The Leaning Tower of St Moritz: a structure on a creeping landslide

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ABSTRACT: The 13th century Leaning Tower of St Moritz, is located in the historic center of the famous Swiss ski resort town in the compression zone of a 10 million m^3 Brattas landslide. Over the hundreds of years, this slowly creeping landslide, which is blocked by a rock outcrop below the tower, had damaged the adjacent St. Mauritius church to such an extent that it had to be demolished already in 1893 due to dangerous differential settlements and cracks. The fact that the 33 m tall tower has survived its 5.40 degrees downslope inclination should not be taken for granted: this is an outcome of a century long effort by a number of outstanding Swiss engineers who came up over the years with original stabilization solutions. This chapter explores the history of the Leaning Tower of St. Moritz, describes its stabilization attempts and pays tribute to some extraordinary minds behind them.

1 INTRODUCTION

The historic Leaning Tower of St Moritz (Figure 1) is located 50 m uphill from the Kulm Hotel in the center of the town. This luxury hotel was built in 1856 by the Swiss tourist industry pioneer Johannes Badrutt, mainly for English tourists, whom Badrutt attracted for the first winter holidays by betting that if they did not come back home suntanned, he would pay for their trip. Needless to say, he won the bet and the winter tourism was born (e.g., Margadant and Maier, 1993). In 1879, the first electric bulb in Switzerland was lit in the Kulm Hotel dining hall. It was also the first hotel in Switzerland to boast a telephone, WC, hydraulic lifts and air heating, as well as winter sports such as curling and skeleton. Johannes Badrutt has been widely recognized and celebrated for these achievements, but there has been one more indication of his genius, which went totally unnoticed: namely, the choice of the hotel location, where not a single structural crack has been detected over the entire 160 years of its history. In contrast, if he built it just 50 m uphill from its present location, by today it would have to be demolished and rebuilt at least twice, if the owners still had the courage and finances to do so. All the historic structures above the hotel were subjected to slowly accumulating ground displacements, and eventually had to be demolished. Except one – the Leaning Tower of St. Moritz. This is its story.



Figure 1: The Leaning tower of St Moritz.

2 THE BRATTAS LANDSLIDE

The Brattas-Fullun landslide, which constitutes the major factor for the special geotechnical conditions of the Leaning Tower, is located on the northern slope above the village of St. Moritz (Sterba et al., 2000). It is composed of a 600 m wide clastic flow bounded on both sides by almost parallel shear surfaces (Figure 2, top). The detachment zone is located on the southern edge of the terraced surfaces of the Val Saluver at an altitude of 2400 m a.s.l., and the area stretches over a horizontal distance of 1.5 km to a lower altitude of 600 m with the average inclination of about 20°. The clastic flow consists of two parts (Figure 2, bottom), with some geological evidence of a rock outcrop at the boundary between them. The upper zone, which extends from the detachment zone between Sass Runzöl and Sass da Muottas to the crest at an altitude of approximately 2100 m, is composed of a rockfall with boulders reaching 2-3 m in diameter. The lower 600-700 m long zone, which is the actual Brattas-Fullun landslide, is composed of a 17-23 m thick silty soil matrix with boulder inclusions, whose downhill movement is constrained at its foot by a rock outcrop. The landslide area is partially wooded at the top and has been developed for construction purposes in its lower portion. Until 2006 the movement has only been measured in the developed areas, reaching 5 cm per year above the construction zone and gradually tapering off to zero near the Via Maistra at the lower landslide edge (Figure 3).

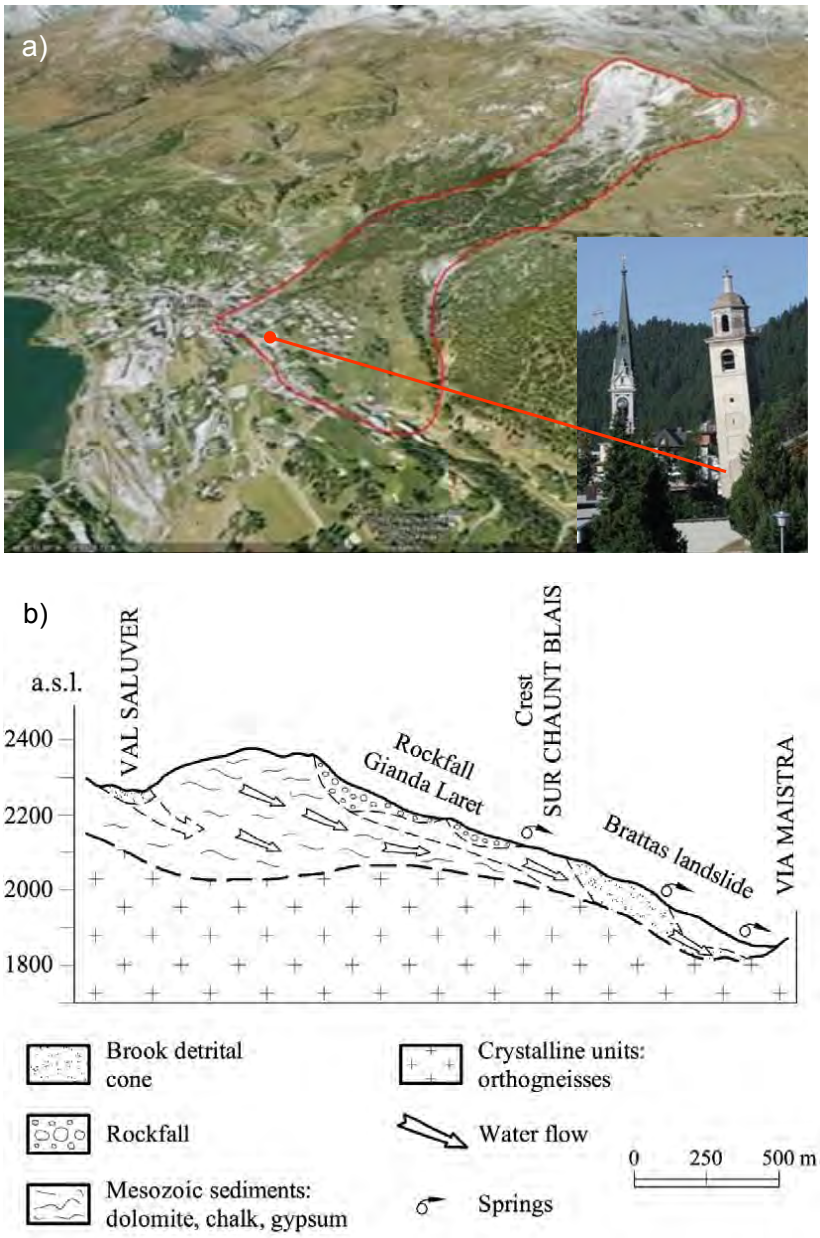


Figure 2: The St. Moritz-Brattas Landslide (top) aerial photo (after Google); (bottom) the geology (Müller and Messina, 1992).

The rockfall event is prehistoric in nature and has been the subject of many investigations by Swiss geologists (e.g., Cornelius, 1912; Heim, 1932; Schlüchter et al., 1982; Schlüchter, 1988; Müller and Messina, 1992; Schlüchter and Helfer, 1991). The large-scale geologic situation of the area resulted from Mesozoic sediments of the Bernina Nappe being pushed over the crystalline rock of the Err Nappe, subjecting all the layers in this region to high tectonic stresses and forming a lithologically very inhomogeneous and strongly weathered weak zone with liassic layers of clay-rich schists, which facilitate formation of slip surfaces.

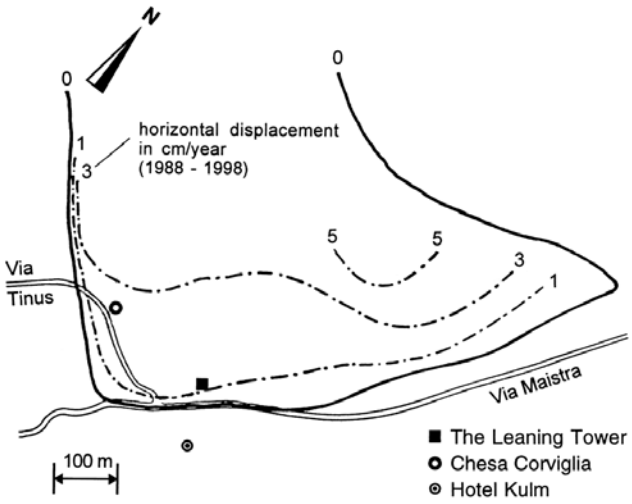


Figure 3: Average horizontal displacement rates over 1988-1998 at the bottom of the landslide (after Tschudi and Angst, 1999).

The hydrological conditions further contributing to the slope instability: the saturation of the slope is due not only to frequent precipitation (average of 1000 mm per year) but also to the permanent seepage from the Schlattain Brook and periodic snow melt from the terraced surfaces of the Val Saluver (Figure 2b), verified through tracer tests by Müller and Messina (1992). Various deep aquifers were observed in the landslide, which create independent water tables in the slope formed by fossil soil layers overlain by later slides. Radiocarbon dating of wood and peat samples (Schlüchter 1988) enabled various movement events in the slope to be identified with the most recent one being approximately 700 years old.

