INCORPORATION OF THE LRC-METHOD INTO CODIFIED WIND LOAD DISTRIBUTIONS

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ABSTRACT

The recently published ISO-document ISO 4354 Wind Actions on Structures [ISO, 2009] is the first code which recommends using the LRC-method [Kasperski et al., 1992] to develop consistent simultaneous wind load distributions which reproduce the extreme actions (drag, lift, overturning moment) or the extreme action effects (structural responses, e.g. bending moments). The paper briefly summarizes the different design scenarios which have to be covered in a code and presents some results for a simple example structure.

KEYWORDS: DESIGN WIND LOAD, WIND LOAD DISTRIBUTIONS, LRC-METHOD, CODES

Introduction

Wind actions on structures and structural elements have to be considered in the design in the scope of different problems. For the fastening of simple cladding elements, extremes of the local wind induced pressures are required. Basically, these extremes can be specified in dependence of the size of the cladding element. In a simplified approach, the extremes averaged over a larger area can also be used if the cladding elements form individual structural systems, e.g. a continuous beam with two or three fields. A more realistic approach uses effective load distributions. The scenarios for the ultimate limit design contain the loss of equilibrium of the structure, considered as a rigid body. The corresponding global actions are drag, lift and overturning moment. Finally, for the design of the load bearing structural system, effective load distributions are required. The paper presents on the example of a simple block shaped building design values of the aerodynamic coefficients for the local loads and for simultaneous pressure distributions reproducing drag, lift and overturning moment. Finally, the pressure distributions are presented which induce the extreme bending moments in the corner of the frame assuming a hinged support.

Required Statistical Analysis for the full LRC-approach

The LRC-method (load-response-correlation method) requires as basic input the mean values, the standard deviations and the correlations of the contributing pressures. These statistical parameters can be obtained in an appropriately scaled wind tunnel experiment. The effective load distribution for a specific command variable w (global action e.g. draft or structural response e.g. bending moment) is obtained as the mean pressure distribution plus a contribution of the weighted standard deviation as follows:

\[ c_{p_{\text{eff}}}(w) = c_{p_{\text{mean}}} + g_w \cdot \rho_{w p} \cdot c_{p_{\text{sdev}}} \]  

(1)
The weighting coefficient for the contribution of the standard deviation is the correlation between the pressure at position i and the command variable under consideration. It can be obtained from the following equation:

\[ \rho_{w,p_i} = \frac{\sum_j a_j \cdot \rho_{p_i,p_j} \cdot c_{p_j,sdev}}{\left( \sum_i \sum_j a_i \cdot a_j \cdot \rho_{p_i,p_j} \cdot c_{p_i,sdev} \cdot c_{p_j,sdev} \right)^{1/2}} \]  

\[ \sum \text{ai} - \text{weighting coefficient} \]

The weighting coefficients \( a_i \) in equation (2) are either influence coefficients (in case of actions) or structural responses due to unit loads (in case of action effects). The latter are obtained from an appropriate structural analysis.

The peak factor \( g_w \) in the basic equation (1) has to be obtained based on a separate extreme value analysis of the extremes of the respective command variable. The design value of the command variable is specified as:

\[ w_{\text{des}} = w_{\text{mean}} + g_w \cdot w_{\text{sdev}} \]  

\[ w_{\text{mean}} - \text{mean value of the command variable} \]

\[ w_{\text{sdev}} - \text{standard deviation of the command variable} \]

The design value of the command variable is obtained from solving the following implicit problem which demands that the exceedance probability of \( w_{\text{des}} \) has to meet a specified target value:

\[ p(w > w_{\text{des}}) = p_{\text{target}} \]  

The exceedance probability of \( w_{\text{des}} \) is obtained from the following basic convolution integral which combines the probabilities of extreme wind speeds and the probabilities of extremes of the command variable:

\[ p(w > w_{\text{des}}) = \int_{v=0}^{\infty} f_v(v) \cdot \int_{c=c_{\min}}^{\infty} f_c(c) \, dc \, dv \]  

where \( v \) is the wind speed and \( c \) is the aerodynamic coefficient corresponding to the respective command variable \( w \), i.e. \( w \) is obtained as:
\[ w = \frac{1}{2} \cdot \rho \cdot v^2 \cdot c \] (6)

