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CONDITON OF STRUCTURES AND PROPERTIES OF CONCRETE OF AN EXISTING OIL SHALE CHEMICAL PLANT

M. KIVISTE[(a)*], J. MILJAN[(a)], R. MILJAN[(b)],
M. KIPRUSHENKOV[(a)]

[(a)] Institute of Forestry and Rural Engineering, Estonian University of Life Sciences, Kreutzwaldi 1, Tartu 51014, Estonia
[(b)] Institute of Economics and Social Sciences, Estonian University of Life Sciences, Kreutzwaldi 1, Tartu 51014, Estonia

A case study for investigating the condition of concrete structures and properties of concrete of an existing oil-shale chemical plant is presented. The condition of concrete structures in the plant (constructed in 1951) was assessed visually on a six-point scale. It was found on visual inspection that concrete structures with cracked or spalled concrete cover need extensive repairs. Compressive strength of cores, carbonation depth, cover, water absorption as well as sulphate, chloride and nitrate content in concrete were determined. According to the results suggestions were proposed to repair deteriorated concrete structures in the plant.

Introduction

The production of oils alternative to petroleum has received worldwide attention in regards to increasing prices of fuel. One of these alternatives is producing oil from oil shale. The production of oil shale oil is successfully competing with oil products and is gaining growing significance [1]. The production of oil from oil shale is now developed in China [2], Bulgaria [3], Brazil, Jordan, Australia, etc.

The effects of pulverized-fired oil shale on the technological equipment in Estonian power stations have been studied in companies involved in production of oil shale. High-temperature corrosion resistance of a number of boiler steels was tested experimentally in laboratory and industrial conditions in the presence of chlorine-containing external deposits [4-6].

However, the deterioration of concrete structures and infrastructures is also a widespread problem in many countries. In order to assess such structures for

* Corresponding author: e-mail *mihkel.kiviste@emu.ee*

continued future service, simple and practical tools need to be developed for evaluating their reliability and performance. Several studies have been dealing with condition and reliability assessment of concrete structures in nuclear power plants [7–9] or bridges [10–12]. However, no attention has been given to the effects of the gases and phenols that arise from oil shale retorting on the load-bearing concrete structures of a chemical plant. The aggressive environment inside the plants, which manufacture shale oil could affect adversely to the material properties and structural capacity inside the building. The residual flexural and shear capacity of the concrete load-bearing structures in the studied plant was determined analytically earlier [13]. This paper presents a case study for investigating the condition of concrete structures and properties of concrete of an existing oil-shale chemical plant.

In order that structural condition be predictable the defining attributes and properties must be quantifiable [14]. For that reason visual inspection of load-bearing concrete structures in the plant was performed. Also, on the basis of visual assessment concrete structures with most severe deterioration were located for subsequent investigation. The deterioration mechanism is analytically divisible into two factors that combine to produce a specific mechanism: 1) the inherent properties of the specific material or system and 2) the atmosphere or environment in which those properties are operating [14].

Compressive strength as the most important property of concrete has been studied in most detail. With respect to durability, carbonation depth, cover, chloride, sulphate and nitrate content and water absorption of concrete were determined in order to have an overview of the inherent properties of concrete.

A brief description of the present and past situation of the indoor environment near generators of the plant is provided. It was found that serious deterioration of load-bearing concrete structures in the studied oil plant building may originate already from the 1950-ies. At that time due to different production technology slag was removed from all generators each day. The columns and beams were exposed to a large concentration of gases exiting through the hatches of a generator during slag removal.

According to the results suggestions are proposed to repair deteriorated concrete structures in the plant.

General description of the generator building

The studied generator building is located in North-Eastern Estonia. The seven-storied building measuring 64 × 15 m has been almost constantly in service since the construction in 1951. The generator processes 1.4 million tonnes of oil shale every year. Oil shale is processed in the plant's generators to produce shale oils, fuel oils and resins. There are 12 generators in the plant. The flow sheet of generator involves transportation of oil shale to the bunkers located in the upper part of the building from where it is led to the generator. In the generator shale oil is separated by the process of retorting. The generators are continuously filled and emptied. The vertical position of the generators allows them to empty by gravity pull. The conveyor under the generator leads semicoke out of the building. The structures carrying generator are located on the 1st and 2nd floors. As semicoke exits the generator immediately after burning, it is quenched with water which produces a lot of gases (Fig. 1).

