Reconstruction of Special Types of Constructions

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ABSTRACT

The silos are cylindrical with a diameter 36.0m with a total height 50.5m. The silo shell is made from steel plates of variable thickness. Capacity of the silo is 60 000t. Gradually, during more than 30 years of operation, local deformations and distinctive 6000mm high cracks in the bottom part of the shells of the silos developed. On the basis of diagnostic inspections and control static calculation a strengthening was proposed. To increase working life of the clinker silos, the shell strengthening over the whole height of the cylindrical body by means of horizontal rings and vertical stiffeners was designed. The total S355 steel consumption for strengthening of one silo shell is 300t.

Original roof structure of the winter stadium in Zvolen was made of timber glued beams with span of 55.5m. During years of serving to its purpose occurred some problems. Diagnostics showed that construction is inconvenient for the ultimate limit state and serviceability. Deflection of the roof construction in the year 2005 was more than 500mm. Authors proposed strengthening of the original beams (overall 34pcs) by prestressing.

KEYWORDS: silo, shell, diagnostic inspections, static calculations, strengthening, stiffeners, roof structure, timber beams, prestressing

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APPRAISAL OF THE TECHNICAL CONDITION OF THE STEEL STRUCTURES OF CLINKER SILOS PC1 AND PC2

Introduction

The structure consists of the actual silo with a dump, a superstructure and of a communication tower with a gangway. The silos are cylindrical with a diameter 36,0m and a height 41,4m. The silo shell is made from steel plates of variable thickness. Capacity of the silo is 60000t. The silo is built on a reinforced concrete ring. In the bottom part of the silo there is a reinforced concrete ring with 400mm width and 1000mm height. The distance between the inner edge of the steel shell and the outer edge of the ring is 500mm.
Gradually, during operation, local deformations and distinctive 6000mm height cracks in the bottom part of the shells of the silos developed. In silo PC1 also the bottom load-bearing ring ruptured at the vertical crack location.

The aim of the appraisal was to determine the cause of these serious defects and to propose actions in order to assure the working reliability, the safety and to prolong the physical serviceable life of the steel structures of the silos PC1 and PC2.
The contractor did not have the original complete project documentation and the static calculation of the load-carrying structure of the silos at the disposal.

Detailed Diagnostic Inspection of Clinker Silos PC1 and PC2

Detailed diagnostic inspections have been performed in March and May 2006 by workers of the Faculty of Civil Engineering, Slovak University of Technology in Bratislava and were aimed mainly on:

- determination of the real geometrical shape of the shell roof and the foundation structures of the silos,
- determination of the thicknesses of the shell plates, ultrasound measurements,
- determination of material quality, non-destructive tests,
- determination of the technical state of the reinforced concrete structures,
- identification of structural details of the silo's steel structures,
- specification of places of anchorage of the technological equipments producing additional loading,
- assessment of mechanical damage, local and global deformations of the load-carrying parts of the silos.

Technical Description of Clinker Silos PC1 and PC2

We can divide the steel structures of the above mentioned facility (Figure 1) into two groups, according to the purpose and character of the structure:

- steel silo,
- other steel structures.

Fig. View of Clinker Silo PC2 with Technology

The steel silo consists of a wall and roof shell and ends with an upper compression ring at the height + 52,50. The shell is considered as a thin-walled cylindrical shell. It is made of plates with thicknesses 33 – 13mm, welded with vertical and horizontal butt welds. The shell is at the bottom part fixed into the foundation and introduces into it compression, a horizontal force and a moment. Loading of the shell comprises the roof, pressure of the clinker filling, technology on the silo, wind and temperature (clinker filling has 90 ºC).

Extreme stresses resulted from the static solution at the bottom edge in the nearest vicinity of the silo anchorage. The original project assumed that when direct effect of the pressure of the material over a certain height from the anchorage will be prevented, the extreme effects will considerably decrease. For this purpose a reinforced concrete monolithic ring with 40cm width and 100cm height was created in a distance of 50cm from the steel shell, which had to carry the pressure of the material and to prevent the direct influence of the temperature on the bottom ring of the shell. A considerable smoother deformation in the vertical direction should have been the result and so a decrease of the moment influence.
The roof is constructed from profiles connected to the bottom tension ring and to the upper compression ring. Into the compression ring the communication tower is anchored, which exhibits also horizontal forces. The cladding is made from a plate, which was cut in form of a trapezoid, laid and welded to the supporting profiles.

