AN INTEGRATED METHODOLOGICAL APPROACH FOR THE UPGRADING OF COASTAL STRUCTURES DUE TO CLIMATE CHANGE EFFECTS

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ABSTRACT

In this work an integrated methodological approach for the upgrading of coastal structures due to climate change impact is presented. The methodology includes the definition of the performance criteria of the structures and the specification of the climate change scenario for the structure's service lifetime. Various concepts for upgrading existing emerged and submerged rubble mound breakwaters are studied and related costs are estimated. The proposed methodology is implemented for two specific coastal structures, located in Northern Greece; the rubble mound breakwater in the Port of Alexandroupoli and the submerged breakwaters in Paralia Katerini. It is shown that the influence of climate change is stronger in terms of sea level rise rather than the increase of the wave heights. Therefore the structures that are studied fail to fulfill the criteria related to wave overtopping and transmission but satisfy the stability criteria, under the climate change scenario. The final selection takes into account the upgrading costs for each alternative.

Keywords: coastal defence structures, Climate change, breakwater, upgrading

1. INTRODUCTION

The impact of climate change on coastal zone is associated with the rise of the mean sea level due to global warming (IPCC, 2007) but also due to the increased storm surge events (Isobe, 2013), that could lead to the increase of extreme wave run-up and overtopping of coastal structures (Chini and Stansby, 2012). This results in rising possibility to cause damage to existing coastal protection structures, such as the instability of armor blocks and breakwaters. The treatment of these effects is to upgrade these structures by adding/modifyng the structure elements so they comply with the original design performance criteria, as suggested by Burcharth et al. (2014) for the case of a typical rock armoured revetment.

In the present study an integrated methodological approach for the upgrading of coastal structures, such as rock armoured emerged and submerged breakwaters due to climate change impact is presented. Initially the definition of performance criteria is given, regarding the hydraulic and structural responses for such structures. A quantitative estimate of the sea level rise and the near shore wave height for specific climate change scenario is given based on available simulations and extreme wave analysis. The weak points of the structures are identified, based on the commonly used design formulas for armour stability and wave transmission and wave overtopping. Then possible alternative upgrades of the structures are given considering geometrical and esthetical restrictions. The final selection of the upgrading is based on the upgrading costs for each alternative. The proposed methodology is implemented for two specific coastal structures, located in Northern Greece; the breakwater in the Port of Alexandroupoli and the submerged breakwaters in Paralia Katerini.

2. METHODOLOGY

2.1 Setting of definitions

2.1.1 Type of coastal structures

The types of coastal structures considered in the present study are rubble mound breakwaters which are common solution for port and coastal protection under wave action. Emerged breakwaters (positive freeboard, \( R_c \)) are dissipative structures that absorb wave energy due to breaking on the front slope and turbulent flows through the breakwater core. A typical cross-section of rubble mound breakwaters is trapezoidal, consisting of rock or concrete armour units on the outside of the structure, at least two layers, an underlayer of rocks and the breakwater core of fine material. Apart from the conventional emerged breakwaters, the use of submerged breakwaters (negative freeboard, \( R_c \)) is systematically investigated in recent coastal protection projects as an environmentally acceptable solution. Their function is based on the reflection of part of the incident wave energy, partial absorption due to wave breaking and partial transmission of the wave energy shoreward. The dissipation, reflection and transmission coefficients, \( K_d \), \( K_r \) and \( K_t \) respectively depend on many parameters related to wave and structure characteristics. Figure 1 shows a typical cross-section of emerged and submerged rubble mound breakwaters.
2.1.2 Service Lifetime

The specification of the service lifetime of the breakwater affects the determination of the climate change scenario to be used as well as the design and the total cost of upgrading. Usually the service lifetime for the design of coastal structures is in the order of 30-70 years (CEM, 2002), for extreme wave events with 50-year or 100-year return period.

