Design of Standard Type Buildings Using NBCC/IBC

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Abstract

Analysis and design of type buildings for Canadian applications are based on specifications provided in NBCC – 2005 and Canadian Limit States Steel Standard CAN/CSA S16-01(R05), as discussed in the following.

Using IBC – 2009 load calculations and load combinations are studied and compared against NBCC - 2005 values.

Development of a formatted spreadsheet application based on NBCC - 2005 provides a rapid load calculating and combinations for type steel buildings.

Results of some type buildings modeled in DrFrame software using NBCC 2005 are studied.

Skills and knowledge required for rapid design of steel buildings are discussed.

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1.0 Introduction

As engineers we need to know what loads the structure is experiencing; i.e. as your analysis skills progress you should be able to compute the bending moment diagram for the structure – which sides of the beam are in tension and which are in compression, which parts are in axial compression and which parts are in axial tension. You should be able to know the elements that are stressed the most which are the parts that you want to concentrate your design time on.

In addition, it is necessary to have a good idea of what the answer should be before doing any computer analysis. While sophisticated computer programs to do structural analysis abound it is still very important to have a good idea of a reasonable range of what the correct answer should be before starting on that analysis. A few quick hand calculations will in many cases get you within 10% of the answer.

This report begins with a loads review in the National Building Code of Canada 2005 and International Building Code 2009 focusing on important and rapid techniques end up load values. Next, some of the common applications are described. Following this is an explanation of the current Canadian design code procedure and the spreadsheet developed based on this code. And finally the relationship between loads and members stresses is discussed in Dr Frame software.

2.0 The Structural Design Process

The aim of the structural design process is to produce a set of drawings that clearly shows how to construct the structural system for the building in a manner that will safely resist the loading specified by the Code. The drawings must be clear as they will be read and interpreted by the building authority, the contractor and future engineers trying to modify the building. The process is often a lengthy one and usually involves coordination between many other disciplines involved in a project.

When designing a particular element or area of framing the aim of the structural designer is to find a structural system that satisfies all the required structural criteria, fits the constraints of the area being designed imposed by the project and is the most economical solution. The most economical structural solution is usually defined as being the one that uses the least weight of structural steel – this is a very crude measure that will give an indication to compare the efficiency of designs and is used by the quantity surveyor. As all fabricators will tell you minimum steel weight does not necessarily constitute the minimum cost. When doing the design we usually start by a determination of the governing structural forces in the member, for a beam this would be the factored bending moment, for a column it would be the factored axial load or for a beam column would include combinations of moment and axial forces. We would then pick the minimum weight element that has the capacity to satisfy this load. For a beam we will select the minimum weight beam that satisfies the criteria that its factored moment resistance exceeds the factored moment on the beam. We then must check other criteria such as the deflection before determining that the element is acceptable. Selecting an element at random and then seeing if that element meets all the structural criteria will seldom produce an economical design.

2.1 The Art of Structural Design

The process of taking a building from conceptual design to final inspection may take several months or even years. During that process it is not unusual for the entire concept of the building to evolve several times.

The process will usually involve interaction with consultants from disciplines including architectural, mechanical and electrical and cost consultants. While the aim of each of these consultants is to complete the building their short-term aims may be in conflict with your structural aims. For example you may determine that deeper joists are more economical from a structural standpoint, however, the additional cost of cladding and building height to accommodate these joists may outweigh the structural savings. Many of the problems that you will encounter in the structural design are not significant technical problems but are conflicts in the vision of the team members that must be solved with diplomacy and tact.

On any project the need for well-organized design notes is essential. This is vital when tracing back to see if a beam has sufficient capacity to pick up some extra load that it may be necessary to carry.

The actual design order of the parts designed will vary – one designer for example may elect to design the lateral system before designing the columns. It is likely that the designer will have at least a concept design of the complete building including a feel for approximate sizes prior to fully completing the design and detailing of any of the building components. The complete design of a building may appear to be a daunting task but broken into manageable pieces the task can be tackled in an efficient manner.

The preparation of structural drawings and the coordination of the job with the architect and other disciplines is usually an on-going task and is seldom left to the end of the project. Many design issues will only become apparent as the drawings are prepared. An independent concept review will result in a better design and more readable drawings containing fewer mistakes.

2.2 Load Sheets

One way to make the notes easier to produce and follow later is to eliminate repeating calculations by reference back to calculations that are common to several elements. An example of this is the preparation of load sheets to provide the design loads for specific areas of the project. Using load sheets it is very quick to determine the live load and factored loading for an element such as a beam or a column by multiplying the loading from the load sheet by the tributary area for the element. There are several ways to produce load sheets but one of the most effective methods is to use a spreadsheet approach and summarize the loading from each source and compute subtotals that apply to the deck, the beams and the girders. The dead load and live load are kept separately as it will be necessary to have both the factored loading and the live loading when designing elements such as beams where both 19 forces due factored loads and deflections due to service live loads are checked. The load sheets show very clearly what items have been included in the design of the structural system. It is worthwhile considering carefully if the loading is reasonable and if all items have been considered. Ignoring the weight of housekeeping pads on mechanical floors or the weight of a raised floor can make it difficult to rationalize the safety of the structure to carry these loads later on. Often too much of the design effort is devoted to designing the resistance of the system without sufficient consideration of the loads that are present.

The beam self weight is added directly in the load sheet to alleviate the need to add it in separately to each beam calculation. This procedure supposes that the self-weight of the beams is estimated using a trial design or on the basis of experience with similar designs. If desired, the estimate of beam weight can be checked following the design of several elements with the calculations for these elements modified if necessary. However, the beam self weight is usually only a small percentage of the total load carried by the beam and as long as some estimate of beam self weight has been included in the beam design the variations in the beam self weight will usually not result in beam size changes. Beam self weight will be a more significant portion of the total beam loading where spans are longer or in cases where the superimposed dead and live load are low.

2.3 Limit States Design

The steel code that we follow is called **Limit *States Design*** *of Steel Structures*. There are several limit states that must be satisfied. These consist of ultimate limit states that include exceeding the load carrying capacity and serviceability limit states that include vibration and deflection. Ultimate limit states are examined using load factors applied to the loads while serviceability limit states use unfactored loads.

Ultimate Limit States:

- Exceeding the load carrying capacity
- Overturning
- Sliding
- Fracture
- Fatigue

Serviceability Limit States:

- Deflection
- Vibration
- Permanent Deformation

The majority of our design time will be spent satisfying the ultimate limit state of not exceeding the load carrying capacity either through yielding or buckling and the serviceability limit state through deflection. The limit state design process involves determining the loads on the element and multiplying these loads by load factors given to us by the building code thus producing the factored load on the member. The factored resistance of the member is determined by calculating its resistance then multiplying by a capacity reduction factor, Ø, to give the factored resistance. The factored resistance of the element must be greater than its factored load. The load factors represent the uncertainty and probabilistic nature of the loads while the capacity reduction factors represent the uncertainty and probabilistic nature of the resistance.

