INVESTIGATION ON THE EFFECTS OF CASPIAN SEA UPON ITS COAST 
AND PRESENTING SUITABLE PROTECTIVE SOLUTIONS

BY

FARHAD MIRFAKHRAEE
PARTICIPANT IN HYDRAULIC ENGINEERING BRANCH B  
LILE

ATTENTION

PROFESSOR MR. VERHAGEN

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Investigation on the effects of Caspian Sea upon its coast and presenting suitable protective solutions

By
Farhad Mirfakhraee
Participant in Hydraulic Engineering branch b, 95-96
I.H.E

Attention
Professor Mr. Verhagen

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ABSTRACT

In this research, the effects of increase of water level of Caspian Sea upon coastal line of Iran has been investigated. While investigating, waves and water level changes of Caspian Sea, the different methods of coastal protection have been surveyed. In this respect, much attention has been paid to the rocky breakwaters which are widely used to protect coast.

Finally, having examined the positive and negative points of the applied designs to the Iranian coasts, attempts have been made to present an appropriate method considering all of the necessary standards in designing of breakwaters. As far as possible, in all stages of the present research, the real value of existence data have been used.

Finally a suitable design is presented for coastal protection considering all design aspects. In the presented method, in addition to the height of wave and the roughness of the corners of armor layer, the type of wave such as surging or plunging, wave period, height of wave in the breaking point, slope of the coast and permeability of core and angle of friction between the blocks have been considered. After determining the weight of rock pieces and considering the problem of rock erosion, the final size of breakwater regarding to the freeboard and wave run-up on the slope is determined.
Chapter 1

Studying of Destructive Effects of Caspian Sea
1.1. Introduction

The protection of ports and costs against the destructive and indefatigable force of agitated sea waves, is the problem which return to the long time ago. Until developing of facilities for natural ports and its adjacent coasts, attention to the problems related to the mentioned areas only was interested to the local people and administration, but after developing of road systems and specially pretty coastal roads, controlling and protection of coastal areas against the roaring and agitated waves becomes very important to pay attention to it. Increasingly using of pretty roads along the coasts, protection of coastal constructions and lands adjacent the sea should be appreciated and for protecting of individual and governmental lands, it has to be prevented against the erosion and the eroded coasts must be amended economically.

Whenever has been erosion upon the coasts, the individual and governmental owners of coastal constructions are interested to any solution for protecting and amending of coasts. Unfortunately, the solutions have been carried out without any specialized research. Although some of these solutions have been successful in some coasts, we can not advise the same solution to any place with particular problem. Sometimes applying the similar solutions for problems upon the coast could be the alternative, but it can not work always.

We have to be careful, however, applying a unsuitable solution for a particular problem can create lot of extra costs. In practice, the erosion of coast are coming from two types of natural water action.

Firstly, the wave are attacking to the coast and secondly coastal currents which are near to the coasts or lakes.

Studying about how the coasts have been eroded, makes it clear that there are lot of complications and variations between wave’s forces. For studying of these types of forces, the designers has to use of sciences such as Oceanography, Aerology, Soil mechanics, Solid mechanics, Structural Design, Geology and so on.

Increasing of height of water level in the Caspian Sea in Northern coasts of Iran during past several years, has been made a lot of difficulties. For protecting respects, administration and the individual owners of constructions and lands along the coastal lines of Caspian Sea, they have performed lot of solutions in a hurry without having any basically knowledge and sufficient scientific reasons.

In this report, the effects of protection plans has been noticed. Also the effects of the rocky protection wall which has been used extensively will be studied.

1.2. Water Level Changes of Caspian Sea

Water level of Caspian Sea has been fluctuating from long time ago considering much available documents. Rising of the water level of Caspian Sea is not new matter and will be not at future.

Fluctuating of Caspian Sea’s water level has lot of effects upon environmental and ecology of coastal areas. That is why during summers and days the weather is cooler than the lands which is located far away from coastal zones and in the other hand, in winters and nights, the weather in those coastal areas is warmer than the non coastal areas. Also Caspian Sea and its coastal lands is one of the most beautiful fertile areas in our country, Iran, noticing to this point that Iran has dry climate, the changes of water level in Caspian Sea has to be studied so seriously.
In this view, both positive and negative points of this subject must be researched carefully and for this purpose we need cooperation of Universities and research centers.

The changes of Caspian Sea’s water level have been shown in Fig. 1.1 which has been prepared by the Port and Shipping Organization of Anzali.

The zero point at the graph shows that the level of seawater is located at -26.36947 in comparison with open sea water levels.

The changes of Caspian Sea water level is compared with Zero point in Main Port and Shipping Organization of Anzali which equals -26.36947 (m) compared with Open Sea water level. The positive and negative numbers at the vertical axis shows the increase or decrease of this difference respectively.

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**Fig. 1.1. Changes of water level in Caspian Sea (1929 - 1995)**

**Fig. 1.2. Changes of water level in Caspian Sea (1992 - 1996)**
Farhad Mirfakhraee, Participant in HE, b., 95-96
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Fig. 1.3. Caspian Sea
1.3. Characteristics of Caspian Sea

Caspian Sea has been recognized as a very rich resource of this area. Increasing of Caspian Sea's water level in the recent years has destroyed both residential and low coastal lands and this phenomenon has made a lot of difficulties for the people who are living alongside of Sea.

The rising of water level in Caspian Sea is the very big problem for the coastal strip of Caspian Sea and this problem is a threat to natural ecosystem of area. When the sea water level is rising, the large area of coastal lands is floating under sea water and salt in the sea water is harmful for surviving of any types of plants and trees, therefore, plants and extensive forests could be destroyed by this phenomenon. In the other hand, the suitable agriculture lands after lying under the sea water become muddy and we know the muddy lands are not suitable for agriculture purposes. The specifications of Caspian Sea are summarized as following:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gross area of river basin</td>
<td>3,700,000 (km$^2$)</td>
</tr>
<tr>
<td>Area of river basin in Iran</td>
<td>256,000 (km$^2$)</td>
</tr>
<tr>
<td>Water volume in Caspian Sea</td>
<td>778,600 (m$^3$)</td>
</tr>
<tr>
<td>Parameter</td>
<td>6525 (km)</td>
</tr>
<tr>
<td>Length of coast of Iran From Astara till Gomishan</td>
<td>995 (km)</td>
</tr>
<tr>
<td>Maximum area of Sea</td>
<td>412,000 (km$^2$)</td>
</tr>
<tr>
<td>Minimum area of Sea</td>
<td>374,000 (km$^2$)</td>
</tr>
<tr>
<td>Medium area of Sea</td>
<td>393,000 (km$^2$)</td>
</tr>
<tr>
<td>Sea Length</td>
<td>1200 (km)</td>
</tr>
<tr>
<td>Maximum Width of Sea</td>
<td>554 (km)</td>
</tr>
<tr>
<td>Minimum Width of Sea</td>
<td>202 (km)</td>
</tr>
<tr>
<td>Average Width</td>
<td>310 (km)</td>
</tr>
<tr>
<td>Depth of Sea in deepest point in South of Sea</td>
<td>960 (m)</td>
</tr>
<tr>
<td>Depth of Sea in deepest point in North of Sea</td>
<td>26 (m)</td>
</tr>
</tbody>
</table>

The most important rivers entering to Caspian Sea are such as, Volga 78.5 of total entrance flowing, Coral %6, Oral %5.2, it can be expressed roughly the %94.7 of water resource of Caspian Sea is supplied from Previous Russian Countries and just only %5.3 of this water is supplied from Iranian rivers.

1.4. The Effects of Sea Water Level Rising upon Coasts of Iran

The rising of sea water level have had a lot of effects upon coasts of Iran. These effects are as following:

1. The lot of people have just lost their occupations that they earn a livelihood from rent of their houses. The rising of water level damages their houses. This problem is observed in cities like as Anzali, Astara, Kalachai, Ramsar and Babolsar.

2. Erosion and sedimentation in the coast observing alongside of South coast of Caspian Sea.

3. Environmental impact of sea water pollution on the coastal lands. It is coming from the oil pollution from refinery of Baku in Russia.

4. Settlement of cities and constructions alongside of coastal zone that rising of water level has been occurred on it.

5. Drying of coastal forests close to the Caspian Sea.

6. The low agricultural lands becomes muddy and salty.
7. Destruction of roads in villages and cities. This problem is observed in Tonekabon, Ramsar, Nowshahr and Mahmoudabad.
8. In correctly working of drainage between agricultural farms.
9. The water of wells have become salty or polluted.
10. Endangering of fishing facilities in Bandare Torkaman.
11. Endangering of national facilities like as Bandare Anzali’s costume.
12. Damage on urban facilities like as water, electricity, ego, telephone and so on.
13. Destruction of other constructions for military instruction.
14. Destruction of residential houses and then providing of suitable houses for this reason.
15. Prevention of suitable entrance of river to the sea and then water level rising in this region.

From whole ranges of these effects, only damages on residential houses and water level rising will be discussed as shown in Table 1.1.

<table>
<thead>
<tr>
<th>Number of residential houses</th>
<th>Area in Hectares</th>
<th>Number of residential Houses</th>
<th>Area in Hectares</th>
<th>Name of Cities</th>
</tr>
</thead>
<tbody>
<tr>
<td>1310</td>
<td>241</td>
<td>882</td>
<td>14.3</td>
<td>Astara</td>
</tr>
<tr>
<td>2227</td>
<td>375</td>
<td>400</td>
<td>215</td>
<td>Bandare Anzali</td>
</tr>
<tr>
<td>88</td>
<td>50</td>
<td>37</td>
<td>13</td>
<td>Chaboksar</td>
</tr>
<tr>
<td>55</td>
<td>140</td>
<td>45</td>
<td>10</td>
<td>Roodsar</td>
</tr>
<tr>
<td>365</td>
<td>110</td>
<td>281</td>
<td>42</td>
<td>Kalachai</td>
</tr>
<tr>
<td>432</td>
<td>150</td>
<td>674</td>
<td>20</td>
<td>Kiashahr</td>
</tr>
<tr>
<td>458</td>
<td>95</td>
<td>---</td>
<td>---</td>
<td>Chamkhale</td>
</tr>
<tr>
<td>4925</td>
<td>1161</td>
<td>2319</td>
<td>314.3</td>
<td>Total</td>
</tr>
</tbody>
</table>

Table 1.1. Definitions about area and levels in Gilan.

The Table 1.1. belongs to the Province of Gilan.
For the Province of *Mazandaran* the explanations will be shown in Table 1.2.

<table>
<thead>
<tr>
<th>Total Area (He)</th>
<th>Population</th>
<th>Area (He)</th>
<th>Level Height</th>
<th>Name of City</th>
</tr>
</thead>
<tbody>
<tr>
<td>1675.32</td>
<td>5821</td>
<td>335.23</td>
<td>Lower Than -22</td>
<td>Ramsar</td>
</tr>
<tr>
<td></td>
<td>4953</td>
<td>285.33</td>
<td>Lower Than -23</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4961</td>
<td>370.15</td>
<td>Lower Than -24</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3674</td>
<td>311.57</td>
<td>Lower Than -25.5</td>
<td></td>
</tr>
<tr>
<td>413</td>
<td>5320</td>
<td>134.97</td>
<td>Lower Than -22</td>
<td>Katalom</td>
</tr>
<tr>
<td></td>
<td>4961</td>
<td>125.86</td>
<td>Lower Than -23</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4821</td>
<td>132.32</td>
<td>Lower Than -24</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2177</td>
<td>55.22</td>
<td>Lower Than -25.5</td>
<td></td>
</tr>
<tr>
<td>1325.64</td>
<td>4757</td>
<td>170.64</td>
<td>Lower Than -22</td>
<td>Tonekabon</td>
</tr>
<tr>
<td></td>
<td>3773</td>
<td>135.32</td>
<td>Lower Than -23</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3267</td>
<td>117.19</td>
<td>Lower Than -24</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2871</td>
<td>102.98</td>
<td>Lower Than -25.5</td>
<td></td>
</tr>
<tr>
<td>394</td>
<td>2585</td>
<td>133.88</td>
<td>Lower Than -22</td>
<td>Nashtavard</td>
</tr>
<tr>
<td></td>
<td>2774</td>
<td>128.25</td>
<td>Lower Than -23</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2689</td>
<td>122.52</td>
<td>Lower Than -24</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1266</td>
<td>59.97</td>
<td>Lower Than -25.5</td>
<td></td>
</tr>
<tr>
<td>490</td>
<td>3595</td>
<td>69.96</td>
<td>Same Above</td>
<td>Salmanshahr</td>
</tr>
<tr>
<td></td>
<td>3595</td>
<td>69.96</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td></td>
<td>953</td>
<td>22.22</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td>419</td>
<td>2802</td>
<td>54.89</td>
<td>Same Above</td>
<td>Kalarabad</td>
</tr>
<tr>
<td></td>
<td>2517</td>
<td>44.21</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2409</td>
<td>54.89</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td></td>
<td>906</td>
<td>15.72</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td>957</td>
<td>9865</td>
<td>299.4</td>
<td>Same Above</td>
<td>Nowshahr</td>
</tr>
<tr>
<td></td>
<td>8668</td>
<td>366.07</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8537</td>
<td>258.78</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6267</td>
<td>190.19</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td>329</td>
<td>8333</td>
<td>99.66</td>
<td>Same Above</td>
<td>Noor</td>
</tr>
<tr>
<td></td>
<td>8127</td>
<td>95.77</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8032</td>
<td>93.78</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2673</td>
<td>34.95</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td>205</td>
<td>3367</td>
<td>126</td>
<td>Same Above</td>
<td>Alamdeh</td>
</tr>
<tr>
<td></td>
<td>2725</td>
<td>69.69</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2363</td>
<td>38</td>
<td>Same Above</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1635</td>
<td>36</td>
<td>Same Above</td>
<td></td>
</tr>
</tbody>
</table>

Table 1.2. Definitions about various area and levels

The summary of mentioned Table will be indicated in Table 1.3.

<table>
<thead>
<tr>
<th>Total Area (He)</th>
<th>Lower Than -22</th>
<th>Lower Than -23</th>
<th>Lower Than -24</th>
<th>Lower Than -25.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>3647.81</td>
<td>3215.29</td>
<td>2918.38</td>
<td>1602.76</td>
<td></td>
</tr>
<tr>
<td>116290</td>
<td>108489</td>
<td>104357</td>
<td>52987</td>
<td></td>
</tr>
</tbody>
</table>

Table 1.3. Summary of Totals.

From mentioned Total area in Table 1.3., the sum of areas lower than -24 has been effected by the waves. Total area and population is 4521.14 and 157344 related to the
Provinces of Gilan and Mazandaran respectively. Center of unexpected disasters had to do accommodation of people who are located in these levels and they were taken out from these regions.

Because of the damage have been come to residential houses the following activities shown in Table 1.4. have been performed. It is clear from this table the number of 4252 residential houses has been damaged. The most number of damaged houses are related to Bandare Torkaman with 1392 residential houses, therefore, considering these problems, have been carried out several alternatives for solving of mentioned difficulties. These are as following:

<table>
<thead>
<tr>
<th>Name of City</th>
<th>Required Land (He)</th>
<th>Total Debit of Previous Loan</th>
<th>Bought Lands (He)</th>
<th>Non Performed</th>
<th>Performed</th>
<th>Total Damaged Units</th>
<th>Required Credit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ramsar</td>
<td>16</td>
<td>155</td>
<td>5.5</td>
<td>195</td>
<td>151</td>
<td>347</td>
<td>300</td>
</tr>
<tr>
<td>Tonekabon</td>
<td>3</td>
<td>290</td>
<td>2.5</td>
<td>390</td>
<td>86</td>
<td>476</td>
<td>250</td>
</tr>
<tr>
<td>Nashtarood</td>
<td>1</td>
<td>54</td>
<td>1.9</td>
<td>145</td>
<td>74</td>
<td>219</td>
<td>150</td>
</tr>
<tr>
<td>Abasabad</td>
<td>-</td>
<td>31</td>
<td>1.6</td>
<td>165</td>
<td>15</td>
<td>180</td>
<td>-</td>
</tr>
<tr>
<td>Salmanshahr</td>
<td>1</td>
<td>19</td>
<td>-</td>
<td>48</td>
<td>-</td>
<td>48</td>
<td>150</td>
</tr>
<tr>
<td>Kalarabad</td>
<td>-</td>
<td>16</td>
<td>3.15</td>
<td>65</td>
<td>-</td>
<td>65</td>
<td>-</td>
</tr>
<tr>
<td>Noshahr, Chalos</td>
<td>33</td>
<td>315</td>
<td>1.5</td>
<td>359</td>
<td>-</td>
<td>359</td>
<td>500</td>
</tr>
<tr>
<td>Noor, Alamdeh</td>
<td>-</td>
<td>169</td>
<td>2</td>
<td>210</td>
<td>78</td>
<td>288</td>
<td>-</td>
</tr>
<tr>
<td>Mahmoodabad</td>
<td>1</td>
<td>4</td>
<td>-</td>
<td>64</td>
<td>-</td>
<td>64</td>
<td>200</td>
</tr>
<tr>
<td>Fereidonkenar</td>
<td>5</td>
<td>241</td>
<td>2.7</td>
<td>349</td>
<td>78</td>
<td>426</td>
<td>750</td>
</tr>
<tr>
<td>Babolsar</td>
<td>-</td>
<td>143</td>
<td>2</td>
<td>94</td>
<td>45</td>
<td>139</td>
<td>-</td>
</tr>
<tr>
<td>Larim, Joabar</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>110</td>
<td>-</td>
<td>110</td>
<td>-</td>
</tr>
<tr>
<td>Kharidabad</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>91</td>
<td>-</td>
<td>91</td>
<td>-</td>
</tr>
<tr>
<td>Bandare Gaz</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>49</td>
<td>-</td>
<td>49</td>
<td>-</td>
</tr>
<tr>
<td>Torkaman</td>
<td>1</td>
<td>309</td>
<td>6.3</td>
<td>200</td>
<td>1192</td>
<td>1392</td>
<td>150</td>
</tr>
<tr>
<td>Total</td>
<td>61</td>
<td>1746</td>
<td>85.86</td>
<td>2533</td>
<td>1719</td>
<td>4252</td>
<td>2450</td>
</tr>
</tbody>
</table>

Table 1.4.

1. Removing of disasters people from floating regions and accommodation of them in the secure places.
2. Constructing of ruble mound breakwaters.

If the costs are considered, it has to be mentioned over 23283 million Rials equal 5.82 million $ has been spent. This amount of money is so much and it is amazing. Therefore, that is why this report is too important and after gaining a practical and economical solution, it could certainly save lot of national currency.

1.5. Geotechnical Aspects of South bottom of Caspian Sea
Bottom’s geotechnics of Caspian Sea between Bandare Anzali and North of Zaghe Marz has been recognized by the bore holes applied by Exploration Center of National Petroleum Company of Iran. Process of investigation has been performed with suitable distance with applying of two bore holes till depth of 32 (m). Types of soil layers till 30 (m) are:

- Gray clay till gray blue sticky clay and till 7 (m) from surface is very soft.
- From 9.20 (m) till 12.5 (m) gray brown stiff clay and clay plus bits of bivalves.
- Till 30 (m) depth, clay plus stony gray clay and sand stone with very tiny articles with minerals of mica plus bits of bivalves and little bit coal materials.
Bore hole no.2 has been dug with distance equal 1.5 (km) from bore hole no.1 and its depth is till 37.5 (m).

Types of soil layers till 30 (m) depth are:
- Till 7 (m) depth gray brown clay a little bit stiff and compacted from 9.5 (M) depth till end of bore hole, clay and gray stony clay till gray blue clay little bit compacted with sandstone with tiny articles plus coal material.
- Bore hole no.3 has been carried out at the depth of 18 (m) with a distance equal 3 (km) from bore hole no.1 till depth of 30 (m) frequently.

In this investigation types of soil layers are as following:
- Till 4.5 (m) gray clay till gray blue sticky clay, so soft till 9 (m) depth gray clay plus gray blue soft clay plus articles of coal.
- Till 12 (m) depth, clay or stony gray clay till gray blue clay, from 12 (m) till 27.5 (m) depth, clay and gray stony clay till gray blue a little bit compacted with strips of sandstone with very tiny materials.
- From 27.5 (m) till 30 (m), recent type of soil is determined. Only at the depth of 27.6 (m) there is one layer with thickness of 5 (cm) with sandstone very tiny materials of gray to clear gray brown.

Introductory studying have been shown a layer of gray brown clay in bore hole no.1 and no.2 at the depth of 9.20 (m) till 12.30 (m).

Direction of this layer is the same direction for the sea bed. Also there is possibility for damaging of mentioned layer.

1.6. Shallow Waves
The shallow waves are established on the free surface of the fluid, because there is equilibrium between stability and fluctuating of fluid. When an object is dropped inside water or on its still surface, a disturbance will be produced on its surface which is called wave. The waves are called disturbance of water surface. There are several reasons for producing of these waves. The main reasons are phenomenon like as wind, earthquake, explosions, the rapidly movement of a large group of fish close to the water surface, tides and settlement of sea’s bed. The most important reasons are the wind and tides.

Generally the winds are produced by blowing of wind, then this type of producing of waves will be considered. In the coastal lines of Iran, the most important factor for producing of waves is the wind blowing, specially in the Persian Golf, Oman Sea and Caspian Sea. Therefore these types of waves will be discussed.

Here, the wave producing by the other factors are not discussed, but, in designing process it will be taken in to account if it is necessary.

1.7. Wave Producing by Wind
When two types of liquid are flowing adjacent each other with different velocities, the shear stress is produced regarding to the viscosity of two layers. Therefore, an unstable flow is established on the surface of water-air by blowing of wind. Consequently, with blowing of wind in different velocities, the waves with various velocities are produced. If the wind blowing continues, the produced waves are lying at the same direction of wind blowing. Finally, the produced disturbance will gradually become two dimensional. The waves locating in the wind blowing region are going to have a greater amplitude named Local Waves. Then. these waves are taking away from blowing zone after forming. In this case, these types of waves are called Swelling
Waves. The swelling waves are free waves and these waves are gradually disappeared by phenomenon like friction, distribution, turbulence and air resistance, however, before disappearing, they can cover the very long distance. In practice, the free waves are symmetry over to the sheet which is lying in the middle of distance between two crests. However, slopes of besides of crests are steeper than the slopes of down parts of waves and the average water surface level is upper than the still water level. Also the regularity of waves which are produced by wind are relative, because these waves are moving one after other continuously with different periods, heights and lengths. Wave recording at the constant point shows the different single waves arriving to that point. For more details the following pattern is considered.

The coordinate is the Cartesian coordinate which has been shown on the Still Water Level. Still Water Level is the surface without any turbulence. The Depth (d) is from the bed till S.W.L., Wave Height (H) is the vertical distance between lowest point of wave and its crest. Wave Length (\( \lambda \)) is the distance between two successive crests, (c) is the Celerity of the Wave propagation. The position of any point from the S.W.L. is recognized by function of \( h(x,t) \).

Generally, when the changes of water level are recorded naturally, the problem is not as simple as the scathe shown in Fig.1.4. As it is clear in the Fig.1.5, determining the period or height of repetition is difficult. So, in these cases, it can be used nominal wave height (HA) and nominal wave period (TA) shown in Fig.1.5.
Fig. 1.5. Period and nominal height of wave.