The mixture of gases contains carbon dioxide (CO_2), sulphur dioxide (SO_2), hydrogen sulphide acid (H_2S), phenols, aliphatic hydrocarbons, methane, ethane, propane, butane, etc. According to classification [15] these gases are of type I and II, which react with $Ca(OH)_2$ to neutralize concrete or produce salts. These salts generate III type of corrosion, which damage the structure of concrete [16].

The conveyor is located on 1st floor extending across the full length of the building. The columns and beams on the 1st and 2nd floors are directly exposed to the gases from the conveyor since the 1st floor is only partially separated from the 2nd floor by a concrete ceiling. The condition of several columns and beams on the 1st and 2nd floor has raised a concern on the durability of those structures.

The cooling of the processed oil shale takes place on the 3rd floor. There is no leakage of gases on the 4th, 5th, 6th and 7th floors. Oil shale is loaded from the bunker onto the generator on the 6th floor. On the 7th floor oil shale is loaded onto the bunker.

The bearing structures of the generator building perform as a monolithic concrete frame the columns of which are made of concrete mark M110 (10.8 N/mm^2) and beams are made of concrete mark M140 (13.7 N/mm^2). Steel reinforcement of class A-I (smooth, yield strength 210 N/mm^2) has been applied both in the beams and columns.

Fig. 1. Concrete structures exposed to the gases generated in water quenching of semicoke.

Experimental and analytical methods

Visual inspection, concrete tests and chemical analysis were performed from February to December 2006. Additional cores were drilled and tested in April 2009.

Visual inspection

The purpose of visual inspection was to: 1) classify the structures according to visually discernible corrosion damage and 2) point out the structures with most severe deteriorations. The elements of the monolithic concrete frame were assessed as individual structures. All concrete columns and beams in the plant were assessed visually on a six-grade scale (Table 1) developed at the department of Rural Building of Estonian University of Life Sciences in the 1970-ies. Grades reflect visually discernible changes in the functional state of the structures on the basis of the condition of steel reinforcement and concrete cover. If even one feature of a lower grade can be determined during the inspection process, this lower grade is assigned to the structure.

Table 1. **Classification of deterioration states of concrete beams and columns [17]**

Grade	Description of condition
5	No corrosion detected
4	Less than 20% of stirrups are corroded (cracks or spalled concrete cover)
3	More than 20% of stirrups are corroded
2	Micro-cracks (width 0-0.3 mm) in the concrete cover of the main reinforcement
1	Cracks (width >0.3 mm) in the concrete cover of the main reinforcement
0	Concrete cover of the main reinforcement has spalled

Compressive strength of concrete cores

The concrete core test was based on Estonian National Standard EVS-EN 12504-1:2003 [18]. In order to determine the compressive strength of concrete 55 cores with diameter of 75 mm were drilled from columns and beams. 36 cores were acquired from the 1st floor (columns) or the 2nd floor (beams) where highest structural loads and most deteriorated structures were present. Cover meter was applied to locate the reinforcement in the structure before drilling. This generally enabled extraction of cores from a such location that they contain no reinforcement.

After that the non-destructive rebound hammer test was conducted. However, the methods and results of the rebound hammer test in this study are omitted. The reasons are briefly stated in discussion.

Cores were cut by means of a rotary cutting drill with diamond bits. The device was properly attached to the beams and columns to prevent shaking during drilling. In this manner, cylindrical specimens were obtained which were marked, brushed with phenolphthalein solution for carbonation depth measurements and transported to testing laboratory. The authors managed to acquire 29 cores from columns and 26 cores from beams. The ends of cores were ground or capped with rapid-hardening cement. Each core was measured in accordance with EVS-EN 12504-1:2003 [18]. Mean cross-sectional area was calculated from five diameter measurements, and the mean height of core was calculated from five height measurements. The core was tested according to EVS-EN 12390-3:2002 [19]. The estimated cube strength ($f_{est.cube}$) was calculated by applying the following equation (1) in BS 6089:1981 [20]:

$$f_{est.cube} = \frac{D}{1.5 + 1/\lambda} \cdot f_{core}, \tag{1}$$

where

- D is 2.5 for cores drilled horizontally (perpendicular to cast direction), or D is 2.3 for cores drilled vertically (parallel to cast direction),
- λ is the height/diameter ratio, and
- f_{core} is found by dividing the maximum load sustained by the core to its average cross-sectional area.

