



Taichung Harbor, Taiwan, ROC—R.L. Wiegel

PART III

COASTAL STRUCTURES AND RELATED PROBLEMS

*Differential movement due to earthquake which occurred during 20th ICCE,
Hualien, Taiwan, ROC—R.L. Wiegel*



CHAPTER 126

Wave Transmission across Submerged Near-Surface Breakwaters

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Abstract: Wave transmission across a submerged breakwater at Santa Monica, California, is examined through a three-dimensional model test. The results agree with empirical criteria previously proposed by Tanaka (1976).

Introduction

In 1983, El Niño came to California. By the time it left, over 20 piers and breakwaters had been damaged, countless beaches had lost sand, and new records for high tide levels had been set. At Santa Monica, the municipal pier was severely damaged. The seaward 120 meters of the pier were completely destroyed as were 110 meters of the adjoining Newcomb Pier. This occurred despite the fact that a submerged breakwater partially protects the pier from offshore swell.

The submerged breakwater was to figure prominently in repair alternatives considered by the City as they sought a plan most responsive to their needs. A major issue was whether to rebuild the breakwater and utilize its protective ability in developing the pier repair plans or to concentrate the reconstruction effort in strengthening the pier and avoid expending resources on the breakwater. To aid in making this decision, the City's consultant, Daniel, Mann, Johnson, & Mendenhall (DMJM), commissioned a three-dimensional model study of the site to investigate the effect that various breakwater configurations had on the alternative pier designs.

This paper compares the data on wave transmission across the submerged breakwater at Santa Monica obtained from the model study to results of wave transmission studies presented by Tanaka (1976). The Santa Monica data tend to corroborate Tanaka's results, suggesting their use in design application.

Setting

Santa Monica is part of the Los Angeles metropolitan area. Figure 1 shows the location of the site. The breakwater and pier are at the head of Santa Monica Bay, a broad open body of water bounded by the Santa Monica Mountains to the north and the Palos Verdes Peninsula to the

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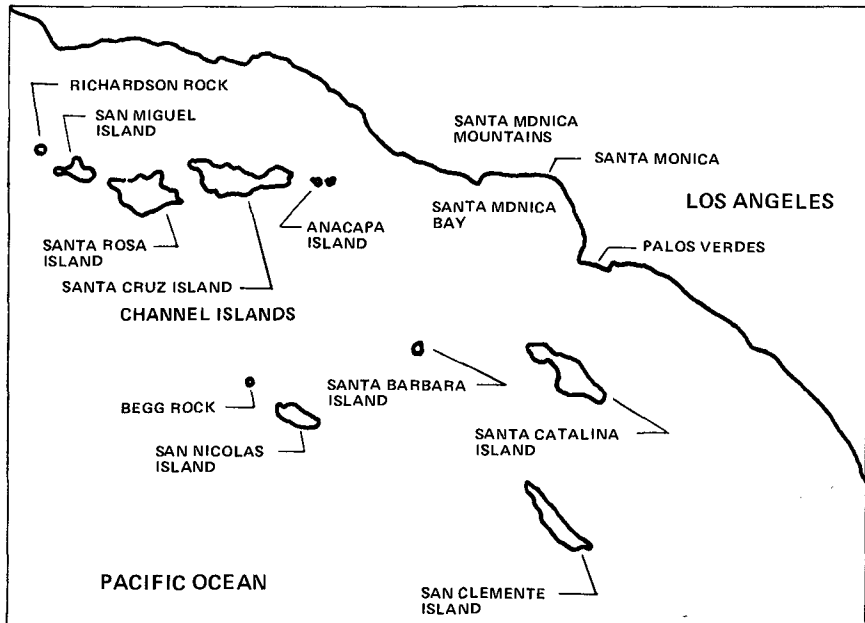


Figure 1. Site Location

south. The site is partially protected from offshore swell by Santa Catalina Island, Santa Barbara Island, San Nicolas Island, the Channel Islands, and Point Conception.

The Santa Monica shoreline is a broad sandy beach at the base of a 9 to 12 meter high bluff. Littoral transport is generally toward the south. Man-made structures such as the Santa Monica breakwater, the Marina del Rey jetties, and the Redondo Harbor breakwater interrupt this flow of sand at several points along the bay until the Redondo submarine canyon finally traps most of the remaining littoral drift at the southerly end of the bay and diverts it offshore.

History

A review of the history surrounding the Santa Monica breakwater will help in understanding several unusual aspects of its configuration. Man-made structures began to be constructed on Santa Monica Beach as early as the 1870s. The first structure on record was a small pier located near the site of the present pier. By 1876, a railroad pier supported by a substantial number of pilings had been built. In 1908, the first municipal pier was built; it failed 12 years later. The present municipal pier was built in 1921.

The Santa Monica breakwater was constructed in 1933 and 1934 by the City to provide a pleasure boat anchorage. The breakwater was

constructed parallel to the shoreline about 610 meters long and was located about 610 meters offshore. Figure 2 illustrates the breakwater configuration as it was constructed. The top elevation was 3 meters