2.1.3 Performance criteria

The performance criteria for the emerged rubble mound breakwaters refer to the selection of design level for several hydraulic and structural responses such as the armour stability, the breakwater toe stability and the admissible wave overtopping discharge. For the submerged breakwaters the breakwater stability criteria and the level of optimum wave transmission coefficient should be determined. More specifically the performance criteria are determined as follows:

- **Armour stability.** For rocks the criterion corresponds to the initiation of armour damage (S = 2–3 in the Van der Meer, 1988 formula, and $K_D = 2$ in the Hudson formula, Shore Protection Manual,1984) for 50-years return period sea state, and repairable damage for the 100–200 years return period sea state (S = 4–5 in the Van der Meer formula, $K_D = 3$ in the Hudson formula). For concrete units (cubes, dolos, accropodes etc.) values of $K_D$ can be found in the Coastal Engineering Manual (U.S.A.C.E., 2002), while according to the Van der Meer 1988, consideration, the no damage criterion is usually used.

- **Toe stability.** It is often used the no damage criterion, toe damage parameter, $N_{od}<0.5$, for the 50-year period (U.S.A.C.E., 2002) or a small acceptable damage, $N_{od}<1.5$ (Burcharth et al. 2014), corresponding for the 200-year return period sea state, for the most critical water levels creating wave breaking directly onto the toe.

- **Wave overtopping discharge.** The admissible wave overtopping discharges are $q=10$ l/sm for the 50-years sea state, and $q=25$ l/sm for the 100 years return period sea state (U.S.A.C.E., 2002; Burcharth et al. 2014).

- **Wave transmission coefficient.** The admissible wave transmission coefficient $K_t$, depends on the purpose of the submerged breakwater; e.g. for small port and marina protection, the design aims in small levels of wave height $H<0.5m$ while for coastal protection, acceptable levels of $H$ regarding coastal erosion is the key design.

The calculation of the hydraulic responses (wave overtopping, wave transmission) and the structural loads and responses (armour and toe stability) is performed using desk study tools which are available. In the present upgrading, design formulae found in the Coastal Engineering Manual (U.S.A.C.E., 2002) and the EurOtop Manual (2007) are used.

2.2 Definition of the climate change scenario

The climate change scenario should be defined with respect to the construction of the related long-term statistics of wave heights, wave periods and water levels (deep waters). Then waves are transformed in front of the structure (shallow water) under the future wave climate scenario. The results from the Research Program "CCSE WAVS: Estimating the effects of climate change on sea level and wave climate of the Greek seas, coastal vulnerability and safety of coastal and marine structures" regarding the effect of climate change to the sea level and wave climate of the Greek seas are used. Particularly the future climate change scenario takes into account the following:

- Increase of Mean Water Level (MWL) due to climate change. The results of Regional Climate Model (RegCM3) simulations for the periods 1950-2000 and 2001-2100, as reported in CCSE WAVS Technical Report WP 1.5 (Tragou et al., 2013) are used. Additionally the contribution of factors related to the addition of water mass from the melting of
continental ice sheets, should be added according to forecasts of the IPCC AR5 (Church et al., 2013). The increase of MWL in Greek Seas due to such effects is found to be 0.20-0.25m in Year 2050 and 0.40-0.50m in Year 2100.

- Increase of MWL due to the effect of storm surge and change of the wave climate. The results from the application of wave and storm models and numerical experiments for the periods 1950-2000 and 2001-2010 developed in the Greek Seas are used, as reported in the CCSEWAVS Technical Report WP 2.1 (Krestenitis et al., 2013). It is shown that the maximum annual levels of storm surge are about 0.20-0.35m for the Years 2050 and 2100. The results of the extreme values analysis on the wave height found in CCSEWAVS Technical Report WP 3.2 (Galiatsatou et al. 2014) are also used.

2.3 Upgrading

2.3.1 Submerged rubble mound breakwater

The basic restriction on upgrading submerged rubble mound breakwaters is that the structure should remain submerged. Therefore two basic concepts of upgrading are:

- Increase of structure's width as shown in Figure 2a.
- Adding an extra armour layer and reduce the structure’s freeboard, if the upgraded structure remains fully submerged, as depicted in Figure 2b.

