

# Seismic Resistance Analysis and innovative Earthquake Refit of a 37 year old Office Building



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## ABSTRACT

The office building housing the Agrisano health insurance company in Windisch was fully renovated and also raised by one storey in the summer of 2011. In addition to aesthetic and energy saving improvements, the buildings seismic resistance was also increased.

The review of the structure's earthquake resistance was carried out on the basis of technical note sia 2018, Seismic Review of Existing Buildings. This technical note is based on the assessment of probabilities of risk and analyses the costs and benefits of potential earthquake strengthening solutions. An initial calculation indicated that only 10% of the earthquake resistance required by the latest standards could be guaranteed without additional strengthening of the structure.

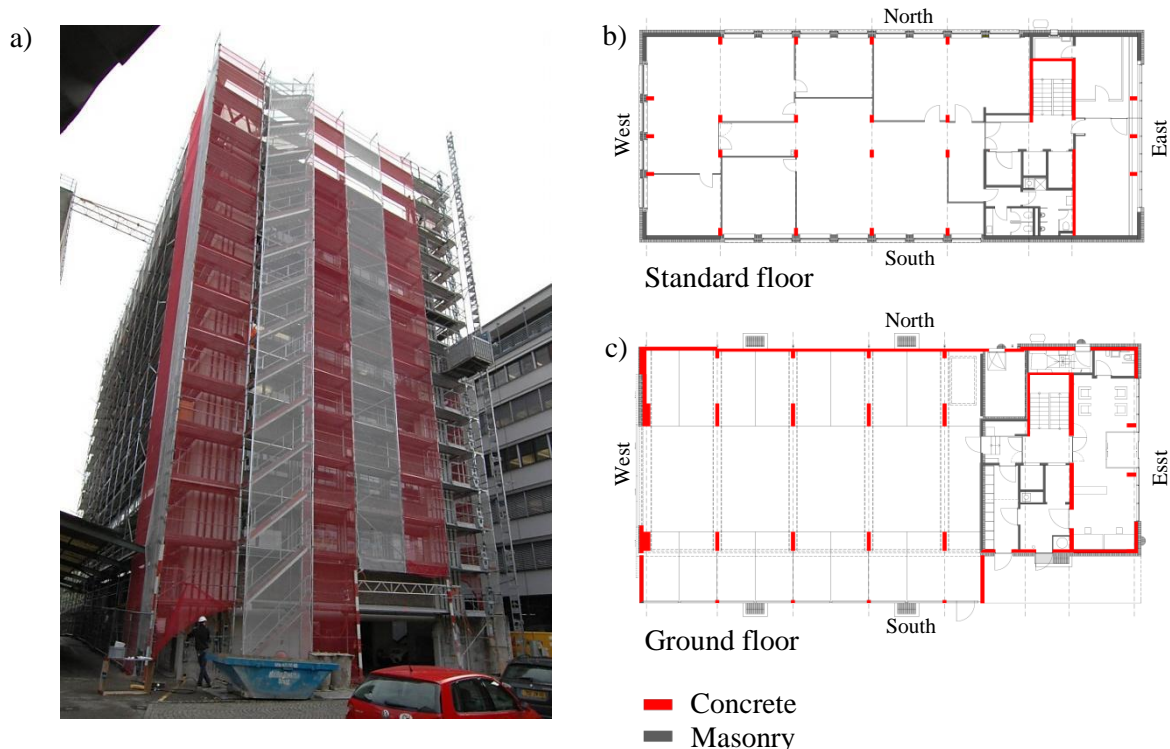
Various different strengthening options were considered for increasing the seismic resistance of the office building and its masonry wall panels. The final decision was made in favour of a strengthening system from StressHead AG which involved the installation and post-tensioning of Sika CarboDur CFRP plates. In total, 16 of these post-tensioned plates with a tensioning force of 220kN were used. The anchorage of the tensioning force was achieved with a combined CFRP StressHead and steel reaction frame to transfer the load directly into the floor slabs. The low space requirement of the CFRP tendons and their end anchors allowed their external application on the existing façade. Additional external insulation was then applied on the façade, which now also covers and hides the strengthening system completely.