Studying of the sea water level can be followed by changing of irregular sea water level to Sine waves with short amplitudes. Also an irregular pattern of sea waves can easily be changed to several Sine wave by using rule of Combination. This subject has been shown in the Fig. 1.6.

Fig. 1.6. Waves with different height and amplitude.

Since the phases of sine curves are random different, so it is advised to use probabilistic theory. It is observed that the successive point in the equal time stages in the irregular measurements will make the Normal or Gaussian distribution curve. Therefore a wave pulse as shown in Fig. 1.7. shows the state of the water level. Also it shows for every period there would be particular height.
Fig. 1.7. Normal distribution

In known place and state the probability of occurrence of the favor wave height is expressed by the Rayleigh distribution and the probability that the height becomes between \((H)\) and \((H+\Delta)\) will be:

\[
\text{Probability} = \frac{\exp\left(-H^2 / \tilde{H}^2\right) \cdot 2H / \tilde{H}^2}{\text{Area}} = \frac{\exp(-H^2 / \tilde{H}^2) \cdot 2H / \tilde{H}^2}{\Sigma H^2 / N}
\]

(1.1)

The Rayleigh distribution is:

\[
\frac{\exp(-H^2 / \tilde{H}^2) \cdot 2H / \tilde{H}^2}{\Sigma H^2 / N}
\]

(1.2)

The Rayleigh distribution is:
Fathad Mirfakhraee, Participant in HE,b, 95-96

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Fig. 1.8. Probability of occurrence

In practical, 1/3 of highest heights are selected which called Significant Wave Height (Hs). This type of the wave height is so real and the real waves can be evaluated by this type of waves. The other statistically height of waves are related to the Significant Wave Height and these proportions are shown in Table 1.5.

<table>
<thead>
<tr>
<th>Significant Wave Height</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean Height</td>
<td>0.64</td>
</tr>
<tr>
<td>Height of 10% of Highest</td>
<td>1.29</td>
</tr>
<tr>
<td>Height of 1% of Highest</td>
<td>1.68</td>
</tr>
<tr>
<td>Maximum Height</td>
<td>1.87</td>
</tr>
</tbody>
</table>

Table 1.5. Proportions among wave heights

\[ H_{max} = 1.87H_{1/3} \]

This equation shows the relation between \( H_{max} \) and Significant Wave Height. The wave heights are varying considerably at the short period of time in less than minutes or hours. Therefore Rayleigh distribution is acceptable at short periods. It does not work in the long periods.

In practice, recording of wave heights during the long periods are not possible, therefore in these cases are used of Roy assumption. The relation between the wave height and probability of occurrence is as following:

\[ H(R) = \left( \frac{H(I)}{5.90} \right) \ln(365R) \]

which \( R \) in (year), \( H \) is the Significant Wave Height. \( H(R) \) gives us the wave height after \( R \) years. In the design of breakwaters \( R \) can take values between 50 till 100 years.

The following table has been filled in the restrict time period by the International Oil Company of Iran in Anzali.

<table>
<thead>
<tr>
<th>Veloc. of Water (m/Sec.)</th>
<th>Mean Period of Waves (Sec.)</th>
<th>Height of Highest Wave (m)</th>
<th>Mean Height of Waves (m)</th>
<th>Mean Velocity of Storm (knot)</th>
<th>Mean Velocity of Wind (knot)</th>
<th>Month</th>
<th>Annum</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.44</td>
<td>9</td>
<td>3.0</td>
<td>1.8</td>
<td>42</td>
<td>22.0</td>
<td>Oct.</td>
<td>88</td>
</tr>
<tr>
<td>0.50</td>
<td>11</td>
<td>2.2</td>
<td>1.4</td>
<td>25.4</td>
<td>23.4</td>
<td>Nov.</td>
<td>88</td>
</tr>
<tr>
<td>0.57</td>
<td>12</td>
<td>6.0</td>
<td>3.0</td>
<td>52</td>
<td>26.7</td>
<td>Dec.</td>
<td>88</td>
</tr>
<tr>
<td>0.38</td>
<td>12</td>
<td>2.80</td>
<td>1.6</td>
<td>24.7</td>
<td>19.5</td>
<td>Jan.</td>
<td>88</td>
</tr>
<tr>
<td>0.38</td>
<td>11</td>
<td>3.7</td>
<td>2.3</td>
<td>30</td>
<td>25.4</td>
<td>Feb.</td>
<td>88</td>
</tr>
<tr>
<td>0.35</td>
<td>11</td>
<td>3.0</td>
<td>1.8</td>
<td>22</td>
<td>18.0</td>
<td>March</td>
<td>89</td>
</tr>
<tr>
<td>0.45</td>
<td>8</td>
<td>2.5</td>
<td>1.5</td>
<td>25.4</td>
<td>18.7</td>
<td>Apr.</td>
<td>89</td>
</tr>
<tr>
<td>0.86</td>
<td>8</td>
<td>3.0</td>
<td>1.6</td>
<td>24.7</td>
<td>20.7</td>
<td>May</td>
<td>89</td>
</tr>
<tr>
<td>0.30</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>26</td>
<td>21.4</td>
<td>June</td>
<td>89</td>
</tr>
<tr>
<td>0.23</td>
<td>7</td>
<td>1.7</td>
<td>1.0</td>
<td>24.7</td>
<td>17.3</td>
<td>July</td>
<td>89</td>
</tr>
<tr>
<td>0.18</td>
<td>16</td>
<td>4.8</td>
<td>2.7</td>
<td>30</td>
<td>25.4</td>
<td>Aug.</td>
<td>89</td>
</tr>
<tr>
<td>0.20</td>
<td>7.5</td>
<td>4.0</td>
<td>1.6</td>
<td>25.4</td>
<td>20.7</td>
<td>Sep.</td>
<td>89</td>
</tr>
<tr>
<td>0.12</td>
<td>11.70</td>
<td>3.9</td>
<td>2.5</td>
<td>34.7</td>
<td>28.0</td>
<td>Oct.</td>
<td>89</td>
</tr>
<tr>
<td>0.38 (Av.)</td>
<td>10 (Av.)</td>
<td>42 (Max.)</td>
<td>23 (Av.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 1.6. Different values of waves’ specifications during one year in Anzali.

If one would like to calculate the Significant Wave Height (Hs), one has:

For highest waves: \( \bar{H}^2 = \frac{\Sigma H^2}{N} = 12.73 \)

For mean waves: \( \bar{H}^2 = \frac{\Sigma H^2}{N} = 3.93 \)
For sum of mean highest : \( \bar{H}^2 = \Sigma H^2 / N = 16.66 \)

<table>
<thead>
<tr>
<th>Intervals</th>
<th>Probability for the Highest Waves</th>
<th>Probability of Mean Waves</th>
<th>Probability of sum of Highest and Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 - 0.50</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>0.50 - 1.00</td>
<td>0.077</td>
<td>0.24</td>
<td>0.06</td>
</tr>
<tr>
<td>1.00 - 1.50</td>
<td>0.145</td>
<td>0.39</td>
<td>0.11</td>
</tr>
<tr>
<td>1.50 - 2.00</td>
<td>0.197</td>
<td>0.43</td>
<td>0.16</td>
</tr>
<tr>
<td>2.00 - 2.50</td>
<td>0.230</td>
<td>0.37</td>
<td>0.19</td>
</tr>
<tr>
<td>2.50 - 3.00</td>
<td>0.240</td>
<td>0.26</td>
<td>0.20</td>
</tr>
<tr>
<td>3.00 - 3.50</td>
<td>0.230</td>
<td>0.15</td>
<td>0.21</td>
</tr>
<tr>
<td>3.50 - 4.00</td>
<td>0.210</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.00 - 4.50</td>
<td>0.180</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.50 - 5.00</td>
<td>0.140</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.00 - 5.50</td>
<td>0.110</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.50 - 6.00</td>
<td>0.080</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.100</td>
<td>2.70</td>
<td>4.95</td>
</tr>
</tbody>
</table>

Table 1.7.

As it is observed from Table 1.7, selecting \( H_s = 2.70 \) (m) as a design value is a little low and the probability of damage after constructing will be high. For solving of this problem, one can use the Return Period. For different return periods with \( H_s = 2.70 \) (m), one has:

\[
H(R) = \left( \frac{H}{5.9} \right) \ln(365R)
\]

\[
H(2) = 3.00 \quad H(3) = 3.20 \quad H(4) = 3.35 \quad H(5) = 3.45
\]

Reviewing of waves shows that wave height of \( H_s = 3.50 \) (m) with period of 10 (s) and return period of 5 (yrs) is acceptable. However, we have to notice the records of waves are so incomplete, then for studying of waves carefully, it is needed continuously waves' record during past several years.

1.8. Wave Producing due to Tides
Tides are the fluctuated movement affecting in Oceans which are created regarding to the gravity of planets. Consequently, the water level becomes up and down periodically. However, the period and height of movement, both in time and from one point to another point varies based on relative position of effective planets. Since the gravity of planets is proportion with their mass, \( M \), and their distance from Earth, i.e. \( M/r^2 \), so only the moon and sun have noticeable effects and the effect of moon due to short distance compared with sun is 2.17 times greater than the effect of sun.

On the basis on fluctuation of water level, some currents are created which the whole water volume effects on it. Also, direction of motion and intensity changes in basis of time and place
1.8.1. Calculations of Currents due to Tides
The height of tides and their occurrence time are determined for particular days. Then the changes of curve are drown between two successive points of tide based on following relations:

\[ H = a \cos (k(T + b)) + c \]  \hspace{1cm} (1.3)

In this equation one has:

- \( H \): Height of Water in (m)
- \( T \): Time in (hrs)
- \( a = (H_1 - H_2) / 2 \)
- \( H_1 \): Height of water at point 1
- \( H_2 \): Height of water at point 2
- \( T_1 \): Time at point 1
- \( T_2 \): Time at point 2
- \( C = (H_1 + H_2) / 2 \)
- \( K = \pi / (T_2 - T_1) \)

Since in the Caspian Sea the tide phenomenon is not too noticeable, and it doesn’t effect the waves in this Sea, therefore, it is avoided to discuss about wave producing due to Tides.

1.9. Proceeding of Waves at the Certain Place
Practically, the such waves are considered which they are produced by the continuously blowing of wind. The important parameter used to calculate of wave height is the Fetch.

Fetch shows the area which the wind is blowing with constant speed and direction in the certain period. The region of mentioned area are recognized by following points:
1. The experiences show in measurement of velocities in a supposed direction the safety factor will be desired when the velocity vector would have 15° obligation regarding to the wind blowing direction and it is undesirable when it is over than 45°.
2. Generally, the Fetch in the certain direction is consist of a sector which their radiuseses have 45° angle regarding to the mentioned direction.
3. The length of straight Fetch is the straight line which connects the target point in the assumed direction to the nearest available shoreline.
4. Calculation of height in the basis of straight fetch length is not common and instead of it one can use the Effective Fetch Length.

1.9.1. The Effective Fetch Method
For any desirable point on the basis of straight fetch axis, one has to draw 7 radius with angles of 6° from both sides. The limitation to draw the radiuseses every both sides is 45°.

1.10. The methods used for Height and Period of Waves

1. Estivenson Method
In this method the following relations show the height of waves. In this formula \( F \) is length of effective Fetch for more than 30 nautical mile.

\[ H = 1.5 F^{0.5} \] \hspace{1cm} (For the Fetch length more than 30 nm.) \hspace{1cm} (1.4)
Fig. 1.9. Calculation of Fetch length

Page 16
2H = 1.5F^{0.5} + 2.5 - F^{0.25} \quad \text{(For the Fetch length less than 30 nm.)} \quad (1.5)

In these equations one has:

- **H**: Wave Height (ft)
- **F**: Length of Fetch (Nm), \(1 \text{(Nm)} = 1.852 \text{ (km)}\)

The other formula which has been indicated in the book published by Mr. Kabir Sadeghi related to the velocity of wind is:

\[ H_{\text{max}} = 0.026V^2 \]

which \(V\) is velocity in (knot), \(1 \text{(knot)} = 0.515 \text{ (m/sec.) (ft)}\), and \(H\) is Wave Height in ft.

2. **Moltiver Method**

This method is based on wind velocity and fetch length as following:

\[ H = 0.17(VF)^{1/2} \]

and for lakes one can use:

(Moltiver-Estivenson)

\[ H = 0.17(VF)^{1/2} + [2.5 - F^{1/4}] \]

In this relation, one has:

- **H**: Wave Height (ft)
- **F**: Length of Fetch (mile), \(1 \text{(mile)} = 1.6 \text{ (km)}\)
- **V**: Wind Velocity (mile/hour), \(1 \text{(mile/hour)} = 0.44 \text{ (m/sec.)}\)

Above formula could be written in the metric system as:

\[ H = 0.032(VF)^{1/2} + [0.75 - 0.27F^{0.25}] \]

which \(F\) in (km), \(V\) in (km)/hour and \(H\) in (m).

The phrase inside the brackets will be neglected for effective length more than 30 (km).

3. **Monk Method**

In this method the height of waves in the deep water are expressed as:

\[ H_0 = 0.0555(VF)^{0.5} \]

\[ T = 0.5(FV^2)^{1/4} \]

which:

- **\(H_0\)**: Wave Height in Deep Water (ft)
- **\(F\)**: Length of Fetch (Nm=1.852 m)
- **\(V\)**: Wind Velocity (knot=0.515 m/sec.)
4. The most modern method related to the waves which are creating by the winds is the Fetch method:

\[ H = 0.0016 \frac{V}{10} (F/g)^{0.5} \quad \text{and} \quad T = 0.30 (\frac{V}{10})^{0.4} \frac{foga}{10^{0.3}} \]

In this formula, \( V \) in (m/sec.), \( F \) in (m), \( g \) in (m/sec.)^2 and \( T \) in (sec.).

The subscript of 10 in \( \frac{V}{10} \) shows the wind velocity has been considered in 10 (m) above water surface.

The mentioned method is used for deep water and it has to be noticed the shape of supply, topography and time of wind blowing will have influence in the wave height.

5. \textit{Pierson and Mosquit} Method.

\[ H = 0.0185 (V_{19.5})^2 \]

which \( H \) in (ft), \( V \) in (knot) and superscript of 19.5 shows the height where the wind velocity has been measured above water level.

It has to be expressed the wind velocity close to the low lands is less than the velocity in high lands.

The Table 1.8. shows the direction and maximum wind velocity in several Northern cities of Iran.

<table>
<thead>
<tr>
<th>Name of City</th>
<th>General Direction</th>
<th>Max Wind Velocity (knot)</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Babolsar</td>
<td>North-East</td>
<td>28 - 23</td>
<td>West</td>
</tr>
<tr>
<td>Bandare Anzali</td>
<td>North</td>
<td>48 - 55</td>
<td>North-West</td>
</tr>
<tr>
<td>Rasht</td>
<td>West</td>
<td>34 - 40</td>
<td>South-West</td>
</tr>
<tr>
<td>Ramsar</td>
<td>South-West</td>
<td>34 - 40</td>
<td>West</td>
</tr>
</tbody>
</table>

Table 1.8. Max. wind velocity and its direction in hours of 6:30, 12:30, 18:30 local.

Since the rubble mound breakwaters in shore line of Iran are designed based on the wind blowing which are mostly blowing in the Mazandaran region, therefore, the wind phenomenon in this zone especially in Babolsar will be studied.

The dominant winds that mostly called seasonal winds, are blowing in the certain direction for long period. Then the specifications and directions of such winds have been indicated in the following tables.

<table>
<thead>
<tr>
<th>Annum</th>
<th>Month</th>
<th>Wind Direction (degree)</th>
<th>Numbers of Days in Month</th>
<th>Velocity (knot)</th>
<th>Velocity (km/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1953</td>
<td>September</td>
<td>270</td>
<td>3</td>
<td>44</td>
<td>81.5</td>
</tr>
<tr>
<td>1956</td>
<td>April</td>
<td>270</td>
<td>21</td>
<td>40</td>
<td>74.1</td>
</tr>
<tr>
<td>1957</td>
<td>February</td>
<td>270</td>
<td>27</td>
<td>40</td>
<td>74.1</td>
</tr>
<tr>
<td>1967</td>
<td>July</td>
<td>20</td>
<td>5</td>
<td>40</td>
<td>74.1</td>
</tr>
</tbody>
</table>

Results: \( 270^\circ \) 35 per annum 44 81.5

Table 1.9. Dominant winds during past 25 years.
Farhad Mirfakhraee, Participant in HE, b, 95-96
Report Title: Investigation on the effects of Caspian Sea upon its coast and presenting suitable protective solutions

<table>
<thead>
<tr>
<th>Annum</th>
<th>Wind Direction</th>
<th>Wind Velocity (knot)</th>
<th>Percentage of time</th>
<th>Percentage of stillness</th>
<th>Numbers of Observation</th>
<th>Velocity (km/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1951</td>
<td>270</td>
<td>7.1</td>
<td>24.6</td>
<td>9.4</td>
<td>1824</td>
<td>13.15</td>
</tr>
<tr>
<td>1952</td>
<td>270</td>
<td>3.4</td>
<td>36.7</td>
<td>8.9</td>
<td>2777</td>
<td>6.30</td>
</tr>
<tr>
<td>1953</td>
<td>270</td>
<td>3.4</td>
<td>30.5</td>
<td>2.7</td>
<td>2920</td>
<td>6.3</td>
</tr>
<tr>
<td>1954</td>
<td>270</td>
<td>3.10</td>
<td>39.5</td>
<td>3.8</td>
<td>2970</td>
<td>5.74</td>
</tr>
<tr>
<td>1955</td>
<td>270</td>
<td>3.3</td>
<td>43.7</td>
<td>2.1</td>
<td>2912</td>
<td>6.11</td>
</tr>
<tr>
<td>1956</td>
<td>270</td>
<td>3.0</td>
<td>41.1</td>
<td>0.4</td>
<td>2928</td>
<td>5.56</td>
</tr>
<tr>
<td>1957</td>
<td>270</td>
<td>4.3</td>
<td>17.9</td>
<td>32.4</td>
<td>2920</td>
<td>7.96</td>
</tr>
<tr>
<td>1958</td>
<td>45</td>
<td>4.3</td>
<td>11.2</td>
<td>51.2</td>
<td>2920</td>
<td>7.96</td>
</tr>
<tr>
<td>1959</td>
<td>270</td>
<td>4.8</td>
<td>8.9</td>
<td>66.7</td>
<td>2920</td>
<td>8.90</td>
</tr>
<tr>
<td>1960</td>
<td>270</td>
<td>4.4</td>
<td>8.4</td>
<td>68.8</td>
<td>2928</td>
<td>8.15</td>
</tr>
<tr>
<td>1961</td>
<td>270</td>
<td>4.9</td>
<td>8.1</td>
<td>70.4</td>
<td>2920</td>
<td>9.08</td>
</tr>
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<td>1962</td>
<td>270</td>
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<td>60.5</td>
<td>2920</td>
<td>10.27</td>
</tr>
<tr>
<td>1963</td>
<td>270</td>
<td>5.2</td>
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<td>68.10</td>
<td>2818</td>
<td>9.63</td>
</tr>
<tr>
<td>1964</td>
<td>270</td>
<td>5.2</td>
<td>12.3</td>
<td>45.7</td>
<td>2927</td>
<td>9.63</td>
</tr>
<tr>
<td>1965</td>
<td>270</td>
<td>4.8</td>
<td>18.3</td>
<td>42.5</td>
<td>2920</td>
<td>8.90</td>
</tr>
<tr>
<td>1966</td>
<td>270</td>
<td>5.9</td>
<td>13.4</td>
<td>37.9</td>
<td>2920</td>
<td>10.93</td>
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<td>1967</td>
<td>270</td>
<td>7.0</td>
<td>10.4</td>
<td>36.6</td>
<td>2920</td>
<td>12.96</td>
</tr>
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<td>1968</td>
<td>270</td>
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<td>11.4</td>
<td>44.6</td>
<td>2928</td>
<td>12.78</td>
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<tr>
<td>1969</td>
<td>45</td>
<td>5.4</td>
<td>10.6</td>
<td>48.2</td>
<td>2893</td>
<td>10.00</td>
</tr>
<tr>
<td>1970</td>
<td>270</td>
<td>6.2</td>
<td>9.6</td>
<td>46.8</td>
<td>2920</td>
<td>11.48</td>
</tr>
<tr>
<td>1971</td>
<td>315</td>
<td>6.1</td>
<td>10.4</td>
<td>47.3</td>
<td>2920</td>
<td>11.30</td>
</tr>
<tr>
<td>1972</td>
<td>45</td>
<td>5.1</td>
<td>9.8</td>
<td>48.0</td>
<td>2761</td>
<td>9.45</td>
</tr>
<tr>
<td>1973</td>
<td>315</td>
<td>6.1</td>
<td>9.9</td>
<td>54.9</td>
<td>2920</td>
<td>11.30</td>
</tr>
<tr>
<td>1974</td>
<td>315</td>
<td>6.1</td>
<td>9.8</td>
<td>56.4</td>
<td>2919</td>
<td>11.30</td>
</tr>
<tr>
<td>1975</td>
<td>315</td>
<td>5.8</td>
<td>10.9</td>
<td>50.8</td>
<td>2882</td>
<td>10.74</td>
</tr>
<tr>
<td>Av. 25 years</td>
<td>270</td>
<td>4.5</td>
<td>16.4</td>
<td>40.9</td>
<td>2858.2</td>
<td>8.23</td>
</tr>
</tbody>
</table>

Table.1.10. Specifications of dominant waves during 25 years (1951 - 1975)

![General direction of wind.](image)
The analysis of statistics from information in years since 1977 till 1994 shows the Max. and Min. humidity has been 89 and 63 respectively and the Max. volume of rainfall in this period has been 466.7 (m.m.) and the changes of temperature has been from 1 till 30 degree of Centigrade.

The Table 1.12, shows the dominant winds in Nowshahr.

