Forensics of a Partially Collapsed Timber Roof

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Abstract

The roofing of an office building partially collapsed after being in use for almost twelve years. The author was appointed as an independent structural expert to investigate the damage event, by approval of all involved parties, i.e. owner, building insurer and roof constructor. The expert report should give answers to several questions related to forensic engineering, e.g. reasons for the partial failure of the roof elements; influence of the snow load present at the moment of partial collapse; faults or deficits in the structural concept, in element production or execution etc. The paper reports on the course of events and actions, the structural and architectural conception of the roofing, its structural behavior, determination of updated failure loads, associated structural safety assessment and degrees of compliance. The paper gives no conclusion on legal or insurance issues.

Keywords: office building; roofing; snow load; partial collapse; timber; structural forensics; structural modelling; structural detailing; degree of compliance.

1 Introduction

In the middle of winter, the roofing of a smaller office building near Fribourg, Switzerland (approx. 650 m altitude), partially collapsed after being in use for almost 12 years, Figure 1.

1.1 Course of events and actions

The partial collapse was reflected by a sudden local deflection of approx. 40 cm of the inferior roofing face in the north-west corner of the roof, at the first interior column (to the east) of the completely glazed north façade. The deflection at the upper face attained approx. 10 cm. The damage took place very quickly and was accompanied by cracking sounds. The concerned building zone was immediately evacuated and emergency strutting was installed, Figure 1. Additionally, a pump was temporarily installed on the roof for evacuating accruing water.

Figure 1: Partially collapsed roof with emergency strutting in place
The damage event was followed by an on-site inspection by an expert of the building insurer the following day. The snow height was measured as approx. 22 cm, and it was further reported that the snow load was repetitively renewed by the snowfalls of the current month.

About one week later, a further on-site damage inspection by local structural engineers took place, submitting a first expert report about two weeks after. It concluded, in particular, that the partial collapse is due to a structural deficiency of the construction system, and on no account due to the snow load present at partial collapse. This was deduced by comparing the design value of actions according to the code in force at the time of construction to an estimated total load at the time of partial collapse.

The roof constructor refused the conclusion of the first expert report regarding the failure cause (i.e. structural deficiency of the construction system) as, according to him, structural design was carried out conforming to standards. Above all, an underestimation of the considered loads or insufficient structural design was excluded, as this should have led to a collapse much earlier and not after twelve years only. According to him, the partial collapse should rather be related to an accidental and long-term overload in the form of water and high-density ice, further increased through a potential extraneous cause such as seismic vibration or other.

This refusal led to appointing the author as an independent expert, by approval of all involved parties (i.e. owner, building insurance company and roof constructor), to investigate the damage event. The expert report should give answers to several forensic engineering questions:

- Why did the roof structure partially collapse?
- Why did the roof elements fail?
- Is the snow (or any other natural phenomenon) responsible for the damages?
- Are there any faults or deficits in the structural concept, in the roof element production or execution, to be suspected as potential causes for the partial collapse?

These questions were formulated by the lawyer representing the owner. The expert report had to give no answers at all on legal or insurance issues, but be strictly limited to structural aspects. This delimitation also concerned the use of technical terms which may have other legal meanings, e.g. design value, collapse, fault, deficit, damage, failure load etc. A more detailed discussion of this issue, i.e. understanding of technical terms by different professions, unfortunately is beyond the permitted size of this contribution.

The author visited the (still strutted) damage zone about eight months after the event and consulted the drawings of the roof construction. He gave his preliminary approval for installing complementary safety measures or repair measures, respectively, in view of the newly approaching winter season. The experts report was submitted nine months after the damage event.

## 2 Roof structure

### 2.1 Available documents

In addition to personal notes and photos, the expert report was based on different documents, such as the questions to be answered (section 1), appointment approvals of the involved parties, first expert report, response of the roof constructor, and as-built drawings.

### 2.2 Composition of the roofing

The roofing construction is composed as follows, from top to bottom (Figure 2):

- 80 mm humus soil
- 30 mm pea gravel
- Ca. 5 mm tar paper waterproofing in two layers
- 30 mm fir paneling, interconnected by tongue-and-groove and screwed onto battens
- Battens 50 mm wide and 80 mm high, spaced at 717 mm (room for ventilation)
- Prefabricated “Wellsteg” elements (Figure 3), being 300 mm high 2.87 m wide and 8.65 m long. 234 mm high and 4 mm thick corrugated webs are spaced at 167 mm. 33 mm thick timber planks with a width of 167 mm serve as top and bottom flanges of each corrugated web, being interconnected by tongue-and-groove.
At the northern support (where the partial collapse of the roof elements occurred), the upper and lower face sheeting in the “Wellsteg” element (highlighted in Figure 2) are threaded for architectural reasons – in a 220 mm high and 230 mm wide glued laminated timber (glulam) beam ①, also requiring to install a transverse beam ② (100 mm wide and 234 mm high). A planked post construction supports the lower flanges of the roof elements at their southern support.

