Load factor calibration for the proposed 2005 edition of the National Building Code of Canada: Statistics of loads and load effects

F.M. Bartlett, H.P. Hong, and W. Zhou

Abstract: The 2005 edition of the National Building Code of Canada (NBCC) will adopt a companion-action format for load combinations and specify wind and snow loads based on their 50 year return period values. This paper summarizes statistics for dead load, live load due to use and occupancy, snow load, and wind load that have been adopted for calibration, and a companion paper presents the calibration itself. A new survey of typical construction tolerances indicates that statistics for dead load widely adopted for building code calibration are adequate unless the dead load is dominated by thin, cast-in-place concrete toppings. Unique statistics for live load due to use and occupancy are derived that pertain specifically to the live load reduction factor equation used in the NBCC. Statistics for snow and wind loads are normalized using the 50 year values that will be specified in the 2005 NBCC. New statistics are determined for the factors that transform wind speeds and ground snow depths into wind and snow loads on structures.

Key words: buildings, code calibration, companion action, dead loads, live loads, load combinations, load factors, reliability, safety, snow loads, wind loads.

Introduction

This is the first of two papers summarizing the load factor calibration carried out for combinations of dead, live, wind, and snow loads specified in the 2005 edition of the National Building Code of Canada (NBCC). Several changes from the 1995 NBCC have been proposed, including (i) adoption of the “companion-action” format and revised load factors, for load combinations involving wind, snow, and live loads due to use and occupancy at ultimate and serviceability limit states; (ii) use of the 1 in 50 year value, which has a return period of 50 years, instead of the 1 in 30 year value for nominal (or specified) snow and wind loads; and (iii) adoption of importance factors for post-disaster shelters and buildings applied to snow and wind loads. The load factor calibration was carried out for the NBCC Part 4 Task Group on Snow and Wind Loads, whose members are acknowledged at the end of this paper.

The companion-action format for load combinations was recommended for the 2005 NBCC during a meeting of the Task Group on Snow and Wind Loads in October 1999. This format is widely recognized for its simplicity and appropriateness for combining loads that can occur simultaneously (e.g., Turkstra 1972; Ellingwood et al. 1980; Kariyawasam et al. 1997) and has been adopted for many codes and stan-
standards including the CEB (1976), AISC (1986), and ASCE (2000). Structural design based on this format uses load combinations of the form

\[ \alpha_p + \alpha_S + \sum_{i \neq j} \alpha_{ij} S_j \]

where \( \alpha_p \) is the load factor applied to the effects due to permanent load, \( P \), such as dead load; and \( \alpha_S \) is the load factor applied to the effects due to the principal transient load, \( S \), assumed to be acting at its maximum value during the lifetime of the structure. The load factors \( \alpha_{ij} \) applied to the companion-action transient loads \( S_j \) represent the companion-action load effects that occur during the period while the principal action is acting at its maximum value.

An early objective of the calibration process was to determine whether specified snow and wind loads should correspond to 50 year or 500 year return periods. The Task Group on Snow and Wind Loads decided in October 2000 to recommend that specified loads be based on 50 year return periods, in part because (i) these values could be readily obtained from existing databases; (ii) preliminary calibration indicated that load factors less than 1.0 were required if 50 year loads were specified; and (iii) 50 year return periods are used for the loads specified in ASCE7-98 (ASCE 2000).

In this paper, the bases for the statistical parameters used to characterize dead, live, snow, and wind loads are presented. These statistical parameters differ slightly from the values used to verify the 1977 NBCC reported by Nowak and Lind (1979). A companion paper summarizes results of the calibration, describes the evolution of the recommended load factors in response to comments from the Task Group on Snow and Wind Loads and others, and summarizes the final load factors and load combinations recommended for the 2005 NBCC.

Dead load

Dead load consists of the weights of a structural member and the structural components that it supports, partitions, and permanent equipment. Although uncertain, it is commonly assumed to remain constant during the service life of a structure. There is a tendency for the dead load to exceed its nominal value because the designer may overlook something in the dead load takeoff, which is typically estimated to within ±10% for buildings. Dead load uncertainty is because of dimensional tolerances and the uncertainty of unit weights of materials. Additional uncertainty is introduced through the process of converting the load into a load effect, which for indeterminate structures can be done accurately only if the construction sequence is known. It is commonly assumed that dead loads are normally distributed, perhaps because tolerances tend to be normally distributed, although no actual data seem to be available to verify this assumption.

The dead load statistical parameters depend to some extent on the size of structure, the construction material used, and the quality control implemented. The weight of large structural components is relatively insensitive to absolute dimensional tolerances, so the dead load bias coefficients and coefficients of variation are reduced. Improved quality-control procedures have the same effect. Geometric and material unit weight variations are less for steel or precast concrete components than for cast-in-place concrete construction. In particular, formwork deflections in cast-in-place concrete floor construction can often cause the dead load to be larger than the nominal value.

A literature review indicated dead load itself has a bias, or ratio of the mean to nominal values, from 1.00 to 1.05 and a coefficient of variation (CoV), the ratio of the standard deviation to the mean value, between 0.06 and 0.09. Modelling and analysis are typically assumed to be unbiased, with CoVs of 0.03–0.07, and increase the CoV of the dead load effect to between 0.05 and 0.10. Most investigators (Standards Association of Australia 1985, South African Bureau of Standards 1989, and European Committee for Standardization 1994, all reported by Kemp et al. 1998; Tabsh 1997; Ellingwood 1999) adopted a bias of 1.05 and a CoV of 0.10, as reported by Ellingwood et al. (1980), which were used for the current investigation. For cases where the dead load counteracts the effects of other loads, the dead load was assumed normally distributed with a bias of 1.00 and a CoV of 0.10.