This is shown below:

Factored Resistance of member > Factored Load on member

Ø*(Calculated Resistance) > Load Factors*(Calculated Loads)

3.0 NBCC – 2005

The basic building code in Canada is National Building Code of Canada and all other building codes are derived from that code with minor regional differences. “National Building Code of Canada 2005” issued by the Associate Committee on the National

Building Code is essentially applied in Canada. Supplement to the National Building Code of Canada 2005 is necessary for snow and wind load diagrams and interpretation of loading as well.

From the structural designers standpoint there are two sections of interest in the Code itself. The first is Part IV, this has the structural loads and procedures for determining the forces on the building, and you will find how to compute the live loads on the floors here and the seismic and wind procedures. The second part of the Code that is important is Part II that gives the referenced standards.

Three loading cases that produce challenges when designing roof beams of steel buildings are wind uplift, snow load and ponding. As part of this report we will examine lateral buckling of beams as this is an issue in wind uplift where the compression flange is not supported. Wind uplift for example is not an issue with concrete roofs as the concrete dead load is sufficient to resist the uplift and with wood beams lateral instability is less of an issue due to the more bulky nature of the members.

3.1 Dead Load

The specified dead load for a structural member consists of the weight of the member itself, all materials of construction and permanent equipment.

3.2 Live Load

The specified live load on an area of floor or roof depends on the intended use and occupancy, and shall not be less than the uniformly distributed load patterns listed in Article 4.1.5.10. in NBCC 2005.

3.3 Wind Load

The NBCC 2005 code differs from previous editions of the building code in the following issues:

- The load factor for wind in NBCC 2005 is 1.4 previously it was 1.5
- There is now an importance factor, I_w, for wind on both the building and the cladding. To insure that certain buildings will be able to act as shelters after a major wind and to ensure that other post disaster buildings remain operational an importance factor is now used to increase the wind load on certain buildings.
- Under NBCC 2005 the wind load for the design of the building structure to resist wind loading and the design of exterior skin support members is based on the average hourly wind speed with a return period of 50 years.
- The concept of open and rough terrain is introduced for computing the exposure factor. In open terrain the gusts will be larger than they are in rough terrain.
- Under the companion action approach to load combinations there is a wind component that will need to be considered in conjunction with snow loading.
- When computing deflections caused by wind we can multiply the deflection in doing this we use the Serviceability Limit State (SLS) importance factor of 0.75.

For Vancouver: q_{50} = 0.48 kPa Used for structural design for buildings and for cladding.
The q_{50} pressures must be adjusted by the multiplying by several coefficients to give the pressures on the roof surface that are used in design. These factors include the importance factor, I_w, that

depends on the importance classification of the building. Following is the explanation of what we need for load values.

3.3.1 Interior Pressure

The building code in Clause 4.1.7.1.3 gives the interior pressure as:

$$p_i = I_w * q * C_e * C_{gi} * C_{pi}$$

I_w = Importance factor to be used for wind strength design = 1.0 and is a new addition in NBCC 2005.

Building Importance Category	ULS I_w (Strength)	SLS I_w (Deflection)
Normal Importance	1.0	0.75
High Importance	1.15	0.75
Post Disaster	1.25	0.75

Table 1- Wind importance factors

To find the values of C_{gi} & C_{pi} we have to go to Commentary I of the *User's Guide – NBC 2005 Structural Commentaries (Part 4 of Division B).* On page I-22 (Item 31) there is guidance as to the values of C_{gi} for internal pressure and the Cpi coefficient. If there are very few openings in the building then it may be beneficial to compute C_{gi} using the procedure outlined in commentary item 22 on page I-10 and I-11 but the size and number of openings in the building skin has to be unreasonably small for this to be worthwhile and in most circumstances C_{gi} will be 2.0.

The exposure factor, C_e, for computing the effect of interior pressure is taken at the mid-height of the building. The interior pressure is a function of the background leakage of the skin of the building, possibly through openings for windows, vents and doors. It is assumed that these are uniformly distributed over the height of the building and hence the wind speed of concern is the one at mid height of the building.

Interior pressure coefficients (from NBCC 2005 Commentary I-31)

Category	C_{pi}	C_g	Description
Category 1	-0.15 to 0.0	2.0 (See note Below)	Buildings without large openings and having small uniformly distributed openings. Applies to mechanically ventilated higrises and windowless warehouses with door systems not prone to storm damage.
Category 2	-0.45 to 0.3	2.0 (See note below)	Buildings in which significant openings can be relied on to be closed but in which background leakage may not be uniformly distributed. Most low buildings fit into this category provided all elements (shipping door especially) are designed to be fully wind resistant.
Category 3	-0.7 to +0.7	2.0	Buildings in which large or significant exist which can transmit gusts to the interior including sheds and buildings where shipping doors and ventilators may be open in the storm.

Table 2 – Interior pressure coefficient

There is lots of opportunity for debate as to which category the building fits into, some offices consider that tilt-up warehouses fit into category 1 but if all the loading dock doors are on one side it would be fair to say that the background leakage would not be evenly distributed and this type of tilt-up building should fit into category 2. There may also be future modifications to the building where a windowless building is modified to have a more storefront appearance resulting in increased background leakage.

Under NBCC 2005 the gust factor has been modified to reflect that the measured interior gust factor has been found by observation to be higher than 1.0. The wind pressures q_{50} are based on a one-hour average wind speed measured in open terrain at a height of 10m. The gust factor accounts for increases in pressures above the pressure derived from the one-hour average wind speed. Under NBCC 2005 it will be difficult to get a C_{gi} of less than 2.0 and the

table above 16 has been modified to reflect this. Under NBCC 1995 Category 1 and 2 buildings had a C_{gi} of 1.0.
C_e the exposure coefficient is taken at mid height of the building for computing the internal pressure the value of the exposure factor for various heights is given in NBCC 4.1.7.1(5) the following formulas:

$$\text{Open Terrain:} \quad C_e = \left(\frac{h}{10}\right)^{0.2} \quad \text{but not less than 0.9}$$

$$\text{Rough Terrain:} \quad C_e = 0.7 * \left(\frac{h}{12}\right)^{0.3} \quad \text{but not less than 0.7}$$

Rough terrain is suburban or urban landscape or trees and is free of open areas including large parking lots, large playing fields, open agricultural land or bodies of water. The commentary has photos of rough terrain and open terrain and there will often be some engineering judgement involved. Rough terrain should not be assumed if the roughness is derived from trees that are to be cleared during the life of the structure.

The rough terrain and exposed terrain conditions were brought in for NBCC 2005 as there were concerns that the higher wind loads for cladding would have a large financial impact. The feeling was that many buildings are in cities and large towns and would therefore benefit from the reduced wind load that can be realized from the rough terrain that exists there.

