801
WWF(M) 20X185	27.	12	27406	795
WWF(M) 20X205	27.	14	35140	789
WWF(M) 20X230	27.	4	11171	782
WWF(M) 20X256	27.	6	18516	776
WWF(M) 22X282	27.	6	20260	771
WWF(M) 22X338	27.	10	40217	766
WWF(M) 22X416	27.	10	49318	764
WWF(M) 22X484	27.	4	22901	762
WWF(M) 22X484 + 2-1" x 20" PL'S	33.	4	36956	785
WWF(M) 22X484 + 2-1/2" x 21" PL'S	27.	4	25920	809
			$\Sigma = 543165$	

Total Cost of Columns = \$543200.

Total Gross Weight = 1379800 lbs.

Average Cost/Ton = 787 \$/Ton

BEAM QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W14X22	30.	36	11410	915
W14X26	30.	44	15622	867
W21X55	30.	10	8157	942
W24X68	30.	180	173448	900
W24X110	30.	10	14054	811
W14X22	35.	2	660	816
W16X31	35.	10	4485	787
W16X36	35.	12	5868	739
W16X40	35.	8	4260	724
W18X45	35.	8	4803	726
W14X22	35.	6	2276	939
W14X26	35.	12	5173	902
W16X31	35.	10	5157	905
W14X34	35.	6	3123	833
W16X36	35.	6	3374	850
W18X40	40.	4	2865	853
W21X62	40.	72	78563	838
W24X84	40.	4	6250	792
			<u>Σ = 349577</u>	

Total Cost of Beams = \$349600.

Total Gross Weight = 805500 lbs.

Average Cost/Ton = 868 \$/Ton

K-BRACE QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W8x31	26.2	8	2995	838
W12x79	26.2	4	3144	691
W14x92	26.2	4	4104	774
W8x31	22.1	20	6601	876
W8x35	22.1	12	4299	842
W8x40	22.1	16	6296	809
W10x45	22.1	8	3549	811
W10x49	22.1	36	16918	789
W10x54	22.1	12	6087	773
W10x60	22.1	16	8832	757
W12x65	22.1	24	13822	729
W8x31	24.6	8	2856	851
W12x72	24.6	8	5486	705
W8x35	20.2	16	5367	863
W8x40	20.2	16	5881	827
W10x45	20.2	16	6627	828
W10x49	20.2	72	31560	805
W10x54	20.2	24	11339	788
			$\Sigma = 145781$	

Total Cost of K-Bracing = \$145800.

Total Gross Weight = 368500 lbs.

Average Cost/Ton = 791 \$/Ton

SUMMARY

Member	Gross Weight (k.)	Total Cost (\$)	Average Cost/Ton (\$/Ton)
Columns	1380	543200	787
Beams & Girders	806	349600	853
K-Bracing	368	145800	791
Structural Skeleton	2554	1038600	813

BLDG # 4COLUMN MEMBERS

TIER	C1	C2	C3	C4	C5
18-20	W10x60	W12x79	W12x65	W8x40	W10x54
16-18	WWF14x92	WWF18x134	WWF16x105	W10x49	W12x72
14-16	WWF18x134	WW20x170	WWF20x150	W12x65	WWF14x92
12-14	WWF20x170	WWF20x230	WWF20x185	W12x79	WWF18x119
10-12	WWF20x205	WWF20x306	WWF20x230	WWF14x92	WWF18x134
8-10	WWF20x256	WWF20x377	WWF20x306	WWF16x105	WWF20x170
6-8	WWF22x282	WWF22x416	WWF20x377	WWF18x119	WWF20x185
4-6	WWF22x338	WWF22x484	WWF20x377	WWF18x134	WWF20x205
2-4	WWF22x416	WWF22x484 +2-1/8"x20" R's	WWF22x416	WWF20x150	WWF20x230
0-2	WWF22x484	WWF22x484 +2-1/8"x20" R's	WWF22x484	WWF20x170	WW20x256

*ALL WWF SECTIONS ARE METRIC - WWF(M)

BEAM MEMBERS

Level	B1	B2	B3	B4	B5
20	W14x22	W16x26	W14x22	W21x62	W18x45
19	W14x34	W18x40	W14x22	W24x110	W24x94
18	W14x26	W16x31	W14x22	W24x68	W21x62
17	W14x26	W16x31	W14x22	W24x68	W21x62
16	W14x26	W16x36	W14x22	W24x68	W21x62
15	W14x26	W16x36	W14x22	W24x68	W21x62
14	W14x26	W16x36	W14x22	W24x68	W21x62
13	W14x26	W16x36	W14x26	W24x68	W21x62
12	W14x26	W16x36	W14x26	W24x68	W21x62
11	W14x30	W16x36	W14x26	W24x68	W21x62
10	W14x30	W16x36	W14x26	W24x68	W21x62
9	W14x30	W16x40	W14x26	W24x68	W21x62
8	W14x30	W16x40	W14x26	W24x68	W21x62
7	W14x30	W16x40	W14x26	W24x68	W21x62
6	W14x34	W16x40	W14x26	W24x68	W21x62
5	W14x34	W18x45	W14x26	W24x68	W21x62
4	W14x34	W18x45	W14x26	W24x68	W21x62
3	W14x34	W18x45	W14x26	W24x68	W21x62
2	W14x34	W18x45	W14x26	W24x68	W21x62
1	W16x36	W18x45	W14x26	W24x68	W21x62

K-BRACE MEMBERS

TIER	K1	K2	K3
20	W8x31	W8x31	W8x31
19	W8x31	W8x40	W10x45
18	W8x31	W8x40	W8x55
17	W8x35	W8x40	W8x40
16	W8x40	W8x40	W10x45
15	W8x40	W10x45	W10x49
14	W10x45	W10x49	W10x49
13	W10x49	W10x49	W10x49
12	W10x49	W10x49	W10x49
11	W10x49	W10x49	W10x49
10	W10x49	W10x49	W10x49
9	W10x49	W10x49	W10x49
8	W10x49	W10x54	W10x49
7	W10x49	W10x54	W10x49
6	W10x49	W10x60	W10x49
5	W10x49	W12x65	W10x49
4	W10x54	W12x65	W10x49
3	W10x54	W12x65	W10x49
2	W10x60	W12x65	W10x54
1	W12x72	W14x92	W12x65

COLUMN QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W8x40	31.	4	2517	882
W10x54	31.	8	6438	836
W10x60	31.	4	3490	816
W12x65	31.	2	1813	782
W12x79	31.	4	4239	753
W10x49	25.	4	2556	907
W12x65	25.	4	3060	819
W12x72	25.	8	6615	799
W12x79	25.	4	3555	783
WWF(M) 14x92	25.	16	18201	860
WWF(M) 16x105	25.	6	7629	842
WWF(M) 18x119	25.	12	16989	828
WWF(M) 18x134	25.	20	31405	815
WWF(M) 20x170	31.	4	9595	792
WWF(M) 20x256	31.	8	28106	770
WWF(M) 22x484	31.	6	39193	757
WWF(M) 20x150	25.	6	10581	818
WWF(M) 20x170	25.	16	31573	807
WWF(M) 20x185	25.	10	21306	801
WWF(M) 20x205	25.	12	28086	794
WWF(M) 20x230	25.	14	36438	787
WWF(M) 20x256	25.	4	11499	781
WWF(M) 22x282	25.	4	12576	776
WWF(M) 20x306	25.	6	20376	772
WWF(M) 22x338	25.	4	14972	770
WWF(M) 20x377	25.	8	33326	769
WWF(M) 22x416	25.	10	45887	767
WWF(M) 22x484	25.	4	21304	766
WWF(M) 22x484 +2-1/2" x 20" R's	31.	4	37288	779
WWF(M) 22x484 +2-1/2" x 20" R's	25.	4	25292	797
			<u>Σ = 535919</u>	

Total Cost of Columns = \$535900.

Total Gross Weight = 1358900 lbs.

Average Cost/Ton = 789 \$/Ton

BEAM QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W14x22	30.	28	8874	915
W14x26	30.	52	18463	867
W21x62	30.	10	8906	912
W24x68	30.	180	173468	900
W24x110	30.	10	14054	811
W14x26	35.	2	750	785
W16x31	35.	4	1794	787
W16x36	35.	14	6846	739
W16x40	35.	8	4260	724
W18x40	35.	2	1105	752
W18x45	35.	10	6004	726
W14x22	35.	2	759	939
W14x26	35.	14	6035	902
W14x30	35.	10	4705	854
W14x34	35.	12	6246	833
W16x36	35.	2	1125	850
W18x45	40.	4	3118	825
W21x62	40.	72	78563	838
W24x94	40.	4	6250	792
			$\Sigma = 351333$	

Total Cost of Beams = \$351300.

