Office Building “Belvedere”, Prestressed in three Directions

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Summary

In 2013 the structural part of the office building “Belvedere” in Vaduz (Liechtenstein) was finished. The building consists of 6 floors (one underground) and is above ground 25m x 65m in plan. Supporting walls are offset 5,6 m from the edges of the slabs thus every slab bears a cantilever of 5,6 m. Only four cores of vertical circulation carry the load of the upper four floor slabs to basement and foundation. The building’s walls are arranged in two planes in chess board shape and are designed to support a 15-m-cantilever. Tendons have been used to reduce the deformation of the slabs and to reinforce the shear walls; hence the building is prestressed in three dimensions. A strut and tie model has been used to calculate the forces in the walls. Particular attention has been paid to the nodes of the trusses. These points were critical for the bearing capacity of the structure.

Keywords: post-tensioning; tendons; slabs; shear walls; cantilever; strut-and-tie model; node design

1. Introduction

In 2011 the first advisory group, an international financial service provider, decided to organize a competition for their new head office in Liechtenstein. In this location the company had rented different offices and wanted to unite all groups in one single building. The architects Hildmann Loenhart Mayr together with the authors won the competition with the building described in this paper.

Fig. 1: Side view of model during competition

2. Description of Structure

The building consists of 6 floors, one of which is underground and below groundwater table. It contains the employees' parking lot and some infrastructures as archive, server room and garbage collectors. The ground floor includes the entrance area and some customer reception offices as well as the restaurant. The clerks' offices are located on floor one to three. These floors emerge over the ground floor and thereby create the airy impression of the building. Whereas the building so far is a reinforced concrete structure the last floor (walls and roof) is made of timber and steel. The roof gives sight for the directors on certain mountain peaks of the Swiss and Liechtenstein's Alps. Therefrom the building's name has been derived: "Belvedere" (nice view), see Fig. 1.

The size of the rectangular upper floors is 25 x 65 m and the underground parking 50 x 85 m. The façade itself is recessed with glazing on all sides. The building does not have an expansion joint; first of all the plates need to transfer normal forces due to the global structural behaviour, see below. Moreover the approach of integral construction follows recent research results as documented in [1], [2].

Four cores with stairwell and lift are arranged to support the building horizontally (wind, earthquake) and vertically. They integrate into two rows of walls laid out in chess board manner. On ground floor level the number of walls is reduced to a minimum. The material of walls and slabs has been chosen to be concrete C 35/45. Yield strength of steel is 500 N/mm² and prestressing steel is of quality Y 1770.

3. Challenges and Solutions in Structural Design

3.1 Slabs

3.1.1 Deflection

Every vertically supporting element is offset from the rim of the slabs along the building length by 5,6 m and along the building's short side by 3 m. Thus every slab of the upper floors has cantilevered edges all around. The room-high glazing is placed 1,4 m from the edge. The reduction of the slab deformation is crucial for the functioning of the façade and also for the visual appearance of the building. The regular floor thickness is 40 cm.

The maximum long term deflection of the slab edges without any intervention was determined to be 10 cm. This deformation could be reduced to 3,5 cm by the implementation of tendons in the building's transverse direction, see Fig. 2 and Fig. 3 and also [3].

In general the compressive force in buildings' plates due to prestressing is partially diverted by walls parallel to the tendons directions or by stiff cores. In the building under consideration however only few walls will take the horizontal force, so the main part of the compression induced by the prestressing will stay in the slab, especially in the cantilever part.