![Concepts for upgrade of a submerged rubble mound breakwater](image)

Figure 2. Concepts for upgrade of a submerged rubble mound breakwater (a) increase of structure’s width (b) add an extra armour layer and reduce the structure’s freeboard.

2.3.2 Rubble mound breakwater

When crest level increase is acceptable, possible upgrade would consist in placing an extra layer of armour units on the front slope and on the crest in order to strengthen the main armour and reduce the wave overtopping (Figure 3a). When increase of the breakwater freeboard is not acceptable, then possible concepts of upgrading are:

- Adding armour units and form an extra layer (Figure 3b).
- Adding armour units and form an extra layer with flatter slope (Figure 3c).
- Adding armour units and form a flat berm (Figure 3d).
- Construct a separate submerged breakwater (Figure 3e).
3. IMPLEMENTATION OF PROPOSED METHODOLOGY-CASE STUDY

The proposed methodology is implemented in two case studies for coastal structures located in Northern Greece, and more specifically the windward breakwater in the Port of Alexandroupoli and the submerged breakwaters in Paralia Katerini, as shown in the map (Figure 4).

Figure 3. Concepts for upgrade of a rubble mound breakwater (a) add armour units and increase breakwater’s freeboard (b) add armour units and form an extra layer (c) add armour units and form an extra layer with flatter slope (d) add armour units and form a berm (e) construct a separate submerged breakwater.
3.1 Breakwater in the Port of Alexandroupoli, North Greece

The Port of Alexandroupoli has coordinates 40°50' N latitude and 25°52' E Longitude and is exposed to SouthWest (SW), South (S) and SouthEast (SE) waves (Figure 5). The Port is mainly protected by the south windward breakwater, with total length of 1715m, which is a armoured with concrete units (accropodes) with volume 5.0m$^3$, which has been constructed in 1995-2001.

3.1.1 Structure response under future wave climate and sea level conditions

The data for the wave conditions and the tide levels are found in the Technical Report of the breakwater design (ADK Engineers, 1999). The breakwater is designed for S-SW waves with $H_s=5.10$ m, $T_p=9.0$s (deep waters), while the tide levels with respect to the MWL are $+0.72$m (high tide) and $-0.63$m (low tide). The following future climate change scenario is taken into account, based on the above consideration (section 2.2):

- Increase of 0.20m of the MWL due to climate change, as result of the Regional Climate Model (RegCM3) simulations and the IPCC forecasts for the Year 2050 (Tragou et al., 2013).
- The maximum annual levels of storm surge in the area of Alexandroupoli are about 0.30m for the Year 2050 (Krestenitis et al., 2013).
Increase on the offshore wave height which is found $H_s=5.50\text{m}$ for the Year 2050 (Galiatsatou et al. 2014).

The cross-section of the windward breakwater and the MWL including Climate Change, storm surge and tide effects is shown in Figure 6.

![Figure 6. Cross-section of the windward breakwater and the MWL including Climate Change, storm surge and tide effects.](image)

For the estimation of the wave height in front of the structure, simple calculation tools are used taking into account the transformation of the wave from deep waters to the near shore zone, due to wave refraction, shoaling and breaking. Regarding the breakwater performance, it should be noted that for the armour stability estimation the case of low tide was considered, while for the estimation of wave overtopping the case of high tide was taken into account. The hydraulic conditions and related performance for existing breakwater under the design period conditions (Year 2000) and the Climate change scenario (Year 2050) are shown in Table 1.

From the evaluation of the performance criteria for the existing structure under the climate change scenario, it is seen that the armour and toe stability criteria are satisfied, however there is shown a significant increase of the average wave overtopping discharge, from $q=7.87\text{l/sm}$ to $q=31.90\text{l/sm}$, exceeding the limit of $q=10\text{l/sm}$.

Table 1. Hydraulic conditions and related performance for existing breakwater under (a) Design period conditions, Year 2000 and (b) Climate Change scenario, Year 2050.