The Leaning Tower is located at the foot of the Brattas slope north of the Kulm Hotel, above the Via Maistra (Figure 3) with its is foundation based in the 15 m thick sliding mass. The underlying rock is highly deformed (Serizite-Muskovite gneiss and schists) and rises towards the Via Maistra to the ground surface, forming a rock ridge against which the creeping mass comes to a halt creating a compression zone with the creep movement of approximately 1 cm per year.

3 THE LEANING TOWER AND THE STABILIZATION OF THE LANDSLIDE (ALBERT HEIM, 1899)

The bell tower of the St. Mauritius church was originally built in Romanesque style in the 13th century, approximately 100 years after the construction of the church (e.g., Haefeli, 1974; Margadant and Maier, 1993). In the course of the following centuries its height was increased, first in the 16th century, accompanied by installation of the chimes and of the clock. The top floor in the late Renaissance style was probably added in the 17th century. Existing records show that in 1797 the tower was displaced by an earthquake, raising concerns about its stability. In 1890, due

to the continuously growing inclination, it was decided to decrease the overturning moment by removing the chimes, which were installed in a specially constructed wooden frame and exhibited in the vicinity of the church. Curiously, while the tower has been inclined almost exactly downslope, the adjacent St. Mauritius church, which was located above the tower, was inclined upslope. Maillart (1931) explained this phenomenon by a possible local heave of the compressed sliding mass between the church and the tower. In 1893, differential displacements of the church reached the level seriously endangering its structural stability and the church was demolished. In 1897 it was also considered to demolish the tower, but thanks to the very good quality of its masonry it was decided to leave the tower in its inclined state (estimated at that time as $2.5° - 3.0°$, Wullimann and Schneller, 1989). Concerned, however, about the continuing displacements of the area and inclination of the tower, the St. Moritz community invited in 1899 Prof. Dr. Albert Heim to look into their potential reasons and to suggest possible mitigation measures.

The choice of the expert was probably not difficult, since Professor Heim (1849-1937) was already at that time one of the top world geologists, considered to be one of the founding fathers of the Engineering Geology. As recent as in 2012, the Joint Technical Committee 1 "Natural Slopes and Landslides" which is a scientific committee created by three of the main societies in the field of Geotechnics: ISSMGE (International Society of Soil Mechanics and Geotechnical Engineering), ISRM (International Society of Rock Mechanics) & IAEG (International Association of Engineering Geology) established an International Lecture dedicated to Albert Heim, that would take place every four years during the International Symposium on Landslides.



Figure 4: Albert Heim and an example of one of his geological profiles (source: ETH – Library).

Albert Heim became full professor of technical and general geology at the ETH Zurich (at that time Polytechnic Institute) in 1872, at the age of twenty-three, which was remarkable even for that time (e.g., Brockmann-Jerosch et al., 1952). Three years later he also became a professor at the University of Zurich and until his retirement in 1911, he gave hundreds of lectures, including "General Geology", "The Geology of Switzerland" and "The Prehistory of Man". His field trips in Switzerland and abroad were legendary among his students and colleagues.

The main focus of Albert Heim's monographic and cartographical works was on the origin of the Alps and on their geology. His innovative use of scientific drawing and photography (Figure 4) for geological documentation has profoundly influenced future generations of geologists. Prof. Heim authored numerous geological reports, e.g., for railway construction projects like the Simplon Tunnel or accidents such as the Vorstadt disaster in Zug (1887). He was involved in various commissions and societies, e.g., as the longstanding President of the Swiss Geological Survey. Beyond his geological interests, he was also an expert in cynology, championing among other breeds the famous Bernese Mountain Dog.

In his expert report of 1899, Prof. Heim established that the main reason behind the continuously increasing displacements of the Leaning Tower of St. Moritz and surrounding structures was a permanent landslide and proposed to reduce its velocity by installing drainage system in the upper part of the landslide. Implementation of these mitigation measures was recognized by Maillart (1931) und Haefeli (1974) as critical for preventing the large-scale deformations and improving the general stability of the slope. It could not, however, stop the displacements completely, as is seen from the results of the continuous tower inclination measurements undertaken starting from 1908 (Figure 5). This is hardly surprising, since the ground water comes into the sliding mass from a very broad area (see tracer tests by Müller und Messina 1992) and cannot be fully drained using a local drainage system. By 1928, inclination of the tower increased to 4.4° (7.7%) and the danger of the eventual tower collapse became imminent (Maillart, 1931).

4 THE 1ST STABILIZATION OF THE TOWER (ROBERT MAILLART, 1928)

In 1927, the concerned municipality invited Engineer Robert Maillart to explore the possibility of the tower stabilization. Robert Maillart (1872 –1940) was a famous Swiss civil engineer and bridge designer. His ground-breaking use of structural reinforced concrete radically changed both the architecture and engineering of bridge construction (e.g., Marti, 1996). In 1991, his Salginatobel Bridge built in 1930 (Figure 6) was declared an International Historic Civil Engineering Landmark by the American Society of Civil Engineers. In 2001, the British journal *Bridge – Design and Engineering* voted Salginatobel Bridge as "the most beautiful bridge of the

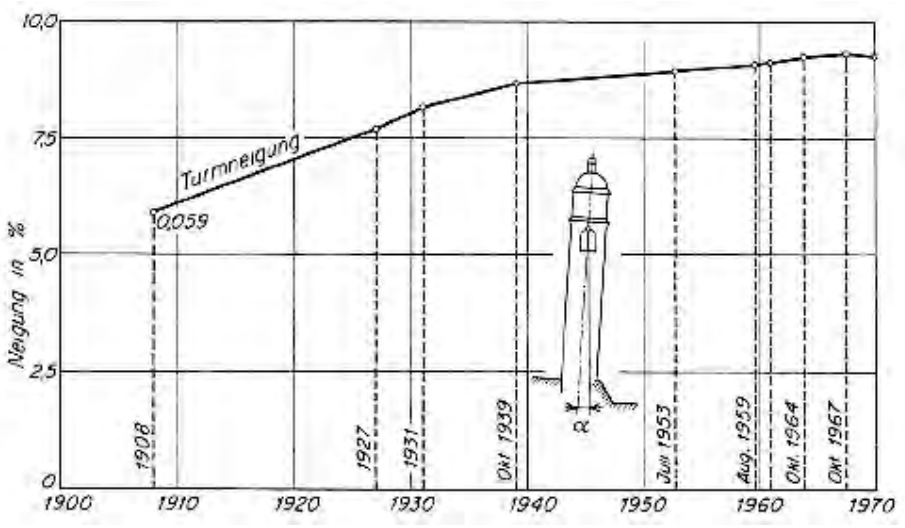


Figure 5: Inclination of the tower in % between 1908 and 1967 (after Haefeli, 1974) .

century". Malliart's designs of the beamless floor slabs and mushroom ceilings for industrial buildings (Figure 6) are another example of his artistic and engineering ingenuity.

Robert Maillart completed his Civil Engineering studies in 1894 at the ETH Zurich, and after working for a few years in industry, established in 1902 his own company *Maillart & Cie*. The company was engaged in large-scale projects in Switzerland and Europe and eventually brought him and his family in 1912 to Russia. Tragically, he was trapped there during the WWI and the revolution, his wife died and his company was nationalized. Escaping in 1918 back to Switzerland as a bankrupt widower with three children, he mastered, however, a remarkable comeback leading to his best designs and projects.

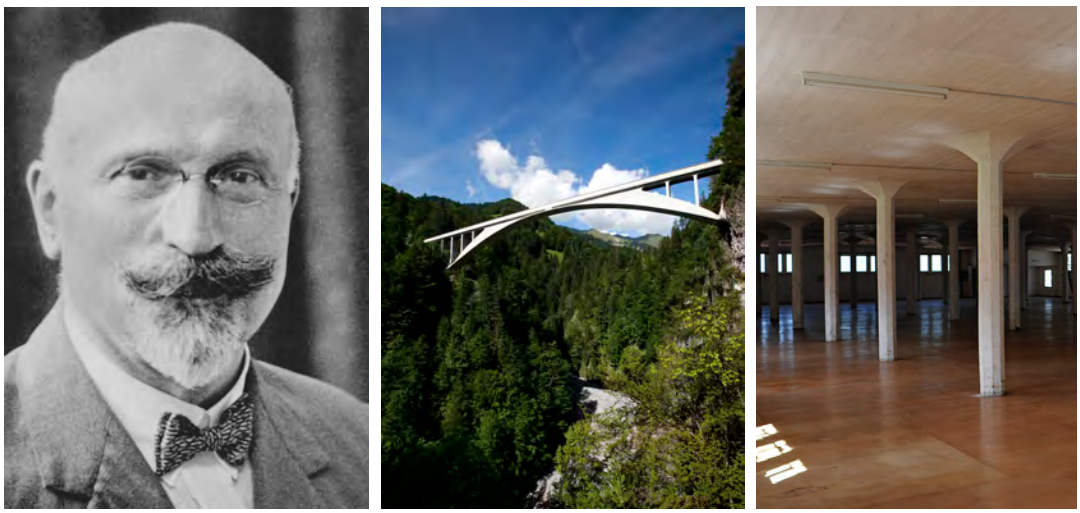


Figure 6: Robert Maillart, his Salginatobel bridge and the mushroom slab (source: Wikipedia).