As with all wind loading there is plenty of scatter in the results, see also the NBCC Structural Commentaries for cases near hills and a transition formula where there are gaps in the upstream roughness.

For low buildings the C_e coefficients can be simplified in table-3 as following:

Height (m)	**Rough Terrain** Exposure factor C_e	**Open Terrain** Exposure factor C_e
0 to 6m	0.7	0.9
>6m to 12m	0.7	1.0
>12m to 20m	0.8	1.1

Table 3 – Exposure factor

3.3.2 Exterior Pressure

The building code in Clause 4.1.8.1.1 gives the exterior pressure as:

$$p = I_w * q * C_e * C_g * C_p$$

The pressures and gust factors to be used for most low rise buildings are included in figures in Commentary I of the NBCC 2005 Structural Commentaries.

The exposure factor, C_e, for computing the effect of exterior pressure is taken at level of roof of concern. If the building has one roof level take that level if the building has two roof levels then evaluate at each level separately. The effect of the wind gusts on the roof are a function of the wind speed at that level and hence the need for the exposure factor appropriate for the roof level.

3.3.3 Wind Uplift

Wind uplift can be a significant problem on roof beams as the compression flange is now unsupported. While the wind uplift load may be less than half of the factored gravity load the unsupported length of the beam has often reduced the capacity of the beam to less than half its supported capacity. The uplift pressure on the roof is a combination of interior pressure and exterior pressure and both must be calculated.

3.3.4 Downward Wind Loading

For flat roofs the wind loading can exert a downward force on the roof. Prior to NBCC 2005 this was often ignored as the snow load was normally considerably larger than the downward load due to wind forces. Under NBCC 2005 the Code now requires that wind be considered as a companion action to snow loading to the principal action dead load and snow. This gives the following load combination:

$1.25D + 1.5S + 0.4W$

Two load cases to consider depending on evaluating the beam for factored downward load or the effects of wind uplift.

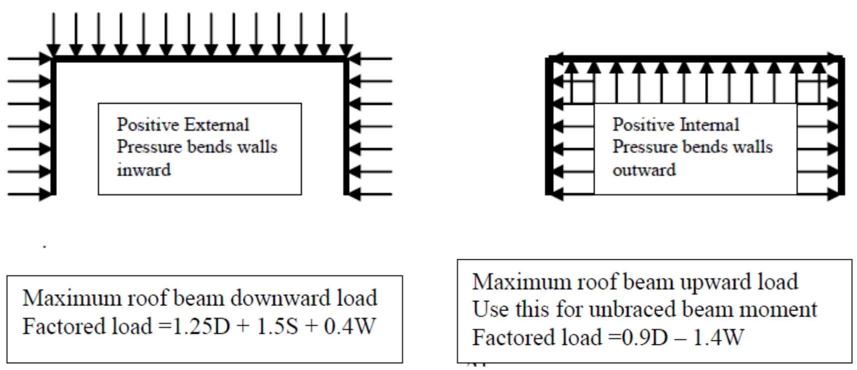


Figure 1 – Wind downward and uplift loads

3.4 Snow Load

The basic roof snow load for a flat roof is determined from clause 4.1.6.2 of the Building Code.

The basic roof snow load equation is:

$$S=I_s * (S_s*(C_b * C_w * C_s * C_a) + S_r)$$

I_s is the importance factor to be used with snow load. As with wind load there is an importance factor to be used with Ultimate Limit State (ULS) for strength design and another to be used with the Serviceability Limit State (SLS) for deflection design.

Building Importance Category	ULS I_s (Strength)	SLS I_s (Deflection)
Normal Importance	1.0	0.9
High Importance	1.15	0.9
Post Disaster	1.25	0.9

Table 4 – Snow importance factors

S_s and S_r are the snow and rain parameters set by the building authority and under NBCC 2005 are based on a return period of 50 years.

C_b is the basic roof snow load multiplier. C_b=0.8 for small roofs (maximum dimension less than 70m). Previous to NBCC 2005 C_b was always a constant of 0.8 which took account of the observed facts that the snow on the roof of a heated building is less than snow on the ground partly because the snow can drift off the roof. However under NBCC 2005 C_b increases to reflect that with large roofs snow can drift from one location to another. For large roofs with C_w=1 (see discussion on C_w below) the value of C_b will be:

$$C_b = 1.0 - \left(\frac{30}{l_c}\right)^2 \quad \text{(Applies to roofs with } l_c\text{>70m)}$$

$$l_c = \text{characteristic length} = l_c = 2w - \frac{w^2}{L}$$

w= shorter roof dimension
L= longer roof dimension

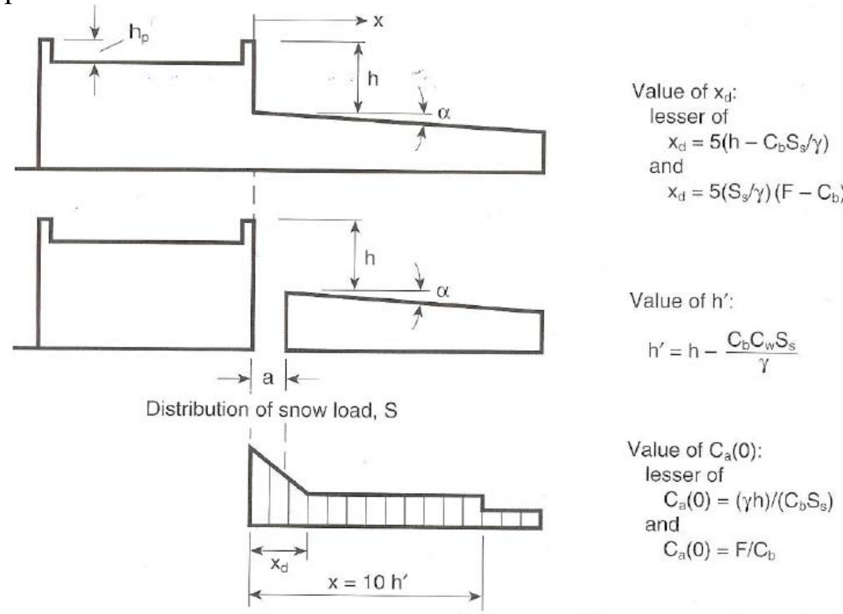
C_s is the slope factor and is 1 for roof slopes less than 15 degree regardless of the roof being slippery or not. Above this slope there are variations depending on whether the slope is classed as being slippery or not. For slippery roofs C_s=0 at 60 while for non-slippery roofs C_s=0.0 when the roof slope exceeds 70 degree, a value of C_s=0 means that the Code considers that there is no snow on the roof.

C_w is a wind factor that should be taken as 1.0 in the Vancouver area. C_w may only be used to reduce snow load if the building is in open terrain on all sides and will remain so for the remainder of its life. C_w may not be used to reduce the snow load on *Post Disaster* buildings.