Total Gross Weight = 812500 lbs.

Average Cost/Ton = 865 \$/Ton

K-BRACE QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W8x31	25.5	8	2934	844
W12x72	25.5	4	2835	702
W14x92	25.5	4	4004	776
W8x31	21.5	8	2589	883
W8x35	21.5	4	1404	848
W8x40	21.5	24	9247	815
W10x45	21.5	8	3474	816
W10x49	21.5	60	27596	794
W10x54	21.5	16	7940	777
W10x60	21.5	8	4318	761
W12x65	21.5	16	9007	732
W8x31	23.8	8	2787	858
W12x65	23.8	8	4904	720
W8x35	19.5	8	2616	871
W8x40	19.5	8	2864	835
W10x45	19.5	16	6454	836
W10x49	19.5	104	44380	812
W10x54	19.5	8	3678	794
			<u>Σ = 143041</u>	

Total Cost of K-Bracing = \$143000.

Total Gross Weight = 358700 lbs.

Average Cost/Ton = 798 \$/Ton

SUMMARY

Member	Gross Weight (k)	Total Cost (\$)	Average Cost/Ton (\$/Ton)
Columns	1359	535900	789
Beams & Girders	813	351300	865
K-Bracing	359	143000	798
Structural Skeleton	2531	1030200	815

BLDG #5COLUMN MEMBERS

TIER	C1	C2	C3	C4	C5
18-20	W10x49	W12x65	W10x60	W8x40	W10x49
16-18	W12x72	WWF16x105*	WWF14x92	W10x45	W12x65
14-16	WWF16x105	WWF18x134	WWF18x119	W10x54	W12x79
12-14	WWF18x134	WWF20x170	WWF20x150	W12x65	WWF14x92
10-12	WWF20x150	WWF20x230	WWF20x185	W12x79	WWF16x105
8-10	WWF20x185	WWF20x256	WWF20x230	W12x85	WWF18x119
6-8	WWF20x230	WWF22x338	WWF20x256	WWF16x105	WWF20x150
4-6	WWF20x256	WWF22x416	WWF22x282	WWF16x105	WWF20x150
2-4	WWF20x306	WWF22x484	WWF22x338	WWF18x119	WWF20x170
0-2	WWF20x377	WWF22x484 22- $\frac{1}{2}$ " x 21" PL	WWF22x416	WWF18x134	WWF20x205

*ALL WWF SECTIONS ARE METRIC-WWF(M)

BEAM MEMBERS

Level	B1	B2	B3	B4	B5
20	W14x22	W14x22	W14x22	W18x50	W18x40
19	W16x31	W16x36	W14x22	W24x100	W24x94
18	W14x22	W16x 6	W14x22	W21x62	W21x55
17	W14x22	W16x26	W14x22	W21x62	W21x55
16	W14x22	W16x26	W14x22	W21x62	W21x55
15	W14x22	W16x31	W14x22	W21x62	W21x55
14	W14x22	W16x31	W14x22	W21x62	W21x55
13	W14x22	W16x31	W14x22	W21x62	W21x55
12	W14x26	W16x31	W14x22	W21x62	W21x55
11	W14x26	W16x31	W14x22	W21x62	W21x55
10	W14x26	W16x31	W14x22	W21x62	W21x55
9	W14x26	W16x36	W14x26	W21x62	W21x55
8	W14x26	W16x36	W14x26	W21x62	W21x55
7	W14x30	W16x36	W14x26	W21x62	W21x55
6	W14x30	W16x36	W14x26	W21x62	W21x55
5	W14x30	W16x36	W14x26	W21x62	W21x55
4	W14x30	W16x40	W14x26	W21x62	W21x55
3	W14x34	W16x40	W14x26	W21x62	W21x55
2	W14x34	W16x40	W14x26	W21x62	W21x55
1	W14x34	W16x40	W14x26	W21x62	W21x55

0
K-BRACE MEMBERS

TIER	K1	K2	K3
20	W8x24	W8x28	W8x31
19	W8x31	W8x31	W8x40
18	W8x31	W8x31	W8x31
17	W8x31	W8x31	W8x31
16	W8x31	W8x31	W8x31
15	W8x31	W8x35	W8x35
14	W8x31	W8x40	W8x35
13	W8x35	W10x45	W8x40
12	W8x40	W10x49	W8x40
11	W10x45	W10x49	W8x40
10	W10x49	W10x49	W10x45
9	W10x49	W10x49	W10x45
8	W10x49	W10x49	W10x49
7	W10x49	W10x54	W10x49
6	W10x49	W10x60	W10x49
5	W10x49	W10x60	W10x49
4	W10x49	W12x65	W10x49
3	W10x54	W12x65	W10x49
2	W10x54	W12x65	W10x49
1	W12x72	W12x85	W12x65

COLUMN QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W8X40	31.	4	2517	882
W10X49	31.	12	8979	857
W10X60	31.	2	1745	816
W12X65	31.	4	3626	782
W10X45	25.	4	2425	937
W10X54	25.	4	2739	882
W12X65	25.	12	9179	819
W12X72	25.	4	3308	799
W12X79	25.	12	10666	783
W12X85	25.	4	3768	771
WWF(M) 14X92	25.	10	11376	860
WWF(M) 16X105	25.	24	30516	842
WWF(M) 18X119	25.	14	19820	828
WWF(M) 18X134	25.	8	12562	815
WWF(M) 18X134	31.	4	7601	796
WWF(M) 20X205	31.	8	22822	781
WWF(M) 20X377	31.	4	20421	760
WWF(M) 22X416	31.	2	11251	759
WWF(M) 20X150	25.	22	38796	818
WWF(M) 20X170	25.	12	23679	807
WWF(M) 20X185	25.	6	12784	801
WWF(M) 20X230	25.	10	26027	787
WWF(M) 20X256	25.	10	28747	781
WWF(M) 22X282	25.	2	6288	776
WWF(M) 20X306	25.	4	13584	772
WWF(M) 22X338	25.	6	22458	770
WWF(M) 22X416	25.	4	18355	767
WWF(M) 22X484	25.	4	21304	766
WWF(M) 22X484 x2-1/2" x 21" R's	31.	4	29640	739
			$\Sigma = 426996$	

Total Cost of Columns = \$427000.

Total Gross Weight = 1070500 lbs.

Average Cost/Ton = 798 \$/Ton

BEAM QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W14x22	30.	44	13945	915
W14x26	30.	36	12782	867
W18x50	30.	10	6885	874
W21x62	30.	180	160308	912
W24x100	30.	10	12995	825
W14x22	35.	2	660	816
W16x26	35.	6	2351	820
W16x31	35.	12	5382	787
W16x36	35.	12	5868	739
W16x40	35.	8	4260	724
W14x22	35.	14	5312	939
W14x26	35.	10	4310	902
W14x30	35.	8	3764	854
W16x31	35.	2	1031	905
W14x34	35.	6	3123	833
W18x40	40.	4	2865	853
W21x55	40.	72	71226	856
W24x94	40.	4	6250	792
			$\Sigma = 323327$	

Total Cost of Beams = \$323300.

Total Gross Weight = 735728 lbs.

Average Cost/Ton = 879 \$/Ton

K-BRACE QUANTITY TAKE-OFF

SECTION	LENGTH	NUMBER	TOTAL COST	PRICE PER TON
W8x24	25.5	4	1244	924
W8x28	25.5	4	1377	876
W12x72	25.5	4	2835	702
W12x65	25.5	4	3271	686
W8x31	21.5	40	12944	883
W8x35	21.5	8	2808	848
W8x40	21.5	8	3082	815
W10x45	21.5	8	3474	816
W10x49	21.5	48	22077	794
W10x54	21.5	12	5955	777
W10x60	21.5	8	4318	781
W12x65	21.5	12	6755	732
W8x31	23.8	8	2787	858
W12x65	23.8	8	4964	720
W8x31	19.5	24	7254	909
W8x35	19.5	16	5233	871
W8x40	19.5	32	11458	835
W10x45	19.5	16	6454	836
W10x49	19.5	56	23897	812
			$\Sigma = 132137$	

Total Cost of K-Bracing = \$132100.

Total Gross Weight = 326100 lbs.

