Identification of the collapse of the load-bearing facade cladding system of a building

D Papán, Z Papánová and M Ščotka
University of Žilina, Faculty of Civil Engineering, Department of Structural Mechanics and Applied Mathematics, Univerzitná 8215/1, 010 26 Žilina, Slovakia
Email address: daniel.papan@uniza.sk

Abstract. The aim of paper is to show how one structural fail can cause collapse of cladding system. This cladding system was one of most popular for use about 40 years ago for civic amenities buildings. The simplified theoretical approach for the collapse system reason is verified by modern FEM calculation. The results of investigation contain in situ inspections and measures on specimens. The main result of research is to amplify interest to older facades and its maintained and inspections.

1. Introduction
One of the most important long-term indicators in the design and planning for reliable buildings and their building components, as well as their "introduction to life" is maintaining of safety in their long-term use. In the event of non-compliance with technological procedures, due to climate change as well as age, each structure degrades and loses its useful properties and durability. At the time the construction is put into use, we can talk about the original state of the construction and its elements. Any change from this original state, that leads to a reduction in safety or service life, is considered a failure of the building structure or its elements. Accumulation of negative influences and factors may result in a serious disrepair or collapse of the structure. In these extreme cases, there is a general threat to the safety of the building or its main construction components, as well as the people in its vicinity.

One of the monitored structural parts of buildings from the safety point of view is, among other things, the facade cladding system. The paper identifies a critical failure of the construction system of the overhanging facade at the Technical University in Zvolen, which led to its overall collapse and public danger.

2. Facade cladding systems on buildings
Currently the market offers various options for solving the final modification of the facades on individual buildings. In addition to a wide range of modern facade plasters and coatings (acrylic, silicate, silicone, mineral, cement, etc.), contact systems and various types of ventilated facade systems are also available (figure 1).

- Contact facade systems (adhesive) are characterized by gluing of the individual cladding elements directly onto the peripheral construction, masonry, plaster or thermal insulation. The base must be level and the weight or height to which the facing will extend must also be taken into account. Gluing is only possible under certain weather conditions [1].

- Ventilated facade systems (hanging facades) eliminate the shortcomings of adhesive cladding. The cladding is attached to the supporting structure by means of anchors, which always creates an air...
chamber needed to vent the moisture penetrating from the interior. This construction solution protects the facade and the air chamber improves the thermal insulation properties of the wall.

Figure 1. Example of contact and ventilated facade system of peripheral walls.

3. General description of the discussed object and specific facade system
The subject part of the discussed object’s building structure relates to the main building of the Technical University in Zvolen. The main building has an uneven floor plan with different heights of individual parts. A ventilated facade system was used for the side peripheral walls. The approximate age of the examined part of the building structure is 37 years. One of these sidewalls destructed in a short period of time, followed by a collapse. However, degradation processes of steel components did not show signs of excessive corrosion (figure 2).

Figure 2. University in Zvolen and collapse of the side wall facade system.

3.1. Description of the used facade cladding system
In terms of construction and technical solution, it is a self-supporting structure, which consists of an aluminium grate, anchored to hot-rolled profiles "U" 120. These profiles are anchored with mechanical anchors HILTI HSA M12×180 to the reinforced concrete load-bearing wall of the building. The components of the supporting grate have an atypical cross-section and are divided into vertical and horizontal aluminium profiles (figure 3). These components are connected by a screw connection using washers and aluminium extrusions. The single welded joint is implemented by connecting the L-shaped aluminium extrusions to the hot-rolled U 120 profiles to secure the vertical aluminium components. The individual travertine stone parts (facade blocks) are inserted into the aluminium grate. Their weight is approximately 16 kg and there are grooves milled around the perimeter for insertion and protection against falling out.
Vertical aluminum profile  
Horizontal aluminum profile  
Hot-rolled steel profile

**Figure 3.** Supporting components of the grate system for stone cladding.

The cladding was inserted to the height of 2.2 m in the lower part under the shading grate. The distance between two mechanical anchors on one profile is 2.75 m and the overhangs were 0.6 m long. The reconstructed system of hot-rolled profiles from which the dimensions and distances of individual elements were identified is on (figure 4).

