

## CHAPTER ONE HUNDRED EIGHTY

### ASPHALT STABILIZATION OF RUBBLE SLOPES

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#### ABSTRACT

During the spring of 1982 a new container terminal was under construction at the Port of Haina, Dominican Republic. Changes in design conditions due to damages to the protective breakwaters at the port entrance required modifications to the design of slope protection under the pier. The stabilization of the rubble slope was, in fact, a stop gap measure in that repairs would be instituted on the breakwaters in the near future.

Due to a lack of suitable stone, the location of structural elements already in place and severe time constraints, an asphalt coating over relatively light weight rubble was selected to armor the slopes under the pier. The asphalt coating was prepared in a locally available batching plant and placed on-site using a crane equipped with a 4 cubic yard bucket. As a result of the use of the asphalt coating, construction delays were minimized and adequate slope protection was provided for the container pier without extensive design modifications.

The purpose of this paper is to discuss the advantages and disadvantages of the use of asphalt in the construction of shore protection facilities in the context of the site specific application at the Port of Haina.

#### INTRODUCTION

The Dominican Republic occupies the eastern portion of the island of Hispaniola in the Caribbean Sea. The Port of Haina is located on the southern coast of the Dominican Republic at the mouth of the Haina River, as shown in Figure 1. The Port is approximately 12 kilometers west of Santo Domingo, the capital city of the Dominican Republic.

The Port of Haina is a fully developed port complex with a wide spectrum of cargo handling capabilities. The complex includes a major sugar processing and loading facility, fertilizer and ferronickel loading wharf, Sealand container offloading facility, diesel fuel unloading area, container wharf under construction as described herein and a proposed coal offloading facility for a planned generating station.

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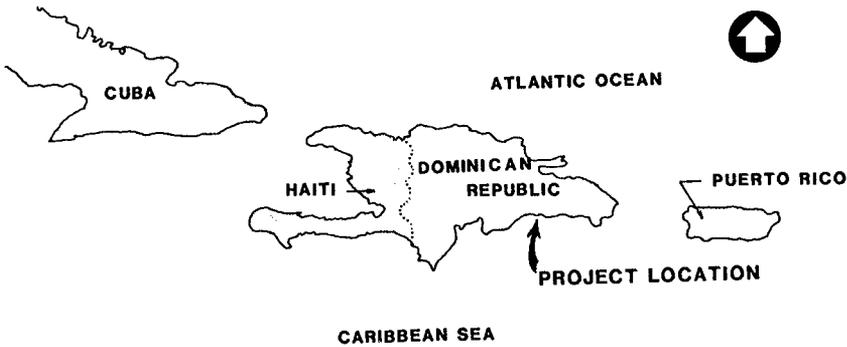


Figure 1  
Location Plan

The harbor is protected against incoming waves by two breakwaters as shown in Figure 2. During the fall of 1979, the site was affected by two hurricanes, Frederick and David, which resulted in considerable damage to the breakwaters. 110 meters of the east breakwater and 65 meters of the west breakwater were essentially destroyed. Since that time, progressive deterioration of the structures has resulted in increased exposure of the south end of the container terminal which was under construction.

The design of the container terminal pier included a pile supported relieving platform constructed over a rock armored dike which surrounded fill material used for the container storage area, as shown in Figure 3. The armor layer consisted of stone with a median weight of 300 kilograms and an average specific gravity of 2.6. Based on Hudson's equation (Reference 1), using a design slope of 1:1.75, this stone was found to be stable for wave heights of up to 1.25 meters. At the time of this study, in April 1983, exposure of the site had increased to a point where the original one meter design wave height could be as high as 2.6 meters. Since this wave height was in excess of the stable wave height for the proposed stone armor layer, a study was initiated to determine alternative methods of temporary reinforcement.