Cores with height/diameter ratio 1 were tested, because cylinders with this ratio have very nearly the same strength as standard cubes [18]. The estimated cube strength was compared to concrete mark from design drawings to verify if the columns and beams were built in accordance with the drawings. Concrete mark (operative until 1984) was calculated as a mean compressive strength of standard cube specimens of side 150 mm in kg/cm^2.

Carbonation depth and cover of concrete cores

Carbonation depth test by the phenolphthalein method was based on EVS-EN 14630:2006 [21]. Phenolphthalein, when applied to the freshly opened surface of concrete turns non-carbonated concrete red, and remains colourless in carbonated concrete. Testing was undertaken by applying a phenolphthalein solution to a freshly drilled surface of concrete cores *in situ*. Carbonation depth was measured by means of a ruler on 10 locations of the core. Carbonation depth measurements should be taken only on the hardened cement paste (not on a place of large piece of aggregate) of the core.

Concrete cover was also measured by means of a ruler on 10 locations in core hole. In most cases the core hole had to be widened to find the nearest reinforcement. Carbonation depth and concrete cover were measured on 19 randomly chosen cores and core holes, respectively.

Chemical analysis of concrete cores

In order to have an overview of deleterious salts in concrete an chemical analysis was performed. After compression test three cores extracted from the beams carrying generator (on the 2nd floor) were sent to the Remmers chemical laboratory by the company REV Special OÜ. From these cores samples were obtained for quantitative analysis of water soluble salts. With respect to durability sulphate, chloride and nitrate content were determined as a percentage of mass of concrete samples. The analysis was performed following German standard DIN 51100 [22].

Water absorption of concrete

In order to determine the water absorption of concrete ten samples were extracted from the columns on the 1st floor. Soviet standard water absorption measuring method [23] was applied since the building was constructed in 1951. Samples were immersed in 20 °C water and weighed every 24 hours until a constant value was reached. After that, samples were oven-dried until reaching a constant dry mass. Water absorption was found with the formula (2):

$$W = \frac{m_H - m_0}{m_0}, \tag{2}$$

where W is the water absorption (%),
m_H is the mass of a water-saturated sample (g), and
m_0 is the mass of a dried sample (g).

Results and discussion

Visual inspection

The condition of each column and beam in the plant was carefully assessed visually. The summary of visual assessment of beams and columns on different floors is presented Table 2.

Table 2 shows that the condition of beams is somewhat worse than that of the columns – the beams on floors 2–4 operate with spalled concrete cover. However, columns with spalled concrete cover are located on floors 1–3. Probably, the condition of beams is worse because they are more exposed to aggressive gases and nearer to the generator. On the upper floors of the generator building the concentration of gases is less intense and the temperature is lower. On the basis of visual assessment concrete structures with most severe deterioration were located for subsequent investigation.

Table 2. **Visual assessment grades of reinforced concrete members**

Floor no.	Mean grade of beams	Mean grade of columns
1	–*	0
2	0	0
3	0	0
4	0	3
5	3	3
6	4	4
7	5	5

* There are no beams on the 1st floor

From the structures to which grade 0 was assigned the beams carrying generator on the 2nd floor were in the worst condition. Numerous visually discernible structural deteriorations occurred on those girders and joists. For example, the concrete cover of tensile (but sometimes also neutral or compressive) reinforcing bars has spalled (Fig. 2–3), many stirrups are loose or broken (Fig. 2–3), concrete is delaminated (Fig. 3) or containing incompatible aggregates such as brick pieces etc. (Fig. 2). Due to uniform corrosion the cross-section of longitudinal rebars was not reduced considerably.

Fig. 2. Bottom view of a girder fragment carrying generator on the 2nd floor.

Fig. 3. Bottom view of a joist fragment carrying generator on the 2nd floor.

The columns on floors 1–3 and beams on floors 3–4 received also grade 0 i.e. operate with spalled concrete cover. Their condition was slightly better in comparison with beams carrying generator on the 2nd floor.

In general, concrete structures with cracked or spalled cover (grade 1 and 0) need extensive repairs from the owner of the building. Cracked or spalled concrete cover does not serve its function of providing fire and corrosion protection as well as bond to the reinforcement. Loose stirrups have to be reattached or replaced during repairs to restore the initial shear capacity of beams on the 2nd floor.

Compressive strength of concrete

The results of compressive strength of 55 cores acquired from columns and beams are presented in Fig. 4 and Fig. 5, respectively.