<table>
<thead>
<tr>
<th>Expression</th>
<th>Wind Direction</th>
<th>Wind Velocity (knot)</th>
<th>Percentage of Time</th>
<th>Percentage of Stillness</th>
<th>Number of Observation</th>
<th>Velocity (km/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>270</td>
<td>5.8</td>
<td>41.2</td>
<td>45.3</td>
<td>239.4</td>
<td>10.74</td>
</tr>
<tr>
<td>February</td>
<td>270</td>
<td>5.2</td>
<td>19.1</td>
<td>41.2</td>
<td>219.1</td>
<td>9.63</td>
</tr>
<tr>
<td>March</td>
<td>270</td>
<td>5.3</td>
<td>16.7</td>
<td>36.1</td>
<td>240.3</td>
<td>9.82</td>
</tr>
<tr>
<td>April</td>
<td>270</td>
<td>5.1</td>
<td>19.2</td>
<td>36.7</td>
<td>232.3</td>
<td>9.45</td>
</tr>
<tr>
<td>May</td>
<td>270</td>
<td>5.3</td>
<td>16.7</td>
<td>37.2</td>
<td>243.0</td>
<td>9.82</td>
</tr>
<tr>
<td>June</td>
<td>270</td>
<td>4.7</td>
<td>19.5</td>
<td>36.3</td>
<td>235.7</td>
<td>8.70</td>
</tr>
<tr>
<td>July</td>
<td>270</td>
<td>4.5</td>
<td>22.9</td>
<td>32.7</td>
<td>244.3</td>
<td>8.33</td>
</tr>
<tr>
<td>August</td>
<td>270</td>
<td>4.3</td>
<td>19.9</td>
<td>37.9</td>
<td>243.9</td>
<td>7.96</td>
</tr>
<tr>
<td>September</td>
<td>270</td>
<td>5.9</td>
<td>17.7</td>
<td>41.1</td>
<td>236.4</td>
<td>10.93</td>
</tr>
<tr>
<td>October</td>
<td>270</td>
<td>4.1</td>
<td>12.6</td>
<td>45.9</td>
<td>244.3</td>
<td>7.59</td>
</tr>
<tr>
<td>November</td>
<td>270</td>
<td>3.9</td>
<td>11.7</td>
<td>47.3</td>
<td>235.3</td>
<td>7.23</td>
</tr>
<tr>
<td>December</td>
<td>270</td>
<td>3.9</td>
<td>12.0</td>
<td>49.4</td>
<td>244.2</td>
<td>7.23</td>
</tr>
<tr>
<td>Result</td>
<td>270</td>
<td>4.5</td>
<td>16.14</td>
<td>40.9</td>
<td>238.2</td>
<td>8.33</td>
</tr>
</tbody>
</table>

Table 1.11. Specifications of dominant waves during 25 years.

Table 1.12. Specification of direction and intensity of dominant winds in Nowshahr and Chaloos

The investigations show in the dominant wind zone, Western-North toward Eastern-South (angle of 270°), the wind is blowing 14.6% of time with average velocity of 8.33 (km/hr) and the strong winds are blowing 35 days of year with 81.5 (km/hr) in the same direction of seasonal winds.
In the Bandare anzali the wind specifications and its directions has been indicated in Table 1.13.

<table>
<thead>
<tr>
<th>Vel. (m/Sc.)</th>
<th>Direction</th>
<th>Vel. (m/Sc.)</th>
<th>Direction</th>
<th>Vel. (m/Sc.)</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>330</td>
<td>13</td>
<td>330</td>
<td>12</td>
<td>290</td>
</tr>
<tr>
<td>8</td>
<td>350</td>
<td>19</td>
<td>340</td>
<td>8</td>
<td>340</td>
</tr>
<tr>
<td>12</td>
<td>340</td>
<td>12</td>
<td>340</td>
<td>10</td>
<td>200</td>
</tr>
<tr>
<td>10</td>
<td>30</td>
<td>12</td>
<td>350</td>
<td>15</td>
<td>20</td>
</tr>
<tr>
<td>8</td>
<td>340</td>
<td>7</td>
<td>350</td>
<td>10</td>
<td>330</td>
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<td>8</td>
<td>340</td>
<td>7</td>
<td>350</td>
<td>11</td>
<td>330</td>
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<td>5</td>
<td>100</td>
<td>6</td>
<td>300</td>
<td>11</td>
<td>40</td>
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<td>15</td>
<td>330</td>
<td>11</td>
<td>300</td>
<td>13</td>
<td>200</td>
</tr>
<tr>
<td>9</td>
<td>20</td>
<td>11</td>
<td>40</td>
<td>7</td>
<td>220</td>
</tr>
<tr>
<td>10</td>
<td>340</td>
<td>11</td>
<td>350</td>
<td>11</td>
<td>330</td>
</tr>
</tbody>
</table>

Table 1.13. Specifications of wind in Bandare anzali in (1989 - 1991)

1.11. Selection of Wave Height and Period to Design
The following assumptions are considered for design wave:

Effective Fetch = 30 Nautical Mile = 30 x 1.852 = 55.56 km

Babolsar, Max. wind velocity = 24 knot
Babolsar, Mean wind velocity = 4.5 knot
Bandare Anzali, Max. wind velocity = 42 knot
Bandare anzali, Mean wind velocity = 23 knot

Then the wave heights are calculated based on different indicated formula shown in Table 1.14.

<table>
<thead>
<tr>
<th>Babolsar</th>
<th>Anzali</th>
</tr>
</thead>
<tbody>
<tr>
<td>Formula No.</td>
<td>Max. Wind Vel. (m/Sc.)</td>
</tr>
<tr>
<td>(1.4)</td>
<td>2.5</td>
</tr>
<tr>
<td>(1.6)</td>
<td>15.10</td>
</tr>
<tr>
<td>(1.7)</td>
<td>2.19</td>
</tr>
<tr>
<td>(1.8)</td>
<td>2.21</td>
</tr>
<tr>
<td>(1.9)</td>
<td>2.17</td>
</tr>
<tr>
<td>(1.10)</td>
<td>2.00</td>
</tr>
<tr>
<td>(1.12)</td>
<td>2.73</td>
</tr>
<tr>
<td>(1.14)</td>
<td>12.46</td>
</tr>
</tbody>
</table>

Table 1.14. Predicting of waves height calculated by different formulas.

Since in the above table has been used from Fetch of 30 Nm., the calculated volumes might be unacceptable. So for solving of this problem is used of 50 Nm. Fetch length. The results after correction have been indicated in Table 1.15.
From the above table the wave height of $H = 3.5$ (m) is selected which is acceptable basis on statistical analysis. Then after choosing of wave height as a design value, the wave period will be calculated as shown in Table.1.16.

### Table.1.15. Predicting of waves height calculated by different formula (Fetch = 50 n.m.)

<table>
<thead>
<tr>
<th>Formula No.</th>
<th>Babolsar Max. Wind Vel. (m/Sc.)</th>
<th>Babolsar Mean Wind Vel. (m/Sc.)</th>
<th>Anzali Max. Wind Vel. (m/Sc.)</th>
<th>Anzali Mean Wind Vel. (m/Sc.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1.4)</td>
<td>3.23</td>
<td>3.23</td>
<td>3.23</td>
<td>3.23</td>
</tr>
<tr>
<td>(1.6)</td>
<td>15.10</td>
<td>0.16</td>
<td>14.00</td>
<td>4.19</td>
</tr>
<tr>
<td>(1.7)</td>
<td>2.43</td>
<td>0.77</td>
<td>2.37</td>
<td>1.76</td>
</tr>
<tr>
<td>(1.8)</td>
<td>2.38</td>
<td>0.73</td>
<td>2.32</td>
<td>1.70</td>
</tr>
<tr>
<td>(1.9)</td>
<td>2.70</td>
<td>0.80</td>
<td>2.63</td>
<td>1.92</td>
</tr>
<tr>
<td>(1.10)</td>
<td>0.80</td>
<td>0.25</td>
<td>0.78</td>
<td>0.57</td>
</tr>
<tr>
<td>(1.12)</td>
<td>3.52</td>
<td>0.36</td>
<td>3.36</td>
<td>1.84</td>
</tr>
<tr>
<td>(1.14)</td>
<td>12.46</td>
<td>0.13</td>
<td>11.40</td>
<td>3.42</td>
</tr>
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</table>

### Table.1.16. Predicting of wave period in Second (Fetch = 30 n.m)

<table>
<thead>
<tr>
<th>Formula No.</th>
<th>Babolsar Max. Velocity</th>
<th>Babolsar Mean Velocity</th>
<th>Anzali Max. Velocity</th>
<th>Anzali Mean Velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1.11)</td>
<td>7.80</td>
<td>2.50</td>
<td>7.60</td>
<td>5.60</td>
</tr>
<tr>
<td>(1.13)</td>
<td>5.6</td>
<td>2.25</td>
<td>5.50</td>
<td>4.33</td>
</tr>
</tbody>
</table>

### Table.1.17. Predicting of waves period in second (Fetch = 50 n.m.)

<table>
<thead>
<tr>
<th>Formula No.</th>
<th>Babolsar Max. Velocity</th>
<th>Babolsar Mean Velocity</th>
<th>Anzali Max. Velocity</th>
<th>Anzali Mean Velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1.11)</td>
<td>8.82</td>
<td>2.82</td>
<td>8.60</td>
<td>6.4</td>
</tr>
<tr>
<td>(1.13)</td>
<td>6.53</td>
<td>2.62</td>
<td>6.41</td>
<td>5.0</td>
</tr>
</tbody>
</table>

In the above tables, the periods have been calculated for 50 and 30 (Nm) fetch in deep water. Consequently, the period of 8 (sec.) is acceptable.
Chapter 2

Wave Theory
2.1. Theoretical Studying of Waves

All types of waves related to the gravity, acoustic and electromagnetism follow the same states of wave equations. The variances of any case depends on type of the physical phenomenon and terminated states. Generally, the surface equations and terminated states may be linear or nonlinear. The surface of wave must follow the particular case of Laplace equations. Before beginning of linear analysis of waves some specifications about shallow waves will be useful.

The used coordinates here is the Cartesian coordinate that has been located on the Sill Water Level (S.W.L). The depth of sea is equal d from S.W.L., wave height is H which is the distance between lowest point of wave till its crest. The wave length is the distance between two successive wave crest and c is the wave celerity of wave propagate.

The changes of water level surface is measured by the function of $h(x,t)$ that x and t is variations. The following boundary conditions has to be reached by the water inside waves:

1. The particles of water are not escaping of free surface of water, in the other hand the particles must be on the free surface. To reach this case the velocity of particles in $z=h$ should be equal to the normal velocity on the free surface, then:

$$V|_{z=h} = V_n$$  \hspace{1cm} (2.1a)

This case refers to the motional surface state. If the flow would be non rotational, then the velocity of fluid may be shown by the potential function:

$$V|_{z=h} = \nabla \Phi|_{z=h} = \partial \Phi / \partial n|_{z=h} n$$  \hspace{1cm} (2.1b)

Which the $n$ is the normal unique axis into upward on the free surface. Another assumption is that the $h$ is too small compared with wave length. Then the mentioned case occurs at $z=0$, so with acceptable approximation $k$ can be replaced with $n$. Then:
\[ V \big|_{z=h} = (\partial h/\partial t)k = \partial \Phi/\partial z \big|_{z=0} k \]  
\[ (2.2) \]

2. In the bottom of sea the fluid particles can not pass from the solid section, then at the \( z = -d \) one has:

\[ V \cdot N = \partial \Phi/\partial N = 0 \]  
\[ (2.3) \]

Which \( N \) is the normal axis on the sea bottom upwardly.

3. The pressure on the open surface for any position of \( x \) and \( t \) will be equal zero. In this case the non rotational flow case has been considered. The *Bernoulli* equations for free surface is:

\[ \partial \Phi/\partial t + gh + V^2/2 = 0 \]  
\[ (2.4) \]

If the free surface is the base surface, therefore the time function in the above equation will become zero. The mentioned equation is called the dynamically equation of surface. Since in this equation there is second power of velocity term, so this relation is nonlinear. For the very little displacement of free surface, the second power of nonlinear term could be neglected compared with other terms. Then the dynamically equation of surface becomes linear.

\[ h = -(1/g) \partial \Phi/\partial t \big|_{z=h} \]  
\[ (2.5) \]

In the physically case, it is assumed that the flow is linear and it is sufficient quiet, then the results of the cinematic energy of particles is very less than the mechanically energy. Here there is one equation is that how much is the little displacement of free surface particles. The answer is related to the surface movement of particular wave. The physical observation under the water shows the particles are moving on the surface close to the circular pattern. The radius of this circle is \( H/2 \) which is half of the wave height. See Fig.2.2.

![Fig.2.2. Approximate movement of particles upon the free surface](image-url)

If one particle moves in the period of \( t \) ( \( t \) is the time for repetition of movement), in this case the particle moves \( \pi H \) and the mean velocity will be: \( \pi H/\lambda \). Generally, it is supposed that linear equations are used for \( H/\lambda \) more than 1/50. The linear free surface state is gained by neglecting of \( h \) and combining of two Eq., i.e. (2.2) and (2.5).
The discussion is about continuous flow inside the wave. Then, the continuity equation must be applied. If the flow is non-circular, by applying of continuity Eq. the velocity will be shown by the potential function:

$$\nabla^2 \Phi = 0$$ \hspace{1cm} (2.7)

which is the same Laplace function.

The solution of above Eq. is:

$$\Phi = X(x)Z(z)T(t)$$ \hspace{1cm} (2.8)

After replacing of it in the Eq. one has:

$$\left(\frac{1}{X}\right)\left(\frac{\partial^2 X}{\partial x^2}\right) = -1/z\left(\frac{\partial^2 Z}{\partial z^2}\right) = -k^2$$ \hspace{1cm} (2.9)

Hence, the $k$ is the constant which is called wave number. The general solution for Eq. for $X, Z$ is:

$$Z(z) = C_1 \cosh(kz + \alpha)$$ \hspace{1cm} (2.10)

$$X(x) = C_2 \sin(kx + \beta)$$ \hspace{1cm} (2.11)

$C_1, C_2$ and $\beta$ are the constants. Special type of $Z(z)$ is recognized by applying of boundary conditions. In the $z = -d$, one has:

$$\frac{dZ}{dz} = C_1 k \sinh(-kd + \alpha) = 0$$

The solution of Eq. will be as $\alpha = kd$, then:

$$Z(z) = C_1 \cosh(kz + kd) = 0$$ \hspace{1cm} (2.12)

The linear state of free surface can be used for determining of function depended to time. Then the equations of (2.8) and (2.9) are replaced in the equation (2.6). One has:

$$\frac{1}{T} \frac{dT^2}{dT^2} + \omega^2 = 0$$ \hspace{1cm} (2.13)

$$\omega = |k \tan(h(kd))| = 0$$ \hspace{1cm} (2.14)

The general solution for equation (2.13) will be:

$$T(t) = C_3 \sin(\omega t + \gamma)$$ \hspace{1cm} (2.15)

After combining of Eq. and neglecting of $\gamma$ and $\beta$ and knowing $\gamma = \beta = 0$, then the potential function of velocity will be:

$$\phi = A \cosh(kz + kd) \sin(kx) \sin(\omega t)$$ \hspace{1cm} (2.16)

The factors of $C_1, C_2, C_3$ have been observed in the amplitude. With considering of velocity potential, the displacement of free surface will become:
\[ h = -i/g(\partial^2/\partial t) \mid = -\omega A/g \cosh(kd) \sin(kx) \cos(\omega t) = a \sin(kx) \cos(\omega t) \]  
\[ \text{(2.17)} \]

which \( a \) is amplitude of the wave. In this Eq. the \( A = -ag/\omega \) is replaced. Then:

\[ \phi = (-\omega A/g)[\cosh(kz+kd)/\cosh(kz)] \sin(kx) \sin(\omega t) \]  
\[ \text{(2.18)} \]

In general solution of the mentioned Eq. there are four functions like as \( \sin, \cos, x, t \), which is expressed four type of wave for it.

\[ h_1 = a \sin(kx) \cos(\omega t) \]
\[ h_2 = a \cos(kx) \cos(\omega t) \]
\[ h_3 = a \sin(kx) \sin(\omega t) \]
\[ h_4 = a \sin(kx) \cos(\omega t) \]

Then, one has:

\[ h' = h_1 + h_4 = a \sin(kx + \omega t) \]  
\[ \text{(2.19)} \]
\[ h'' = h_2 + h_3 = a \cos(kx - \omega t) \]  
\[ \text{(2.20)} \]

Then the potential of velocity will be:

\[ \phi' = -\frac{ag}{\omega} \frac{\cosh(kz+kd)}{\cosh(kd)} \cos(kx + \omega t) \]  
\[ \text{(2.21)} \]
\[ \phi'' = \frac{ag}{\omega} \frac{\cosh(kz+kd)}{\cosh(kd)} \sin(kx - \omega t) \]  
\[ \text{(2.22)} \]

If the acceptable coordinate coinciding with the transition of wave, so the angle of \( \cos \) in above Eq. becomes constant, then:

\[ kx - \omega t = \text{Cte.} \Rightarrow kdx - \omega dt = 0 \]

In the other hand the celerity is:

\[ C = \frac{dx}{dt} = \frac{\omega}{k} = \frac{1}{T} = f\lambda \]  
\[ \text{(2.23)} \]

which \( f \) is the frequency of wave in Hertz, \( \lambda \) wave length and \( t \) time of period. Then it is replaced \( \omega \) from Eq.2.14 in the above Eq.

\[ c = \left| \frac{g}{k} \tanh(kd) \right|^{1/2} \]  
\[ \text{(2.24)} \]
\[ \lambda = C.T = \frac{gT^2}{2\pi} \tanh(\frac{2\pi d}{\lambda}) \]  
\[ \text{(2.25)} \]

The last Eq. shows the wave length in deep water is going to become close to \( gT^2/2\pi \) and in the shallow water becomes close to \( (gd)^{1/2} \). T. In the other hand, it
shows the wave length is the function of gravity and period in the deep water or depth
and period in shallow water. It has been shown the wave length and celerity increases
with depth in shallow water. Now, the movement of one particle is considered. The
velocity components of water going toward right direction are:

\[
\begin{align*}
u &= \frac{\partial \phi}{\partial x} = \frac{agk}{\omega} \frac{\cosh (kz + kd)}{\cosh (kd)} \cos (kx - \omega t) \\
u &= \frac{\partial \phi}{\partial z} = \frac{agk}{\omega} \frac{\sin (kz + kd)}{\cosh (kd)} \sin (kx - \omega t)
\end{align*}
\]

(2.26) \hspace{2cm} (2.27)

A fixed point like \((x_0, y_0)\) on the wave is considered. As it is clear from above Eq., \(u\) and \(v\) are the Sin variants in the term of time and location. The points of \(x\) and \(y\) are
the average points for particles. Now, noticing Fig.2.3 it is applied two new axis
named \(\xi\) and \(\zeta\) for expressing of instantaneous location of particles. The velocity
components can be expressed in the term of new axis.

![Fig.2.3. Location vector for a particle](image)

\[
\begin{align*}
u &= \frac{\partial \xi}{\partial t} \\
v &= \frac{\partial \zeta}{\partial t}
\end{align*}
\]

(2.28) \hspace{2cm} (2.29)

By neglecting of \(u\) and \(v\) and comparing them with above Eq. and integrating from
t= 0 till \(t=t\) the component of displacement for \(\xi\) and \(\zeta\) will be:

\[
\begin{align*}
\xi &= \int_{0}^{t} \frac{\partial \phi}{\partial x} dt \bigg|_{x_0,y_0} = -\frac{agk}{\omega^2} \frac{\cosh(kZ_0 + kd)}{\cosh(kd)} \sin(kx_0 - \omega t) \\
\zeta &= \int_{0}^{t} \frac{\partial \phi}{\partial z} dt \bigg|_{x_0,y_0} = -\frac{agk}{\omega^2} \frac{\sinh(kZ_0 + kd)}{\cosh(kd)} \cosh(kx_0 - \omega t)
\end{align*}
\]

(2.30) \hspace{2cm} (2.31)
The above Eq. shows for any position of particles, i.e. \((x_0, y_0)\) the pattern of fluid particles could be expressed by ratio of depth to wave length. i.e. \(d/\lambda = kd/2\pi\).

In the Fig.2.3, the position of any point has been shown by vector of \(r\) by using of \(\xi\) and \(\zeta\) relations and noticing previous Eq. will be:

\[
n = \xi i + \zeta k = \frac{a}{\sinh(kd)} \left[ - \cosh(kZ_0 + kd) \sin(kx_0 - \omega t)i + \sinh(kZ_0 + \lambda) \cos(kx_0 - \omega t)k \right]
\]

The various amount of \(kd\) determine the general figure of particles' pass in the Hyperbolic function.

**Case 1:** (Deep water \(1/2 \leq d/\lambda < \infty\))

For this confined, the differentiation of \(d/\lambda\) follow:

\[
tanh(kd) \approx 1, \quad e^{kd}/2 \approx \sinh(kd) \approx \cosh(kd)
\]

Then the Eq. for \(r\) will be:

\[
r = a e^{kz_0} [-\sin(kx_0 - \omega t)i + \cos(kx_0 - \omega t)k]
\]

when the \(z_0\) is minus, it shows the under of Still Water Level. The above Eq. shows the circular radius that it decreases in the power in depth. The waves in this case are called Short Waves. That the wave length is very less than the depth. Fig.2.4a.

**Case 2:** (Water in the intermediate Depth \(1/2 < d/\lambda < 1/2\))

For his case, the Eq. shows one ellipsoid pattern with big and small diameter that it decreases in depth based on power. Fig.2.4b.

**Case 3:** (Shallow Water \(0 < d/\lambda \leq 1/20\))

The Hyperbolic function, in this case will change to the:

\[
\sinh(kd) \approx \tanh(kd), \quad \cosh(kd) \approx 1
\]

Then the Eq. of \(r\) will be as follows:

\[
r = \frac{a}{kd} [-\sin(kx_0 - \omega t)i + (kd + k\lambda) \cos(kx_0 - \omega t)k]
\]
From the celerity of wave and wave length, one has:

\[ C = (gd)^{1/2} \]  

(2.35)  

\[ \lambda = \frac{2\pi}{k} = (gT^2d)^{1/2} \]  

(2.36)  

These relations show the velocity and wave length decreases in the shallow water. Now, the horizontal component of velocity in the shallow water will be:

\[ u = \frac{\alpha g k}{\omega} \cos(kx - \omega t) = \frac{\alpha g}{(gd)^{1/2}} \cos(kx - \omega t) \]  

(2.37)  

From this Eq. it is clear the increase of particles’ velocity is proportion inversely with water depth. Comparing between Celerity \( c \) and horizontal component of velocity \( u \), they will be equal when the depth becomes same value in both case. This phenomenon is created in the wave crest. Then \( \cos(kx - \omega t) = 1 \). In this case the wave is called breaking wave. And when the horizontal component of velocity will be greater than the Celerity \( c \), the wave is called spilling wave.

The proceeding of wave pattern is possible by using of linear theory. One breaking wave with pointed crest has been shown in the Fig. 2.5. The depth of water in the breaking point \( d = a \) could be calculated to be equal to Eq. of \( c = (gd)^{1/2} \) and Eq. of \( u \) with using of \( \lambda = (g T^2 d)^{1/2} \).