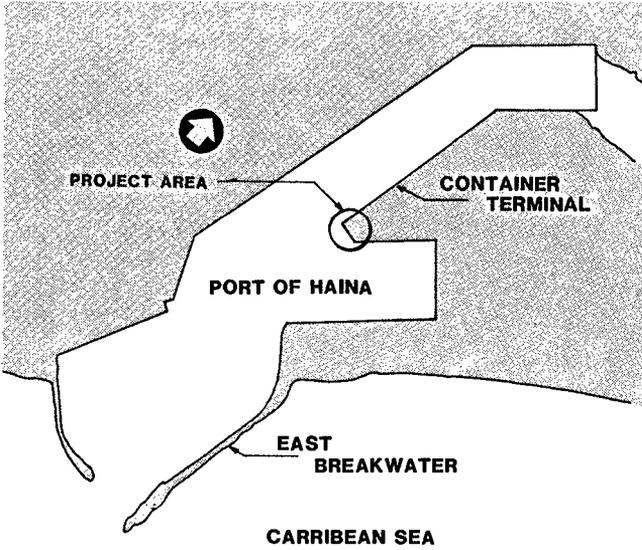


Figure 2  
Site Plan

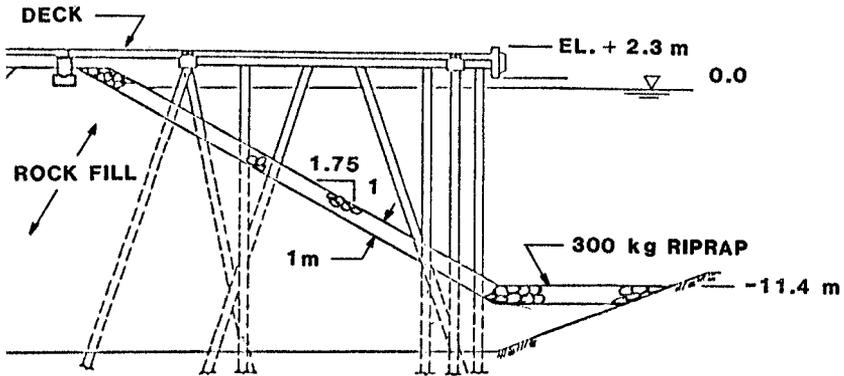


Figure 3  
Typical Section of Container Berth As Designed

It should be noted that at the time of this effort, a substantial amount of construction was completed on the container berth. Piles and pile caps were in place and the majority of the deck was completed. Figure 4 provides an aerial view of the facility around this time. Only the southern end of the structure was of concern since the deterioration of the breakwaters resulted in a high exposure level to southerly waves at this "head" section. A total of approximately 150 linear meters of the structure were involved.



Figure 4  
Container Facility Under Construction  
(View from the north)

Modifications to the original design of the section were not instituted earlier in the project because reconstruction of the breakwaters was anticipated. This reconstruction, which would have reduced the wave action to an acceptable level, was delayed for a variety of reasons beyond the scope of this discussion. Suffice it to say that the net result was a severe time constraint on design and construction of any possible alternative. Due to the inherent difficulties associated with the repair of the rubble dike under the relieving platform, completion of the head section was not judged to be prudent as discussed in the following paragraphs.

## ALTERNATIVES CONSIDERED

Various slope protection alternatives were considered in light of the expected 2.6 meter design waves, including:

- increase the primary armor stone size
- addition of artificial armor units to the slope
- placement of concrete filled mats
- construction of a wave skirt on the fender panels
- placement of asphalt grout or stone asphalt on the slope protection stone

The following paragraphs discuss these alternatives and present the rationale for the final selection of asphalt grout. The basic criteria utilized to assess the effectiveness of the alternatives were:

- constructibility
- cost
- engineering adequacy.

**Do Nothing:** The do nothing alternative implied construction of the dike slope protection as originally designed and acceptance of damages during periods with wave heights exceeding 1.25 meters at the structure. This alternative was deemed unacceptable due to the unknown period of time until the breakwaters were repaired and the consequences of a slope failure. In addition, after construction of the deck was completed, a major reconstruction effort would be required to make repairs to the rubble slope.