The inferior flange of the “Wellsteg” element also creates the visible ceiling of the office building. The voids between the corrugated webs are filled well-set with thermal insulation flakes “Isolflac”.

3 Structure safety assessment

3.1 Overall structural behavior

The structural behavior of the roof construction is surprisingly more complex than what would be expected at a first glance. As the flanges of the “Wellsteg” elements are connected by tongue-and-groove (i.e. a connection able to transfer shear but no bending), the girder elements can also transfer loads perpendicular to the corrugated webs (i.e. parallel to the support lines). Hence, the structural system is highly statically indeterminate where the stiffness distribution influences the internal effort distribution (i.e. stiffer elements principally attract higher internal forces).

The governing support of the roof elements is provided by a transverse glulam beam (“BLC 220/230” ① in Figure 2). In comparison to the roof elements, this support cannot be considered rigid but is flexible. The support zones of this beam (i.e. above columns steel ROR, Figure 2) are stiffer than...
the zones between columns. Hence, the support stiffness for the roof elements also varies resulting in a non-uniform support reaction distribution for the “Wellsteg” elements. Similar effects can be observed in steel-concrete slim-floor structures [1]. The corrugated webs in the region of the first interior column are the most loaded ones of the roof elements, thus explaining why the roofing collapsed there (and not elsewhere), section 1.1.

3.2 Governing structural detail

The failure of the “Wellsteg” elements is governed by the – inexistent – connection of the corrugated webs to the transverse beam (serving as segregation of thermal insulation) which itself is connected to the support beam, Figure 4. The webs are not structurally connected to the transverse beam as could be deduced from the spatial distribution of the thermal insulation flakes at the collapse spot (Figure 4, middle).

The “Wellsteg” webs have the task to transfer the loads by shear to the supporting transverse beam at the edge of the elements. The transverse beam, in turn, transfers its loads to the glulam support beam by shear-loaded screws (“tire-fonds” in Figure 2). As webs and transverse beam are not connected, the shear force \( V_d \) in the web must be transferred through the flanges of the “Wellsteg” element (Figure 4, left). In the initial project, the owner explicitly required that no mechanical connectors or other support means are visible at the ceiling.

The interface between web and upper flange is loaded in tension, demanding a stress transfer across the glue-bonded connection between web and flange. The interface between top flange and transverse beam, in turn, is primarily loaded in contact pressure. For the lower interfaces between web and flange, and flange and transverse beam, respectively, the stress signs are inverted.

As the shear force is applied at one side only, both interfaces between flange and transverse beam are loaded eccentrically, associated with further stress increases. Ultimate resistance of the support detail is principally governed by the interfaces loaded in tension perpendicular to the timber grain, i.e. between lower flange and transverse beam or between web and upper flange, respectively, highlighted with dotted lines in Figure 4 (left).

3.3 Derivation of failure loads

The structural system is assumed to be a 8.25 m single span beam, thus supposing the northern support of the “Wellsteg” elements at the inner lateral face of the transverse beam (Figure 2). It is also presumed that all corrugated webs are equally loaded, i.e. neglecting a non-uniform distribution of support reaction (section 3.1), to avoid more complex structural modelling at this stage of assessment.

It is further assumed that the total shear force at the support is equally shared between upper and lower flange.
lower flange of the “Wellsteg” element, as stiffness of both flanges are equal. The evaluations shown hereafter confirm that these simplifications result in acceptable precision for realistically estimating partial collapse loads.

For the estimation of failure loads, the interface between lower flange and transverse beam is considered to be governing as it is loaded in tension and bending moments from the eccentrically applied shear force (section 3.2). Inspection results confirm the failure in tension perpendicular to the grain (Figure 4, right).