2.2. Waves Group  
When we are looking at sea level, there is possible to see waves and determine their directions. These type of waves could be produced by the storms, large group of fish’s passing close to the water level. The waves that they are in the same line are called waves group.
The studying of wave group is begun with two linear waves with the same amplitude, but different wave length. Noticing the previous relations and supposing of Combination Rule, it can be possible determining of displacement of free surface. one has:

\[ h = a \{ \cos(kx - \omega t) + \cos[(k + \Delta k) x - (\omega + \Delta \omega) t] \} \]  

(2.38a)

Then:

\[ h = 2a(\cos \left( \frac{-\Delta k}{2} x - \frac{\Delta \omega}{2} t \right) \cos \left[ \frac{1}{2} (2k + \Delta k) x - \frac{1}{2} (2\omega + \Delta \omega)t \right] \]  

(2.38b)

When \( k \gg \Delta k \) and \( \omega \gg \Delta \omega \), then in the above Eq. the variation of first term of \( \cos \) becomes too less than the second one. If the angle of first term of \( \cos \) is constant, then the velocity of wave group will be:

\[ c_g = \lim_{\Delta k \to 0} \frac{\Delta \omega}{\Delta x} = \frac{d\omega}{dk} \]  

(2.39)

Similarly, if the angle of second term of \( \cos \) becomes constant, then the velocity of wave group will be:

\[ c_g = \lim_{\Delta k \to 0} \frac{\omega + \Delta \omega}{k + \Delta k} = \frac{\omega}{k} = \left(\frac{k}{k} \tanh(kd)\right)^{1/2} \]  

(2.40)

The relation between group velocity and particle velocity is:

\[ c_g = \frac{d\omega}{dk} = \frac{d(kc)}{dk} = \frac{1}{2} C[1 + \frac{2kd}{\sinh(2kd)}] \]  

(2.41)

In deep water \( kd \to \infty \) then \( c_g \to C/2 \)
In shallow water \( kd \to 0 \) then \( c_g \to C \)

Generally it can be written:

\[ c_g = nC , n = \frac{1}{2} [1 + \frac{2kd}{\sinh(2kd)}] \]  

Which \( n \) is the 0.5 for deep water and for the shallow water will be 1.
2.3. Wave Energy

The shallow wave will be produced due to transformation of energy from an external resource like as ship, earthquake and storm. In this case there will be both Cinematic and Potential energy. A wave assumed as following specifications. If the unique dimension is considered on the sheet, the mass of element of water level will become:

Then the potential energy is equal:

\[
\Delta m = \rho h \Delta x
\]

\[
\Delta m \frac{gh}{2} = \rho g \frac{A_{x} h^{2}}{2}
\]

After replacing of \( h \) from previous Eq., the integration along the wave length will gain the whole of potential.

\[
E_{p} = \rho g \frac{a^{2}}{2} \int_{0}^{\lambda} \cos^{2}(kx - \omega t) dx = \frac{\rho g a^{2}}{2k} \left[ \xi_{/2} + \frac{\sin(\xi) \cos(\xi)}{2} \right]_{-\omega t}^{k \lambda - \omega t}
\]

\[
= \frac{\rho g a^{2}}{4} \lambda
\]

Hence, the \( \xi \) is the variant of integrate. Then mass of water which has been indicated in Fig.2.7. The kinetic energy of this mass is:

\[
\Delta E_{k} = \frac{1}{2} \rho (u^{2} + v^{2}) \Delta x \Delta z
\]

After replacing of \( u \) and \( v \) from regarding Eq., the whole of kinetic energy will be obtained in the wave length.

\[
E_{k} \approx \frac{1}{2} \rho \int_{0}^{\lambda} \int_{0}^{\lambda} (u^{2} + v^{2}) dx dz = \frac{1}{2} \rho a^{2} g^{2} k^{2} \frac{\cosh^{2}(kh)}{\omega^{2} \cosh^{2}(kh)} \int_{0}^{\lambda} \int_{0}^{\lambda} \left[ \cosh(\kappa z + kd) \cos^{2}(kx - \omega t) + \sin^{2}(\kappa z +kd) \sin^{2}(kx - \omega t) \right] dx dz = \frac{\rho g a^{2}}{4} \lambda
\]

The upper limit of \( z \) will be zero, because the displacement of free surface in any point is horizontal and too small, then:

\[
\int \cosh^{2}(a \xi) d\xi = \frac{1}{4a} \cosh(2a \xi) \pm \frac{\xi_{/2}}{2}
\]
and from \( \omega \) Eq.

\[
\omega^2 = gk \tanh (kd)
\]

(2.45)

In the basis of Bernoulli Eq. , the energy in unique volume in one point will be:

\[
\rho \frac{\partial \phi}{\partial t} = - \rho g = - p + F(t)
\]

(2.46)

Then:

\[
\Delta E = \rho \frac{\partial \phi}{\partial t} \cdot V = \rho \frac{\partial \phi}{\partial t} \cdot \nabla \phi
\]

With replacing of velocity potential \( \Phi \) by the regarding phrase and integrating of results upon period of \( T \), depth of \( d \), the average energy for one wave will be gained:

\[
E = \frac{\rho a^2 g^2}{4C \cosh^2 (kd)} \int_0^T \int_0^d \left[ \cosh(kz + kd) \sin(kx - \omega t) \cosh(kz + kd) \cos(kx - \omega t)i + \sinh(kz + kd) \sin(kx - \omega t)k \right] dt \cdot dz = \frac{\rho a^2 C_g}{4} \left[ \frac{2kd}{\sinh(2kd)} + 1 \right] i = \frac{\rho a^2 C_g}{4} (2.47)
\]

Hence, \( C_g \) is the group velocity. The integration of \( k \) term in the above Eq. naturally is zero, when the velocity component \( V \) will be parallel with vertical sheet. If the whole energy i.e. \( E = E_p + E_k \) is divided to wave length, the average energy of wave will be given:

\[
E = \frac{E}{\lambda} = \frac{1}{2} \rho a^2
\]

It is known, in the deep water the transitional energy is half of celerity of wave and in the shallow water, transitional energy will be equal the celerity.

2.4. Nonlinear Waves

It was expressed the linear theory could not be used for proceeding of breaking waves. Then for this purpose, the waves with the restricted height that the proportion of height to wave length \( (H/\lambda) \) greater than \( 1/50 \), The nonlinear theory will be used. Then the waves with restricted wave height in the nature are nonlinear. There are some nonlinear theory consisting of Stock's Theory (1847-1880), Grestner Theory (1809). The using of these theories in the practical engineering uses is too difficult and not common. Preferably, the linear theory is used for this purpose.

2.5. Wave reflection Encountering with one obstacle

2.5.1. Fixed Structures

In this section, the momentum and mechanical energy transferring between surface of waves and fixed materials is studied. It is mentioned this section is not necessarily related to a special structure, but, this case is common in engineering problems.
2.5.1.1. The hydrostatic Pressure lower than shallow Wave
The change of hydrostatically pressure is so simple inside fluid. And it depends only to depth of fluid and therefore it is written as:

\[ P = \int_{z}^{0} \gamma dz = -\gamma z \quad (2.48) \]

Which \( z \) is the normal axis to the Still Water Level and it is positive in upward. If the waves only are considered in S.W.L., then the horizontal and vertical velocity of particles due to linear Eq. show the effective movement of fluid in the depth of 1.5 times of wave length in the deep water or equal to, the to the depth from sea bed. Therefore, for non circular flows, the pressure changing can be obtained from *Bernoulli* Eq. It is assumed the flow is going to the right direction, if the flow is enough quiet, then the nonlinear term of \( 1/2 \rho V^2 \) can be neglected. Then:

\[ P = \rho \frac{\partial \phi}{\partial t} - \rho g z = \rho g a \frac{\cosh(kd + kz)}{\cosh(kd)} \cos(kx - \omega t) - \rho g z \quad (2.49) \]

Hence, the free surface is the base surface of energy. If the \( \cos(kx - \omega t) = 0 \), this shows the point called node. The hydrostatic Eq. of movement is Max. in these points. In this case, the general Eq. of (2.49) is changed to Eq. (2.48). It means the time variation in pressures acting in the horizontal direction of particle movement. Now, it is considered \( \cos(kx - \omega t) = \pm 1 \) which it shows the distribution of pressure below the crest (+) or the space between two wave (-). From the Eq.2.49. one has:

\[ \Delta P = \rho g \left\{ \pm a \left[ \cosh(ka) + \tanh(kd) \sinh(ka) \right] - a \right\} \neq 0 \quad (2.50) \]

For the linear waves, the remaining pressure for the \( a \) and \( \lambda \) could be neglected. The \( z = 0 \) is for the kinetic case and \( z = h \) is for the dynamically case. Therefore, the boundary conditions regarding to the incorrectly pressure in the common surface of air and sea is contradictory.

for the deep water and \( \cosh(kd) = e^{0.5kd} \) the remaining pressure will be:

\[ P = \rho g e^{KZ} \cos(kx - \omega t) - \rho g z \quad (2.51) \]

If \( z < -d/2 \), then the pressure changing with depth is linear. It means the hydrostatic pressure variations which is expressed by Eq.2.88., below the surface, is in the interaction surface of waves. If it is divided to \( \rho g \lambda \), one has:

\[ \frac{p}{\rho g \lambda} = \frac{a}{\lambda} e^{KZ} \cos(kx - \omega t) - \frac{z}{\lambda} \]

If \( Z = z/\lambda \) and \( P = P/\rho g \lambda \) then:
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\[ P = \frac{a}{X} e^{2xz} \cos(kx - \omega t) - Z \]  

(2.52)

The given results from above Eq. supposing steep wave has been shown in Fig. 2.8. It can be seen the remaining pressure in crest and between tow wave is too little and equal: 0.006. In the depth of half of wave length, the amounts of P under crest, between tow waves and nodes are: 0.5009, 0.4996 and 0.500 respectively. Since the remaining pressure difference is too little, then the linear theory with sufficient confidence will be acceptable in the analysis of engineering problems.

![Fig. 2.8. Dimensionless distribution of a Waves’ pressure with the steepness of H/λ = 1/50](image)

2.6. Studying of Wave Encountering upon Vertical Plane Wall

When the moving wave encounters to the vertical plane wall like as caisson, is reflected and interferes with the waves which are coming into the wall. Then from this phenomenon, it is created standing waves. Height of these waves are two time of initially waves, but the length and period is the same. If the wave is broken close to the wall, the force acted to the structure depends on weather or not the wave has already broken, the wave has just broken upon the wall, the wave has been broken just before encountering to the wall. These cases have been investigated by the Begnold (1939). Hence, the analysis is only for the breaking waves that they are reflecting completely from the vertical wall.

The complete reflection has been made by adding of free surface displacements related to the moving wave to the right and its mirror picture to the left. Fig. 2.9.
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Fig.2.9. Profile of the transitional wave a) To the right b) Mirror picture c) Standing waves

The waves which are going to the right and left position has been shown as following:

\[ h^+ = a \cos(kx - at) \]  \hspace{1cm} (2.53)

\[ h^- = a \cos(kx + \omega t) \]  \hspace{1cm} (2.54)

Then, the reflecting waves will be:

\[ h = h^+ + h^- = 2a \cos(kx) \cos(\omega t) \]  \hspace{1cm} (2.55)

The above Eq. shows the standing wave will have the amplitude tow time of the coming wave. Practically, because of viscosity and other phenomenon and also elasticity and permeability of wall, the amplitude of standing waves will not be exactly tow times of coming waves. Even these phenomenon are absorbing some energy of waves. The large amplitude of standing waves is a problem for the mooring ships in the ports. Regarding to this problem, during the strong storms, the ships are taken into the sea. The velocity potential of standing waves are got by adding of potential of moving waves to the right and left side. They come from the Eq. of (2.53) and (2.54):
\[ \phi = \phi^+ + \phi^- = \frac{-\left(H\rho\right)}{\omega} \cosh(kd + kz) \frac{\cos(kx) \sin(\omega t)}{\cosh(kd)} \]  

(2.56)

By using this relation in the Bernoulli Eq., the pressure in any depth of water upon the wall will be calculated as:

\[ p = \rho \frac{\partial \phi}{\partial t} - \rho g z = \frac{H}{2} \rho g \frac{\cosh(kd + kz)}{\cosh(kd)} \cos(\omega t) - \rho g z \]  

(2.57)

Here, the wall is at \( x = 0 \). The wave force acting upon wall will be obtained by integrating of pressure Eq. upon the wet surface. In one meter of wall width, the acting force upon the wall's surface is calculated as:

\[ F(t) = \int_{-h_0}^{h_0} \rho g z \cosh(kd + k\theta) \cos(\omega t) - \rho g z \]  

(2.58)

\[ = -\rho g V_0 \cot(\omega t) - \frac{\rho g}{2} (h_0^2 - d^2) \]

Fig. 2.10. Force and Momentum by the standing wave

In this, the \( h_0 \) is displacement of free surface at \( x = 0 \) and \( V_0 \) is the vertical velocity of water particles upon the free surface adjacent to the wall.

The moment at the toe of wall depends on the time acting of force. on has:

\[ M_d(t) = -\int_{-h_0}^{h_0} \left( z + d \right) \rho g z \cosh(kd + k\theta) \cos(\omega t) - \rho g \int_{-d}^{h_0} \left( z + d \right) \cosh(kd + k\theta) \cos(\omega t) - \rho g z \]  

(2.59)
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\[
\frac{H_{p}}{k_{0}} \frac{\partial \phi}{\partial x} = \frac{\partial \psi}{\partial z} = \left( \frac{H_{p}}{\omega} \frac{gk}{\omega} \frac{\cosh(kd + kz)}{\cosh(kd)} \right) \sin(\kappa x) \sin(\omega t) \\
\frac{\partial \phi}{\partial z} = \frac{\partial \psi}{\partial x} = \left( \frac{H_{p}}{\omega} \frac{gk}{\omega} \frac{\sinh(kd + kz)}{\cosh(kd)} \right) \cos(\kappa x) \sin(\omega t)
\]

(2.61)  

(2.62)

The wall is located at \( x=0 \) and from above Eq.:

\( u = 0 \Rightarrow \kappa x = 0, -\pi, -2\pi, \ldots \) and also:

\( V = 0 \Rightarrow \kappa x = -\pi/2, -3\pi/2, \ldots \)

Therefore water particles in the standing waves will remain in the \( x = -\lambda/2, -\lambda, \ldots \)

The line of particles coincide with constant value of flow function occurs when the \( \psi \) becomes tangent of velocity vector. Therefore, Integration of Eq. (2.101) with \( z \) relation and Integration of Eq. (2.102) with \( x \) relation will express the following function:

\[
\psi = \left( \frac{H_{p}}{\omega} \frac{gk}{\omega} \frac{\sinh(kd + kz)}{\cosh(kd)} \right) \sin(\kappa x) \sin(\omega t)
\]

(2.63)

In the Fig.2.15. the zero in the flow function has been introduced in the final flows which at \( x = 0, -\lambda/2, \ldots \) and also \( z = -d \) the \( \psi = 0 \) the other lines has been shown as well.
The pressure depending to the time spending can damage a porous concrete wall in sea. If the concrete becomes porous, the water will be absorbed by the concrete and disappeared. Finally it will destroy the free surface of concrete. The hydraulic gradient of pressure inside the wall which is creating by frequently attacking of waves will cause to damage. In the Fig.2.12. the damage of wall has been shown.

Fig.2.12. Manner of damage in the porous concrete wall

It has to be explained the above analysis has performed only for nonbreaking waves. If the standing waves resulting of reflecting and attacking waves are prevented to be spilled, the depth of $d$ at the toe of wall must be at least $1.5H$.

Practically, in the constructions that there would be possibility of damage due to spilling of standing waves, the depth of $2H$ will be considered. Fig.2.13. shows the pressure changing upon the vertical wall in the case of nonbreaking waves.

Fig.2.13. Graph of pressure changes upon the wall
If the encountering of waves with the wall would not be normally and be under an angle, their reflections will be like as the light reflections. From combining of initially waves and reflecting waves will be established the complicated movement on water surface which are called Corrugation.

2.7. The Waves upon the Flat Steep Wall

It is considered a noncircular shallow wave upon the flat impermeable steep wall which could have been a coast. When the flow is assumed noncircular, both final energy and flow energy can vary in depth. The flow energy which is equal with transitional power is expressed by the following phrase:

\[ E = \frac{\rho a^2 g c_g}{2} = \frac{\rho H^2 g c_g}{4} \]  

(2.64)

Which the \( c_g \) is the group velocity. When the group velocity becomes close to the celerity, the water depth increases and then, the wave height should be calculated by following relation:

\[ H = 2a = k / (c_g)^{0.5} \]

The \( k \) is the wave number and is constant, and in deep water the property of waves are expressed based on subscript of zero.

\[ K = H_0 (c_g)^{1/2} \]

By using of Eq. (2.41), one has:

\[ H = H_0 \left( \frac{C_0}{C_g} \right)^{1/2} = H_0 \left\{ \frac{C_0}{C(1 + 2kd/Sinh(2kd))} \right\}^{1/2} \]  

(2.65)

It has to be noticed that both wavelength and wave number varies in depth. In the Fig. 2.14, the proportion of wave length's variation and also the \( d / \lambda_0 \) function has been shown.
The steepness of $d/\lambda_0$ decreases. This phenomenon was observed by Wiegel (1959) experimentally. When $d/\lambda_0 \to 0$, the $h/h_0$ will be close to one amount. By using of breaking criterion and applying of deep water results in the Eq. of (2.24) and (2.105) after combining of results, the following consequence will be found:

$$H_b = h_0 \left( \frac{C_n}{2C_b} \right)^{1/2} = h_0 \left( \frac{C_n}{2\sqrt{gd_b}} \right)^{1/2}$$

Subscript of $b$ shows the breaking case. The wave height in the breaking case will be:

$$H_b = 2d_b = h_0^{4/5} \left( \frac{C_n}{\sqrt{2g}} \right)^{2/5}$$

Then, the wave height and depth of breaking has been introduced by the linear theory which it depends on the deep water property. Now, one impermeable coast under angle of $\alpha$ as shown in the Fig.2.15. is considered.
If the interaction of coast line and still water level becomes the position of the coordinate axis, the depth of water in any position is:

\[ d = x \tan \alpha \]  

(2.67)

From the Eq. (2.67), the breaking wave case will be:

\[ x_b = \frac{H_n^{4/5}}{2 \tan(\alpha)} \left( \frac{C_n}{\sqrt{2g}} \right)^{2/5} \]  

(2.68)

When the waves are coming to the coast line under angle between coast line and wave crest, the refraction phenomenon will be created. The Fig.2.16. shows the shape from this phenomenon in shallow waves. The wave velocity decreases over depth and the coming waves are going to receive the initial waves. This causes the wave length becomes shorter and the crest of wave becomes inclined. In limitation when \( h \to 0 \), The wave’s front becomes in the line as same as the coastal line. The refraction of waves in water is similar to the refraction in acoustic waves in Snell Law:

\[ \frac{\sin \beta}{\sin \beta_0} = \frac{C}{C_0} \]  

(2.69)

Which \( \beta \) is the angle between shore line and wave’s front.
The tangent of angles for drown lines, is called celerity or phase speed that is vertical on orthogonal. The *Micke* has shown the distance which has been spent by the wave upon the step wall, in top and below of Still Water Level, is calculated from following Eq.:

\[ \Delta = \frac{H}{\sin \alpha (\pi/2 \alpha)^{0.5}} \]

When the wave passes through the intermediate surface, its speed becomes Max. which ic equal to:

\[ V = \frac{\pi H}{\sin \alpha (g/\pi \lambda)^{0.5}} \]

2.8. Studying of Sea Bottom’s effects over property of Wave in Shallow Water

As the wave becomes close to the shallow water, the effects of bottom upon wave increases and the wave specification will vary by the phenomenon like as Reflection, Absorption, Wave set up, Diffraction, Wave breaking and Refraction. Although these factors have a influence on wave coming from deep water to shallow water, but the relative importance of them is completely variable. Generally, in a ordinary coast, the refraction and diffraction are the most important factors and the reflection and absorption have the least influence.
Reflection:
The Reflection is created when the wave energy is reflected by an obstacle. The most Reflection is related to the Vertical Wall. For the ordinary coasts with the acceptable slope, the reflection could be neglected.

Absorption:
The loss of energy due to friction of bottom and penetrating to it is called absorption which is negligible, unless in the cases which the wave groups spend a lot of distance in shallow water.

Wave Run Up:
The wave Run Up takes place when the coast affects upon the apparent speed of energy over to the phase speed of $c$. This phenomenon can cause either extension or compression of energy. Therefore, the amplitude of waves are increased or decreased. For finding of Run Up effect, it can be used by Wave Run Up Coefficient:

One has:
$$K_s = \left[ \frac{C_0}{C} \left[ 1 + \frac{4\pi d}{\sinh(4\pi d)} \right] \right]^{1/2}$$

$c_0 =$ Wave Speed in Deep Water
$c =$ Wave Speed in Shallow water
$d =$ Depth of Water
$\lambda =$ Wave Length

Diffraction:
The Diffraction is one of the factors which varies the wave specifications. There are two types of Diffraction. The Diffraction due to the wave encountering to an obstacle lying under the water, and the other one is the diffraction producing of obstacle which they come out of water level.

Refraction:
It is due to the dependent of apparent wave speed to the depth of water and it is coming when the wave touches the sea bottom. As wave moves toward coast, the wave velocity decreases. This phenomenon has been already expressed. The Fig.2.17. shows how the waves are changed in direction due to refraction.

Finally, for calculating of wave height in shallow waters, the wanted wave height must be multiplied at the factors of Run up and Refraction in deep water.
Refraction Along a Straight Beach with Parallel Bottom Contours

(a)

Refraction by a Submarine Ridge (a) and Submarine Canyon (b)

(b)

Refraction Along an Irregular Shoreline

Fig. 2.17. Refraction of waves in the coast lines
Chapter 3

Designing of Different parts of Ruble Mound Breakwaters
3.1. The Ruble Mound Breakwaters

The breakwaters and caissons are designed mostly to protect of ports and coasts. These types of constructions are designed one core made by gravel, broken stones and rubble. It is also consists of several intermediate layers connected to the top protection layer. Damage of these types of structures can be due to removing and damaging of the armor layers, overtopping of waves, to be washed out of toe materials, loss of core materials and sometimes it can be due to difficulties related to the their foundations.

The armor layers pieces in the top layer are mostly considered from the large pieces of stones or prefabricated concrete large units which are made in the different shapes. Very often, the design formulas for designing of this type of constructions only has been given for the protection layers and the other layers are considered regarding to the protection layers. Some of these Formulas are as following:

1. Hadson Formula (1958):

\[
W = \frac{\gamma_s H^3}{KD (\gamma_s - \gamma_w)^3 \cot \alpha} \frac{Cota y_w}{\gamma w} 
\]

One has:

H : Wave Height
\(\gamma_s\,\gamma_w\) : Density of Stones and Water respectively
\(\alpha\) : Slope of Armor Layer
W : Weight of Stones Units

The Hadson Formula can be used for any type of structures that the slope’s varying will be between 1.5 to 1 till 5 to 1. The values of \(KD\) are related to the sharpness of blocks’ edges and degree of consistency as shown in Tab.3.1.

<table>
<thead>
<tr>
<th>Armor Units</th>
<th>Number of Units</th>
<th>Breaking Wave</th>
<th>Nonbreaking Wave</th>
<th>Breaking Wave</th>
<th>Nonbreaking Wave</th>
<th>Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth Minestone</td>
<td>2</td>
<td>1.7</td>
<td>2.4</td>
<td>1.1</td>
<td>1.9</td>
<td>3-1.5</td>
</tr>
<tr>
<td>Smooth Minestone</td>
<td>&gt;3</td>
<td>1.6</td>
<td>3.2</td>
<td>1.4</td>
<td>2.3</td>
<td>3-1.5</td>
</tr>
<tr>
<td>Coarse and Broken Minestone</td>
<td>2</td>
<td>2</td>
<td>4</td>
<td>1.9</td>
<td>3.2</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>&gt;3</td>
<td>2.2</td>
<td>4.5</td>
<td>2.1</td>
<td>4.2</td>
<td>3-1.5</td>
</tr>
<tr>
<td>Tripod</td>
<td>2</td>
<td>9</td>
<td>10</td>
<td>8.3</td>
<td>9</td>
<td>1.5</td>
</tr>
<tr>
<td>Dolloss</td>
<td>2</td>
<td>15.8</td>
<td>31.8</td>
<td>8</td>
<td>16</td>
<td>2</td>
</tr>
<tr>
<td>Tetrapode</td>
<td>2</td>
<td>8</td>
<td>8</td>
<td>5</td>
<td>6</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Table.3.1. Values of KD in Hadson Formula without damage and little overtopping
The period of wave and irregularity of waves is not taken into account in Hadson Formula.