In response to comments from members of the Task Group on Snow and Wind Loads and the Canadian Standards Association (CSA) Technical Committee on Reinforced Concrete Design, the statistical basis for dead load factors, including the accuracy of the analysis of dead load effects, was investigated. Findings are presented in this paper concerning the original dead load statistics (Ellingwood et al. 1980) and data from a new survey of concrete floor thickness variability. The companion paper summarizes related proposed revisions to NBCC Commentary G that reflect the accuracy of the analysis of dead load effects.

“Original” dead load statistics and load factors

For the past two decades, code calibrators around the world have characterized dead load effects as having a bias of 1.05 and a CoV of 0.10. The most detailed rationale for these numbers is presented in Appendix B of Ellingwood et al. (1980, p. 145) and is based on the statistics shown in Table 1. The mean dead load is \( 1.0 \times 4.8 + 1.1 \times 1.9 = 6.89 \text{kPa} \), with a standard deviation of \( (1.0 \times 4.8 \times 0.06)^2 + (1.1 \times 1.9 \times 0.15)^2)^{1/2} = 0.43 \text{kPa} \) and CoV of 0.43/6.89 = 0.062. The bias for the dead load effect, accounting for the load model and analysis parameters, is therefore 6.89/6.70 × 1.0 × 1.0 = 1.03, with a CoV of (0.062^2 + 0.05^2 + 0.05^2)^{1/2} = 0.094. In these computations, the bias and uncertainty of the superimposed dead load were relatively large, and the magnitude of the superimposed dead load is also large. The impact of the superimposed dead load was diminished, however, because it is a small fraction of the total dead load.

In the derivation of the load factors for the 1975 edition of the NBCC, the dead load effect was assumed to have a bias of 1.00 and a CoV of 0.10, which included a CoV of 0.07 for the transformation of dead load to dead load effect (Allen 1975). This calibration suggested a dead load factor of 1.30, but this was reduced to 1.25 to maintain the same ratio of dead load factor to live load factor as was used at the time for ultimate strength design of concrete buildings.

Survey in 2000 of concrete floor thickness variability

Members of the CSA Technical Committee on Reinforced Concrete Buildings were invited to estimate typical floor
thickness deviations for cast-in-place concrete construction. Three responses were received, two with quite detailed (and consistent) information as summarized in Table 2. The values for one- and two-way slabs are consistent with thickness tolerances specified in standard CSA-A23.1-00 (CSA 2000). Bias values shown are based on the average of the reported deviations. Coefficients of variation are determined assuming the range represented by the typical difference represents two standard deviations, and the range represented by the worst difference represents four standard deviations.

The tolerances shown in Table 2 indicate that the thickness of cast-in-place toppings on precast construction and cast-in-place cover slabs on unshored composite construction can be extremely variable. It is assumed that the excess thickness is not uniform, but varies parabolically from a maximum, \( \Delta \), at midspan to zero at the ends. The extra load (or shear) in this case is equivalent to a uniform excess thickness of \( 2\Delta/3 \) along the full length of the member. The extra midspan moment for a simply supported member is \( \frac{2}{3} \Delta \) along the full length of the member. If a 50 mm topping is placed on a 200 mm precast hollow core slab, then the overall dead load of topping and precast has a bias of 1.044 and a CoV ranging from 0.053 to 0.132. If a 50 mm topping is placed on a 400 mm precast double tee beam, then the overall dead load of topping and precast has a bias of 1.068 and a CoV ranging from 0.068 to 0.154. The bias and CoV for these cases are slightly greater than those shown in Table 1.

Unshored composite construction was further identified as a potential problem, even though the definition of dead load in clause 7.1.1 of CSA-S16.1-95 (CSA 1995) was amended to include “the additional weight of concrete and finishes resulting from deflections of supporting members”. No guidance about acceptable tolerances for the cover slab thickness is given in the CSA-A23.1-00 (CSA 2000) or CSA-S16.1-95 (CSA 1995) standards. As noted previously, the impact of greater-than-specified slab thickness is more significant for bending moment than for shear force or reactions. Combining the worst values shown for cover slabs in Table 2 with data for typical steel beams and deck, the overall dead load (or shear) of the concrete slab, deck, and steel beam has a bias of 1.22 and a CoV of 0.25. The moment of the slab, deck, and beam has a bias of 1.33 and a CoV of 0.25. These parameters would require dead load factors in the order of 1.8–2.0 to achieve acceptable levels of reliability.

The survey responses also indicated that the weight of connections in steel roofs is known only after the roof is detailed by the fabricator and can exceed the 10% allowance that some (but probably not all) designers use. For example, the allowance necessary for connection weight of the roof of the Skydome in Toronto was 30% (C.M. Allen, personal communication, 2000).

The present study assumed a CoV for dead load analysis of roughly 0.05–0.075, which implies that the actual dead load force effects should generally be within 10–15% of calculated values. Although this margin may seem small, current codes and standards generally require the user to design ductile members and structures that have the capability to redistribute the load if the actual distribution of force effects is not exactly as predicted by an elastic analysis.

### Live load due to use and occupancy

In the present study, live load refers to that associated with the use and occupancy of a building. It includes the weight of people, equipment, and furnishings and materials in storage. The total live load may be represented by a sustained live load component, which remains essentially constant for a period of time that is often associated with the duration of a tenancy, and a transient (or extraordinary) live load component that has a short duration or is instantaneous. The magnitude of each component is conventionally modelled by a Gamma distribution (Peir and Cornell 1973; Chalk and Corotis 1980; Ellingwood et al. 1980), and the arrival of each is modelled as a Poisson process.

### Maximum lifetime live load

For most reliability analyses or code calibrations, the maximum live load during the life of the structure has been represented by a time-independent random variable with a Gumbel probability distribution (Allen 1975; MacGregor 1976; Ellingwood et al. 1980; European Committee for Standardization 1994; Kariyawasam 1996; Tabsh 1997; Ellingwood 1999). The reliabilities obtained by considering the live load as a Poisson process may in some cases differ from those obtained by modelling the maximum live load during the life of the structure using an “equivalent” Gumbel distribution. In practice, however, it is acceptable to represent the maximum live load as a Gumbel variate, and this representation was adopted.