*Keywords: Seismic engineering, masonry strengthening, post-tensioned CFRP plates*

## 1. PROJECT DESCRIPTION

To improve its facilities, the Agrisano health insurance company wanted to extend its existing office building at Steinackerstrasse 7 in Windisch, Switzerland, by adding a new cafeteria and service areas. The building is located between the Brugg railway station and the campus of the University of Applied Sciences for North West Switzerland, and as space around the building was limited, it was decided to do this by raising it by one additional storey.

Construction work on the project began in spring 2011 on the office building that dates originally from 1975. In addition to the additional storey being added, the building underwent extensive insulation and energy saving improvements, plus its seismic resistance was considerably increased. The reception area on the ground floor had ceased to meet the owner's standards and was also completely revamped in conjunction with the other works.



**Figure 1:** The Agrisano office building in Windisch, Switzerland

The existing building consisted of 2 basement floors, the ground floor and 7 additional floors. The dimensions in rectangular plan were 14 x 34m. Before the additional storey was added the building was 25m tall. After raising it by one storey and locating the additional building services and equipment on the roof, it is now 30m tall.

The building is a skeleton structure, which consists of a grid of columns with a core on the access side, where the main shaft and all of the building services, toilets, stairs and the lifts are located. The space allocation in the rest of the building is flexible and based on movable partition walls.

### 1.1. Structure and materials

The additional 8<sup>th</sup> floor was produced as a lightweight structure, with the walls and the ceiling being made of a timber construction. This meant that the existing building structure only had to accommodate low additional loadings and the existing foundations were adequate. For stiffening purposes, some of the walls in the original core section were reinforced concrete. By using this timber construction it was therefore possible to add the extra storey and to only impose minimal extra load on the existing structural system.

The structural system acting vertically consists of prefabricated columns and the reinforced concrete core. The concrete panels forming the core are responsible for horizontal load transfer. As shown in Figure 1b), there are also some reinforced concrete walls included for horizontal load transfer in the transverse direction of the building in the standard floors. However in the longitudinal direction, all of the panel walls are masonry. Since the building is nearly 40 years old, the precise nature and condition of the masonry materials was unclear. Investigation and analyses showed that the panels were the Durisol Walling System from Durisol AG. All of the existing masonry walls in the building were formed using this system. The Durisol Walling System consists of hollow concrete blocks which are subsequently filled with concrete in situ. With this system the infill concrete core assumes the structural function and the Durisol blocks only act as formwork for the concrete and provide some thermal insulation. The Durisol Walling System therefore has the same structural function as an unreinforced concrete panel.

The ability of the Durisol walling to absorb shear forces from a seismic impact is limited. There is no reinforcement in the Durisol Walling System and its relatively low weight prevents it generating a sufficiently high force to divert the shear forces into compressive force. Longitudinal earthquake re-engineering with an external strengthening system was therefore now considered essential.

To give the office building the insulation value equivalent to a min-energy building, additional external insulation of 20cm was designed to be added as part of the energy improvements. The intention was therefore to locate the strengthening system for the seismic improvements invisibly, either on the inside of the walls or externally but hidden within the new insulation.

## **2. SEISMIC ANALYSIS**

### **2.1.SIA Data Sheet 2018**

The seismic analysis was carried out on the basis of technical note 2018, Seismic Review of Existing Buildings. This technical note is based on the assessment of probabilities of risk and analyses the cost and benefits of potential earthquake strengthening solutions.

In addition to reviewing the conventional earthquake safety of a structure, the hazard analysis also examines its economic integration and dependence. From a national economic standpoint, structures of national or regional importance and buildings of low importance are assessed differently in terms of the consequences of an earthquake. Basically, the additional costs for compliant seismic re-engineering of new buildings are insignificant. In contrast, considerable expenditure - with high construction and subsequent operating costs - is generally incurred for re-engineering or increasing the earthquake resistance and safety of existing structures.

Older structures designed to meet former standards have much lower earthquake resistance than is specified in the new structural standards SIA 260 to 267. Technical note 2018 can be used to determine how the seismic resistance of existing structures can be reviewed and assessed, by comparison with new buildings and in accordance with the principles of the new structural standards. The data sheet offers advice on whether a building should be re-engineered, or whether its existing condition should continue to be accepted. It is based on a risk probability and cost-benefit analysis, which allows that all structures do not have to be upgraded to the level for new buildings in the standards. Whether the project costs are proportional and reasonable can therefore be taken fully into account when deciding the extent of the earthquake resistance increase required.