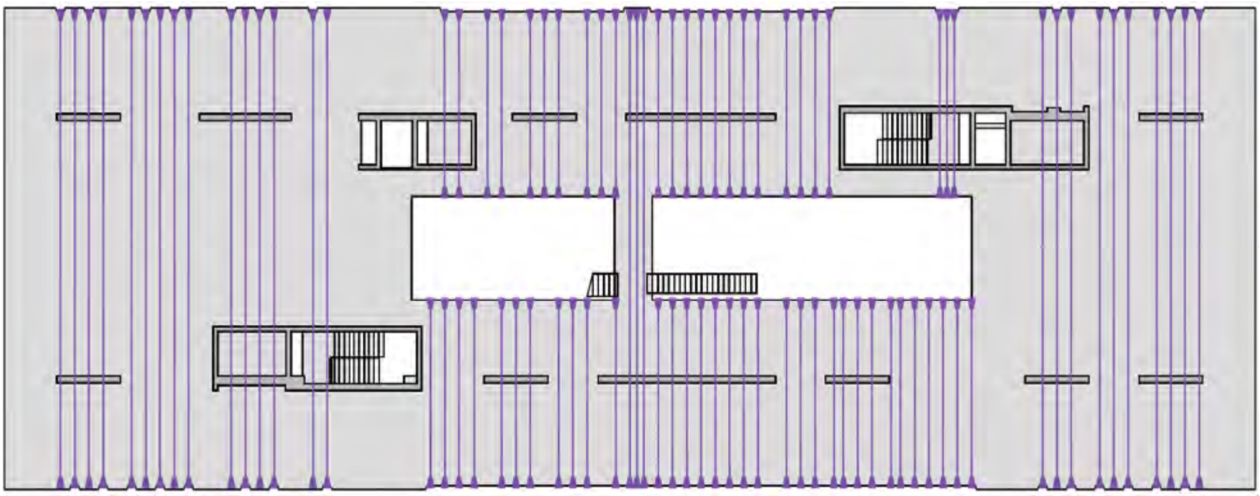

Fig. 2: Arrangement of tendons in ceiling above first floor

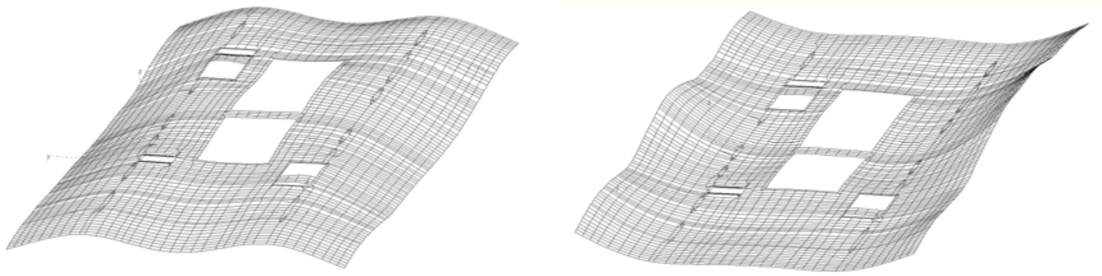

Fig. 3: Deformation of slab under self-weight (left) and by prestressing (right)

Considering this, under permanent load combination cracking could be prevented in most areas. As the relative deflection of the cantilevers is decisive for the façade and the absolute deflection for the visual appearance the relevant parameters of the implemented concrete were verified. Usually only the compressive strength is guaranteed by the contractor but for the estimation of deflection the tensile strength, modulus of elasticity and creeping and shrinkage are the important values. For this particular building the tensile strength and the modulus of elasticity of C35/45 have been controlled before construction at the specimen's age of 28 days, yielding an average tensile strength f_{ctm} = 2,9 N/mm² and a secant modulus E_{cm} = 37.100 N/mm², whereas a compressive strength of $f_{cm,cube,dry}$ = 60,1 N/mm² was measured (f_{ck} = 0,8*0,92* 60,1 N/mm² - 8 N/mm² = 36,2 N/mm² > 35 N/mm²). The tensile strength was less than expected (3,2 N/mm² according to EN 1992 or SIA 262 [5]) and the modulus of elasticity considerable larger. This demonstrates how important it is to verify other parameters apart from the compressive strength when planning a slim structure with stiffness oriented design methods [4].

Deformation measurements of the slabs were arranged for the time of removal of temporary supports and the following four months (further measurement are planned in future). The results show that the immediate deformation only a few millimetres and the construction (shear walls) was quite stiff. Also the deflection of the slabs was not more than 17 mm and after four months had increased by only 3 mm. Thus the long term deformation is expected to remain less than the calculated 35 mm.

3.1.2 Multifunctional slabs

The mentioned tendons are not the only insertions in the concrete slabs. Especially the upper ceiling contains also the building component activation, waste water ducts, electric cables, ventilation ducts, the sprinkler system and of course four or more layers of reinforcement. So it is not just for static reasons that the slab above the third floor needs to have a 50-cm-high cross section.

