Nuclear Engineering and Design 416 (2024) 112763

Contents lists available at ScienceDirect

Nuclear Engineering and Design

journal homepage: www.elsevier.com/locate/nucengdes

FE modeling of the reconstructed Bituminated radioactive waste storage of Ignalina NPP

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A B S T R A C T

Decommissioning of nuclear facilities is a complex of various activities strongly related to safety. In 2004 and 2005, both reactors of Ignalina NPP were closed. One of the tasks that must be solved is what to do with the radioactive waste storage facility (Ignalina AE Building 158). Possible solutions: – complete destruction of the radioactive waste storage with the need to empty the canyons and store the waste in a special place or, as an alternative, turning the storage into a repository. Converting storage to repository also has many different options. The installation of protective barriers of different thicknesses or even the construction of a sarcophagus is possible. The cheapest way to form a protective barrier on an existing building structure. The problem is the relatively high weight of the barrier structure, which must be supported by the existing building structure.

The article presents an analysis of the durability of the structures of the radioactive waste storage, assessing the influence of the self-weight of the building's reinforced concrete structural elements and the mass of the bituminous radioactive waste stored in the canyons, as well as the effect of additional loads acting on the floor and roof of the first floor during the planned conversion of the storage facility into the repository. Numerical simulation was performed using ANSYS computer code and the results were compared with the relevant standards and norms in order to assess the building's ability to undergo such transformations.

Spatial volumetric geometric and calculation models of the construction structures of the 158 building were created. These models have been modified after considering future changes during the transformation of the building into a repository. Perform calculations of the state of stresses and strains of the landfill under the load of the self-weight of the structural elements and the hydrostatic pressure of the bituminous waste. On the basis of the results of the calculations, conclusions were formulated about the level of load on the construction structures of the building and the margin under the current operating loads.

The developed numerical models can be easily extended to evaluate the effects of new building elements and additional loads on structures (specifically walls and ceilings) after the transformation of a waste storage into a repository.

The article presents not only the idea of reorganizing the waste storage structure to repository but also the methodology of modelling such a structure by assessing the age of the building's soil and additional loads. This methodology can be useful and applied to the closure of other nuclear power plants when it is necessary to turn the radioactive waste storage into a repository.

1. Introduction

Decommissioning of nuclear power plant activities brings new engineering and safety challenges and requires detailed planning to keep facility under same safety level as during operation time. One of the tasks in this procedure radioactive waste storage decommissioning.

With the changing safety requirements for radioactive waste management, storage and disposal, waste acceptance criteria, as well as unforeseen problems during long-term waste storage, it is necessary to upgrade existing storage facilities or convert already closed storage facilities into repositories. There are quite a great number of radioactive waste storage facilities and repositories in the world that are being upgraded in one or another way. The IAEA document (International Atomic Energy Agency, 2005) provides a detailed analysis of the remedial action process, possible remedial actions and technologies for surface repositories. International practice in upgrading existing storage facilities or repositories in many parts of the world is presented. The condition of some storage facilities and repositories has been reviewed in more recent publications (Csullog, 2005), (Bergström et al., 2011), (International Atomic Energy Agency, 2001). Typically, the modernisation of storage or waste disposal facilities involves the specification of new WACs and containers, as well as the construction of additional engineering barriers, the installation of waterproofing barriers, the improvement of coating systems, and better control of leaching and surface water runoff.

A performed analysis of surface repositories or low-level radioactive waste disposal facilities has shown that changes in safety requirements for radioactive waste management, storage and disposal, the formation of new waste facilities, expansion of existing storage capacities, and unforeseen problems during long-term storage make it necessary to

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https://doi.org/10.1016/j.nucengdes.2023.112763
Received 27 September 2023; Received in revised form 9 November 2023; Accepted 10 November 2023

upgrade existing repositories or convert already closed storage facilities to repositories.

However, when turning storage facilities into repositories, there is another problem, whether the structure of the existing building will withstand the additional loads from engineering barriers taking into account that the storage facility was built several decades ago. This requires an analysis of the load-bearing capacity of storage facilities turned into repositories. It is practically impossible to do this with analytical methods. Applying the finite element method is also not a simple task. The structure of the building is quite complex. Both the floor and the roof are heavily reinforced, all the reinforced concrete structures of the building have already been built a long time ago. When performing the calculations, the influence of the soil under the building must also be evaluated.

Of course, similar problems have been reviewed in recent publications. More than twenty years ago (Omraci et al., n.d) observed physical and mechanical parameters of the lateritic soil discharge and demonstrated the heterogeneity of this material using FEM analysis. A few studies have compared soil-structure modelling results obtained with different finite element codes. Xu et al. (2006) compared predictions made using SASSI (Lysmer et al., 1999) and LS-DYNA (Livermore Software Technology Corporation (LSTC) (2013)). Latter an assessment of the industry-standard soil-structure interaction analysis codes, SASSI, and LS-DYNA for linear and nonlinear SSI analyses of safety-related nuclear structures and commercial buildings was demonstrated by Bolisetti et al., (2018). Coleman et al., (2016) presented a methodology for performing nonlinear soil-structure analysis in the time domain and identified the need for mesh and time-step sensitivity study. Farahani et al., (2016) described three-dimensional finite element modelling and seismic soil-structure interaction analysis of a nuclear power plant diesel generator building that is founded on soil in degraded concrete stiffness condition. Bouhjiti et al., (2020) propose a global stochastic finite element method to model the effects of concrete ageing uncertainties on the serviceability and durability of large reinforced and prestressed structures with a containment role. Coronado et al., (2011) presented the results of a mesh sensitivity study for concrete panels (i.e., slabs and walls) with aspect ratios and thicknesses that fall within the practical range of configurations used in nuclear power plant construction. Shell and solid elements available in the ANSYS (ANSYS Inc., 1999) finite element program were used for this purpose.

Soil-structure interaction analysis using the direct method can also be performed using other commercial finite-element codes such as ABAQUS (Abaqus manual, 2014), or the open source codes, OpenSees (Mazzoni et al., 2009) and MOOSE (Gaston et al., 2009).

In this work ANSYS Workbench (ANSYS Workbench, 2000) software was used to evaluate the structures of the radioactive waste Storage, assessing the influence of the self-weight of the building's reinforced concrete structural elements and the mass of the bituminous radioactive waste stored in the canyons, as well as the effect of additional loads acting on the floor and roof of the first floor during the planned conversion of the storage facility into the repository.

2. Structure of building 158 of Ignalina NPP before the transformation it into a repository

The bituminised radioactive waste is stored by the SE Ignalina Nuclear Power Plant in building 158, located in the north-western part of the Power Plant area about 200 m to the west from Unit 1 (Fig. 1). The storage facility (Building 158) is connected to building 150 (building for processing, bituminisation, and cementing of LRW) by pedestrian and technology galleries from the east side and to building 158/2 (interim storage facility for LRW) from the west side. The minimum distance between the storage facility and the aforementioned buildings is about 10 m.

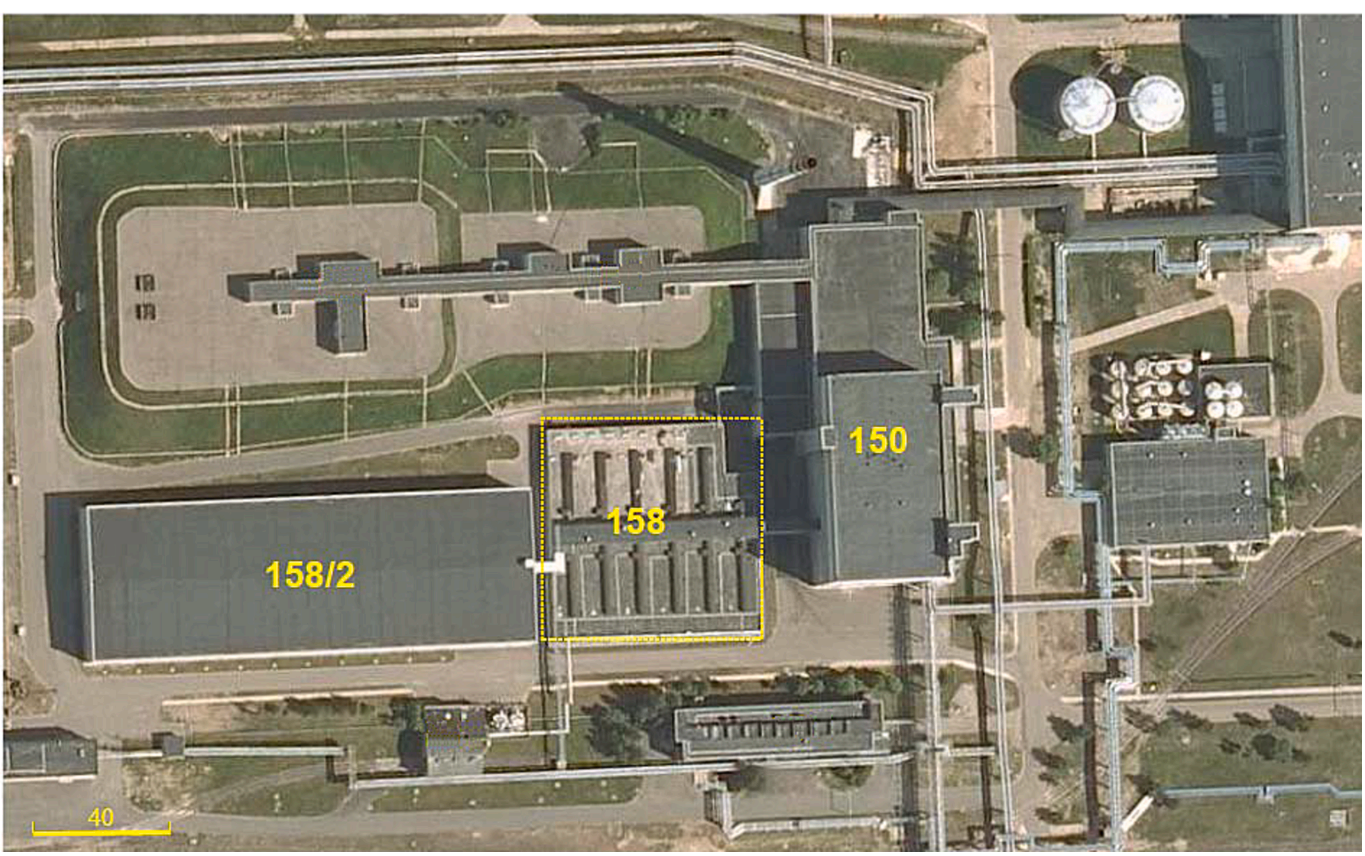


Fig. 1. Building 158 in the Ignalina NPP territory.

Analysis of the structure of Ignalina's NPP building 158 and its geometrical and computational models are based on the design and as-built drawings provided by the employer and other related documents (annexes to the Technical Specifications, Safety Analysis Report, Reports on previously performed investigations, Feasibility study for the conversion of the storage facility into the repository, various legal acts, photos of the inside and outside parts of the building, etc.).