The mean core strength (derived to the mean estimated cube strength) drilled from columns (in Fig. 4) was 15.8 N/mm^2 with the standard deviation of 6.9 N/mm^2.The mean core strength drilled from beams (in Fig. 5) was

18.3 N/mm^2 with the standard deviation of 4.4 N/mm^2. The mean core strength of cores drilled both from columns and beams exceed the Soviet compressive strength marks M110 (10.8 N/mm^2) and M140 (13.7 N/mm^2), respectively. It should be mentioned that Fig. 4 and Fig. 5 present the compressive strength of unbroken cores only. Eight cores broke during drilling as a result of cracks or large voids (e.g. core 13* in Fig. 6) and could not be repaired for compressive test.

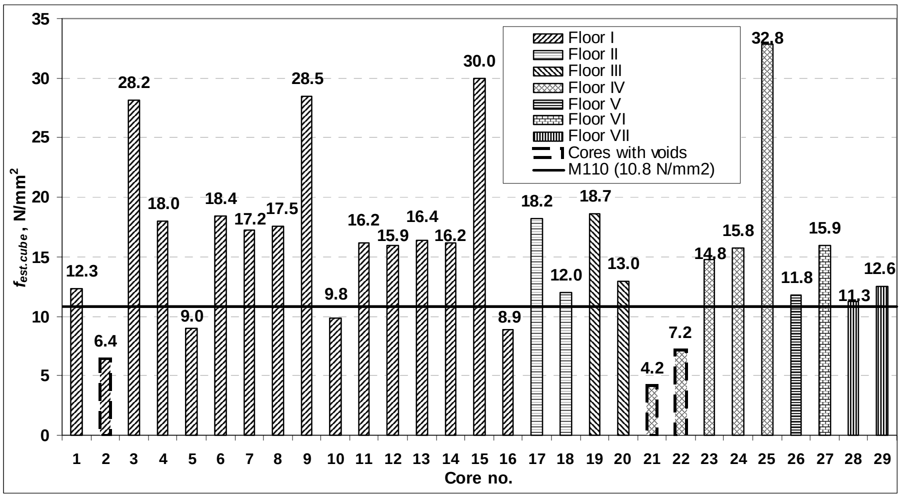

Fig. 4. Compressive strength of 29 cores drilled from columns.

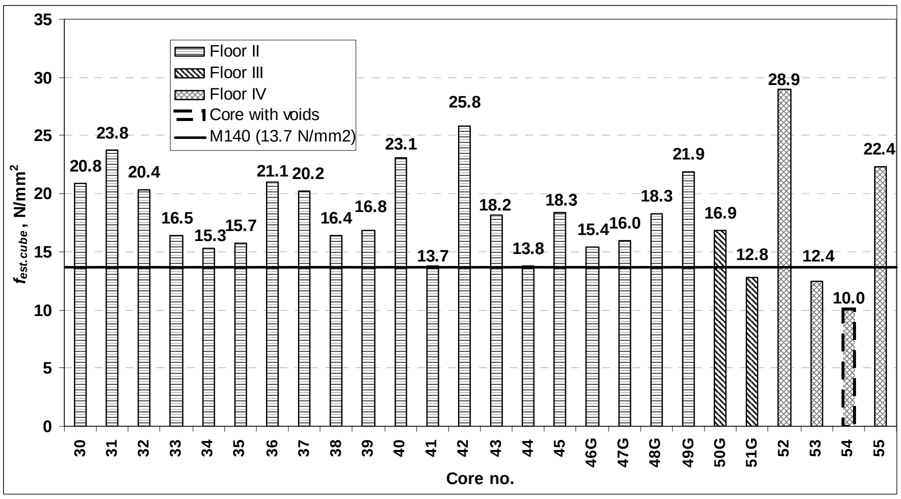

Fig. 5. Compressive strength of 26 cores drilled from beams. *Note: G behind core number denotes cores drilled from beam carrying generator.*

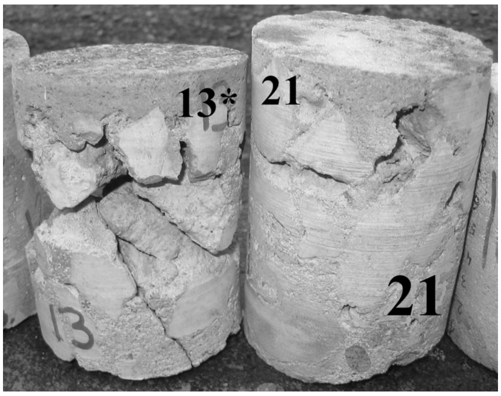

Fig. 6. Concrete cores 13* and 21 after drilling from columns.

