

CHAPTER 209

BERM BREAKWATER FAILURE AT ST. PAUL HARBOR, ALASKA

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1.0 INTRODUCTION

St. Paul Island, Alaska, is located at 50°10'N latitude and 170°15'W longitude in the south central Bering Sea. It is the most northward and largest island of the Pribilof Island group. The area of the island is about 70 square miles (180 square kilometers), with the city and harbor of St. Paul located at a cove (Village Cove) on the southern coastline. The Pribilof Islands are of volcanic origin and are generally hilly with much of the coastline consisting of precipitous rocky cliffs. Moderate to strong winds are characteristic throughout the year, causing the island to be treeless. It is predominantly covered with grasses, sedges, and wild flowers.

The Pribilofs are a natural haven for a variety of flora and fauna. More than a quarter of a million seabirds nest each year along the coastal cliffs. About two-thirds of the world's population of northern fur seals migrate annually to the Pribilofs for mating purposes. The Pribilof Island area of the Bering Sea is also one of the most abundant and richest seafood grounds in the world. Due to a recent moratorium, the harvest of fur seals in the Pribilofs has been discontinued. In order to maintain existing cultural and economic resources, the City of St. Paul has elected to construct a harbor facility at Village Cove to provide services to commercial fishing vessels operating in the central Bering Sea. The maximum natural water depth in the Village Cove area is 26 feet (7.9m) relative to mean lower low tide datum (MLLW=0.0). Mean higher high tide level is 3.2 feet (1m) above MLLW, with extreme high tide during storm periods being estimated at between 5.0 and 6.0 feet (1.5 to 1.8m) above MLLW. Waves approaching from the southwest sector have the most effect on St. Paul Harbor. During the winter months, breaking waves with heights of 25 feet (7.6m) and 13-16s periods can be expected at Village Cove several times each year.

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2.0 1984 BREAKWATER CONSTRUCTION

2.1 ORIGINAL DESIGN

The St. Paul Harbor was to be constructed in Village Cove using a shore-connected breakwater commencing at the base of Village Hill, and extending approximately 2,000 feet (600m) in a north-northwest direction. The original design of the rubblemound breakwater followed a conventional 3 layer system. This consisted of a quarry stone core protected by 2 layers of 8-17 ton stone (Class II). The seaward slope of this structure would have been 1 vertical to 2.5 horizontal and have a crest elevation of +30 feet (9m) MLLW. The head section was to be constructed using a heavier armor layer of 17-24 ton stone (Class I) with a slope of 1:3. A typical cross-section is illustrated in Figure 1.

Due to a low and insufficient production of Class I and II armor stone at the selected island quarry sites, the contractor and engineer agreed to redesign the breakwater to better suit the quarry stone production. The modified design was then based on the berm breakwater concept, hereby referenced to as the "A-B" design. In theory, this approach to breakwater design would have maximized the use of all quarry stone by minimizing stone by-product, and therefore resulting in a more economical unit stone production cost. The breakwater was then completed to a length of approximately 870 feet (260m) by early October of 1984 using the "A-B" design.

2.2 MODIFIED BREAKWATER DESIGN ("A-B" DESIGN)

The "A-B" design called for an outer layer of "A" stone which ranged from 0.75 tons to 8 tons with a median stone size of 1.5 tons. A 60-foot (18.2m) wide berm was placed on the seaward side. The core material ("B" stone) also composed an outer berm section, with a gradation similar to the original Class V core stone. The crest height of the "A-B" breakwater was +28 feet (8.5m) MLLW (see Figure 1). This design was based on 2-D tests conducted at the Danish Hydraulic Institute (DHI). The tests used wind, wave, and storm parameters developed in a Pribilof Island wave study prepared by DHI in June 1982.

The breakwater was completed with a temporary head section consisting of "A" and "B" stone only. No armor stone of the Class I or II type was placed on the head as an armor layer. It was assumed at that time that construction would continue the following year.

2.3 BREAKWATER PERFORMANCE WITH "A-B" DESIGN

Substantial damage to the breakwater resulted from storms occurring on 13 November and 7 December 1984. A hindcast analysis showed that the first storm produced a deepwater significant