2. Iribarren Formula (1938):

$$W = \frac{\gamma s K H^3}{(\gamma s -1)^3 (\cos \alpha - \sin \alpha)^3}$$

(3.2)

One has:

$H_b$: Wave Height at breaking point
$\alpha$: Angle of Slope

In this formula $k = 0.015$ for the any stones and $k = 0.019$ for concrete blocks.

3. Hadson - Iribarren Formula (1951):

$$W = \frac{\gamma s K' \tan^3 \phi \cdot H^3}{(\gamma s -1)^3 (\tan \phi \cdot \cos \alpha - \sin \alpha)^3}$$

(3.3)

One has:

$\phi$: Internal Friction of Stones or Blocks
$H$: Wave Height
$k'$: will be as shown in Table 3.2.

<table>
<thead>
<tr>
<th>$K'$</th>
<th>Slope</th>
<th>$K'$</th>
<th>Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0365</td>
<td>1.3</td>
<td>0.0035</td>
<td>1:1.25</td>
</tr>
<tr>
<td>0.0325</td>
<td>1.4</td>
<td>0.0083</td>
<td>1:1.5</td>
</tr>
<tr>
<td>0.03</td>
<td>1.5</td>
<td>0.0175</td>
<td>1.2</td>
</tr>
<tr>
<td>0.0285</td>
<td>1:2.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.2. Values of $K'$ with various slopes


$$W = \frac{\gamma s}{(\gamma s -1)^3} KH^3 \left( \frac{1}{\cot \phi - 0.8} -0.15 \right)$$

(3.4)

For the natural stones $k = 0.25$
For concrete blocks $k = 0.12$

In the above formula, mostly the effects of many parameters like as types of wave breaking on the slope, wave impact, wave inclining, permeability of core, roughness of corners and consistency of blocks have not been taken in to account. Although in general formula for this purpose, all of these parameters should be considered as well,
but, considering of whole of these parameters will be so difficult in the formula. Therefore, it would be better to take in to account most important parameters.

3.2. Permeability and Frictional Force Effect

When a wave arrives to the surface, related to the permeability of surface, part of it or whole of wave will be reflected. If the surface is assumed vertical and porous, the part of flow will enter into obstacle and another part of it will be reflected. Upon the steep and impermeable surface, the wave is running up of the surface. When the slope is permeable, the wave run up will be reduced. Now, this effect is considered for a wave in porous section with porosity of $n_1$ which is coming to be close to another section with porosity of $n_2$.

It is assumed that flow speed will be zero in the second porous section, the wave height of $H$ and approaching depth of $d$. If the wave speed is $c$, one has:

$$V \left( H + d \right) = n_1 C_1 (H1 - H) + n_2 C_2 H_1$$  \hspace{1cm} (3.5)

$H_1$: Wave Height after encountering to porous section

$c_1, c_2$: Velocity in the porous sections

By using consistency and energy Eq. , one has:

$$\left( C - \frac{V}{n_1} \right)^2 + d + H = \frac{C}{2g} + d$$  \hspace{1cm} (3.6)

$$n_1 CV = (n_1 C - V) (d+H)$$  \hspace{1cm} (3.7)

After finding of $V$ from (3.7) Eq. and replacing of it in (3.5) , one has:

$$H_1 = \frac{(1 + C/c1) H}{1 + n_2 c_2/n_1 c_1} \approx \frac{2H}{1 + n_2 n_1}$$  \hspace{1cm} (3.8)
Therefore, if the wave with wave height of $H$ encounters to permeable section with porosity of 0.4, the amplitude of reflected wave will be $1.43H$. Practically, this wave will be disappeared because of gravel existence. It has been found that waves are disappeared regarding to the frictional force. The following relation shows rule:

$$\frac{H_n}{H_1} = \frac{1}{1 + kx \frac{H_0}{2dD}}$$

Equation (3.9)

One has:

$H_1$ : Wave Amplitude in porous section
D : Dimension of Stones
$d = y$ : Water Depth
$k = 2$ : Friction Factor of Stones
$x$ : Distance

The Fig. 3.2. shows the mentioned relations graphically.

![Graph showing amortization of reflected wave in the rubble mound](image)

Fig. 3.2. Amortization of reflected wave in the rubble mound

The stones decrease the wave amplitude which pass through it. The relations about decreasing upon frictional non-porous surface will be:

$$\frac{H}{H_0} = 1/\sqrt{1 + \frac{kx H_0^2}{2Dd^2}}$$
3.3. Permeability of sublayer of Armor Layers

If there is the breakwater with permeable core, the water will penetrate into the core and will be disappeared. For the waves with the large period, much amount of water will penetrate into the core and the pheriatric line will be down. In these cases the wave force will be decreased and stability of slope increase. But, in the impermeable core, the water movement inside the to layers produces the large force under it. Therefore, the water flowing inside armor layer will increase the unstability of mound. When the breakwater is homogeneous, it will cause more reduction of wave force and more stability of construction. The permeability of the core affects both on the wave run up upon the slope and upon the dimensions of the stones. This matter is discussed as following.

Effect of Sublayer's Permeability:

Van der Meer (1987) and Wiebe (1991) have been working as following.

Firstly, it is explained a critical surf parameter which is depends on the permeability and angle of slope.

\[
A_m = (6.2 \times P^{0.31} \sqrt{\tan \alpha})^{1/(p + 0.5)}
\]  

(3.9)

and the another relation is:

\[
A_m = \frac{\tan \alpha}{am} \quad am = \frac{HSL}{(g/2\pi) Tm^2}
\]
When $A_m > A_{mi}$ there will be Surging Waves:

$$\frac{H_s}{D_{50}} = P^{-0.13} \left( \frac{S}{V_N} \right)^{0.2} \sqrt{\cot \alpha} \, A_m^{P}.$$  \hspace{1cm} (3.11)

When $A_m \leq A_{mi}$ there will be Plunging Waves.

$$\left( \frac{H_s}{D_{50}} \right) \times V_A m = 6.2 \, P^{0.18} \left( \frac{S}{V_N} \right)^{0.2}$$ \hspace{1cm} (3.12)

In above relations $\Delta = (\gamma_s - \gamma_f) / \gamma_f$ and $P$ is the permeability of sublayer.

$D_{50}$ : Average Diameters of Stones

$T_m$ : Average Wave Period

If the $H_s$ is the Significant Wave Height, the $H_{SL}$ will be:

$$T_m = 0.5 + 2.8 \sqrt{H_S}$$ \hspace{1cm} (3.13)

$$H_{SL} = \gamma' br . d$$ \hspace{1cm} (3.14)

$$\gamma' br = B \left(1 + \frac{A d}{L D} \right)^{-1}$$ \hspace{1cm} (3.15)

One has:

$d = d_m + h$

$h$ = Height of Wave Crest till Mean Water Level

$d_m$ = Mean Water Height

$LD = (g/2\pi) \cdot (T_m)^2$

$T_m$ = Mean Wave Period

$g$ = Gravity

$$A = 6.96 \left[1 - cxP (-19 \beta)\right]$$ \hspace{1cm} (3.16)

$$B = 1.56 \left[1 + cxP (-195 \beta)\right]^{-1}$$ \hspace{1cm} (3.17)

Which $A$ and $B$ are the constants and the $\beta$ is the angle of beach slope. The Fig.3.4. shows the Rubble Mound breakwater.
In the above relation \( N \) is the number of wave attacking to the slope and \( S \) is called damage level.

\[
S = \frac{(AR)}{D_{50}^2}
\]  
(3.18)

\( AR \) is the erosion surface that has been indicated in Fig. 3.4.

For the different slopes the Table 3.3. shows the highest and lowest damage level.

<table>
<thead>
<tr>
<th>Damage of slope</th>
<th>Damage Level of S</th>
<th>Angle of Slope (Cotα)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>2</td>
<td>1.5</td>
</tr>
<tr>
<td>8</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>12</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>17</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>17</td>
<td>3</td>
<td>5</td>
</tr>
</tbody>
</table>

Table 3.3. Various damage levels

For the stability case, the \( S \) should be between 1 and 3 and the damage of slope will be occurred when the \( S > 10 \) which it is avoided.

The following Eq. shows the relation between \( N \) and \( S \):

\[
S(N) / S(5000) = 1.3[1- exP(-3 \times 10^4 N)]
\]  
(3.19)

In the above Eq., having type of wave \( (A_m) \), angle of slope \((α)\), damage level \((S)\), number of waves \((N)\), density of stones over to density of water \((Δ)\), significant wave height and permeability \((P)\), the mean Diameter of stones could be calculated. What is obtained from all of above mentioned relations, is that the \( D_{50} \) or dimension of stones or blocks must be enough big that they can not be removed by the waves.
The stability of stone blocks depends on their shape, size, weight, the method of stones’ dropping, angle of slope, drainage, cohesion of stones to the sub soil, wave specifications like as height, period, shape, amplitude, particles velocity, incidence angle to slope and amount of air inside he water. A simple analysis could express the stability of armor layers noticing the Fig.3.5.

In tow cases it is possible removing of stones blocks, firstly when the wave is running up and secondly when the wave is running down. In the last case the weight of stone will help the instability. For both cases it is assumed the waves’ forces affect in parallel to the slope. The Eq. which is written for stability of stones blocks during the wave run up is:

\[ F_w \cos \alpha - F_u + (F_d - F_w \sin \alpha) \tan \phi = 0 \]  

(3.20)

Which \( F_w \) is the buoyant weight of stones.

\[ F_w = K_1 (S_r - 1) \gamma_f D^3 \]  

(3.21)

The frictional force (\( F_d \)) and upward force (\( F_u \)) is as following:

\[ F_d = k_1 H \gamma_f D^2 \]  

(3.22)

\[ F_u = k_3 H \gamma_f D^2 \]  

(3.23)

In the mentioned relations, one has:

\( \phi \): Internal Friction Angle

\( \gamma_f \): Specific Weight of Water
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Sr : Relative Density of Stones
H : Wave Height
K₁ , K₂ , K₃ : Constants which are related to the purity of stones.

After replacement in the main Eq. , one has:

\[ D = \frac{K_r H}{(S_r -1) \cos \alpha (1- \tan \alpha \tan \phi)} \]  (3.24)

Which Kᵣ is the constant value that expresses the position of stones to each other. When the wave is running down, the water comes out from the blocks. In this case \( \tan \phi = 0 \), then the nominal diameters of stones is:

\[ D = \frac{K_s H}{(S_r -1) \cos \alpha} \]  (3.25)

Kₙ : value related to the stones blocks and other armors

Values of Kₙ and Kᵣ can be increased till %10 for the strong waves and decreases till %10 for the still waves. It has been proved for the mean slopes i.e. 1:2 or less, the wave running down will determine the dimension of stones and for the steep slopes the wave run up will be. The USBR by the different tests has submitted another formula which has been improved by the Tailor as follows:

\[ D = K_h H^\frac{1}{3} (S_r -1) (\cot \alpha)^{1/3} \]  (3.26)

e for the small waves i.e. H < 1.5 (m) will be 1. and for the big waves will be decreased till 0.67. The values of Kᵣ , Kₙ , Kₕ have been shown in the Table.3.4. for the armor layer without damaging.

<table>
<thead>
<tr>
<th>Type of armors</th>
<th>Kᵣ</th>
<th>Kₙ</th>
<th>Kₕ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rounded Gravel</td>
<td>0.58</td>
<td>0.37</td>
<td>1.00</td>
</tr>
<tr>
<td>Angular Gravel</td>
<td>0.55</td>
<td>0.35</td>
<td>0.90</td>
</tr>
<tr>
<td>Hand set Stones</td>
<td>0.44</td>
<td>0.28</td>
<td>0.75</td>
</tr>
<tr>
<td>Concrete Blocks</td>
<td>0.35</td>
<td>0.30</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Table.3.4. Values of Kᵣ , Kₙ , Kₕ

The stability formulas have to consider two cases as already expressed. Firstly, the waves affect upon the slope during run up. Secondly, the waves affect upon the slope during running down. After extensive studying, it has been proved that the behavior of armor layers depends upon the permeability of sublayer and core. Therefore the permeability and impermeability has to be taken in to account. Also, the internal friction angle and consistency degree should be considered. In the formula for stability, it must be considered the most critical case where it has to be stable.

The Hedar (1986) has considered theses conditions as well and calculated the weight of blocks as:
One has:

\[ W = \frac{\pi}{6} \gamma_s K^3 \] (3.27)

\[ K : \text{Diameter of spherical block equal of block weight} \]

The different values of \( K \) are found as follow:

**Case A: Run up**

**Case A-a: Sublayer is permeable**

\[ K = \frac{0.33 (d_b + 0.7 H_b) (\tan \phi + 2)}{(\gamma_s - 1) (3.6 - 1) \cos \alpha (\tan \phi + \tan \alpha)} \frac{\gamma f}{e^{4 \tan \beta}} \] (3.28)

**Case A-b: Sublayer is impermeable**

\[ K = \frac{0.41 (d_b + 0.7 H_b) (\tan \phi + 2)}{(\gamma_s - 1) (3.3 - 1) \cos \alpha (\tan \phi + \tan \alpha)} \frac{\gamma f}{e^{4 \tan \beta}} \] (3.29)

In the mentioned formulas one has:

\[ \beta = \alpha + (\phi - 48^\circ) \]

**Case B: Running down**

**Case B-a: Sublayer is permeable**

\[ K = \frac{(D_b + 0.7 H_b) (\tan \phi + 2)}{(\gamma_s - 1) (c^{4 \phi} + 13.7) \cos \alpha (\tan \phi - \tan \alpha)} \frac{\gamma f}{e^{4 \tan \beta}} \] (3.30)

**Case B-b: Sublayer is impermeable**

\[ K = \frac{1.6(D_b + 0.7 H_b) (\tan \phi + 2)}{(\gamma_s - 1) (c^{4 \phi} + 16.5) \cos \alpha (\tan \phi - \tan \alpha)} \frac{\gamma f}{e^{4 \tan \beta}} \] (3.31)

In the mentioned formulas, for cases B one has:

\[ \beta = \alpha - (\phi - 48^\circ) \]

Which:

\( d_b \): water depth at breaking point

\( H_b \): wave height in breaking point

The different tests have been shown if the sublayer is impermeable, the weight of blocks must be increased. In the other hand, the being of tiny particles in the core or
caissons or any other concrete construction will be caused the high weight of blocks. The formula for impermeable sublayer could be used when the armor layers would be located on the impermeable surface or impermeable structure.

The condition for using of impermeable sublayer formula is that the sublayer has to have porosity of %40. The internal friction angle of blocks depends on the property of the blocks that it can be varied. If the angle of slope is equal $$\phi$$, the slope will be unstable and the value of ($$\phi - \alpha$$) is the sign of stability resistance against the wave forces. The angle of $$\phi$$ is one of the specification of consistency degree of blocks in armor layer and the high value of $$\phi$$ shows the blocks are so connected with together and have been locked. Also the angle of $$\phi$$ could be increased artificially by placing of blocks upon together one by one which the free space between the blocks becomes minimum. The consistency of blocks along the time will be reduced by factors like as shaking by the impacting of waves, erosion by waves, damaging by different settlement which is produced in construction and sinking inside sea bed. Although the lying of blocks one by one upon together increases the angle of internal friction as mentioned above, this case causes the armor layers have the more consistency comparison with core layer. Therefore, there would be suddenly damage in the armor layer. The internal friction angle ($$\phi$$) has a large effect on the above formulas. This value ($$\phi$$) can be available by using of Graph shown in Fig.3.6, which it gives the different value of $$\phi$$ noticing the roughness of corners and stones. In the special cases, the value of $$\phi$$ can be increased till 1.05 for the stones with corners.

![Fig.3.6. Stability angle of rubble mound](image-url)

Although in the Hedari’s design formulas for the weight of armor blocks has not been used of permeability parameter, but this parameter has been consider inside value of K and it is more useful comparison with other formula, because it can easily calculate the weight of armor layers.
3.4. Wave Run Up

When the wave encounter to an obstacle, it is either reflected or absorbed. One vertical impermeable and plane obstacle reflects the wave which its amplitude is tow time of main wave. The approximately horizontal surface which it is coarse and permeable will absorb the wave.

![Wave Run Up Diagram](image)

**Fig.3.7. Schematized pattern of wave run up**

When the wave is running up upon the slope it loses its energy. The wave run up, loss of energy, and reflection value depends on both specification of wave and slope surface. The rubbles with the coarse angular materials will have a more effect to loss of energy, but it would be more unstable. In the other hand, a plane surface will cause the big run up. Then it will need to the more free distance. Therefore, the wave run up and possibility of overtopping should be considered in design process.

Iribarren (1974) for reaching of confidence level for that wave breaks upon the slope with angle of slope ($\alpha$) in a impermeable surface. The following relation will show:

$$
tan\alpha \leq \frac{8}{T} (H/g_T)^{1/2} 
$$

(3.32)

One has:

- $H$: wave height at toe of slope
- $T$: wave period

If the steepness of slope becomes greater than above formula, the wave will be reflected and for the steepness less than it, the wave will break upon the slope. For the gentle slopes, the wave run up for breaking waves is the vertical height above the Still Water Level as following:

$$
R_s/h_0 = 0.41 \tan\alpha (H_0/g_T^2)^{0.5}
$$

(3.33)

One has:
H₀ : wave height at deep water
T : wave period
Rₐ : vertical height from S.W.L.

The wave run up upon the rubble mound slope will be decreased if the porosity and coarseness reduces. In the porous surface the previous relation will be changed to:

\[ \frac{R_s}{H_0} = 0.41 \tan \alpha \left( \frac{H_0}{gT} \right)^{0.5} \times r \]  

(3.34)

Which \( r \) is the reduction factor calculated as experimental and its value has been indicated in Table.3.5.

<table>
<thead>
<tr>
<th>Type of Cover</th>
<th>Value of ( r )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat and Impermeable</td>
<td>1</td>
</tr>
<tr>
<td>Concrete Slab</td>
<td>0.9</td>
</tr>
<tr>
<td>Concrete Blocks</td>
<td>0.85 - 0.9</td>
</tr>
<tr>
<td>Grass on the Clay</td>
<td>0.85 - 0.9</td>
</tr>
<tr>
<td>One Layer of Impermeable Stones</td>
<td>0.8</td>
</tr>
<tr>
<td>Accidental Rubbles</td>
<td>0.5 - 0.8</td>
</tr>
<tr>
<td>More than Two Layer of Stones</td>
<td>0.5</td>
</tr>
<tr>
<td>Concrete Tetrapodé Blocks</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table.3.5. Values of reduction factor of \( r \).

The calculation process of wave run up \( R_s \) is as following. The wave height in deep water (\( H_0 \)), wave period and depth of water (\( d \)) should be found. After that, wave period (\( T \)), wave length (\( L \)) will be calculated noticing of depth and celerity by the Graph shown in Fig.3.8.
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Then the wave length at deep water ($L_0$) is obtained:

$$L_0 = \frac{gT^2}{2\pi}$$  \hspace{1cm} (3.35)

Then, after determining of above values, other unknowns will be found by following Graph indicated in Fig. 3.10.
After getting $H_0$ and $L_0$ and other unknowns, the wave height and water depth in the breaking point could be known:

\[
\begin{align*}
\frac{H_b}{H_0} &= 0.38 \left( \frac{H_0}{L_0} \right)^{-1/3} \\
\frac{d^b}{L_0} &= 0.49 \left( \frac{H_0}{L_0} \right)^{-1/3}
\end{align*}
\]

Finally, the $R_s$ will be found by the following formula as already indicated:

\[
\frac{R_s}{H_0} = 0.41 \tan \alpha \left( \frac{H_0}{gT^2} \right)^{0.5} \times r
\]

The other solution which is more simple has been presented by the Wiebe de Haan (1991). One has:

\[
\begin{align*}
HSL &= \gamma br . d, \quad d = dm + h \\
\gamma br &= B \left( 1 + \frac{Ad}{LD} \right)^{-1} \\
A &= 6.96 \left[ 1 - \exp \left( -19 \beta \right) \right] \\
B &= 1.56 \left[ 1 + \exp \left( -19.5 \beta \right) \right]^{-1}
\end{align*}
\]

After that the $R_s / H_{SL}$ is calculated by the following formulas noticing the wave type:

\[
\begin{align*}
R_s / H_{SL} &= 0.72 A_m & 0.5 < A_m \leq 2.8 \\
R_s / H_{SL} &= 0.88 A_m & 1.5 < A_m \leq 2.8 \\
R_s / H_{SL} &= 1.35 & A_m > 2.8
\end{align*}
\]

In the mentioned formulas, the $A_m$ is known already. With determining of $R_s$ and knowing $H_s$, the value of $F$ or Freeboard needed for non overtopping case will be found.

\[
H_s / H_{SL} = 0.478 \left( 1.16 - \frac{F}{R_s} \right)
\]

The value of $F$ will be added to wave height which it is above the S.W.L.. Then the result value will give the height of crest in the breakwater. The other solution is regarding to the Van der Meer (1991). In this formula, firstly $A_m$ must be found. The run up for the smooth slopes ($A_m$) : $R_s / H_{SL} = 1.5$

When the $0.5 < A_m < 2$ , the run up will be:

\[
\begin{align*}
R_s / H_s &= a A_m & A_m \leq 1.5 \\
R_s / H_{SL} &= b A_m & 1.5 < A_m \leq 2.8 \\
R_s / H_{SL} &= d & A_m \geq 2.8
\end{align*}
\]

The values of $a, b, c, d$ for the different level of permeable slope will be as Table.3.6.
Table 3.6.

Comparing the run up relation for the smooth slopes and rubble rocky slopes, it is clear that the run up upon the rocky rubble mound slopes is approximately 50% of the smooth slopes and practically this value will be between 0.5 - 0.6 of run up upon the smooth slopes. After calculating of $R_e$ from the previous method, the freeboard of $F$ will be found.

$$\frac{H_r}{H_{SL}} = 0.478 \left( 1.16 - \frac{F}{R_s} \right)$$

Finally, the value of $F$ will be added to wave height above the S.W.L. which it gives the height of crest in breakwater.

3.5. Thickness of Armor Layer

After determining of weight of blocks or stones, now the determination of thickness is studied. If the protection layers are established in $n$ layers with thickness of $t_1$, the value of $t$ will be:

$$t = n k_D \left( \frac{W}{\rho_s} \right)^{1/3}$$  \hspace{1cm} (3.41)

Which $\rho_s$: density of stones

Sometimes $D_{50}$ is called nominal diameter of stones and expressed as following:

$$D_{50} = \left( \frac{W}{\rho_s} \right)^{1/3}$$  \hspace{1cm} (3.42)

Then the thickness of $t$ is obtained as:

$$t = n k_D (D_{50})^{1/3}$$  \hspace{1cm} (3.43)

Which $K'_{D}$ is the dimensionless factor for the layer and the number of layers ($N$) for protecting of determine area of ($A$) are found:

$$N = An \left( 1 - \frac{P}{100} \right) \left( \frac{D_{50}}{W} \right)^{2/3}$$  \hspace{1cm} (3.44)
In the mentioned formula, P is the permeability in percentage. The different value of P have been indicated in Table 3.7.

