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Feasibility Study and Optimization of the Structural Design of Locks made out of Plain Concrete

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ABSTRACT

Modern ship locks are made of steel reinforced concrete. A crucial issue, similar to other hydraulic structures, is the intensive exposure to water and therefore consequently the contact to a highly varying physical and chemical environment. This may cause rapid deterioration of the concrete cover due to the ingress of chloride, oxygen and moisture leading to excessive corrosion of the embedded reinforcement. Consequently, a strong decrease in structural strength is inevitable. Against this background a study was conducted in which only a plain concrete structure was considered. This construction method had been used successfully in former times and was then replaced by reinforced concrete constructions in the 1960s. The study considered a typical concrete ship lock structure without any reinforcement, with length of 190 m, width of 12.5 and a fall of 10 m. The structural analyses are focused on the lock chamber walls as they are the most labor and cost intensive components rather than on the lock head and gates. The structural and geotechnical verifications were conducted in accordance to latest Eurocodes and German standards along with Finite Element Analysis using ANSYS. The structure was designed for three extreme operating conditions depending on different water level inside the chamber. Furthermore dimensional optimizations are preformed using linear programming and sensitivity analysis is conducted by variation of input parameters. Whereas this paper would only focus on effects due to variation in groundwater level, concrete and soil type. The findings suggest that the most critical loading condition is when there is no water in the chamber. In this case a gravity wall base length of 13.28m is required to reach sufficient stability. However, the bearing capacity of soil and the tensile strength of the concrete are the most critical safety checks for this type of structure.

Keywords: Ship-Lock Structures, Plain Concrete, Static Structural Analysis, ANSYS, Feasibility, Optimization

1. INTRODUCTION

Steel reinforced concrete can withstand high stresses in slender constructions which make it the obvious choice for hydraulic structures in general and modern ship locks in specific. However, these structures are permanently in contact with water and therefore subjected to highly variable physical and chemical environments depending on their location and usage. Processes like carbonation, freezing-thawing cycles and acid attack progressively reduce the durability of the concrete structure. Rapid deterioration of concrete cover leads to an increasing ingress of chloride, oxygen and moisture causing excessive corrosion of the embedded steel reinforcement and a strong decrease in material durability and service life (Westendarp et al. 2014). In addition, “Navigation structures are subjected to significant weathering and deterioration that can result in spalling, scaling, and increased surface roughness along with scour and erosion in underwater locations.” (USACE 2016).

It had been observed by the experts at German Federal Waterways Engineering and Research Institute (Bundesanstalt für Wasserbau, BAW) that the maintenance requirement for reviving the structural integrity of young reinforced concrete structure is comparatively higher when compared with older plain concrete structures. Similar problems are reported in United States navigation infrastructure, “Steel and other metallic materials may be damaged by corrosion, fatigue, mechanical overloading, stress-assisted corrosion and embrittlement” (USACE 2016). In addition to high susceptibility of reinforcement to damage, it was found that the most dominant failure mode in existing hydraulic infrastructure under the BAW and Waterways & Shipping Board (WSV) was cracking, which was experienced in almost 25% of inspected structures (Kunz and Bödefeld 2001). The crack allows the
penetration of water and air which additionally aids corrosion and exponential decrease of stress bearing capacity of structure.

The construction method employing only plain concrete in hydraulic structures had been extensively used in former times until in 1960s when the reinforced concrete design was introduced. It is estimated by BAW that about 65% of the existing structures in the waterway’s infrastructure are made out of plain concrete or light reinforced concrete. Most of the old unreinforced infrastructure still fulfills its functional requirements with little required maintenance. In view of this advantage a study was initiated by BAW in cooperation with Department of Hydraulic Engineering and Water Resources Management, University of Stuttgart with the research target of optimized structural and dimensional requirements of a modern ship lock structure. The research employed the concepts of using plain concrete under consideration of the current standards and design philosophies of Eurocodes and German standards. Similar concepts have been used in the past and reported in (PIANC 1986) and (Partenscky 1984) but the ship size and functional requirement of that time were lesser than present times. Therefore, a typical modern ship lock was chosen with a length of 190 m, width of 12.5 m with a fall of 10 m was selected.