\( \rho \) - air density, usually considered as a deterministic value

\( f_v \) - probability density of extreme wind speeds

\( f_c \) - probability density of extreme aerodynamic coefficients

The lower limit \( c_{\text{lim}} \) in the second integral depends on the design value of the command variable and the actual value of the wind speed in the first integral:

\[ c_{\text{lim}} = \frac{2 \cdot w_{\text{des}}}{\rho \cdot v^2} = \frac{v_{\text{des}}^2 \cdot c_{\text{des}}}{v^2} \] (7)

The second integral can be replaced by the cumulative probability distribution of \( c \) leading to the fundamental equation for specifying the design value of the command variable as follows:

\[ p(w > w_{\text{des}}) = \int_0^\infty f_v(v) \cdot [I - F_c(c_{\text{lim}})] \, dv = p_{\text{target}} \] (8)

with \( F_c \) - cumulative probability distribution of \( c \)

The target value of the exceedance probability can be specified for different classes of structures. The recently published version of ISO 4354 distinguishes four classes:

A - structures with a special post disaster function (hospitals, schools, transmission lines, bridges)

B - buildings which as a whole contain people in crowds (high-rise buildings, stadia, concert halls)

C - normal structures (office buildings, commercial buildings, factories, residential buildings)

D - structures presenting a low degree of hazard to life and other properties (farm buildings, house chimneys, roofing tiles)

The corresponding target values for the ultimate limit state are specified in ISO 4354 with reference to a single year and are summarized in table 1. In a more general approach the target probabilities refer to the projected working life of the structure. The respective values are also given in table 1. Both demands are approximately the same for 50 years projected working life.

Table 1: Target values of the exceedance probability of the design value \( w_{\text{des}} \) of the wind load or wind load effect for the ultimate limit state

<table>
<thead>
<tr>
<th>structural class</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>( p_{\text{target}} / \text{year} [\text{ISO 4354, 2009}] )</td>
<td>1/2000</td>
<td>1/1000</td>
<td>1/500</td>
<td>1/200</td>
</tr>
<tr>
<td>( p_{\text{target}} / \text{working life} [\text{Kasperski, 2009}] )</td>
<td>0.025</td>
<td>0.05</td>
<td>0.10</td>
<td>0.20</td>
</tr>
</tbody>
</table>
Simplified approach as adopted in ISO 4354 (2009)

Solving the convolution integral in equation (8) has to be based on iterations. Strictly speaking, the design value of $w_{\text{des}}$ not only depends on its own statistical properties, but also on the specific statistical properties of the extreme wind speeds. The design value can be expressed as the mean extreme times an adjusting factor as follows:

$$w_{\text{des}} = \hat{w}_{\text{mean}} \cdot c_{\text{ad}}$$  \hspace{1cm} (9)

$\hat{w}_{\text{mean}}$ - mean value of the extremes of $w$

$c_{\text{ad}}$ - adjusting factor

In [Kasperski, 2003], the convolution integral has been solved, specifying the required adjusting factor in equation (9) for a large variety of combinations of the type of extreme value distributions for $v$ and $c$ and the respective variation coefficients. However, ISO 4354 recommends a simplified approach which is close to the approach by Cook, published already 30 years ago [Cook & Mayne, 1978]. The ISO-approach supposes that both extreme values $v$ and $c$ follow the type I extreme value distribution which is given as follows:

$$F(x) = \exp \left[ -\exp \left( -\frac{\pi}{\sqrt{6}} \cdot \frac{x - m}{\sigma} \right) \right]$$  \hspace{1cm} (10)