Storage facility (building 158) is a over ground two-story reinforced concrete structure of rectangular form (75.3×74.1 m) (construction of which was commenced in 1981, operation commenced in 1987). The first floor of the building includes 12 premises for bituminised radioactive waste (canyons), the second floor contains industrial premises, communication corridors and channels, control and measurement rooms (Fig. 2).

Storage facility for bituminised radioactive waste (building 158) is installed on the engineering base (bed) of 1.6 m thickness formed using no-fine concrete of M50 grade instead of surface soil of poor properties (Fig. 2).

This layer is covered with a levelling concrete later of M100 grade of 100 mm thickness on which four monolithic reinforced concrete foundation slabs are installed: three rectangular slabs of ПМ-1 type and one L-shaped slab of ПМ-2 type (a gap between the adjacent slabs is 50 mm). Foundation slabs are made of concrete of M200 grade, thickness of slabs is 400 mm, only under external walls and internal partitions thickness is increased up to 600 mm. Slabs are reinforced with carbon steel rods of 25/12 mm diameter in the upper layer and of 16/12 mm diameter (along/across canyons, respectively) rods in the bottom layer (maintaining a 200/300 mm step), both reinforcement layers are installed ~35 mm from the upper and the bottom surfaces of the slabs. Thickness of foundation slabs under the exterior walls and internal partitions is increased due to special channels of 300 mm depth and 400 mm width formed on the upper surface which are used for embedment of reinforced concrete plates of the exterior walls and internal partitions of the first floor. The inside part of foundation slabs in the canyons is covered with waterproofing and levelling concrete layers of M200 grade (both of 50 mm thickness) with waterproofing ruberoid interlayer between them.

The first floor of the storage facility (from –0.25 to + 6.25 altitudes) is divided into twelve compartments (canyons) of 6 m height and 12x36 m area using internal partitions; canyons are grouped into four blocks of three canyons each. Volume (structural) of 11 canyons is 2500 m^3 (operational volume is 2000 m^3), of one canyon is halved, 1000 m^3 (operational volume is.

800 m^3). Total operational volume of all 12 canyons is 22800 m^3. The layout of canyons is shown in Fig. 3.

Bearing reinforced concrete walls of the first floor of the storage facility are made of prefabricated reinforced concrete plates of П-1 grade (concrete of M200 grade) of 300 mm thickness, 6.3 mm length, and 1.15 m width embedded into the grooves of the foundation slabs ПМ. The same length and thickness but slightly narrower reinforced concrete plates of П-1–1, П-1–2 and П-1–3 types with small differences in reinforcement were also used).

To reduce radiation emission to the environment all exterior walls of the building are covered from the outside with concrete (M200 grade) blocks СБ (6.0x1.2 m, thickness 300 mm), installed in parallel to the walls at 200 mm distance from them, a gap shall be filled in with concrete of M300 grade. This way the total thickness of the exterior walls of canyons is ~800 mm.

It is planned to construct flooring of the first floor using original reinforced concrete beams of T-shaped cross-section and 12 m length, however, due to the failure to get them, prefabricated reinforced concrete (M200 grade) plates of NP-1 type of a halved length (5980 mm length, 1490 mm width, and 220 mm thickness) were used. Since width of the canyons reaches 12 m, 7 reinforced concrete pillars of 400 x 400 mm cross-section were placed on bearing blocks to support flooring plates in the middle of the canyons (5.5 m along the canyons). Reinforced concrete collars ИБ9-3 of 300x800 mm cross-section are installed on the pillars. Canyons are covered with reinforced concrete plates of NP-1 type ensuring that a gap between floorings of groups of canyons (four of three canyons) is equal to 50 mm (the same as between foundation slabs). This concrete plate flooring is covered with four monolithic reinforced concrete (of M300 grade) plates forming the second layer of the canyon flooring. Three rectangular plates of ПМ-3 type cover canyons 1–3, 4–6 and 7–9, plate of ПМ-4 type (L-shaped) covers canyons 10–12. Thickness of the plates forming the ridge roof varies from 480 mm in the centre to 280 mm along the perimeter of the building 158, this variation ensures maintenance of a slope required for water discharge. The flooring itself is coated with a thermal insulation layer made of Polystyrene covered with bitumen, and several layers of Ruberoid. In order to ensure good waterproofing characteristics, the roof was additionally covered with membrane coating Wolfin GWSK of ~2 mm thickness in 2003 and 2005.

Internal surfaces of canyons No. 1 (UF44B01) and No. 2 (UF44B02) are coated with 10 mm thickness cement-silicate ('concrete-glass') layer (i.e., fire-resistant material). Internal surfaces of canyons No. 3 (UF44B03), No. 4 (UF44B04), No. 5 (UF45B01), No. 6 (UF45B02), No. 12 (UF59B01), No. 11 (UF59B02) and No. 10 (UF59B03) are seamlessly covered with carbon steel sheets up to 4.5 m height (thickness of a steel sheet is 4 mm) and with stainless steel sheets from the altitude 4.5 m up to 5.5 m (thickness of a steel sheet is 3 mm). Bottom of each canyon is covered with cement mortar of B25 grade (high strength concrete), thickness of a layer is 50 mm. Surface of cement mortar is additionally covered with rubber-bitumen mastic, varying in thickness from 5 to 50 mm. Walls and bottom of canyons No. 10 (UF59B03) and No 11

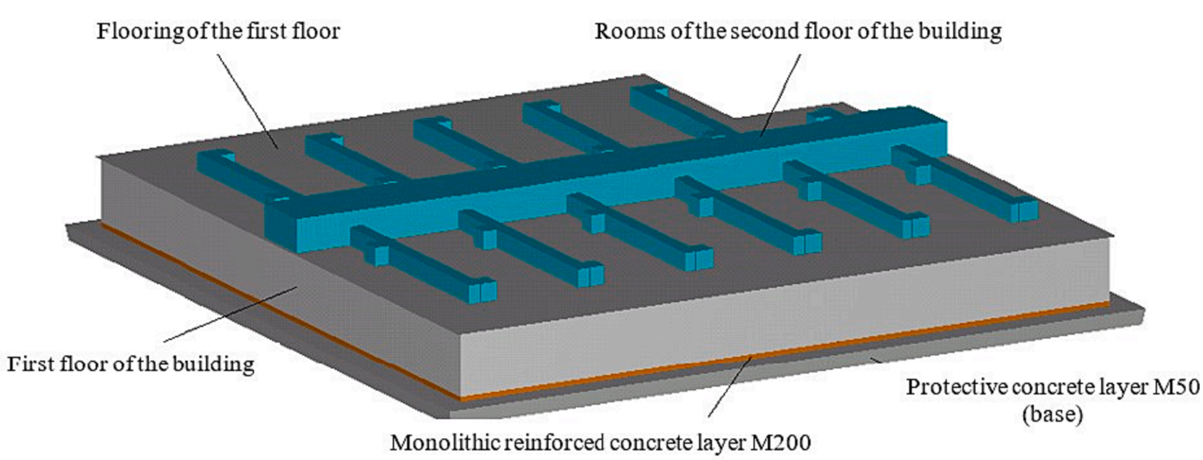


Fig. 2. General view of building 158 of Ignalina NPP (simplified diagram).

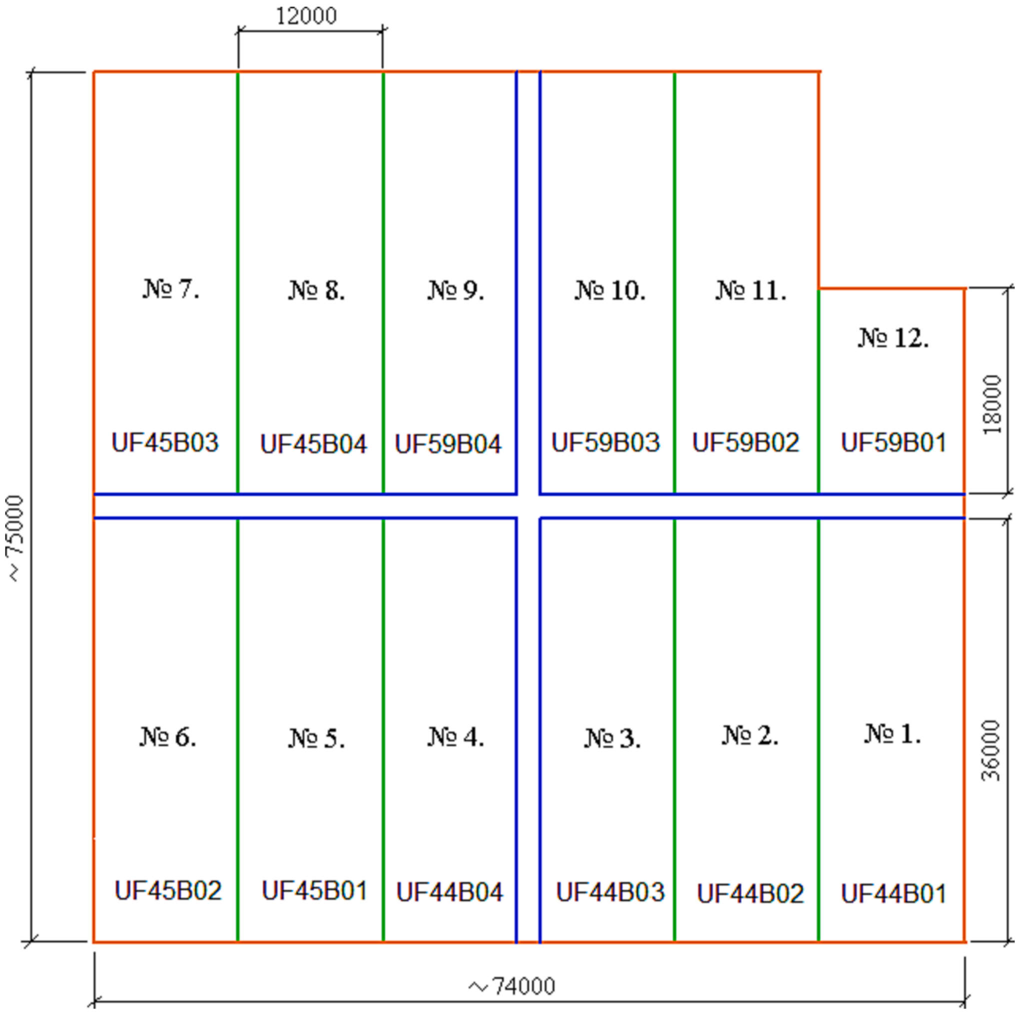


Fig. 3. Layout plan of canyons of bituminised radioactive waste storage facility.

(UF59B02) are seamlessly covered with carbon steel sheets (sheet thickness 4 mm).