The mean value of the maximum equivalent uniformly distributed live load for office floors during a 50 year reference period was derived following Kariyawasam (1996). From simulation results (Ellingwood and Culver 1977), the

<table>
<thead>
<tr>
<th>Variable</th>
<th>Bias</th>
<th>CoV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete density</td>
<td>-1.00</td>
<td>0.03</td>
</tr>
<tr>
<td>Dimensions and density combined</td>
<td></td>
<td></td>
</tr>
<tr>
<td>150 mm slab</td>
<td>1.00</td>
<td>0.08</td>
</tr>
<tr>
<td>Slab and beam floor</td>
<td>1.00</td>
<td>0.07</td>
</tr>
<tr>
<td>Column</td>
<td>1.04</td>
<td>0.04</td>
</tr>
<tr>
<td>Total (100 pound-force per square foot = 4.8 kPa)</td>
<td>1.00</td>
<td>0.06</td>
</tr>
<tr>
<td>Superimposed dead load: (40 pound-force per square foot = 1.9 kPa)</td>
<td>1.10</td>
<td>0.15</td>
</tr>
<tr>
<td>Load model</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Dead load analysis</td>
<td>1.00</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Table 1. Dead load statistics for cast-in-place concrete (Ellingwood et al. 1980).
expected value of the 50 year maximum equivalent uniformly distributed live load, $L_{\text{max}}$ (in kN/m²), is

\[ L_{\text{max}} = 0.895 + 7.6 / \sqrt{A_I} \]

where $A_I$ is the influence area (in m²). The corresponding nominal live load, $L$ (in kN/m²), from the 1995 NBCC is

\[ L = 2.4 \times (0.3 + \sqrt{9.8/B}) \]

where 2.4 is the specified live load (in kN/m²) and $B$ is the tributary area (in m²). According to ASCE7-98 (ASCE 2000), the influence area equals the tributary area for two-way slabs, twice the tributary area for interior beams, and four times the tributary area for columns. Using these relationships, the bias is simply the ratio of eq. [2] to eq. [3] and so depends on the type of element and its influence area.

The 50 year maximum live load bias is shown in Fig. 1 for realistic ranges of influence areas of two-way slabs, beams, and columns. The ASCE (2000) curve in Fig. 1 indicates the bias if the nominal live load is reduced with increasing influence area in accordance with ASCE7-98 instead of eq. [2]. The 30 year extreme live load model used for previous Canadian design code calibration (Allen 1975) has a bias of 0.70 and a CoV of 0.30; the equivalent 50 year maximum live load has a bias of 0.78 and a CoV of 0.27 and is also shown in Fig. 1.

The present study adopted a bias for the 50 year maximum live load effect of 0.90, based on several considerations. Two-way slabs and other components with small influence areas have greater bias, but the consequences of failure of these elements are less severe than those for elements supporting larger tributary areas. Provided they are not weak in shear, two-way slab systems possess considerable strength reserves that are not usually considered by the designer. The average bias for beams, as computed for the 12 values shown by the markers in Fig. 1 and so weighted towards the smaller influence areas, is 0.94. The average bias for columns, as computed for the 16 values shown by the markers in Fig. 1 and so weighted towards the smaller influence areas, is 0.87. Although larger bias occurs for larger influence areas, the large dead load that the column must sustain in this case makes the design relatively insensitive to the magnitude of the live load. Figure 1 also indicates that the live load reduction specified in the NBCC is conservative with respect to that in ASCE7-98 (Ellingwood 1999). The average ratios of NBCC bias to ASCE7-98 bias, as determined by averaging the points shown by markers in Fig. 1, are 0.95 for beams and 0.84 for columns. Lastly, the bias of 0.90 for use and occupancy load is greater than the value 0.78, which is equivalent to that used to calibrate the 1975 edition of the NBCC (Allen 1975).

The CoV of the maximum equivalent uniformly distributed live load for office floors during a 50 year reference period is also based on the simulation results for office buildings (Ellingwood and Culver 1977). This study reported the variance of the 50 year maximum live load, $\sigma^2_{L_{\text{max}}}$ (in kN/m²), is

### Table 2. Survey of concrete floor thickness variability from 2000.

<table>
<thead>
<tr>
<th>Case</th>
<th>Typical</th>
<th>Worst</th>
<th>Bias</th>
<th>CoV</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case</td>
<td>Min.</td>
<td>Max.</td>
<td>Min.</td>
<td>Max.</td>
<td></td>
</tr>
<tr>
<td>50 mm topping on precast</td>
<td>-5</td>
<td>+10</td>
<td>-10</td>
<td>+20</td>
<td>1.00-1.10, 0.140-0.400 Depends on camber of precast member</td>
</tr>
<tr>
<td>65–75 mm cover slab, steel deck</td>
<td>-5</td>
<td>+5</td>
<td>—</td>
<td>—</td>
<td>1.00-1.23, 0.077-0.176 Extreme values are for unshored construction</td>
</tr>
<tr>
<td>150 mm one-way slab between beams</td>
<td>-5</td>
<td>+5</td>
<td>-5</td>
<td>+25</td>
<td>1.02-1.06, 0.049-0.078 Slopes tough, especially if sloped in two directions</td>
</tr>
<tr>
<td>200 mm two-way slab</td>
<td>-10</td>
<td>+20</td>
<td>—</td>
<td>—</td>
<td>1.03-1.06, 0.062-0.065 Always have more around the columns; there is a buildup there</td>
</tr>
<tr>
<td>300 mm two-way slab</td>
<td>-10</td>
<td>+10</td>
<td>-10</td>
<td>+20</td>
<td>1.00-1.06, 0.033-0.056</td>
</tr>
</tbody>
</table>

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Thus the CoV of the live load is obtained by dividing the square root of eq. [4] by eq. [2]. The resulting relationship between the CoV and the influence area is represented by the curve marked "load only" in Fig. 2. The CoV values are relatively insensitive to the influence area considered, ranging from 0.162 to 0.182.