3.4.1 Snow Drifting

Snow drifting can be a significant problem requiring changes to the framing with additional beams or joists around roof projections such as penthouses to prevent the roof deck from becoming overstressed.

Figure G-4 from commentary G of the *User's Guide – NBC 2005 Structural Commentaries (Part 4)* gives snow drifting adjacent to higher roof such as would occur adjacent to a mechanical penthouse or adjacent to a building setback. The above figure covers a large number of snow drifting cases where a step in the roof occurs or where the roof is adjacent to a mechanical penthouse.

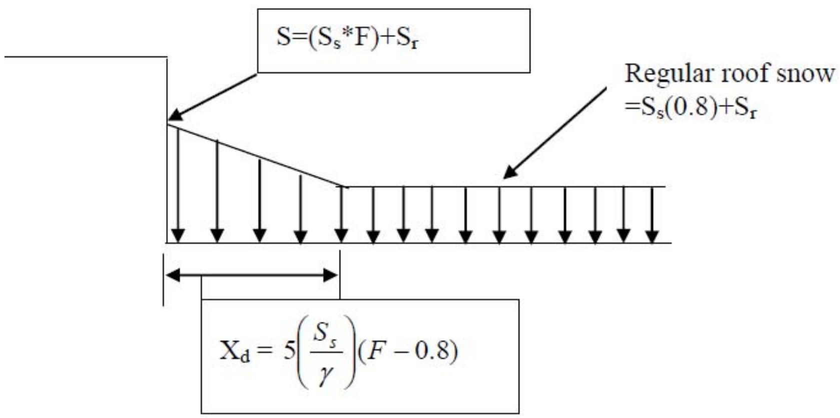


x	Factors(2)		
	C_w	C_s	C_a
0	1.0	$f(\alpha)$(1)	$C_a(0)$
$0 < x \le x_d$	1.0	$f(\alpha)$(1)	$C_a(0) - \frac{(C_a(0) - C_a(x_d))\, x}{x_d}$
$x_d < x \le 10\,h'$	1.0	$f(\alpha)$(1)	1.0
$> 10\,h'$	1.0(3)	$f(\alpha)$(1)	1.0

Table 5 – Snow drift factors

Let's simplify this figure further for one of the more common situations restricted to the following conditions:
1) Located in Vancouver or lower mainland or a sheltered area where $C_w = 1$.
2) Upper and lower roofs are essentially flat. (<10^o slope)
3) Upper and lower roofs have dimensions small enough to give C_b=0.8. (i.e. less than 70m on a side if square, longer if oblong).
4) Upper roof has a parapet that is small enough to ignore (h_p=0).

These simplifications result in a simplified loading diagram form is as follows:



$$F = 0.35 * \sqrt{\left(\frac{\gamma l_c}{S_s}\right)} + C_b \geq 2..0$$

$$\text{characteristic length of upper roof} = l_c = 2w - \frac{w^2}{l}$$

γ = density of snow. The value of γ varies considerably depending on the geographical location, the elevation, the age of the snow and the time of the season. NBCC Commentary G item 6 states that snow at time of deposition fresh snow can be 0.5 kN/m^3 to 1.0 kN/m^3 and increases to 2.0 kN/m^3 to 5.0 kN/m^3. For the lower mainland a value of 3.0 kN/m^3 would be appropriate to use in the snow drift calculations while in dryer and higher areas a value closer to 1.0 kN/m^3 to 2.0 kN/m^3 would be more appropriate.

3.5 Ponding

Ponding is best covered by the attached report from the Part IV of the building code committee of the Association of Professional Engineers of British Columbia. This report includes an example and discusses when ponding would be expected to be most significant. Note that the moment of inertia for the joist is required for the ponding calculation; however, this number is not given directly in some joist catalogues. To get the effective joist moment of inertia it will be necessary to work backwards from the load to produce the allowable deflection in the joist catalogue.

The structural design of many flat roofs will be governed, at least in part, by rain loading. It is important that these rain load criteria not be overlooked by designing the roof only for snow loading. The severity of rain loading is most pronounced in roofs where water from one area can flow and accumulate in another area. In some cases a condition know as "ponding instability" can cause roof deflection under rain loading to increase as the depth of ponded water increases, in turn causing more deflection. Ponding loading is a second order effect, with the load depending on the deflected shape. It is possible to determine an approximate magnitude of the problem using simple assumptions. Rain loads can cause catastrophic failure of the roof. Collapse due to rain loads has occurred in Canada and areas subject to milder climates.

Rain loads will be of particular concern in the design of roofs with one or more of the following conditions:

- "Flat" roofs with slopes greater than 6 inches. If roof slopes are not known, long distances between drain locations can be an indication of possible problems due to the accumulation of water.
- Flexible roofs with joist spans greater than 60 feet or those with beams with span-to-depth ratios greater than 20.
- Roofs located in an area where the one-day rainfall exceeds 4 inches.
- Roofs located in an area where the snow load is less than 30 psf.
- Roofs without scuppers or long distances to scuppers.
- Roofs with beams or joist that run parallel to a roof valley and are located in the valley.
- Roofs where rain from one surface is allowed to collect on a lower surface.

3.6 Minimum Roof Loading

Under the building Code table 4.1.5.3, Specified Uniformly Distributed Live loads on an area of Floor or Roof, the use and occupancy of roofs are listed with a load of 1.0 kPa. We are directed to 4.1.6.1., however, we should probably be directed to 4.1.5.5.2 which states:
"...roofs shall be designed for either the uniform *live loads* specified in Table 4.1.5.3., the concentrated live loads listed in Table 4.1.5.10, or the snow and rain loads prescribed in Subsection 4.1.6, whichever produces the most critical effect on the members concerned."

The intent of the minimum roof load was to provide a live load allowance for workers on the roof and to provide minimum levels of rigidity. For most roofs subject to wind and snow loads in Canada the minimum live load will usually not govern the design of the roof. The minimum roof loading of 1.0 kPa will govern for interior roofs such as the roof over a retail unit in a high interior space such as an airport. The minimum roof live load of 1.0kPa has perhaps unintentionally been taken into account in the computation of column loads of multi-level buildings when using the companion action approach and we will discuss this further in the column section of program.

3.7 Concentrated Loads

Under building Code clause 4.1.5.10 we are required to design for concentrated loads of 1.3 kN spread over 750mm x 750mm to produce the maximum effects. 1.3 kN represents the weight of a worker with tools and will seldom govern unless there is a very short beam and even then it would be difficult to have either the beam and its connections not satisfy 1.3 kN. As discussed in the section on minimum roof loading, above, the concentrated load is considered to act alone and need not be combined with either minimum uniform roof load or snow loads. If there is large mechanical units on the roof the loading for these should be considered as point loads including the effect of any housekeeping pads.