Walls of premises located on the second floor (service rooms, communication corridors and ducts, control and measuring equipment rooms) of 1.8 m high, are made of concrete blocks ΦC-4 and ΦC-3, their flooring of reinforced concrete plates of 180–300 mm thickness. Walls of the auxiliary rooms are made of silicate bricks, their roof is made of reinforced concrete plates of 50–300 mm thickness.

In preparation for the conversion of the storage facility to the repository, the second-floor structural elements will be dismantled, and are therefore not analysed and evaluated by calculations in more detail in this report. In this case, another factor to consider when designing the RW repository is the weight of the bituminised radioactive waste in the canyons (foundation slabs subjected to its weight). Currently, canyons Nos. 1–6, 10, 12 are completely loaded with waste, three canyons (Nos. 7, 8 and 9) are still completely empty (their location see Fig. 3), loading of canyon No. 11 commenced in 2015, but for various reasons IAE was accepted the decision to stop filling bituminous waste into this canyon. In preparation for the conversion of the storage facility to the repository is planned all remaining canyons and empty cavities should be completely filled with inert material (e.g. sand) in order to equalize the load acting on the foundation.

3. Concept of the future repository

Conversion of Ignalina NPP bituminised radioactive waste storage facility into the repository by installing steel and reinforced concrete bearing structures for the engineering barrier of 5.8 m thickness on the flooring of reinforced concrete structure of building 158 (general view shown in Fig. 4).

Spatial volumetric geometric model of the engineering barrier above the group of three 12x36 m canyons rested on the flooring of building 158 along all walls of the building (including partitions between canyons) (based on the diagram provided in Fig. 5) is given in Fig. 6. Unlike the bearing frame in this model (Fig. 6) is made of h-beams of profile HD310x310x500 and the beams ensuring rigidity (H-beam HE1000B profiles) (Fig. 7) are not directly covered with soil forming the engineering barrier (several layers of soil) (Fig. 8), but are concreted longwise at 500×986 mm cross-section (Fig. 7). Concreting in this case both increases the bearing capacity of the beams (prevents them from tripping) and protects them from corrosion. Fig. 9.

During the reconstruction and conversion of the storage facility into the repository, the waterproofing of building 158 and its foundations will be performed using a chemical material which main action is related to the crystallisation of concrete. The method of waterproofing and the waterproofing material will be specified during the preparation of the technical design. Metal structures of the future repository shall be covered with an appropriate anti-corrosion coating, the steel elements of the engineered barrier (main supporting H-beams) shall be concreted with cold-cycle concrete (e.g., using F200 frost-resistant concrete) after coating with anti-corrosion primers, paints or special coatings.

Installation of the engineering barrier of 5.8 m thickness ensures protection of the structure from environmental (atmospheric) influences (temperature, moisture, mechanical impact, etc.). The protection

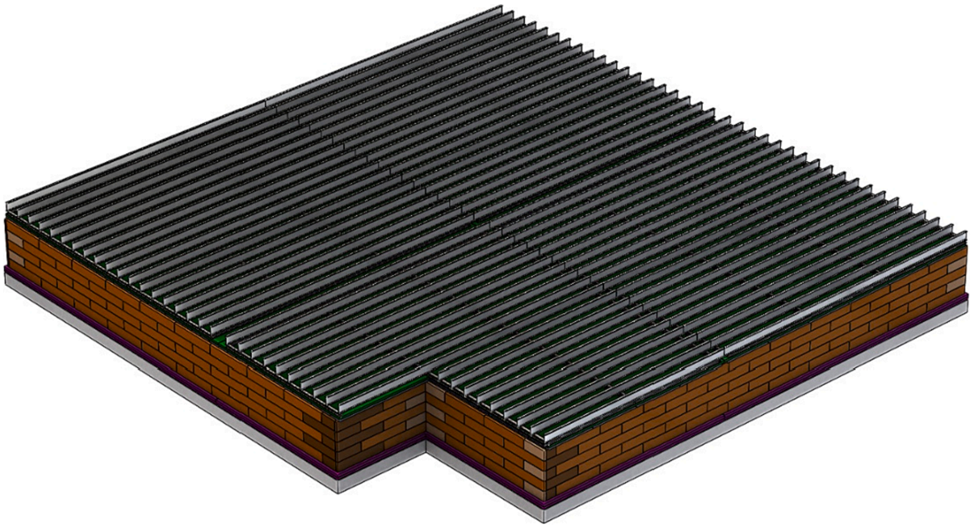


Fig. 4. Conversion of the storage facility (building 158) into the repository by installing the engineering barrier of 5.8 m thickness. The storage facility with bearing structures of the engineering barrier placed on the flooring.

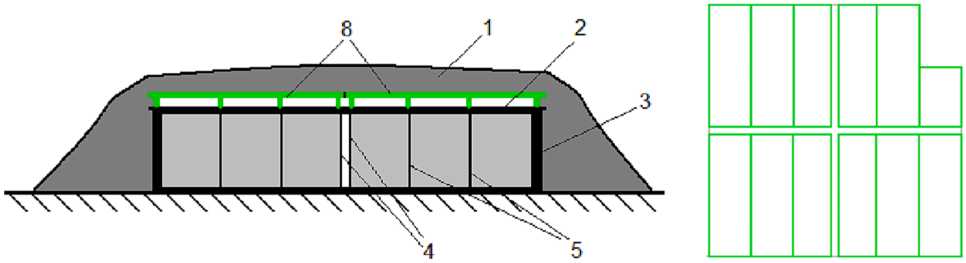


Fig. 5. Engineering barrier constructed by pouring soil layer of the barrier on the additional structure rested on the floor of the storage facility along all walls (flooring of the first floor) (including partitions between canyons).

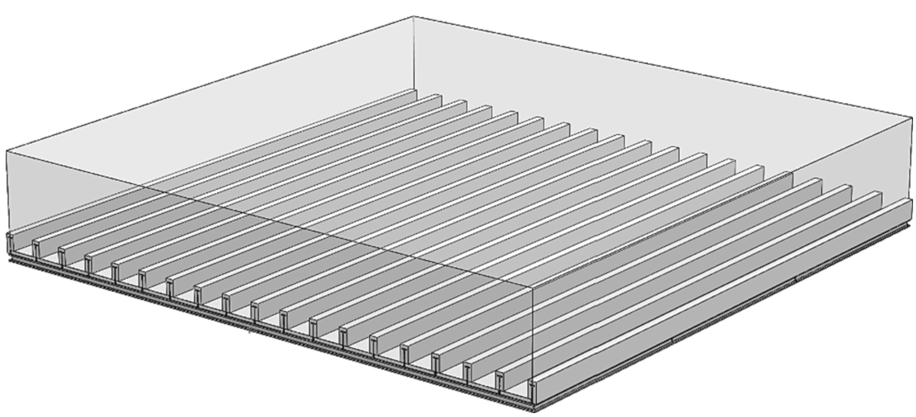


Fig. 6. General view of an engineering barrier of 5.8 m thickness (spatial volumetric geometric model).

measures listed above ensure the permanence of all features of the construction elements of building 158 and their unchangeability for at least 100 years.

Bearing structures are covered with soil layers of different purpose (thus, and of different properties) layer by layer and compacted to form the engineering barrier (composition is provided in Fig. 8), which is additionally covered with a top vegetation layer formed by afforestation of the Repository and hiding (protecting from outside influences) the embankment with region-specific vegetation (Fig. 8).

4. Analysis of building 158 transformed into a repository applying the finite elements method

Computational analysis applying finite element method is used to solve the tasks mentioned in Section 1.3. To perform it, first of all, 3D (spatial) volumetric (solid) geometric models of the facility under investigation (in this particular case – Ignalina NPP bituminised radioactive waste storage facility – building 158) have been developed based on the working drawings and other documentation provided by the Employer; the models include foundation slabs of the building, exterior and internal walls, and reinforced concrete floorings – roof plates and

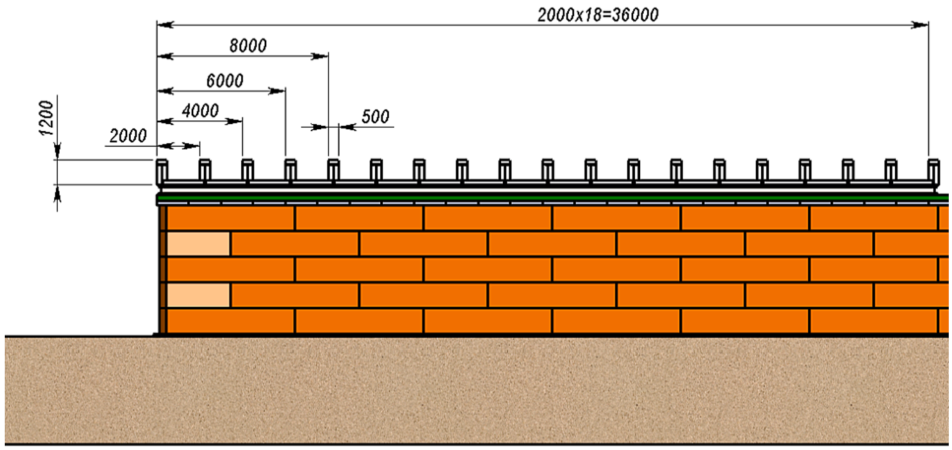


Fig. 7. Fragment of the storage facility bearing structures of the engineering barrier on the flooring.

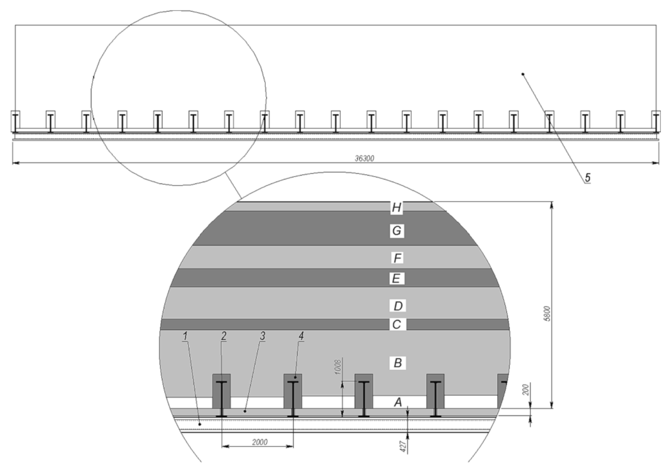


Fig. 8. Diagram of cross-section of the engineering barrier of 5.8 m: 1 – H-beam profile HD310x310x500, 2 – H-beam profile HE1000B, 3 – concrete slab of 200 mm thickness, 4 – concreted H-beam profiles HE1000B, 5 – layered engineering barrier of 5.8 m thickness (A – drainage layer of 0.2 m thickness (fine sand); B – insulating layer of 2.0 m (limnoglacial clay); C – drainage layer of 0.3 m thickness (gritty sand); D – protective layer of 0.6 m thickness (moraine clay); E – drainage layer of 0.6 m thickness (dusty sand); F – drainage layer of 0.7 m thickness (gravel); G – drainage layer of 0.8 m thickness (crushed stone); H – vegetation layer of 0.2 m thickness.

bearing collars and pillars. Geometric models of main structural elements (foundation and wall slabs and panels, pillars, collars, etc.) are created by using 3D geometric modelling software SolidWorks assembly model of the entire building 158 was also created by SolidWorks by inclusion of relevant constraints between structural elements constituting the entire building.