Assessment of *in-situ* compressive strength directly from core tests is based on the reference method described in EVS-EN 13791:2007 [24], which enables to compare the results to concrete strength classes applied today [25]. The lower value of the estimated *in-situ* characteristic strength (according to Approach A in [24]) of cores drilled from columns and beams was 5.6 N/mm^2 and 11.8 N/mm^2, respectively. Therefore, the strength of cores drilled from beams corresponds to the lowest strength class - C8/10. The strength of cores drilled from columns was lower than any strength class applied today.

The strength of concrete should be taken into account when considering the bond between repair mixture (or concrete) and original concrete in the repairs of concrete structures of the plant.

Compressive test revealed some cores with questionably low compressive strength (eg. cores 2, 21, 22 in Fig. 4 and core 54 in Fig. 5). In engineering practice, the strength of concrete at a given age and cured in same conditions at a prescribed temperature is assumed to depend primarily on two factors only: the water/cement ratio and the degree of compaction [26]. According to construction drawings of the building the water/cement ratio of concrete is the same on different floors of the given structure (beam or column). However, the actual water/cement ratio may vary a little in monolithic concrete structures as a result of segregation and bleeding. Segregation involves larger aggregate particles falling towards the lower parts of the freshly placed concrete. Bleeding is the process of the upward migration or upward displacement of water. They often occur simultaneously. The other factor affecting the strength of concrete is the degree of compaction. As a result of insufficient compaction air voids may occur in concrete. The presence of voids in concrete greatly reduces its strength: Approximately 1% voidage decreases the strength by 5-8% [26]. Also in this study, voids of cores 2, 21 (in Fig. 6), 22 and 54 reduced significantly their strength in comparison with others. Voids in columns and beams were a result of insufficient compaction of concrete during the construction of the plant in 1951.

The mean estimated cube strength ($\overline{f}_{est.cube}$) as well as standard deviation of cores drilled from columns and beams on different floors are presented in Table 3.

Table 3 shows no trend in core strengths drilled from columns or beams on different floors. These results contrast with the results of visual inspection, where clear trend of grades on different floors was observable. On the basis of Table 2 and Table 3, the visual condition and the strength of the material (concrete) of the structure are not related. However, since the majority of cores were drilled from either the 1st floor (columns) or the 2nd floor (beams) no detailed comparison of strength can be performed on cores drilled from different floors in Table 3.

In this study also non-destructive rebound hammer test was conducted. However, rebound hammer test reflects only the surface of concrete. The measured rebound number is an indication of about the first 30 mm depth of concrete. Changes affecting only the surface of the concrete such as degree of saturation, carbonation, temperature, surface preparation, etc., have little influence on the properties of concrete at depth of a structure. That also explains why statistically insignificant (p-value = 0.11) and very weak (R^2 = 0.089) relationship was found between core strength and rebound number. More detailed information about the results of rebound hammer test can be found in a separate paper of the authors [27].

The rebound hammer test is largely comparative in nature. It was found on each floor that rebound hammer values *i.e*. surface strength for the beams carrying generator, thus, close to generator (*ca*. 0.4 m) were lower than for those located at some distance (*ca*. 1.5 m) [13].

The mean strength of six cores drilled from beams carrying generator (*i.e*. cores 46G-51G in Fig. 5) was 16.9 N/mm^2 with the standard deviation of 3.0 N/mm^2. The mean strength of 20 cores drilled from other beams was 18.7 N/mm^2 with the standard deviation of 4.7 N/mm^2. The mean strength of cores drilled from beams carrying generator was slightly lower (by 1.8 N/mm^2) that of the cores drilled from other beams. However, due to small difference in mean strength and high standard deviation no clear trend can be found in the results.