Average Cost/Ton = \$10*/Ton

SUMMARY

Member	Gross Weight (k.)	Total Cost (\$)	Average Cost/Ton (\$/Ton)
Columns	107	427000	798
Beams & Girders	756	323300	879
K-Bracing	326	132100	810
Structural Skeleton	2183	882400	828

Appendix D

Building Skeleton Costs

This appendix presents a summary of the structural steel skeleton costs for the formed concrete slab building scheme from Appendix C. These costs are combined with the costs calculated for foundations, base plates and fire protection, resulting in a final building skeleton cost for the five buildings. These designs are modified to suit the steel deck building scheme and the costs adjusted accordingly.

Summary of Steel Skeleton Costs (from Appendix C)

Building No.	Storey Height (ft.)	Floor System Dead Load (psf.)	Steel Skeleton Cost (\$)
1	12.5	80	956,500
2	11.5	80	900,700
3	13.5	80	1,038,600
4	12.5	100	1,030,200
5	12.5	60	882,400

Footings

- for ease of calculation the exterior columns rest on square spread footings and a mat foundation is used in the core
- the allowable bearing capacity of the soil is assumed to be 8 ksf, allowing .5 ksf for overburden a net allowable bearing capacity of 7.5 ksf was used
- concrete: $f'_c = 3$ ksi
- reinforcing steel: $f_y = 60$ ksi
- depth of the footing is governed by punching shear and the size of the footing by allowable bearing pressure of the soil
- unit prices for concrete in place including forms and reinforcing steel were obtained from "Mean's Building Construction Cost Data 1978" (12)

Spread Foundations, over 5 cy. = 90 \$/cy.

Mat Foundations, over 20 cy. = 68 \$/cy.

Footing Designations

Building #1:

Foundation	No.	Size (ft. x ft.)	Depth (ft.)	Volume (cy.)	Cost (\$)
M1	1	50.5 x 75.5	4.8	678	46100
F1	4	19 x 19	3.8	203	18270
F2	4	12.5 x 12.5	2.5	58	5220
F3	8	15.5 x 15.5	3.2	228	<u>20520</u>
					\$90110

Building #2:

Foundation	No.	Size (ft. x ft.)	Depth (ft.)	Volume (cy.)	Cost (\$)
M1	1	50 x 75	4.5	625	42500
F1	4	18.3 x 18.5	3.8	193	17370
F2	4	12 x 12	2.5	53	4770
F3	8	15.5 x 15.5	3.2	228	<u>20520</u>
					\$85160

Building #3:

Foundation	No.	Size (ft. x ft.)	Depth (ft.)	Volume (cy.)	Cost (\$)
M1	1	51 x 76	4.8	689	46850
F1	4	19 x 19	3.9	208	18720
F2	4	12.5 x 12.5	2.5	58	5220
F3	8	15.5 x 15.5	3.2	228	<u>20520</u>
					\$91310

Building #4:

Foundation	No.	Size (ft. x ft.)	Depth (ft.)	Volume (cy.)	Cost (\$)
M1	1	51 x 76	4.9	703	47800
F1	4	20 x 20	4.5	267	24030
F2	4	13 x 13	2.6	65	5850
F3	8	16.5 x 16.5	3.4	274	<u>24660</u>
					\$102340

Building #5:

Foundation	No.	Size (ft. x ft.)	Depth (ft.)	Volume (cy.)	Cost (\$)
M1	1	50 x 75	4.5	625	42500
F1	4	17.5 x 17.5	3.6	163	14670
F2	4	11.5 x 11.5	2.4	47	4230
F3	8	14.5 x 14.5	2.9	181	<u>16290</u>
					\$77690

Base Plates

- the steel was assumed to have a yield stress of 36 ksi
- the size of the base plate is governed by the bearing capacity of the concrete
- the thickness of the base plate was calculated using the equation from page 4-119, CISC-Limit States Design Handbook (15)

$$t_p = \sqrt{\frac{2 C_f m^2}{BC F_y}}$$

- the cost of the base plates was determined using the method outlined in the CISC publication "A Project Analysis Approach to buildings" (11)

Building #1:

Column	No.	Size (ft. x ft.)	Thickness (in.)	Weight (lb.)
C1	4	4.3 x 4.3	5.25	15860
C2	4	5.0 x 5.0	6.75	27560
C3	2	4.3 x 4.3	5.75	8680
C4	4	2.5 x 2.5	2.25	2300
C5	8	3.2 x 3.2	3.50	<u>11710</u>
				66110

Cost = Weight x Connection Factor x Cost Factor x Cost Index
of Steel

Cost = $66110 \times 1.02 \times 1.2 \times .40 = \32370

Building #2:

Column	No.	Size (ft. x ft.)	Thickness (in.)	Weight (lb.)
C1	4	4.1 x 4.1	5.00	13730
C2	4	5.0 x 5.0	6.50	26540
C3	2	4.3 x 4.3	5.25	7930
C4	4	2.5 x 2.5	2.25	2300
C5	8	3.1 x 3.1	3.50	<u>10990</u>
				61490

Cost = $61490 \times 1.02 \times 1.2 \times .40 = \30110

Building #3:

Column	No.	Size (ft. x ft.)	Thickness (in.)	Weight (lb.)
C1	4	4.4 x 4.4	5.50	17390
C2	4	5.2 x 5.2	7.00	30910
C3	2	4.3 x 4.3	5.25	7930
C4	4	2.6 x 2.6	2.50	2760
C5	8	3.2 x 3.2	3.50	<u>11710</u>
				70700

Cost = $70700 \times 1.02 \times 1.2 \times .40 = \34610

Building #4:

Column	No.	Size (ft. x ft.)	Thickness (in.)	Weight (lb.)
C1	4	4.5 x 4.5	5.75	19010
C2	4	5.4 x 5.4	7.25	34530
C3	2	4.6 x 4.6	5.75	9940
C4	4	2.6 x 2.6	2.50	2760
C5	8	3.4 x 3.4	4.00	<u>15110</u>
				81350

Cost = $81350 \times 1.02 \times 1.20 \times .40 = \39830

Building #5:

Column	No.	Size (ft. x ft.)	Thickness (in.)	Weight (lb.)
C1	4	4.0 x 4.0	5.00	13070
C2	4	4.7 x 4.7	6.00	21650
C3	2	4.0 x 4.0	4.75	6210
C4	4	2.4 x 2.4	2.25	2120
C5	8	3.0 x 3.0	3.25	9550
				52600

Cost = 52600 x 1.02 x 1.20 x .40 = \$25750

Fire Protection

- spray on fire protection was used for the spandrel beams (B1, B4 & B5)
- the exterior columns (C1, C4 & C5) and the K-bracing in BF-1 were enclosed by drywall for fire protection
- the members of BF-2 and BF-3 were fire proofed by providing an extra layer of drywall each side of the core wall that encloses them

Building #1:

Beam	Section	No.	Length (ft.)	Contact Area (ft. ²) per ft	Contact Area (ft. ²)
B1	W14x22	8	35	3.49	977
B1	W14x26	12	35	3.52	1478
B1	W14x30	8	35	3.94	1103
B1	W16x31	2	35	3.97	278
B1	W14x34	6	35	3.97	834
B1	W16x36	4	35	4.33	606
B4	W21x55	10	30	5.45	1635
B4	W24x68	180	30	6.12	33048
B4	W24x110	10	30	6.95	2085
B5	W18x40	4	40	4.71	754
B5	W21x55	72	40	5.45	15696
B5	W24x94	4	40	6.22	995

Total Contact Area = 59490 ft.²

Length of F/P for exterior columns:

$$\text{Total length} = 16 (18 \times 12 + 2 \times 18) = 4032 \text{ ft.}$$

Length of F/P for BF-1 K-bracing:

$$\text{Total length} = 4 (18 \times 21.5 + 2 \times 25.5) = 1752 \text{ ft.}$$

Area of core wall needing extra drywall layer:

$$\text{Total area} = 2 (60 + 35) \times (18 \times 12 \times 2 \times 18) = 47880 \text{ ft.}$$

Spandrel Beams:	59490 ft. ² @ .25 \$/sf.	= 14872
Exterior Columns:	4032 ft. @ 11 \$/ft.	= 44352
BF-1 K-bracing:	1752 ft. @ 11 \$/ft.	= 19272
Extra Drywall on Core Walls:	47880 ft. ² @ .80 \$/sf.	= 38304
Total Cost		= \$116800

Building #2:

Spandrel Beams:	59370 ft. ² @ .25 \$/sf.	= 14843
Exterior Columns:	3712 ft. @ 11 \$/ft.	= 40832
BF-1 K-bracing:	1712 ft. @ 11 \$/ft.	= 18832
Extra Drywall on Core Walls:	44080 ft. ² @ .80 \$/sf.	= 14843
Total Cost		= \$109800