The main focus of the initial design is on the lock chamber walls due to two reasons. Firstly, because it is well-known that concrete cannot withstand high tensile forces, therefore using plain concrete in head structures is not possible. Secondly, the lock chambers are the most labor and cost intensive component, hence its design and optimization would have a major impact on the feasibility of building such a structure. In order to assure the stability and safety of the structure, compliance with all current codes and standards is mandatory. The static structural and geotechnical verifications are made and additional verification for internal stresses was conducted using Finite Element Analysis of ANSYS. This study is conducted in accordance with the standards and design philosophies of Eurocodes (EN 1990, 1991, 1992, 1997) and German standards (i.e. DIN 19702). The structure was designed for three operating conditions, when the water level is at maximum, minimum operation water level and when there is no water in the lock chamber (maintenance condition). The last part of the research included the optimization of structural dimensions for each of loading conditions and varied input parameters. A literature review was conducted for selection of appropriate optimization algorithm. Several algorithms have been used for various objective functions for hydraulic structures, PSO (Particle Swarm Optimization Algorithm) was employed for hydraulic structural optimization field (Xinmiao et al. 2008), Genetic Algorithm (GA) was used in optimization model for optimum dimensions of the dam section under dynamic loadings due to seismic excitation (Ahmed et al. 2014), computation and comparison was conducted for optimized dimensions of a concrete gravity dam structure using gravitational search algorithm; particle swarm optimization; weighted least squares support vector machine (Salajegheh and Khosravi 2011). But all these advanced methods are requires high computational power and input data. Therefore a more generalized method “Linear programming” was used, considering the advantage that it requires no generation of new data points and its functioning could be understood intuitively. Consequently for this study the dimensional optimizations was performed by using linear programming for design loading cases (water level in chamber) as well as for assessment of most sensitive parameters by varying the groundwater level, concrete type and soil type.

2. METHODOLOGY

2.1. Codes and Standards

In the last few decades Europe has seen a major shift in engineering design methodology with the inclusion of limit states design in its standards. This study considers both the European codes (EN) as well as German codes (DIN). The standards that were considered in the study are enlisted below.

European Standard (Eurocodes) with German Annexes

- DIN EN 1990 Eurocode 0 - Basis of structural design
- DIN EN 1991-1-1 Eurocode 1: Actions on structures - Part 1-1:
- DIN EN 1997-1 Eurocode 7: Geotechnical design - Part 1: General rules

DIN - Deutsches Institut für Normung (German Institute for Standards)

- DIN 19702 Solid structures in hydraulic engineering – Bearing capacity, serviceability and durability
Since the Eurocodes were used, the structure was designed and analyzed against ultimate and serviceability limit state using the corresponding partial safety factor for different actions and materials. This ensures that the design resistance ($R_d$) should not be less than the design effects of actions ($E_d$). Furthermore a “safety ratio” was calculated which is the ratio between design resistance and design effect of actions. The key purpose of safety ratio is to translate all calculation of each limit state to a single value which later on assisted in simplification of the optimization process. The limit states considered in the research include limit state for failure by hydraulic uplift (UPL), by loss of static equilibrium (EQU), by failure of structure (STR) and geotechnical (GEO) failure.

### 2.2. Design load conditions and materials

Ship lock structures experience diverse environments; hence several loading conditions, corresponding shapes and sizes are possible. One of the investigated shapes consists of three disjoint structures, two concrete gravity retaining walls and an unreinforced concrete base slab. The preliminary shape and dimensions of the structure is shown in Figure 1. The applied loads include hydro-static pressures, effective earth pressures, ground water forces, surcharge loads, up-lift pressures, self-weight of structures, weight of overburden soil. The application of these forces is indicated in Figure 2.