The communication tower is anchored at the ground level into a massive foundation and anchored against horizontal actions at the level + 43,0 into the bottom chord of the silo roof. Acting on the tower is also an additional load from the steep conveyor, which is anchored to a cantilever approximately situated at the level + 35,00.

The gangway, which connects the communication tower with the silo superstructure, is anchored to the tower and to the superstructure and is made from two truss girders with a lower bridge deck and a stiffener at the upper chord. It is a classic conveyor bridge.

The silo superstructure has a relatively complicated structure, because it is placed on the upper circular ring with a diameter of 9,0m and at the same time the axial system of the external load-carrying structure is 9,0m, which results in an inconvenient position of the corner columns. The grid and the floor at level + 52,50 is stiffened in such a way, that this system is able to carry the relatively large actions from the technological equipment.

During operation (more than 30 years) two vertical cracks with a length of ~ 6,0m developed in the bottom part of each silo. Views of the repair of the last crack in the clinker silo PC1 are on Figures 2 and 3.

Fig. 2 View of the Crack in Silo PC2 (November 2005)

Fig. 3 View of the Finished Repair at the Damaged Location of Clinker Silo PC2

Control of the Integrity of Load - Carrying Structures of Clinker Silos PC1 and PC2

Detailed diagnostic inspections of structures were performed in March and April 2006. Controlled were the following:

- shell and roof of silos PC1 and PC2, anchorage into the foundations,

- exit from the communication tower,
- connecting gangway from the communication tower,
- superstructure and layout of the technology,
- dump to the small silo.

During the control it was found out that:
- the load-carrying structures are integral and no member was removed by non-professional interference,
- two cracks emerged during operation at the bottom side of the shell of silos PC1 and PC2, which were professionally repaired.

Plate Thickness Measurement of the Roof Shell

Shell thicknesses of silos PC1 and PC2 were measured by a digital ultrasound thickness meter Digital Wall Thickness Meter DM – 3 with precision of 0,05mm. After measurement we succeeded to obtain the original projected plate thicknesses of the shell and the roof.
The plate width in strip 1 is ~ 2,0m; strips 2 to 16 have a width of 2,5m and strip 17 is ~1,5m wide. The probes at the individual locations were marked. At each probe the thicknesses were measured in 3 places, for verification the mean value from these 3 measurements was taken.

The largest corrosion losses were at the connection of the bottom plate strip of the shell with the base plate (Figure 4). After cleansing a value $t \doteq 30{,}1$mm near the weld was measured at probe S3. Measurements of the roof plate were also carried out. The mean value of the 3 probes is $t \doteq 6{,}2$mm.

Fig. 4 State of the Shell in Contact with the Base Plate

Material Quality Measurement, Non - Destructive Method

Before measurement, the place of the probe was cleaned to bare metal. Hardness tests were carried out by a dynamic hardness tester EQUOTIP PICCOLO on the silo shell and on the roof plates.

We consider the measured data only as informative. The real material quality can be determined only by tensile tests of the steel (destructive). Bottom strip 1 and upper strips

14 to 17 and the roof plates were measured. The equipment statistically evaluates the individual measurement sets.

Measured values:
Bottom strip 1 $f_u \doteq 421MPa$
Strips 14 to 17 $f_u \doteq 435MPa$
Roof $f_u \doteq 385MPa$

These values correspond to material quality S275, where $f_u \doteq 430MPa$ and $f_y \doteq 275MPa$. The roof approaches material quality S235, where $f_u \doteq 360MPa$ and $f_y \doteq 235MPa$.