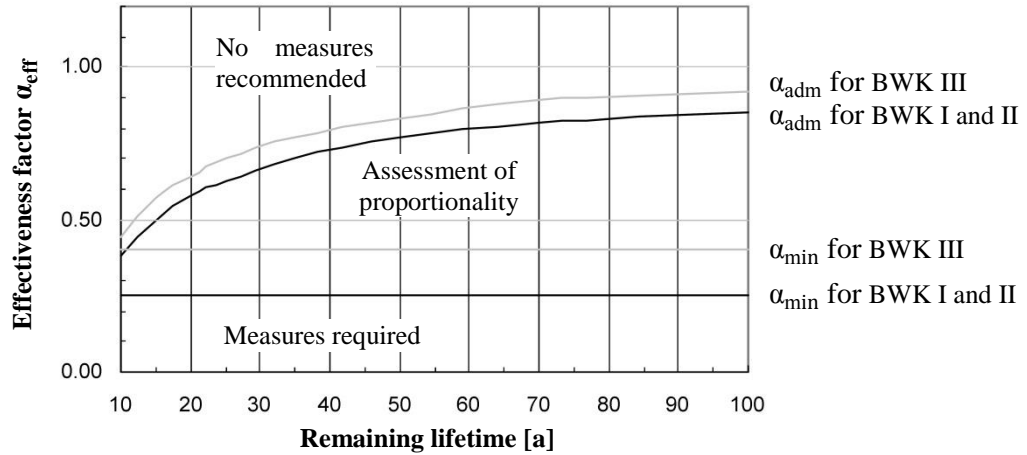
The mathematical seismic safety analysis is assessed by an effectiveness factor  $\alpha_{\text{eff}}$ . It is obtained by comparing the impact requirements in accordance with the standard, to the resistance of the existing building under the standard.

### **2.2.Seismic analysis, Agrisano office building**

The seismic analysis by the Consulting Engineers Gerber & Partner AG of Windisch, showed that the building had significant earthquake resistance deficiencies longitudinally. The model for this analysis was the response spectra method for multi-storey buildings. Detailed information on this method is contained in the work of Priestley, Calvi, Kowalsky, Displacement Base Seismic Design of Structures (2007). The conclusion reached was an  $\alpha_{\text{eff}}$  of 0.10, which corresponds to a resistance of only 10% relative to the impact resistance required according to the latest standards.

Transversely the seismic resistance was within the permissible range. As shown in Figure 1b), there are some concrete walls in the core section of the standard floors for horizontal transfer of load. The remainder of this document is concerned only with the strengthening to increase resistance in the longitudinal direction.

For buildings in structure classes (BWK) I and II, to which this office building belongs, an  $\alpha_{\min}$  of 0.25 [Fig. 2] is required so that the individual risk and reasonableness are covered. This level must be reached for all buildings of any category and remaining life. The assessment of proportionality resulted in an  $\alpha_{\text{adm}}$  of 0.80. As Figure 2 shows, this value represents the upper limit of cost and benefit proportionality. The remaining lifetime and occupancy of the building are factors in the calculation of  $\alpha_{\text{adm}}$ . The relationship of the  $\alpha$  values is shown in formula 1.



**Figure 2:**  $\alpha_{\min}$  and  $\alpha_{\text{adm}}$  thresholds (cf. SIA 2018 (2004). Fig. 6)

$$\alpha_{\min} \leq \alpha_{\text{eff}} \leq \alpha_{\text{adm}} \quad (1)$$

Since an increase in  $\alpha_{\text{eff}}$  to 0.25 had to be made in any case on reasonableness grounds, further analyses were carried out to increase the proportionality to a maximum  $\alpha_{\text{eff}}$  value of 0.80. It was concluded that the safety-related investment costs to increase the  $\alpha$  value from 0.25 to 0.80 were only 175,000 CHF. According to the data sheet, proportionality is always achieved if the investment costs for the seismic safety measures are lower than 175,000 CHF. An initial estimate of the investment costs indicated that the strengthening would come to less than 175,000 CHF. It was therefore decided to raise the  $\alpha$  value to 0.80. A detailed strengthening concept with the necessary additional capability could then be developed.