3.2 Shear Walls

The second challenge for the structural engineer was the design of the shear walls as they were supposed to realise the overhangs of the construction of 10 and 15 m at the buildings ends. The walls were arranged in chess board shape and only 36 cm overlap of the walls between two floors were given for architectural reasons, see Fig. 4.

With a wall thickness of 35 cm the capacity of the shear walls and especially the transfer of forces from one wall to the next were the cruxes of structural design in the lower floors. Bonded tendons were arranged in the walls as space allowed, see Fig. 5. They helped to reduce the compression forces in the nodes but did not carry the whole load.

Fig. 4: Side view of building “Belvedere” at end of structural work

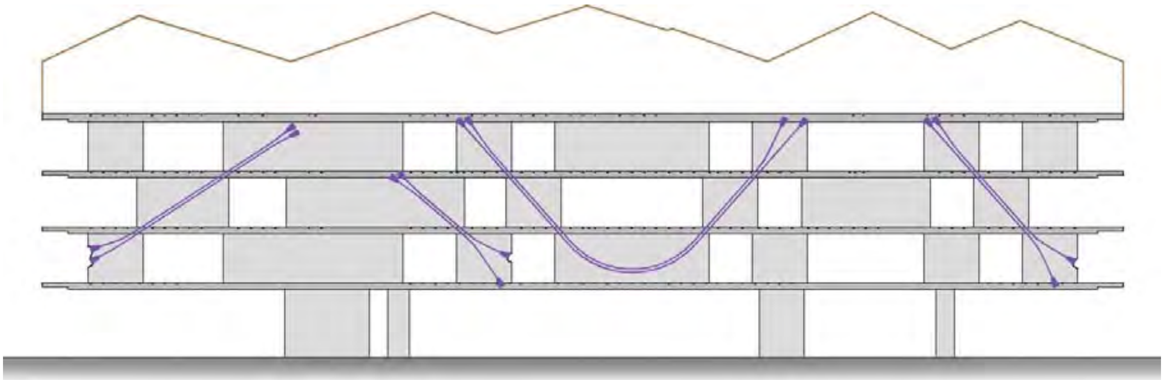

Fig. 5: Bonded tendons in shear walls

The finite element model of the structure gave a value of the deformation but did not help to verify the capacity of the nodes. Therefore the strut-and-tie method - introduced in [6] - was applied for the design of walls and nodes, see Fig. 6. This model yielded the design forces of each node. The node capacity was determined according to the relevant Swiss code [5].

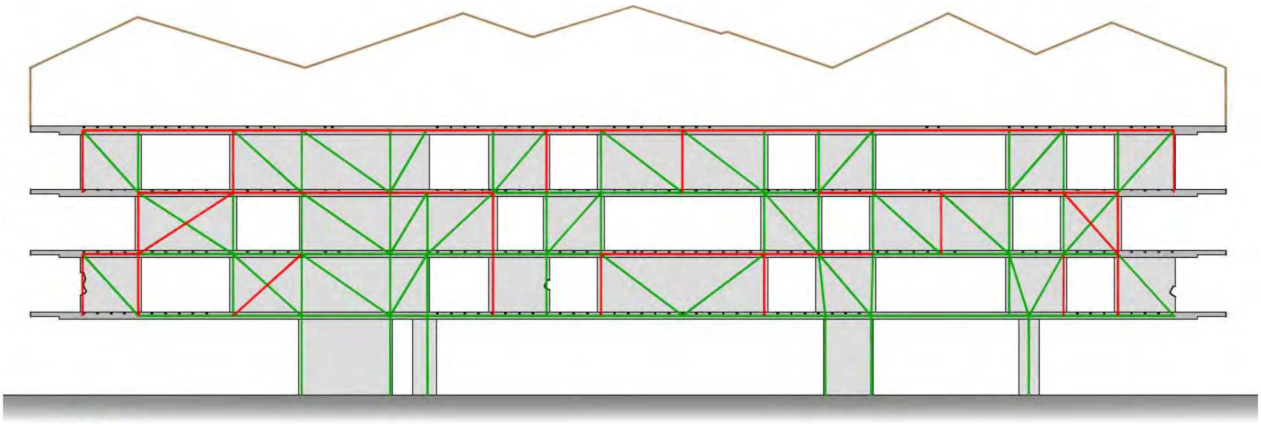

Fig. 6: Simplified strut-and-tie model of one shear wall axis

In several nodes the compression strength of the concrete alone war not sufficient to resist the acting forces. Thus reinforcing bars were introduced to carry a part of the load through the node. In one node even a steel cross was arranged to transfer the forces between two walls see Fig. 7.