Control of the Technical State of Reinforced Concrete Structures and of the Silo Anchorage

Controlled was mainly the anchorage of the silo shell to the reinforced concrete foundations. In the case of silo PC2 the anchor bolts and the whole anchorage is not accessible. For control the anchorage of silo PC1 was made accessible.

At some places the grout under the base plate is damaged and the shafts of the anchor bolts are visible and the anchor bolts are corroded.

During repair, these places must be cleansed and grouted, so their static function as well as anticorrosion protection will be assured.

From the results of the diagnostic inspection follows that:

- the height of the "dead" clinker supplies in silo PC1 reaches up to 13,8m; the dead mass is non-uniformly distributed along the walls with heights of 6,6 to 13,4m.
- the measured, real plate thicknesses mainly in the bottom part of the silo are smaller than the projected ones. During operation a wear of mainly the internal side of the shell occurred and the original thickness is decreased also by corrosion losses. The largest corrosion losses were determined at the connection of the bottom shell plates to the base plate; the real thickness was tp =30,1mm against the original one 33mm. The loss is up to 3mm.
- the strength of the silo shell, determined by a non-destructive method, by a hardness test with a dynamic hardness tester EQUOTIP PICCOLO reaches only 421 ÷ 435MPa against the expected one 520MPa (steel "52"), that is only 81 ÷ 84%. The real mechanical and technological material characteristics of the shell can be determined only by test specimens taken from the silo shell.

During diagnostics a geodetic and photogrammetric spatial survey of the real shape of the clinker silo shells was performed.

- height differences of the bottom steel ring are at silo PC1 -4,0 to 50,0mm; the upper edge of the foundation strip is not horizontal. The ring of silo PC2 was not accessible for control.
- deviations from the theoretical shell diameter r = 18,0m had been found out in various heights. At silo PC1 the maximal deviations are in profile B -57mm (into the silo's interior) and +94mm (out of the silo). In profile F the silo shell is deformed into the silo's interior -76 to -29mm. At silo PC2 the largest deformations are in profile I -50 and +105mm. Because of the magnitude of deformations, it is necessary to make once

in a year a control of the geometric shape of the silos (on the set marks) and in the case of an increase of deformations to take adequate action to ensure silo operation.

Fig. 5 Local Deformation of Silo PC1 Shell

Fig. 6 Local Deformation at the Damaged Location of Silo PC2

On the basis of diagnostic inspections, survey of the real silo shape, geotechnical assessment of the foundation and control static calculation a strengthening by means of horizontal rings and vertical stiffeners to a height 15,13m above the horizontal foundation ring was proposed. The total steel S355 need for strengthening one silo was 183,0 t.

Two samples were taken from each of the silos to specify the mechanical steel properties of the silo shells after evaluation of the appraisal results. The results of the tests confirmed that the material of the silo shell is steel of grade S355, its fracture toughness is very low and it is susceptible to brittle failure.

Considering the structural importance of the clinker silos and to increase their working life, the client agrees with the shell strengthening over the whole height of the cylindrical body.

Control Static Calculation of the Clinker Silo Structure

Static Verification of the Silo

Assessment of actions:

- Calculation of actions from the stored material – according to DIN 1055 (year 1987)
- Temperature actions – according the measured values provided by HOLCIM.
- Non-uniform settlement of foundations – modelling of the non-uniform settlement of foundation structures were done in the „cosine“ shape, where the maximum value of amplitude was determined on the basis of survey as ±10 mm.
- Other actions – these actions were determined according to STN 73 0035 (1980) – Actions on structures.
- Partial load factors and combination factors – these values were determined according to STN 73 0035 – Actions on structures.

Calculation model:

- silo diameter = 36,0 m
- height of the silo wall = 41,1 m
- height of the conical roof = 9,4 m
- total silo height = 50,5 m

The aim of the calculation was to verify the existing clinker silo structures, to determine possible causes of failures and to propose necessary adaptations of the clinker silo structures. The load-carrying silo structures were checked according to the ultimate and serviceability limit states.

Fig. 7 Structural Model of the Silo

Preliminary static calculation of the original structure was performed on the basis of theory of elasticity in programme MathCad.