<table>
<thead>
<tr>
<th>Armor Blocks</th>
<th>Number of Layer n</th>
<th>K'D</th>
<th>Porosity %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth Minestone</td>
<td>2</td>
<td>1.02</td>
<td>38</td>
</tr>
<tr>
<td>Coarse Minestone</td>
<td>2</td>
<td>1.00</td>
<td>37</td>
</tr>
<tr>
<td>Concrete Tripod</td>
<td>2</td>
<td>1.02</td>
<td>54</td>
</tr>
<tr>
<td>Concrete Dolloss</td>
<td>2</td>
<td>0.94</td>
<td>56</td>
</tr>
<tr>
<td>Coarse Minestone</td>
<td>&gt;3</td>
<td>1.00</td>
<td>40</td>
</tr>
<tr>
<td>Concrete Tetrapode</td>
<td>2</td>
<td>1.10</td>
<td>50</td>
</tr>
</tbody>
</table>

Table 3.7. Coefficient of K'D and porosity for various armor blocks.

Van der Meer after determining of stones weight, presented following formula to calculate of thickness:

\[
t = n \times k\Delta \times D_{50}
\]  
(3.45)

Generally, \( n = 2 \) has been considered for armor layer and \( k\Delta = 1.02 \) is called leakage coefficient. Comparing of tow formula about thickness shows the first formula focuses on porosity of stones during the setting, also for different types of blocks has been considered particular K_D. In some cases, the high value of overtopping can damage the structure. For preventing of mentioned phenomenon, the width of crest and armor layers which is related to the overtopping criterion and structure method, should be more than the Min. b considering as following:

\[
b_{\text{min}} = 3 k' (\frac{W}{D_5})^{\frac{1}{3}} = 3 k' (D_{50})^{\frac{1}{3}}
\]  
(3.46)

As it is shown from the above formula, the width of crest is 3 times of armor layers' thickness.

After determining of all design aspects by design formula the main shape of the construction will be as shown in Fig. 3.11.
As it is clear from the above figure, the second layer of filter continues till filter of toe layer and it has been considered a berm which protects the toe against to be washed.

3.6. Scouring Berm

As it is expected, after some time since using of structure, its toe may be washed out. Therefore, this case could be prevented by considering a protecting berm against the erosion.

For calculating of width of berm and other values shown in Fig.3.12., following stages are applied.

![Fig.3.12. Schematized pattern of scouring berm](image)

For calculating of $L$ which is the length the armor layers must be extended, depends on the type of classification for used stones in armor layers. For this purpose, in addition having of nominal diameter of $D_{50}$, the diameters of $D_{85}$ and $D_{15}$ for used stones will be needed.

\[
\frac{L}{D_{50}} = K_L + 11.05 \frac{D_{85}}{D_{15}} - 1.47 \left( \frac{D_{85}}{D_{15}} \right)^2 \tag{3.47}
\]

Then the $K_L$ will be found as:

\[
K_L = 26.3 + 1.15 \left[ \frac{D_{85}}{D_{15}} \right]^{219} \tag{3.48}
\]

The volume of stones per unit length of armor layers is calculated as:

\[
\frac{A}{D_{50}^2} = KA + 194.2 \frac{D_{85}}{D_{15}} - 26.8 \left( \frac{D_{85}}{D_{15}} \right)^2 \tag{3.49}
\]
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\[ KA = 148 + 30 \left( \frac{Hs}{y_t} - 1 \right) D_{50}^{1.91} \] (3.50)

And the width of the berm will be:

\[ \frac{B}{D_{50}} = KB + 7.52 \left( \frac{D_{as}}{D_{15}} \right)^2 + 6.12 \, P_R \] (3.51)

\[ K_B = -10.4 + 0.51 \left( \frac{Hs}{y_t} - 1 \right) D_{50}^{2.5} \] (3.52)

The only unknown parameter in the above formula is the \( P_R \) which is the percentage of stones' Roundness. This value only has effect on the width of berm. In the other cases, this value (\( P_R \)) will be negligible when the stones are floating inside of water.

3.7. Design of Suitable Filter

Design of classified filter for preventing of entrance of small particles into big stones have been more attractive. Generally, filters are applied in unavailable parts. And the possibility of inspection is very low or there would be non. For instance, after spending of time the soil with tiny materials is settled and the stones are settled down. The later case will be the most stable situation for it. But if the design of filter is not correct, the space between the stones can be filled by the small grains. The erosion can create the hole in filter layer. Stability against the water leakage and drainage specification depends on the dimension and distribution of filter classification and sublayer materials. The size of natural particles has a limitation. The different classifications can be combined with together and created suitable materials. By drawing of percentages of particles smaller than the special sieve in logarithmic scale, the classification curve will be produced.

Now, tow criterion for recognizing of particle size are introduced.

\[ d_{85} = \] which is 85 percent of sieve weight less than determined diameter
\[ d_{15} = \] which is the diameter that only 15% of materials is smaller than it.

These tow diameter determine the classification of a natural material and are used in design of filter. The filter layers are designed based on that the large layers are getting away gradually from the layers made of tiny materials. This type of filter is called inverse filter. The number of layers depends on diameter of coarse and tiny particles. The suitable filter has to have following conditions:

3.7.1. Stability Criterion

For preventing of erosion of sublayer materials into filter layer, the smaller particles of filter should be enough small that they can prevent the influence of greater particles. This subject has been shown in Fig.3.13.

It is noticeable that from free space among three equal sphere contacted together, the spheres just only can throw from this space which their diameter is smaller than 1/6.5 times of diameters of bigger spheres. For the uniform natural materials the stability criterion will be:

\[ s = \text{sublayer materials} \]
\[ f = \text{filter materials} \]
3.7.2. Grains Separation
The curves of layers' classification should be approximately parallel with together and for making minimum of grains separation the following criterion should be considered.

\[
\frac{d_{50}(f)}{d_{50}(s)} < 25
\]  

(3.54)

This relation can be neglected for plastic clay.

3.7.3. Permeability Criterion
Permeability of filter layer has to be so enough which the hydraulic gradient could be neglected compared with available value in sublayer materials. Permeability of soils depends on the smaller grains and \( d_{15} \) can be selected to express the permeability for both soils and filter. The criterion will be as:

\[
\frac{d_{15}(f)}{d_{15}(s)} > 5
\]

(3.55)

These conditions are called Terzaghi Filter Design. For the rip rap and grovels the filter conditions can be simplified. Practically, the Max. nominal diameter is 2.5 times of Min. diameter. Since the classification curve of rubbles approximately is vertical, then the Max. diameter can be applied instead \( d_{85} \) and the Min. diameter can be used instead of \( d_{15} \). The stability criterion will be like as:

\[
\frac{df}{ds} < 10
\]

(3.56)
For preventing of filter or rip rap scouring, the Max. dimension of stones in every layer should not be more than 10 times of Min. diameter of stones in the same layer. Also, the grains of filter must be enough big that it is not broken inside the free space of sublayer stones due to velocity of water. Then, for this purpose the criterion will be:

\[ d = K \frac{V_v^2}{g_i} \]

\( K \) is 2 for gravel and \( i \) is the steepness of slope.

\( V_v \): Velocity inside the stones' space that it must not more be more than the scouring velocity of sublayer materials. The above Eq. is based on the velocity of wave running down. The Min. total thickness of filter layer should not be less than half of block's diameter in top layer. This is as a safety factor to prevent movement due to dropping of stones. However, the internal stability and permeability criterion is valid.

Core and intermediate layers like as foundation and armor layer are so important and they must have a special specifications. The core in spite of having enough compressibility, it has to show the feasibility to pass of water into it. Of course, the tiny grains in the core must have certain limitation.

The biggest dimension of stones in the core should not be more than 2.5 times of smallest dimension. Since this increasing will cause to create big holes and coming damages. The layers under the armor layers (filter layer) to keep the core must have tow type of specification.

1. Protection of core during execution
2. Providing of required condition to execute of armor layers.

The experience has shown that the structure will have a more stability and internal friction between blocks when the dimensions or weight of sublayer blocks become close to the armor layer's specifications. The general conditions for the armor layer and core will be:

\[ \frac{D_{15} (\text{Sub})}{D_{85} (\text{C})} < 4.5 \]

\[ 4 < \frac{D_{15} (\text{Sub})}{D_{15} (\text{C})} < 20 \]

\[ \frac{D_{50} (\text{Sub})}{D_{50} (\text{C})} < 25 \]

In the case of uniform armor layer:

\[ \frac{D_{85} (\text{Sub})}{D (\text{Ccr})} < 2 \]

\[ \frac{D_{15} (\text{Ar})}{D_{85} (\text{S} \cdot \text{Ar})} < 4.5 \]

\[ \frac{D_{15} (\text{Ar})}{D_{15} (\text{S} \cdot \text{Ar})} < 20 \cdot 25 \]
3.8. Toe Scouring
Whole of the constructions which are contacted with water have to be protected against the scouring. The following alternatives will be used:

1. Putting of stones in front of slope with natural stones or artificial blocks.
2. Using of Gabions which are filled with natural stones and becomes stable by vertical rods hampered into earth.
3. Using of sheetpile (made of steel, wood, concrete)

In the selecting of wooden sheetpile, it has to be made of hard wood and they should be located very closely to each other and there must be no space between them, because the tiny materials will come out from the same holes. The advantage of steel sheetpile is that this type is insulated and the tiny materials can not be come out from the sheetpile.

3.9. Sample Design of Breakwater by the Design Formulas
For reviewing of design formulas, the some of them are used just as a example.

Design of breakwater by using of Wiebe’s (1991) and Van der meer’s (1987) formulas:

\[ A_{mi} = (6.2 \cdot P^{0.31} \cdot \tan \alpha^{0.5} \cdot \gamma^{0.5})^{1/(P + 0.5)} \]
\[ P = 0.4 - 0.5 \Rightarrow P = 0.40 \Rightarrow A_{mi} = 3.33 \]
\[ \tan \alpha = 0.4 \Rightarrow \alpha = 21.8 \]
\[ A = 6.46 \cdot [1 - \exp(-19\beta)] \Rightarrow A = 1.824 \]
\[ B = 1.56 \cdot [1 + \exp(-19.5\beta)]^{-1} \Rightarrow B = 0.90 \]
\[ \beta = 0.016 \text{ Slope of Beach} \]
\[ LD = \left(\frac{g}{2\pi}\right) T_m^2 \]
\[ T_m = 0.5 + 2.8 \cdot (H_s)^{0.5} \Rightarrow T_m = 5.74 \Rightarrow LD = 51.41 \]
\[ H_s = 3.5 \]
\[ d = d_m + h = 2 + 3.5/2 = 3.75 \]
\[ d_m = 2 \text{ (Average depth in the location of structure)} \]
\[ \gamma_{br} = B \cdot (1 + \frac{A \cdot D}{L \cdot D})^{-1} = 0.79 \approx 0.8 \]
\[ H_{SL} = \gamma_{br} \cdot d = 0.79 \times 3.75 = 3.0 \]
\[ \alpha_m = H_{SL}/(G/2\pi) \]
\[ T_m^2 = 0.058 \Rightarrow A_m = \tan \alpha/\sqrt{\alpha_m} = 1.66 \]
\[ A_{mi} = 3.33 \geq A_m = 1.66 \Rightarrow \text{(Plunging Waves)} \]
\[ \rho_{s} = 2.5 , \rho_{r} = 1.10 \Rightarrow \Delta = \left(\frac{\rho_s}{\rho_r}\right) - 1 = 1.2773 \]
\[ \text{Starting of Damage} s = 2 , N = 3000 \Rightarrow D_{50} = 1.306 \approx 1.30 \text{ (m)} \]
\[ \left(\frac{H_s}{\Delta D_{50}}\right) A_m^{0.5} = 6.2 \cdot P^{0.18} \left(s/\sqrt{N}\right)^{0.2} \]
\[ W_{50} = \rho_s \left(D_{50}\right)^3 = 5.49 \approx 5.5 \text{ ton (Stones are required with the weight of 5.5 tons)} \]

After determining of \( D_{50} \), the eroded are of \( A_R \) will be:

\[ A_R = S \cdot D_{50}^2 \Rightarrow A_R = 3.38 \text{ (m\(^2\))} \]
\[ S = 2 , D_{50} = 1.30 \]

The Run Up is found for the rocky slope:
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\[ H_{SL} = 3.0, \ A_m = 1.66 \]
\[ A_m = 1.66 \Rightarrow 1.5 < A_m \leq 2.8 \Rightarrow R_e / H_{SL} = 0.88A_m^{0.41} \]
\[ R_e = 3.25 \text{ (m)} \]

After determining of \( R_e = 3.25 \text{ (m)} \), the Freeboard of \( F \) will be:
\[ H_e / H_{SL} = 0.478 \ (1.16 - F/R_s) \Rightarrow F \approx 0.00 \]
\[ H_e = 3.5, \ H_{SL} = 3.00, \ R_e = 3.25 \]

Then the height of crest in breakwater will be:
\[ 1/2 H_e + R_e + F + d_m = d + R_e + F = 7.0 \text{ (m)} \]

The thickness of armor layers:
\[ t = n.K\Delta D_{50} \Rightarrow t = 2.6 \]
\[ K_D = 1.02, \ n = 2 \ (\text{Number of armor layer}) \]

Min. crest width for overtopping:
\[ b_{min} = 3K'\Delta (D_{50})^{1/3} \Rightarrow b_{min} = 3.20 \]
\[ K'\Delta = 1.00 \]

Now, the width of scouring berm and the length till point of starting to erosion will be:
\[ L/50 = K_L + 11.05 \ (D_{85}/D_{15}) - 1.47 \ (D_{85} / D_{15})^2 \]
\[ K_L = 26.3 + 1.15 \ [H_e/(H_{SL}/\gamma_I - 1) \ D_{50}]^{0.19} = 32 \text{ (m)} \Rightarrow L = 14.70 \text{ (m)} \]
\[ \gamma_s = 2.5, \ \gamma_I = 1.10, \ D_{85}/D_{15} = 5.00, \ D_{50} = 1.30, \ H_e = 3.3 \text{ (m)} \]
\[ A' / D_{50} = K_A + 194.2D_{85}/D_{15} - 26.8 \ (D_{85}/D_{15})^2 = 476 \]
\[ K_A = 148 + 30[H_e/(H_{SL}/\gamma_I) - 1]D_{50}]^{0.91} = 273 \]
\[ B/D_{50} = K_B + 7.52D_{85} / D_{15} - 1.07 \ (D_{85}/D_{15})^2 + 6.12P_R \]
\[ K_B = -10.4 + 0.51[H_e/(H_{SL}/\gamma_I) - 1]D_{50}]^{2.5} = -7.10 \]
Assuming of \( P_R = 0 \), the Min. width of Berm will become: \( B = 4.80 \text{ (m)} \)

Design by Hedar’s Formula:

Case A: Wave running up and sublayer is permeable.
\[ K = 0.33(d_b + 0.7 \ H_b)(\tan\phi + 2)/[((\gamma_s / \gamma_I) - 1)(0.33 - 1/e^{\tan\phi}\cos\alpha(\tan\phi + \tan\alpha))] = 0.88 \]
\[ K = 0.90 \]
\[ \beta = \alpha + (\phi - 48^\circ) \Rightarrow \beta = 13.8^\circ \]
\[ \phi = 40^\circ, \ \alpha = 21.8^\circ, \ H_b = 3, \ d_b = 2.00 \]

Case B: Wave running down and sublayer is permeable.
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K = (d_b + 0.7H_b)(tan\phi + 2) / [(\gamma / \gamma_t - 1)(e^{4tan\beta} + 13.7)\cos\alpha(tan\phi + tan\alpha)] = 1.15
\beta = \alpha - (\phi - 48^\circ) = 29.8^\circ

Noticing the above cases the nominal diameter of stones which is equal to spherical their diameter, D = 1.15 (m) will be obtained which is a little bit less than the previous case.
From the USBR formula, one has:

D = K_b H^2 / [(S_r - 1)(Cot\alpha)^{1/3}] \Rightarrow D = 1.25 (m)

For the rounded and angular stones k_h = 0.95, for the big waves e = 0.67
Comparing of three above solutions, it will be clear the nominal diameter app. 1.20 (m) with specific weight of 2.5 t/m^3 will be acceptable for resisting against the waves. If one would like to use formula related to the both wave running down and wave running up at the same time, one will have:

D = K_e H / [(S_r - 1) \cos\alpha] = 1.25
D = K_e H / [(S_r - 1) \cos\alpha] = 1.30
K_e = 0.44 , K_r = 0.28

For example, if the sublayer becomes impermeable by the Van der Meer formula the diameter of stones will be calculated greater, but in the other formulas the difference between type of core has not been taken in to account.
Chapter 4

Coast Protection and Studying of Performed Methods
4.1. Coast Protection
The word of protection for coast is used to express of projects which prevent the coat against the flooding. Also, it protects the available situation of coast against the erosion. In the Coastal Engineering aspects the following parameters should be considered.

The factor to make erosion, direction, wave height, flooding, tides level (not noticeable in Caspian Sea), sedimentation transport, effects of constructed buildings upon the coasts and economically, environmentally and sociologically aspects. The most important projects for coast protection can be like as quay walls, breakwaters and beach nourishment.

4.2. Constructions of Coast Protection
The constructions of coastal protection are divided into three types:
1. Constructions far away from coast
2. Constructions for protecting of coast
3. Constructions located in the coast with special purpose

4.2.1. Constructions far away from coast
From this types of constructions the Breakwaters can be pointed. The breakwaters are constructions protecting of ports, harbors and harbor basins against the sea waves. The breakwaters are built inside of various depths of sea water. The seaward of breakwater should be completely resisted against the breaking waves due to storms and also against the hydrostatic pressure due to continuously attacking of waves. When the breakwater is built with slope, the attacking wave will run up on the slope and the velocity of water will change to pressure which it is due to shooting of water on the slope when the wave is spilling. These pressures are applied suddenly upon the slope while they are irregular. These pressures that are acted suddenly upon the slope, they will produce wave similar to the same impact done in water pipe lines which they can damage the breakwater. When the wave runs down, the exiting of water has pressure and the water tries to move the blocks. That is why the blocks must be enough stable against these pressures. If the rubble mound slope with the same slopes from the both sides is attacked by the waves, gradually the equivalent state will be produced which the seaward will have the smooth slope and it starts the point below the Still Water Level. However, in the top it will be below the initial level of construction.

The materials that are going away from the head of structure, they will have a natural slope on the other side which it is due to the internal friction of materials. See Fig. 4.1.

![Fig. 4.1. Schematized pattern of a breakwater](image-url)
If the blocks are enough big, the equivalent point of slope will be smooth and it will be close to the initial situation. The light weight of blocks will cause the breakwater to damage after time spending. Therefore, the blocks must have high weight to have stability against the waves. The high costs of constructing of breakwaters diminishes their use as a coast protection. In the other hand, the special condition to building of breakwater to protect of coastal line and beach sands increases noticing to its construction. The breakwaters affects on the sediment transportation near to the coast. Generally, the breakwaters are built parallel to the coast line and they protect an area against the waves. The refraction and diffraction of waves cause the wave energy decreases behind the breakwater. Because of this the sedimentation will be occurred in the protected region. The breakwaters which are extended along the coast and connected to it act as a dike. It is clear before constructing of breakwater, the sediment transportation has had the equilibrium. After constructing of it, there would be sedimentation in the upward. Due to the decreasing of sediments density in the coast currents, the erosion will occur in the downward. After spending of time, sedimentation will reach another equilibrium and the sediments will move along the breakwater and then the siltation will be occurred in the downward.

To day's, the breakwaters are built mostly from stones and concrete blocks. The core is built by the coarse gravel and broken stones and the whole body are protected by the concrete or rocky armor blocks. See Fig. 4.2.
When there is no enough stones in the area or from the economically point of view providing and transporting of big blocks and very heavy is not possible, it can be used from prefabricated concrete units in various shapes. The very common blocks are the Tetrahedron and Tetrapode which are used for armor layers. For example, the stones have been used as armor layers in the Bandare Nowshahr and the cubic units in the Anzali as armor layers.

4.2.2. Constructions of Coast Protection
As it is clear from the word, this type of constructions protects the lands behind themselves by absorbing of waves' effects. using of these type of constructions is the most effective solution to reduce of wave energy.

The sand beaches have been established by eroding of sea bottom in deep waters. Also the river flowing can take the material like sand to the beaches. These materials are moving continuously and they are moved by the current along the beach and they remove again to the sea or beach by the waves. Whenever there would be large sediment transport to beach or vice versa, constructing of the breakwater (epee) can protest the beach against the erosion and it can reclaim the beach. The Epee is the construction to protect of beach against the erosion and scouring which is built normal to the coast line. The reasons to building of epee are widening of beach, reduction of coastal currents and protection of beach against the erosion. The most important factors which affect the studying of epees are as following:

1. Damage value acted to the beach in downward.
2. Flow velocity
3. Feasibility of epee to be stable against the overturning from both sides.
4. Allowable coastal changes due to changes of angle of wave attacking.
5. Economical comparison with the other protection constructions.

The epees are made from wood, concrete, stone, gravel or concrete blocks. the selection of type of materials depends on the their availability in the region and the regarding costs. The length of epee depends on the sediment transport in the coast and the height of epee is calculated noticing of the depth of sedimentation. In initial stages after building of epee, the artificial beach nourishment by sand may be necessary in downward to prevent of undesirable erosion in downward. But after time spending, sediments will fill behind the epee and also they fill the holes between them till the beach reaches the new equilibrium. See Fig.4.3.

![Fig.4.3. Schematized pattern of spur](image-url)
4.2.2.1. The manner of Epees’ Working
The epees are the transversal constructions to protect the coasts. These types are core filled constructions that they make the obstacle against the longitudinal sediment transport till certain height. The currents in the windward are going toward coast and then are coming back to the sea parallel to the other epees. These currents may be caused scouring of coast and disconnection of epees with coast. In this case, the epees must be protected by one longitudinal building. The epees are built for protecting of beach outside of the water and they prevent the sand beaches to be scoured into the sea. See Fig. 4.4.

Fig. 4.4. Manner of working of spur

4.2.2.2. Situation of Spur
If the waves have a certain direction the most of time, the spur must be located parallel with that direction. But, if the waves are coming from various directions, the spurs will be built normally to the coast line. The spurs are built under certain angle with the coast line when the attacking angle of dominant waves to the coast line are 30 degree or more than it.

The spurs should be like an obstacle to prevent the sediment transport. In the other hand, their end at the seaward must be till wave spilling’s line during the low water level. (if there is any tides.)

It has been suggested that the end of spur must be extended till depth of 2 (m). Practically, it is not running down more than low water level. The distance between spur has been suggested between 1 to 4 (m). SPM suggests this value be between 2 to 3. If the sea is so windy and probability of occurrence for strong storms becomes so higher, this distance must be decreased. In the case of less longitudinal sediment transport, the distance between epees could be increased.