![Figure 1. Shape and initial dimensions of the locks structure](image1)

![Figure 2. Forces and pressures applied on walls of the lock](image2)

The combinations of these forces lead to three design loading conditions. The material considered for the study was concrete class C 20/25, C 25/30 and C 30/35. The material strength and partial safety factor for the design compressive and tensile strength were taken from EN 1992(2014). For the design conditions concrete class C 20/25 was considered with design compressive strength ($f_{cd}$) of 9.33 MPa and design tensile strength ($f_{cd,0.05}$) of 0.70 MPa. The properties of the earthen material for design conditions include the specific weight of the unsaturated soil with 20 KN/m$^3$ and of the saturated soil with 10 KN/m$^3$, the angle of internal friction was set to $\phi = 30^\circ$. For initial conditions, the earth pressure is 50% active earth pressure and 50% passive earth pressure resulting in an effective earth pressure coefficient of $K_{eff} = 0.39$. 

- DIN 19703  Locks for waterways for inland navigation –Principles for dimensioning and equipment
- DIN 1054  Subsoil –Verification of the safety of earthworks and foundations –Supplementary rules to DIN EN 1997-1
Nine other conditions were evaluated in the sensitivity analysis by parameter variation. The following Table 1 indicates the conditions and its respective changes.

<table>
<thead>
<tr>
<th>Group No</th>
<th>Parameter</th>
<th>Situation</th>
<th>Case</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Water level in the chamber</td>
<td>Highest operational water level in the chamber ((w.l +14.0 \text{ m}))</td>
<td>Design conditions</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Lowest operational water level ((w.l + 4.0 \text{ m}))</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>No water in chamber/maintenance case ((w.l + 0.0 \text{ m}))</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Water level in the ground</td>
<td>Water at 2/3 of total height of structure</td>
<td>Sensitivity analysis 1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Water at the ground surface level</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Water at lowest level of structure</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Different concrete strengths</td>
<td>Concrete class C 20/25</td>
<td>Sensitivity analysis 2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Concrete class C 25/30</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Concrete class C 30/35</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Soil type</td>
<td>Fine sand (\gamma = 20 \text{ KN/m}^3, \gamma' = 10 \text{ KN/m}^3, \phi = 30^\circ)</td>
<td>Sensitivity analysis 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Clay (\gamma = 19.5 \text{ KN/m}^3, \gamma' = 9.5 \text{ KN/m}^3, \phi = 22.5^\circ)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gravel (\gamma = 21 \text{ KN/m}^3, \gamma' = 12 \text{ KN/m}^3, \phi = 37.5^\circ)</td>
<td></td>
</tr>
</tbody>
</table>

### 2.3. Static verifications & Finite Element Analysis

For each limit state function corresponding actions, materials and partial safety factors were selected from the Eurocodes and DIN as mentioned in the section earlier. In total seven static verifications were made, which include safety against uplift, overturning, sliding, eccentricity, bearing capacity failure, shear forces and compressive strength check. The respective resistances and actions were calculated and a safety ratio was evaluated for each case. The details are given in the Table 2.

<table>
<thead>
<tr>
<th>Static verifications</th>
<th>Limit State condition</th>
<th>Remark</th>
<th>Reference literature</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure by hydraulic uplift</td>
<td>Vertical design destabilizing force  (&lt;) Vertical design stabilizing force</td>
<td></td>
<td>EN 1997-1, DIN 1054</td>
</tr>
<tr>
<td>Overturning stability</td>
<td>Factored overturning moments (&lt;) Factored reacting moments</td>
<td>For both pivot points, heel &amp; toe</td>
<td>EN 1997-1, DIN 1054</td>
</tr>
<tr>
<td>Sliding resistance</td>
<td>Total horizontal sliding forces (&lt;) Resisting forces</td>
<td>No cohesion between concrete &amp; soil</td>
<td>EN 1997-1, DIN 1054</td>
</tr>
<tr>
<td>Load eccentricity</td>
<td>Factored load eccentricity (&lt;) Allowable eccentricity</td>
<td>for both conditions Only permanent force &amp; with variable force</td>
<td>DIN 1054</td>
</tr>
<tr>
<td>Bearing capacity check</td>
<td>Stress on soil (&lt;) Allowable ultimate bearing capacity of soil</td>
<td>For both about heel &amp; toe</td>
<td>DIN 1054</td>
</tr>
<tr>
<td>Shear forces check</td>
<td>Shear stress (design load) (&lt;) Concrete design strength(shear &amp; compression)</td>
<td>Shear stress in the concrete from axial load</td>
<td>EN 1992-1-1</td>
</tr>
<tr>
<td>Compressive strength check</td>
<td>Compressive stresses (&lt;) Concrete design compressive strength</td>
<td>With &amp; without pore water pressure</td>
<td>DIN 19702, EN 1992-1-1</td>
</tr>
</tbody>
</table>