$m$ - mean value of extremes of $x$

$\sigma$ - standard deviation of extremes of $x$

$\gamma$ - Euler constant = 0.5772

The design value of $v$ is obtained from applying the target probabilities as specified in table 1; for the design value of $c$, the 80%-fractile value of the extremes is used. This value is the rounded version of the Cook-Mayne coefficient, which corresponds to the 78%-fractile. The 80%-fractile value can be obtained by the following relation:

$$c_{\text{des}} := \hat{c}_{80\%} \approx \hat{c}_{\text{mean}} \cdot (1 + 0.7 \cdot \text{cov}(\hat{c}))$$  \hspace{1cm} (11)

with $\text{cov}(\hat{c}) = \frac{\hat{c}_{\text{sdev}}}{\hat{c}_{\text{mean}}}$

$c_{\text{des}}$ - design value of the aerodynamic coefficient

$\hat{c}_{80\%}$ - 80%-fractile of the extreme aerodynamic coefficient

$\hat{c}_{\text{mean}}$ - mean value of the extreme pressure coefficients

$\hat{c}_{\text{sdev}}$ - standard deviation of the extreme aerodynamic coefficients
It is important to note that using a type III distribution for the estimation of the design wind speed or fitting \( v^2 \) to the type I distribution usually requires a higher design value of \( w \) than is provided by the 80%-fractile value [Kasperski, 2009]. Furthermore, the approach is less appropriate if the variation coefficient of the aerodynamic coefficient exceeds 15% to 20% [Kasperski, 2009].

Strictly speaking, the basic demand of the 80% non-exceedance probability refers to the duration of a storm. In Germany, the average duration of storms induced by frontal depressions is 3 hours, however with a clear decrease of the relative intensity for the second and third storm hour [Kasperski, 2002]. Consequently, the design values of aerodynamic coefficients have to be related at least to the reference period of one hour, i.e. the ensembles of extremes sampled in a wind tunnel experiment have to correspond to a sampling time of 3600 s under design wind speed conditions in full scale. The appropriate sampling time in the wind tunnel than is obtained with the time scale as follows:

\[
T_{\text{wt}} = T_{\text{fs}} \cdot \lambda_T
\]

\( T_{\text{fs}} \) - reference duration of the storm in full scale, e.g. 3600 s

\( T_{\text{wt}} \) - required period in the wind tunnel experiment to sample the extremes

\( \lambda_T \) - time scale

The time scale depends on the geometric scale \( \lambda_L \) and the velocity scale \( \lambda_v \) as follows:

\[
\lambda_T = \frac{\lambda_L}{\lambda_v} = \frac{L_{\text{wt}}}{v_{\text{fs}}} = \frac{L_{\text{wt}}}{L_{\text{fs}}} \cdot \frac{v_{\text{wt}}}{v_{\text{fs}}}
\]

\( L_{\text{wt}} \) - characteristic dimension of the wind tunnel model of the tested structure

\( L_{\text{fs}} \) - corresponding dimension of the structure in full scale

\( v_{\text{wt}} \) - mean wind speed at a reference height in the wind tunnel

\( v_{\text{fs}} \) - design wind speed at the corresponding height in full scale

While for a single specific project there is only one specific value of the design wind, for the codification of the wind loads usually a range of design wind speeds has to be covered. If the sampling period in the wind tunnel is shorter than the required one, the sampled extremes are too small, if the sampling period in the wind tunnel is larger than the required one, the sampled extremes are too large. Supposing that the wind tunnel sampling period contains all decisive statistical features of the analysed process, the mismatch of the sampling period can be considered based on the relation of the accumulation of probability in repeated chance experiments:

\[
p_{\text{target, sub-set}} = \frac{T_{\text{sub-set}}}{T_{\text{target}}} \cdot p_{\text{target}}
\]

\( p_{\text{target}} \) - target non-exceedance probability for the appropriate reference period
\( p_{\text{target, sub-set}} \) - target non-exceedance probability in the sub-set

\( T_{\text{target}} \) - appropriate reference period,
  e.g. 1 hour for storms induced by strong frontal depressions

\( T_{\text{sub-set}} \) - duration of the sub-set

With extremes sampled from 10-minute sub-sets, the target sub-set non-exceedance probability in equation (14) becomes 0.963 to meet the final target value of 80\% for one hour. The design value of the aerodynamic pressure coefficients can be estimated with the respective statistical parameters from the sub-sets as follows:

\[
c_{\text{des}} = \hat{c}_{\text{mean, sub-set}} \cdot (1 + 2.1 \cdot \text{cov} (\hat{c}_{\text{sub-set}}))
\]