Table 3. **Concrete strength of columns and beams on different floors**

Floor no.	$\overline{f}_{est.cube} \pm s$ from columns, N/mm^2 (no. of cores)	$\overline{f}_{est.cube} \pm s$ from beams, N/mm^2 (no. of cores)
1	16.8 ± 7.1 (16)	–[a]
2	15.1 ± 4.4 (2)	18.6 ± 3.4 (20)
3	15.8 ± 4.0 (2)	14.9 ± 2.9 (2)
4	14.9 ± 11.1 (5)	18.4 ± 8.8 (4)
5	11.8 (1)	–[b]
6	15.9 (1)	–[b]
7	11.9 ± 0.9 (7)	-[b]

[a] – There are no beams on the 1st floor
[b] – No cores were drilled from beams on floors 5-7

Estonian Standard EVS-EN 12504-1:2003 [18] specifies that concrete strength is influenced when the core diameter is less than three times the maximum size of aggregate. In such cases the drilling operation can affect the bond between the aggregate and the surrounding hardened cement paste. As the maximum size of aggregate increases, the strength of the core decreases. The effect is more pronounced for small diameter cores [28].

In this study cores with diameter 75 mm were drilled. The maximum size of aggregate was not known. It is possible that 75 mm diameter cores violated the requirement of a minimum ratio of core diameter to aggregate size. However, cores of larger diameters were not drilled because of the risk of structural damage and congestion of the reinforcement. Overall, in view of the numerous factors influencing the strength of cores, the effect of core size can be considered to be unimportant. However, small cores have a higher variability than standard-size cores [24, 26]. Thus, an increased number of cores has to be tested. Therefore, in this study additional 35 cores were drilled besides the initial 20 cores.

Carbonation depth and cover of concrete

The results of carbonation depth as well as concrete cover measurements are presented in Fig. 7.

Figure 7. shows that mean carbonation depth of cores is considerably lower than the corresponding concrete cover. Thus, in general carbonation front has not reached the vicinity of the surface of the rebar. Only one core exists in Fig. 7 where mean carbonation depth (core 19) was nearly the same

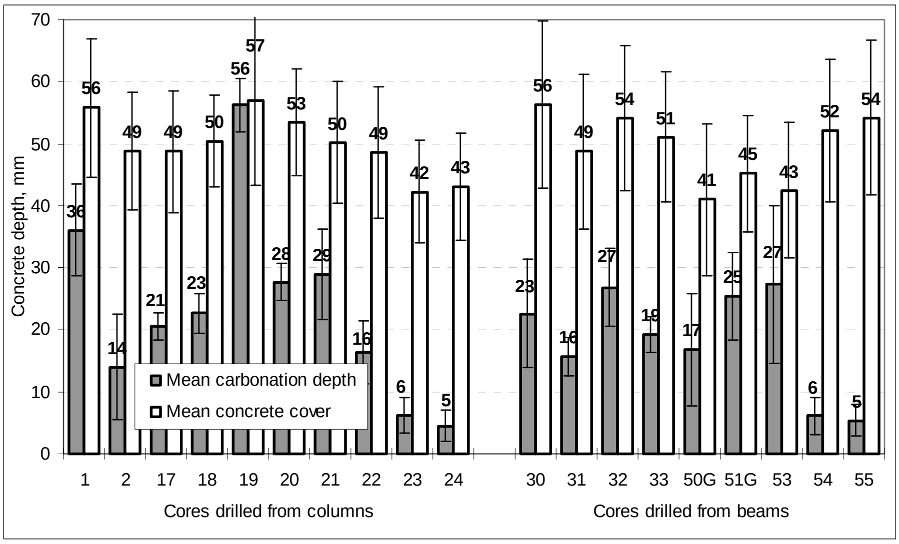

Fig. 7. Carbonation depth and concrete cover on cores drilled from columns and beams. “Whiskers” on bars denote standard deviation of the measurements. *Note: G behind core number denotes cores drilled from beam near generator.*

as concrete cover. Also, a single carbonation depth measurement on core 53 could overreach the cover (shown as an overlap of standard deviations of cover and carbonation depth on core 53). Still, according to the data sample presented in Fig. 7, the corrosion of steel reinforcement in the studied generator building was not carbonation-induced.

A photograph of the most representative cores is presented in Fig. 8. Because of the presence of coarse aggregate, carbonation depth may vary considerably on the same core e.g. core 50G in Fig. 8.

It might also be noted that, if cracks are present, CO_2 can ingress through them so that carbonation "front" advances locally from the penetrated cracks. In many cases, corrosion can take even when the full carbonation front is still a few millimetres away from the surface of the steel if partial carbonation has taken place [29].