In order to evaluate the internal stresses and possible failure of the material, Finite Element Analyses (FEA) was conducted. ANSYS Mechanical was used for the FEA for the proposed structure. For the current study a traditional approach was adapted for the static analysis of the subject structure, whereas the future work could
possible include dynamic analysis. The approach included selection of element type as “Solid 185” with eight nodes and homogenous isotropic structural solid elements. The concrete material properties for FEM modelling considered material as isotropic linear elastic concrete with a young’s modulus of 30 GPa, Poisson’s ratio \(v = 0.2\) and unit density of 24000 kg/m\(^3\) with a typical concrete material failure stress strain curve. For generation of the computational mesh the triangular element shape was selected along with Ansys automatic meshing option. The boundary conditions, displacements \(U_{xyz}\) and rotation \(R_{xyz}\) were selected in accordance to the structural systems considered in the codes and standards, where the chamber retaining wall was considered a cantilever with fixation at the base only. In the current feasibility study the variation in the soil adjacent to the chamber was not considered and earth pressure was considered as a fixed load. The consideration and modelling of soil-structure interaction in the FEM model might be a part of the subsequent future studies. The pressures and forces systems considered are shown in the Figure 2. The overall handling of the FEA from element selection to post processing of results was automated by developing a code in ANSYS-APDL (Ansys Parametric Design Language).

In post processing both nodal and parametric solutions were extracted. These solutions include tensile stresses, compressive stresses, maximum horizontal shear stresses \(T_{xy}\) and maximum horizontal normal stresses on vertical planes (compressive, \(\sigma_x\)) for assessment of ultimate limit state. For evaluation of the serviceability limit state, the nodal solution of maximum displacement in x-direction was evaluated and compared with the permissible values of the standards. For validation of the FEA results, the solutions from Ansys and results of static verifications with regards to compressive stress in principle axis, horizontal planes and vertical planes are compared with those computed by ANSYS using FEA. Once an approximation is found between the analytical calculations and FEA results, the boundary conditions, mesh size, shape and force system are fixed for other loading conditions and cases.

### 2.4. Optimization and Sensitivity Analysis

Upon completion of static verification and FEA in accordance to respective limit states, each case is condensed to one value, “the safety ratio”. The ratio is evaluated by comparing calculated value of limit state (static/FEM) and values allowed by the standards. Table 2 enlists some of static verification and limit states. In general this ratio indicates the order of sufficiency of the proposed structure in context to verifications enforced by standardization, a value smaller than one means that the proposed structural dimensions are insufficient and need to be revised. Whereas, a safety ratio more than one indicates more than code required sufficiency and a need of optimization for reduction of dimensions.

As discussed before as well, there are several optimization schemes and algorithms used for structural optimization which include but not limited to Particle Swarm Optimization Algorithm PSO (Xinmiao et al. 2008), Genetic Algorithm GA (Ahmed et al. 2014), gravitational search algorithm; particle swarm optimization; weighted least squares support vector machine (Salajegheh and Khosravi 2011). Since the orientation and requirement of the subject study was limited to assess feasibility, therefore selection of method was based on ease of application and less computational power requirement. Considering this, linear programming optimization scheme was selected. A linear programming model was constructed based on the objective of minimizing the base length. The constraints of the model were that all safety ratios must be at least equal to 1.0. For acceptance of the optimal design, the performance function was the most critical verification whose safety ratio must be equal to 1.0. The optimization process did not only give the optimal dimensions of the proposed geometry but also indicated the critical parameters and limit states for various loading scenarios. Similarly each design case was optimized and dimensions corresponding to the worst case scenario were selected. Subsequently a sensitivity analysis was conducted by changing the input parameters. The influences of three parameters were analyzed, which include change in ground water level, concrete type and soil type. Each input change was applied to the dimensions finalized from design conditions and then static verification and FEA was done to assess sufficiency. For each change, safety ratios were computed to determine the most critical safety aspect and then optimization was performed to evaluate the dimensional requirements.