![Figure 4. Reconstruction of the load-bearing system of a fallen facade wall.](image)

**Figure 4.** Reconstruction of the load-bearing system of a fallen facade wall.

4. **Critical failure analysis**

The analysis of the accident was greatly simplified by a video recording, which indicated the mechanism of the system failure. This mechanism led to the failure of mechanical anchors, which were attached to the supporting “U” profiles to the existing building on which the facade was suspended. It was clear from the video that the anchors were broken first, which caused the fall of the entire facade system. Due to the impact force from the self-weight of the block, plastic joints began to form on the structure resulting in the formation of a kinematic mechanism, which collapsed unstable into a horizontal position. The consequence of collapse can be seen in (figure 5).
Since there are several facade blocks such as those on the building, the upper parts of the load-bearing system were uncovered on the next block in order to examine the design solution of the not-yet-collapsed construction. After revealing, other failures of the support system were detected (figure 6):

- Mechanical anchor damaged by skidding
- Missing mechanical anchor
- Improper use of mounting washers

One of the reasons for the incorrect design solution was an excessive unevenness of the reinforced concrete wall. In order to eliminate the inaccuracies created between the load-bearing grate of the hanging facade and the existing building, a considerable number of washers were used for mechanical anchors. This solution significantly affects the stress in these anchors and reduces their load-bearing capacity.

4.1. Description of the used facade cladding system

As mentioned in above chapter, the entire collapse was initially caused by the failure of the mechanical anchoring of the supporting grate to the existing building. For this reason, the mechanical anchor itself can be considered a critical element of the structure and the cause of its collapse. In the calculation, only the actual weight of the facade was considered, except for climatic or temperature change loads. The self-load distribution was considered ideal [2], it means uniform for all mechanical anchors. The actual weight of the facade is given in table 1:
Table 1. Self-load of facade elements.

| MEMBER                                | Length [m] | Area [m²] | Number of pieces | Unit mass [kg] | Mass all members [kg] |
|---------------------------------------|------------|-----------|------------------|----------------|----------------------|
| stone facade block (thk. 30.1 mm)     | 79.26      | 67.6      | 5357.976         |                |                      |
| stone facade block (thk. 27.2 mm)     | 79.26      | 59.1      | 4684.266         |                |                      |
| horizontal aluminium profile          | 210        | 1.58      | 331.80           |                |                      |
| vertical aluminium profile            | 56         | 2.93      | 164.08           |                |                      |
| steel washers                         | 108        | 0.61      | 65.88            |                |                      |
| hot-rolled profiles “U 120”           | 35.1       | 13.4      | 470.34           |                |                      |
| connecting material (aprox.)           | 1000       | 0.10      | 100              |                |                      |
| **Sum (block thk. 30.1 mm)**          |            |           |                  |                | **6490.076**         |
| **Sum (block thk. 27.2 mm)**          |            |           |                  |                | **5816.366**         |

As the facade cladding has a large dispersion of thicknesses, the resulting vertical force will be determined for two variants:
- for the minimum measured thickness (ie 27.2 mm corresponding to the weight of the facade block $m_1 = 5816.366$ kg)
- for the maximum measured thickness (ie 30.1 mm corresponding to the weight of the facade block $m_2 = 6490.076$ kg)

These two limit weights, with sufficient reliability, represent the weight of all possible thickness variations that can occur on the facade. The part with a specified weight is suspended on 18 anchors, where the limit state designs of the force on the anchor were determined, i.e. $V_{id (1)} = 3.54$ kN for variants with a stone block thickness of 30.1 mm and $V_{id (2)} = 3.17$ kN for variants with a cladding thickness of 27.2 mm.

After determining the load during collapse, the above-mentioned unsuitable design solution plays a significant role, when the anchor becomes a bracket, strained on the lever arm, which has a significant effect on the anchor’s cross-sectional stress. For this reason, a sensitivity analysis was performed, the parameter of which was the distance between the existing building and the anchored "U" profile at a constant load. Ideally, the anchor should be under shear load, without any free play. It was this ideal case that was the starting point for the calculation of the sensitivity analysis, which the values were compared with at an increasing free play.

4.2. Sensitivity analysis of anchor resistance

The calculation of anchors’ bearing capacity was carried out in accordance with ETAG 001 "METAL ANCHORS USE IN CONCRETE". Annex D – Design methods for anchorages. A schematic illustration of an anchor stressed on a lever arm is shown in figure 7.

![Figure 7. Mechanical anchor strain on the lever arm.](image)
The first case (figure 7a) represents a situation without mounting washers and the second case (figure 7b) a situation when using mounting washers. Sensitivity analysis was performed in both cases, with image b) satisfying the resulting collapse. Computational models representing the use of mounting washers are shown in figure 8.