Geometric model is used as a basis for creation of the volumetric (spatial) computational model of the building under investigation: the corresponding boundary conditions are specified (fixtures and loads), model elements (solid and surface) are assigned with the relevant material properties, and, the stress-strain state of the model elements under the appropriate load combination is calculated by solving a static linearly elastic task. ANSYS Workbench software (ANSYS Workbench, 2020) was used to create the computational model, to preform computations and process the results of computations performed.

Major part of structural units of building 158 are reinforced concrete, i.e., reinforced blocks or plates, however, at this stage of the investigation, reinforcement was modelled only for the foundation slabs. This was done by omitting modelling the reinforcement rods (this would make the model very complex and time-consuming), and creating two equivalent homogeneous steel sheet-type layers in terms of cross-section area (based on the cross-sectional area of the thinnest reinforcement 12 mm in diameter placed in concrete every 200 mm), within 35 mm of the upper and lower surfaces of the foundation slabs.

All the components forming the model (foundation slabs, wall panels and blocks, aggregates and flooring (and panels forming them)) were considered to be rigidly bonded together, i.e., the structure of the whole building was treated as monolithic. Such simplification has been adopted taking into account the requirements of the technical documentation used in the construction process for the construction technology and materials and the nature of the load (only compressive forces prevail).

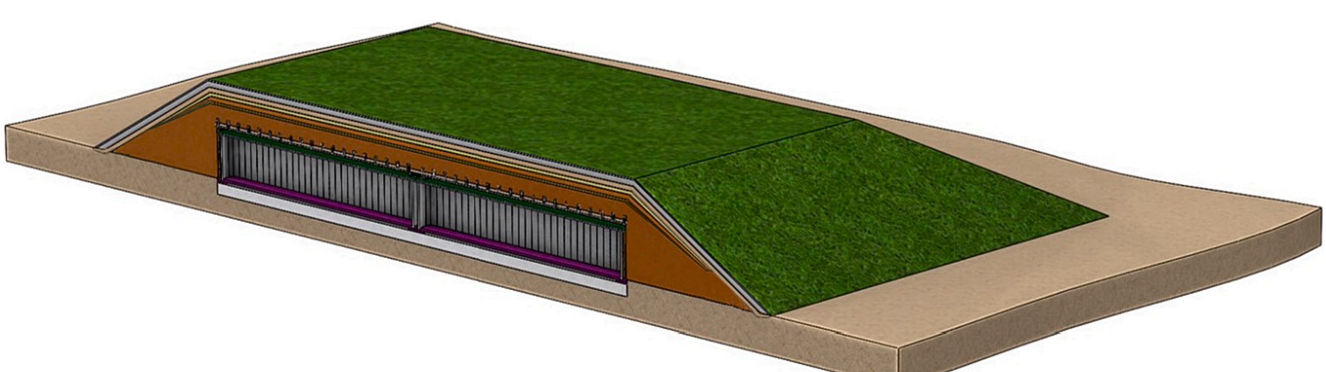


Fig. 9. The repository with engineering barrier of 5.8 m thickness. Isometric view section.

4.1. Loads

The following loads have been evaluated in calculating the stress-strain state of building 158 – bituminised radioactive waste storage facility transformed into a repository:

- Own weight of the main structural units in the finite element model shall be assessed directly by assigning bulk densities of materials (see chapter 4.2 herein) to the relevant volumes of the model (foundation slabs, wall and flooring panels);
- Conversion of the storage facility into the repository foresees complete loading of canyons (up to the ceiling level, i.e., up to the 6 m height mark) with inert material (leaving no air gap where moisture could accumulate), therefore, for the purpose of this scenario, the weight of bituminised RW shall be estimated by recalculating their quantity and weight in canyons.
- The additional metal – reinforced concrete flooring with an engineering barrier of 5.8 m thickness installed on the roof of the building (including 0.5 m thickness of the reinforced concrete flooring of Building 158) load. This barrier is formed of reinforced concrete layer of 0.2 m thickness, drainage layer of 0.2 m thickness (fine sand); insulating layer of 2.0 m thickness (limnoglacial clay); drainage layer of 0.3 m thickness (gritty sand); protective layer of 0.7 m thickness (moraine clay); drainage layer of 0.5 m thickness (dusty sand); drainage layer of 0.7 m thickness (gravel); drainage layer of 0.8 m thickness (crushed stone); vegetation layer of 0.2 m thickness (equivalent density of the barrier soil layer is – 1910 kg/m^3), its structure is analogous to the structure of engineering barrier of 2.1 m thickness.
- Wind and snow loads. The calculations also estimated the design wind and snow loads (basic reference value of wind speed $v_{ref} = 24$ m/s (according to STR 2.05.04: 2003, partial wind reliability coefficient $\gamma_Q - 1.3$), 200 kg/m^2 snow cover load (according to STR 2.05.04: 2003, snow load in the INPP zone $-$ 1.6 kN/m^2, partial reliability coefficient of snow effect is $\gamma_Q - 1.3$) Calculations were performed by simultaneously adding all loads, which makes the computational model conservative. Extreme situations (seismic load, hurricanes and hurricanes) were not taken into account in the calculations, as the building is not directly dangerous due to the absence of voids in the building and its transformation into a repository essentially equivalent to a monolithic block buried under a large layer of earth. Slope stability provided by a 3:1 slope and technological measures such as the properties of the poured soil, the slope support bar proper drainage of rain and ice water.

4.2. Materials

When compiling the calculation model of the repository, it was assumed that the construction structures of the buildings are purely concrete (concrete from M50 to M300), only the reinforcement of the foundation plates (equivalent layers at a distance of 35 mm from the lower and upper surfaces) is structural steel. The mechanical properties (isotropic, elastic) of the mentioned materials are calculated as follows:

Soil:

- Elasticity modulus $E_c = 2.5\ 10^9$ Pa.
- Density 2230 kg/m^3.

M50 concrete (GOST 5802–86) (or B3,5 in accordance with LST 1300:2000):

- Elasticity modulus $E_c = 9.5\ 10^9$ Pa.
- Density 2400 kg/m^3.
- Strength limit under compression $R_{b,ser} = 2.7$ MPa.
- Strength limit under tension $R_{bt,ser} = 0.39$ MPa.

Concrete M200 (GOST 5802–86) (or C12/15 in accordance with LST EN 206–1:2002/B15 in accordance with LST 1300:2000):

- Elasticity modulus $E_c = 23.0\ 10^9$ Pa.
- Density 2400 kg/m^3.
- Strength limit under compression $R_{b,ser} = 15.0$ MPa.
- Strength limit under tension $R_{bt,ser} = 1.15$ MPa.

Concrete M300 (strength class B25 in accordance with GOST 2663–85) (or C20/25 in accordance with LST EN 206–1:2002/B25 in accordance with LST 1300:2000):

- Elasticity modulus 30.0 10^9 Pa.
- Density 2400 kg/m^3.
- Strength limit under compression $R_{b,ser} = 18.5$ MPa.
- Strength limit under tension $R_{bt,ser} = 1.6$ MPa.

Compressive strength of the cylindrical specimen in accordance with Eurocode-2, (T. 3.1) is:

- Concrete M300 (C20/25) $f_{ck} = 20$ N/mm^2 (or MPa).
- Concrete M200 (C12/15) $f_{ck} = 12$ N/mm^2 (or MPa).

Structural steel:

- Elasticity modulus $2.1 \cdot 10^{11}$ Pa.
- Density 7850 kg/m^3.
- Poisson's constant 0.3.
- Yield limit under compression $2.5 \cdot 10^8$ Pa; yield limit under tension $2.5 \cdot 10^8$ Pa.
- Strength limit under tension $4.6 \cdot 10^8$ Pa.

Equivalent density of bituminised RW 1200 kg/m^3.

4.3. Assessment of calculated stress-strain state

Assessment of the stress and strain state in structures according to Eurocode-2 includes the analysis of stresses and strain in concrete and reinforcement. Particular case included direct assessment of the reinforcement of foundation slabs exclusively. Bearing capacity of building 158 was assessed based on the stresses and strains formed in the concrete and the reinforcement.

Fig. 10 shows diagrams for compressive deformation of concrete, describing the non-linear or two-dimensional elastic-plastic behaviour of concrete. The tensile strength of concrete is assumed to be zero. Calculations based on the finite element method enable easy presentation of the said dependencies in the form of material characteristics and execution of physically non-linear analysis, but in this case the calculation time is greatly increased. Simplified methodologies use a linear elastic model, the deformations being determined from the stress values obtained based on the diagrams referred to in Fig. 10, and the conclusions on the strength reserve are based on the size of the deformation of the concrete.

Generally, the critical parameter for the assessment based on the limit state for long-term operation is the crack opening width, calculated as described in Clause 4.4.2.4 of Eurocode-2.

Design crack opening width shall be calculated as follows:

$$w_k = \beta s_{rm} \varepsilon_{rm} \tag{1}$$

here:

w_k– crack opening width;

β– a factor which, in the range of section thickness from 300 to 800 mm, obtains values in proportion from 1.3 to 1.7 respectively.

ε_{rm}– average value of deformation determined by the empirical formula.