The transformation of the live load into a live load effect introduces modelling and analysis factors. The modelling factor addresses the idealization of the actual live load as an equivalent uniformly distributed load and has been assumed unbiased with a CoV of 0.20 (Ellingwood et al. 1980). Kariyawasam (1996) assumed these values pertain to columns, and a reduced CoV of 0.10 pertains to beams. The analysis factor reflects the transformation of load to load effect and has been assumed unbiased with a CoV of 0.05 (Ellingwood et al. 1980). Using these values, the variation of CoV for the 50 year maximum equivalent uniformly distributed load with influence area is plotted in Fig. 2. The CoV values for beams proposed by Kariyawasam are consistently smaller than those for columns. All CoV values are relatively insensitive to the influence area. The CoV values for columns are similar to that assumed by Allen (1975) for calibration of previous editions of the NBCC, if the difference between the 30 year and 50 year reference periods is taken into account.

In past investigations, a Gumbel distribution has been used to represent the total live load effect, including modelling and analysis factors (Ellingwood et al. 1980; Kariyawasam 1996). In the present investigation, the live load was assumed to be a Gumbel variate with a bias of 0.9 and a CoV of 0.17. The combined effect of the modelling and analysis factors has a bias of 1.0 and a CoV of [0.202 + 0.052]1/2 = 0.206 and was assumed to be time-independent. It is further assumed that the overall effect of modelling and analysis is normally distributed, because its uncertainty is dominated by a single factor, the modelling uncertainty, and because in the absence of data or studies concerning the modelling uncertainty, a normal distribution seems as reasonable as any other.

Point-in-time live loads

Statistical analyses of point-in-time live load have been carried out by others (Ellingwood and Culver 1977; Chalk and Corotis 1980; Ellingwood et al. 1980). The load may be modelled as a sustained component, with a duration from 2 to 8 years, and an extraordinary live load that occurs once or twice per year. For the present study, it is assumed that the duration of the point-in-time live load, which includes both the sustained component and the extraordinary live load event, is 6 months.

The point-in-time live load is assumed to have a Weibull distribution, with parameters derived from those of the 50 year maximum live load distribution following the procedures presented by Wen (1990). The bias is 0.273 and the CoV is 0.674. The transformation factors used to convert load into load effect were assumed identical for the 50 year maximum live load and the point-in-time live load.

Wind load

The simplified procedure in the 1995 NBCC adopts a simple “gust factor” approach to estimate wind load on buildings. The wind pressure, \( p \), against the surface of a building is

\[ p = q w \]

where \( q \) is the reference velocity pressure, \( w \) is the exposure factor, \( C_p \) is the external pressure coefficient, and \( C_w \) is the gust factor. Although values for each of these variables are specified in the NBCC, all are uncertain. As described in the next section, the reference velocity pressure depends on the climatic conditions at the site and, assuming these conditions do not change, can be considered as a stochastic process. Thus the distribution of the maximum pressure that will occur over a given design life can be derived from the annual maximum pressure distribution. The various coefficients that transform the reference velocity pressure to a pressure on the surface of a building will be considered time-independent random variables.

Reference wind pressures

In Appendix C of the 1995 NBCC, the 1 in 1 year reference wind pressure, \( q_v \), is determined from the 1 in 1 year wind velocity, \( V_v \), as

\[ q_v = \frac{1}{2} \rho V_v^2 \]

where \( \rho \) is the density of air (1.2929 kg/m³ for dry air at 0°C and standard atmospheric pressure). The density of air was considered deterministic because its CoV, roughly 2.5–5.0% (Ellingwood et al. 1980; MacGregor et al. 1997), is considerably less than the CoV of \( V_v \).

Reference velocity pressures presented in Appendix C of the 1995 NBCC are derived from 1 h wind velocity data from over 100 sites, typically airports, with 10–22 years of record (NBCC 1995). Maps were prepared showing contours of the 1 in 30 year wind velocity and dispersion parameters for Gumbel distributions fit to the data. For the locations
listed in Appendix C, which represent the centres of urban areas and so generally differ from the sites where data were collected. In 30 year wind velocities and dispersion parameters were obtained from the maps by interpolation. Then 1 in 10 year and 1 in 100 year velocities were determined for each site, from which 1 in 10, 1 in 30, and 1 in 100 year reference pressures were computed using eq. [6]. Thus the reference pressures listed in Appendix C of the NBCC generally do not exactly fit a Gumbel distribution, though it is possible to back-calculate wind velocities from the published reference pressures that do. There is no documentation available concerning the factors used to transform the velocities and dispersion parameters from the data collection sites to the locations listed in Appendix C. Generally the CoVs of maximum annual wind velocities back-calculated from the wind pressures in Appendix C seem greater than the corresponding CoVs for nearby sites where data were collected.

The present investigation considered the wind velocity, not the wind pressure, as a random variable with a Gumbel distribution to be consistent with the methodology and assumptions adopted to compute the NBCC reference pressures. The ratio of the mean maximum velocity expected in a 50 year design life, $V_{50}$, to the specified 1 in 50 year velocity, $V_{50}$, depends on only the CoV of the maximum annual wind velocity, $\text{CoV}_a$, and is

$$\frac{V_{50}}{V_{50}} = 1 + 3.050 \text{CoV}_a + 2.592 \text{CoV}_a^2$$

Annual maximum wind velocity data were provided for 311 sites by the Engineering Climatology Section of the Canadian Meteorological Centre in Downsview, Ontario, of which 223 had at least 10 years of record. The CoV of the maximum annual wind velocity varies as shown in Fig. 3. The horizontal axis in Fig. 3 is the difference in degrees between the longitude of the site and the Yukon–Alaska border, so the points on the left-hand side of the figure correspond to sites in Yukon and British Columbia and the points on the right-hand side correspond to sites in the Atlantic provinces. The $\text{CoV}_a$ values range from 0.028 (Old Glory Mountain, British Columbia) to a maximum of 0.394 (Squamish Airport, British Columbia), with an overall mean value of 0.134. For the 223 sites with at least 10 years of record, $\text{CoV}_a$ ranges from 0.055 (Vancouver Harbour, British Columbia) to 0.230 (Iqaluit, Nunavut), and the overall mean is 0.135. For these 223 sites, 90% of the $\text{CoV}_a$ values are within the range 0.087–0.183. There is a marked tendency for the standard deviation of the maximum annual wind velocity to increase as the mean value of the maximum annual wind velocity increases, as shown in Fig. 4. A linear regression line forced through the origin has a slope, which corresponds to the $\text{CoV}_a$ of 0.135 as shown.