3.8 Seismic Load

3.8.1 Seismic Forces Using Code Static Procedures

The seismic loading on a building comes from movement and imposed deformations and the effects of inertia forces. The load on the building will be a function of the dynamic properties of the structure interacting with the effects of the seismic event at the site. While seismic forces are dynamic and complex in nature, it is common for the seismic design of low-rise buildings to be performed using the static procedures outlined in Clause 4.1.8 of the building code. The limiting seismic force on a building with a seismic system having an R_d of 1.5 or greater is given below.

$$V = \left(\frac{2}{3}\right) * \frac{S(0.2) * I_E W}{(R_d R_o)}$$

Where:
S(0.2) = Site spectrum acceleration at 0.2 seconds determined in accordance with code established procedures that take into account the firm ground acceleration expected and the soil at the site. These values can be found in appendix C of the building code. Values for the seismic coefficients can also be found from the earthquakes Canada website (at no charge) by inputting latitude and longitude of the site. For Vancouver use S(0.2) = 0.94. The values are given by the Geological Survey of Canada and are the mean values with an expectation of being exceeded of 2% in 50 years.
I_E = Importance factor =

- 1.0 for normal buildings
- 1.3 for elementary, middle and high schools, community centres and buildings containing large amounts of toxic material.
- 1.5 for hospitals and post disaster.

W=Weight of the structure including partition load and 25% of snow load. Use C_a=1.0, C_b=0.8 (no drifting) when evaluating snow load for seismic weight. Items that are permanent in the building such as roofing material and mechanical equipment should also be included in the weight. Where there is storage loading 60% of the storage loading should be included and the full weight of liquids in tanks. As the partition load is regarded as being quite conservative with use of modern gyproc partitions, the Code now states that when computing seismic loads we need only use half the code required partition load of 1kPa that was used when computing vertical loads.

R_d= Ductility related force modification factor reflecting the capability of the structure to dissipate energy through inelastic behaviour. The values for R_d are given in the Building Code and to use the value desired it is necessary to follow the detailing rules in the appropriate code. For a steel system must follow the requirements outlined in clause 27 of S16-01. The R-value represents the amount of ductile behaviour that can be expected from the system for steel seismic systems the value of R_d ranges from 5.0 for very ductile systems to 1.5 for systems that are not detailed for ductility.

R_o = the over-strength related force modification factor accounting for the dependable portion of the reserve strength in a structure designed to the provisions of the steel code. The over-strength factor is prescribed in the Code but it is a function of several factors as shown below:

$$R_o = R_{size} * R_Ø * R_{yield} * R_{sh} * R_{mech}$$

Where:
R_{size} = Rounding of sizes and dimension.
$R_Ø$= Difference between nominal and facorted resistance (i.e. 1/Ø)

R_{yield} = Ratio of actual "yield" to minimum specified yield (Specify F_y=345 Mpa but actually comes out higher).
R_{sh}= Overstrength due to strain hardening.
R_{mech}= Overstrength arising from mobilizing full capacity of structure (getting a collapse mechanism).

For steel structures the values of Rd and Ro are given in the table-6 from NBCC:

SFRS Ductility-Related Force Modification Factors, R_d, Overstrength-Related Force Modification Factors, R_o, and General Restrictions[1]
Forming Part of Sentence 4.1.8.9.(1)

Type of SFRS	R_d	R_o	Restrictions[2]				
			Cases Where $I_EF_aS_a(0.2)$				Cases Where $I_EF_vS_a(1.0)$
			< 0.2	≥ 0.2 to < 0.35	≥ 0.35 to ≤ 0.75	> 0.75	> 0.3
Steel Structures Designed and Detailed According to CAN/CSA-S16							
Ductile moment-resisting frames	5.0	1.5	NL	NL	NL	NL	NL
Moderately ductile moment-resisting frames	3.5	1.5	NL	NL	NL	NL	NL
Limited ductility moment-resisting frames	2.0	1.3	NL	NL	60	30	30
Moderately ductile concentrically braced frames							
Non-chevron braces	3.0	1.3	NL	NL	40	40	40
Chevron braces	3.0	1.3	NL	NL	40	40	40
Tension only braces	3.0	1.3	NL	NL	20	20	20
Limited ductility concentrically braced frames							
Non-chevron braces	2.0	1.3	NL	NL	60	60	60
Chevron braces	2.0	1.3	NL	NL	60	60	60
Tension only braces	2.0	1.3	NL	NL	40	40	40
Ductile eccentrically braced frames	4.0	1.5	NL	NL	NL	NL	NL
Ductile frame plate shear walls	5.0	1.6	NL	NL	NL	NL	NL
Moderately ductile plate shear walls	2.0	1.5	NL	NL	60	60	60
Conventional construction of moment frames, braced frames or shear walls	1.5	1.3	NL	NL	15	15	15
Other steel SFRS(s) not defined above	1.0	1.0	15	15	NP	NP	NP

Table 6 – Ductility factors

This table also gives height restrictions, for example a chevron brace can be a maximum of 40m high is a high seismic areas while "Other Seismic Systems not defined above" are marked as NP which means that they are NOT PERMITTED for use in high seismic areas. Those marked "NL" have no height limitation.

3.8.2 Determine the Seismic Shear

The building is a simple building and if the static design procedure is to provide reasonable results for any structure it would be a simple structure such as this one. Propose to determine the seismic forces by using the static design procedure and then verify that it is appropriate to use this procedure.

3.8.3 Simple Hand Procedure

For stiff buildings the cut-off formula from the static procedure will govern. This formula is conservative to use as the required base shear will always be less than this.

$$V = \left(\frac{2}{3}\right) * \frac{F_a * S_a(0.2) * I_E W}{(R_d R_o)}$$

$S_a(0.2)$ are the same as the values listed in Appendix C to NBCC 2005. The value of F_a depends on the site class and the value of $S_a(0.2)$.

This limiting shear can now be computed using a simple one-page worksheet, which also computes the fundamental period. Under NBCC 2010 the limiting formula above does not apply to buildings that are on type F soils which include those subject to liquefaction. For buildings located on this type of soil the forces will be much higher unless ground improvement occurs that changes the site class to be used in design.

Summary (Using 4.1.8.11)

Calculated Base Shear $V = \dfrac{S(T_a) * M_v * I_E * W}{(R_d R_o)}$

Base shear must exceed $V = \dfrac{F_v * S(2.0) * M_v * I_E * W}{(R_d R_o)}$

But need not exceed $V = \left(\dfrac{2}{3}\right) * \dfrac{F_a * S_a(0.2) * I_E * W}{(R_d R_o)}$

Note that at the low periods the cut-off formula will often govern and there is no need to determine the period accurately.