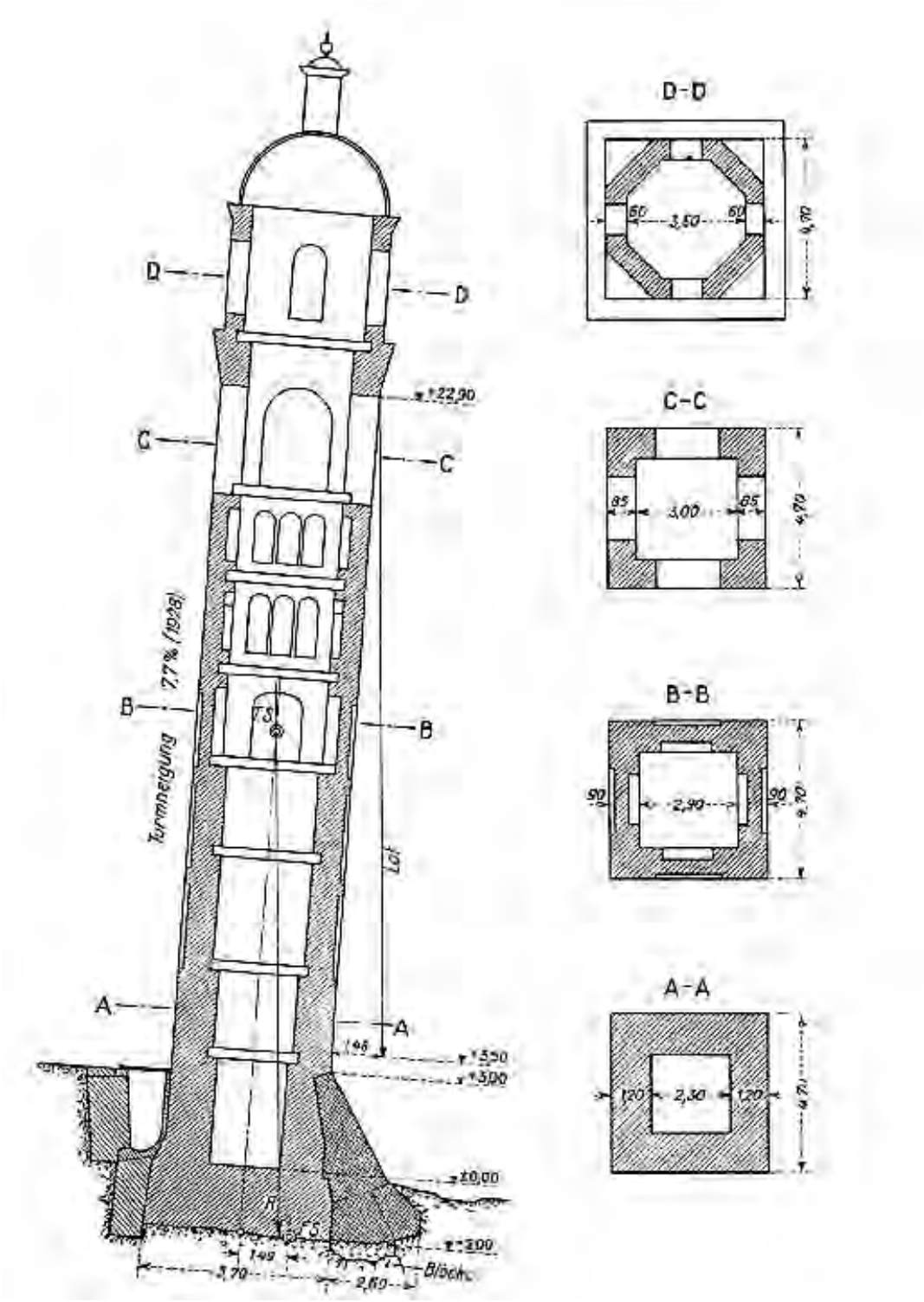


Figure 7: The first stabilization of the Leaning Tower (after Maillart, 1931) .

For the Leaning Tower, Maillart established that with the lower edge of the foundation experiencing the load of 700 kPa, this did not only contribute to the tower inclination, but could also bring it close to the bearing capacity failure (Maillart, 1931). It was decided to implement stabilization in two steps (Figure 7): (i) decreasing the acting earth pressures upslope, and (ii) in-

creasing the foundation area downslope. The work was carried out by the firm Prader & Cie (R. Coray). The first step was accomplished by excavating a trench behind the tower (Figure 7), which was later supported by concrete plates. In the second critical step, a concrete base was constructed starting from 3 m above the ground level, connected to the existing masonry and transferred the load to the extended foundation (Figure 7). This allowed do shift the eccentricity of the resultant force from 1.23 m downslope to 0.26 m upslope, reducing the load on the lower edge of the foundation from 700 kPa to 200 kPa (without the wind load) and eliminating the imminent bearing capacity failure.

Needless to say, the task was challenging since it involved excavation into soil and masonry at the overstressed downslope part of the tower. In order to avoid potential collapse during stabilization works, the tower was pulled back by 16 tension cables connected to the top openings in the tower and anchored back in the heavy concrete blocks. Additionally, the construction of the downslope concrete base, excavation and extension of the foundation was carried out in five narrow stages. During the works and after their completion in November 1928 the inclination rate increased (Figure 5), which is hardly surprising, since the new extended foundation required certain displacement to mobilized the contact stress. After 1939, however, significant slowing down was observed, which bought the tower another 30 years of life.

5 THE 2ND STABILIZATION OF THE TOWER (ROBERT HAEFELI, 1968)

From 1961 inclination of the tower started accelerating (Figure 5) and the municipality of St. Moritz felt it necessary to invite an expert opinion – this time of Professor Robert Haefeli. Robert Haefeli (1898-1978) was not only the pioneer of the geotechnical research in Switzerland but also one of the founders of the international Soil Mechanics (the Third World Congress of the still young International Society of Soil Mechanics and Foundation Engineering took place in 1953 in Zurich). Professor Haefeli was also one of the founders of the flagship international journal *Geotechnique.* His contribution was widely acknowledged during the celebration of the 60th Geotechnique anniversary in 2008 (Burland, 2008). Remarkably, his achievements in Snow Mechanics and Glaciology are considered in these disciplines to be as significant as in Soil Mechanics. It was largely due to Haefeli that a major expedition was organized to study the Greenland ice sheet. He was one of the co-founders of the Swiss Institute for Snow and Avalanche Research, and in 1954-1957 he was the president of the International Commission for Snow and Ice. His passion for rocks and soils, snow and ice went far beyond his research: Robert Haefeli loved the Alps and the open-air life, rock climbing and skiing.

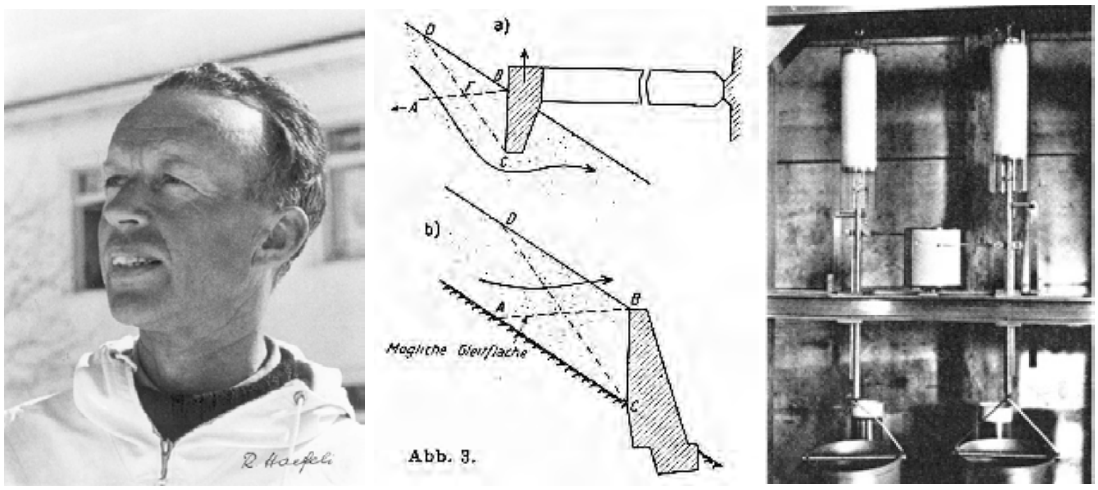


Figure 8: Robert Haefeli (after Burland, 2008), his theory of landslide pressure (center, after Haefeli, 1944) and snow experiments in 1936 (right, source E. Bucher, ETHZ).

After graduating in Civil Engineering from ETH Zurich, Robert Haefeli worked for a few years in Spain, where he was engaged in waterpower development schemes. His interests in soil, snow and ice mechanics were inspired by his work on the construction of dams in the Alps. In 1935 he was invited to establish the first soil mechanics section (including a laboratory) at ETH Zurich, where he carried out fundamental studies on the shearing resistance of soil, snow and ice. His doctoral thesis, published in 1939, was entitled "Snow mechanics with reference to soil mechanics." He was appointed a professor at ETH Zurich in 1947, and lectured on soil and snow mechanics together with avalanche mitigation and preventive measures (Figure 8). He was extremely active in research and his formulas for calculations of the landslide pressures are still extensively used in geotechnical design (Haefeli, 1944).