Determination of failure loads is based on linear-elastic stress analysis of the rectangular interface cross-section. The uniformly distributed failure load \( q_u \) of the roof elements is

\[
q_u = \frac{4 \cdot f_{t,90} \cdot 1}{L} \cdot \frac{1 + e}{A} \cdot \frac{1}{W}
\]

where \( L = 8.25 \text{ m} \) is the span; \( f_{t,90} \) is the tensile strength perpendicular to the timber grain (Table 1); \( A = 100 \cdot 167 = 167'700 \text{ mm}^2 \) is the interface area; \( e = 50 \text{ mm} \) is the eccentricity of the shear force from the interface axis; \( W = 167 \cdot 100^2 / 6 = 278,3 \cdot 10^3 \text{ mm}^3 \) is the interface’s elastic section modulus; and \( s = 167 \text{ mm} \) is the spacing of the corrugated webs.

Table 1 shows the considered values of \( f_{t,90} \) in the determination of failure loads according to Eq. (1). Note that the design value according to [3] was defined rather conservatively. This applies even more [4] for the admissible stress according to [2].

The associated values may thus be considered lower-bound extremes of \( f_{t,90} \). On the other hand, the tensile strength of a C24 (for deriving failure load 2) rather is an upper-bound value for short-term loading, as derived in laboratory testing [4], and should be further reduced by load duration factors in the forensic assessment.

### 3.4 Considered actions

The assessment considers the permanent and snow loads presented in Table 2. The roof elements were designed according to [6] and [2], both in force at the time. The structural design considered a permanent load of 2.97 kPa, corresponding to 100.5% of the updated code value or 105.2% of the more realistic value, respectively (see footnote in Table 2).

A snow load of 1.53 kPa [6] was initially considered, overestimating the value according to the current code [7] by 5.5% as the roof altitude was conservatively presumed at 680 m. [6] and [7] result in the same design snow load.

The initial structural design considered a total load of 4.50 kPa for verification of Ultimate Limit State (ULS). However, the code [2] for timber structures in force at the time was based on a design concept of admissible stresses, i.e. ULS verifications considered no partial safety factors for loads or resistances, but the factored load according to [6] was simply divided by 1.5 [2]. Subsequently, acting stresses were calculated by linear elasticity theory and compared to the admissible stresses of [2].

For determining design values of actions in buildings, an average partial safety load factor of

### Table 1: Values for tensile strength perpendicular to the timber grain and associated failure loads

<table>
<thead>
<tr>
<th>Reference</th>
<th>( f_{t,90} ) [MPa]</th>
<th>Failure load ( q_u ) [kPa], Eq. (1)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>[2]</td>
<td>0.05</td>
<td>0.61</td>
<td>Admissible stress ( \sigma_{z,\perp} )</td>
</tr>
<tr>
<td>[3]</td>
<td>0.10</td>
<td>1.21</td>
<td>Design value ( f_{t,90,d} )</td>
</tr>
<tr>
<td>Failure load 1 (*)</td>
<td>0.245</td>
<td>2.97</td>
<td>Derived from ( f_{t,90,d} ) [3] (*)</td>
</tr>
<tr>
<td>Failure load 2 (*)</td>
<td>0.576</td>
<td>6.98</td>
<td>Derived from ( f_{t,90,d} = 0.4 \text{ MPa (C24)} ) (*)</td>
</tr>
</tbody>
</table>

(*) considering average material strength (deduced from design or characteristic strength, respectively), admitting conversion and partial safety factors with an overall product of 1.7 [3]

(1) \( f_m = 1.44 \cdot f_t = 1.44 \cdot 1.7 \cdot 2.45 \cdot f_{m} \), assuming 20% COV [4] of log-normal distribution [5]

(2) C24: habitual strength class of structural timber [3]
1.4 could habitually be assumed [6] while more refined determination considered a partial safety factor of 1.3 for permanent loads and of 1.5 for governing live loads. The division by a factor of 1.5 for deriving admissible loads generally results in a structural safety deficit of 6.7% (but in comparison to rather conservative admissible stresses).

### 3.5 Degrees of compliance

Structural safety assessment is based on degrees of compliance \( n \) [8], being defined as the ratio between an updated structural resistance and an updated action. It is thus the inverse of a “unity check” as performed in other countries.

If \( n \geq 1 \), structural resistance equals or exceeds the applied action (i.e. sufficient structural safety). The degree of compliance also immediately reflects the available safety margin or deficit. Depending on the case (i.e. code-conforming verification or collapse load assessment, respectively), design or average values are considered in the results in Table 3.

#### 3.5.1 Code-conforming structural safety

The first two lines of Table 3 present nominal ULS verifications according to codes.

The comparison of the ultimate load determined from admissible stresses according to [2] to design loads according to [6] considers a conversion factor of 1.5 (section 3.4). The related degree of compliance represents the structural safety of the initial roof structure as demanded by codes in force at the time of construction, considering an updated resistance provided by the governing detail (section 3.2).