**Increased Stone Sizes:** Utilizing Hudson's equation for quarystone randomly placed in two layers, the required median stone size was found to be on the order of 2.3 metric tons for the design wave of 2.6 meters. Increasing the size of the rock used as the primary armor layer in this area to 2.3 metric tons was not an acceptable option since rock of the size and quality necessary is not locally available. As a consequence, material would have to be brought in from distant sources at high cost and would result in unacceptable delays in the completion of the facility. Discussions with the contractor on site confirmed this finding.

Additionally, the increased thickness of the required two layers and underlayers would necessitate removal of materials already placed in order to avoid intrusion on the berth area, and placement between piles would be, at best, difficult. As a result of the high anticipated costs, delayed construction and the encroachment problem, this alternative was ruled out.

**Concrete Filled Mats:** Concrete filled mats have been used in some shore protection applications and in numerous hydraulic structures. They are not, however, generally used in structures subject to large wave heights. Several questions were raised about the application of concrete filled mats to the project at the Port of Haina, including:

- the effects of the expected increase in wave runup due to the relatively smooth slopes
- the possibility of cracking the relatively thin mats during design wave events and disintegration of the slope
- lack of availability on short notice of the mats and equipment necessary for placement

Further, placement between existing piles and caps would be difficult. Consequently, this alternative was dropped from further consideration.

**Wave Skirt:** An indirect method of increasing the stability of the rock dike involved reducing the wave heights affecting the slope. Due to the relatively low deck elevation, a wave skirt placed below the fender face would function on the principle of stopping the wave energy in the upper part of the water column from reaching the rock dike under the structure. Energy during storm conditions would be decreased on the dike via reflection from the skirt and overtopping of the deck. Although technically feasible, this type of structure has not apparently been employed for this purpose. Additionally, structural connections would be difficult and might cause excessive loadings on the deck structure. It was felt that this option would require a significant amount of engineering, and further that construction might also be somewhat complicated causing further delays in the project. This option was not considered appropriate for further consideration.

**Asphalt Grout Or Stone Asphalt:** The final alternative consisted of the placement of an asphaltic compound in the interstices of the armor stone; or mixing armor stone with an asphaltic compound and placement on the structure. The asphalt then acts as a binder increasing the effective "weight" of the armor layer and therefore its stability. This type of approach has been employed successfully in both Holland (References 2 through 7) and the United States (References 2 , 5) on similar projects in the past. The material can be placed either as a grout or as a stone asphalt layer.

The use of asphalt had several distinct advantages for stabilization of rubble slopes, particularly from the standpoint of this project. First, the basic materials were available locally. Secondly, construction could be accomplished with standard construction equipment, e.g. cranes and buckets already at the site. Third, the basic cross section of the dike would be essentially unchanged so encroachment in the berth area would not pose a problem. Construction could presumably be accomplished before placement of the deck sections without significant delays in the contractor's work. The end result then would be stabilization of the structure at a relatively low cost to the owner.

Discussions with a specialty contractor involved in this type of construction revealed that the mixes utilized, although slightly different from routine pavement asphalt, could be manufactured in any asphalt plant in good working order. A limiting wave height on these structures of around 4.0 meters is normally assumed. This level was certainly within those anticipated at this location, and this alternative was selected for further analysis.

## HISTORICAL USES OF ASPHALT

A review of available literature (References 2 through 7) was conducted to develop a basis for the design of the asphalt grout for use at the Port of Haina. Based on this review, it was found that asphalt had been used in marine structures since the late 1920's, but it was not until the early 1930's that systematic applications began (Reference 3).

Some of the earliest uses of asphalt involved the construction or rehabilitation of groins, primarily along the coast of Holland. These structures utilized asphalt both as a grout over stones and in sand mixtures which could stand alone. Typical

mixes used in the asphalt grout included 70-72 percent sand, 10 percent filler and 18-20 percent asphalt (Reference 3).

Later uses of asphalt in marine structures along the coast of Holland included an asphalt and sand mixture for the Harlingen Harbor Breakwater. This structure, constructed in 1948, was subject to severe overtopping due to its low crest elevation. A second structure, the breakwater at Scheveningen, was grouted using an asphalt mix during this same year. Eight ton blocks on the Scheveningen Breakwater were successfully grouted during a repair project (Reference 3).