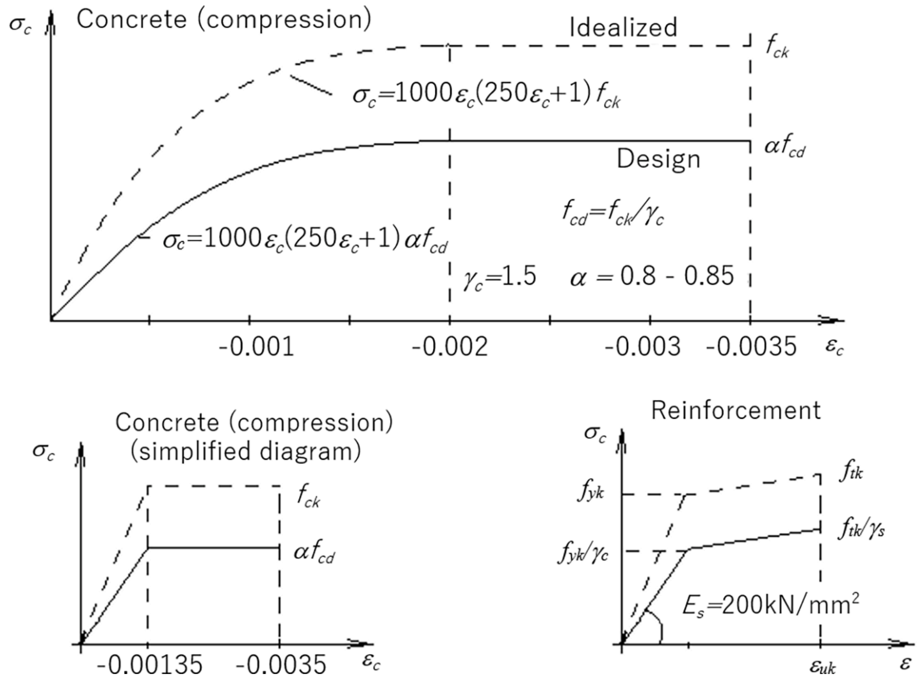


Fig. 10. Diagrams of compressed concrete (a), (a') and diagrams of reinforcement (b) in accordance with Clause 4.2 of Eurocode-2: ε_c – concrete strain; f_{ck} – strength limit of compressed concrete (characteristic value); $f_{cd} = f_{cd}/\gamma_c$ – strength limit of compressed concrete (design value); $\gamma_c = 1.5$ – strength margin factor of compressed concrete (Clause 2.3 of Eurocode-2); $\alpha = 0.8 - 0.85$ – safety margin assessing durability of operation; ε – reinforcement strain; f_{yk} – viscosity limit of reinforcement (characteristic value); f_{tk} – strength limit of the reinforcement (characteristic value); $f_{yd} = f_{kd}/\gamma_s$, $f_{td} = f_{td}/\gamma_s$ – relevant design values; $\gamma_s = 1.15$ – strength margin factor of reinforcement (Clause 2.3 of Eurocode-2); E_s – reinforcement elasticity modulus.

$$\varepsilon_{rm} = \frac{\sigma_s}{E_s}\left(1 - \beta_1\beta_2\left(\frac{\sigma_{sr}}{\sigma_s}\right)^2\right) \qquad (2)$$

σ_s– tensile stresses in the reinforcement at the base of the crack cross-section.

σ_{sr}– tensile stresses in the reinforcement at the base of the crack cross-section under the load resulting in crack opening; here accepted $\sigma_{sr} = \sigma_s$.

$\beta_1 = 1.0$ for high cross-sections.

$\beta_2 = 0.5$ under continuous loads.

$s_{rm} = 50 + 0.25 k_1 k_2 \frac{\phi}{\rho_r}$ – crack length.

ϕ– cross-section of a single reinforcement rod.

$k_1 = 0.8$ for high cross-sections.

$k_2 = 0.5$ in case of prevailing bending strains.

$\rho_r = \frac{A_s}{A_{c,eff}}$ effective reinforcement ratio.

A_s– cross-section area of a reinforcement in the zone of extension strains.

which area is $A_{c,eff}$.

As reinforcement in building structures is not assessed in the case under study, Eurocode-2 also provides other recommendations for the assessment of limit state for long-term operation:

- no plastic deformations in the reinforcement (not evaluated in the case under study).
- compression strains within the compressed concrete layers shall not exceed $0.6 f_{ck}$.

here f_{ck} – characteristic cylinder compressive strength of the concrete. When describing the material properties in Chapter 1.5, the value of this parameter is also given for M300 and M200 concrete. Eurocode-2 does not include any values of the aforementioned parameter for M100 concrete, consequently, state of the model elements made of M100 concrete has been evaluated by replacing the f_{ck} with the strength limit under compression $R_{b,ser} = 5.5$ MPa (for M200 and M300 concrete this value is slightly smaller than the f_{ck}).

Durability of the concrete is governed by the number of cold cycles. Normative documents STR 2.05.05:2005 and LST EN 206:20014 provide concrete grades according to resistance to cold cycles – from F50 to F1000.

A source (Feasibility study for conversion of the interim bituminised radioactive waste storage facility into the Repository (Long-term safety justification). Final report. 2 revision. S/14–796.6.7/PSR-FRI/R:2, LEI, 20 October 2009) indicates that there are about 15 cold cycles a year in Lithuania (Chapter 12.18 "Frost resistance"), thus, the class of concrete resistance to cold cycles can be used to calculate how many years that concrete should withstand, e.g., F300 should withstand 300/15 = 20 years. However, this (frost-resistant) concrete was not used (or no such information was found) for Building 158 – according to the relevant data, the building structures consist of M200 and M300 concrete for which the degradation properties are not standardised, as a result, neither the actual state of the concrete nor changes in its properties over time can be predicted with reasonable accuracy.

Eurocode-2 has several exposure classes defined based on EN206-1: XO, XC, XD, XS – effects of various types of corrosion, XA – chemical impact and XF – effects of cold cycles, which, as mentioned above, indicates that the compression strains within the compressed concrete shall not exceed $k_1 \cdot f_{ck} = 0.6 \cdot f_{ck}$.

4.4. 3D solid geometric model of of Ignalina NPP building 158

3D solid geometric model of Ignalina's NPP Building 158 has been developed using geometric modelling (3D CAD) software Solidworks (Fig. 10). It includes a soil layer of 4 m thickness, containing installed engineering base of 1.6 m thickness, concrete levelling layer of 100 mm thickness, reinforced concrete foundation slabs of 400/600 mm thickness (Fig. 11b and c), exterior walls assembled using reinforced concrete plates of 300 mm thickness and concrete blocks of 300 mm thickness with a concrete filling of 200 mm thickness, internal partitions of 300 mm thickness, flooring of the first floor made of reinforced concrete

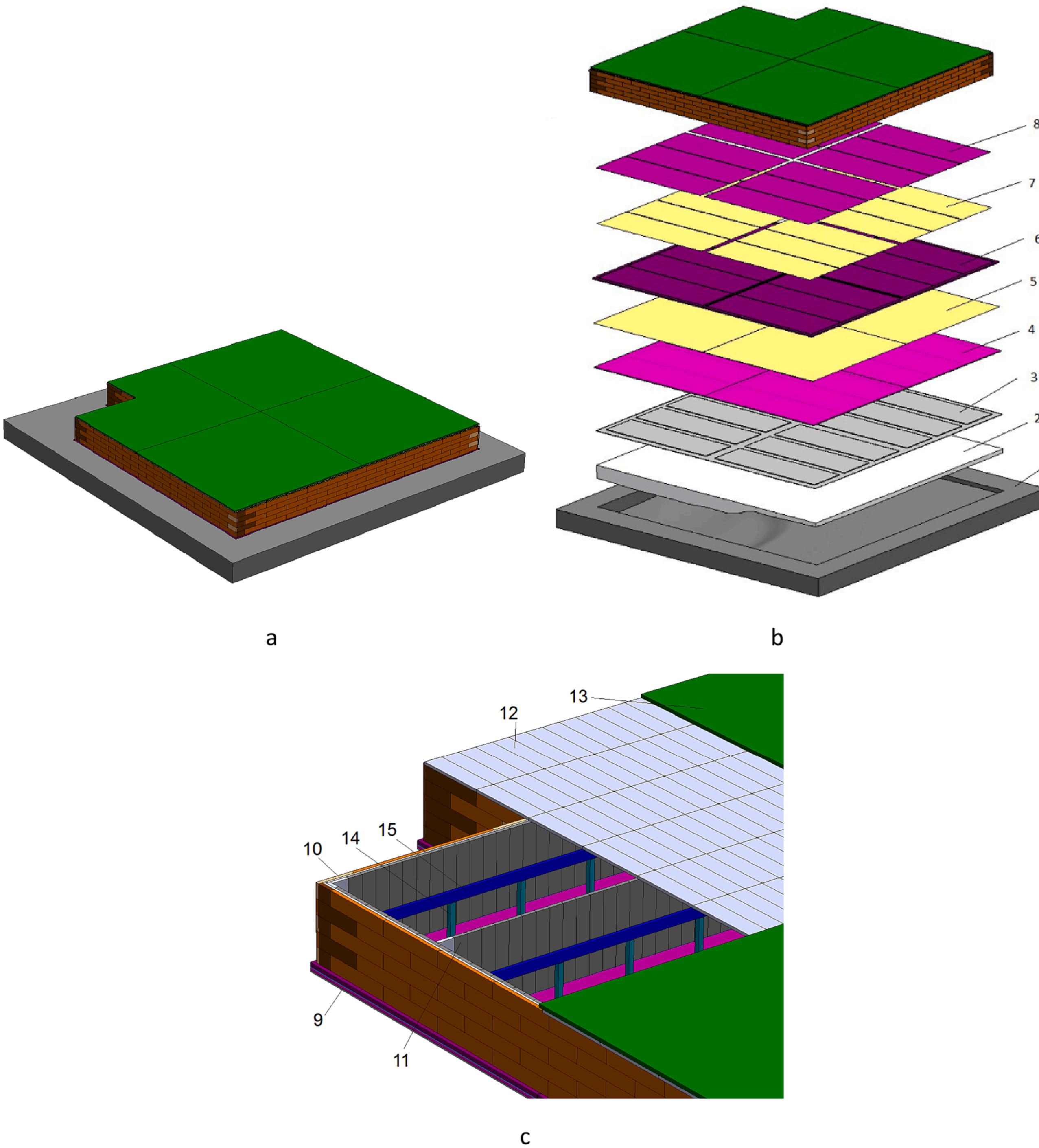


Fig. 11. Spatial volumetric geometric model of Ignalina NPP building 158: a – general view; b – scattered view of the bottom part; c – fragment of the model without one of the top flooring panels and part of the bottom flooring panels: 1 – soil; 2 – engineering base ("bed"); 3 – levelling concrete layer; 4, 6 and 8 – the lower, middle and upper concrete layers of reinforced concrete foundation slabs, respectively; 5 and 7 – lower and upper equivalent reinforcement layers of reinforced concrete foundation slabs, respectively; 9 – reinforced concrete foundation slabs (include positions 4 to 8); 10 – exterior walls (3 layers); 11 – interior walls; 12 – lower layer of the flooring; 13 – upper layer of the flooring; 14 – pillars; 15 – collars.

plates (2 layers – the bottom layer assembled using reinforced concrete plates of 220 mm thickness and the upper layer of monolithic reinforced concrete plates of 280–480 mm thickness), as well as bearing supporting pillars (of 400x400 mm cross-section) and collars (of 300x800 mm cross-section) (Fig. 10). All structural elements of the building are modelled as solid bodies except the equivalent reinforcement layers of the foundation slabs – they are shown as zero-thickness surfaces in the geometric model (Fig. 11b, positions 5 and 7).

As described in Chapter 1, during the conversion of the storage facility into the Repository, structural units of the second floor of the building shall be dismantled and, therefore, they have not been included in the geometric model of building 158.

4.5. *Computational model of building 158 transformed into a Repository*

Spatial computational model of Building 158 has been developed in order to analyse bearing capacity of Ignalina NPP building 158 subject to the own weight of structures and weight of bitumen filling based on the spatial geometric model of Ignalina NPP building 158 described in Chapter 4.4.