The immense value of his achievements becomes only more evident against the background of tragically lengthy periods of depressive ill health, which Haefeli suffered over the years and which forced him to leave ETH Zurich in 1953. He continued, however, to be active as a private consultant and it was in this capacity that the St. Moritz municipality invited in 1967 his expertise on the next stabilization of the Leaning Tower, which at that time increased its inclination to 5.37° (9.4%).

In his assessment of the Leaning Tower conditions, Prof. Haefeli calculated stresses in the structure and in the soil, caused by both the tower inclination and the wind load, which had not been considered before. His conclusion was that the stresses in the tower masonry were far from reaching a failure state. The stresses in soil have increased since the last stabilization of Mallart in 1928 from 200 kPa to 290 kPa, and have also not yet reached the bearing capacity. The main

concern was, therefore, the continuing inclination of the tower with increasing average rate (from 0.02% per year in 1939-1961 to 0.05% per year in 1961-1964), which could, eventually, bring the stresses to the critical values. Thus, the solution was needed to slow down the inclination rates and, if possible, to find a way to control the contact stresses.

Professor Haefeli proposed to carry out the stabilization in three stages (Haefeli, 1974). In Stage I, the foundation of the tower was anchored with the help of four 300 kN, 20 m long Duplex anchors in the soil behind the tower (Figure 9). The main goal of this stage was to keep the tower stable during the works for the following stages. Stressing the anchors has caused 0.01° rotation in the upslope direction (opposite to the tower inclination), confirming the effectiveness. Stage II included installation of the drainage system below the tower foundations. Recognizing that the drainage installed in1899 by Prof. Heim 150 m above the tower improved the slope stability, it was clear that it hardly affected the local conditions around the tower. To address that, three horizontal 25 m long, 131 mm diameter wells spread in a fan were drilled below the tower (Figure 9). The three drainage pipes had special filter perforation in the rear 15 m, bringing water into a collector. In spite of the fact that only one pipe brought a few drops ground water, the measure has proven to be rather effective causing the tower to rotate almost immediately backwards by 0.03°. A possible explanation is that lowering one of the local ground water horizons caused increase of effective stresses, leading to the settlements behind the tower.

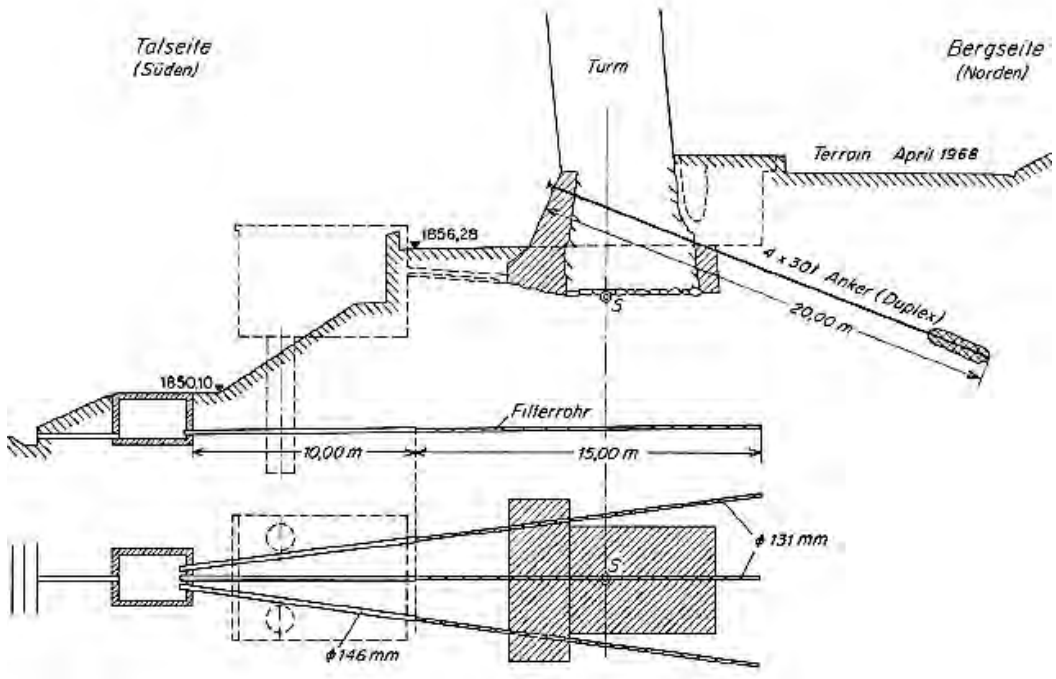


Figure 9: The second stabilization of the leaning tower (after Haefeli, 1974).

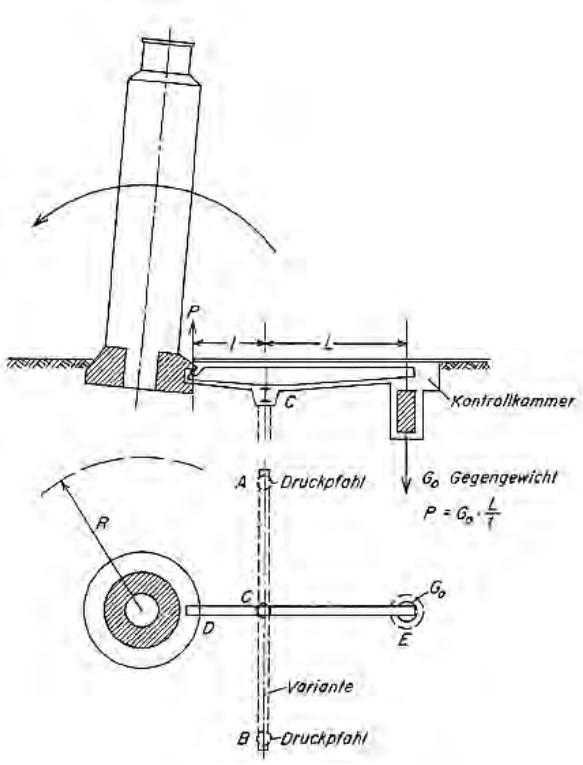


Figure 10: Proposal for a lever system to control the moments in a leaning tower (Haefeli, 1974).

Although, the effectiveness of the first two stages caused the Stage III to be abandoned, it is still worth mentioning as an illustration of Prof. Haefeli's creative thinking. In order to be able to apply resisting moment to the tower foundation, he proposed to build a lever system (dashed lines in Figure 9). The principal function of this system is illustrated in Figure 10. Prof. Haefeli suggested that applying the moment opposite to the overturning moment could not only decrease the stresses in soil and structure but also affect kinematics of the inclination by triggering the soil creep in the opposite direction. Furthermore, in his paper (Haefeli, 1974) he presented a comparison between the Leaning Tower of St. Moritz and the Leaning tower of Pisa and suggested that the lever system can be also used for stabilization of the Leaning Tower of Pisa!

6 THE 3RD STABILIZATION OF THE TOWER (H.-J. LANG, R. WULLIMANN, F. SCHNELLER, 1983)

Professor Haefeli continued to monitor the tower after the 2nd stabilization. In 1973 the anchors, which apparently lost some force were pre-stressed back to 300 kN. In 1976 due to his health problems Professor Haefeli had to terminate his consulting activity and the municipality contracted the Institute of Geotechnical Engineering, ETH Zurich (at that time Institut für Grundbau und Bodenmechanik, IGB) to continue the monitoring of the tower. The institute had grown from the soil mechanics section founded by Haefeli in 1935 and became independent in 1970 under the leadership of Professor Hans-Jürgen Lang.

Hans-Jürgen Lang (b. 1931) graduated in Civil Engineering at ETH Zürich and after a distinguished career in industry was appointed in 1968 the first Professor of Soil Mechanics and Geotechnical Engineering at the ETH Zurich. He headed the institute for 28 years through a very exciting and productive period, when apart from its extensive teaching and research activities the institute supported many major geotechnical projects in the country. Professor Lang has not only been an outstanding engineer but also a legendary teacher, admired by generations of ETH students for his interesting and well structured lectures. His classical textbook "Bodenmechanik und Grundbau", which has already survived 10 editions, is being used in many German-speaking universities and can be found on the shelves of all geotechnical consultants in Switzerland. Written independent of national codes, it focuses on basic mechanical principles of geotechnical problems and inspires engineers towards creative thinking.

The beginning of the IGB monitoring activity coincided with a rather alarming event: on 6th of May 1976 a Friaul earthquake of M_S=6.5 took place in Northern Italy. The tower reacted with additional tilt of about 0.01°, i.e., almost a yearly value. Actually, during the 2nd stabilization in April 1968 the tower reacted similarly to a small earthquake (Haefeli, 1974), but at that time the ground anchors had not yet been pre-stressed. Sensitivity of the tower to such relatively weak seismic loads became a concern and it was decided to review the stability of the tower based on the results of existing site investigations, inclinometer and piezometer measurements. Based on the limit analysis and finite element calculations it was concluded that the safety factor against bearing capacity failure below and outside the tower was as small as FS=1.04 and could easily drop to unity as a result of relatively moderate increase in the ground water level (Wullimann and Schneller, 1989). It has, however, to be noted that the strength values adopted for these calculations were rather conservative, which probably explains why the tower could survive almost 2.5 times higher contact stresses before its 1st stabilization in 1928.