A number of additional examples of the use of asphalt and asphalt grout were found throughout the European Coastline (Reference 6). These included the asphalt grouting of 4 ton blocks forming the foundation of a vertical wall breakwater located in Marseilles, France. The blocks were located at a depth of approximately 12 meters (40 feet). A mix consisting of 60 percent fine sand, 20 percent limestone filler and 20 percent bitumen, 40-50 penetration was used. The asphalt was lowered in a bucket which was opened just above the rock surface. The initial temperature of the asphalt was 180 degrees C. Grouting of large concrete blocks was also accomplished at the River Adour northern breakwater also located in France. At this structure, 15 ton concrete blocks were grouted. Another example was the Pointe de Grave breakwater in France, where three to five ton stones were grouted using a mix consisting of 12 percent filler, 34 percent beach sand up to 6.0 mm, 34 percent dune sand up to 0.05 mm and 13 percent bitumen, 60-70 penetration.

At Porto Levante on the Po Delta in Italy, an asphalt grout consisting of 70 percent fine sea sand, 10 percent filler and 20 percent bitumen, 40-50 penetration, was placed on a breakwater at temperatures of 180 to 200 degrees C. Finally, a breakwater at Hirtshals Harbor in Denmark was grouted from 4.0 meters below the zero datum to 1.5 meters above. The mix used on this structure consisted of 72 percent sand, 10 percent limestone filler and 18 percent bitumen, 80-100 penetration.

In the United States, one of the earliest and most successful uses of asphalt grouting was during the rehabilitation of the Galveston, Texas jetty in 1935-1936 (References 2, 5). The armor on the south jetty at Galveston consisted of 6 to 10 ton stone, and the jetty cross section had deteriorated and become porous. After several mixes were tried, the jetty was grouted with a mix consisting of 70 percent Galveston beach sand, 12 percent Mississippi River loess (filler) and 18 percent asphalt cement, 30-40 penetration. The asphalt was placed at 204 degrees C (400 degrees F.).

## SELECTED DESIGN

The evaluation of the available literature on the use of asphalt in shore protection structures made one major factor eminently clear: very little published information exists on the design of such structures. The choice, however, was still desirable given the historical successes of the basic method, the rapid construction necessitated by project requirements, locally available materials and relatively low cost.

The basic design for this project consisted of placement of asphalt as a grout to fill the interstices of the existing primary armor. From the top of structure to a level 4.0 meters below the datum, the asphalt would be placed as a continuous cover. The lower level was selected since it is approximately 1.5 times the design wave height below the surface. Below - 4.0 meters, the material would be "pattern"

grouted to result in approximately 50% coverage. The purpose of the pattern grouting was to permit relief of pore pressures in the rock-fill. Penetration of the asphalt to a depth of 0.3 meters was judged to be adequate to bind the primary armor. The design section is shown in Figure 5.

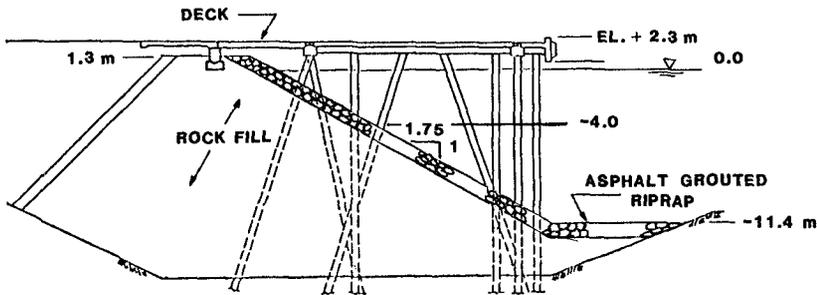


Figure 5  
Revised Cross Section With Asphalt Grout

Critical to the successful placement of the asphalt was the determination of a reasonable mix for the material. Some valuable information was obtained from work by Mulders (Reference 8), however due to the lack of available detailed design information, selection of the mix was an iterative process.