Building #3:

Spandrel Beams:	59560 ft. ² @ .25 \$/sf.	= 14889
Exterior Columns:	4352 ft. @ 11 \$/ft.	= 47872
BF-1 K-bracing:	1792 ft. @ 11 \$/ft.	= 19712
Extra Drywall on Core Walls:	51680 ft. ² @ .80 \$/sf.	= 41344
Total Cost		= \$123800

Building #4:

Spandrel Beams: 59670 ft.² @ .25 \$/sf. = 14918
 Exterior Columns: 4032 ft. @ 11 \$/ft. = 44352
 BF-1 K-bracing: 1712 ft. @ 11 \$/ft. = 19272
 Extra Drywall on Core Walls: 44080 ft.² @ .80 \$/sf. = 38304
 Total Cost = \$116800

Building #5:

Spandrel Beams: 55770 ft.² @ .25 \$/sf. = 13942
 Exterior Columns: 40320 ft. @ 11 \$/ft. = 44352
 BF-1 K-bracing: 1712 ft. @ 11 \$/ft. = 19272
 Extra Drywall on Core Walls: 44080 ft.² @ .80 \$/sf. = 38304
 Total Cost = \$115900

Formed Slab Building Skeleton Costs

Building No.	Structural Steel (\$1000)	Foundations (\$1000)	Base Plates (\$1000)	Fire Protection (\$1000)	Total (\$1000)
1	956.5	90.1	32.4	116.8	1196
2	900.7	85.2	30.1	109.8	1126
3	1038.6	91.3	34.6	123.8	1288
4	1030.	102.4	39.8	116.8	1289
5	882.4	77.7	25.7	115.9	1102

Steel Deck Building Skeleton Costs

- to accommodate the 10 ft. beam spacing the core of the example building was shifted 5 ft.
- by shifting the core, the loads on the columns in the braced frames change but these changes offset one another
- the shift also increases torsional loads due to earthquake but upon checking, this small increase is absorbed in the slight over estimate of the distribution factors, it should also be pointed out that wind loads govern in the lower levels

Steel Deck Building Scheme

- for the above reasons the cost calculated for columns and braced frames for the example building with a concentric core will be used in estimating the cost of the structural skeleton with the shifted core
- due to the change from 15 ft. to 10 ft. spacing of beams the loads on the spandrel beams B4 and B5 are reduced, therefore they will be redesigned and the cost adjusted accordingly
- the following costs will be used in the evaluation of the steel deck floor system

Steel Deck Building Skeleton Costs

Building No.	Structural Steel/ (\$1000)	Foundations (\$1000)	Base Plates (\$1000)	Fire Protection (\$1000)	Total (\$1000)
1	929.5	90.1	32.4	115.4	1167
2	873.7	85.2	30.1	108.3	1097
3	1016.7	91.3	34.6	122.4	1265
4	1013.1	102.4	39.8	115.5	1271
5	856.3	77.7	25.7	115.3	1075

Appendix E

Floor Slab Costs

Eleven formed concrete slabs of different unit weight, strength and thickness were designed so that their economy in the example building could be determined. Five floor slabs consisting of 3 in. deck and a concrete cover slab with beams at 10 ft. centers were also designed so that the cost of the formed concrete slab could be compared to a steel deck design. This appendix outlines the design assumptions, summarizes the designs and presents the cost of these sixteen floor slabs.

The formed concrete slabs have a 2 hr. fire resistance rating as does the steel deck design with the 3.25 in., semi-lightweight cover slab. However, the steel deck design with a 2.5 in., normal weight cover slab requires sprayed on fire protection to the underside of the deck to obtain a 2 hr. fire resistance rating. When combined with a suspended ceiling, both slab constructions provide a sound transmission rating in excess of STC 60 and an impact noise rating in excess of $INR + 5$.

Formed Concrete Slab

- the slabs were designed according to CSA Standard A23.3-1973 (16)
- the moment coefficients for continuous slabs were used
- the clear span was taken as 24'-6" allowing for a 6" flange width
- dead loads:

Mech & Elec.	5
Ceiling	5
Flooring	<u>2.5</u>
	12.5 psf.
- + self-weight of slab
- live loads:

Use and Occupancy	50
Movable Partitions	<u>20</u>
	70 psf.
- short and long term deflections of the slab were checked against the allowable deflections
- in two of the designs the slab was thickened in the exterior spans to meet deflection requirements
- slab bents:

Cost of Concrete:

Slab No.	Volume (cy./100 sf.)	Unit Supply and Place Cost (\$/cy.)	Supply and Place Cost (\$/sf.)	Finishing Cost (\$/sf.)	Total Concrete (\$/sf.)
12	1.19	40.49	.48	.12	.60
13	1.19	42.63	.51	.12	.63
14	1.19	46.33	.55	.12	.67
15	1.42	68.33	.97	.12	1.09
16	1.42	71.08	1.01	.12	1.13

Cost of Steel Deck Designs:

Slab No.	Concrete (\$/sf.)	Temperature Wire Mesh (\$/sf.)	Steel Deck (\$/sf.)	Flashing of Deck (\$/sf.)	Fire Protection (\$/sf.)	Total Cost (\$/sf.)
12	.60	.13	.94	.03	.36	2.06
13	.63	.13	.94	.03	.36	2.09
14	.67	.13	.94	.03	.36	2.13
15	1.09	.13	.94	.03	-	2.19
16	1.13	.13	.94	.03	-	2.23

Slab Designs:

Slab No.	Slab Thickness (in.)		Weight of Concrete (psf.)	Strength (f_c) (Ksi)
	Exterior Span	Interior Span		
1	5.0	4.5	115	4
2	5.0	5.0	150	3
3	5.5	5.0	115	3
4	5.0	5.0	150	4
5	5.0	5.0	115	4
6	5.0	5.0	150	5
7	5.5	5.5	150	3
8	5.5	5.5	115	3
9	5.5	5.5	150	4
10	5.5	5.5	115	4
11	5.5	5.5	150	5

Arrangement of main reinforcing steel:

Bar Designation: 4 #106T
 #4 ——— Top
 10.6"

Number of bars required in 40' span:

Strip	B1	B2	B3	M1	M2		
Quantity	4	4	14	4	16		
Design	$\frac{wl^2}{24}$	$\frac{wl^2}{10}$	$\frac{wl^2}{11}$	$\frac{wl^2}{11}$	$\frac{wl^2}{16}$		
Moment	24	10	11	11	16		
Bar	4026T	4106T	4050T	4096T	4046T	4130B	4120B
Slab	(hooked)						
1	31	31	31	28	28	49	38
2	31	31	31	28	28	56	37
3	35	28	28	25	25	45	34
4	31	30	30	28	27	55	37
5	31	28	27	25	24	49	33
6	31	30	30	27	27	54	37
7	35	28	28	25	25	50	35
8	35	25	25	23	22	45	35
9	35	28	27	25	25	50	35
10	35	25	24	23	22	45	35
11	35	27	27	25	24	49	35

Number of bars required in 35' span:

Strip	B1	B2	B3	M1	M2		
Quantity	2	2	3	2	4		
Design	$\frac{wl^2}{24}$	$\frac{wl^2}{10}$	$\frac{wl^2}{11}$	$\frac{wl^2}{11}$	$\frac{wl^2}{16}$		
Moment	24	10	11	11	16		
Bar	4026T	4106T	4050T	4096T	4046T	4130B	4120B
Slab	(hooked)						
1	27	27	27	25	24	43	33
2	27	27	27	25	24	49	33
3	31	25	24	22	22	40	30
4	27	27	26	24	24	48	32
5	27	24	24	22	21	43	29
6	27	26	26	24	23	47	32
7	31	25	24	22	22	44	30
8	31	22	22	20	20	40	30
9	31	24	24	22	21	43	30
10	31	22	21	20	19	39	30
11	31	24	23	22	21	43	30

Deflection:

- the deflections were calculated in accordance with clause 7.6.2, CSA, A23.3-1973
- using the moment distribution technique the following moment coefficients (C) were obtained

$$M = C \times w \ell^2$$

Moment coefficients:

- using the equation for deflection:

$$\Delta_m = \frac{\ell^2}{48EI} (M_s - \frac{1}{10} (M_a + M_b))$$

where: M_s = mid span moment

M_a & M_b = end span moments

Δ_m = mid span deflection

- the deflection for spans AA', A'B, and BB' was checked against the allowable deflection

$$\text{allowable short term deflection} = \frac{\ell}{360} = \frac{14.5^2 \times 12}{360} = .483''$$

$$\text{allowable long term deflection} = \frac{\ell}{240} = \frac{14.5^2 \times 12}{240} = .725''$$