In any case, regardless of what was the true safety reserve, it was clear that with the gradually increasing tower inclination it would sooner or later become insufficient. And because the Stage III of the 2nd stabilization proposed by Prof. Haefeli (Figure 10) had not been implemented, there was still no mechanism in place to control the stresses and inclination. The idea of the 3rd stabilization proposed in 1982 by IGB (Eng. Rudolf Wullimann and Prof. H.-J. Lang) and the engineering firm Edy Toscano AG (Eng. Fredi Schneller), however, differed radically from the Haefeli lever system and included the following components (Figure 11). First, the tower was underpinned by two reinforced concrete foundation walls carefully sank to the depth of 10 m, i.e., about 3 m above the sliding surface (Wullimann and Schneller, 1989). Next, pre-stressed reinforced concrete collars were placed in the area at the foot of the tower (Figure 12) to pre-stress

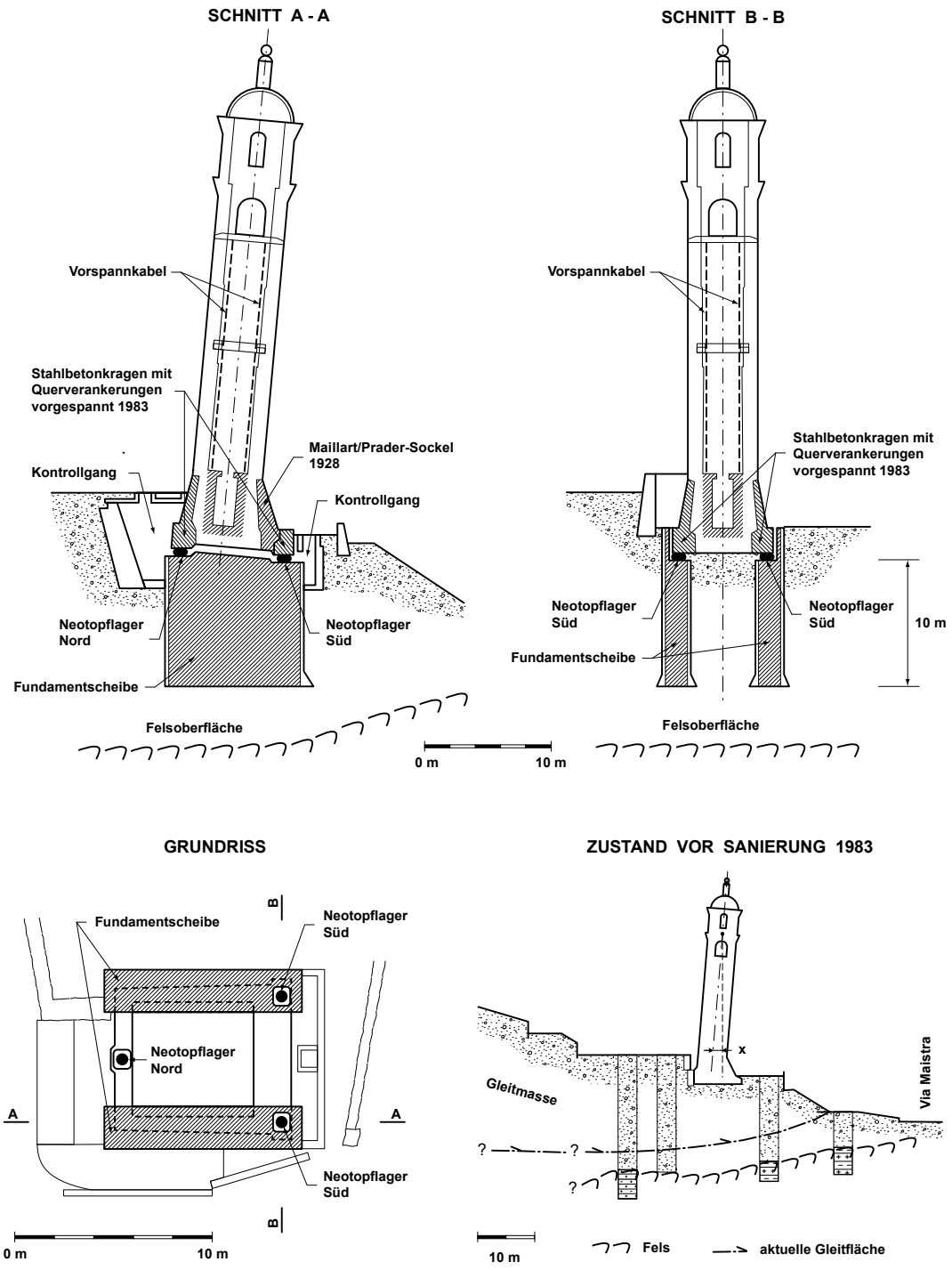


Figure 11: Schematic drawing of the 3rd stabilization of the Leaning Tower of St. Moritz (information poster of St. Moritz community).



Figure 12: Construction of the pre-stressed reinforced concrete collars at the tower footing.

the masonry of the foundation and bind together the original foundation and the footing extension constructed by Maillart in 1928. The tower weight (1264 t) was then lifted from the ground using hydraulic jacks and transferred onto the foundation walls through three Teflon bearing pads located between the collar beams and the foundation walls (Figure 13). Finally, the masonry of the tower was reinforced using vertical internal pre-stressing by means of the tension cables. Similar to the 1st stabilization in 1928, the safety of the tower during the construction works was enabled by the cables and ground anchors (Figure 14), with cables placed in the pipes to reduce temperature strains.



Figure 13: Teflon bearing pad (left) and installation of the new plate during the 2005 tilt correction (right).



Figure 14: Safety cables during the construction works for the 1st stabilization in 1928 (left, after Maillart, 1931) and the 3rd stabilization in 1983 (right, after Wullimann and Schneller, 1989).

After the completion of construction works in 1983, the tilt of the tower was decreased by about 0.27°, bringing it 30 years back in time (no one needs a "straight" leaning tower!). The engineering solution for the 3rd stabilization left the Teflon bearing pads accessible allowing for the future corrections of the tower tilt by lifting its downslope edge and introducing additional steel plates into the bearing pads (Figure 13). Since 1983 the inclination of the tower has been carefully monitored by the St. Moritz municipality (Mr. Pietro Baracchi) using two independent measurement technics: theodolite outside and Zenitlot inside of the tower (Figure 15). The measurements indicated that in 2005 and 2013 the tower approached its 1983 inclination (Figure 16), which in 2005 was corrected by the engineering firm Edy Toscano AG (Eng. Dino Menghini) with the ETH Zurich support (Eng. Ivo Sterba and Dr. Dominik Hauswirth) by lifting its downslope edge and adding a plate into each of downslope bearing pads, and again in 2013 by lifting its upslope edge and removing a plate from the upslope pad.



Figure 15: External theodolite (left) and internal Zenitlot (right) measurements.

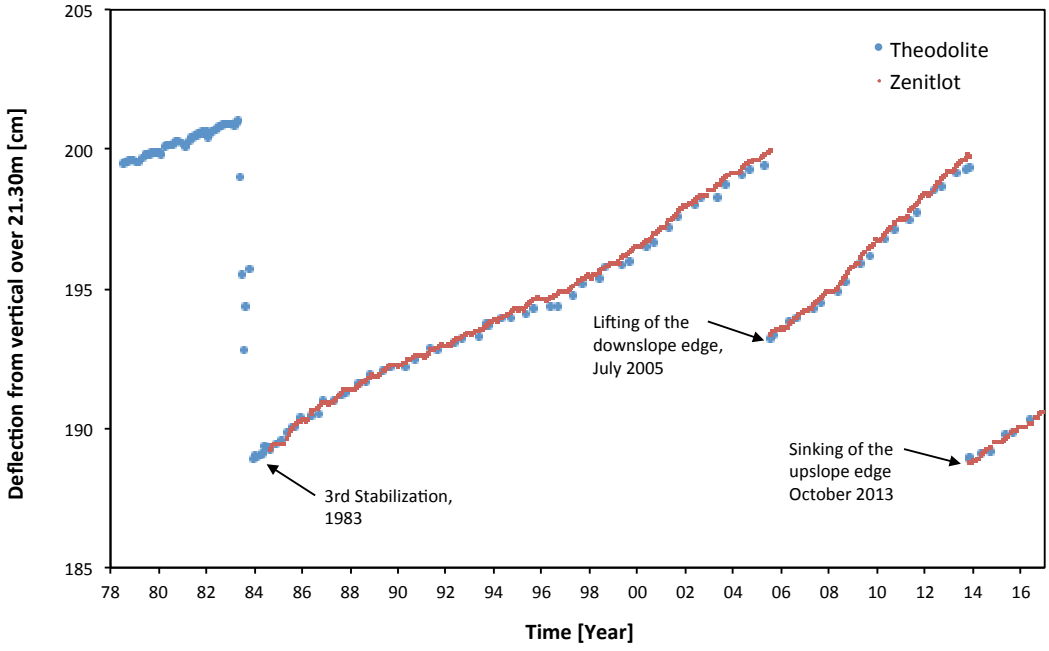


Figure 16: Recent measurements of the tower deflection from the vertical over the 21.30 m height.