![Computational models considering mounting washers.](image)

**Figure 8.** Computational models considering mounting washers.

The above-mentioned standard, on the basis of which the calculation of the load-bearing capacity of anchors was carried out, takes into account these mounting washer using the $\alpha_M$ coefficient, also shown in figure 8 for both cases. The total calculation according to the given standard is calculated as follows:

a) Shear load without lever arm [3]:

The characteristic resistance of the anchor in case of steel failure, $V_{Rk,s}$, is given by eq.(1)

$$V_{Rk,s} = 0.5 \cdot A_s \cdot f_{uk} \ [N]$$

b) Shear load with lever arm [3]:

The characteristic resistance of an anchor, $V_{Rk,s}$, is given by equation (2)

$$V_{Rk,s} = \frac{\alpha_M \cdot M_{Rk,s}}{l} \ [N]$$

where

- $\alpha_M$ = see figure 8
- $l$ = lever arm
- $M_{Rk,s}$ = $M_{0,Rk,s}(1 - N_{rd}/N_{Rd,s})$ [Nm]
- $N_{Rd,s}$ = $N_{Rk,s}/\gamma_{Ms}$
- $N_{rd,s}/\gamma_{Ms}$ = to be taken from the relevant ETA
- $M_{0,Rk,s}$ = characteristic bending resistance of an individual anchor.

After substituting specific values into the calculation, the resistance of the anchor was obtained as the ideal state.

$$V_{Rk,s} = 29.5 \text{ kN}$$

In the same way, the values of the resistance of the anchor at an increasing gap were obtained, see table 3.
Table 2. Input parameters of HILTI anchors for bearing capacity calculation.

| MECHANICAL PROPERTIES - Anchor size | M6   | M8   | M10  | M12  | M16  | M18  |
|-------------------------------------|------|------|------|------|------|------|
| Nominal tensile strength $f_{u,thread}$ | HSA, HSA-BW, HSA-F [N/mm$^2$] | 650  | 580  | 650  | 650  | 700  | 650  | 700  |
| Yield strength $f_{y,thread}$        | HSA-R | 650  | 580  | 650  | 600  | 625  |
| Stressed cross-section $A_s$ [mm$^2$] | HSA-R | 20.1  | 36.6  | 58  | 84.3 | 157  | 245  |
| Moment of resistance $W$ [mm$^2$]    | HSA-R | 12.7  | 31.2  | 62.3 | 109.2 | 277.5 | 540.9 |
| Char. bending resistance $H$ [N/m]   | HSA-R | 9.9  | 9.9  | 21.7 | 21.0 | 48.6 | 454.4 |

CHARACTERISTIC RESISTANCE - Anchor size

| Eff. Anchorage dept $h_{ef}$ [mm] | M12 | M16 |
|----------------------------------|-----|-----|
| Tension $N_{Rk}$                 |    |     |
| HSA, HSA-BW HSA-R2, HSA-R       | 17.9 | 26.5 | 35.0 | 26.5 | 36.1 | 50 |
| HSA-F                            | 17.9 | 26.5 | 35.0 | 26.5 | 36.1 | 50 |
| Shear $V_{Rk}$                   |    |     |
| HSA, HSA-BW HSA-R2, HSA-R       | 29.5 | 29.5 | 29.5 | 51  | 51  | 51 |
| HSA-F                            | 29.5 | 29.5 | 29.5 | 51  | 51  | 51 |

DESIGN RESISTANCE - Anchor size

| Eff. Anchorage dept $h_{ef}$ [mm] | M12 | M16 |
|----------------------------------|-----|-----|
| Tension $N_{Rd}$                 |    |     |
| HSA, HSA-BW HSA-R2, HSA-R       | 11.9 | 17.6 | 23.3 | 17.6 | 24.1 | 33.3 |
| HSA-F                            | 11.9 | 17.6 | 23.3 | 17.6 | 24.1 | 33.3 |
| Shear $V_{Rd}$                   |    |     |
| HSA, HSA-BW HSA-R2, HSA-R       | 23.6 | 23.6 | 23.6 | 40.8 | 40.8 | 40.8 |
| HSA-F                            | 23.4 | 23.4 | 23.4 | 45.2 | 45.2 | 45.2 |

Table 3. Load capacity of the anchor strain as a lever arm.

| LEVER ARM [mm] | 1  | 2  | 4  | 8  | 15 | 30 | 50 |
|----------------|----|----|----|----|----|----|----|
| without effective washer $\alpha_{M} = 1.0$ $V_{RR,\alpha}$ [kN] | 9.17 | 4.59 | 2.29 | 1.15 | 0.61 | 0.31 | 0.18 |
| with effective washer $\alpha_{M} = 2.0$ $V_{RR,\alpha}$ [kN] | 18.35 | 9.17 | 4.59 | 2.29 | 1.15 | 0.61 | 0.37 |

4.3. FEM results verification

For FEM results verification was used computing environment “ANSYS Workbench”. Geometrical model of anchor is available on the manufacture’s websites (“HILTI”) [4]. This verification has only principled character, because in computational model was used simplified material model. That is a reason why all result below describe reality only qualitative, but no quantitative. However, principle this simplification sufficiently fulfils the purpose of verification. The calculation was performed in three variants as was the case with the analytical solution. Length of lever arm was 50 mm, pure shear was considered without any space. In all variants was used same finite element mesh density and was used same element type. This is a reason why was possible mutual comparison. Loading and boundary conditions correspond with analytical solution.