### 3. Results

Three design situations depending on the water level in the lock chamber, respectively water level at +14.00 m, +4.00m and no water in the chamber, were selected for this study. The initial base length of gravity wall was considered to be 12.0 m. The results of the analysis for design case 1 show that the structural dimensions are sufficient, while the eccentricity is the most critical parameter. For optimization the eccentricity check was set
as performance function and base length as parameter. The optimized length was found out to be 9.78 m. The same procedure was carried out in case of the water level at +4.00 m, which indicated the bearing capacity as critical safety check; the optimized length was 11.65 m. Also case 3 representing no water in the chamber showed that the eccentricity was the critical safety check where the safety ratio was decreased to 0.92. The corresponding optimized base length was calculated to 13.28 m and was set as design value for all further analysis. The analysis of the design conditions show that bearing capacity, eccentricity and sliding resistance are among the most critical checks. In the Finite Element Analysis, the lowest safety ratios were found for the safety check of tensile stresses, where the safety ratio decreased from 2.89 to 1.53 for the decrease of the water level in the chamber as shown in Table 3. The location of the maximum tensile stresses is at the toe shown in Figure 3. The safety ratios for shear stresses decrease from 4.12 to 1.96 with the decrease in water level. The structure is abundantly safe against compressive stresses. The safety ratios increase as the stresses inside the structure increase due to the bigger size and higher weight. This is validated from the results in static verifications. The maximum compressive stress is 1.79 MPa whereas the allowed compressive stresses are 9.33 MPa giving a safety ratio of at least 5.21. The location of the maximum compressive stress (least safety ratio) is at the heel of the structure in design case 2 as shown in Figure 3. Concluding that the optimized base length of 13.28 m is sufficient to fulfill all limit states under design conditions and loadings.

Table 3. Results of safety ratios in design conditions and optimization

<table>
<thead>
<tr>
<th>Group 1 (Design Conditions)</th>
<th>Design 1</th>
<th>Design 2</th>
<th>Design 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Std = Standard, Opti = Optimized</td>
<td>Std Opti</td>
<td>Std Opti</td>
<td>Std Opti</td>
</tr>
<tr>
<td>Optimizing parameter Base length</td>
<td>12 9.78</td>
<td>12 11.65</td>
<td>12 13.28</td>
</tr>
<tr>
<td>Uplift stability</td>
<td>2.02 2.03</td>
<td>3.25 3.25</td>
<td>4.97 4.96</td>
</tr>
<tr>
<td>Overturning stability</td>
<td>1.65 1.20</td>
<td>1.27 1.23</td>
<td>1.43 1.61</td>
</tr>
<tr>
<td>Sliding resistance</td>
<td>1.69 1.39</td>
<td>1.22 1.19</td>
<td>1.35 1.49</td>
</tr>
<tr>
<td>Eccentricity limit</td>
<td>1.42 1.00</td>
<td>1.42 1.35</td>
<td>1.13 1.34</td>
</tr>
<tr>
<td>Bearing capacity</td>
<td>1.87 1.70</td>
<td>1.02 1.00</td>
<td>0.92 1.00</td>
</tr>
<tr>
<td>Shear strength</td>
<td>2.57 2.10</td>
<td>2.67 2.59</td>
<td>2.60 2.88</td>
</tr>
<tr>
<td>Compressive strength</td>
<td>10.40 8.72</td>
<td>8.21 7.98</td>
<td>7.74 8.55</td>
</tr>
<tr>
<td>Principle tensile stress</td>
<td>2.89 2.04</td>
<td>1.33 1.32</td>
<td>1.53 1.90</td>
</tr>
<tr>
<td>Principle compressive stress</td>
<td>10.02 8.79</td>
<td>5.21 5.17</td>
<td>6.31 6.69</td>
</tr>
<tr>
<td>Max shear stress Txy</td>
<td>4.12 3.69</td>
<td>1.96 1.77</td>
<td>2.76 2.94</td>
</tr>
<tr>
<td>Max normal tensile stress σy</td>
<td>6.72 3.61</td>
<td>1.72 1.67</td>
<td>2.43 3.33</td>
</tr>
<tr>
<td>Max displacement-x</td>
<td>7.98 5.68</td>
<td>4.53 4.32</td>
<td>4.43 5.27</td>
</tr>
</tbody>
</table>

Figure 3. Tensile and compressive stress distribution through Ansys-APDL.