Figure 5 shows the relationship between the reference wind pressures for various return periods and the CoV of the maximum annual velocity distribution. The reference wind pressures have been normalized using the 1 in 30 year wind pressure. Figure 5 can be used to estimate the wind load factors for specified 1 in 50 year reference wind pressures that give factored wind loads identical to those in the 1995 NBCC, where the load factor of 1.5 is applied to specified 1 in 30 year reference wind pressures. For the average $\text{CoV}_a$ of 0.135, the load factor applied to the 1 in 50 year reference wind pressure is 1.38, and for the $\text{CoV}_a$ limits of 0.087 to 0.183 it corresponds to 1.35 and 1.42, respectively. If the load factor selected for the 1 in 50 year specified wind press-
sure exceeds these values, and the resistance factors are unchanged, designs to the 2005 NBCC will require a greater nominal resistance than designs to the 1995 NBCC.

Figure 5 can also be used to estimate the importance factor for wind load for post-disaster buildings, which in the 1995 NBCC are designed for the 1 in 100 year wind load. These provisions are equivalent to an importance factor that ranges from 1.14 to 1.26, with a mean value of 1.21.

Calibration for wind load was carried out for Regina, Rivière du Loup, and Halifax. The annual maximum wind velocity was treated as a Gumbel variate with bias computed using eq. [7] and CoVq values derived from the Canadian Meteorological Centre data. These values are summarized in Table 3.

**Transformation factors**

As implied by eq. [5], the reference velocity pressure must be transformed using gust, exposure, and pressure coefficients to obtain the pressure applied to the surface of a building. Given that the transformation is represented as a product of independent random variables, the Central Limit Theorem implies that the overall transformation factor has a log-normal distribution. It was further assumed that the transformation factor is time independent. The current literature presents a considerable range of statistical parameters for these factors; rationale for the particular values selected is summarized briefly in the following paragraphs.

The literature pertaining to the wind load approach in ASCE7-98 (ASCE 2000) has limited relevance because some factors differ markedly from those in the NBCC.

Statistical parameters for the exposure coefficient, $C_{ex}$, and the combined gust and pressure coefficients, $C_gC_p$, reported by others are presented in Table 4. The $C_gC_p$ parameters adopted for the current investigation are consistent with those proposed by Davenport (1982, 2000). The $C_gC_p$ bias adopted is conservative with respect to values proposed for low (Davenport 1982) or tall buildings (Davenport 2000), and the CoV adopted is at the higher end of the range of values reported by others.

ASCE7-98 (ASCE 2000) specifies a factor of 0.85 for directionality that was neglected in the present investigation because the gust and pressure coefficients used for low building design in the NBCC have been reduced to account for directionality and other factors. Also, the pressure coefficients in the NBCC for taller buildings are less severe than those in ASCE7-98 because they permit the designer to compute the leeward suction at the mid-height of the building, not at the top.

Thus the overall bias coefficient accounting for exposure, gust, and pressure coefficients, ignoring wind directionality, is $0.80 \times 0.85 = 0.68$, with a CoV of 0.219, say 0.22. As shown in Table 4, these values are similar to those proposed by Ellingwood and Tekie (1999) and are comparable to values proposed by Davenport (1981, 1982).

**Point-in-time wind velocities**

Point-in-time wind loads were derived assuming that the wind velocity (and therefore pressure) is a stochastic process and the factor that transforms the reference pressure to a load against the building surface is a time-independent random variable. The duration of each wind pulse was assumed to be 3 h, so the annual number of pulses is $365.25 \times 24/3 = 2922$. The central tendency and dispersion parameters for the point-in-time distributions are summarized in Table 3.

**Snow load**

The actual snow load on a roof is the difference between the quantity of snow or rain that has accumulated and the quantity that has been removed by wind, melting, or evaporation (Ellingwood and O’Rourke 1985), but data are not readily available to quantify snow load in this manner. Instead, the model used for design, which expresses the snow load on the roof of a structure, $S$, as a snow component and a rain component, was adopted:

\[ S = (C_hC_aC_wS_h) + S_r \]

where $S_h$ is the ground snow load, $S_r$ is the associated rain load, $C_h$ is the basic roof snow load factor and equals 0.8, $C_w$ is the wind exposure factor, $C_a$ is the slope factor, and $C_g$ is the accumulation factor. The transformation factor, $C_{gr}$, that converts the ground snow load at a given site to an appropriate roof snow load is

\[ C_{gr} = C_hC_aC_gC_r \]

According to the 1995 NBCC, $S_r$ need not be taken greater than $C_{gr}S_h$.

**Ground snow load**

The ground snow load, $S_h$, is determined in Appendix C of the 1995 NBCC as the product of the depth of snow, $d$, and the unit weight of snow, $\gamma$. These values were computed (NBCC 1995; Newark et al. 1989) by first fitting Gumbel distributions to maximum annual accumulated snow depth data for 1618 stations with 7–38 years of record and calculating 1 in 30 year values. Then snow densities were determined, for various geographical regions with common climatic features as manifested by forest type, that averaged 2.01 kN/m³ east of the continental divide, 2.55–4.21 kN/m³ west of the continental divide, and 2.94 kN/m³ above the treeline in the Northwest Territories and Nunavut. The loads were normalized to account for the site elevation, assuming a linear variation of load above sea level, and smoothed contour maps were prepared. The final loads corresponding to the various geographical locations in Appendix C were derived by interpolation and rectified to account for elevation. There is no documentation available concerning the interpolation weighting factors used to transform the ground snow load values from the data collection sites to the locations listed in Appendix C.