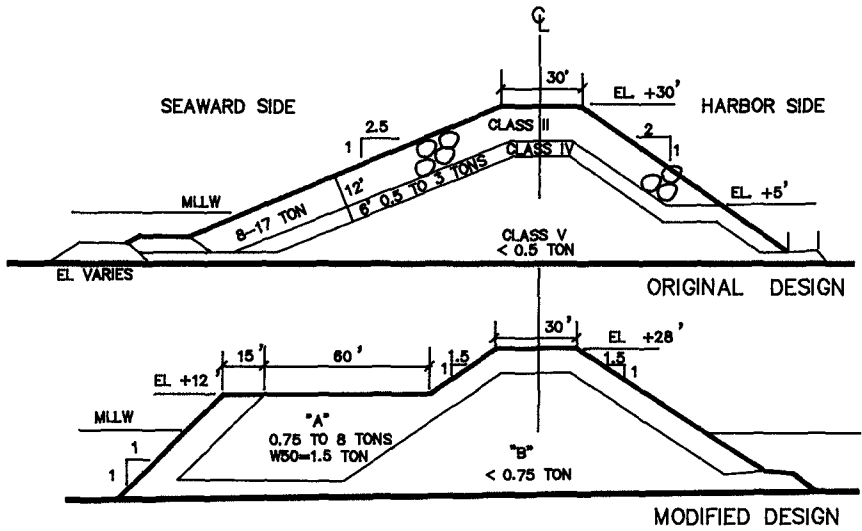


FIGURE 1 BREAKWATER CROSS SECTIONS

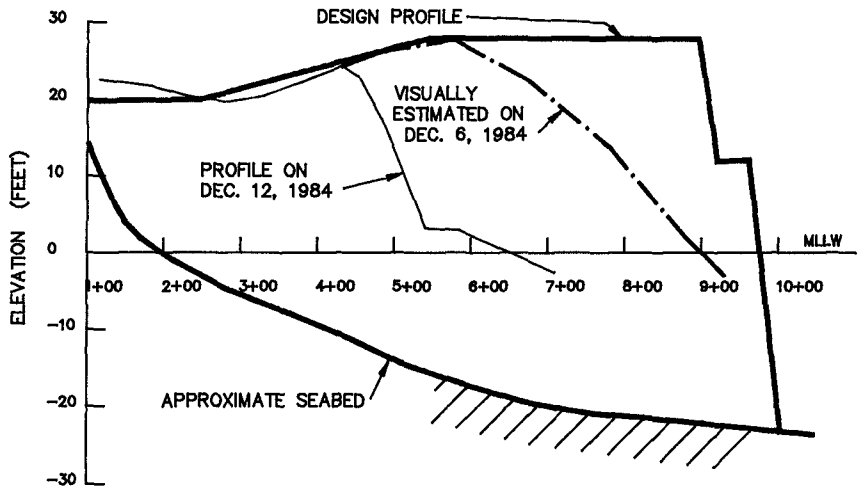


FIGURE 2 COMPARISON OF CENTERLINE PROFILES

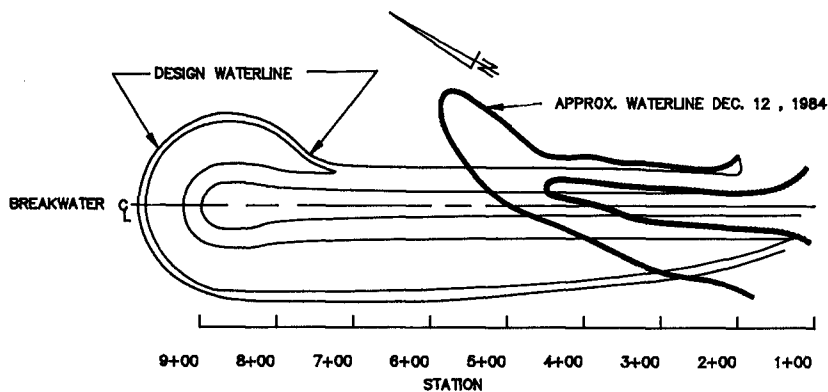


FIGURE 3 BREAKWATER PLANVIEW

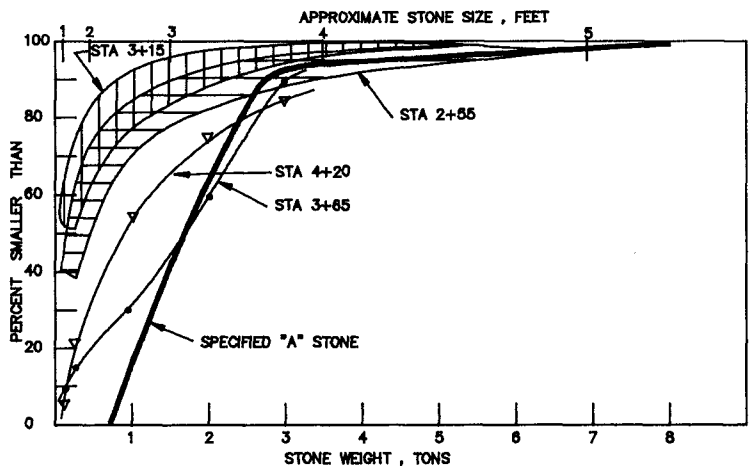


FIGURE 4 ESTIMATED GRADATION OF "A" STONE

wave height, H_s , of 30 feet (9.1m) with peak periods of 16 seconds, and the second storm had an H_s of 22 feet (6.7m) with 13 second peak periods. Storms of these magnitudes can be expected to occur several times a year in the Bering Sea. Tide levels were estimated at +3.5 to +4.0 feet (1.1 to 1.2m) MLLW for both storms. The 13 November storm resulted in considerable redistribution of the "A" stone along the entire structure length. Approximately 300 feet (91m) of the original crest and 100 feet (30m) at the waterline were lost. Data gathered after the 7 December storm indicated that an additional 200 feet (60m) of crest and 250 feet (75m) of waterline were lost, for a total damage of 500 feet (150m) of crest and 350 feet (100m) at the waterline. A comparison of centerline profiles before and after the storms is shown in Figure 2. It was evident that material at the head section was transported into the harbor area and formed a low, wide underwater mound east of the initial centerline, producing a reef type structures. Plan views of the before and after structure are shown in Figure 3.