7 LANDSLIDE ACCELERATION

Recent investigations of the landslide (Puzrin and Schmid, 2011; Schwager and Puzrin, 2014; Oberender and Puzrin, 2016) have provided some important insights into its mechanism. It has been established that after a long period of decreasing displacement rates, from the beginning of the 1990s the landslide started to accelerate (Figure 17). The "usual suspect" for the creeping landslide acceleration is an increase in precipitation. Indeed, the landslide displacements exhibit sensitivity to precipitation rate changes (e.g., in 2001-2006, when deviation of the precipitation rates from the average value is observed in Figure 17). Around the onset of acceleration, however, no precipitation increase could be observed, which was also confirmed by fairly constant groundwater table (Figure 18), measured using a piezometer pipe installed in a borehole about 37 m above the rock outcrop at the depth of 14 m. No rising in the groundwater table has been observed around the onset of acceleration in 1991. On the contrary, the groundwater table around 1991 seems to drop below the average level (with the average depth of 9.92m), which appears to be the same both in the period of 1984-1996 (around the onset of acceleration) and during the entire measurement period of 1978-2010.

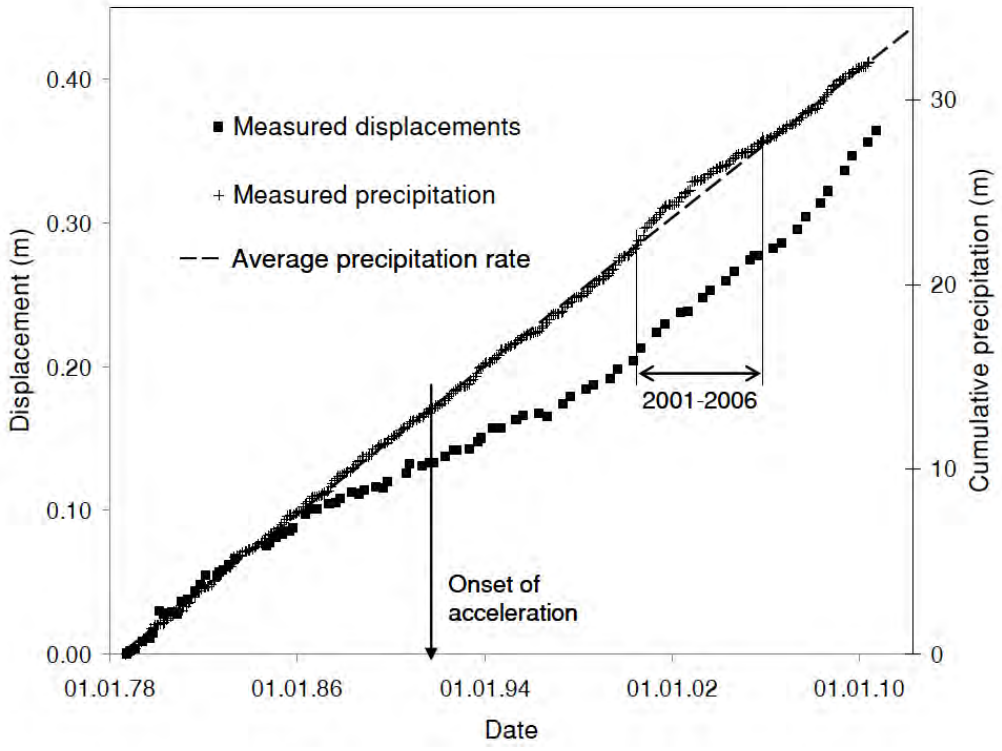


Figure 17: Displacements and cumulative precipitation in 1978-2010. Displacements were measured in the vicinity of the Leaning Tower (21.1 m away from the rock outcrop), in the direction of the slope gradient (after Puzrin and Schmid, 2011).

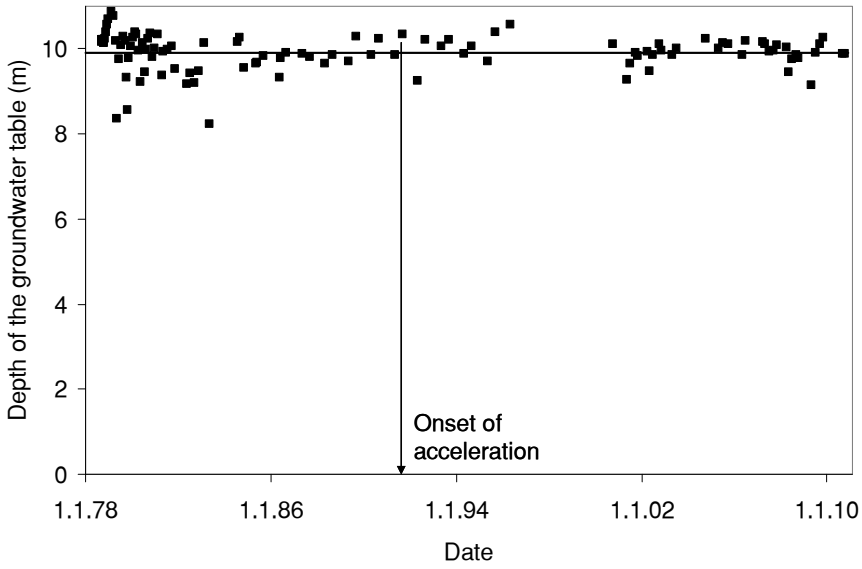


Figure 18: Depth of the groundwater level in the vicinity of the tower in 1978-2010 (after Puzrin and Schmid, 2011).

A more likely reason for the landslide acceleration could be the passive failure of soil at the bottom of the landslide, in the zone characterized by high compression strain rates (Figure 19) within the constructed area in the lower 200 m of the landslide in 1988-1998, close to the latitudinal center of the landslide. Puzrin and Schmid (2011) provide both theoretical and experimental evidence supporting the passive failure hypothesis.

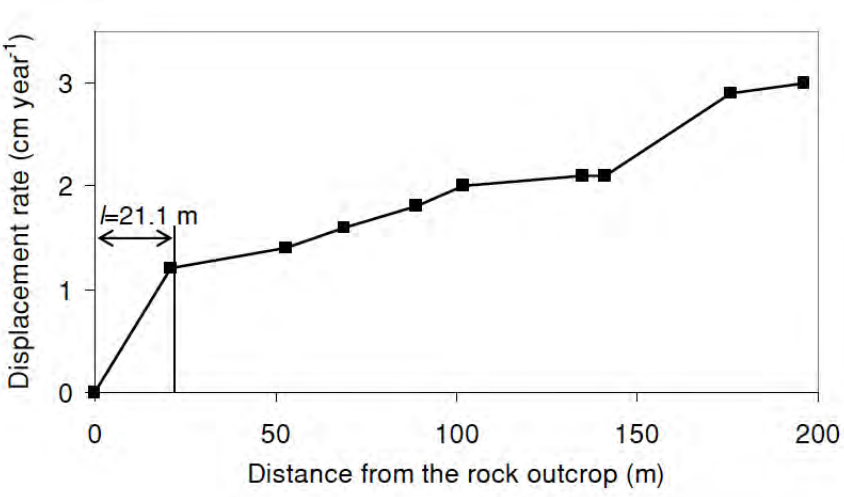


Figure 19: Average displacement rates within the constructed area in the lower 200 m of the landslide in 1988-1998 (after Puzrin and Schmid, 2011).

Measuring in 2006-2010 for the first time displacement rates along the entire landslide length (close to the latitudinal center of the landslide) has shown (Figure 20) that the rates continue increasing upslope and can reach up to 0.5 m/yr (600-700 m above the Via Maistra and just below the foot of the rockfall). This is an order of magnitude higher than the maximum average displacement rates in the constructed zone and suggests that the compression zone along the Via Maistra (including the tower location) can be a subject of significant earth pressures.

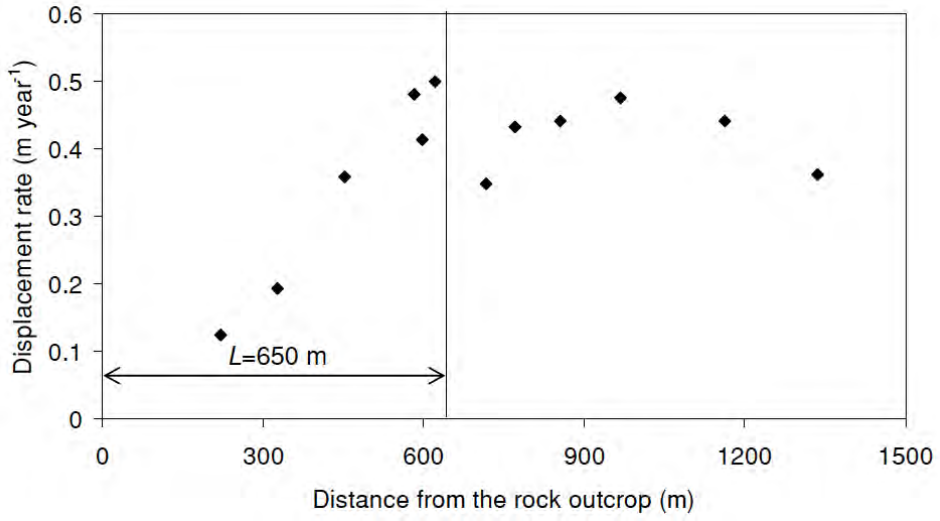


Figure 20: Average displacement rates in 2006-2010 above the constructed area. Displacements were measured using GPS device with the accuracy of 0.02 m (after Puzrin and Schmid, 2011).