The present investigation considered the maximum annual snow accumulation depth to have a Gumbel distribution, for consistency with the basis for computation of the specified

---

**Table 3. Statistical parameters for maximum wind velocity in 50 year and 3 h periods.**

<table>
<thead>
<tr>
<th>Site</th>
<th>Maximum 50 year</th>
<th>Maximum 3 h</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CoV</td>
<td>Bias</td>
</tr>
<tr>
<td>Regina</td>
<td>0.108</td>
<td>1.039</td>
</tr>
<tr>
<td>Rivière du Loup</td>
<td>0.170</td>
<td>1.054</td>
</tr>
<tr>
<td>Halifax</td>
<td>0.150</td>
<td>1.049</td>
</tr>
</tbody>
</table>

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values in Appendix C of the 1995 NBCC. Annual maximum snow depth data were provided for 1278 data-collection sites by the Engineering Climatology Section of the Canadian Meteorological Centre. The CoV of the annual maximum depth ranges from 0.08 to 1.34, with a mean of 0.491 and a median of 0.47. Approximately 95% of the CoV values lie between 0.22 and 0.95. Statistical parameters for maximum snow depth in a 50 year reference period are summarized in Table 5. The ratio of the mean value of the maximum depth in a 50 year reference period to the 1 in 50 year specified snow depths were computed using eq. [7]. The ranges of bias coefficients and CoV values for the normalized maximum 50 year distribution are quite small, so the values shown in the last row of Table 5 were adopted.

Snow densities used to derive the 1 in 30 year values specified in the 1995 NBCC were assigned based on the type of forest in the region of the site. Kariyawasam (1996) has summarized statistical parameters for snow density, $\rho$, from Newark (1984), as reproduced in Table 6. In the current investigation snow densities were assumed normally distributed with a bias of 1.0 and a CoV of 0.17. The CoV value adopted is the average for all regions except tundra and taiga, which are not densely populated.

### Associated rain load
The associated rain load represents the weight of rain falling on snowpack, and values for Appendix C of the 1995 NBCC have been derived based on historical data for winter

---

**Table 4.** Statistical parameters for the exposure coefficient, combined gust and pressure coefficients, and overall wind load transformation factor reported by others.

<table>
<thead>
<tr>
<th>Source</th>
<th>Bias</th>
<th>CoV</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposure coefficient, $C_e$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Allen 1975</td>
<td>—</td>
<td>—</td>
<td>Bias = 0.85; CoV = 0.13 assumed for combination of $C_gC_p$ and modelling</td>
</tr>
<tr>
<td>Ellingwood et al. 1980</td>
<td>1.0</td>
<td>0.16</td>
<td>Exposure factor different from NBCC</td>
</tr>
<tr>
<td>Davenport 1981</td>
<td>1.0</td>
<td>0.10</td>
<td>As reported by Kariyawasam 1996</td>
</tr>
<tr>
<td>Davenport 1982</td>
<td>0.80</td>
<td>0.16</td>
<td>Low buildings</td>
</tr>
<tr>
<td>Ellingwood and Tekie 1999</td>
<td>0.93</td>
<td>0.129</td>
<td>Low building</td>
</tr>
<tr>
<td>Davenport 2000</td>
<td>0.80</td>
<td>0.20</td>
<td>For simplified method; tall building</td>
</tr>
<tr>
<td>Current investigation</td>
<td>0.80</td>
<td>0.16</td>
<td></td>
</tr>
</tbody>
</table>

| Gust coefficient and pressure coefficient, $C_gC_p$ | | | |
| Allen 1975 | 1.0 | 0.10 | Pressure coefficient only; model error factor of 0.85 included separately |
| Ellingwood et al. 1980 | 1.0 | 0.11 | Gust |
| | 1.0 | 0.12 | Pressure |
| | 1.0 | 0.16 | Combined |
| Davenport 1981 (as reported by Kariyawasam (1996)) | 1.0 | 0.05 | Gust |
| | 1.0 | 0.10 | Pressure |
| | 1.0 | 0.11 | Combined |
| Davenport 1982 | 0.80 | 0.15 | For low buildings |
| Ellingwood and Tekie 1999 | 0.97 | 0.093 | Gust |
| | 0.88 | 0.067 | Pressure (same nominal values as NBCC) |
| | 0.85 | 0.115 | Combined |
| Davenport 2000 | 0.80 | 0.21 | For simplified method; tall building |
| Current investigation | 0.85 | 0.15 | |

| Overall wind load transformation factor | | | |
| Allen 1975 | 0.72 | 0.174 | Basis for calibration of NBCC |
| Ellingwood et al. 1980 | 0.85 | 0.239 | Not specifically for low buildings (as reported by Kariyawasam (1996)) |
| Davenport 1981 | 0.85 | 0.150 | Low buildings; only external pressures |
| Davenport 1982 | 0.54 | 0.325 | For low buildings, high-wind regions |
| Rosowsky and Cheng 1999 | 0.73 | 0.264 | Not specifically for low buildings; all factors have normal distributions |
| Ellingwood and Tekie 1999 | 0.70 | 0.208 | Simple method |
| | 0.57 | 0.250 | Detailed method |
| | 0.71 | 0.150 | Wind tunnel and meteorological study |
| Davenport 2000 | 1.00 | 0.087 | |
| Current investigation | 0.68 | 0.22 | Log-normal |