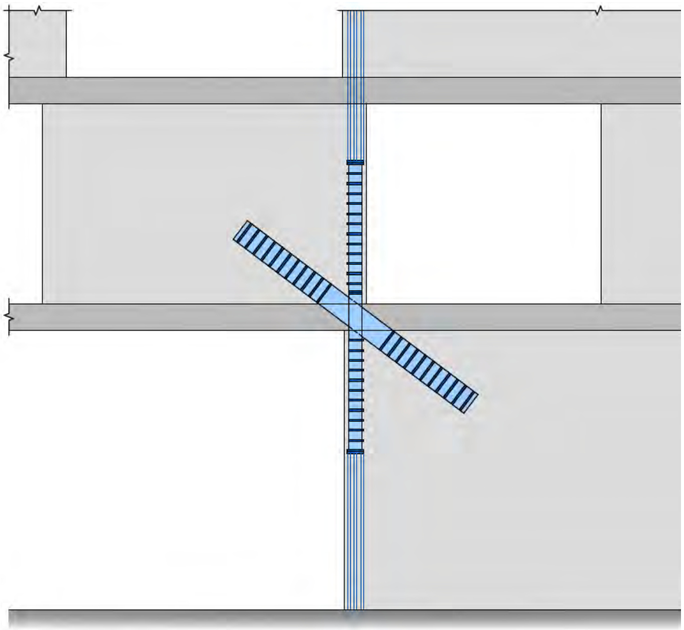

Fig. 7: Steel inlay for truss node

In some shear walls not only the nodes but also the compressive force of the diagonal strut was larger than its resistance. This was because the ultimate compressive stress had to be reduced according to the Swiss code SIA 262 [5] by 40% due to the conventionally vertical and horizontal reinforcement not being aligned with the main stress directions. To improve the situation and solve that problem the reinforcement was turned parallel to the diagonals in order to be able to apply the factor of 0,8 instead of 0,6, see Fig. 8.

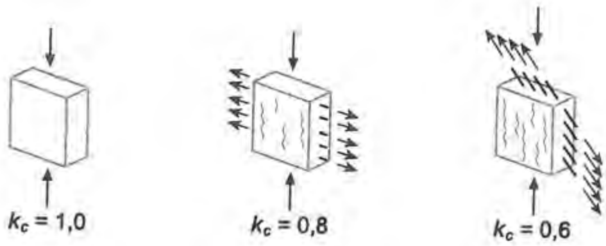

Fig. 8: Reduction factor of SIA 262 [5] for compressive strength and shear wall with inclined reinforcement

The horizontal ties of the strut-and-tie model in the upper floors yielded values of 10 and 11,5 MN. To avoid excessive cracking in slabs and to make use of their higher strength steel tendons were introduced in the longitudinal direction of the building, see Fig. 9, unfortunately complicating the problem of the multifunctional slabs described in chapter 0.