Detailed strength calculation was performed with programme IDA NEXIS 32, 3.60.15 from SCIA company. A 3D plate-wall model of the steel structure of the clinker silo shells was used.

The results of the preliminary and the detailed calculation of the original structure agreed very well.
Results of the calculation:

- The stress state of the structure has been evaluated on the basis of equivalent Von Misses stresses. The maximal stresses were calculated for the existing state of the structure and also for the new strengthened structure.
- The control calculation of the original structure of the clinker silo has shown that the maximal equivalent stress in the structure taking into account all actions and their combinations exceeds in some areas of the shell the design strength of the silo material S 355.
- By strengthening of the original clinker silo structure by a system of rings and vertical stiffeners a sufficient decrease of the maximal level of the equivalent stress in the structural shell was achieved. That means that the maximal equivalent stress of the strengthened structure and of its new parts (stiffening members) does not exceed the design strength of the silo material S 355.

Stability Verification of the Silo

The check was performed according the Slovak standard STN 73 1401:1984. Stability of shells is dealt with in Annex VI. There are different cases covered in this Annex.

Unstiffened shell with inner pressure:

- The silo loaded with clinker and non-uniform settlement was up to 2/3 height not satisfactory.

Shell with longitudinal stiffeners (without inner pressure):

- The silo loaded with clinker and non-uniform settlement was in the middle part not satisfactory. This is because the positive influence of the inner pressure was not taken into account (STN does not cover this case). That is why we then took the greater resistance of the stiffened shell without inner pressure or the unstiffened shell with inner pressure (but with stresses on the stiffened shell). The silo was found satisfactory (only in some strips the resistance was exceeded by maximum 5%, which will be offset by the influence of the inner pressure).
- The longitudinal stiffeners are checked for bending and axial compression as beam columns. The bending moment distribution was found on a strut 3D model which took into account the favourable circumferential tension into account by modelling “fictitious” rings that carried tension (i.e. the local bending moments were reduced in comparison with a continuous beam supported by ring stiffeners only). The circumferential tension had also a favourable effect on the buckling lengths. They were also calculated on the same 3D strut model.

Horizontal ring stiffeners:

- The ring stiffeners have to have adequate stiffness to provide support for the curved panels and longitudinal stiffeners as well. The dimensions of the ring stiffeners were calculated according to Buckling of Steel Shells, European Recommendations, ECCS 1998. The stress Sigmax was taken as 1.2-times the design value of buckling compressive stress (of the cylinder stiffened by longitudinal stiffeners, buckling between the ring stiffeners – see the ECCS publication).

Proposal of Silo Shell Strengthening

The steel silo shells will be strengthened by a system of vertical stiffeners and horizontal rings. After strengthening the cylindrical shell will act as an orthotropic (orthogonal anisotropic) shell.

Horizontal rings are divided in primary and secondary. The primary rings have an angular shape, they are made of web (400.12 - 115768) and flange (250.25 – 115768).
There are five intermediate rings and one upper ring, which will be situated closely bellow the silo roof level.

The secondary rings are from flat steel 250.25 – 113154. The bottom ring is located over the foundation ring, the other 4 secondary rings are always minimally 200mm above the existing horizontal weld. This rule is valid also for the location of the primary horizontal rings.

The vertical stiffeners are from flat steel with variable dimensions. They will be located around the outer silo perimeter (totally 80pcs) in spacings ~ 1514mm. Their minimal distance from the original vertical welds is 150 to 200mm.

The dimensions of the vertical stiffeners (in the upward direction from the foundation ring):
- Part 1 300.30 (~ 7000mm)
- Part 2 300.30 (~ 7500mm)
- Part 3 300.30 (~ 7500mm)
- Part 4 300.25 (~ 7500mm)
- Part 5 220.22 (~ 5750mm)
- Part 6 180.16 (~ 5750 mm)

The total S355 steel consumption for strengthening of one silo shell is 300 t.

To execute the proposed strengthening of the silo shells it is necessary to work out:
- detailed workshop documentation of the structural strengthening elements,
- precise and complete the welding specifications for strengthening and a
- detailed erection procedure.