4.2.2.3. Longitudinal Profile of Spurs
In the longitudinal profile of spurs the problem is that how much height has to be given to the initial profile of beach. This height in the sand beaches must be little, app. 1(m) and in the gravel beaches this value should be more than 1(m). Noticing of this point the profile of epee will consist of horizontal section (AB), slopes parallel to the beach, and finally at the end below the low water level in tides (BM) a steep slope (DE), but this slope should be stable against the scouring and erosion by sea. Smooth slopes in
the sand beaches is about 1/20 till 1/25 and in the gravel beaches is about %15. See Fig. 4.5.

Fig. 4.5. Longitudinal profile of spur

4.2.3. Longitudinal Constructions located in the Cost
The protection constructions located on the coast are required when there would be erosion on the beach and there would be no equilibrium between movement of beach materials. Also, these constructions will be necessary when the strong storms become dangerous for the epees. The longitudinal constructions mostly are built along the coast line and consisted of quay walls with steep slope, quay walls with medium slope, and also with little slope. The coastal walls with more slope are the heavy constructions using to protect of upwards lands against the storms. The main duty of coastal wall with medium slope is retaining of embankment. And that is why these type are built from light structures. The coastal walls with little slope are built from stones, concrete and so on to prevent beach erosion. The following items must be considered in these cases:

1. Building of vertical constructions or with steep slope should be avoided, because there would be danger of scouring.
2. The danger of scouring also is available for constructions with smooth slope. Therefore, it has to be built the impermeable constructions in toe of coastal wall to protect of it against the scouring.
3. If the construction is built impermeable, the noticeable pressure will be acted to it. Therefore, in such structures the suitable drainage must be considered to do exit of water and the structure must be resisted against this pressure.

The coastal walls with steep slope mostly are built by concrete. Considering of some conditions like as fluctuating, type of foundation materials and other specification of waves are built in various forms. And these are gravity structures.

For standing against the waves, noticing of wave direction these walls are constructed in various patterns. For the various projects, they will be also different. The general forms are like as vertical, app. vertical, with slope, step by step, convex and cave. Every type has own advantages and disadvantages. It can be used combination of mentioned types considering of local conditions.
The main studying performed on the coastal wall with steep slope shows in the various forms the vertical walls are the most effective of walls in reduction of spillage and setting up of waves after cave walls. In the case of vertical walls with much slope the wave reflection and coast erosion will be Max. in front of the wall. The studying on the model shows the scouring of materials under the construction will be stopped when the depth of erosion reaches to the wave height under low water level. Consequently, in the whole range of vertical wall the toe of these structures should be protected by steel sheets to prevent the construction from scouring of its underlayers.

The purpose of coastal walls with smooth slope is as same as the walls with steep slopes. The main purpose to construct of these types is stability against the soil pressure. While the coastal wall with steep slope is built to prevent the erosion and other damages against the waves attacks. Standing against the waves is the second goal of this type and they are lighter than the coastal walls with steep slope. Therefore, it is used of concrete sheets or protection sheets in its body. The coastal wall with little slope protects the lands and buildings against the wave attacks. These types are not designed as a retaining walls. Since these types are resting on the soil and supported by the subsoil, therefore they are built in the same angle of their repose.

4.2.4. Artificial Beach Nourishment by Sand

The artificial nourishment method for beach may be a replacement method instead of epees or besides epees can be used to protect of beach. The beach nourishment is consisted of adding suitable materials into beach and dropping of them in suitable place where they can be distributed by waves action. The materials may be provided from borrow pit, mines, or from deep parts of sea by dredging. The amount of required materials and costs per annum consisted of transportation, loading and unloading depends on coastal erosion. The artificial nourishment of beach is not extensively used, but it can be used to fill behind the breakwaters that they have become gradually empty. The method of work is that the sand will be pumped by strong pumps which it comes out from holes around the breakwater and this sand is guiding to the target place where is needed to be filled by sand. this way has tow main goals. Firstly, dredging of siltation that is main problem in ports. Secondly, stabilization and reclamation of beaches by sand.

For example, there has been the same problem in port of Nowshahr. In the recent years, the new section of the breakwater has been added to the West section of breakwater and in the East side has been extended into the sea where has been built the area for Loading and Unloading. The water level rising of Caspian Sea and extension of port have created scouring and strong erosion in the East bank of Caspian Sea. The reached damage to the residential areas have been so heavy and the erosion of beach has reached several meters to the road connecting between recent city and other cities in province. A short looking at the location of port and harbor area shows that large amount of siltation made of tiny materials (silt and sand) has been settled behind of Western Breakwater, as same as in South of breakwater made of grovels and stones. These sedimentation has been caused little efficiency in this region and the port area is going to be filled by these sediments. Considering of borrow pit close to the same port area, by the artificial beach nourishment method, the erosion and scouring of beach can be prevented and reduced the received hazard.
Farhad Mirfakhrae, Participant in HE, 95-96
Report Title: Investigation on the effects of Caspian Sea upon its coast and presenting suitable protective solutions
4.3. Durability of Stones in Protection Constructions and Breakwaters

The most important parameters regarding to the destruction of stones are both physical and chemical phenomenon. The gradually erosion of stones in armor layers is a physical phenomenon. The chemical dissolved is another phenomenon which is occurred, but it is not so important. The chemical dissolved causes, the carbonate stones and the salts are solved. Also, it causes the Ferro Combinations are combined with Oxygen and Hydrogen and become swelling. Therefore, the extra volume of this phenomenon will damage the stones.

The physical phenomenon to damage of stones are divided to three category:
1. Abrasion
2. Cracking
3. Breaking

These mechanisms damage the armor layers that they defer only in quantity.

1. Abrasion:
Abrasion is the phenomenon that causes the particles of stones are separated by any external factor. This phenomenon can be occurred by particles of floating sands, shakes and fluctuation of stone upon another one. Even the sea water itself can make a erosion upon smooth or loose materials that it can penetrate into holes of stones due to hydrostatically pressure.

2. Cracking:
Due to attacks of salts, the big holes will be made in top layer of stones and this case the most common mechanism to destruction. Also, the frequency of freezing and defrosting related to the water and ice, various layers of stones, expounding of clay materials could make cracking of stones.

3. Breaking:
In this case the stone is broken to tow or several pieces. For example, these are occurs in the initial weak stones. these sheets can be appeared in stones during the extraction of stones from the mines or during the transportation or dropping of stones. Also it may be occurred during the strong storms in sea due to movement of armor layers.

It is clear these different phenomenon can be affected itself or besides together and one of them may help the other one to be stronger. therefore it is too difficult to proceed importance of one of them itself to making damage, but the conclusion will be destruction.

4.3.1. Effective Factors in Controlling of Erosion
The relative importance of damage mechanism is controlled by factors as:
1. Resting location of armor layers
2. Local climate conditions
3. Physical environs of wave
4. Types of stones and degree of its erosion

The resting location of armor layers in the top layer of breakwater has a relative importance. Considering of Fig.4.7. the body of breakwater is divided to four regions, but in the local studying these four regions can not be separated and generally it is
divided to three regions: Floating region, Intermediate region in tides and supper region of tides. These three regions are more suitable to control.

The climate condition is important from this point of view that it changes the arrangements of destruction factors. Destruction due to physical factors, salt of air and salty attack in the hot and dry climate will be too strong. Because the evaporation causes the crystallization becomes Max. and the resulted salts will be gathered on the stones and there would be no possibility of washing and coming back to the sea. Probably, the salt will be the most important factor of corruption, because this salt can be gathered on the tiny and dusty material between armor layers in the supper region of tides. The damaging of armor layer in the particular environs can be suddenly and serious. But, the salty attack will have a less importance where there are low evaporation and lot of rainfall. Since damage due to salty attack in the temperate climates is too little. Therefore, this effect in Caspian Sea would be neglected. But for the Persian Gulf and Oman Sea has to be considered. See Fig. 4.7.

Fig. 4.7. Divided sections in the breakwater

The frequency of freezing and defrosting in the cold climate is the important phenomenon that the penetrated water after freezing will swell and become expounded and makes the cracks wider and extended. Therefore this case is a harmful case, because the increasing of volume of water after freezing causes the stones will be destroyed. Combination of abrasion and big breakage can take effect from climate conditions and mostly it depends on place, location of storms in breakwaters and also depends on physical environs. The location of stones is important in the protection constructions. For example, the unsuitable stones for armor layer could be used for

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filter layers or unsuitable stones for section of tides changes may be used in floating area.

Water depth, wave conditions regarding to the physical aspects are so important to design of breakwaters or longitudinal protection walls. If the breakwater was designed well, it should be taken in to account the important factors like as durability of armor layers, type of stones and geological aspects of stones and degree of aeration. Degree of aeration and geology of stones can be estimated by testing when it comes out from the mines.

Since durability of stones is related to the type and tissue of stones and its natural cracks, therefore, the stones with aeration and microscopic cracks and weak cohesion between their particles will have less durability in the breakwater buildings compared the same stones but fresh.

The distance between layers, sheets and cracking inside the stones are so important and it controls the biggest pieces of stones using for breakwaters as the armor layers. The initial cracks may be created during transportation or dropping in the stones. It is clear the created cracks along the stones’ sheets can be restricted the dimension of produced blocks. The tiny cracks are also important case must be avoided, because it will help the abrasion and erosion phenomenon after constructing of breakwater.

4.3.2. The Damage Mechanism and its Effects on the Stones
To be rounded of armor layers’ stones is one of the important factors to design and execution of breakwaters. Since the armor layer blocks are rounded during breakwater’s life, therefore, it will be lighter and the ratio of free space between the armor layers is changed and also, the degree of blocks’ locking with the neighbor blocks are reduced. The reduction of weight and degree of locking in the armor layers will cause the instability of armor layers. While the changes of free space ratio varies the wave distribution specification.

For studying of these effects, the roundness phenomenon will be studied. The most solutions are arranged to tiny grains like as sands, so the studying of blocks will be so difficult.

Comparing of rounded blocks in breakwaters with the same stones which have been rounded artificially in the laboratory, the loss of weight during to be rounded can be estimated. The large number of tests have shown the roundnessibility of materials in the above area of tides compared to tide area is so little. These studying shows the limestone can be rounded easily. The free space ratio for various armor layers have been indicated in Table 4.1.

From the table it is clear the dependent between age of construction and ratio of free space. What is so important is that the shape of stones and their location in these constructions. Some of salts has more destructive effect compared with other salts what is related to the pressure of their hydration of salts. For example, the \( \text{Na}_2\text{SO}_4 \) is more effective than the \( \text{NaCl} \). Also the abrasion of stones will increase the roundness criterion of stones.
Table 4.1.
The abrasion of stones due to attack of salts increases rounded of limestone in the dry and hot climates. High void ratio will reduce the danger of salts' attacks. The big breakages among the stones increases the number of small stones in the armor layers which they can be washed out by storms. Therefore their space will be free. These holes can be filled by tiny materials which are carried by waves, consequently, the void ratio will be reduced.

Due to every big crack the stones will be changed to two or more pieces. The best way to reach to relative importance of damage or destruction, an selected section from armor layer will be evaluated as following.

1. Holes: Free space in armor layer
2. Cracks: The armor layer stones are changed to several pieces, but remained in their location.
3. Below the standard level: Quality and weight of stones is less than acceptable specification
4. Instability: Stones in the natural conditions of sea have fluctuation and movement. The evaluation of big cracks is returned to the No.2 and No.3.

Extension of damage or erosion in the first armor layer can be divided in three category:

1. Progress of cracks and breakages: The unstable blocks smaller than standard size will be conclusion of various mechanisms in damage.
2. Progress and increase of holes in the armor layers.
3. Damage of second protection layer and failure of whole construction.

The progressing of stages from 1 till 3 does not occur in naturally conditions and exception of erosion the destruction and corruption will be slowly. Of course the strong conditions will increase the stages of 2 and 3. One of the tests can express the relation between to be rounded of materials is Los Angeles Test. The another way is called mill wheel. In this way several samples of stones written the certain number
upon them will be rolled upon each other for certain limit accidentally. Then the samples are taken out and the roundness of materials are recognized.

For the various types of stones the acted roundness varies by year. For example, for the limestones with chalk one hour inside the mill equivalent with 5 years becoming rounded in the hot and dry climate with mean energy of waves. While for the hard limestones plus carbonate ferrous one hour in the mill is equivalent with 102 till 103 years in the mild climates.

4.4. The Durability of stones and Regarded Tests

There are several tests to evaluate of stones’ qualification for making concrete and its condition and building stones. The abrasion and cracking are the main damages factors to rocky armor layers. These factors consists of separating grains from each other. Therefore, the inconsistency, cracks, stone tissue probably are the most important factors which must be taken in to account during tests. Among the tests this determination of density, compression strength, abrasion strength and percentage of weight loss in the sodium sulfate solution will be the most important tests.

In the case of evaluating against the sulfates can be never compared the salt attacks from the sea. Since the used temperature during the test is so much compared with real values in nature, therefore we have to be very careful to interpretation of results.

In Iran to evaluate of quality for stones in the Northern Coasts, the types of tests and their limitations are as following:

1. For various stones:
   1.1. ASTM C 88-73, Reaction in salty environs (Sodium Sulfate Solution)
   1.2. ASTM C 79-70, Saturated specific weight and degree of water absorption.
   1.3. ASTM C 170-70, Compression Strength

2. For protection Stones
   2.1. ASTM C 535-75, Bearing Abrasion Strength

The studying from available stones mines, the Bala zarandin’s stone mine for the project of Babolsar and Bandare Torkaman and the Leersar’s mine stone for the projects of Tonekabon and Nowshahr have been recognized more desirable.

Considering required quality and specific weight of 2.2 till 2.6 t/m³ and also for the economically point of view the rubble mound breakwater will be more preferable. From the above mentioned discussion about the durability of stones it can be concluded the following points.

<table>
<thead>
<tr>
<th>Type of Stone</th>
<th>Weight</th>
<th>Shape Factor</th>
<th>Max. Solvency of Na₂SO₄ ASTM C 535-75</th>
<th>Max. Abrasion ASTM C 535-75</th>
<th>Min Saturated Specific Weight (t/m³)</th>
<th>Max. Water Absorption</th>
<th>Min Compression kg/cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Broken Stones for core</td>
<td>As Map</td>
<td>0.2</td>
<td>20</td>
<td>60</td>
<td>2.2</td>
<td>5</td>
<td>200</td>
</tr>
<tr>
<td>Filter Layer</td>
<td>As Map</td>
<td>0.4</td>
<td>8</td>
<td>30</td>
<td>2.4</td>
<td>3</td>
<td>500</td>
</tr>
<tr>
<td>Armor Layer</td>
<td>As Map</td>
<td>0.4</td>
<td>8</td>
<td>30</td>
<td>2.4</td>
<td>3</td>
<td>500</td>
</tr>
</tbody>
</table>

Table 4.2. Desirability limits for stones.
The size and shape of rock armor layer is function of inconsistency which is available in the mine. If the careful ways of explosions, careful transportation and dropping methods becomes minimum, the maximum dimension of blocks will be obtainable for the armor layers. Safe dropping of stones inside the water will cause to have good locks between blocks and having acceptable void ratio. After putting the blocks in considered places, after some time the shapes of the blocks will be changed by mechanisms like abrasion and cracking. These effects will be the Max. in the intermediate section of breakwater. These effects are important, because they change the shapes, weights, interlocking degree and the void ratio of armor layers. Also it requires to consider the type of effect. The special studying must perform to select of stones which it will enable the required design is recognized considering the changes occurring during the construction’s life.

The factors must be taken in to account are the evaluating of materials’ quality, aerology conditions, wave aspects and return period of storms. The Table.4.3. shows summary of stones’ durability in various conditions of weather.

<table>
<thead>
<tr>
<th>Type of Stones</th>
<th>Rounded and Abrasion</th>
<th>To be Flaked</th>
<th>Big Breakage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>Climates</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Freezing</td>
<td>2</td>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td>Mild</td>
<td>3</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Dry and Hot</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Tropical</td>
<td>4</td>
<td>5</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 4.3. Predicting of corruption of stones in various climates

A: Acidic stones like as Granite, Sandstone, Getice
B: Base stones like as Basalt, Shiest, Andegit
C: Carbonate stones like as Limestone, Marmarit, Dolomite
S: Stable like as Carbonate Limestones
W: Weak like as Limestones and Chalk
1: Very strong against damage
2: Strong against damage
3: Medium strength against damage
4: Weak against damage
5: Very weak against damage

The aerology conditions have a little effect on the durability of stones in the sea climate, unless in the special coasts. For example, freezing and defrosting in the cold weathers has a very minus effect on the stones having microscopic cracks and breakage. In the other hand in the hot and dry climates, the effects of salt is more important than the other factors.

Wave conditions and return period of storms have a great importance which the damage of breakwater body will be noticeable. In the normal conditions, the gradually varying is created related to shape and weight of blocks, but during the storm conditions, it starts to make damages caused rapidly failure of construction. The pattern of damage is the results of wave energy. The quality of rock armor layers is so important. If there are some very thin cracks in the stones, it will be caused to break.
Also the quality of resting, degree of well locked and closed of blocks and void ratio are important.

The unstable blocks are the most common factors of damage which it can be related to the low interlocking between the blocks. Since in the bad weather condition with much energy, these blocks could be scoured and washed out. Therefore they will make the holes in the breakwater surface which it follows the structure to failure. Considering of above statements, the most important factors causing gradually damage will come below.

4.5. The Effective Factors in Gradually Damage
1. Absence of not enough hydraulic and engineering studying and using unstable plan
2. Existence of stones with less weight than the minimum required weight for standing against the waves.
3. Existence of big holes between the stones due to execution method during building which caused the local damage and finally having the complete damage.
4. Absence of suitable filter and filter layer in the section of protection construction.
5. Absence of required controlling on the execution operations.
6. Absence of suitable distribution of armor layer as the core is contacted directly by the more impacts.
7. Using the unsuitable execution solutions.
8. No using of intensification of waves.
9. No considering of scouring matters.
10. Absence of noticing to buoyancy of sea bottom under section of breakwaters.
11. Inadequate quality in concrete making in the concrete blocks.
12. Inadequate studying of design wave height related to the overtopping and finally making damage to the construction
13. Breakage of protection blocks due to dropping and initially existence cracks during taking out from mine and transportation.
14. Erosion of stones and loss of their weight and becoming smaller due to wave attack and weather conditions.
15. Absence of noticing to the bottom materials that the settlements due to this case could create the difficulties.
16. Absence of using the suitable blocks with high durability in the considered construction during its life.

4.6. Manner of Damage in Protection Constructions
1. Movement of smaller grains from the armor layer and spillage of them into the toe of slope.
2. After getting empty the around of big stones, over turning and loss of equilibrium, movement of these stones to the toe.
3. Effects of waves in the small rocks of armor layer and shaking and rolling of them.
4. Scouring parts of core and reduction of useful their width.
5. Dispersion of outer layer stones that are unstable due to the waves attacks are moved and displaced.
7. Cracking the blocks due to wave impacts finally caused their movements.
8. Overtopping of waves in breakwaters due to inadequate height of construction.
9. Starting of damage initially due to buoyancy of sea bottom and then intensification of armor layer damage and then whole failure.

10. In the case of tetrapods breakwaters, the relative damage are high, because the big initially internal friction angle is decreasing by spending of time and also the difference of friction angle between this layer and sublayer will be a factor to damage.

11. Inadequate resistance against scouring at toe.

In the recent years using of sea resources and unfeasibility to reaching of stones with high density and suitable tonnage, using the concrete blocks has been increased. In this respect, the cubic concrete unit shows the more durability compared with the others. Determination of type of bottom's materials, their layer thickness also the resting situation of layers under each other in design process and finally stability of construction will be the so important factors.

The toe protection is both for the soft bottom and for the rocky bottoms. When the direction of foundation in rocky bottom is in the same direction of longitudinal axis, the excavation will be normal to the longitudinal axis of breakwater and excavation in the toe section will be necessary for stability of breakwater. If the bottom is soft and compressive (silt or soft clay) it should be removed and replaced to the suitable material.

Here, several examples of designs which have been performed in Iran and other countries are summarized and damage of these constructions, explanation of acted activities and types of damages will be expressed.

1. Fishery Port of Khark:
Starting of its damage have been since 1973 till now. It has been constructed in years of 1972-1973. The materials used in the outer layer are limestones with the weight of 2 - 7 tons and the significant wave height has been 3.00 - 3.50 (m). The slope of armor layer is 1 over 2 till 1 over 1 in the seaward.

Movement of smaller grains between the protection stones, displacement and dropping down to the toe of slope, emptying the around of greater stones, overturning and movement of these stones into the toe have been process of its damage.

Existence of stones lighter than the Min. required weight standing against the wave attacks and existence of big holes between the stones due to type of execution has caused the local damage. Also the absence of suitable filter layer, inadequate controlling during the execution, non enough hydraulic and engineering studying have been the reasons of damage.

The only solution to maintain of it is review in design considering of blocks with higher weights.

2. Fishery Port of Rostamy:
Starting of its damage has been since 1979 - 1980 and it has been built since 1975 till 1976. The used materials in the outer layers are from limestones in the various dimension with the weights of 2 - 5 tons and the wave height has been considered 0.50 - 2.00 (m) various. The slope of armor layers is 1 over 1.5 in the waves side.

The distributed stones in armor layer has been removed due to wave attacks while they have been unstable. Their movement has made a lot of damage. In the other hand the core is so unstable and due to wave attacks has been washed out and caused the
crest becomes narrow. Whole of these mentioned factors are the expression for manner of damage.

Using of unacceptable primary plan which it has been without any filter layer and distributed outer layer, using of rounded corner materials in the core section, unsuitable execution of operations, having no controlling during the building have been most important reasons of damage.

The only solution for maintenance is the using of big stones in the armor layer.

3. Tripoli Port in Libya:
Starting of damage has been since 1981 and it has been completed in 1978. The used materials are Tetrapodes as the armor layers with the weight of 19 tons and the significant wave height to design is 9.00 (m) and the wave height during the damage has been 8.10 till 9.00 (m).

The breakage of Tetrapodes after storms, movement, damaging of wall upon breakwater, height difference between top protection layer and top part of wall upon the breakwater, overtopping of waves have been expressed the manner of damage. Wave combination and creating intensification case increasing by wind blowing and time of blowing, studying of hydraulic models by the uniformed waves, interaction, intensification of height and consequently increasing of wave energy and having error in the Fetch length have been expressed as most important factors of damage.

The new plan has been performed for the wave height of 10.10 (m) and using of cubic concrete units with the various weight and dimensions.

4. Gioia Tauro Port in Italy:
Starting of damage has been performed during the execution of construction. The used materials in the armor layers are from Dolloss with 15 tons weight and the significant wave height in design has been 7.00 (m) and the slope of armor layers is 1 over 2.

Preliminary, the starting of damage in the breakwater has been due to thrust of sea bottom under effecting of wave influence with the wave height of 8.00 (m). After that cracking of Dollosses due to wave producing by the storms have been mentioned the manner of damage.

Waves’ combinations, intensification of waves, changes of sea bottom, therefore the increasing of intensification and the effect of water impact have been the most important reasons of damage.

Providing of new plan has been based on more wave height till 8.00 (m) and the maintenance of damage of parts by special binterlockss have been expressed the only method for maintenance.

4.7. The Examination of Performed Protection Plans
The water level rising in Caspian Sea in addition making lot of economically problems, has been caused the some offices and institutions become busy to have the schedule against these problems. From those, the group of boundary waters of Force Ministry is introduced.