Phenolphthalein test is easy to perform and is rapid but it should be remembered that the pink colour indicates the presence of $Ca(OH)_2$ but not necessarily a total absence of carbonation. Indeed, the phenolphthalein test gives a measure of the pH but does not distinguish between a low pH caused by carbonation and by other acidic gases.

The authors found statistically insignificant (p-value = 0.48) relationship between carbonation depth and the strength of cores. Figure 7 shows that carbonation depth and concrete cover do not differ substantially between the cores drilled from columns or beams. Also, carbonation depth and concrete cover were not differing on different floors.

According to the design drawings the cover to longitudinal reinforcement of both columns and beams was 50 mm. The actual concrete cover depends on the quality of casting of monolithic concrete. The mean cover on different columns and beams in Fig. 7 varied from 41 to 57 mm. Also, relatively high standard deviation of concrete cover was measured on the same column or beam. As an extreme example a cover from 38 to 65 mm was measured on the same column on the 3rd floor. The variable results of cover measurements in this study characterize the quality of concrete placing in the 1950ies.

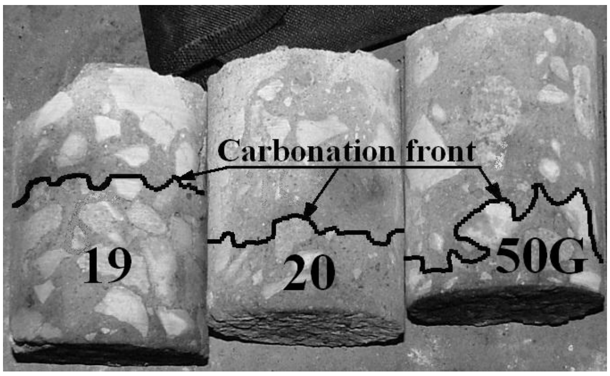

Fig. 8. Carbonation front estimated by phenolphthalein method on cores 19, 20 and 50G.

Chemical analysis of concrete

The results of chemical analysis of water-soluble deleterious salts in concrete cores are presented in Table 4. The content of water soluble salts in concrete was interpreted following WTA guidelines 4-5-99 [30] in Table 5.

Table 4 and Table 5 show that the sulphate content near the surface of concrete beams carrying generator on the 2nd floor was from middle (in samples 46G and 48G) to very high (in sample 46G). In depth of a concrete beam carrying generator on the 2nd floor the sulphate content was low.

Generally, the chloride content in surface as well as in depth of concrete was medium. Nitrate content was found to be low in all samples presented in Table 4.

Only four samples are not enough for thorough conclusions. However, it is evident that the content of deleterious salts in concrete beams carrying generators is too high. Solid salts do not attack concrete but, when present in solution, they can react with hydrated cement paste. The effect of sulphate and chloride ions to concrete is presented as follows.

Sulphate ions can penetrate the concrete and react with components of the cement matrix to cause expansive chemical reactions. Swelling may occur that, starting from the corners of a concrete structure gives rise to cracking and disintegration. Sulphate attack can also manifest itself as a progressive loss of strength of the cement paste due to loss of cohesion between the hydration products. In this study the strength of concrete cores 46G-48G (in Fig 5.) was not substantially lower than that of cores drilled from other beams. Therefore, a sulphate content ranging from middle to very high had not reduced the strength of cores.

Chloride contamination of concrete is a frequent cause of corrosion of reinforcing steel. Chloride-induced corrosion can only take place once the

Table 4. **Results of chemical analysis of deleterious salts in concrete**
Note: G behind core number denotes cores drilled from beam near generator

Core no.	Sample location in core	Content of water soluble salts in concrete, percentage of mass			
		Sulphate $(SO_4)^{2-}$	Chloride Cl^-	Nitrate NO_3^-	Total
46G	Surface	3.817	0.214	0.045	4.075
46G	Middle	0.173	0.654	0.012	0.839
47G	Surface	0.738	0.421	0.044	1.202
48G	Surface	0.751	0.236	0.021	1.009

Table 5. **Classification of the content of water soluble salts in concrete [30]**

	Content of water soluble salts in concrete, percentage of mass		
	Low	Middle	High
Sulphate	< 0.5	0.5–1.5	> 1.5
Chloride	< 0.2	0.2–0.5	> 0.5
Nitrate	< 0.1	0.1–0.3	> 0.3

chloride content of concrete in contact with the steel surface has reached a threshold value. Chlorides lead to a local breakdown of the protective oxide film on the reinforcement in alkaline concrete, so that a subsequent localized corrosion takes place. The morphology of the attack is that typical of pitting corrosion. However, no pitting was found on the exposed reinforcement of columns and beams (Fig. 2 and 3) as a result of visual inspection.