After loading the spatial solid geometric model of Ignalina NPP bBuilding 158 to the ANSYS Workbench all the elements have been assigned with the relevant mechanical properties of materials (soil, concrete, and steel) (Chapter 4.2) and boundary conditions (fixtures and loads). Specifically, the properties of the materials were specified for the equivalent reinforcement layers of the foundation slabs – assuming that thinner reinforcement rods of 12 mm diameter are laid in concrete maintaining a 200 mm step, it was determined that thickness of the equivalent reinforcement layer reaches 0.5655 mm, therefore, these layers were modelled as surfaces which thickness is specified in the computational model. Completely rigid contact type connections were formed between the contacting elements of the model. In the computational model of building 158, the bottom surface of the soil element was rigidly fixed (by capturing all degrees of freedom), and the gravity load was set for the entire model by including the acceleration of the free fall in the vertical direction equal to $g = 9.8\ m/s^2$. Floor of the canyons was loaded with a pressure (or distributed force) corresponding to the weight of the filling contained in canyons as described in Clause 4.1 (Fig. 12).

Volumes of the model have been divided into second order (with additional nodes in the middle of the edges) solid type finite elements (mainly of a rectangular parallelepiped shape using *Hex dominant* mode for meshing solids and surfaces). Their nominal size ranged from 200 mm (pillars and collars) to 1500 mm (soil and top roof panels). The total number of finite elements in the model is 176,248 and the number of nodes is about 1,177,677. Fig. 13 represents general view of the computational finite elements model of Building 158, Fig. 14 – fragments of the model.

4.6. Computational analysis of building 158 converted into the repository

During the calculations, it is assumed that the engineering barrier consists of the following layers of soil of various properties (F. 42b) formed on the bearing structure and 0.2 m thick concrete slabs of the barrier rested on the flooring of building 158 along all walls of the building (from the lower to the upper): drainage layer of 0.2 m thickness (fine sand); insulating layer of 2.0 m thickness (limnoglacial clay); drainage layer of 0.3 m thickness (gritty sand); protective layer of 0.7 m thickness (moraine clay); drainage layer of 0.5 m thickness (dusty sand); drainage layer of 0.7 m thickness (gravel); drainage layer of 0.8 m thickness (crushed stone); vegetation layer of 0.2 m thickness.

The mass load of the engineering barrier corresponds to the mass effect of the barrier mass of 5.8 m thick soil layers of various properties (a distributed load of 2.3 MPa was assigned to the upper surfaces of the 300 mm thick concrete slabs of all walls).

Results of computations are expressed as the distribution fields of the model total displacements (mm) shown in Fig. 15, the distribution fields of normal stresses (MPa) in the X, Y and Z axes shown in Figs. 16-18, respectively (the positive values of the normal stresses in the ANSYS Workbench software correspond to the tensile state of the material, the negative values to the compressive state). Fig. 19 shows distribution fields of elastic deformation of concrete elements in the directions of X, Y and Z axes, Figs. 20 and 21 provide for distribution fields of equivalent Von-Mises stresses (MPa) and equivalent elastic deformations (strains) in the equivalent reinforcement layers of foundation slabs respectively.

As shown in Fig. 15, the maximum total displacements of the model range from 25.5 to 28.6 mm (for walls and floorings of the first floor). Changes in displacements through the object are quite even (no sudden jumps or unexplained nature variation of displacement distribution). It should be noted that the aforementioned values of displacements include the deformation of the soil and the engineering base (with displacements of 19.1–22.2 mm). Excluding the latter, the “settling” of the structural elements of the building 158 under these loads would be just 6 mm.

As shown in Figs. 16-18, the values of the normal stress components in the main concrete elements of the object (foundation plates, walls and flooring of the first floor) in both tensile and compressive concrete layers in the X, Y, and Z axes directions (system of coordinates of the model is provided in Fig. 7) are respectively 1.2/−1.81, 2.44/−4.59 and 1.18/−2.33 MPa (the positive values correspond to the tensile state of the material, the negative – to the compressive state). Maximum compression of the concrete is seen in the Y axis direction (vertical) (maximum normal stress – 4.59 MPa – in the flooring above the canyon internal walls and along the pillars supported by the collars), tensile – also in the Y axis direction (2.44 MPa – in the flooring plates above canyon walls). For the other axes (X, Z) the stresses are less for approximately two

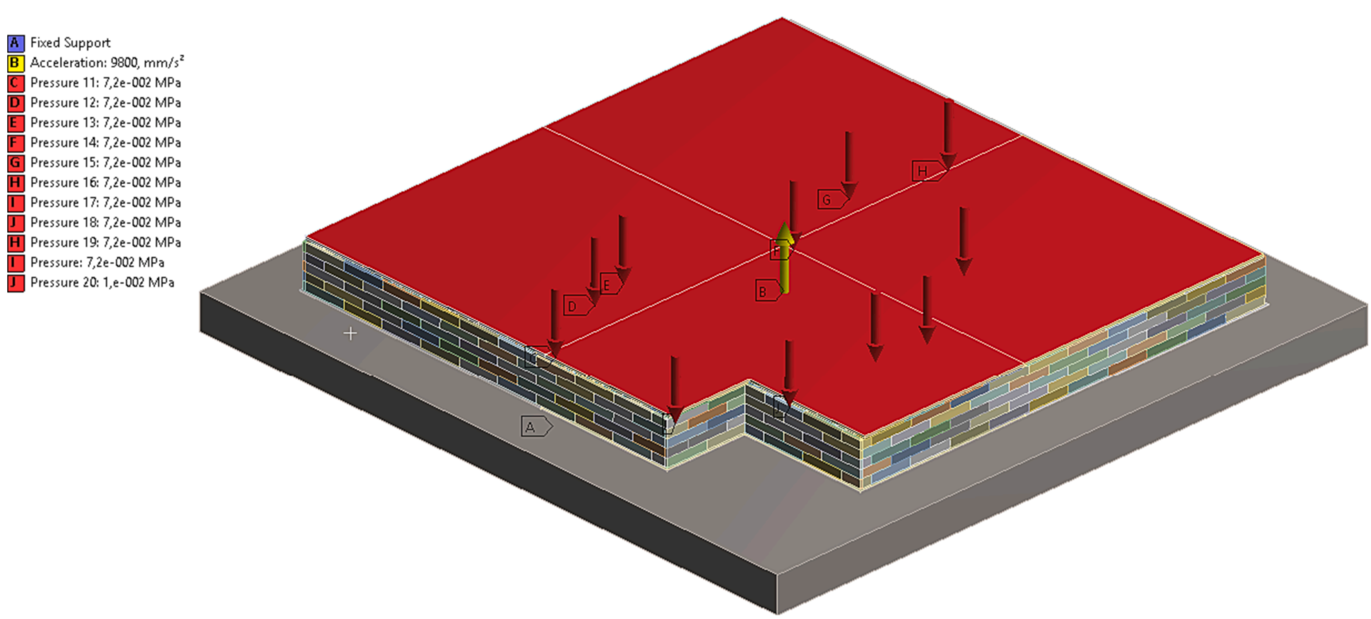


Fig. 12. Computational model of Building 158 with the applied boundary conditions: with all the canyons completely loaded (filled up to the ceiling) with bituminised RW and gravity load of engineering barrier installed on the flooring of the building in case of conversion of the Storage facility into the Repository.

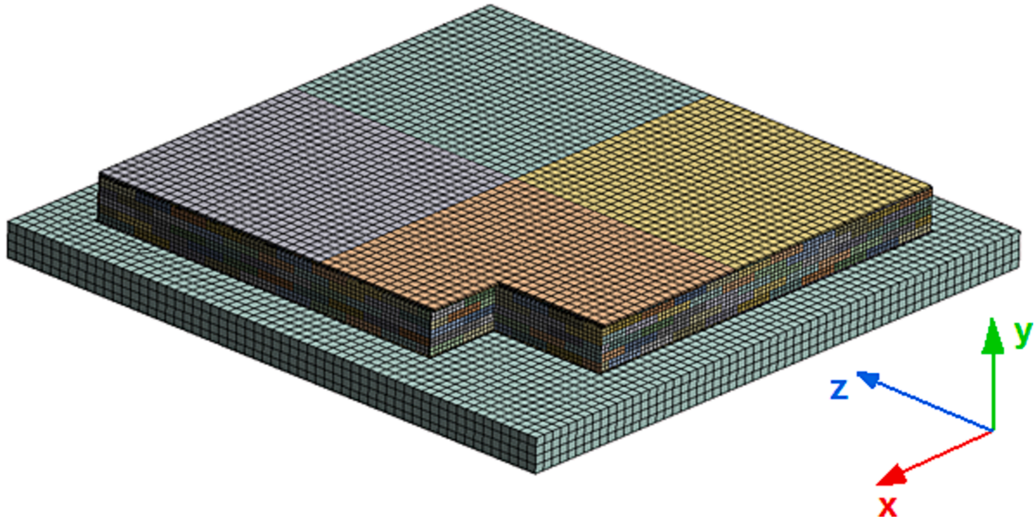


Fig. 13. General view of the computational finite elements model of entire building 158.

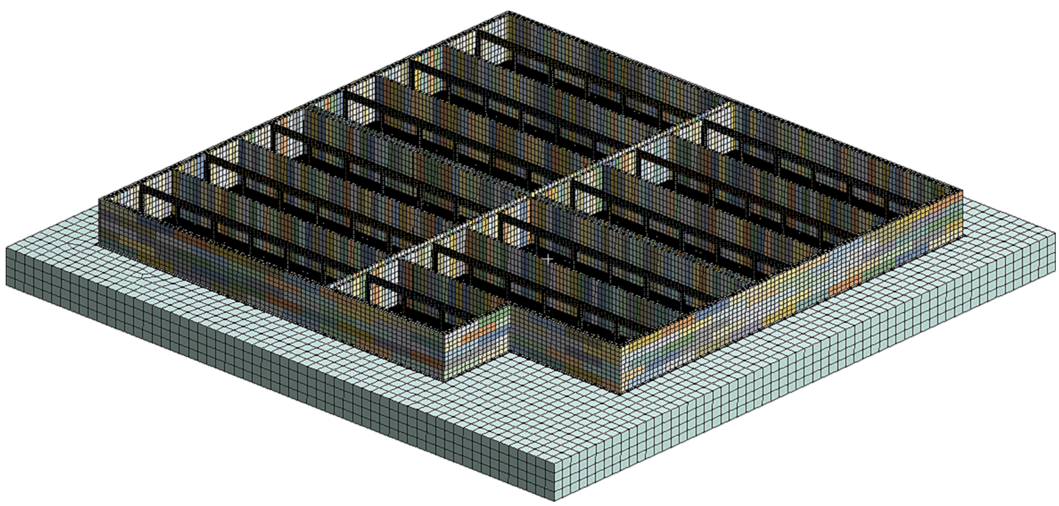


Fig. 14. Fragments of computational finite elements model of building 158: a – without flooring elements; b – magnified view of three canyons block.

times.

State of concrete deformation is analogous: maximum compressive and tensile deformations (-0.0001544 and 0.0000825, respectively) occur in the Y axis direction (Fig. 18b, system of coordinates is provided in Fig. 13). Compressive deformations in the X-axis are less than twice (-0.000063), tensile deformations are also less than about 2.5 times (0.000033), in the Z axis direction concrete compressive and tensile deformations are also less than twice (−0.000081 and 0.000045 respectively).