Fig. 8 Reconstruction Works on the Clinker Silo

Fig. 9 Detail of the Strengthening in the Anchorage Area

RECONSTRUCTION OF THE ROOF OF THE ICE STADIUM IN ZVOLEN

Introduction

The ice stadium in Zvolen drew attention in the beginning of 2006, when the safe service of the building was threatened. The roof structure reached a 550mm deflection in the middle of the span. The original roof structure, made of glued timber girders was not satisfactory for the ultimate and serviceability limit states. The proposed reconstruction strengthens the original girders by means of prestressing.

The structure consists of glued timber girders fixed on the left side to a timber stay and on the right side to a reinforced concrete structure. The length of the timber girder is 55.5m, the height 2.6m and slope is 12.53°.

Description of the Technical State Before Reconstruction

Control inspections have been made of the timber roof structure of the ice stadium, which accompanied the data obtained from the client. During the inspection the basic dimensions of the roof structure in transversal and longitudinal direction as well as the dimensions of some selected roof girders were checked.

The main cause of the extensive deformations (the measured value in 2006 was 463.5mm) was an insufficient resistance of bolted connections.

Girder Strengthening

Every original timber glued girder with a total height 2.6m (totally 34 pieces) was strengthened by means of prestressing according to Figure 10. The assembly of the new steel members of anchor brackets and tie rods was performed from the inside of the building (Figure 11).

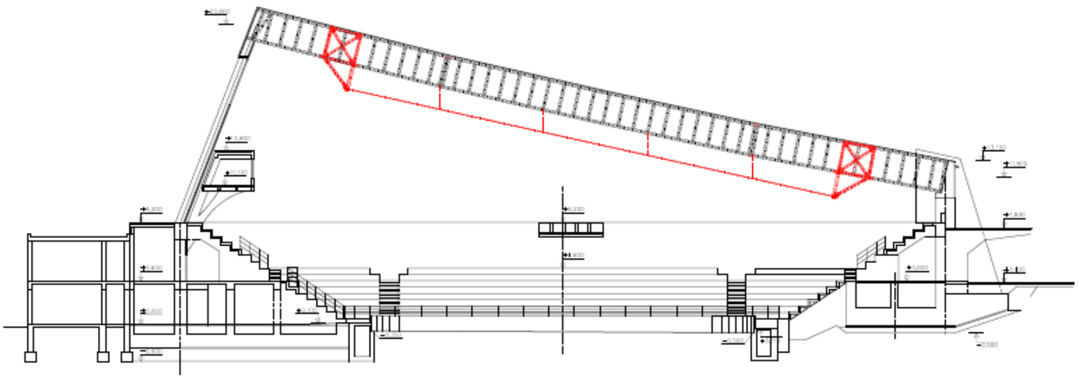

Fig. 10 Strengthening of the Timber Girders by Prestressing

The steel truss anchor brackets were manufactured of steel S355. The anchorage trusses were supplied to the site as individual members from rolled U-sections of light weight. The members of the anchor brackets have been connected to the timber girders by means of steel bolts with diameter 24 and 32mm. The field joints of steel members were bolted to gusset plates with M16/10.9 bolts.