In this respect, the providing of hydraulic models for the shore protection constructions has been managed by this group. The plan which will be explained, has been provided in cities of Tone Kabon, Astara, Kalachay. In the coasts which have had 50 (m) of residential buildings, roads inside the sensitive areas that this plan only has been executed in the Astara and Kalachay.
This plan consists of a dike and a breakwater resting in front of dike. The used materials in the dike consisted of core materials with diameters of (0 - 10 cm) that its surface has been covered with cement bags. These bags have been later connected to the dike body by the injection of cement. The positive point of this plan is noticing to the local materials.

For constructing of planed breakwater has been used of Running Sand as a core materials, the stones with the weight of 50 - 200 kg., one layer of stones with the weight of 1 ton as a filter layer, a layer of stones with the weight of 2.5 tons as an armor layers. The details are clear in the Appendix.

The Max. and Min. datum of water level and the wave height using for the design has been considered as shown in Table 4.4.

<table>
<thead>
<tr>
<th>State</th>
<th>Water Depth (m)</th>
<th>Height (m)</th>
<th>Period (Sc.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.6</td>
<td>1.6</td>
<td>7</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>2.7</td>
<td>7</td>
</tr>
</tbody>
</table>

Table 4.4. Specifications for design.

The main points have been taken into account in the plan of model are as following:

1. Studying of stability of part of breakwater in the coastal wall with considering optimization of armor layer dimensions.
2. Studying of wave overtopping and wave transmission from the breakwater body.
3. Studying of wave Run Up upon the dike.
4. Studying of breakage of materials under the armors in the hill of breakwater.

Whole of those mentioned factors are the most important cases. The non considering of one of those factors will cause the failure of construction.

4.7.1. Model Structure and Used Values

For the observation of the wave action on the model has been used of a box with the length of 40 (m), width of 1 (m) and height of 0.80 (m) including the glass part to observation. Also it can produce the waves. In the building of model breakwater has been used the stones with specific weight of 2.57 t/m³. This model, has been provided based on similarity rule using of Froude number. Noticing of restrictions about model building, the scale of 1/8 has been selected. Noticing of scale, the relation between the various physical quantities in the original structure and model will be indicated in Table 4.5.

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Length</th>
<th>Time</th>
<th>Volume</th>
<th>Weight</th>
<th>n</th>
<th>Velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scale</td>
<td>1/8</td>
<td>1/2.8</td>
<td>1/512</td>
<td>1/512</td>
<td>1/1</td>
<td>1/2.8</td>
</tr>
</tbody>
</table>

Table 4.5. Relation between the main quantities and models.

Considering the used scale, specification of used waves in the model are expressed in Table 4.6.

<table>
<thead>
<tr>
<th>State</th>
<th>Water Depth (m)</th>
<th>Height (m)</th>
<th>Period (Sc.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td>20</td>
<td>2.5</td>
</tr>
<tr>
<td>2</td>
<td>37.5</td>
<td>33.7</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Table 4.6. Specification of design wave
Based on this case, the specification of used stones are shown in Table 4.7.

<table>
<thead>
<tr>
<th>Equivalent Diameter in Model</th>
<th>Equivalent Diameter in Structure</th>
<th>Weight of Stone in Model (kg)</th>
<th>Weight of Stone in Structure (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sph. (cm)</td>
<td>Cub (cm)</td>
<td>Sph. (cm)</td>
<td>Cub (cm)</td>
</tr>
<tr>
<td>15.25</td>
<td>12.25</td>
<td>122</td>
<td>98</td>
</tr>
<tr>
<td>14.13</td>
<td>11.38</td>
<td>113</td>
<td>91</td>
</tr>
<tr>
<td>12.78</td>
<td>9.63</td>
<td>103</td>
<td>77</td>
</tr>
<tr>
<td>11.25</td>
<td>9</td>
<td>90</td>
<td>72</td>
</tr>
<tr>
<td>5.25</td>
<td>4.25</td>
<td>42</td>
<td>34</td>
</tr>
<tr>
<td>4.1</td>
<td>3.4</td>
<td>33</td>
<td>27</td>
</tr>
</tbody>
</table>

Table 4.7. Dimension of performed stones.

4.7.2. Explanations of Carried out Tests on the Model

The tests have been performed in the tow mentioned cases.

Stability Test:

In this test the design wave have been acted 1000 times continuously to the section and the various effect of it has been studied. The movement value of armors and its analysis shows the value of acted damage.

In the first case, the studying of model before and after testing has been shown the movements of armor rocks has been not occurred.
In the second case, when the coastal wall was completely floated, again there would be no change in the armor layer. Then the toe cases show the stability test will be acceptable and the armor layer has required weight for stability and this weight regarding to the wave specifications to design is acceptable.

Wave Overtopping Test:

Since the breakwater is a permeable construction, a section of wave is transmitting from the body of breakwater. But, this transmission in the case of short waves are little, because the core materials prevents from this transmission so much.
In the first case, the studying shows the transmission is too little.
In the second case, the overtopping of wave has been measured upon the crest and shows the overtopping of wave comparatively is much. Therefore, the overtopping wave could make damage to the dike which is resting behind the breakwater.
Noticing in both cases some values of wave energy passes from the breakwater section. (at first case from the body of breakwater and at second case over the crest of breakwater). Therefore, some parts of energy encounters to the dike and because of arriving of wave upon the slope, it runs up upon the slope and that energy is caused wave running up.

The wave transmission for tow previous cases and after breakwater has been measured by the sensors. Their values has been shown in the Fig. 4.8. and Fig. 4.9.
Water Research Center Co.

Fig. 4.8.

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Fig. 4.9.
If the wave run up is equal to Max. vertical height which this value is running up as they measured from the bottom level, one has:

<table>
<thead>
<tr>
<th>State</th>
<th>Depth of water in Model (cm)</th>
<th>Height of Wave in Model (cm)</th>
<th>In Model (cm)</th>
<th>In Main Structure (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td>20</td>
<td>30</td>
<td>2.4</td>
</tr>
<tr>
<td>2</td>
<td>37.5</td>
<td>33.7</td>
<td>85</td>
<td>6.8</td>
</tr>
</tbody>
</table>

Table 4.8. Wave running up

The above values show to prevent of Run Up, the height of breakwater and protection dike must be increased. The following suggestions will be considered:

1. Decreasing of weights of armors with the weights of 2.5 tons
2. Increasing of breakwater's height
3. Increasing of dike's height

In the case of 1 and 2 the weight of stones decreased from value of 2.5 tons to value of 1.5 tons and the height of breakwater increased to 3 meters. In the case of point 3 after repeating of tests and interpretation the final results are as following.

After testing it was observed the armor layers have had little movement without any vertical movement, then the weight of stones are enough heavy to be stable. In the case of wave overtopping, the test has been carried out for the both cases which the values after and before the testing have been shown in the Fig. 4.10. and Fig. 4.11.
Before B.H.

Channel 0

After B.H.

Channel 1

Fig. 4.11.

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As the first stage the value of running up has been measured that the results are indicated in the Table 4.9.

<table>
<thead>
<tr>
<th>State</th>
<th>Depth of Water in Model (cm)</th>
<th>Wave Height in Model (cm)</th>
<th>In Model (cm)</th>
<th>In Structure (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td>20</td>
<td>5</td>
<td>0.4</td>
</tr>
<tr>
<td>2</td>
<td>37.5</td>
<td>33.7</td>
<td>37.5</td>
<td>3</td>
</tr>
</tbody>
</table>

Table 4.9. Wave run up

After studying of above statements, the remained problem is the scouring of toe materials of breakwater that it can be mostly the main problem to damaging of breakwaters. In the performed tests, this matter was considered and it was observed that the small granule materials between the armor layer are removed of their own location and they are transformed into in front of the breakwater. The materials with diameter of more than 5 (cm) are suitable. Considering of results of performed tests, the final pattern of design for constructing of breakwater will be as following and its section is available in the Appendix.

1. For the armor layers has been used of stones with the weight of 1.5 - 2 tons.
2. Height of breakwater is extended till 3 (m).
3. For the backward dike, its height is extended till 6 (m).
4. For the filter layer under the hill armors is used the materials with mean diameter of more than 5 (cm).
5. The other specifications are performed as the map of protection wall.

In addition of tow above mentioned projects which has been executed in the Kalachai and Astara, for the regions of Tonekabon, Bandare Torkaman the other suggested plan has been carried out that it will be explained as following.

Rubble Mound Breakwater of Tonekabone, Babolsar and Bandare Torkaman:
Noticing of extension of plan area and distribution of coastal zones, the situation of coastal line naturally will be differed. The Table 4.10. shows the results of water depth measurements in the several points.

<table>
<thead>
<tr>
<th>Dist. from Beach (m)</th>
<th>Mirrood</th>
<th>Slope</th>
<th>Sorkhrood</th>
<th>Slope</th>
<th>Roodsar</th>
<th>Slope</th>
<th>Chaloos</th>
<th>Slope</th>
<th>Aver. Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0.7</td>
<td>0.014</td>
<td>0.5</td>
<td>0.01</td>
<td>0.9</td>
<td>0.018</td>
<td>1.00</td>
<td>0.02</td>
<td>0.015</td>
</tr>
<tr>
<td>100</td>
<td>1.7</td>
<td>0.02</td>
<td>1.3</td>
<td>0.016</td>
<td>1.7</td>
<td>0.016</td>
<td>1.8</td>
<td>0.016</td>
<td>0.017</td>
</tr>
<tr>
<td>150</td>
<td>3.5</td>
<td>0.036</td>
<td>3.2</td>
<td>0.038</td>
<td>3.60</td>
<td>0.038</td>
<td>3.7</td>
<td>0.038</td>
<td>0.038</td>
</tr>
<tr>
<td>200</td>
<td>4.5</td>
<td>0.02</td>
<td>4.2</td>
<td>0.02</td>
<td>4.60</td>
<td>0.02</td>
<td>4.70</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>250</td>
<td>-</td>
<td>-</td>
<td>4.7</td>
<td>0.01</td>
<td>5.10</td>
<td>0.01</td>
<td>5.20</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>300</td>
<td>5.00</td>
<td>-</td>
<td>-</td>
<td>6.00</td>
<td>6.00</td>
<td>0.018</td>
<td>6.15</td>
<td>0.019</td>
<td>0.018</td>
</tr>
</tbody>
</table>

Table 4.10. Results of water depth measurements in different locations.
Also because of resting of building in the plan areas where the breakwaters' lines pass from it, the water depth varies from zero till 2.5 (m) in the quiet sea. Therefore the water depth and design wave height will be various in the different areas. Anyway, the design criterion will be indicated in Table 4.11.

<table>
<thead>
<tr>
<th>Depth of Sea Water</th>
<th>0 - 2.5 (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. Water Level Rising (assumption)</td>
<td>1.00 (m)</td>
</tr>
<tr>
<td>Max. Instantaneous level Rising</td>
<td>± 0.5</td>
</tr>
<tr>
<td>Significant Wave Height</td>
<td>0.8 - 2.7 (m)</td>
</tr>
<tr>
<td>Wave Run Up (External slope 2 to 1)</td>
<td>0.5 - 1.8 (m)</td>
</tr>
<tr>
<td>Freeboard</td>
<td>zero</td>
</tr>
</tbody>
</table>

Table 4.11. Assumptions of design

If the below relation is accepted for crest of breakwater:

Height of Crest : \(d + \frac{1}{2} H_s + R_u + F\)

one has:

\(d : \text{Water Depth}\)
\(H_s : \text{Significant Wave Height}\)
\(R_u : \text{Run Up}\)
\(F : \text{Freeboard}\)

Then:

Max. Height : \(3.5 + \frac{2.7}{2} + 1.8 = 6.70\) (m)
Min. Height : \(1.0 + \frac{0.8}{2} + 0.5 = 1.90\) (m)

In the above calculations ±0.5 due to the air pressure effect has not been taken in to account. These values should be added to the water level heights to calculate of height of breakwater. The used dimension of stones in the various layers are as following:

1. Stones Type 1:
   For the sublayer, the weight of every block is between 20 till 50 kg which has to be cleaned from silt and organic materials.
   Execution of this layer has been that the materials were dropped from the landward. The free space between the stones have been Min. and the executed layer has to be compacted.

2. Stones Type 2:
   This is used as a core materials and the weight of every block is between 50 till 100 kg and at least %50 of materials must have more than 75 kg weight.
   For executing of this layer, the materials were dropped from the landward by the trucks and then by the suitable equipment the considered slope will be produced.
3. Stones Type 3:
Stones of filter layer which weight of every block is % 10 of weight of rock armor layer.

4. Stones Type 4:
Rock armor layer which is used in the various weights for different locations and depths have been indicated in Table 4.12.

<table>
<thead>
<tr>
<th>Final Depth of Design</th>
<th>Specific Weight of Stones (at least)</th>
<th>Weight of Stones</th>
</tr>
</thead>
<tbody>
<tr>
<td>till 2 (m)</td>
<td>2.6</td>
<td>0.5 - 0.8</td>
</tr>
<tr>
<td>till 2.5 (m)</td>
<td>2.6</td>
<td>0.7 - 1.0</td>
</tr>
<tr>
<td>till 3 (m)</td>
<td>2.6</td>
<td>1.1 - 1.7</td>
</tr>
<tr>
<td>till 3.5 (m)</td>
<td>2.6</td>
<td>1.8 - 2.6</td>
</tr>
<tr>
<td>till 2 (m)</td>
<td>2.4</td>
<td>0.6 - 0.9</td>
</tr>
<tr>
<td>till 2.5 (m)</td>
<td>2.4</td>
<td>0.9 - 1.3</td>
</tr>
<tr>
<td>till 3 (m)</td>
<td>2.4</td>
<td>1.5 - 2.3</td>
</tr>
<tr>
<td>till 3.5 (m)</td>
<td>2.4</td>
<td>2.4 - 3.6</td>
</tr>
</tbody>
</table>

Table 4.12.

Priority of execution of layers is:

1. Preliminary layer:
This layer helps to stability of sea bottom and section of plan. Therefore execution of this layer is too necessary before executing of core layer. The continuing of work is permitted after performing of this layer.

2. Core layer:
It is necessary for filling of plan spaces.

3. Filter layer:
For preventing of sinking of armor layer.

4. Armor layer:
For preventing of damage and stability of breakwater against the wave attack. This armor layer has been considered to prevent of damage in the core layer. During the execution the slopes of both sides should be covered by the protector layer. To prevent the damage to the filter layer, the seaward surface of breakwater must be covered with the armor layer.

Any damage exerting by the waves, storms, or any other phenomenon should be maintained. The thickness must be controlled to have any required action. Totally, to protect of execution of plan, the Max. length equal 25 (m) could be remained without any cover or other layer, more than this value, it must be covered by the another layer. It has to be avoided putting the filter layer instead armors, otherwise, due to wave energy they would be washed out.

The concrete walls have been designed behind the breakwater with the height from bottom 12.5 (m), height from water level 4.5 (m), thickness of wall 0.6 (m) made of reinforced concrete. The enclosed map shows the total pattern of breakwater.
4.7.3. Manner of damage in the Breakwaters of Astara and Kalachai
1. Movement of used stones in the outer layer with the low weight, their results have been movement of stones.
2. Emptying of around space of used stones due to absence of right filtration’s execution, the result of this is the over tuning of stones and burying of stones inside the bottom.
3. Having not enough required height for breakwater due to selection of unsuitable wave height to design.
4. Wave breaking upon the breakwater that it causes damage of stones in some cases and movement of stones upon the dike slopes.
5. Down flowing of waves from the top of dike into the back of dike in land ward which have made the inundation to the residential buildings.
6. Absence of right controlling of execution processes and during the dike constructing which caused the cement bags are not connected with together with cement grout. This matter has made damage on some part of dike.
7. Since the dike body is approximately plane, it needs much height to prevent of wave overtopping over the crest. Then the increasing of height is necessary.
8. Inattention to the wave intensification in the studying of model of plan.

4.7.4. Reparative Methods for Suitable Re-exploitation of Performed Plan
1. Carrying out the master studying about design wave.
2. Increasing of dike height in the required value after carefully studying upon the previous dike.
3. Replacing of stones with the weight of more than 2 tons in the armor layer.
4. Electing of suitable filtration in the various beds where the breakwater is built, because in the gravel and rocky beds the executing of breakwater is suitable, but the sand beds have not the enough quality to resting of breakwater.
5. Right execution of breakwater and continuously controlling of it by setting of one by one of the armor blocks to create the Max. friction between the blocks.
6. Selecting of suitable stones that they have good stability against the transportation damage and attacks of waves impacts.

4.7.5. Damage Manner of Breakwaters of Tonekabon, Babolsar and Bandare Torkaman
Preliminary, the Water Organization has built the short rocky breakwater in the level of -24 meters of open seas because the extension of Province of Mazandaran. But, due to intensity of exerted damages, this level of height was changed and it was increased. Then tow types of breakwaters, short and high, have been built which are expressed in the primary report.

4.7.6. Damage Manner of Rocky Breakwaters
1. No setting of stones one by one which causes the Min. friction between the stones. This is the factor of unsuitability of armor layers.
2. Non execution of bed layer which has caused sinking of toe stones.
3. Non execution of suitable filter which caused exiting of core layer materials and sinking of armor stones.
4. Wave overtopping from the crest causing scouring of core materials and the sand under the breakwater.
5. No enough width of crest which prevents of core scouring.
6. Using of small blocks of stones due to considering of low design wave height.
7. No enough controlling during the constructing.
8. Wave breaking upon the slope and toe of slope causing more destruction and movement of armor layers.
9. Non execution of scouring berm to preventing of destruction of armor layer.
10. Non execution of concrete wall behind the breakwater.

Above mentioned factors have been most important factors showing the unsuitable ability of short rocky breakwater. These problems have been caused the high rocky breakwater to be considered. The high rocky breakwater has a required height for preventing of wave transmission inside the dike body and overtopping, i.e. noticing to prevention of wave run up and wave dissipation. In these cases there has been enough height. However, it is observed some overflowing of waves which it is acceptable. The local studying of high rocky breakwaters shows the following cases.

1. Non execution of scouring berm. It should be executed but, it has not yet been carried out.
2. Non execution of concrete wall behind the breakwater.
3. Using of smaller stones than the Min. required stones causing damage of armor layers.

Considering above mentioned cases, the manner of damage and destruction of high rocky breakwater is as following.

1. Non using of suitable design wave height which has caused the movement of stones.
2. Non execution of bed cover which causes the sinking of stones in toe of breakwater.
3. Non execution of scouring berm causing destruction of seaward armor layers.
4. Exiting of filter layer due to unsuitable design.
5. Using of stones with less weight in the armor layers. This caused the movement of armors due to wave attack.
6. Using of Hadson formula to design of armor layer which is so old and it has not so workability.

Suggestions to Suitable Re-exploitation of High Rocky Breakwater:
1. Using of exact design wave height by carefully studying.
3. Maintaining of damaged part of armor layers by bigger stones.
4. Making of scouring berm for preventing of gradually damage of armor layers.
5. Making of suitable filtration where the maintenance of slope would be possible.
6. Increasing of height of high breakwaters approximately 1.00 m to prevent of wave overtopping.

4.8. Suggestion of Suitable Plan
As it explained already, a construction such as breakwater protects the coast by absorbing of destructive effects of waves’ attacking. Also, it is the most effective method for decreasing of wave energy. Whenever the sedimentation in the coast would be observed, constructing of breakwater reclaims the coast and protects it. In the other
hand, the longitudinal constructions are required for protecting of upward beach. Whenever the sedimentation would be not enough for coast reclamation and there would be erosion in the beach, the longitudinal constructions of coast would be more effective.

From the above statements about the breakwaters and spurs, it is clear to construct the coastal constructions the knowing of sediment transport will be necessary. To provide the plans for spurs, it is necessary to have value of materials which they are displacing during the wave impacts and sea currents. Noticing of this case that there is little information about sediment transport, however, the visual observation from the various locations shows the values of sediment transportation are too much. So, the best method to reduce of waves’ energy is constructing of breakwaters. In this respect, the spurs have to be helped by a longitudinal protection construction.

The purpose of constructing of wall and spur is the prevention of the erosion in the coasts. The building of these two structures in the same time seems to be necessary, because constructing of spur itself may be have no enough effect for absorbing of wave energy considering of waves; height and types of coastal lands (gravel and mostly sands). Also there would be possibility that the sea continues its destructive effect by progressing and turning around the spur. Constructing of coastal wall (as it performed) without spur system could not prevent completely from the coast erosion and sea bottom near to the beach, unless the other types of longitudinal walls are used after adequate studying and modeling.

Unfortunately, it has not been yet carried out important project about building of spurs, but in the other countries the lot of works in this respect with the various materials have been performed.

What could be observed in the coastal zone of Caspian Sea is the existence of lot of stone mines with the good quality in the acceptable economically distances. Therefore, building of spurs by the stones would be economically.

For designing of spurs (various layers) could be used from the same relations for designing of breakwaters. Of course, using of wave height with the mean height or less is advised. The reason for this is that after some time that the spur was built, the coast would obtain some stabilization and it can be maintained in the future. Therefore, by studying of various types of effective coast protection constructions, tow following method considering the previous performed works are suggested.

1. Building of spur besides the longitudinal protection buildings.
2. Reconstruction and maintenance of longitudinal buildings.

The first case would be most effective method to protection. Because, both used from the longitudinal protection buildings and also, by building of spurs the coastal areas will be stabilized.

Consequently, both the upward areas will be protected from inundation due to water level rising and the coastal zones are stabilized. Therefore, the destruction effects of sea are prevented by effecting of both spur and protection building with together. In the second case, which it is the feasibility of maintenance or reconstructing of longitudinal protection buildings, it has to be expressed this work in addition maintenance and rebuilding the coast, only will protect upward areas against the flooding. The manner of its maintenance has been already expressed.
The important point is that the constructing of spur will have the execution problems due to building inside of the water, consequently, it will have more costs. However, the favorite feasibility of its profit makes it more desirable.

4.9. The Results and Advises Must be Learnt
1. Carrying out of perfect studying of wave situations at least for a complete yearly period.
2. Special attention to the various stages of any protection structure.
3. To avoid of using of particular concrete units which they are locked with together like as Tetrapodes weightier than 15 tons.
4. Special attention to the geotechnical aspects in controlling of section stability of protection walls, particularly due to dynamically waves’ force.
5. Perfect performing of hydraulic models studying with the careful method by the specialists team and equipped laboratory.
6. Special attention to prevention of gradually erosion consequently damage of protection construction.
7. Gaining enough confidence of technical specifications of using cement compared with the standards specifications.
8. Not too much displacing of protection blocks which this case will produce the breaking cracks.
9. Special attention to using of resistant materials against the erosion and scouring.
10. Regular setting of concrete units or armor stones besides together making Max interlocking between them.
11. Using of blocks with high consistency degree and coarse and rough corners and considering less consistency in design.
12. Using of scouring toe to prevent of toe scouring.
13. Continuously controlling in execution process by the experienced team in this case.
14. Continuously controlling of project after constructing and removing the defects.
Report Title: Investigation on the effects of Caspian Sea upon its coast and presenting suitable protective solutions
Cross-Section of Bandare Torkaman's Breakwater Type.
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