The changes in earth pressures were back-calculated from the changes in the shape of inclinometer pipes measured in 2008-2012 using the novel inclinodeformeter (Figure 21, after Schwager and Puzrin 2014). The average yearly increase in earth pressure in the compression zone was 0.3 kPa, an order of magnitude lower than the pressure increase outside this zone. At the same time, the strain rates in the compression zone are an order of magnitude higher than outside (Figure 19). This implies plastic flow at practically constant stresses, directly confirming the failure hypothesis.

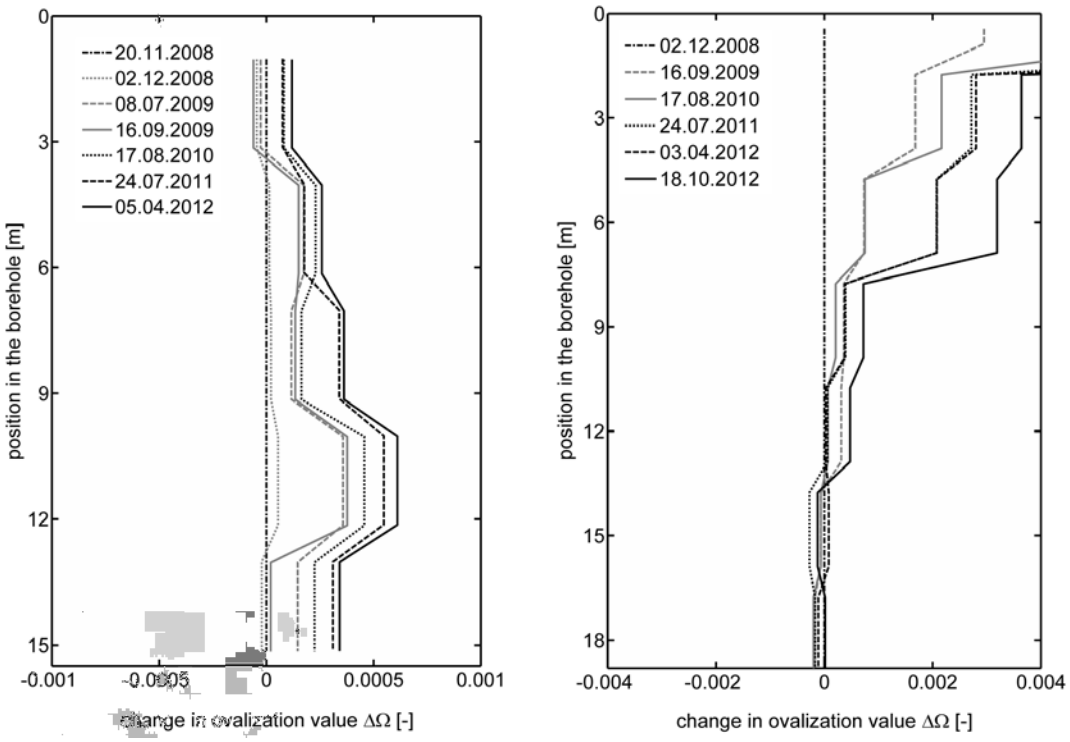


Figure 21: Evolution of the inclinometer pipe shape within the failed compression zone (left) and above the failed zone (right). Ovalization value Ω is calculated as the difference of the perpendicular pipe diameters normalized by the nominal outer radius of the inclinometer pipe R (after Schwager and Puzrin, 2014).

8 MONITORING

The question what exactly causes today the continuously increasing inclination of the tower can be answered using the results of displacement measurements. The displacements and rotations of the tower have been occasionally measured since 1908 (Figure 22). From the beginning of the 1980s, the tower has been monitored on a regular basis using several complimenting types of manual measurements. Consistent with other observations, these measurements show that in the last three decades the ground surface in the vicinity of the tower experienced acceleration of average downslope displacement rates (Figure 23): 6.5 mm/year in 1986-1996; 12 mm/year in 1996-2006 and 19.5 mm/year in 2006-2016. However, as is seen from inclinometer readings (Figure 24), displacement rates decrease with depth, resulting at the slip surface in about 10 mm/year (in 2006-2010). The tower foundation walls are 10 m deep and are likely to be effected by this non-uniform velocity field. Their rotation expresses itself as a differential settlement/heave at the four corners of the tower at the ground surface and leads to the increasing tower inclination. Using the measurement base distance of 21.3m (Figure 23), the average tower

deflection rate of 6mm/year corresponds to a tilt rate of around 0.016°/year or about 3 mm per 10 m of the foundation wall depth, consistent with the inclinometer readings in Figure 24.

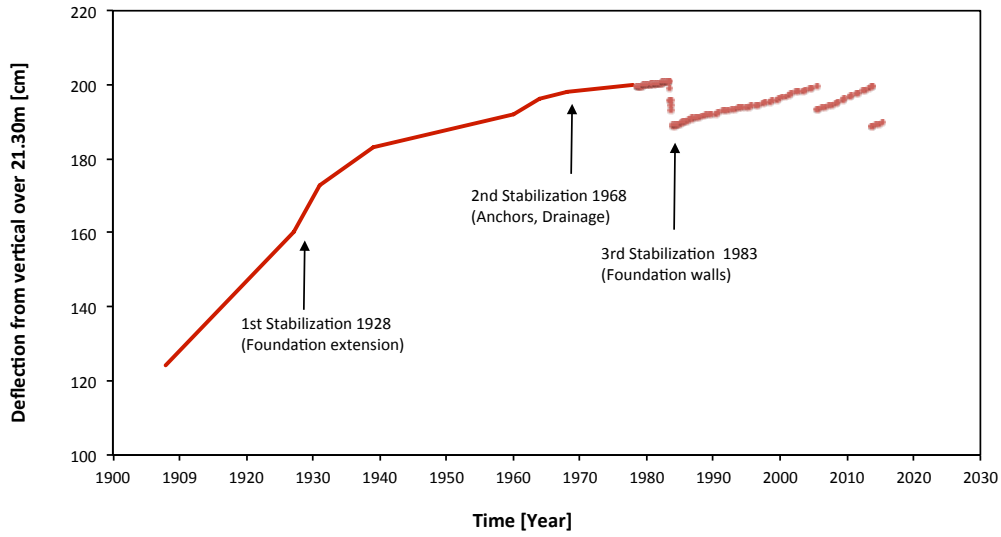


Figure 22: Approximate historic measurements of the tower deflection from the vertical over the 21.30 m height.

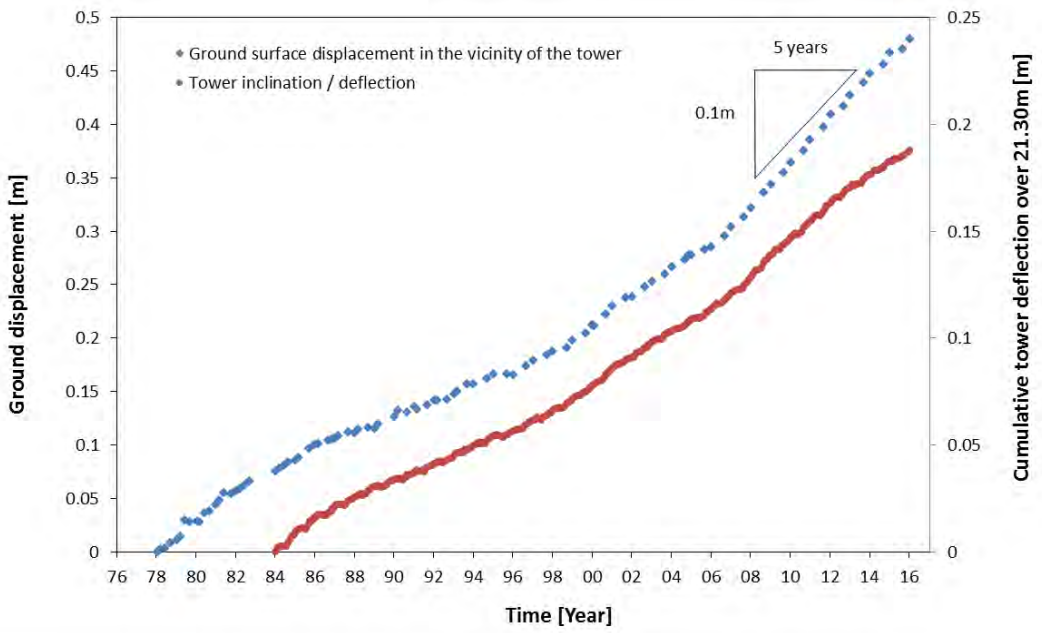


Figure 23: Ground displacements and cumulative inclination of the tower in 1978-2016.

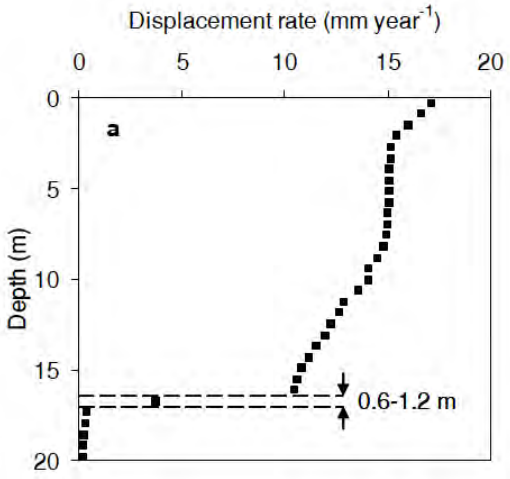


Figure 24: Average displacement rates in depth measured in 2006-2010 by an inclinometer in the vicinity of the Leaning Tower (after Puzrin and Schmid, 2011).