3.8.4 Determining If Static Procedure Can Be Used

Under NBCC 2005 the default seismic design procedure is perform dynamic analysis, we are permitted to use static analysis if the building satisfies any of the criteria outlined in clause 4.1.8.7(1). Paraphrasing and commenting on these clauses we get:

a) In cases where $I_E F_a S_a(0.2)$ is less than 0.35.
b) In regular structures less than 60m in height with a fundamental period less than 2.0 seconds. This is probably true but involves reviewing all the irregularity criteria laid out in table 4.1.8.6.
c) The structure has any type of irregularity except irregularity type 7 (Torsional sensitivity on a rigid diaphragm building) and the building is less than 20m in height and has a fundamental period of less than 0.5seconds in each direction. It can be shown that our building with brace bays on all the exterior faces is not torsionally sensitive and the period is less than 0.5seconds and the height less than 20m so it is permitted to use the static procedure.

3.8.5 Distribute to Levels

The shear is distributed to all levels using the formulas outlined in clause 4.1.8.11(6). As the building period is less than 0.7 seconds there is no shear concentrated at the roof and F_t=0. Force at suspended level 2 (roof level) = F_2 while force at level 1 is F_1.

$$F_x = V * \frac{W_x * h_x}{\sum_i^n W_i h_i}$$

3.9 Load Combinations

The load combinations for use with NBCC 2005 are reproduced below:

Table 4.1.3.2.
Load Combinations for Ultimate Limit States
Forming Part of Sentence 4.1.3.2.(2)

Case	Load Combination(1)	
	Principal Loads	Companion Loads(2)
1	1.4D	
2	(1.25D(3) or 0.9D(4)) + 1.5L(5)	0.5S(6) or 0.4W
3	(1.25D(3) or 0.9D(4)) + 1.5S	0.5L(6)(7) or 0.4W
4	(1.25D(3) or 0.9D(4)) + 1.4W	0.5L(7) or 0.5S
5	1.0D(4) + 1.0E(8)	0.5L(6)(7) + 0.25S(6)

Table 7 – NBCC load combinations

Companion loads are only used with the principal loads if they make the situation worse in the case of wind uplift the 0.5S would not be used as it helps to counteract the wind force as snow cannot be relied on to be present when the wind is acting on the structure.

4.0 IBC 2009

Building Codes provide minimum design requirements to ensure construction of a building withstands natural or man made forces without threat to life or welfare of the public. Lessons learned from past earthquakes have lead to increased enforcement and improvement of building codes.

Two areas still need attention:

- Protection of non-structural (MEP)
- Risks outside California

International Building Code Seismic Requirements And Design Load Procedure is as following.

- Determine the Seismic Use Group (I,II or III). 1616.2
- Determine Soil Site Class. Table 1615.1.1
- Determine the mapped max considered ground acceleration (in %g), Ss and S1. Figure 1615(1) and (2)
- Apply site coefficients to the ground acceleration(1615.1.2)
- Calculate SDS and SD1 per 1615.1.3
- Determine the resulting Seismic Design Category (A to F). Table 1616.3(1) and (2).
- Determine equipment Component Importance Factor (Ip)
- Calculate seismic lateral force
- Design restraint system to resist

Seismic Use Group	Importance Factor, I	Nature of Occupancy
Group I	1.0	Non-critical or hazardous; no risk to general public
Group II	1.25	Educational; Assembly; Nursing homes; Correctional facilities; Any bldg with 5000+ occupancy; Non-essential power and water treatment facilities
Group III	1.5	Fire, rescue and police stations; Hospitals; Use Group I and II having surgery or emergency treatment facilities; Emergency preparedness centers; Post-earthquake recovery vehicle garages and aircraft hangers; Emergency communications and control towers; Bldgs with high hazard or toxic material.

Table 8 - Building seismic use group and importance factor

SITE CLASS	SOIL PROFILE
A	Hard Rock
B	Rock
C	Very dense soil and soft rock
D	Stiff soil profile
E	Soft soil profile
F	Very soft - liquefiable

Table 9 - Site soil class

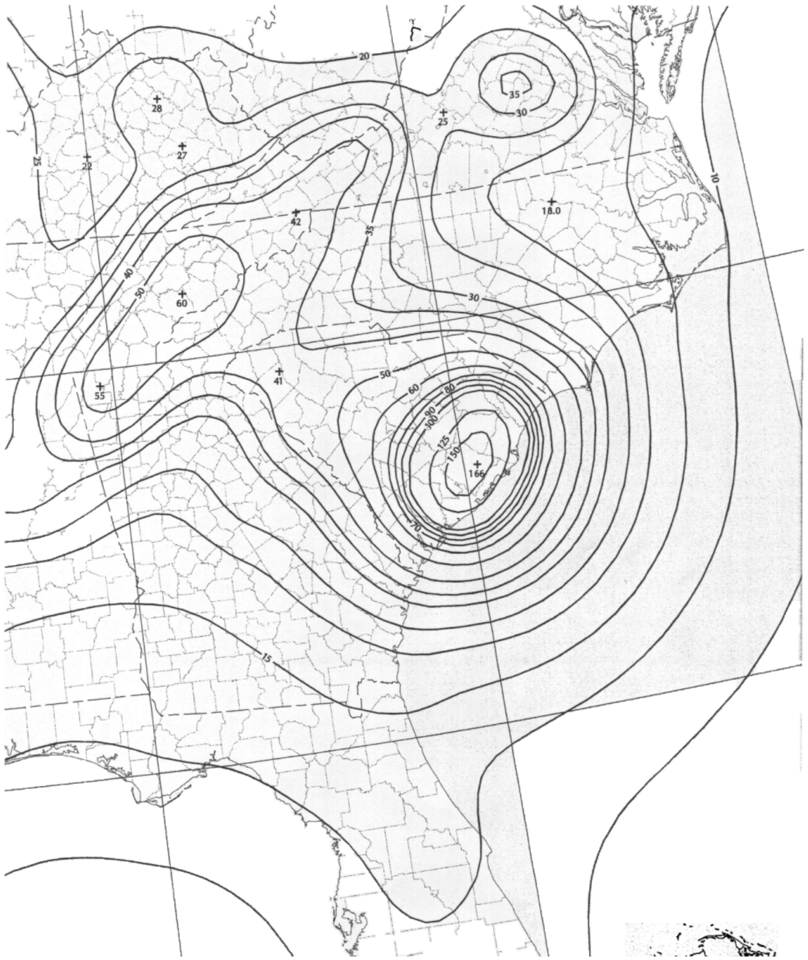


Figure 2 - Ss - Max spectral response acceleration map at 0.2 sec.

Load combinations are as following.