At least the beams carrying generator on the 2nd floor should be treated with steam to reduce the concentration of sulphates and chlorides in concrete. The mortar applied during repairs has to protect the steel by both physical means (i.e. preventing the ingress of deleterious substances) and by chemical means (providing repassivation).

Water absorption of concrete

Water absorption was determined on ten concrete samples, which were extracted from the columns on the 1st floor. The mean water absorption was 5.2% with the standard deviation 1.1%. It should be noted that two pitchy concrete samples had also the lowest water absorption values. Therefore, the mean water absorption could have been higher if clean concrete samples would have acquired.

The Soviet Building Code [31] distinguishes between three different concrete types: normal permeability (N), lowered permeability (P) and particularly low permeability (O) with water absorption values of 4.8-5.7%, 4.3-4.7%, 4.2% and under, respectively.

According to the mean water absorption value concrete of normal permeability (N) was applied on the columns on the 1st floor.

As mentioned before the columns on the 1st floor should be treated with steam to reduce the concentration of pitch from concrete during repairs. Considering the aggressive indoor environment in generator building the repair mixture has to be of low permeability.

Aggressive indoor environment in the plant in the 1950-ies

As mentioned in introduction, the studied oil plant was launched in 1951. At that time besides oil production these generators supplied the nearby gas plant with heating gas. The gas plant was supplying former Leningrad (St. Petersburg) with domestic gas through more than 200 km long gas pipeline (under a banner “Gas for Leningrad”).

The middle part of the 125-tonne generator designed by Lengiprogaz was more constricted when compared to preceding generators. This caused an unequal temperature distribution in those generators. As a result uncomposed oil-shale pieces fell together with semicoke into the gasification chamber at the lower part of generators. Oil shale pieces ignited with air contact and raised the temperature in the gasification chamber whereby oil-shale ash melted to become slag. Slag had to be removed (by raking) from the generator since it constrained the outlet of semicoke. Slag was raked through all four hatches of the generator manually. Hot slag was broken more efficiently by spraying water into the generator. Therefore, ash dust and harmful gases in large concentration left through hatches of the generator during slag removal. Air addition into the generator was suspended during raking. Slag raking was performed in all generators more than once a day by a schedule. Slag was removed more rarely after reconstruction of generators from the end of the 1950-ies to the beginning of the 1960-ies. Later the amount of air added to the generator was reduced. From the start of the 1980-ies the gasification process of generators was discarded and hatches were opened only for repairs [32].

Employees of the plant had noticed exposed reinforcement *i.e.* spalled concrete cover of load-bearing concrete structures near gas generators already at the beginning of the 1960-ies. Therefore, the deterioration of load-bearing concrete structures in the studied gas-generator building originates probably from the 1950-ies.

Conclusions and suggestions

A case study for investigating the condition of concrete structures and properties of concrete of an existing oil-shale chemical plant is presented. The studied seven-storied plant, located in North-Eastern Estonia, was constructed in 1951.

It is found that the deteriorations of load-bearing concrete structures in the studied oil plant building may originate already from the 1950-ies. At that time, due to a different technology, slag was removed from all generators each day. The columns and beams were exposed to a large concentration of aggressive gases exiting through the hatches of the generator during slag removal. These gases were the most probable cause of deterioration of concrete structures.

The following steps of repair based on the results of the current study are suggested:

- Removal of cracked and delaminated concrete to expose the surface of the damaged steel.
- Steaming of structures to reduce the concentration of chlorides, sulphates as well as pitch from concrete.
- Treatment of the steel rebars to remove rusting layers. Application of protective coating to the steel. Reattachment or replacement of loose stirrups. Placement of additional steel bars if necessary.
- Application of bond coat on substrate concrete to provide bond with repair mortar.
- Application of cement-based and low-permeability repair mortar to replace the damaged concrete that was removed. The repair mixture has to protect the steel by both physical means (*i.e.* preventing the ingress of deleterious substances) and by chemical means (providing repassivation of steel).

Deteriorated concrete structures on the 1st and 2nd floor of the studied generator building were repaired quite similarly to the above steps by the company REV Special OÜ. The authors recommend repairing also deteriorated concrete structures on the floors 3–7.

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