- the live load deflection (Δ_L) is determined using the live load alternate spans
- the dead load deflection (Δ_D) is determined using full loading on all spans
- 40% of the live load is assumed to be a sustained load

Span	AA'		A'B		BB'	
Dead Load	$\frac{wL^4}{171EI}$		$\frac{wL^4}{654EI}$		$\frac{wL^4}{348EI}$	
Deflection	$\frac{wL^4}{171EI}$		$\frac{wL^4}{654EI}$		$\frac{wL^4}{348EI}$	
Live Load	$\frac{wL^4}{105EI}$		$\frac{wL^4}{137EI}$		$\frac{wL^4}{126EI}$	
Deflection	$\frac{wL^4}{105EI}$		$\frac{wL^4}{137EI}$		$\frac{wL^4}{126EI}$	
Slab (in)	Short Term	Long Term	Short Term	Long Term	Short Term	Long Term
1	.381	.690	.345	.569	.367	.641
2	.349	.660	.172	.309	.181	.356
3	.323	.605	.275	.466	.291	.529
4	.247	.481	.117	.226	.121	.261
5	.381	.690	.181	.333	.263	.484
6	.180	.381	.010	.190	.100	.225
7	.212	.426	.101	.199	.105	.231
8	.323	.605	.153	.289	.221	.419
9	.140	.317	.081	.160	.082	.191
10	.223	.454	.123	.231	.126	.272
11	.097	.255	.066	.137	.065	.163

Temperature steel:

- the temperature steel is the same for all slabs because the maximum spacing for #4 bars of 18" governs

Temperature Bar List:

$$20 \times 10-4026 = 400 \text{ lb.}$$

$$20 \times 10-4360 = 4810 \text{ lb.}$$

$$16 \times 10-4100 = 1070 \text{ lb.}$$

$$6 \times 10-4270 = 1080 \text{ lb.}$$

$$7360 \text{ lb./floor}$$

- see page 190 for arrangement of temperature reinforcing steel

Cost of Concrete:

Slab	Volume of Concrete per floor (cy.)	Supply and Place Cost (\$/cy.)	Supply and Place Cost (\$/floor)	Supply and Place Cost (\$/sf.)	Forming Cost (\$/sf.)	Finishing Cost (\$/sf.)	Concrete Cost (\$/sf.)
1	216	71.08	15350	1.01	1.50	.12	2.63
2	234	40.49	9480	.63	1.50	.12	2.25
3	239	68.33	16330	1.08	1.50	.12	2.70
4	234	42.63	9980	.66	1.50	.12	2.28
5	234	71.08	16630	1.10	1.50	.12	2.72
6	234	46.33	10840	.72	1.50	.12	2.34
7	257	40.49	10410	.69	1.50	.12	2.31
8	257	68.33	17560	1.16	1.50	.12	2.78
9	257	42.63	10960	.72	1.50	.12	2.34
10	257	71.08	18270	1.21	1.50	.12	2.83
11	257	46.33	11910	.79	1.50	.12	2.41

Cost of Slab:

Slab	Weight of Main Reinf. (lb./floor)	Weight of Temp. Reinf. (lb./floor)	Total Weight of Reinf. (lb./floor)	Total Weight of Reinf. (lb./sf.)	Cost of Reinf.* in place (\$/sf.)	Cost of Concrete (\$/sf.)	Cost of Slab (\$/sf.)
1	15050	7360	22410	1.48	.29	2.63	2.92
2	15160	7360	22520	1.49	.29	2.25	2.54
3	13550	7360	20910	1.38	.27	2.70	2.97
4	14970	7360	22330	1.47	.29	2.28	2.57
5	13450	7360	20810	1.37	.27	2.72	2.99
6	14800	7360	22160	1.46	.29	2.34	2.63
7	13900	7360	21260	1.40	.28	2.31	2.59
8	13140	7360	20500	1.35	.27	2.78	3.05
9	13860	7360	21220	1.40	.28	2.34	2.62
10	13090	7360	20450	1.35	.27	2.83	3.10
11	13750	7360	21110	1.39	.28	2.41	2.69

*Cost of reinforcing steel in place = 395 \$/Ton.

- arrangement of temperature reinforcing steel:

• Steel Deck and Concrete Cover Slab

- the dead and live loads are the same as specified for the formed concrete slab

- the steel deck was selected from the Westeel-Rosco design tables (17) for a 10 ft. continuous span

- the deck acts compositely with the concrete cover slab

Steel Deck Designs:

- .036" thick, Westeel-Rosco T-30V Hi-Band, 3" steel deck was used with the following cover slabs

- spray on fire protection to underside of deck is indicated if required

Cover Slabs:

Steel Deck Design No.	Steel Deck	Cover Slab Thickness (in.)	Concrete Strength f'_c (Ksi)	Weight of Concrete (pcf.)	Fire Protection
12	T-30V .036	2.5	3	150	Y
13	T-30V .036	2.5	4	150	Y
14	T-30V .036	2.5	5	150	Y
15	T-30V .036	3.25	3	115	N
16	T-30V .036	3.25	4	115	N

Temperature Mesh:

- use a 6" x 6" - 10/10 mesh with a 3 ft. wide double layer
over beams B8 for crack control

- allow 5% for lap splices

- cost: 6" x 6" - 10/10 = 55.60 \$/1000 sf.

place mech = .06 \$/sf.

Area of Mesh = $(15150 + 180 \times 3) \times 1.05 = 16475 \text{ ft.}^2$
per floor

Cost of Mesh = $16475 \left(\frac{55.60}{1000} + .06 \right) = \1905

= .13 \$/sf.

Flashing of Deck:

Column Flashing = 22 columns @ \$3.50 ea. = \$ 77

Concrete retainer and flashing = 530 ft. @ .83 \$/lf. = \$440
around perimeter

Total Flashing Cost = \$517

= .03 \$/sf.

Spray on fire protection:

- the contact area ratio for 3" deck is 1.4

Cost of fireproofing = $1.4 \times .25 \text{ $/sf.} = .36 \text{ $/sf.}$

Cost of Concrete:

Slab No.	Volume (cy./100 sf.)	Unit Supply and Place Cost (\$/cy.)	Supply and Place Cost (\$/sf.)	Finishing Cost (\$/sf.)	Total Concrete (\$/sf.)
12	1.19	40.49	.48	.12	.60
13	1.19	42.63	.51	.12	.63
14	1.19	46.33	.55	.12	.67
15	1.42	68.33	.97	.12	1.09
16	1.42	71.08	1.01	.12	1.13

Cost of Steel Deck Designs:

Slab No.	Concrete (\$/sf.)	Temperature Wire Mesh (\$/sf.)	Steel Deck (\$/sf.)	Flashing of Deck (\$/sf.)	Fire Protection (\$/sf.)	Total Cost (\$/sf.)
12	.60	.13	.94	.03	.36	2.06
13	.63	.13	.94	.03	.36	2.09
14	.67	.13	.94	.03	.36	2.13
15	1.09	.13	.94	.03	-	2.19
16	1.13	.13	.94	.03	-	2.23

Appendix F

Composite Floor Beams

In this appendix the steel sections outlined in the alternative floor design section will be combined with the floor slabs of Appendix E. Beams B6, B7, and B8 will be designed and costed for the formed concrete slab floor systems. For the steel deck floor systems the beams B6, B7, B8(40') and B8(50') must be designed and costed.

The design of the composite beams was carried out in accordance with GSA S16.1-1974(8). All the designs meet the floor performance requirements of section 3.2. The long-term deflections were calculated using twice the modular ratio (E_S/E_C) to account for the effects of creep. The costs for the beams include: W or WWF section, cover plate, haunch, shear studs, shoring, cambering, and fire protection. All the possible beam configurations for B6 were designed and costed with each of the sixteen floor slabs. Upon examining the results it was found that little or no saving in the steel beam was achieved by thickening the slab or increasing the concrete strength. Beam B6 accounts for over 60% of the floor beam cost. Therefore, the slab premiums for thicker slabs and higher concrete strengths are not justified and beams B7 and B8 will only be combined with slabs number 1, 2, 12 and 15. The designs and costs for the over 400 beams investigated are summarized in the following table.