### 3. STRENGTHENING CONCEPT

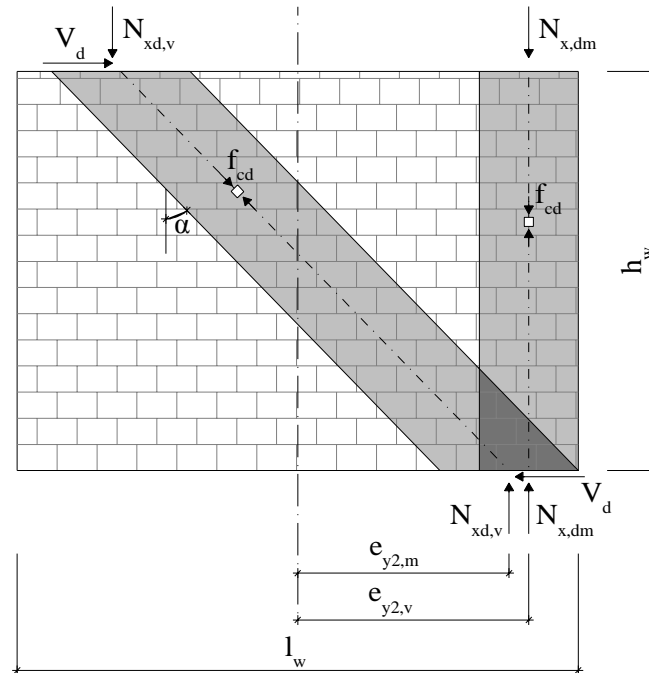
#### 3.1. Strengthening options

Various strengthening systems were available for the earthquake re-engineering of the office building. Basically 2 different options were considered in greater detail. Either reinforced concrete walls could be installed or the existing masonry walls could be strengthened.

Earthquake re-engineering using new concrete walls is a common solution. However this option was rejected for the Agrisano health insurance building because retention of the flexible space allocation on the upper floors was wanted and the weight of these additional concrete walls would have generated considerable extra loads on the structural system. The construction and sequence of the concreting works would also have been difficult and complex.

Existing masonry walls can be strengthened with various materials. On this building strengthening with steel and CFRP plates was examined. The principle of load transfer in the existing walling system on this building can be viewed on approximately the same theoretical basis as is considered for

traditional masonry in Switzerland. Figure 3 shows the stress fields of an unreinforced concrete or masonry panel. The shear force is transferred by means of the applied force, as a load diagonally.



**Figure 3:** Stress fields of the walling system

The verification of the ultimate limit state is based on the lower bound theorem of the theory of plasticity. An allowable state of stress has to fulfil the conditions of equilibrium and the yield criterion.

The normal force  $N_{xdv}$  increases due to the added normal force from the masonry strengthening. A further consequence of the increase in normal force, at a constant angle  $\alpha$ , is also a rise in the maximum shear force to be absorbed  $V_d$  resulting from an earthquake impact. Detailed information is contained in Zimmerli, Schwartz, Schwegler, Masonry: Design and Construction (1999).

Unlike concrete, the strengthening option with steel plates could provide a high tensile force with a small additional cross-sectional area, but there was a mismatch between the optimum material characteristics and the materials installation. Because the earthquake strengthening of the individual storeys always had to reach down to the ground floor, it would have required either very long and expensive steel plates, also involving transport problems, or steel plates with numerous joints.

CFRP plates were another option. Carbon fibre reinforced plastic plates are light, easy to deliver to the site in the required long lengths and are extremely durable and corrosion resistant. The fact that CFRP plates are easier to handle than steel plates is a particularly important factor on high-rise buildings where long strengthening lengths are required. However the combination of purely elastic material properties without plastic deformability, and localized deformation in the event of cracks can cause brittle failure of the strengthening with plates applied without additional tensioning, even with only slight deformation. This also means that the very high tensile strength of the material is not fully utilized. This problem would occur mainly in the joint area from the walls to the floor slabs. Slight displacement in this area could cause the CFRP plates to weaken. Another disadvantage for this type of strengthening with plates applied without additional tensioning is the anchorage length required. As with traditional steel reinforcement, the plates also need an anchorage length. The necessary tensile force is developed through the bond with the adhesive. Here again, any unevenness or displacement could subsequently lead to detachment of the plates.

It was concluded as a result that earthquake strengthening with post-tensioned CFRP plates represented the best solution for the Agrisano health insurance building. The post-tensioning will

prevent any premature failure of the strengthening due to detachment of the plates and the outstanding characteristics of the CFRP material can be fully utilized. In addition the StressHead CarboStress system from StressHead AG Lucerne provides a post-tensioning method which also has a compact CFRP stress head for the end anchorages.