$1.4(D+F)$	**(Equation 16-1)**
$1.2(D+F+T)+1.6(L+H)+0.5(L_r \text{ or } S \text{ or } R)$	**(Equation 16-2)**
$1.2D+1.6(L_r \text{ or } S \text{ or } R)+(f_1L \text{ or } 0.8W)$	**(Equation 16-3)**
$1.2D+1.6W+f_1L+0.5(L_r \text{ or } S \text{ or } R)$	**(Equation 16-4)**
$1.2D+1.0E+f_1L+f_2S$	**(Equation 16-5)**
$0.9D+1.6W+1.6H$	**(Equation 16-6)**
$0.9D+1.0E+1.6H$	**(Equation 16-7)**

Table 10 – IBC load combinations

5.0 Spreadsheet Application

For this project a spreadsheet application was developed to compute the wind load, snow load and combination loads. NBCC 2005 was used as the source for the equations and input values. A printed copy of this spreadsheet is provided in Appendix A of this report.

Where the input parameters are limited to a specific value, such as the importance factors, the user is able to select the desired value from drop down lists.

The spreadsheet works in three different ways:

- Computes the snow load based on NBCC 2005
- Computes the wind load based on NBCC 2005
- Calculates the load combinations including wind uplift and wind downward loads

The spreadsheet which used is built on a formatted sheet provided by Prof. Stiemer and is available for download at www.sigi.ca/engineering/learning_rigor.html.

This spreadsheet allows the user to define variables and write the formulas in text format. The FormatSheet macro is used to parse the formulas and write the excel formula in an adjacent cell.

It is important to note that the FormatSheet macro in the spreadsheet developed for this project was modified to allow variables to contain excel functions. A short check was added to ensure that the reserved word was not located inside a variable before adding it to the reserved words list. A copy of this macro with the clearly demarcated changes is provided in Appendix A of this report.

6.0 Modeling in DrFrame

The most common way to determine the forces in moment frames prior to doing capacity design is to use a frame analysis program and compute the forces. Some of these programs will have programs that do automatic design of steel members but the user should exercise extreme caution in such cases, as the resulting designs will seldom meet seismic design requirements.

The major problems being:

- The programs will often introduce different columns in one lift as opposed to making the column continuous over the maximum length to eliminate costly column splices.
- The programs that do automatic design of steel structures will seldom check the column for the forces derived from yielding of the beam.
- Many moment frames will be drift controlled (making moment frames satisfy the drift requirements will often produce member sizes that make it pointless to detail to requirements greater than Limited Ductility R_d=2.0). Most automated design programs concentrate on adjusting member size to meet strength requirements not drift limitations. When calculating drift the elastic drifts should be multiplied by (R_dR_o/I_E) and should be increased for P-Delta effects by multiplying by the U_2 factor which is increased in accordance to 27.1.8 to account for seismic effects and limited to a maximum of 1.4.

Sizes resulting from analysis of this frame including meeting the strength requirements and normal importance drift limit requirements of 0.02hs (evaluated using an elastic structure with forces corresponding to R_dR_o=1 and IE=1) are shown in the following.

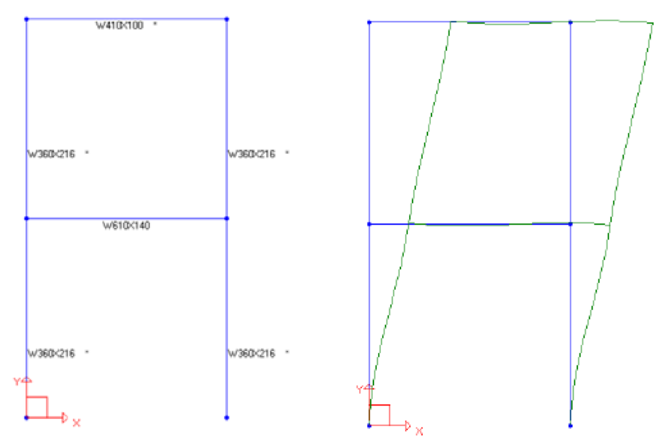


Figure 3 – Structure model of steel frame

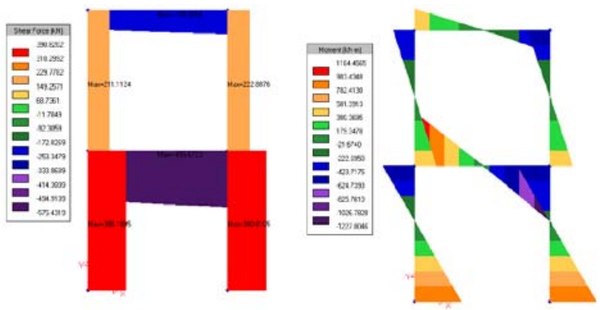


Figure 4 – Analysis outputs of steel frame

DrFrame has been designed to be simple to use for both basic and advanced analyses. For use of load evaluation, DrFrame is the unique software which fits most. Use the various tools in the Tool Palette to create and modify loads, supports, members, etc. the structure will respond immediately to these actions. Selected objects and groups of objects can also be modified using the arrow keys. A few type buildings were modeled in DrFrame to investigate the relationship between the different stresses in members and applied loads.

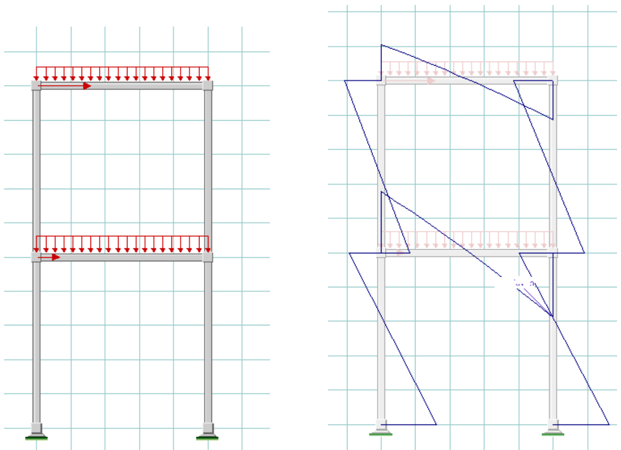


Figure 5 – Structure modeled in Dr Frame

Using DrFrame significantly shows the linear relationship between flexural stress in the members and applied forces. Although this software is simple and easy to use, it is very powerful and handy.

8.0 Conclusion

We need to think more about the loads, the majority of our design time and structural training is spent on the resistance side of the equation but a correct interpretation of the loads will have a huge influence on the effectiveness or economy of the structural system.

Forces don't jump; they need to be provided with a load path. Don't assume that someone else is going to provide that link for you. We need to get a feel for the forces that we are working with.

For instance, for many problems a quick structural analysis can be performed by assuming the point of inflection and determining moments from there.

Analysis and design of type buildings for Canadian applications are based on specifications provided in NBCC – 2005 and Canadian Limit States Steel Standard CAN/CSA S16-01(R05).

Development of a formatted spreadsheet application based on NBCC - 2005 provided a rapid load calculating and combination for type steel buildings.

Skills and knowledge required for rapid load evaluation of steel buildings can be progressed by modeling in Dr Frame software.