<table>
<thead>
<tr>
<th></th>
<th>2000</th>
<th>2050</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_s$ (m)</td>
<td>5.10</td>
<td>5.50</td>
</tr>
<tr>
<td>$T_p$ (s)</td>
<td>9.00</td>
<td>9.00</td>
</tr>
<tr>
<td>$d_h$, water depth in front of structure (m)-low tide</td>
<td>8.20</td>
<td>8.40</td>
</tr>
<tr>
<td>$d_h$, water depth in front of structure (m)-high tide</td>
<td>9.55</td>
<td>10.05</td>
</tr>
<tr>
<td>$R_c$, freeboard (m) high tide</td>
<td>3.75</td>
<td>3.25</td>
</tr>
<tr>
<td>$H_{1/3}$ in front of structure (m)</td>
<td>4.00</td>
<td>4.40</td>
</tr>
<tr>
<td>$K_d$, (Hudson formula)</td>
<td>9.34</td>
<td>12.44</td>
</tr>
<tr>
<td>No damage (Van der Meer, 1993)</td>
<td>ok</td>
<td>ok</td>
</tr>
<tr>
<td>$N_{td}$, Toe damage (Van der Meer, 1995)</td>
<td>0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>$q$, Average overtopping discharge (l/sm)</td>
<td>7.87</td>
<td>31.90</td>
</tr>
</tbody>
</table>

3.1.2 Design of upgrade alternatives

Three different alternatives for upgrading the breakwater are designed aiming the reduction of wave overtopping, in order to meet the threshold of $q=10\text{l/sm}$. The construction cost of each upgrade alternative, is calculated from the Official Greek Government Rates as stated in Greek Government Gazette (FEK 363B/19-2-2013). The upgrade alternatives are:

- **Add a berm.** Add rock armour units with weight similar to the breakwater toe, in order to form an horizontal berm, with main purpose the reduction of wave overtopping, as calculated from empirical formulae found in the Coastal Engineering Manual (USACE, 2002). The required weight of rock have been calculated using empirical formulae (Pilarczyk and Zeidler, 1996) in order to satisfy the submerged breakwater and toe stability criteria with damage parameter $S<2$, $N_{td}<0.5$.

- **Add an extra armour layer on the front slope and on the crest.** In this case an extra layer of accropodes, each volume $5.0\text{m}^3$, is added in order to increase the crest of the breakwater.

- **Add a submerged breakwater.** A separate submerged rubble mound breakwater is constructed in order to attenuate the incident waves and consequently reduce the wave overtopping. The proposed submerged breakwater achieves transmission coefficient $K_t=0.85$ for the design wave conditions. The required weight of rock have been calculated using empirical formulae (Pilarczyk and Zeidler, 1996) with $S<2$.

The results for the different upgrade alternatives are shown in Table 2 and Figure 7. As can be seen all solutions are designed in order to meet the threshold for the average overtopping discharge $q=10\text{l/sm}$. From the economic point of view the construction of a berm is the most favorable, while the extra armour layer is the most expensive one. However if the...
stability criteria for the existing breakwater were not satisfied by the climate change scenario, probably the second alternative which improves significantly the armour stability would be required.

Table 2. Average overtopping discharge and construction cost of upgrading the rubble mound breakwater for different alternatives

<table>
<thead>
<tr>
<th></th>
<th>A. Adding a berm</th>
<th>B. Adding an extra armour layer on the front slope and on the crest</th>
<th>C. Adding a submerged breakwater</th>
</tr>
</thead>
<tbody>
<tr>
<td>q, Average overtopping discharge (l/sm)</td>
<td>9.88</td>
<td>6.28</td>
<td>10.07</td>
</tr>
<tr>
<td>W (t), Armour weight</td>
<td>5.00</td>
<td>11.75</td>
<td>1.23</td>
</tr>
<tr>
<td>Construction cost per meter (€/m)</td>
<td>510.00</td>
<td>4500.00</td>
<td>1064.00</td>
</tr>
</tbody>
</table>

Figure 7. Cross-section of upgraded structure for alternative solutions (a) adding a berm, (b) adding a submerged breakwater, (c) adding an extra armour layer on the front slope and on the crest.