Formed Concrete Slab Floor Beams 0

Steel Deck Floor Beams

Designs for B6

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (Fy) (Ksi)	Haunch	Shored or Cambered	Cost (\$)
1	W21x62	-	-	44		S	1014
1	W21x62	-	50	44		S	1042
1	W21x55	-	50	44		S	950
1	W18x60	-	50	44		S	952
1	W18x55	-	50	44		S	888
1	W18x50	-	50	44		S	822
1	W18x45	-	70	44		S	765
1	W18x40	-	60	44	x	S	739
1	W16x45	-	50	44	x	S	776
1	W16x40	-	70	44	x	S	718
1	W16x36	-	100	44	x	S	675
1	W16x31	5"x1/4"x10'	100	44	x	S	685
1	W21x62	-	50	44		C	1147
1	W21x55	-	50	44		C	1055
1	W18x60	-	50	44		C	1032
1	W18x55	-	50	44		C	968
1	W18x50	-	50	44		C	902
1	W18x45	-	50	44	x	C	874
1	W18x45	6.5"x1/4"x10'	50	44		C	905
1	W18x45	-	50	50		C	883
1	W18x40	6.5"x1/4"x17'	50	44		C	893
1	W18x40	-	50	50	x	C	857
1	W18x40	-	50	60		C	857
1	W16x45	6.5"x1/4"x15'	50	44		C	895
1	W16x45	-	50	50		C	841

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (F _y) (Ksi)	Haunch	Shored or Cambered	Cost (\$)
1	W16x40	6.5"x1/4"x20'	50	44		C	863
1	W16x40	-	50	50	x	C	807
1	W16x40	-	50	60		C	807
1	W16x36	6"x3/8"x23'	50	44		C	849
1	W16x36	-	50	60	x	C	784
1	W16x31	4.5"x1/4"x19'	50	60		C	804
1	W16x31	4.5"x1/2"x24'	50	50		C	834
1	W16x31	4.5"x1/4"x14'	50	60	x	C	809
2	W21x68	-	-	44		S	1091
2	W18x50	-	60	44		S	827
2	W18x45	-	100	44		S	778
2	W16x40	-	100	44	x	S	729
2	W16x36	-	100	50	x	S	714
2	W16x36	6"x1/4"x8'	100	44	x	S	732
2	W16x31	4.5"x3/8"x17'	100	44	x	S	738
2	W18x55	-	50	44		C	968
2	W18x50	-	50	50	x	C	991
2	W18x50	6.5"x1/4"x14'	50	44		C	997
2	W18x45	6.5"x1/4"x19'	50	44		C	964
2	W18x45	-	50	60		C	926
2	W18x40	6.5"x3/8"x24'	50	44	x	C	960
2	W18x40	-	50	60	x	C	895
2	W16x45	-	50	60	x	C	919
2	W16x40	6"x1/2"x25'	50	44		C	932
2	W16x36	6"x1/4"x19'	50	60		C	875

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (P_y) (ksi)	Haunch	Shored or Cambered	Cost (\$)
2	W16x40	6"x1/2"x25'	50	50		C	913
2	W16x31	4.5"x1/2"x24'	50	60		C	866
3	W21x62	-	-	44		S	1014
3	W18x45	-	60	44		S	767
3	W16x40	-	80	44	x	S	728
3	W16x36	-	100	44	x	S	683
3	W18x55	-	50	44	x	C	973
3	W18x50	-	50	44		C	908
3	W18x45	-	50	50		C	889
3	W18x45	-	50	44	x	C	879
3	W18x40	6.5"x1/4"x18'	50	44		C	905
3	W18x40	-	50	60		C	864
3	W18x40	-	50	50	x	C	862
3	W16x40	6"x1/4"x21'	50	44		C	889
3	W16x40	-	50	60		C	813
3	W16x36	-	50	60	x	C	789
3	W16x31	4.5"x3/8"x21'	50	60		C	835
4	W21x68	-	-	44		S	1091
4	W18x45	-	100	44		S	770
4	W16x40	-	100	44	x	S	722
4	W16x36	-	100	50	x	S	707
4	W16x36	6"x1/4"x5'	100	44	x	S	704
4	W16x31	4.5"x3/8"x16'	100	44	x	S	731
4	W18x55	-	50	44		C	963
4	W18x50	-	50	50		C	950

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (Ksi) (Ry)	Haunch	Shored or Cambered	Cost (\$)
4	W18x50	6.5"x1/4"x13'	50	44		C	985
4	W18x45	-	50	60		C	920
4	W18x45	6.5"x1/4"x19'	50	44		C	960
4	W18x40	-	50	60	x	C	889
4	W18x40	6.5"x3/8"x24'	50	44		C	956
4	W16x40	6"x1/2"x25'	50	44		C	928
4	W16x36	6"x1/4"x19'	50	60		C	870
4	W16x31	4.5"x1/2"x24'	50	60		C	861
4	W14x38	6"x1/4"x17'	50	60	x	C	890
4	W14x34	6"x3/8"x22'	50	60	x	C	882
4	W14x30	5.5"x1/2"x26'	50	60	x	C	866
5	W21x62	-	-	44		S	1014
5	W18x45	-	60	44		S	761
5	W16x40	-	80	44	x	S	722
5	W16x36	-	100	44	x	S	675
5	W18x50	-	50	44		C	902
5	W18x45	-	50	50		C	883
5	W18x45	-	50	44	x	C	874
5	W18x45	6.5"x1/4"x10'	50	44		C	905
5	W18x40	-	50	60		C	857
5	W18x40	-	50	50	x	C	857
5	W18x40	6.5"x1/4"x18'	50	44		C	900
5	W16x40	-	50	60		C	807
5	W16x40	6"x3/8"x21'	50	44		C	883
5	W16x36	-	50	60	x	C	784

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (Fy) (Ksi)	Haunch	Shored or Cambered	Cost (\$)
5	W16x36	6"x1/2"x25'	50	44	x	C	879
5	W16x36	6"x1/4"x21'	50	50		C	853
5	W16x31	4.5"x3/8"x20'	50	60		C	823
5	W16x31	4.5"x1/4"x17'	50	60	x	C	828
6	W21x68	-	-	44		S	1091
6	W18x45	-	100	44		S	768
6	W16x40	-	100	44	x	S	721
6	W16x36	-	100	50	x	S	706
6	W16x36	6"x1/4"x5'	100	44	x	S	704
6	W16x31	4.5"x3/8"x15'	100	44	x	S	717
6	W18x55	-	50	44		C	962
6	W18x50	-	50	50		C	948
6	W18x50	6.5"x1/4"x13'	50	44		C	985
6	W18x45	-	50	60		C	918
6	W18x45	6.5"x1/4"x19'	50	44		C	959
6	W18x40	-	50	60	x	C	888
6	W18x40	6.5"x3/8"x23'	50	44		C	948
6	W16x40	6"x1/2"x24'	50	44		C	918
6	W16x36	6"x1/4"x19'	50	60		C	869
6	W16x31	4.5"x1/2"x24'	50	60		C	860
7	W24x68	-	-	44		S	1193
7	W21x73	-	-	44		S	1154
7	W18x50	-	70	44		S	832
7	W18x45	-	100	44		S	778

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (Fy) (Ksi)	Haunch	Shored or Cambered	Cost (\$)
7	W16x40	-	100	44	x	S	729
7	W16x36	-	100	50	x	S	714
7	W16x36	6"x1/4"x9'	100	44	x	S	738
7	W16x31	4.5"x3/8"x17'	100	44	x	S	738
7	W21x55	-	50	44		C	1056
7	W18x55	-	50	50		C	1025
7	W18x55	-	50	44	x	C	1004
7	W18x50	-	50	60		C	1001
7	W18x50	-	50	50	x	C	991
7	W18x50	6.5"x1/4"x16'	50	44		C	1010
7	W18x45	-	50	60		C	926
7	W18x45	6.5"x3/8"x21'	50	44		C	996
7	W18x40	-	50	60	x	C	895
7	W18x40	6.5"x1/2"x24'	50	44		C	981
7	W16x40	6"x1/4"x15'	50	60		C	909
7	W16x36	6"x1/4"x21'	50	60		C	888
8	W21x68	-	-	44		S	1091
8	W18x45	-	70	44		S	772
8	W16x40	-	80	44	x	S	728
8	W16x36	-	100	44	x	S	683
8	W16x31	5"x1/4"x10'	100	44	x	S	693
8	W18x50	-	50	44		C	908
8	W18x45	-	50	50		C	889
8	W18x45	6.5"x1/4"x15'	50	44		C	944