Due to the relatively uniform loading and subsequently quite even deformation of the building 158 (in terms of the building area), the stresses in the equivalent reinforcement layers of the foundation slabs are also quite low – 4.42–2.62 MPa (in the upper and bottom layers respectively). Their peak areas are located near the junctures of loaded and empty canyons (Fig. 19). Equivalent elastic deformations of reinforcement are also small: of the upper layer – 0.0000221, of the bottom layer – 0.0000135 (Fig. 20).

As shown in chapter 4.3, the strength of reinforced concrete structures of building 158 after the transformation into a repository.shall be assessed according to the following criteria:

Based on the critical limit state

- maximum reinforcement deformation in the upper reinforcement layer of the foundation plates $0.0000221 < 0.01$ (Fig. 21a).
- maximum reinforcement deformation in the bottom reinforcement layer of the foundation plates $0.0000135 < 0.01$ (Fig. 21b).
- concrete deformation in the compressive layer $0.000154 < 0.002$ (Fig. 19b).

Based on crack opening width

$$\varepsilon_{rm} = \frac{2.87 \times 10^{6}}{2.1 \times 10^{11}}(1 - 1 \times 0.5 \times 1) = 10.53 \times 10^{-6}$$

$$s_{rm} = 50 + 0.25 \times 0.8 \times 0.5\frac{12}{\frac{1}{400}} = 530$$

$w_k = \beta s_{rm}\varepsilon_{rm} = 1.7 \times 530 \times 6.83 \times 10^{-6} = 0.0095$ mm <0.3 mm, here w_k – crack opening width.

Due to the long service life of the building, the following two other conditions are checked:

- Deformations of the reinforcement shall not be as high as the plastic deformation limit – *the relevant condition is met (see assessment on the critical limit state)*.

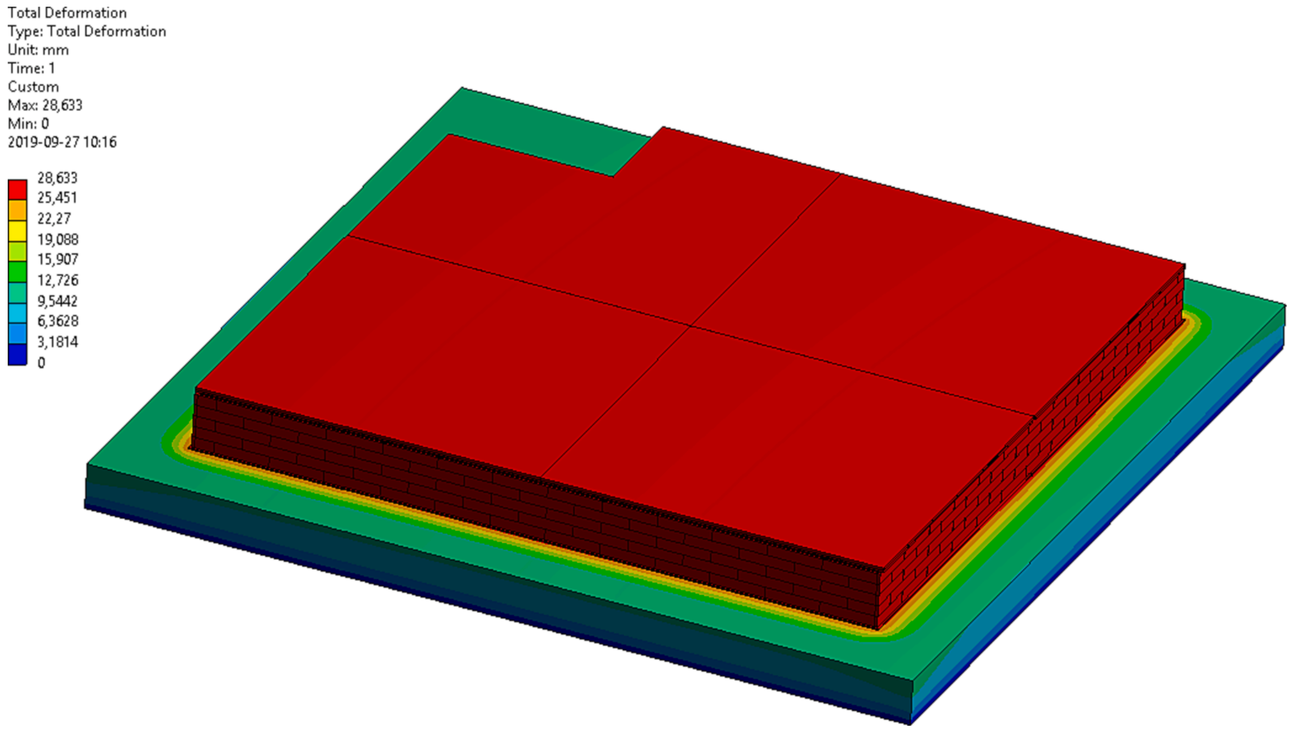
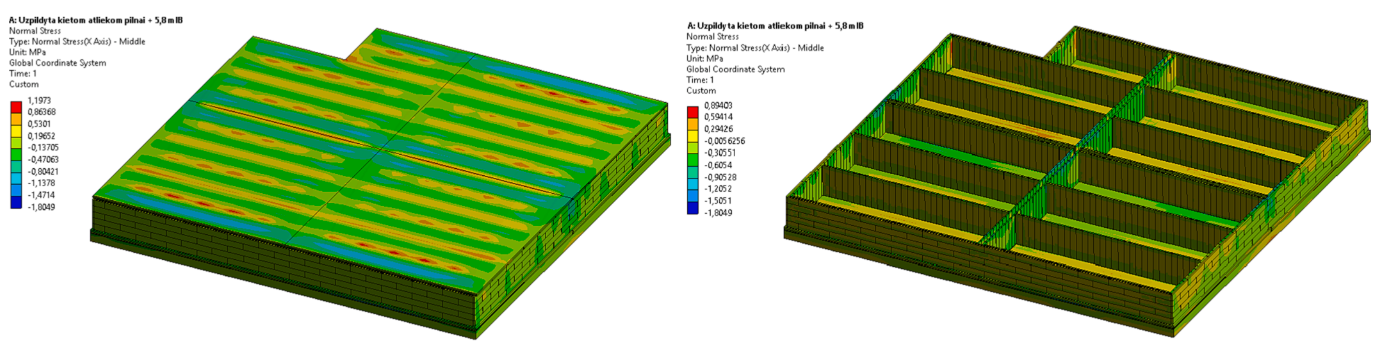


Fig. 15. Total displacements fields of Ignalina NPP building 158 model (mm).



a b

Fig. 16. Distribution fields for the normal stresses of Ignalina NPP building 158 model (MPa) in the X-axis: a – of the entire building; b – of the building with hidden upper plates of the roof and flooring blocks.

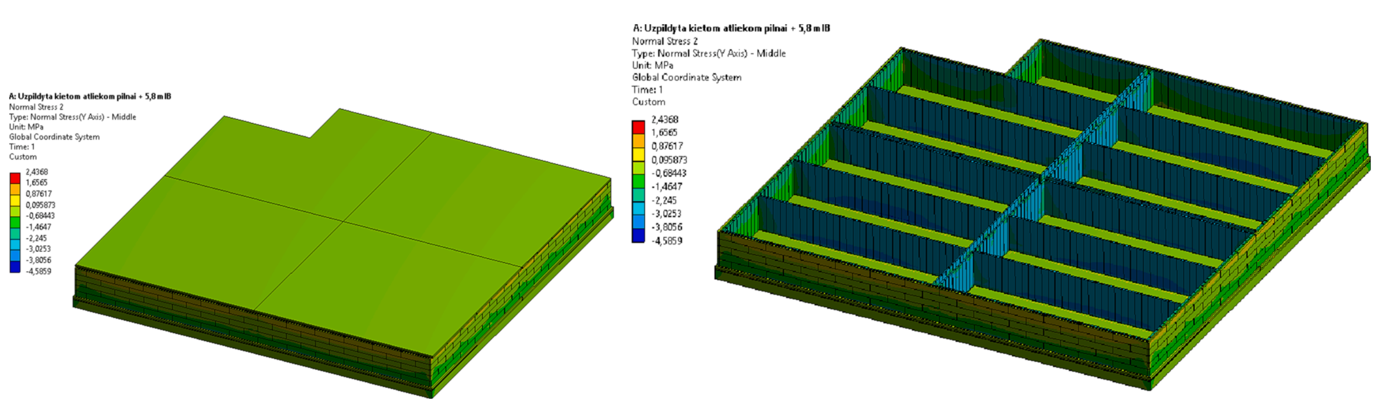


Fig. 17. Distribution fields for the normal stresses of Ignalina NPP building 158 model (MPa) in the Y-axis: a – of the entire building; b – of the building with hidden upper plates of the roof and flooring blocks.

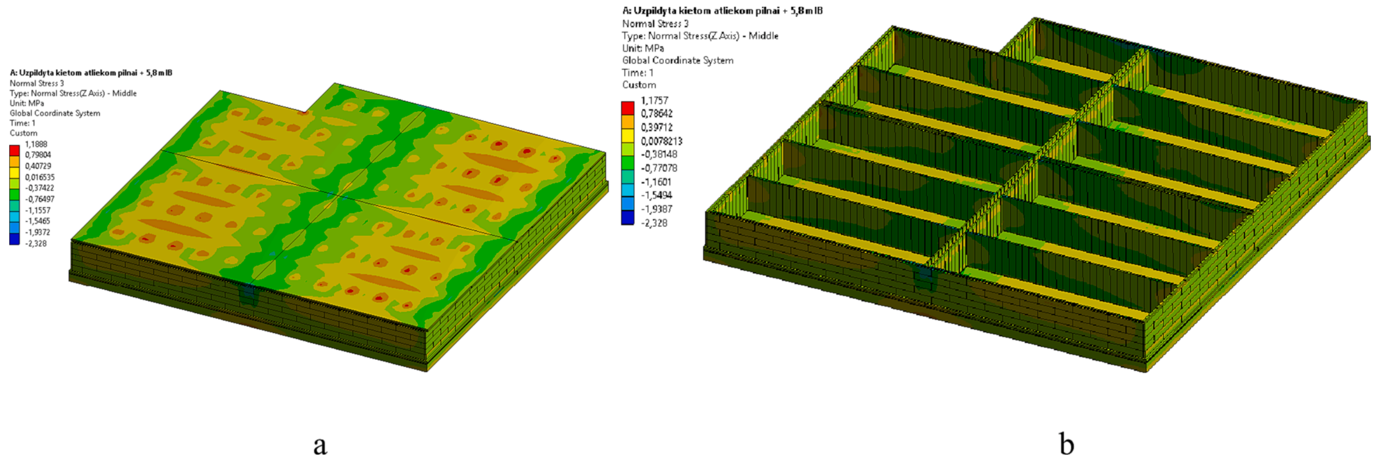


Fig. 18. Distribution fields for the normal stresses of Ignalina NPP building 158 model (MPa) in the Z-axis: a – of the entire building; b – of the building with hidden upper plates of the roof and flooring blocks.