3.2 Submerged breakwaters in Paralia Katerini, North Greece

The submerged breakwaters are located on the coast of Paralia Katerini, South of the Gulf of Thessaloniki, and has coordinates 40°15’ N latitude and 22°37’ East Longitude, while the coast is oriented SE - NW (Figure 8). The submerged breakwater has been constructed as a part of a general project to in order to treat the erosion problem of the coast. The cross-section of the submerged rubble mound breakwater is shown in Figure 9, where the weight of the armour rock is W=1.4t.
3.2.1 Structure response under future wave climate and sea level conditions

In the framework of the coastal engineering study (Koutitas et al., 2004) the submerged breakwater has been designed in order to achieve an average transmission coefficient $K_t=0.42$, for the representative wave. The Borah and Balloffet (1985) Eq. [1] is used in order to derive the median wave height $H$, based on the wave height $H_i$ and wind frequency $f_i$ for each case including all existent wind strengths. The results showed that the representative wave is SE and S with $H_{so}=1.34$ m, $T_p=4.5$ s (deep waters) for the construction time conditions (Year 2010).

Regarding the stability of the submerged structure, it is designed for the extreme wave case in the area of Katerini, which is SE waves with $H_{so}=3.78$ m, $T_p=6.5$ s (Year 2010). The tide level in the areas is 0.22 m.

The following future climate change scenario is taken into account, based on the above consideration (section 2.2):
- Increase of 0.20 m of the MWL due to climate change, as result of the Regional Climate Model (RegCM3) simulations and the IPCC forecasts for the Year 2050 (Tragou et al., 2013).
- Increase 17% in the maximum annual levels of storm surge in the area of Katerini for the Year 2050 (Galiatsatou et al. 2014).
- Increase 3.5% on the offshore wave height for the Year 2050 (Galiatsatou et al. 2014).

For the estimation of the wave height in front of the structure, simple calculation tools are used taking into account the transformation of the wave from deep waters to the near shore zone, due to wave refraction, shoaling and breaking.

The freeboard of the structure is $R_c=0.30$ m, while $R_c'$, accounting for the effects of the tide, storm surge and increase of MWL due to climate change, is calculated as:

$$ R'_c = R_c + \zeta_w + \zeta_t + \zeta_{CC} $$

where, $\zeta_w$ is the elevation of MWL due to storm surge, $\zeta_t$ due to high tide and $\zeta_{CC}$ due to climate change.

The submerged breakwater’s performance regarding the stability criterion ($S<2$) and the design wave transmission criterion ($K_t=0.42$), is evaluated for the future climate change scenario. The wave transmission is calculated using the formulae found in Van der Meer et al. (2005) and d’Angermond et al. (1996). It is seen that the climate change effect
results in increasing the MWL and therefore has positive effect on the stability of the armour units of the breakwater; the damage parameter $S$ decreases. Regarding the wave transmission the various wave conditions tested, including the representative wave, show an increase of $K_t$ about 12% ($K_t = 0.47$). The hydraulic conditions and related performance for existing breakwater under the construction period conditions (Year 2010) and the Climate change scenario (Year 2050) are shown in Table 3.

Table 3. Hydraulic conditions and related performance for existing submerged breakwater under (a) construction period conditions, Year 2010 and (b) Climate change scenario, Year 2050.