### 3.2. StressHead CarboStress system

The compact anchorage area heads called StressHeads developed by StressHead AG are made of CFRP like the strengthening plates. This concentrated anchorage at the end of the tendon has several advantages. Because the flow of force is clearly visible, the system can even be used on lower quality substrates. With the 100mm long StressHeads, the end anchorage can be formed even when there is limited space behind the support. The system also has installation advantages. This post-tensioned system can therefore be installed to achieve the strengthening and seismic resistance improvements required in a relatively short time and without any major construction works.

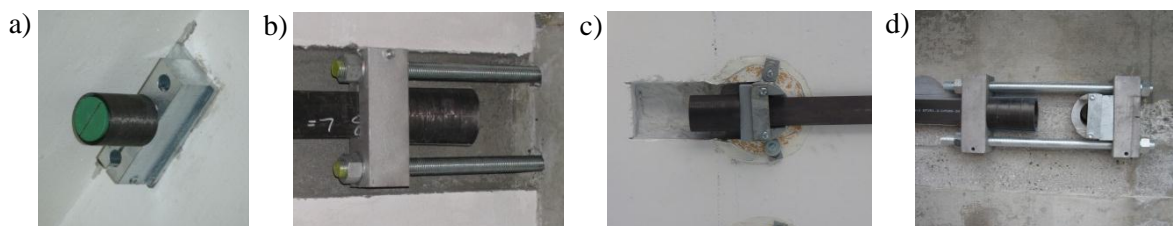


**Figure 4:** Schematic drawing of the StressHead CarboStress post-tensioning system

In the limit state for structural safety, the StressHead CarboStress system is based on the principle of external post-tensioning without bonding. The application of an active force on the building also gives structural advantages over the non-tensioned options already referred to in section 3.1. This simple post-tensioning system generates a controllable flow of force and makes full use of the beneficial material characteristics of the CFRP plates. The maximum tensioning force is 220kN. Because the tendon is bonded to the surface of the structure, the bonding forces in the service limit state must also be considered and included in the calculations.

The main part of the post-tensioning system is formed by the well-known and extensively used CarboDur CFRP plates. The tendon consists of a CFRP plate, Sika CarboDur S626 from Sika AG, and 2 CFRP heads, StressHead 220, which are installed on both ends. Anchorage in the substrate is guaranteed by a fixed anchor and a movable anchor which forms the tensioning end.

The standard StressHead AG range contains 2 anchorage systems (types II and III). Both types II and III have a both a movable and a fixed anchor. Anchorage type III [Fig. 5c and 5d] applies the tensioning forces into the substrate as shear force through a shear connector. The force is applied parallel to the tensioning plane. Unlike type II, the tensioning force behind the support is transferred through a steel anchor directly into the substrate and as normal force. The force in anchorage type II [Fig. 5a and 5b] is applied at right angles to the tensioning plane.



**Figure 5:** Anchorage types II and III

The post-tensioned CFRP plate anchors are adaptable to the local constraints on the structure. Dependent on the depth of the external reinforcement layer, part of the anchor can be recessed locally

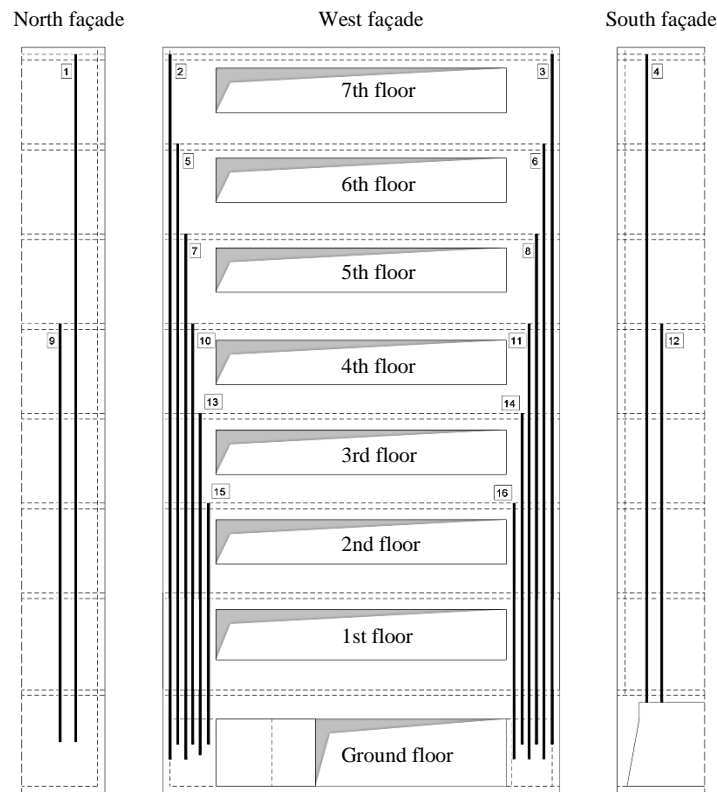
into the structure. This reduces the projection of the steel section of the end anchor from the concrete surface by 50mm.