8.0 References

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User's Guide – NBCC 2005 Structural Commentaries (Part 4 of Division B).

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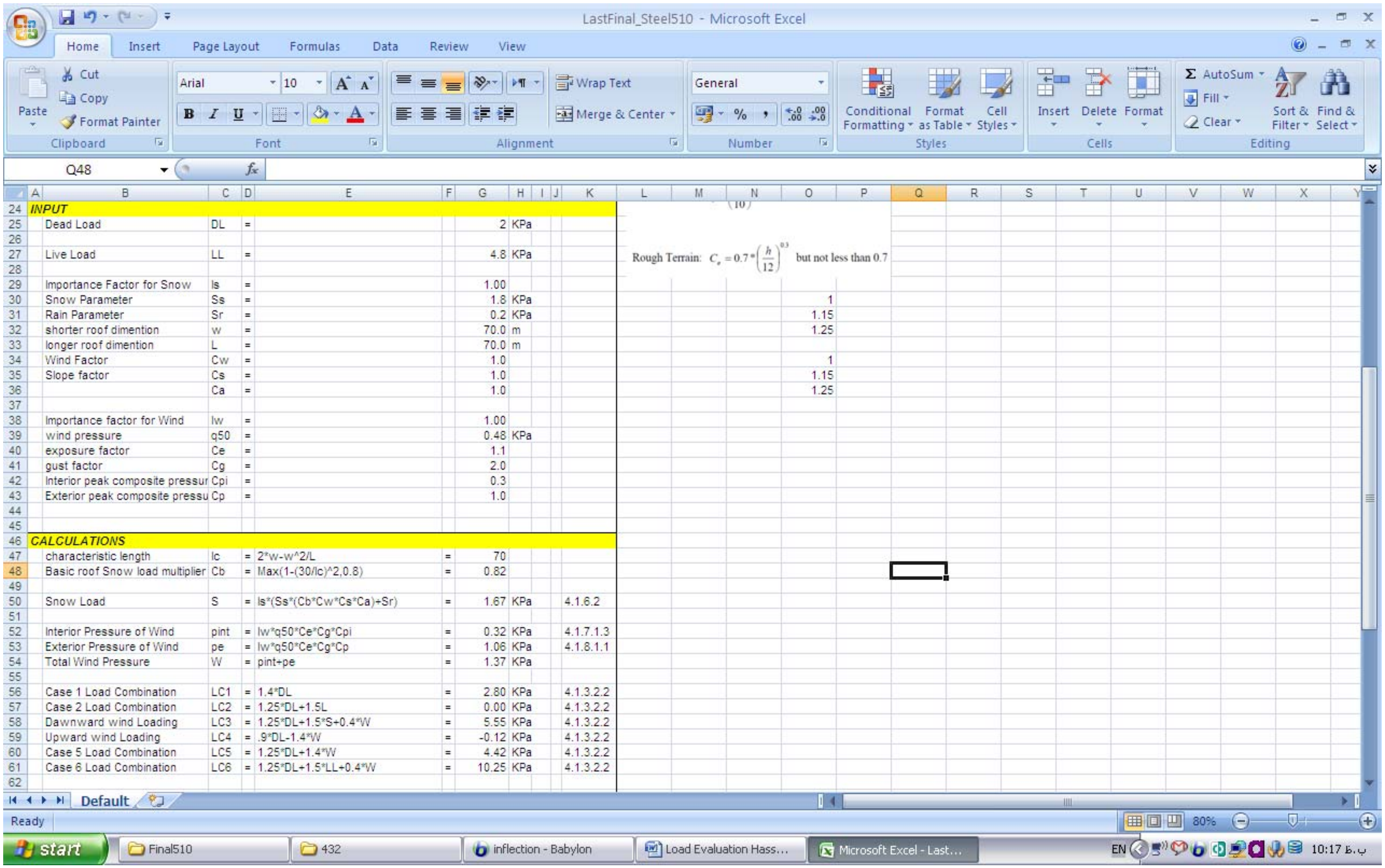
Minimum design loads for buildings and other structures, 2006, ASCE, American society of civil engineers.

DrFrame software user guide.

Appendix A. Spreadsheet



	B	C	D	E	F	G	H	K	O
24	INPUT								
25	Dead Load	DL	=			2	KPa		
26									
27	Live Load	LL	=			4.8	KPa		
28									
29	Importance Factor for Snow	Is	=			1.00			
30	Snow Parameter	Ss	=			1.8	KPa		1
31	Rain Parameter	Sr	=			0.2	KPa		1.15
32	shorter roof dimention	w	=			70.0	m		1.25
33	longer roof dimention	L	=			70.0	m		
34	Wind Factor	Cw	=			1.0			1
35	Slope factor	Cs	=			1.0			1.15
36		Ca	=			1.0			1.25
37									
38	Importance factor for Wind	Iw	=			1.00			
39	wind pressure	q50	=			0.48	KPa		
40	exposure factor	Ce	=			1.1			
41	gust factor	Cg	=			2.0			
42	Interior peak composite pressur	Cpi	=			0.3			
43	Exterior peak composite pressu	Cp	=			1.0			
44									
45									
46	CALCULATIONS								
47	characteristic length	lc	=	2*w-w^2/L	=	70			
48	Basic roof Snow load multiplier	Cb	=	Max(1-(30/lc)^2,0.8)	=	0.82			
49									
50	Snow Load	S	=	Is*(Ss*(Cb*Cw*Cs*Ca)+Sr)	=	1.67	KPa	4.1.6.2	
51									
52	Interior Pressure of Wind	pint	=	Iw*q50*Ce*Cg*Cpi	=	0.32	KPa	4.1.7.1.3	
53	Exterior Pressure of Wind	pe	=	Iw*q50*Ce*Cg*Cp	=	1.06	KPa	4.1.8.1.1	
54	Total Wind Pressure	W	=	pint+pe	=	1.37	KPa		
55									
56	Case 1 Load Combination	LC1	=	1.4*DL	=	2.80	KPa	4.1.3.2.2	
57	Case 2 Load Combination	LC2	=	1.25*DL+1.5L	=	0.00	KPa	4.1.3.2.2	
58	Dawnward wind Loading	LC3	=	1.25*DL+1.5*S+0.4*W	=	5.55	KPa	4.1.3.2.2	
59	Upward wind Loading	LC4	=	.9*DL-1.4*W	=	-0.12	KPa	4.1.3.2.2	
60	Case 5 Load Combination	LC5	=	1.25*DL+1.4*W	=	4.42	KPa	4.1.3.2.2	
61	Case 6 Load Combination	LC6	=	1.25*DL+1.5*LL+0.4*W	=	10.25	KPa	4.1.3.2.2	
62									

Rough Terrain: $C_e = 0.7 * \left(\frac{h}{12}\right)^{0.3}$ but not less than 0.7