The degree of compliance from comparing design and ultimate load according to [3], [7] and [9] denotes the structural safety if the roof was constructed identically today. Both degrees of compliance indicate a major structural safety deficit.

#### 3.5.2 Partial collapse loads

Table 2: Considered loads

<table>
<thead>
<tr>
<th>Material</th>
<th>Nominal dimension</th>
<th>Characteristic value</th>
<th>Surface load [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Humus soil</td>
<td>80 mm</td>
<td>21 kN/m³</td>
<td>1.68</td>
</tr>
<tr>
<td>Pea gravel</td>
<td>30 mm</td>
<td>20 kN/m³</td>
<td>0.60</td>
</tr>
<tr>
<td>Waterproofing</td>
<td>2 layers</td>
<td>0.02 kPa/layer</td>
<td>0.04</td>
</tr>
<tr>
<td>Fir paneling</td>
<td>30 mm</td>
<td>5 kN/m³</td>
<td>0.15</td>
</tr>
<tr>
<td>Battens</td>
<td>Section 50/80 @ 717 mm</td>
<td>5 kN/m³</td>
<td>0.03</td>
</tr>
<tr>
<td>“Wellsteg” elements</td>
<td>300 mm</td>
<td>5 kN/m³</td>
<td>0.36</td>
</tr>
<tr>
<td>Thermal insulation</td>
<td>234 mm</td>
<td>0.4 kN/m³</td>
<td>0.09</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material</th>
<th>Nominal dimension</th>
<th>Characteristic value</th>
<th>Surface load [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent load</td>
<td></td>
<td></td>
<td>2.95 [k]</td>
</tr>
<tr>
<td>Design snow load</td>
<td>Alt. 657 m, Swiss midlands</td>
<td>1.81 kPa-0.8</td>
<td>1.45</td>
</tr>
<tr>
<td>Estimated snow load at collapse</td>
<td>22 cm snow height</td>
<td>3.0 kN/m³</td>
<td>0.66</td>
</tr>
</tbody>
</table>

\(^{(*)}\) according to [7]. Realistically, the average density of timber rather amounts to 3.8 kN/m³ [4], resulting in a characteristic value of permanent load of 2.82 kPa.

### Table 3: Degrees of compliance for different cases

<table>
<thead>
<tr>
<th>Case</th>
<th>Perm. load [kPa]</th>
<th>( \gamma_r )</th>
<th>Snow load [kPa]</th>
<th>( \gamma_q )</th>
<th>Total [kPa]</th>
<th>Share of snow load (LRFD level)</th>
<th>Failure load [kPa] (Table 1)</th>
<th>Conversion factor</th>
<th>( n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>[2],[6]</td>
<td>2.95</td>
<td>1.30</td>
<td>1.45</td>
<td>1.5</td>
<td>6.01</td>
<td>0.36</td>
<td>0.61</td>
<td>1.5</td>
<td>0.15</td>
</tr>
<tr>
<td>[3],[7],[9]</td>
<td>1.35</td>
<td>1.45</td>
<td>1.5</td>
<td>6.16</td>
<td>0.35</td>
<td>1.21</td>
<td>0.35</td>
<td>1.0</td>
<td>0.20</td>
</tr>
<tr>
<td>Failure load 1</td>
<td>2.82</td>
<td>1.0</td>
<td>0.66</td>
<td>1.0</td>
<td>3.48</td>
<td>0.19</td>
<td>2.97</td>
<td>1.0</td>
<td>0.85</td>
</tr>
<tr>
<td>Failure load 2</td>
<td>2.82</td>
<td>1.0</td>
<td>0.66</td>
<td>1.0</td>
<td>3.48</td>
<td>0.19</td>
<td>6.98</td>
<td>0.645</td>
<td>1.29</td>
</tr>
</tbody>
</table>

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The last two lines of Table 3 present verifications of effective failure. Failure load 1 considers an average interface resistance, derived from a conservative assumption for the tensile strength perpendicular to the grain (Table 1). The associated collapse load is confirmed with an accuracy of 15%.

Failure load 2 considers a short-term value of tensile strength and should be corrected for short-medium- and long-term load duration [4]. Permanent loads with a duration beyond 10 years require a reduction factor of 0,6 to be applied to the nominal failure load. For snow load, a weighted reduction factor is considered between medium-term loading (i.e. load duration 7 days to 6 months, with a reduction factor 0,8) and short-term loading (i.e. load duration inferior to 7 days, with a reduction factor 0,9). The medium-term snow load is assumed to be the frequent value of the variable action, and the associated value amounts to 0,62 for 657 m altitude [9]. The residual snow load, i.e. 1 – 0,62 = 0,38, is considered as short-term.