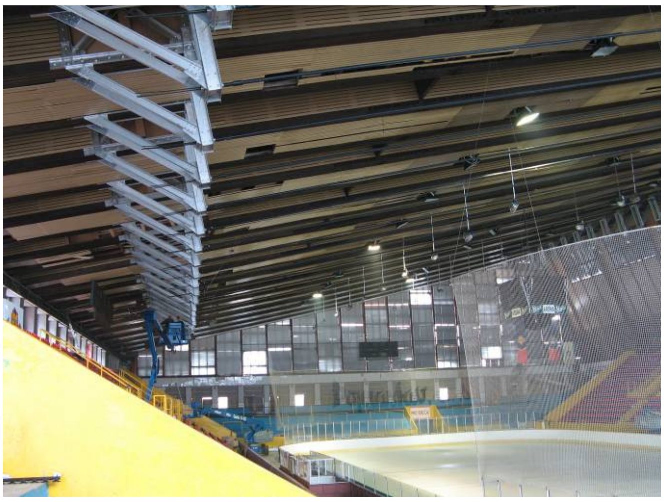

Fig. 11 View of the Reconstructed Roof Structure

The prestressing tie rods have been situated approximately 2.0m under the bottom face of the timber girders. The tie rods are situated in longitudinal axes of the timber girders. The self weight of the prestressing tie rod is carried to the timber girders by vertical hangers located in 4 points along the length of the tie rod. At the design stage rods MKT460 SYSTEM of steel S460 have been proposed. Because of long delivery times Triostrand cables were used, which have been available at that time.

The prestress to a maximal axial force in the tie rod S_{vRd}=184kN has been applied gradually in four stages. The assumed loading during prestressing was dead load of the girders and the load of the roof cladding. Stressing was performed at the bottom anchor bracket (Figure 12). The prestressing forces are in Table 1.

Fig. 12 Anchorage of the Cable Tie Rod to the Steel Brackets

Table 1 Prestressing Forces

Stage	Cable force [kN]	Axial force in the upper chord in the middle of the span [kN]	Axial force in the bottom chord in the middle of the span [kN]
		Combination = dead weight + cladding + prestress	
1	45.9	-438.8	373.5
2	91.8	-403.6	304.2
3	137.7	-368.7	232.6
4	183.6	-333.5	163.9

Measurements and Results of the Measurements

The forces during prestressing have to be measured on each prestressing cable. The strain gauges have been glued directly to the prestressing cables. The prestressing force diagram is on Figure 13. We can see how the prestressing force disappears in the structure. The cause can be the deformation and creep of the girder. Therefore it was necessary to additionally prestress the cables to the prescribed force.

Besides the cable forces also the vertical deflection of one girder was measured during prestressing. The measurement equipments have been located at the bottom girder connection. The deformation of the opened connection and the vertical girder deformation have been measured as well.

It has been determined during the measurement, that the cracks do not close during prestressing and that the girder has raised 13.42mm at the location of the bottom connection. This deformation was smaller than was expected previously. It could be caused by deformed original fastening devices at the girder connection.

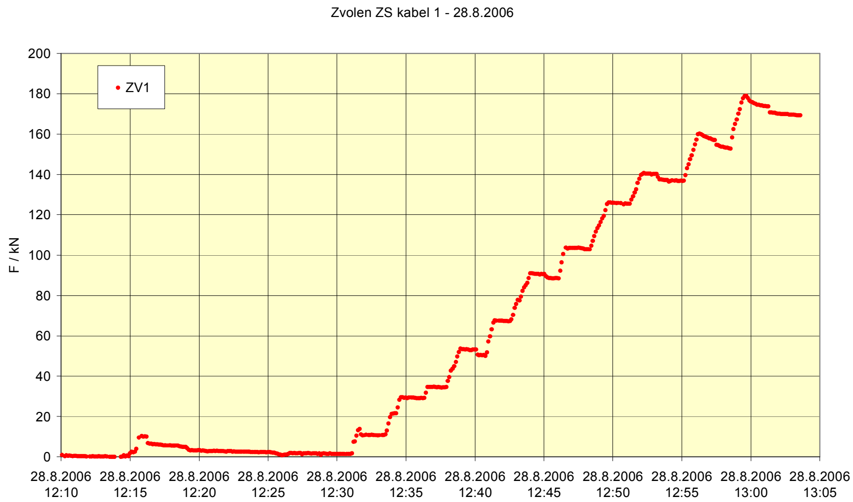

Fig. 13 Forces in the cable during prestressing

Conclusion

Because of the age of the structure, the defects and the fact that the use of glues was not examined enough at the construction time even at the European level, it is complicated to determine the service life of the structure, which is mainly influenced by the durability of the glue in the flanges and mainly in the flange-to-web connection, where two materials join. The state of the structure after strengthening and the state of the individual girders will be regularly checked in the future.

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