**Table 1:** StressHead CarboStress post-tensioning system characteristics

StressHead		CFRP plate	
Type	StressHead 220	Type	Sika CarboDur S626
Cross-section	d=80mm, l=100mm	Cross-section	60x2.6mm
Post-tensioning system			
Tensioning force $P_{0,max}$	220kN	Tensioning $\sigma_{P0,max}$	1,410N/mm <sup>2</sup>
Max. force to be applied in the anchorage $F_{u,min}$	300kN	Elongation $\epsilon_{P0,max}$	8.5‰
E-modulus, longitudinal	$\geq 165,000\text{N/mm}^2$	Tensile strength $\sigma_u$	$\geq 2,800\text{N/mm}^2$

### 3.3. Configuration concept

The number of strengthening units required results from the tensile forces in the seismic analysis. Because the earthquake impact increases in the building from top to bottom, all of the strengthening systems were not installed right up to the top storey. This makes the configuration perfectly adapted to the potential earthquake impact. In total 16 of the systems were installed, 12 on the west façade and 2 each on the north and south façades [Fig. 6].



**Figure 6:** Configuration diagram of the post-tensioned strengthening systems

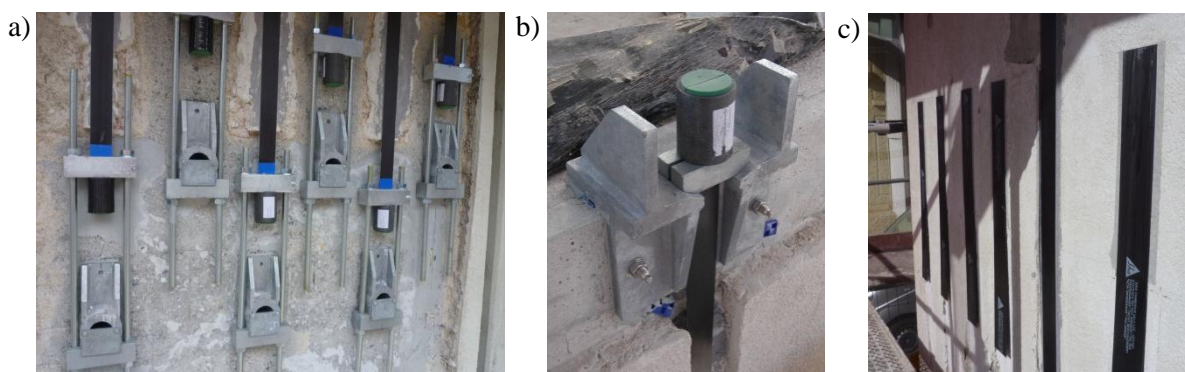
All of the movable anchors were located on the ground floor. As can be seen in Figure 1c), there were existing concrete walls on the west side of the office building. These walls provided suitable supports for the anchors. The force is applied through steel sections and a shear connector. This goes back to the StressHead AG standard type III solution [Fig. 5d)]. It is only on the south façade (systems 4 and 12) that no concrete walls were available. The solution here is the StressHead AG standard type II [Fig. 5b)] which anchors the plates directly into the floor slab with threaded rods.



Application of force on the fixed side posed a problem. As mentioned in section 1.1., the walling is formed by Durisol blocks. The outermost course of blockwork just acts as insulation and can only absorb low compressive forces. The load forces are therefore taken by the concrete inside the Durisol blocks. As a result the tensioning force had to be transferred from the floor slabs as directly as possible into these concrete cores.