Another auxiliary structure considered for design was the base slab of the lock structures. Two types of construction possibilities were considered in the initial part of the research. Firstly a U frame monolithic construction where all components of the structure are connected and secondly a construction considering three disjoint sub-structures which is indicated in Figure 1. The U frame monolithic construction with plain concrete was analyzed and it was found as an unfeasible shape as the tensile stress at the connection of base slab and chamber walls exceeded the concrete design tensile strength of 0.7 MPa. The details of this design and analysis
are not a part of this paper. Whereas the plain concrete base slab design and consideration for a disjoint system would be discussed briefly here. Similar to the chamber wall structure, the base slab was also considered for three design cases, two of which meaning the critical. Firstly, when the water level in the chamber is at maximum operational level and secondly when no water is within the chamber and only the self-weight of the structure could resist the forces. The following figure indicates equivalent factored loading system for the both critical design cases. Considering the beam theory, respective applicable standards and the evaluated bending moment the appropriate depth of the slab was calculated to be 3.0 meters.

![Design Case 1](image1.png)

![Design Case 3](image2.png)

*All values are in KN/m

All design cases considered a groundwater level up to two third of the height of the structure. The ground water level is now altered to evaluate the sensitivity of this parameter on the safety of the lock structure. To assess the variation in safety ratios due to change in ground water level, three loading conditions were assessed. First being that the ground water level equals to structure height (maximum level), it was witnessed that a rise in uplift pressure improves the bearing capacity but reduces all other stabilities making the eccentricity limits and sliding safety to be the most critical. However, the results show that a base length of 13.28 m is still sufficient for all stabilities. Although the safety ratios for eccentricity limits (1.03) and sliding resistance (1.05) with base length 13.28 m are marginally higher than 1.0 as indicated in the graph figure below. Nevertheless the optimization was conducted and an optimized base length is found to be 13.05 m. The change in ground water level was reduced to a minimum level. In this case the original base length of 13.28 m failed to fulfill required bearing capacity check and the safety ratio reduced to 0.93, this is shown in the figure with comparison to other ground water level. To fulfill this requirement the dimensions were increased and the respective optimized length was 14.0m. The following figure indicates the various static and FEM verifications for the respective structure with the variation in the ground water level where the aspects of interest are those with safety ratio around 1.
The next sensitivity analysis is conducted considering the change in concrete quality. The stability related to structure geometry and geotechnical properties are not sensitive to any small change in concrete properties. Since the changes of the strength class will keep the weight and density almost at the same value. For C20/25, C25/30 and C30/35, only the safety ratios related to the material strength i.e tensile, compressive and shear strength are improved slightly. But none of these stabilities are critical and the bearing capacity has the least a safety ratio of 1.00. Therefore the optimized base length remains at 13.28 m, regardless of an improved concrete class. This is indicated in the figure below with identical safety ratios for critical aspects.

Sand was considered as design soil, the sensitivity of a change in soil type was evaluated. The soil type was varied choosing clay and gravel with basic properties written in table 1. The results indicate that clayey soil reduces the stabilities, hence decreasing the bearing capacity ratio to 0.95. Therefore, the required base length

Figure 5. Sensitivity analysis for change in ground water level with base length of 13.28 m

Figure 6. Sensitivity analysis for change in concrete type with base length of 13.28 m
was computed to be 14.11 m. Gravel soil provided more stability and a higher bearing capacity resulting in a reduced optimized base length of 12.58 m.

A general perspective of the sensitivity analysis carried out in this study shows that for retaining gravity walls made out of plain concrete, the stability is most critically restricted by the bearing capacity of soil and eccentricity limits. However, the safety against internal stresses (tensile stresses) is the most critical aspect which needs to be corroborated.