(15)

with \( \text{cov} (\hat{c}_{\text{sub-set}}) = \frac{\hat{c}_{\text{sdev, sub-set}}}{\hat{c}_{\text{mean, sub-set}}} \)

\( \hat{c}_{\text{mean, sub-set}} \) - mean value of the extreme aerodynamic coefficient from sub-sets

\( \hat{c}_{\text{sdev, sub-set}} \) - standard deviation of the extreme aerodynamic coefficient from sub-sets

Example of Application

For a simple block shaped building with the basic geometry \( h/d/l = 0.6/1/2 \) (h-height, d-span, l-length) a series of wind tunnel experiments has been performed in the wind tunnel of the Department of Civil and Environmental Engineering Sciences at the Ruhr-University Bochum. Pressures have been measured along the centre bay of the building as shown in figure 1. The flow direction has been varied in 10\(^\circ\)-steps from 0\(^\circ\) (wind parallel to the longer axis) to 90\(^\circ\) (wind parallel to the shorter axis). For each flow direction, 100 independent runs have been performed to sample the extremes. The geometrical scale is supposed to be 1:400.

![Figure 1: Basic geometry of the model, position of taps and definition of flow directions](image-url)
In this study, design values are presented for local pressures, for global wind induced actions and for some structural responses. The weighting coefficients for drag and lift are simply the influence lengths for each of the taps. The overturning moment refers to the downwind lower corner of the building for the wind direction 90°, the respective influence factors are the influence length times the lever arm. The drag coefficient is normalized to the height $h$ of the building, the lift coefficient to the span $d$ and the overturning moment to the product $h \cdot d$. All aerodynamic coefficients refer to the mean velocity pressure at building height $h$. It is important to note that codes usually present aerodynamic coefficients which refer to a gust velocity pressure. Therefore, the aerodynamic coefficients presented in this paper can not be directly compared to values presented in codes.

Figure 2 compares the wind tunnel profiles of the mean wind speed and the turbulence intensity to the expected range of values for flat open country using the ESDU-specification [ESDU, 1985 & 1986]. For the wind tunnel boundary layer, a geometric scale of 1:400 is applied. Both wind tunnel profiles are in the respective range. Strictly speaking, the geometric scale also has to be checked for the further parameters describing the turbulent wind fields in the wind tunnel and the full scale, e.g. for the integral length scales which describe the mean dimensions of the eddies in the turbulent flow. The respective value for the along wind component is about 35 cm in the wind tunnel at $h/2$, which corresponds to 140 m in full scale. The German DIN 1055 part 4 [DIN, 2005] specifies for the corresponding parameter a value of about 150 m for flat, open country.

![Figure 2](image.png)

Figure 2: Comparison of the wind tunnel flow applying $\lambda_L = 1:400$ to the range of values as specified in ESDU for flat, open country ($z_0 = 0.03$ m to 0.1 m)

For didactical reasons, ISO 4354 recommends specifying the non-simultaneous design values for the local pressures in tables and presenting the simultaneous design pressure distributions in figures. Table 2 summarizes the extremes of the local pressures along the center bay for the flow direction 90°. Basically, two values - the largest positive and the largest negative value - are specified. As design wind speed, 37.5 m/s at building height $h$ is used. This value corresponds to the hourly mean wind speed.