**Table 5.** Statistical parameters for maximum snow depth in 50 year period.

<table>
<thead>
<tr>
<th></th>
<th>CoV</th>
<th>Bias</th>
<th>CoV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower 3% fractile</td>
<td>0.220</td>
<td>1.064</td>
<td>0.132</td>
</tr>
<tr>
<td>Mean</td>
<td>0.491</td>
<td>1.099</td>
<td>0.197</td>
</tr>
<tr>
<td>Upper 3% fractile</td>
<td>0.953</td>
<td>1.126</td>
<td>0.244</td>
</tr>
<tr>
<td>Current investigation</td>
<td>1.10</td>
<td>0.20</td>
<td></td>
</tr>
</tbody>
</table>

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rainfall (Taylor and Allen 2000). Subjectivity was required in the process of sketching 0.1 kPa isolines on maps (Newark et al. 1989). The difficulty of deriving statistical parameters for this load is further exacerbated because the necessary data are not readily available and, even if they were, would require careful filtering to eliminate winter rainfall on nonexistent snowpacks. It was therefore assumed that probability distributions for the roof snow load include any further bias and uncertainty due to the associated rain load.

Ground-to-roof transformation factor

Although there is a significant body of literature concerning the ground-to-roof snow load transformation factor, few studies provide results in a format that is compatible with code calibration. Ellingwood and O’Rourke (1985) cite the following relationship, from O’Rourke et al. (1982):

\[ C_{gr} = 0.47 E T_e \]

where \( E \) is a wind exposure factor ranging from 0.9 to 1.3; \( T \) is a thermal characteristic factor ranging from 1.0 to 1.2; and \( \varepsilon \) is the error term, which has a log-normal distribution with a mean value of 1.0 and a CoV of 0.44. The transformation factor computed using eq. [10] ranges from 0.53 to 0.91 times the basic roof snow load factor of 0.8 specified in the 1995 NBCC.

For the present investigation, parameters were adopted from a recent study by Taylor and Allen (2000) that are consistent with the definition of the ground-to-roof snow load factor as defined in eq. [8]. Various results, summarized in Table 7, consider the ratio of the maximum measured roof snow load to the corresponding maximum ground snow load as converted to an equivalent roof load using the NBCC criteria. The statistics are based on 13 years of data for 112 roofs in four Canadian cities. The bias for data obtained in Halifax, Chicoutimi, and Ottawa and the drift location in Saskatoon tend to fall within the ranges suggested by Ellingwood and O’Rourke (1985). The CoV values are also similar.

Point-in-time snow load

Point-in-time snow load parameters were derived assuming that the snow accumulation is a stochastic process and the factors that transform the depth to a load and the ground snow load to a roof snow load are time-independent random variables. It was assumed that the point-in-time pulse has a magnitude that follows the Weibull distribution and a duration of 14 days during only 3 months of the year, so there are six events per year. The central tendency and dispersion for the point-in-time distribution represent a bias with respect to the specified 1 in 50 year snow depth of 0.196 and a CoV of 0.882.

Summary

This paper summarizes statistical parameters for dead load, live load due to use and occupancy, snow load, and wind load that have been adopted for calibration of load and load combination criteria for the 2005 National Building Code of Canada (NBCC). Table 8 summarizes the bias and CoV values and distribution types adopted for the calibration presented in a companion paper. A new survey of typical construction tolerances indicates that statistical parameters for dead load presented by Ellingwood et al. (1980) are ade-

---

**Table 6.** Statistical parameters for snow density (Kariyawasam 1996).

<table>
<thead>
<tr>
<th>Forest region</th>
<th>Mean (kg/m$^3$)</th>
<th>SD (kg/m$^3$)</th>
<th>CoV</th>
<th>Typical sites</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acadian</td>
<td>220</td>
<td>50</td>
<td>0.23</td>
<td>Fredericton, Halifax, St. John’s, Charlottetown</td>
</tr>
<tr>
<td>Aspen grove</td>
<td>220</td>
<td>40</td>
<td>0.18</td>
<td>Edmonton, Saskatoon, Winnipeg, Thunder Bay</td>
</tr>
<tr>
<td>Boreal</td>
<td>190</td>
<td>60</td>
<td>0.32</td>
<td>Northern Prairies, mid-northern Ontario and Quebec</td>
</tr>
<tr>
<td>Coast</td>
<td>430</td>
<td>25</td>
<td>0.06</td>
<td>Coastal British Columbia</td>
</tr>
<tr>
<td>Columbia</td>
<td>360</td>
<td>35</td>
<td>0.10</td>
<td>Southeastern British Columbia</td>
</tr>
<tr>
<td>Great Lakes</td>
<td>220</td>
<td>60</td>
<td>0.27</td>
<td>Southern and central Ontario, southern Quebec</td>
</tr>
<tr>
<td>Montane</td>
<td>260</td>
<td>25</td>
<td>0.10</td>
<td>Interior British Columbia and Yukon</td>
</tr>
<tr>
<td>Prairie</td>
<td>210</td>
<td>40</td>
<td>0.19</td>
<td>Northern Prairies, Regina</td>
</tr>
<tr>
<td>Subalpine</td>
<td>360</td>
<td>30</td>
<td>0.08</td>
<td>Vancouver and Fraser Valley</td>
</tr>
<tr>
<td>Tundra</td>
<td>300</td>
<td>80</td>
<td>0.27</td>
<td>Arctic</td>
</tr>
<tr>
<td>Taiga</td>
<td>200</td>
<td>80</td>
<td>0.40</td>
<td>Subarctic, Yellowknife</td>
</tr>
<tr>
<td>Current investigation</td>
<td>0.17</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Note:** SD, standard deviation.