4. CONCLUSIONS

When considering latest standards and concrete without reinforcement, for design of a modern ship lock structure, the construction arrangement consisting of three disjoint sub-structures was found more feasible, in contrast to the currently employed typical reinforced concrete U frame structure which has rapid reinforcement deterioration and maintenance issues. The suggested arrangement consists of two identical concrete gravity retaining chamber walls and a plain concrete base slab. The static verification and finite element analysis in accordance with Eurocodes and German standards was conducted for a lock with a length of 190 m, width of 12.5 m and a fall of 10 m. From the three design conditions considered it is found that critical loading conditions occur when no water is inside the chamber. The most critical verifications were the eccentricity check and bearing capacity of the soil. A linear programming optimization scheme was employed. This provided the optimized base dimension of 13.28 m. The critical loading conditions were applied to the base slab resulting into a final thickness 3.00 m, which was determined using beam theory.

Upon completion of verification of the design conditions, a sensitivity analysis was conducted to investigate the impact of the change in input parameters on the dimensional requirements and stabilities. The conducted analyses determine that stabilities are sensitive to changes in the ground water level. The increase of the ground water results into a decreased stability and lowered safety ratios for all verifications with exception to the bearing capacity verification, where the safety ratio increased. The reason for the observed reduction is the increase in vertical forces due to uplift. The second sensitivity analysis determined the influence of change in concrete type. The analysis revealed that such change will have no impact on the structure’s critical verifications i.e. eccentricity and bearing capacity which depend on the weight of the structure rather than on the strength of concrete. The third change in the input parameters was the change in soil type. The analysis showed that coarse soils provide a better environment and higher stabilities, thus reducing the dimensions of the structure. Soils having finer material or smaller angles of friction require a longer base length to guarantee the stability.
It can be concluded that bearing capacity verification is most critical, whereas eccentricities check can be ranked second for this type of concrete structure. Since the base length is a function of base pressure and eccentricity, it is recommended to elongate the base of the structure and gradually reduce the other dimensions along with the height for an optimal shape. As established before the change in concrete class does not have an impact on the optimal dimensions of the structures. The bearing capacity remains the most critical verification, which is independent of concrete strength.

The sensitivity analyses indicate that coarse soils will improve the bearing capacity and reduce the earth pressures on the walls, the proposed structural arrangements would be more feasible and stable in regions with gravel and rock foundations. Soil replacement could be an option in cases where the soil is clayey. In conditions where foundation soils have a very low bearing capacity, a pile foundation could be employed.

An increased ground water level has an impact on the sliding resistance of the structure, hence recommendations are presented. Firstly, providing a key at the bottom of the structure would improve the stability. Secondly, the application of a ground water drainage system along the earthen side of the structure is suggested. Furthermore a drainage system along the chamber walls is of vital importance when the structure is undergoing maintenance conditions (no water in the chamber). Since this loading condition is critical but at the same time temporary, the reduction of the natural ground water level in the vicinity of the structure is suggested, however this short duration will not have any major impact on the regional ground water system.

The same methodology could be adapted for the assessment of stabilities for existing hydraulic structures made out of plain concrete which consists of about 60 % of navigation infrastructure of Germany. Further studies regarding fracture mechanism and location of potential failure planes within the structure due to tensile stresses are recommended. This would ensure the safety of the overall structure especially foundations against progressive development of uplift pressures leading to hydraulic fracturing and deterioration. A better understanding of the soil-structure interaction and fluid-structure interaction considering dynamic loadings and soil variation would be highly beneficial for additional optimization on the structure in terms of shapes and dimensions. Other structural configurations involving decrease in height of retaining wall by cascading smaller lock structures in series could further improve the scheme which needs to be investigated. However, an economic and life-cycle cost analysis would be pivotal in making the final decision concerning selection and usage of lock structures made out of plain concrete.

5. REFERENCES

ANSYS Inc. (2013). ANSYS Mechanical APDL Structural Analysis, 15.0 ed. Canonsburg, USA.