The raw values in table 2 can be processed further by rounding and/or by summarizing the values into zones. The question arises, how close the further simplified values have to be to the raw design values. In regard to an underestimation, the exceedance probability of $w_{des}(1-\alpha)$ can be used as basic criterion. In figure 3, the trace of the yearly non-exceedance probability is shown for $v$ and $c$ following a type I distribution. The variation coefficient of $v$ is assumed to be 0.125, which is the typical value of the German wind climate. For the variation coefficient of $c$, a value of 0.15 is assumed. The trace is shown in the Gumbel
probability paper and has been normalized to reproduce the design value of 1 for the yearly exceedance probability of 1/1000. Additionally, the exceedance probability of \( w_{des} (1-\alpha) \) is shown for a working life of 50 years. An underestimation of 10% is going to double the exceedance probability in 50 years lifetime, an underestimation of 15% leads to a value which is three times larger than the initial target value. It is worth mentioning that doubling the target value of the exceedance probability is shifting the structure to the next lower structural class. Hence, as basic demand, an accuracy of 0.95 can be specified, leading to a deviation from the target exceedance probability of 40%.

Figure 3: Change of exceedance probability for an underestimation of \( w_{des} \)

Table 2: Design values for the local extremes, flow direction 90°, center bay

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</table>
To develop a simplified approach for the code, in figure 4 the design values of the maximum and minimum local pressures are plotted for the two walls and the roof. Additionally, the allowable underestimations are marked. The basic R-S-T-demand to any code (Right-Simple-Transparent) is difficult to meet in case of the roof pressures using constant values for different zones along the roof. This approach either leads to a large number of zones along the roof or leads to large overestimations if only a few zones are specified. Therefore, a linear approach is proposed. For the negative pressures, there is an almost constant design value for suction close to the leading edge. The depth of this zone is about 0.15 relative length along the roof, the corresponding design value is -4.4. The adjacent zone shows a linear decrease to -1.3 as design value for the trailing edge. This proposal meets the demand that the underestimations remain smaller than 5%. The overestimation in the zone at the leading edge is about 3%, the largest overestimation in the zone with the linear drop is about 20%. For the alternative pressures, the zoning is different. Starting at the leading edge, a small zone with no change of sign is followed by a zone which shows a linear increase of the design value of the positive pressure. Then, the design value of the positive pressure is constant. Finally, it shows a linear decrease until the trailing edge.

The height-dependence of the design values of the pressures on the walls often is tried to be reduced by referring the respective values to the velocity pressures at the actual height instead of using the uniform value of the velocity pressure at the height of the building. This approach is used in the European wind load code [CEN, 2005], which specifies e.g. for the upwind wall three zones for the reference pressure: a bottom and a top zone with a constant value of the reference pressure and an intermediate zone with a reference pressure depending on the height above ground. Strictly speaking, this concept only shifts the basic problem of making things simple from the pressure coefficients to the reference pressure. The total effort of getting the design values of the local wind loads is the same. Therefore, the more flexible approach with a linear run of the design values and a uniform reference pressure is adopted here.

For the upwind wall, it is usual practice to specify a uniform design value. Considering the largest deviation of 0.95 to the unsafe side and using rounded values for the code leads to a design value of 2.4. Strictly speaking, the pressure coefficients indicate that the extreme positive pressures at the upwind wall are influenced by different flow regions. The lower end of the upwind wall is influenced by the horseshoe vortex, which leads to smaller extreme positive pressures. A refined zoning uses 2.0 as design value for the lower 1/3 of the upwind wall. Similarly, for the top end of the upwind wall a reduced value of 1.8 can be used as design value for the local pressures; the corresponding zone has a relative height of 0.15.

Specifying a uniform design value for the downwind wall, as is the usual practice in wind load codes, is less appropriate since the design value at the top end differs considerably from the other values. Obviously, for the investigated geometry, the flow re-attaches on the roof and separates again at the trailing edge, leading to larger local design values at the top end of the downwind wall. A simplified approach specifies a constant value of -0.6 for the bottom zone with a size of 0.4 relative height, followed by a zone with a linear decrease to -1.3 at the top end of the downwind wall.