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (Fy) (Ksi)	Haunch	Shored or Cambered	Cost (\$)
8	W18x40	-	50	60		C	864
8	W18x40	6.5"x1/4"x24"	50	44		C	944
8	W16x40	-	50	60		C	813
8	W16x40	6"x3/8"x22"	50	44		C	896
8	W16x36	6"x1/2"x25"	50	44		C	886
8	W16x36	6"x1/4"x15"	50	60		C	855
8	W16x31	4.5"x3/8"x23"	50	60		C	848
9	W24x68	-	-	44		S	1103
9	W21x73	-	-	44		S	1154
9	W18x50	-	70	44		S	826
9	W18x45	-	100	44		S	770
9	W16x40	-	100	44	x	S	722
9	W16x36	-	100	50	x	S	707
9	W16x31	4.5"x3/8"x17"	100	44	x	S	731
9	W21x55	-	50	44		C	1051
9	W18x55	-	50	50		C	1019
9	W18x55	-	50	44	x	C	999
9	W18x50	-	50	60		C	994
9	W18x50	-	50	50	x	C	986
9	W18x50	6.5"x1/4"x16"	50	44		C	1005
9	W18x45	-	50	60		C	920
9	W18x45	6.5"x3/8"x21"	50	44		C	991
9	W18x40	-	50	60	x	C	889
9	W18x40	6.5"x1/2"x24"	50	44		C	976

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (Fy) (Ksi)	Haunch	Shored or Cambered	Cost (\$)
9	W16x40	6"x1/4"x16'	50	60		C	910
9	W16x36	6"x1/4"x21'	50	60		C	883
10	W21x68	-	-	44		S	1091
10	W18x45	-	60	44		S	761
10	W18x40	-	100	44		S	718
10	W16x36	-	100	44	x	S	675
10	W16x31	5"x1/4"x9'	100	44	x	S	679
10	W18x50	-	50	44		C	902
10	W18x45	-	50	50		C	883
10	W18x45	6.5"x1/4"x14'	50	44		C	931
10	W18x40	-	50	60		C	857
10	W18x40	6.5"x1/4"x20'	50	44		C	913
10	W16x40	-	50	60		C	807
10	W16x40	6"x3/8"x22'	50	44		C	890
10	W16x36	6"x1/4"x14'	50	60		C	842
10	W16x31	4.5"x3/8"x21'	50	60		C	830
11	W24x68	-	-	44		S	1103
11	W21x73	-	-	44		S	1154
11	W18x50	-	70	44		S	824
11	W18x45	-	100	44		S	768
11	W16x40	-	100	44	x	S	721
11	W16x36	-	100	50	x	S	706
11	W16x31	4.5"x3/8"x16'	100	44	x	S	724
11	W21x55	-	50	44		C	1050

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (F) (KSI)	Haunch	Shored or Cambered	Cost (\$)
11	W18x55	-	50	50		C	1018
11	W18x55	-	50	44	x	C	999
11	W18x50	-	50	60		C	993
11	W18x50	-	50	50	x	C	985
11	W18x50	6.5"x1/4"x16'	50	44		C	1004
11	W18x45	-	50	60		C	918
11	W18x45	6.5"x3/8"x19'	50	44		C	975
11	W18x40	-	50	60	x	C	888
11	W18x40	6.5"x1/2"x24'	50	44		C	975
11	W16x40	6"x1/4"x15'	50	60		C	902
11	W16x36	6"x1/4"x20'	50	60		C	875
11	W16x31	4.5"x1/2"x25'	50	60		C	867
12	W18x55	-	-	44		S	862
12	W18x40	-	50	44		S	706
12	W16x40	-	50	44		S	681
12	W16x36	-	60	44		S	633
12	W16x31	-	100	44		S	594
12	W18x40	-	50	44		C	786
12	W16x40	-	50	44		C	736
12	W16x36	-	50	50		C	722
12	W16x36	6"x1/4"x10'	50	44		C	751
12	W16x31	-	50	40		C	692
12	W16x31	4.5"x1/4"x19'	50	44		C	747
12	W16x26	4.5"x1/4"x21'	50	50		C	720

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (F _y) (ksi)	Haunch	Shear or Cambered	Cost (\$)
12	W16x26	4.5"x1/4"x13'	50	60		C	697
12	W14x34	-	50	60		C	692
12	W14x30	6"x1/4"x6'	50	60		C	674
12	W14x26	4"x1/4"x17'	50	60		C	681
12	W14x22	4"x3/8"x24'	50	60		C	675
13	W18x55	-	-	44		S	862
13	W16x31	-	90	44		S	583
13	W16x40	-	50	44		C	732
13	W16x36	-	50	50		C	716
13	W16x36	6"x1/4"x9'	50	44		C	739
13	W16x31	-	50	60		C	687
13	W16x31	4.5"x1/4"x19'	50	44		C	743
13	W16x26	4.5"x1/4"x13'	50	60		C	691
13	W14x34	-	50	60		C	687
13	W14x30	6"x1/4"x6'	50	60		C	668
13	W14x26	4"x1/4"x17'	50	60		C	677
13	W14x22	4"x3/8"x24'	50	60		C	670
14	W18x55	-	-	44		S	862
14	W16x31	-	90	44		S	582
14	W16x40	-	50	44		C	731
14	W16x36	-	50	50		C	715
14	W16x36	6"x1/4"x9'	50	44		C	738
14	W16x31	-	50	60		C	686
14	W16x31	4.5"x1/4"x19'	50	44		C	743

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (F _y) (Ksi)	Haunch	Shored or Cambered	Cost (S)
14	W16x26	4.5"x1/4"x13'	50	60		C	690
14	W14x34	-	50	60		C	686
14	W14x30	6"x1/4"x6'	50	60		C	667
15	W18x50	-	-	44		S	799
15	W16x31	-	80	44		S	593
15	W16x40	-	50	44		C	742
15	W16x36	-	50	44		C	688
15	W16x31	-	50	60		C	698
15	W16x31	4.5"x1/4"x17'	50	44		C	740
15	W16x26	4.5"x1/4"x10'	50	60		C	684
15	W14x34	-	50	50		C	665
15	W14x30	-	50	60		C	637
15	W14x26	4"x1/4"x15'	50	60		C	675
15	W14x22	4"x3/8"x22'	50	60		C	667
16	W18x50	-	-	44		S	799
16	W16x31	-	80	44		S	586
16	W16x36	-	50	44		C	683
16	W16x31	-	50	60		C	692
16	W16x31	4.5"x1/4"x17'	50	44		C	735
16	W16x26	4.5"x1/4"x10'	50	60		C	677
16	W14x34	-	50	50		C	659
16	W14x30	-	50	60		C	630
16	W14x26	4"x1/4"x15'	50	60		C	669
16	W14x22	4"x3/8"x22'	50	60		C	662

Designs for Beam B7

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (F _y) (Ksi)	Haunch	Shored or Cambered	Cost (\$)
1	W18x55	-	-	44		S	762
1	W16x40	-	50	44		S	600
1	W16x36	-	80	44		S	564
1	W16x31	-	80	44	x	S	548
1	W16x26	-	100	44	x	S	493
1	W14x34	-	80	44	x	S	555
1	W14x30	-	100	44	x	S	513
1	W14x26	4"x1/4"x5'	100	44	x	S	504
1	W16x40	-	50	44		C	655
1	W16x36	-	50	50		C	641
1	W16x36	6"x1/4"x12'	50	44		C	687
1	W16x31	-	50	60		C	616
1	W16x31	4.5"x3/8"x18'	50	44		C	683
1	W16x26	4.5"x1/4"x15'	50	60		C	641
1	W14x34	-	50	60		C	612
1	W14x34	6"x1/4"x18'	50	44		C	669
1	W14x30	6"x1/4"x11'	50	60		C	633
1	W14x26	4"x1/4"x18'	50	60		C	621
2	W21x55	-	-	44		S	822
2	W18x40	-	60	44		S	627
2	W16x40	-	90	44		S	615
2	W16x36	-	100	44		S	571
2	W16x31	-	100	44	x	S	555
2	W14x34	-	100	44	x	S	561
2	W14x30	4"x1/4"x5'	100	44	x	S	551
2	W18x45	-	50	44		C	754
2	W18x40	-	50	50		C	740
2	W16x40	-	50	60		C	724
2	W16x36	-	50	60		C	671