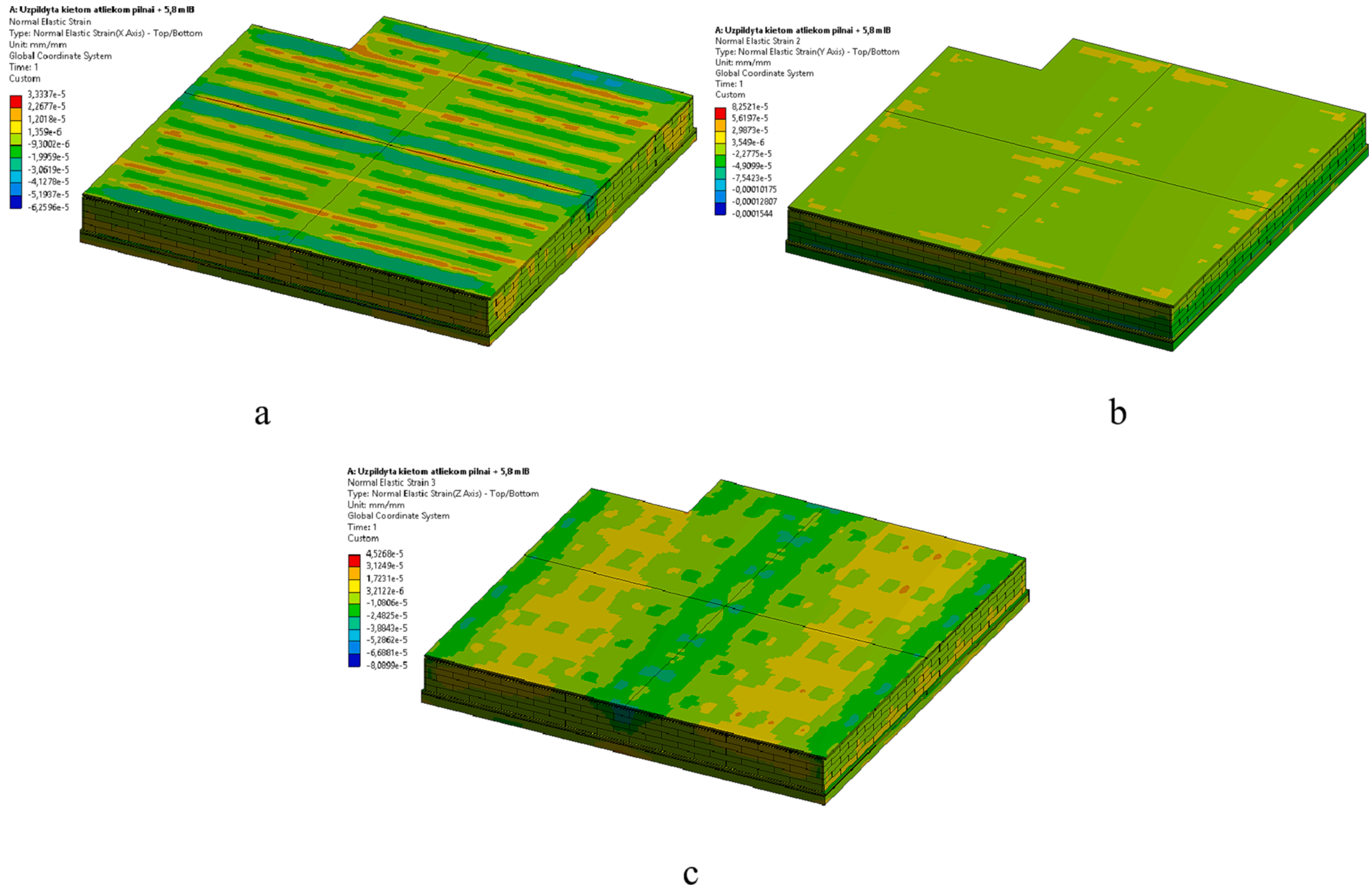
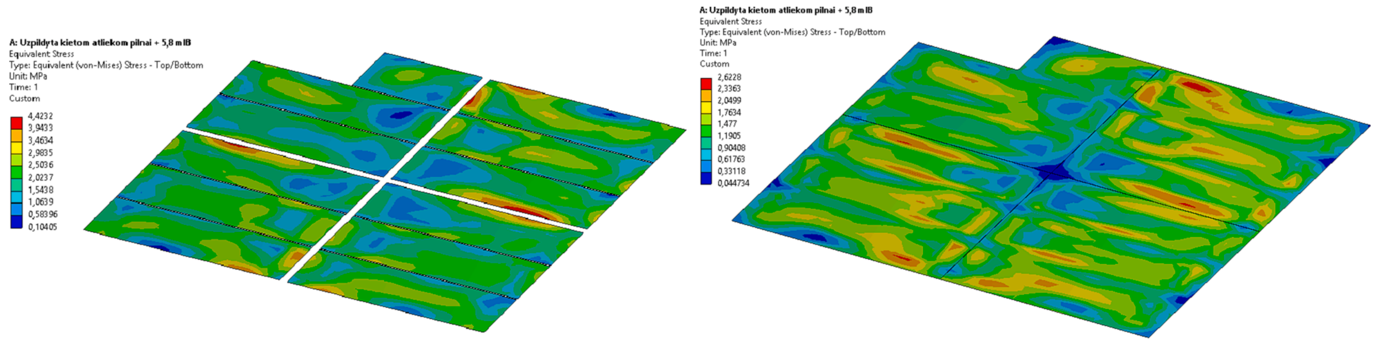


Fig. 19. Fields of distribution of elastic deformation components of concrete elements of Ignalina NPP building 158 model: a – in the X axis direction; b – in the Y axis (vertical) direction; c – in the Z axis direction.

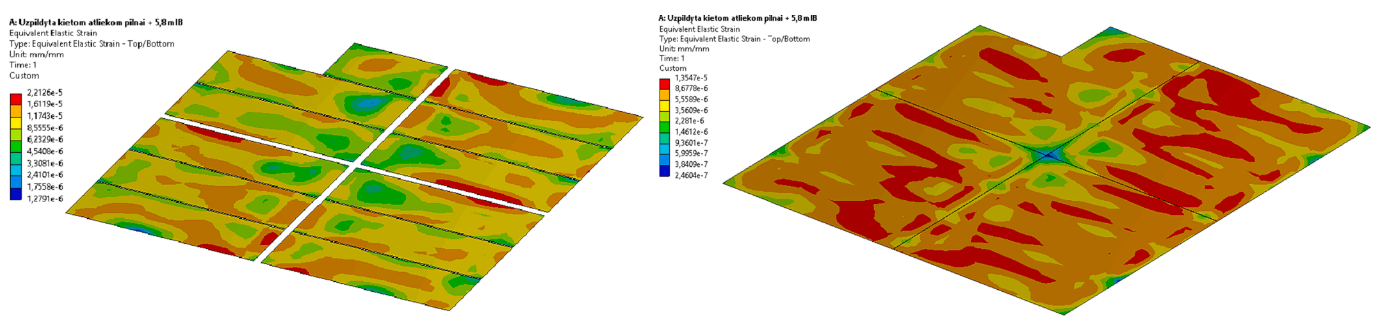
- Compression strains within the compressed concrete layers (maximum in the Y direction, Fig. 17b) shall not exceed $0.6f_{ck}$ (as rated in Eurocode-2 in accordance with EN206-1 for Class XF concrete impact – impact of cold cycles): 4.59 MPa $<$ $0.6 \times 15/1.35=$ 6.67 MPa – *the relevant condition is met.*

It should be noted that the analysis of stress-strain state of the concrete of building 158 was performed based on the computation model excluding the weight load of the structural elements of the second floor, and of the structural steel reinforcement elements in the plates of the walls and flooring of the first floor, as well as bearing structures of the engineering barrier – its mass has been evaluated by loading the upper surfaces of plates of all walls (of 300 mm thickness) of building 158 applying the force equal to the total mass of the barrier (mass of the bearing structures and a soil layer of the engineering barrier).



a b

Fig. 20. Distribution fields of equivalent Von-Mises stresses (MPa) in the elements of the equivalent reinforcement layers of foundation slabs of Ignalina NPP building 158 model: a – the upper layer; b – the lower layer.



a b

Fig. 21. Distribution fields of elastic deformations (equivalent strain) in equivalent reinforcement layers of foundation plates of Ignalina NPP building 158 model: a – the upper layer; b – the lower layer.

5. Summary and conclusion

3D solid/surface geometric and computational models of Ignalina NPP bituminised radioactive waste storage facility – building 158 have been developed based on the current situation; the models include the installed engineering base, concrete levelling layer, monolithic reinforced concrete foundation slabs with equivalent reinforcement layers, exterior and internal walls assembled using reinforced concrete plates, and flooring of the first floor of the building (containing two layers).

When determining the opportunity to reconstruct and convert the storage facility into the repository and analysing the stress and strain state in the elements of structural units within the scope of the aforesaid determination, all the requirements of valid legal acts (Eurocode-2 and STR) applicable to the reinforced concrete structural units have been complied with.

Computation of stress-strain states of building 158 converted into the repository under the loads described in chapter 4.1 herein shows the sufficient strength of concrete elements of the construction both in terms of critical limit state and crack opening width, reinforcement deformations or compression strains within the compressed concrete layers. The reserve in terms of compressive strains in the compressive concrete layers is equal to about 1.2 times (6.67<8.22 MPa), for the reinforcement it reaches several hundred times (0.0000221<0.01), in terms of crack opening width – 30 times (0.0095<0.3). Based on the provided information it could be stated that structures of the storage facility are suitable for reconstruction and conversion into the repository. Stability and durability of the repository structures will directly depend on the waterproofing finishing of reinforced concrete elements and imposed number of thermal cycles (impact of cold cycles (XF in accordance with Eurocode-2)), when applying the permissible compression strains for the concrete resistance under the factor K1 = 0.6.

CRediT authorship contribution statement

V. Eidukynas: Methodology, Software. **G. Dundulis:** Conceptualization, Writing – original draft. **D. Eidukynas:** Writing – review & editing, Visualization.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

The data that has been used is confidential.

Acknowledgments

The authors would like to express their deep gratitude to the specialists at Svertas Group for the proposed idea of reconstruction of bituminous radioactive waste storage into a landfill, as well as to the administration and technical staff at the Ignalina NPP for providing the information regarding the operational procedures and operational data.

Funding

This research was funded by from European Union Ignalina program grants, No. 1B.15/01/BWR.01 “Bituminized radioactive waste Storage Facilities design of transformation into a Repository” contract 2022-11-30, No. St-150(13.74E) under implementation of the activities of the B20 project of the Ignalina nuclear power plant.

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