**Table 7.** Statistical parameters for ground-to-roof transformation factor (Taylor and Allen 2000).

<table>
<thead>
<tr>
<th></th>
<th>n</th>
<th>Bias</th>
<th>CoV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheltered locations</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Halifax</td>
<td>35</td>
<td>0.61</td>
<td>0.45</td>
</tr>
<tr>
<td>Chicoutimi</td>
<td>8</td>
<td>0.71</td>
<td>0.44</td>
</tr>
<tr>
<td>Saskatoon</td>
<td>8</td>
<td>0.30</td>
<td>0.11</td>
</tr>
<tr>
<td>Recommended value</td>
<td></td>
<td>0.60</td>
<td>0.42</td>
</tr>
<tr>
<td>Exposed locations</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Halifax</td>
<td>11</td>
<td>0.47</td>
<td>0.46</td>
</tr>
<tr>
<td>Ottawa</td>
<td>12</td>
<td>0.44</td>
<td>0.41</td>
</tr>
<tr>
<td>Saskatoon</td>
<td>5</td>
<td>0.33</td>
<td>0.22</td>
</tr>
<tr>
<td>Recommended value</td>
<td></td>
<td>0.50</td>
<td>0.42</td>
</tr>
<tr>
<td>Drift locations</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chicoutimi</td>
<td>4</td>
<td>0.86</td>
<td>0.30</td>
</tr>
<tr>
<td>Ottawa</td>
<td>6</td>
<td>0.60</td>
<td>0.38</td>
</tr>
<tr>
<td>Saskatoon</td>
<td>23</td>
<td>0.61</td>
<td>0.43</td>
</tr>
<tr>
<td>Recommended value</td>
<td></td>
<td>0.60</td>
<td>0.42</td>
</tr>
<tr>
<td>Current investigation</td>
<td></td>
<td>0.60</td>
<td>0.42</td>
</tr>
</tbody>
</table>

**Note:** n, number of roofs in the sample.
quantum unless the dead load is dominated by thin, cast-in-place concrete toppings. Statistical parameters for live load due to use and occupancy were derived that pertain specifically to the live reduction factor equation used in the NBCC. Statistics for snow and wind loads were normalized using the 1 in 50 year values that will be specified in the 2005 NBCC. New statistical parameters were determined for the factors that transform wind or snow loads to the force effect because of these loads.

Acknowledgements

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Table 8. Summary of statistical parameters for loads.

<table>
<thead>
<tr>
<th>Load type</th>
<th>Bias</th>
<th>CoV</th>
<th>Distribution type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load</td>
<td>1.050</td>
<td>0.100</td>
<td>Normal</td>
</tr>
<tr>
<td>Use and occupancy live load</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50 year maximum load</td>
<td>0.900</td>
<td>0.170</td>
<td>Gumbel</td>
</tr>
<tr>
<td>Point-in-time load</td>
<td>0.273</td>
<td>0.674</td>
<td>Weibull</td>
</tr>
<tr>
<td>Transformation to load effect</td>
<td>1.000</td>
<td>0.206</td>
<td>Normal</td>
</tr>
<tr>
<td>Wind load (1 in 50 year specified)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50 year maximum velocity</td>
<td>1.039</td>
<td>0.081</td>
<td>Gumbel</td>
</tr>
<tr>
<td>Regina</td>
<td>1.054</td>
<td>0.112</td>
<td>Gumbel</td>
</tr>
<tr>
<td>Rivière du Loup</td>
<td>1.049</td>
<td>0.103</td>
<td>Gumbel</td>
</tr>
<tr>
<td>Halifax</td>
<td>0.156</td>
<td>0.716</td>
<td>Weibull</td>
</tr>
<tr>
<td>Point-in-time velocity</td>
<td>0.064</td>
<td>1.149</td>
<td>Weibull</td>
</tr>
<tr>
<td>Regina</td>
<td>0.084</td>
<td>1.001</td>
<td>Weibull</td>
</tr>
<tr>
<td>Rivière du Loup</td>
<td>0.680</td>
<td>0.220</td>
<td>Log-normal</td>
</tr>
<tr>
<td>Halifax</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transformation to load effect</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Snow load (1 in 50 year specified)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50 year maximum depth</td>
<td>1.100</td>
<td>0.200</td>
<td>Gumbel</td>
</tr>
<tr>
<td>Point-in-time depth</td>
<td>0.196</td>
<td>0.882</td>
<td>Weibull</td>
</tr>
<tr>
<td>Density</td>
<td>1.000</td>
<td>0.170</td>
<td>Normal</td>
</tr>
<tr>
<td>Transformation to load effect</td>
<td>0.600</td>
<td>0.420</td>
<td>Log-normal</td>
</tr>
</tbody>
</table>


List of symbols

- $A_l$ influence area
- $B$ tributary area
- $C_a$ snow accumulation factor
- $C_b$ basic roof snow load factor
- $C_e$ exposure factor for wind load
- $C_g$ gust factor for wind load
- $C_{gr}$ ground-to-roof snow load transformation factor
- $C_w$ wind exposure factor for snow load
- $C_s$ slope factor for snow load
- $C_{wa}$ coefficient of variation of annual maximum load
- $d$ snow depth
- $E$ wind exposure factor
- $L$ live load due to use and occupancy
- $I_{max}$ mean 50 year maximum live load due to use and occupancy
- $n$ number of roofs in the sample
- $p$ wind pressure
- $P$ effect due to permanent load
- $q$ reference velocity pressure
- $q_T$ reference wind pressure with $T$ year return period
- $S$ snow load
- $S_i$ principal transient load
- $S_j$ companion-action transient load
- $S_k$ rain load associated with ground snow load
- $S_g$ ground snow load
- $T$ return period or thermal characteristic factor
- $V_T$ wind velocity with 50 year return period
- $V_{max}$ mean 50 year maximum wind velocity
- $V_{50}$ with 50 year return period wind velocity
- $\alpha$ load factor
- $\alpha_l$ load factor applied to the effects due to the principal transient load
- $\alpha_j$ load factor applied to the companion-action transient loads
- $\alpha_P$ load factor applied to the effects due to permanent load such as dead load
- $\gamma$ unit weight of snow
- $\Delta$ maximum excess thickness of cast-in-place topping
- $\varepsilon$ error term in ground to roof snow load transformation factor
- $\rho$ density
- $\sigma_{lmax}$ variance of 50 year maximum live load due to use and occupancy