The StressHead CarboStress system cannot be anchored in masonry by these standard anchors. Special solutions were already being developed by StressHead AG for masonry, as in the example discussed here. StressHead AG's intention was to develop standard anchors for masonry in collaboration with Lucerne University of Applied Sciences – Engineering and Architecture, which was also involved in the earlier development of the post-tensioning systems by testing the tendons and anchors.

Therefore a steel anchor was developed which transferred the force efficiently. The objective was to bring the tensioning force as far back as possible and directly into the concrete cores of the double-skin walling, but eccentricity of the anchor and/or the StressHead could not be prevented. The resultant moment is therefore absorbed by a Hilti AG anchor rod, which anchors the tensile force in the floor slab concrete. However, there is another disadvantage to this re-positioning of the tendon to be further back. The displacement forces generated can act transversely on the weak external masonry block walls. It has been found that the blocks can be damaged even with relatively low compressive forces. To reduce these effects, the CFRP plates were positioned under the anchorage. This enables the displacement force to be supported directly in the slab. Laborious concrete demolition work was necessary to move each of these tendons further back. A non-tensioned CFRP plate was installed in advance to form a displacement saddle so that the plate could slide easily on the substrate.



**Figure 6:** a) Movable anchor, b) Fixed anchor, c) Non-tensioned CFRP plate forming a displacement saddle

After installation of all of the steel sections and hanging the tendons, the post-tensioning process began. The tendon was tensioned on the movable side using a hydraulic cylinder press in several steps up to a force of 220kN. The threaded rods taking the tensioning force from the plate to the steel anchors were then fixed and trimmed. Finally the tendon was treated with a protective coating. The post-tensioning, pre-treatments and the post tensioning treatment works required a total of only 15 working days.

Due to the new 20cm thick insulation, it was possible to fit the tendons and the end anchorages within the façade. When the construction works were completed, the earthquake resistance refit was invisible.

#### 4. CONCLUSIONS

The scientific knowledge gained from seismic research has led to a tightening of the relevant structural standards in recent years. Many older buildings therefore now have inadequate seismic resistance. But strengthening with full adaptation to the new standards makes sense in very few cases because the remaining life of the building is shorter and the refit costs are high. Therefore the SIA created Data



Sheet 2018, Seismic Review of Existing Buildings. That data sheet reviews the rehabilitation measures required on the basis of the proportionality between costs and benefits.

The 8-storey office building of the Agrisano health insurance company, originally built in 1975, was re-engineered for seismic safety as part of an overall refurbishment and improvement project. As part of this upgrading, the building was also raised by one storey. The new storey is of a timber construction and consequently only had a minimal additional influence on the structure and the earthquake refit.

Analysis of the seismic resistance under SIA Data Sheet 2018 showed that longitudinally the building could only absorb 10% of the earthquake impact required in the new structural standards SIA 260 to 267. Based on the proportionality of the acceptable risk and an evaluation of the investment costs versus the benefits, it was concluded that the best solution would be strengthening to raise the seismic resistance to 80% of the levels required by the latest standards for new buildings.

The StressHead CarboStress system from StressHead AG was selected for the earthquake refit to the office building. The system post-tensions CarboDur CFRP plates to a maximum force of 220kN. The tensioning force is anchored through the StressHead, a compact head also made of CFRP, and then through steel anchors which transfer the force from the StressHead into the structure. Since only masonry walls were available for the fixed anchors, the tensioning force was applied indirectly through the floor slabs. The walling of the existing building consists of two skins used both as insulation and as formwork for the subsequently placed concrete core. Specially made steel sections were required for anchorage in the floor slabs. On the tensioned side located on the ground floor, the force could be applied directly into the existing concrete walls.

To increase the seismic resistance to 80% of the potential impact, 16 post-tensioned systems with different tensioned lengths were necessary. Because the earthquake impact decreases in proportion to the building height, only 4 systems had to be extended over the full height of the structure. The other systems were then installed staggered storey by storey. The system was applied in an extremely short time. Including all of the necessary pre-treatment and post-treatment works, only 15 working days were needed for the complete installation of all 16 of the systems.

The corrosion resistance and low space requirements of the CarboDur CFRP plates and the StressHead CarboStress system were used to best effect on this example. On completion of the works it was therefore possible to cover and hide the systems within the 200mm of new external insulation. The earthquake refit is now invisible.

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