<table>
<thead>
<tr>
<th></th>
<th>2010</th>
<th>2050</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_{sw}$ (m)</td>
<td>3.78</td>
<td>3.91</td>
</tr>
<tr>
<td>$T_p$ (s)</td>
<td>6.50</td>
<td>6.50</td>
</tr>
<tr>
<td>$H_{1/3}$ (m) in front of structure</td>
<td>2.57</td>
<td>2.71</td>
</tr>
<tr>
<td>$d$, water depth in front of structure (m)</td>
<td>3.50</td>
<td>3.70</td>
</tr>
<tr>
<td>$R_c$, freeboard (m) at high tide</td>
<td>-0.60</td>
<td>-0.82</td>
</tr>
<tr>
<td>$S$, armour damage (Van der Meer, 1993)</td>
<td>0.70</td>
<td>0.20</td>
</tr>
<tr>
<td>$K_t$, average transmission coefficient</td>
<td>0.42</td>
<td>0.47</td>
</tr>
</tbody>
</table>

3.2.2 Design of upgrade submerged breakwater

In order to fulfill the design requirements the submerged breakwater should be upgraded. The basic restriction is that the upgraded structure should remain fully submerged, which denotes that decreasing the freeboard is not accepted in the specific case, where the freeboard is marginal $R_c = 0.30$m. Therefore the upgrading refers to increase of the structure’s width $W = 5.5$m in order to obtain $K_t = 0.42$. Based on these restrictions the required structure's width is $W = 9.5$m as seen in Figure 10 where the cross-section of the upgraded submerged rubble mound breakwater, is shown. Table 4 summarizes the wave conditions and the transmission coefficient for the existing breakwater for design period conditions, Year 2000, the existing breakwater under climate change scenario, Year 2050 and the upgraded structure under climate change scenario, Year 2050. The required weight or rock and the construction cost for the upgraded structure is shown in Table 5.

Figure 10. Cross-section of upgraded submerged rubble mound breakwater.

Table 4. Wave conditions, freeboard and transmission coefficient for (a) existing breakwater for design period conditions, Year 2000 (b) existing breakwater under climate change scenario, Year 2050 and (c) upgraded structure under climate change scenario, Year 2050

<table>
<thead>
<tr>
<th>no</th>
<th>$R_c$ (m)</th>
<th>$H_{sw}$ (m)</th>
<th>$T_p$ (s)</th>
<th>$K_t$</th>
<th>$R_c$ (m)</th>
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<tr>
<td>1</td>
<td>-0.80</td>
<td>1.34</td>
<td>4.50</td>
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<td>1.39</td>
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<tr>
<td>2</td>
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<td>1.21</td>
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<td>0.539</td>
<td>0.493</td>
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<tr>
<td>3</td>
<td>-0.55</td>
<td>2.58</td>
<td>7.40</td>
<td>0.431</td>
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<td>2.67</td>
<td>0.464</td>
<td>0.409</td>
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<td>4</td>
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<td>8.50</td>
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<td>3.91</td>
<td>0.404</td>
<td>0.353</td>
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<tr>
<td>5</td>
<td>-0.15</td>
<td>1.34</td>
<td>4.50</td>
<td>0.302</td>
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<td>1.39</td>
<td>0.363</td>
<td>0.323</td>
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<tr>
<td>6</td>
<td>-0.60</td>
<td>1.34</td>
<td>4.50</td>
<td>0.436</td>
<td>-0.82</td>
<td>1.39</td>
<td>0.496</td>
<td>0.456</td>
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Table 5. Weight or armour unit and construction cost for upgraded submerged rubble mound breakwater

<table>
<thead>
<tr>
<th>Increase of breakwater width</th>
<th>$W$ (t), Armour weight</th>
<th>Construction cost per meter (€/m)</th>
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<tr>
<td>1.50</td>
<td>150.00</td>
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4. CONCLUSIONS

Various concepts of upgrading of existing emerged and submerged rubble mound breakwaters have been studied and related costs estimated. The results from the Research Program CCSEAWAVS regarding the effect of climate change to the sea level and wave climate of the Greek seas have been used, considering the structure service lifetime of 50 years. The calculation of the hydraulic and structural responses has been performed using readily available formulae from literature. The results showed that the influence of climate change is stronger in terms of sea level rise as well as the storm surge increase rather than the increase of the wave heights. Therefore the structures that are studied fail to fulfill the criteria related to wave overtopping and transmission but satisfy the stability criteria, under the climate change scenario. The final selection of the upgrading alternatives is taken from the lower construction cost.

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