Slab Number	Beam	Cover Plate	Shear Connection (%)	Grade (F _y) (Ksi)	Haunch	Shored or Cambered	Cost (\$)
2	W16x31	4.5"x1/4"x13'	50	60		C	698
2	W16x26	4.5"x3/8"x20'	50	60		C	685
2	W14x38	-	50	60		C	665
2	W14x34	6"x1/4"x13'	50	60		C	699
2	W14x30	6"x1/4"x18'	50	60		C	679
12	W18x45	-	-	44		S	653
12	W16x36	-	50	44		S	561
12	W16x31	-	50	44		S	514
12	W16x26	-	80	44		S	463
12	W14x30	-	60	44		S	477
12	W14x26	-	100	44		S	449
12	W16x31	-	50	44		C	569
12	W16x26	-	50	60		C	554
12	W16x26	4.5"x1/4"x15'	50	44		C	600
12	W14x34	-	50	44		C	560
12	W14x30	-	50	50		C	542
12	W14x26	-	50	60		C	519
12	W14x22	4"x1/4"x12'	50	60		C	538
15	W18x40	-	-	44		S	605
15	W14x26	-	90	44		S	453
15	W14x22	-	100	50		S	428
15	W14x22	4"x1/4"x5'	100	44		S	450
15	W16x31	-	50	44		C	573
15	W16x26	-	50	50		C	537
15	W14x34	-	50	44		C	564
15	W14x30	-	50	50		C	546
15	W14x26	-	50	60		C	524
15	W14x22	4"x1/4"x9'	50	60		C	525

Designs for Beams B8, B8(40'), B8(50')

Slab Number	Member	Beam	Cover Plate	Shear Connection (%)	Grade (F _y) (Ksi)	Shored or Cambered	Cost (\$)
1	B8	WWF31x103	-	-	44	S	2081
1	B8	WWF28x95	-	50	44	S	1960
1	B8	W24x120	-	50	44	S	2065
1	B8	W24x110	-	50	44	S	1922
1	B8	W24x100	-	50	44	S	1780
1	B8	W24x94	-	80	44	S	1716
1	B8	W24x100	-	50	44	C	1910
1	B8	W24x94	-	50	50	C	1974
1	B8	W24x94	8"x1/4"x18'	50	44	C	1947
1	B8	W24x84	-	50	60	C	1858
1	B8	W24x84	8"x1/4"x21'	50	44	C	1823
1	B8	W24x76	-	50	60	C	1726
1	B8	W24x76	8"x3/8"x23'	50	44	C	1742
1	B8	W24x68	8"x1/2"x26'	50	44	C	1683
1	B8	W24x68	8"x1/4"x19'	50	60	C	1730
1	B8	W21x73	7.5"x1/4"x19'	50	60	C	1773
1	B8	W21x68	7.5"x1/4"x21'	50	60	C	1704
1	B8	W21x62	7.5"x3/8"x25'	50	60	C	1657
1	B8	W21x55	7"x1/2"x27'	60	60	C	1583
2	B8	WWF35x113	-	-	44	S	2317
2	B8	W24x110	-	60	44	S	1928
2	B8	W24x100	-	70	50	S	1956
2	B8	W24x100	10"x1/4"x19'	100	44	S	1962
2	B8	W24x120	-	50	44	C	2192
2	B8	W24x110	-	50	50	C	2228

Slab Number	Member	Beam	Cover Plate	Shear Connection (%)	Grade (F _y) (Ksi)	Shored or Cambered	Cost (\$)
2	B8	W24x110	10"x1/4"x17'	50	44	C	2177
2	B8	W24x100	-	50	50	C	2069
2	B8	W24x100	10"x1/4"x20'	50	44	C	2057
2	B8	W24x94	-	50	60	C	2019
2	B8	W24x94	8"x3/8"x22'	50	44	C	1994
2	B8	W24x84	8"x1/4"x17'	50	60	C	1977
2	B8	W24x76	8"x1/4"x20'	50	60	C	1866
2	B8	W24x68	8"x3/8"x23'	50	60	C	1780
2	B8	W21x73	7"x1/2"x24'	50	60	C	1845
2	B8	W21x68	7"x5/8"x26'	50	60	C	1804
12	B8(40')	WWF28x101	-	-	44	S	1803
12	B8(40')	W24x84	-	100	44	S	1409
12	B8(40')	W24x76	-	100	50	S	1417
12	B8(40')	W24x76	8"x1/4"x14'	100	44	S	1405
12	B8(40')	W24x68	-	100	60	S	1336
12	B8(40')	W24x68	8"x1/2"x22'	100	44	S	1403
12	B8(40')	W21x73	7"x1/2"x23'	100	44	S	1451
12	B8(40')	W21x68	7"x5/8"x24'	100	44	S	1416
12	B8(40')	W24x94	-	60	44	C	1641
12	B8(40')	W24x84	-	60	50	C	1635
12	B8(40')	W24x84	8"x1/4"x8'	60	44	C	1570
12	B8(40')	W24x76	-	50	60	C	1551
12	B8(40')	W24x76	8"x1/4"x14'	80	44	C	1523
12	B8(40')	W24x76	8"x1/4"x14'	80	44	C	1523
12	B8(40')	W24x68	-	70	60	C	1447

Slab Number	Member	Beam	Cover Plate	Shear Connection (%)	Grade (F _y) (Ksi)	Shored or Cambered	Cost (\$)
12	B8(40')	W24x68	8"x1/2"x22'	70	44	C	1514
12	B8(40')	W21x73	7"x1/4"x5'	100	60	C	1535
12	B8(40')	W21x68	7"x1/4"x8'	100	60	C	1483
12	B8(40')	W21x62	7"x1/4"x14'	100	60	C	1436
12	B8(40')	W21x55	7"x3/8"x22'	100	60	C	1408
15	B8(40')	WWF28x101	-	-	44	S	1803
15	B8(40')	W24x84	-	50	44	S	1403
15	B8(40')	W24x76	-	80	44	S	1335
15	B8(40')	W24x68	-	80	50	S	1331
15	B8(40')	W24x68	8"x1/4"x20'	60	44	S	1350
15	B8(40')	W24x94	-	50	44	C	1659
15	B8(40')	W24x84	-	50	50	C	1653
15	B8(40')	W24x84	8"x1/4"x5'	50	44	C	1567
15	B8(40')	W24x76	-	50	60	C	1576
15	B8(40')	W24x76	8"x1/4"x12'	50	44	C	1515
15	B8(40')	W24x68	-	50	60	C	1460
15	B8(40')	W24x68	8"x3/8"x20'	50	44	C	1489
15	B8(40')	W21x73	-	50	60	C	1495
15	B8(40')	W21x73	7"x1/2"x21'	60	44	C	1544
15	B8(40')	W21x68	7"x1/4"x5'	50	60	C	1456
15	B8(40')	W21x62	7"x1/4"x11'	50	60	C	1409
15	B8(40')	W21x55	7"x1/4"x20'	60	60	C	1397
12	B8(50')	WWF35x128	-	-	44	S	2832
12	B8(50')	WWF31x109	-	50	44	S	2442
12	B8(50')	WWF31x103	-	70	44	S	2345

Slab Number	Member	Beam	Cover Plate	Shear Connection (%)	Grade (F _y) (Ksi)	Shored or Cambered	Cost (\$)
12	B8(50')	WWF28x110	-	100	44	S	2456
12	B8(50')	WWF28x101	-	90	50	S	2469
12	B8(50')	WWF28x101	10"x1/4"x17'	100	44	S	2421
12	B8(50')	WWF28x95	10"x1/4"x20'	100	44	S	2335
12	B8(50')	WWF31x109	-	50	44	C	2662
12	B8(50')	WWF31x103	-	70	44	C	2525
12	B8(50')	WWF28x110	-	100	44	C	2611
12	B8(50')	W24x110	-	50	60	C	2507
12	B8(50')	W24x100	-	80	60	C	2342
12	B8(50')	W24x94	8"x1/4"x14'	100	60	C	2342
12	B8(50')	W24x84	8"x1/4"x20'	100	60	C	2201
12	B8(50')	W24x76	8"x1/2"x26'	100	60	C	2151
12	B8(50')	WWF35x128	-	-	44	S	2832
15	B8(50')	WWF31x109	-	50	44	S	2466
15	B8(50')	WWF31x103	-	50	44	S	2357
15	B8(50')	WWF28x110	-	50	44	S	2448
15	B8(50')	WWF28x101	-	100	44	S	2341
15	B8(50')	WWF28x95	10"x1/4"x15'	100	44	S	2346
15	B8(50')	WWF28x110	-	50	44	C	2603
15	B8(50')	W24x100	-	50	60	C	2347
15	B8(50')	W24x94	-	60	60	C	2345
15	B8(50')	W24x84	8"x1/4"x18'	60	60	C	2191
15	B8(50')	W24x76	8"x1/4"x23'	70	60	C	2114
15	B8(50')	W24x68	8"x1/2"x27'	70	60	C	2028

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