Used material model: linear elastic - assumed linear relationship between stress and strain.

Used element type: SOLID186 - a higher order 3-D 20-node solid element that exhibits quadratic displacement behaviour. The element is defined by 20 nodes having three degrees of freedom per node: translations in the nodal x, y, and z directions. [7].
Results from FEM solution are in the following series of figures. Where are on the left side displacements and on the right sides are equivalent stresses due Von Mises theory. Figures 9-11 are in same order as it was in analytical solution: 1. pure shear, 2. lever arm without spacer and 3. lever arm with spacer.

Figure 9. FEM solution of pure shear situation.

Figure 10. FEM solution of situation with lever arm without spacer.

Figure 11. FEM solution of situation with lever arm and spacer.

From figures is the clear that the FEM solution is in principle as well as proportionally similar to the analytical solution from the previous chapters. This solution satisfactorily meets the initial assumptions of the FEM calculation.

Both of solution clearly show, that with increasing gap between anchored object and the substrate the resistance of the anchors rapidly decreases. Spacer partially help against resistance decreasing of anchor, but compared to the ideal shear condition, is it the reduction of resistance still quite large.

5. Conclusions
In general, the building’s facade systems described in the paper are frequently used in Slovak Republic. This contribution obtained in previous chapters shows on real example how can be danger wrong technical realisation. Even when engineering design is not performed unsatisfactorily. Civil engineers in 1983 designed the load-bearing system for facade stone cladding blocks but there were none to check the reality. That was the reason that anchoring did not works properly (ideal inertial forces redistribution). Many researchers and scientific authors are focused in this engineering area, not only main load bearing structures of buildings [8, 9]. That is because the facades, windows, doors, etc. are first stiff barriers for any static and dynamic load from environment. The reason of research investigation published in this paper is to increase interest on this self-load-bearing structures. In TU Zvolen was an accident of real facade system where only one factor “lever arm of anchor” caused collapse after 37 years of structure lifetime. Theoretical approach based on theory of elasticity considered in anchoring system manuals showed that the anchor without lever arm had large reserve of load-bearing capacity. But the real technical connection between steel anchor and reinforced...
concrete wall were performed “with lever arm”. The load bearing capacity of such connection decrease several times rapidly depending on lever arm. This fact is confirmed by modern FEM calculation. Unfortunately, during the lifetime of investigated structure gradually the anchors loses the capacity and the system redistributed forces to stronger (better mounted) anchors. The last stage of redistribution was destruction. Today, no one will know if this fail was on civil structure constructers or accelerated completion due to the communist prestige of the then totalitarian leaders. From this case it is necessary to take the need to pay increased attention to diagnostics and inspections of exposed construction parts. All building’s parts must be safe and maintained because it can also be danger for people moving around. The next process for the investigated building was to unmount the rest of facade parts built same way.

Acknowledgement
We kindly thank to Department of Structural Mechanics and Applied Mathematics for support in research activities.

References
[1] Electronic online source in Slovak https://www.stavebnik.sk/clanky/zateplovanie-obvodovych-stien-zateplovacim-systemom.html
[2] Standard STN EN 1991-1-1 Eurocode 1: Actions on structures. Part 1-1: General actions. Densities, self-weight, imposed loads for buildings
[3] ETAG 001, Edition 1997, Guideline For European Technical Approval Of Metal Anchors For Use In Concrete, European Organisation for Technical Approvals
[4] Hilti HSA EXPANSION ANCHOR, Technical Datasheet
[5] Design of Bonded Anchors, Tech. Report, European Organisation For Technical Approvals
[6] Bathe K J 2006 Finite Element Procedures (Cambridge: MA: Klaus-Jürgen Bathe)
[7] Ansys® Academic Research Mechanical, Release 20.1, Help System, Element Reference, ANSYS, Inc.
[8] Figuli L, Zvaková Z and Bedon C 2017 Design and analysis of blast loaded windows Procedia Engineering
[9] Herbut A, Rybak J 2017 Guidelines and recommendations for vibration control in the case of rapid impulse compaction, Advances And Trends In Eng. Sciences And Technologies II