Independent gradation estimates were made at various locations along the breakwater above the waterline. The results of this work showed that the in-place "A" stone outer layer was generally finer than the specified gradation for "A" stone (see Figure 4). Some signs of slumping on the harbor side south of STA 4+50 slope were also visible.

2.4 DISCUSSION OF DAMAGE

During the field investigation, slope measurements indicated substantial adjustment of the outer "A" stone layer, similar to that experienced in the DHI 2-D tests. However, north of STA 4+50 and with respect to the construction of the head section, the excessive damage was due to the fact that the "A" stone size used was inadequate for the design wave conditions of 20-25 foot (6-7.6m) breaking waves with periods ranging from 13-16 seconds. At the head, the direction of wave attack will be at an angle which will cause displaced stones to travel laterally and into the harbor and entrance channel. The head then receded until the water depth limited the wave height to less than 10 feet (3m), where a 1.5-2 ton stone on a 1:5 slope can be stable.

Breaching of the breakwater trunk was also witnessed by City of St. Paul officials during the 13 November 1984 storm. The reduced porosity of an extremely well-graded material, such as "A" stone in combination with "B" stone, probably increased the run-up potential of the design wave conditions. This, in turn, resulted in excessive overtopping, and finally breaching of the "A-B" breakwater.

3.0 BERM BREAKWATER DESIGN CONSIDERATIONS

The investigation on the construction of this berm breakwater, as well as the events and circumstances that culminated in its

completion, have raised various items of concern when designing and constructing these types of structures. Several of these items are presented below.

1. The procedures for production and inspection of armor stone for any rubblemound type structure are extremely important. It may be more difficult to determine if an armor stone class with a wide gradation meets design specifications, as compared to conventional armor stone with a narrow gradation. The performance of a berm breakwater, as well as the potential degree of damage, would also have to be assessed if the median stone size in the berm armor stone is less than that specified. Does a small reduction in median stone size or skewed gradation result in a disproportionate degree of damage under design conditions?
2. The determination of potential long shore movement of material along a berm type structure should be addressed in order to evaluate its long-term maintenance needs, and consequently estimate the annualized maintenance costs. A design with a significant savings in capital costs may not be the most economical if the maintenance costs are excessive.
3. Due to the inherent capability for the seaward slope of berm type structures to adjust in direct relation to the impinging sea state, some guidelines need to be established for the definition and assessment of potential damage.

These are a few of the items that need to be addressed by the engineer and planner when evaluating berm type breakwaters.

4.0 BREAKWATER - 1985 TO PRESENT

Following the "A-B" breakwater damage in 1984, the Alaska Department of Transportation hired Tetra Tech, Inc. to develop a breakwater damage assessment. The City then retained Tetra Tech, Inc. to provide assistance in re-designing the breakwater, designing a 200-foot (60m) length dock, preparing plans and specifications, and advertising and awarding construction contracts. A 25-foot (7.6m) breaking wave was selected for the re-design, which required armor stone of 14-ton on the breakwater trunk section and 18-ton on the head section. Seaside trunk slope was 1:2.5 and 1:3 for the head. This redesign also incorporated the criteria of near-zero percent annualized maintenance (minimum 50 year design life). Since the 200-foot (60m) dock, and future extension thereof to 1000 feet (300m) in length, would be positioned along the harbor side of the breakwater, the breakwater crest elevation was designed for no wave over-topping and established at +37 feet (10.1m) MLLW. The 200-foot (60m) long by 40 feet (12m) wide dock was a pre-stressed, pre-cast concrete caisson design. The caisson was constructed in Tacoma, Washington, towed to St. Paul Island, Alaska and placed on a

specially constructed foundation. During the period May 1985 to January 1986, the breakwater was constructed. The concrete caisson/dock was installed, and a 200-foot (60m) wide, 300-foot (90m) long channel dredged in the summer of 1986.

The breakwater with armored head has been subjected to the design wave a number of times during the 1985, 1986 and 1987 winter seasons. Visual inspections indicate no armor stone displacement along the trunk or head of the breakwater. The dock system has been utilized extensively for vessel off-loading of cargos which previously had to be lightered. St. Paul Harbor is a Federally Authorized Project and since December 1986, the U.S. Army Corps of Engineers (Alaska District) and the City of St. Paul have been carrying out further studies of the existing design for the St. Paul Harbor Project. This has included 2 and 3 dimensional hydraulic model studies, and complete re-evaluation of economic benefits for the project. At this time, findings of the hydraulic model tests indicate the seaward slope of the breakwater can be steepened to 1:2 using 18-ton armor stone. Presently it is planned to construct additional navigation features in 1989. This includes the extension of the breakwater length to Station 18+00, adding 700 feet (210m) of dock, constructing a 1000-foot (300m) second detached breakwater and final excavation of a mooring basin.