The initial mix selected for use is shown in Table 1 along with several iterations leading to the final mix. The initial proportions were selected based on information in the literature from past uses. Prior to placement, however, the mix was modified because of the highly fluid nature of the original mix. It appeared that penetration into the rock might be too great therefore increasing the volume of asphalt necessary to complete the work.

The second mix, with 25% one half inch aggregate, retarded flow of the material in the interstitial spaces to a greater extent than desirable. The final mix shown in Table 1 resulted in a reasonable level of flow and good filling of the voids in the armor layer.

TABLE 1  
ASPHALT MIXES

	INITIAL MIX	2nd TRIAL	FINAL MIX
ASPHALT	15	12	12
SAND	70	52	58
FILLER	15	11	15
GRAVEL	—	25	15

- NOTES: 1. Asphalt penetration 60-70  
2. Asphalt mixes and temperatures were controlled to some extent by the capabilities of the existing plants

Placement temperatures ranged from a low of 99° C (210 °F) to a high of 215° C (410°F). Both the low and high ends of the temperature range were troublesome since low temperatures resulted in poor flow; high temperatures in a spongy mass in the water and excess steam formation. The majority of the material was placed at approximately 165° C (330° F) which resulted in good flow and reasonable under water placement. The asphalt was supplied from a plant approximately 10 kilometers from the site. Delivered to the site in 15 cubic meter batches by truck, the material was dumped into a rock bin (steel holding box) as shown in Figure 6.

The material was placed on the structure with a 4 cubic yard clamshell as shown in Figure 7. When placed above water workmen utilized shovels and rakes to move the asphalt as required, although on the slopes it flowed well. Below water, the clamshell was opened near the water surface and material was dropped in mass to the rock surface. The pattern was obtained by locating the bucket relative to the pile caps and horizontal members already in place.

Two hundred and sixty cubic meters of asphalt were delivered and placed on the structure. With an adequate supply, approximately 60 cubic meters were placed in a working day. A total of 1150 square meters of rock protection surface between elevation +1.3 and -4.0 meters were completely grouted; 1450 square meters from -4.0 to -10.8 meters were pattern grouted with approximately 50% coverage achieved. Figures 8 and 9 provide views of the above water section and near the water line, respectively.

Unfortunately, photographs below water are not available, however sections under-water were inspected by divers who reported both adequate penetration, coverage and a durable mass.



Figure 6  
Asphalt Being Dumped into Rock Bin at the Site



Figure 7  
Asphalt Placement With Four Cubic Yard Bucket



Figure 8  
**Asphalt Grout Partially in Place Near the Crest**



Figure 9  
**Asphalt Grouted Rock Slope Near Water Line**

## SUMMARY AND CONCLUSIONS

Initial placement of the asphalt began on 26 April 1983 and was completed on 11 May 1983. As a result of the timely manner in which the redesign and construction of the armor layer was conducted, the contractor was able to maintain his work schedule and ultimately completed overall construction ahead of schedule.

Based on the experience gained on this project in the use of asphalt grout at the Port of Haina, the major advantages of the material included:

- the use of locally available materials
- a flexible finished structure
- minimized armor stone size
- relative ease of construction

The disadvantages of the asphalt grout included a lack of available design criteria, potentially increased runup and pore pressure buildup. One further aspect of the use of asphalt which may be of great concern in some areas is the environmental impact of placement in the water. Although no information could be found in the literature, several on site observations are worthy of note.

Placement of the material resulted in a substantial cloud of steam which was anticipated because of the high temperatures. Shortly after placement, a film of oil was visible on the water surface which dissipated relatively quickly. Fish normally seen around the structure returned within an hour or two of placement and seemed to be feeding on the asphalt surface. It is difficult to draw any firm conclusions from these observations, and environmental impacts of this method of slope stabilization should be considered where appropriate.

In conclusion, the placement of asphalt at the Port of Haina project has manifested itself as a technically and economically feasible method of increasing the stability of the stone armor layer to withstand wave attack. Inspection of the armor layer after slightly more than one year indicated that the asphalt grouted slope was stable and performing satisfactorily. Several periods of severe wave conditions occurred during this time, although it is unlikely that the design conditions were experienced.

## ACKNOWLEDGMENTS

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