Acceleration of the landslide displacement rates caused by the soil failure in the compression zone has been closely followed by the tower inclination (Figure 23). In order to monitor these developments closer and to be able to react to extreme events, in addition to the manual measurements an automatic monitoring system was designed by the Institute of Geotechnical Engineering, ETH Zurich, and installed by Solexperts AG (Figure 25). The system includes an inclination sensor (A) placed in a heated steel box to enable winter measurements; temperature and air pressure sensor (B); two in-place inclinometers (C1) and (C2) and a piezometer (D), all placed at the sliding surface level in boreholes outside the tower; and three accelerometers (E1 at the rock outcrop in the basement of the Kulm Hotel, E2 at the tower foundation and E3 at an upper floor level of the tower). All the data is accessible online and automatic text messages are sent when the measured values reach alarm values.

The system is extremely sensitive and could detect very small changes in tower inclination and pore water pressures caused by construction works in the vicinity of the tower. During the tilt correction in 2013, the inclination sensor (A) readings were successfully validated against the Zenitlot measurements (Figure 26). Vibrations of the tower, which were caused by a slight impact when the tower was lifted and brought back into the contact with the bearing pad after a plate was removed, were detected by the accelerometer (E3) (Figure 27), allowing for assessment of the natural frequency of the tower. The 2016 earthquakes in Italy have also been detected by all three accelerometers and have been used for validating the site response spectra.

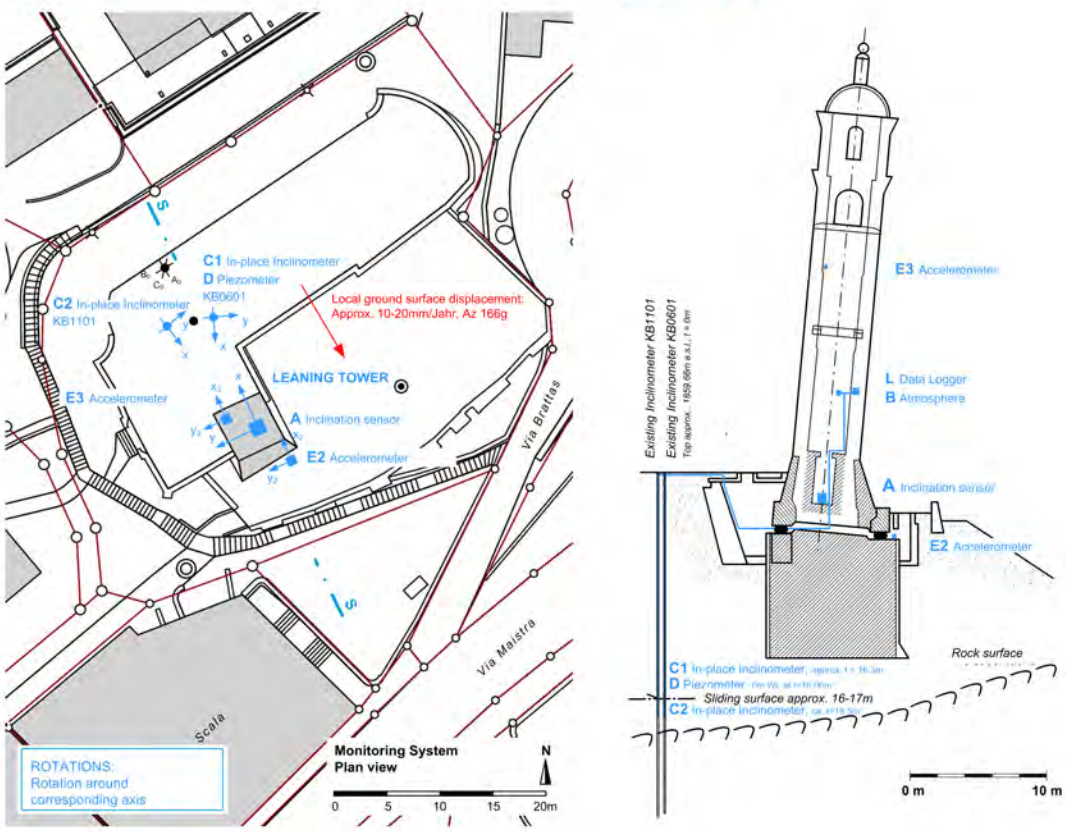


Figure 25: Plan view and section S – S of the automatic monitoring system.

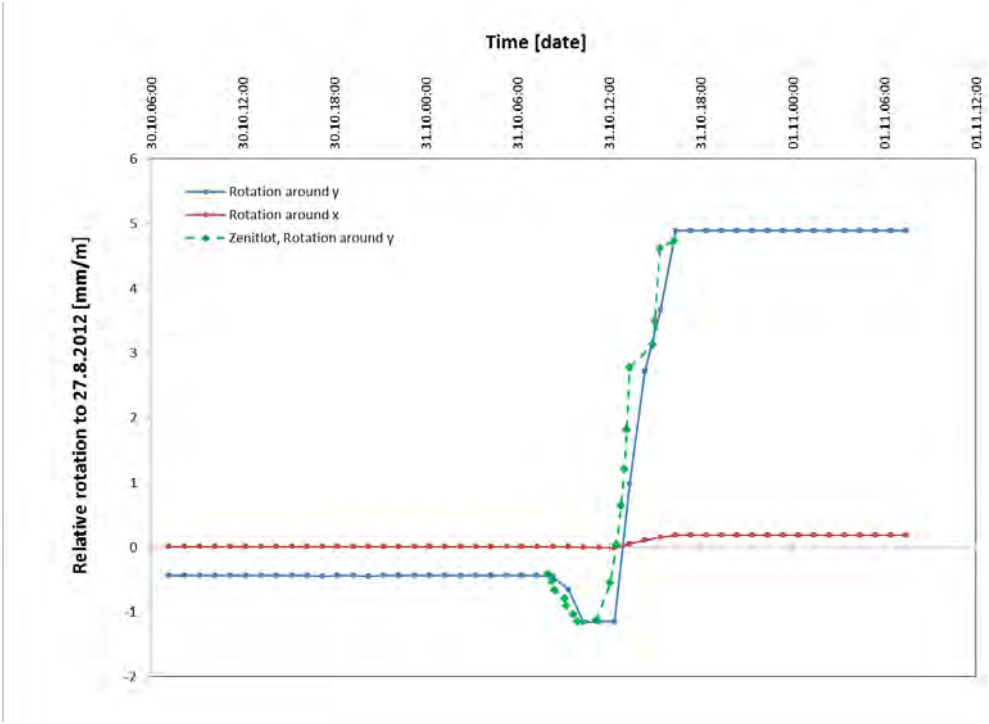


Figure 26: Rotation of the tower during the tilt correction on 31.10.2013.

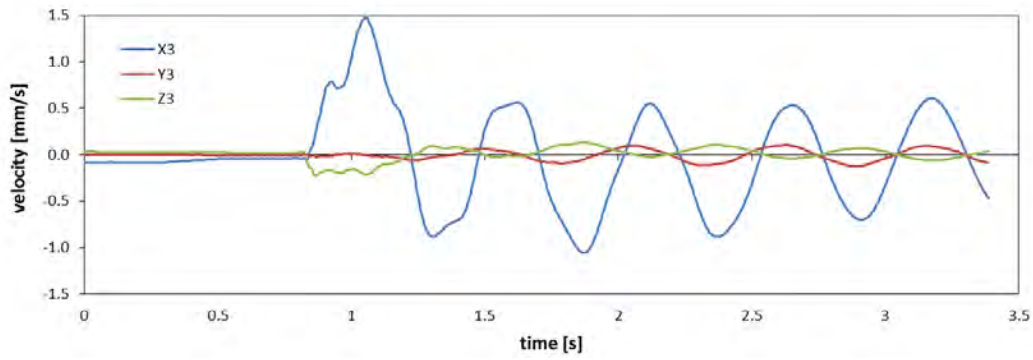


Figure 27: Free vibrations of the tower (velocity, sensor E3) during the tilt correction on 31.10.2013.

9 OUTLOOK

More than a hundred years of efforts by brilliant Swiss engineers have kept the Leaning Tower of St. Moritz alive, long after all other structures in its vicinity were demolished, rebuilt and demolished again. How to keep a historic tall narrow structure stable in a compression zone of a creeping landslide is not something a geotechnical engineer is taught at school. The enormous, 10 million cubic meter landslide is not going to stop, on the contrary it has been currently accelerating, while the existing Teflon bearing pads have exhausted their capacity for changing their height by adding/removing plates. Furthermore, it has been established (Edy Toscano AG) that the existing solution does not satisfy the most recent norms for seismic stability.

Stochastic models of landslide displacements (Oberender and Puzrin, 2016) predict that the tower will reach its maximum historical inclination of 5.40 degrees again in about 10 years. Before this happens a new solution has to be found and implemented, including seismic isolation. Hopefully, the Leaning Tower of St. Moritz will continue challenging new generations of engineers for many years, decades and centuries to come!

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