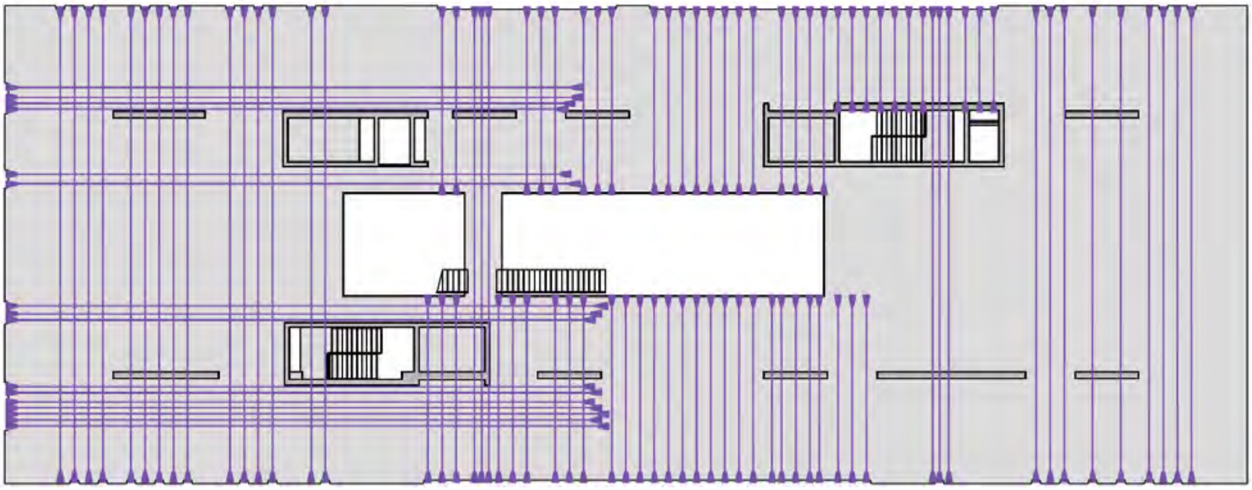

Fig. 9: Arrangement of tendons in ceiling above second floor

3.3 Experiences with Swiss Code SIA 262

Compared with the European codes the Swiss codes are rather compact. Thus the responsibility of the structural engineer is higher. For the designers of the project it was a positive experience. The codes gave the freedom to develop a non-usual building without too many normative restrictions. On the other hand, for more guidance eurocodes and other literature were consulted.

However the authors were somehow puzzled by the fact that in a certain points the verification of punching shear according to SIA 262 was achieved without any punching reinforcement, whereas according to European codes the resistance was not enough even with punching reinforcement and large amount of bending reinforcement. Yet it seems that the meanwhile introduced new Swiss code SIA 262 (2013) is closer to European codes.

4. Construction Process

The location of the building in the Rhine Valley implies large wind forces, fast temperature changes and a groundwater table of 1,5m under surface. A steel sheet pile wall was used to keep the water out of the construction pit and 54.000 litres of water per minute had to be pumped away constantly. The cast-in-situ drilled piles of diameter 60 cm had to prevent the basement from floating during construction (tension forces) and to support the structure after completion (compression forces).

Every slab was prestressed just after it reached the minimal concrete strength for prestressing after about 10 days. The prestressing forces lifted the ceiling from the scaffolding and its supports had to be adjusted right after. As described above the structure of the building did function only after the completion of the last concrete slab. Thus all floors needed to be supported until then. The support loads were much larger than the capacity of usual scaffolding piles; tree trunks and also temporary concrete columns were used instead.

After the completion of all floors and the prestressing of the slabs and shear walls the temporary bracing could be removed. The removal started on ground floor level to avoid excessive compression in the temporary supports in the load bearing axis. The concrete columns under the cantilever were unloaded by hydraulic jacks on a bracing system and thereafter taken out, see Fig. 10. The building's ends could then be lowered slowly and safely. The deformation was monitored during the removal of the temporary supports. With a maximum value of 5 mm the deflection was less than estimated (11 mm).

Fig. 10: Demolition of temporary concrete columns

5. Conclusions

The owner could be persuaded to trust a slim and "athletic" structure. An office building with prestressed slim floors was established in Vaduz (Liechtenstein). Important therefore is the precise control of the deflections and hence the demands from the execution phase are greater compared to usual structures, because additional concrete properties have to be monitored and guaranteed (modulus of elasticity, tensile strength). The reliability of the prediction of deformations can be improved by the restriction of the concrete tensile strength in plates and walls.

Tendons in shear walls and the design of the wall using a strut-and-tie model lead to a structure with chess board shaped shear walls and cantilevers of the building of up to 15 m. The unconventional inclination of the reinforcement of some shear walls and a welded steel cross were necessary to transfer the building's load into the four cores and achieve the floating impression of the four upper floors.

6. Acknowledgements

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