As shown in table 2, the sign of the local pressures may change at almost any position along the center bay, i.e. although the wind induces on average negative pressures along the roof, there are short periods where positive pressures may occur. The distribution of the corresponding alternative extremes may differ considerably from the distribution of the 'classical' peaks which have the same sign as the respective mean pressures. At least at first sight, this makes things more complicated. As can be seen however in table 3, the final results can still be clearly arranged for the design practice.
Figure 4: Intermediate step to develop simplified design values of local pressures for codification

Table 3: Final table of the design values of local pressures as it is going to appear in the code
flow direction 90°, center bay

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<th>0</th>
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<table>
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<th>min</th>
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upwind wall

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roof
downwind wall
The LRC distributions for the extreme drag, lift and overturning moment are shown in figure 5. Strictly speaking, the pressure coefficients on the roof are not required for the reproduction of the extreme drag. Analogue, the pressures on the walls are irrelevant for reproducing the extreme lift. For sake of simplicity, these two figures can be melted into one figure, presenting the combination of pressures required to reproduce drag and lift. The distribution, which reproduces the extreme overturning moment, lies somewhere between the respective distributions for drag and lift.

The question arises, if a separate distribution has to be specified for the command variable overturning moment. To answer this question, the overturning moment for the LRC distributions for drag and lift and the combined distribution is calculated, which allows evaluating the degree of under- or overestimation, respectively. Additionally, the three global wind actions are estimated based on the envelope of the extreme local pressures. The results are summarized in table 4. The three LRC-loads lead to consistent results, i.e. the command variable shows the largest value for the corresponding LRC-load distribution and smaller values for the two other distributions. The combined distribution, i.e. taking the wall pressures from the drag-distribution and the roof pressures from the lift-distribution, leads to an overturning moment which is about 6% larger than the 'true' design value. Hence, a single distribution is able to reproduce drag, lift and overturning moment with sufficient accuracy.

The basic need of providing in a code more than the information on the local extremes can be identified by comparing the results from the envelope of the local pressures which are marked in table 4 with the heading max/min c\textsubscript{p}. Large to very large overestimations are obtained, i.e. specifying only the local peaks leads to over-conservative and uneconomic design.

![Figure 5: Identified LRC-distributions for drag, lift and overturning moment, flow direction 90°, center bay](image)

(all values with reference to the mean velocity pressure at building height h)

<table>
<thead>
<tr>
<th></th>
<th>LRC drag</th>
<th>LRC lift</th>
<th>LRC o.t.m.</th>
<th>max (c_p)</th>
<th>min (c_p)</th>
<th>comb. drag lift</th>
</tr>
</thead>
<tbody>
<tr>
<td>drag</td>
<td>1</td>
<td>0.777</td>
<td>0.910</td>
<td>1.290</td>
<td>1.059</td>
<td></td>
</tr>
<tr>
<td>lift</td>
<td>0.809</td>
<td>1</td>
<td>0.920</td>
<td>1.722</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>o.t.m.</td>
<td>0.961</td>
<td>0.954</td>
<td>1</td>
<td>1.611</td>
<td>1.059</td>
<td></td>
</tr>
</tbody>
</table>

all values normalized to the respective 'true' design value
Beside local and global wind induced actions, structural responses have to be considered when preparing the final codal provisions. A typical example of the load bearing structure for the investigated block shaped building is a steel portal frame. The structural responses to be analyzed are the bending moments in the frame and the support reactions. The influence coefficients required for the LRC-method strongly depend on the structural system (hinged support, fixed support, 3-pin-jointed frame) and the ratio of the stiffness of the columns and the rafter. In this example, only one case is studied, which is a frame with a hinged support and same stiffness for column and rafter. In figure 6, the influence lines for the two support reactions and the bending moment in the frame's corner are shown.

![Influence lines](image.png)

**Figure 6**: Influence lines for the two support reactions and the bending moment in the corner of the frame, all values per unit load and per m distance of the frames, all loads normal to the respective surface

The influence lines for the vertical support reaction are the same as for the overturning moment, i.e. the vertical support reaction is not a further decisive command variable. However, the horizontal support reaction and the bending moments in the frame's corners are three typical examples of decisive command variables.
The influence line for the bending moment in figure 6 can be used to explain the problem of favorable load effects. If the wind is coming from the left, pressures on the upwind wall generally will be positive. On the roof and on the downwind wall, pressures generally will be negative. For the upwind bending moment, the load effects (product of pressures times amplitude of the influence line) from these three sub-areas have all the same sign, i.e. an over-estimation of the roof suction leads to an estimation of the upwind bending moment on the safe side. For the downwind bending moment, however, the sign of the product pressure times amplitude of the influence line changes its sign. The higher the suction on the roof, the smaller will be the absolute value of the negative bending moment, i.e. the load contributions are favorable for the downwind bending moment. Overestimating the suction on the roof leads to unsafe values for the downwind bending moment.