Further considering the share of snow load of the total load, these assumptions result in a conversion factor of (1 – 0,19)∙0,6 + 0,19∙(0,62∙0,8 + 0,38∙0,9) = 0,645, applicable to failure load 2, resulting in an overestimation of the partial collapse load by 22%.

The average degree of compliance of 1,07 from failure loads 1 and 2 confirms the collapse load with adequate accuracy. Differences can be attributed to the following:

- The snow load present when the damage occurred may be somewhat underestimated. Assuming a slightly higher density reduces the degrees of compliance.
- The average tensile strength perpendicular to the grain derived from the design value of [3] is rather conservative while assuming a strength class C24 may overestimate it. More detailed evaluations would allow to further approach the upper- and lower-bound results.
- Failure loads were determined with a simplified approach (section 3.3). This may influence the resulting degrees of compliance to the favorable or unfavorable.

4 Conclusions

4.1 Object-related conclusions

The qualitative reflections on the global structural behavior (section 3.1) explain why the roof failed in this area (and not elsewhere).

The structural behavior of the governing structural detail (section 3.2) shows that the interface between lower flange and transverse beam of the “Wellsteg” elements at the northern support governed the partial collapse load.

With regard to the initial questions to be answered by the expert report, the following conclusions can be drawn:

- Why did the roof structure partially collapse?
  - Collapse is principally defined by the excess of bearing capacity. Comparing failure loads and permanent load shows (Table 3) that the latter could be supported during the experienced service life but it already exploited the majority of the structural safety margin; the partial collapse thus is associated to snow loads.
  - The overall structural behavior of the roof combined with a brittle failure from exceeding the tensile strength perpendicular to the timber grain additionally worsens the situation: the webs of the roof elements in the region of the first interior column initially undergo an increased loading, leading to a local failure of the interface. This leads to a load increase on the adjacent webs initiating their failure and so forth, thus explaining the sudden and progressive collapse.

- Why did the roof elements fail?
  - The collapse of the roof elements is due to the failure of the interface between inferior flange and transverse beam of the most loaded corrugated web of a roof element.
  - This initial failure subsequently requires transfer of the whole load through the interface between web and upper flange
(section 3.2). A simplified estimation of stresses shows that the glue bonding between web and upper flange cannot transfer the applied loads, resulting in the collapse of the whole “Wellsteg” element.

- Is the snow (or any other natural phenomenon) responsible for the damages?
  - It is the snow load that finally led to the partial collapse, and it did not attain the value required by codes.
- Are there any faults or deficits in the structural concept, in the roof element production or in execution, to be suspected as potential causes for the partial collapse?
  - General faults in production or in execution of the roof elements could not be identified. The major deficit of the roof structure is associated with the governing detail between corrugated web and transverse beam which is unable to transfer the loads. This deficit is also founded in the owner’s initial request that no mechanical connectors shall be visible at the lower face of the roof elements (section 3.2). If such means could have been applied, they would have considerably increased the robustness of the roof structure and may even have prevented the partial collapse.
  - The screws mounted between upper flange and transverse beam probably prevented the total collapse of the roof as they allowed activating membrane forces in the upper flanges of the “Wellsteg” elements.
  - A connection of the corrugated webs and the transverse beam by bond gluing cannot be produced since the gluing process requires a compacting pressure during hardening. This pressure cannot be supported by the webs due to their corrugated form (wave).

4.2 General and personal conclusions

The questions to be answered were formulated by a legal representative of the owner and reflect a view on a damage case different from the one of structural engineers. This covers, among others, the use of technical terms such as collapse, fault, deficit, failure load etc. In the author’s opinion, an associated delimitation or terminology chapter should be a part of every expert report.

The governing of the critical connection detail for the overall structural performance confirms it (again): the devil hides in the details.

The Ultimate Limit State according to codes should not be confused with the real collapse state. They concern different material failure probabilities (i.e. strength values) and safety margins.

Simplified analytical approaches and considering extreme values of prevailing parameters often already allow determining the correct order of magnitude of the final result.

Current (Swiss) codes principally represent the state-of-the art but they sometimes require more sophisticated interpretation and background knowledge.

5 Acknowledgments

The owner is thankfully acknowledged for giving the author the right to publish the findings of this expert report.

6 References