In figure 7, the LRC-load distributions are shown for the upwind and downwind corner bending moment. Additionally in table 5, the structural responses are summarized for different LRC-load distributions. All results are normalized by the respective extreme response. The combined load distribution, which reproduces exactly drag and lift and gives a fair estimation for the overturning moment also works for the horizontal support reaction (deviation less than 2%) and for the upwind bending moment (deviation only 7.4% to the safe side). However, the downwind bending moment is underestimated by more than 20%, i.e. a second load distribution is required for the code. The universal equivalent static wind load as stated in [Katsumura, Tamura, Nakamura 2007] does not exist.

Table 5: Comparison of the structural responses due to different load distributions

<table>
<thead>
<tr>
<th>LRC</th>
<th>drag</th>
<th>lift</th>
<th>o.t.m.</th>
<th>( M_{up} )</th>
<th>( M_{down} )</th>
<th>( H )</th>
<th>max/min</th>
<th>comb.</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{up} )</td>
<td>1.000</td>
<td>0.902</td>
<td>0.968</td>
<td>1</td>
<td>0.896</td>
<td>1.002</td>
<td>1.558</td>
<td>1.074</td>
</tr>
<tr>
<td>( M_{down} )</td>
<td>0.898</td>
<td>0.433</td>
<td>0.703</td>
<td>0.782</td>
<td>1</td>
<td>0.855</td>
<td>0.792</td>
<td>0.774</td>
</tr>
<tr>
<td>( H )</td>
<td>0.987</td>
<td>0.803</td>
<td>0.913</td>
<td>0.945</td>
<td>0.903</td>
<td>1</td>
<td>1.304</td>
<td>1.019</td>
</tr>
</tbody>
</table>

Figure 7: Identified LRC-distributions for the bending moments in the corners of the frame, flow direction 90°, center bay
(all values with reference to the mean velocity pressure at building height h)
The final step is to smooth the raw LRC data, which refer to the discrete tapping of the model, to averaged values for sub-areas. To keep the final codal provisions as simple as possible, it is tried to simplify the distributions to four constant pressure values, two for the walls and two for the two halves of the roof. The final load distributions are compared to the LRC-loads for the two bending moments in figure 8. Only the roof pressures are different, which reflects the need to specify for the roof pressures the unfavorable contributions for the upwind bending moment and the favorable contributions for the downwind bending moment. Table 6 summarizes for the different command variables the comparison of the final code values to the respective design values. The identified solution is able to meet the demand to safety and economy in an almost perfect way.

![Figure 8: Final load distributions as they appear in the code compared to the identified LRC-distributions for the bending moments in the corners of the frame, flow direction 90°, center bay (all values with reference to the mean velocity pressure at building height h)](image)

<table>
<thead>
<tr>
<th>command variable</th>
<th>drag</th>
<th>lift</th>
<th>o.t.m.</th>
<th>$M_{up}$</th>
<th>$M_{down}$</th>
<th>H</th>
</tr>
</thead>
<tbody>
<tr>
<td>ratio</td>
<td>0.990</td>
<td>0.967</td>
<td>0.964</td>
<td>1.026</td>
<td>0.998</td>
<td>0.968</td>
</tr>
</tbody>
</table>

**Conclusions**

The LRC method offers the possibility to generate more efficient and reliable wind load codes which are able to consider the demands to safety and economy. Although basically more than one load distribution will be required, for most applications the increase of volume will remain reasonable. Therefore, the recently published ISO 4354 - Wind action on structures recommends the LRC method to specify consistent simultaneous pressure distributions. The worked example is based on a simple block-shaped building. The non-simultaneous design values of the local pressures are presented in tables, the simultaneous pressure distributions in figures.
**References**

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