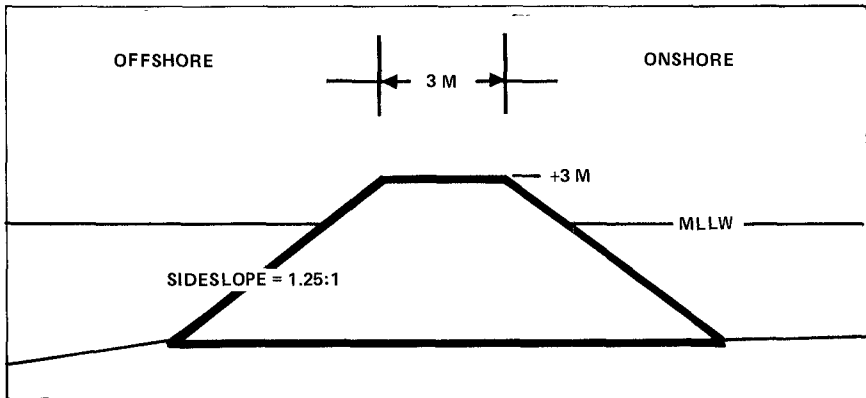


Figure 2. Original Santa Monica Breakwater Configuration

above mean lower low water (MLLW) and had a crest width of 3 meters. It had fairly steep sideslopes of 1.25 horizontal to 1 vertical. A rapid siltation of the anchorage followed the breakwater construction, and the beach behind the breakwater began advancing seaward. By 1937, as many as 245 anchorages had been lost to siltation. At the same time, the breakwater began deteriorating, and there are accounts of wave damage creating a gap in the breakwater as early as August 1934.

Based on survey data of a typical cross-section 30 meters from the northern end contained in the damage survey report of the Federal Emergency Management Agency dated July 21, 1983, the breakwater lost approximately 0.3 meter of its crest height after the first year of construction, another one meter by 1956, and an additional 0.2 meter by 1972. A survey of this breakwater after the 1983 storms showed the crest elevation of this section had lowered another 3 meters since 1972, although it is difficult to determine if the reduction was due to the storm or if it occurred gradually throughout the 11-year period. Figure 3 shows the site in 1975 and 1983 and illustrates the relative position between the pier and the breakwater. The shoreline bulge in the lee of the breakwater seen on the 1975 picture has retreated by 1983, possibly as a result of the deterioration of the breakwater. Figure 3 also illustrates the damage to the pier caused by the 1983 storms.

The 1983 configuration is illustrated in Figure 4. The crest height is -1.6 meters MLLW and the crest width at that level is 13.4 meters. Material from what was the upper portion of the breakwater has fallen down, creating a more stable sideslope of 2 horizontal to 1 vertical.

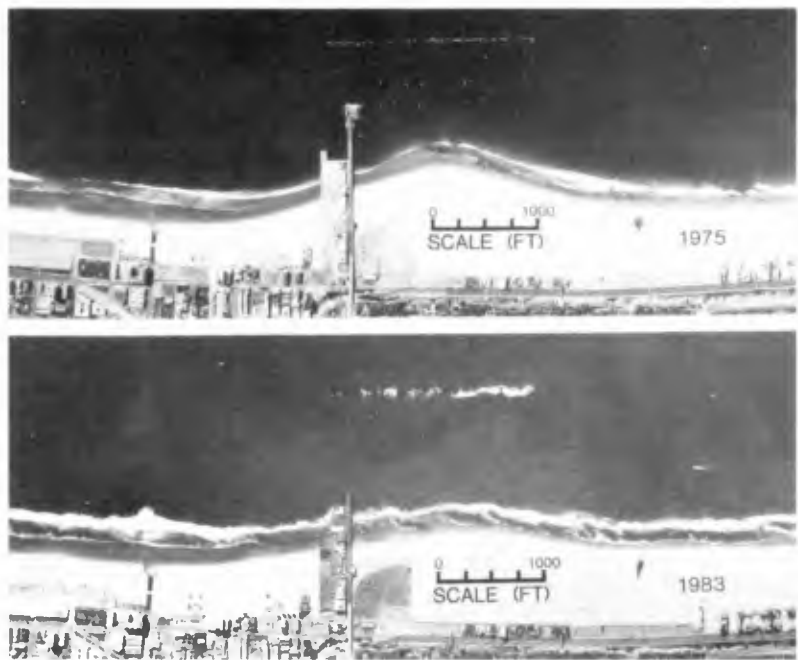


Figure 3. Comparison of Shorelines for 1975 and 1983

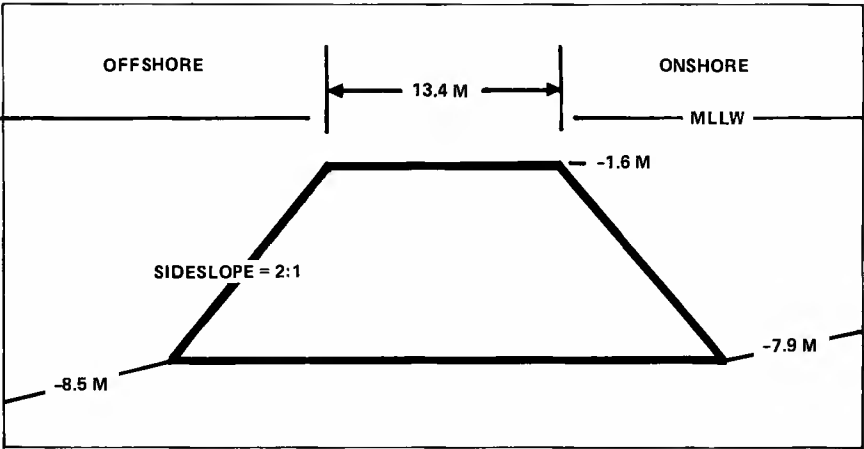


Figure 4. 1983 Breakwater Configuration

This was the condition of the breakwater when DMJM began its analysis. The breakwater was essentially submerged, especially at high tide, although the crest elevation was nonuniform in height and projected above the water surface in several locations. These complications played a role in the decision to utilize a model study in analyzing the wave action on the pier.

Model Study

The three-dimensional hydraulic model study was conducted by the Offshore Technology Corporation to determine the protection provided to the pier by various breakwater configurations. The results of the existing submerged configuration are analyzed in this paper. The model scale was 1 to 50. It covered an area of 37 meters by 24 meters, which represents 1,850 meters by 1,200 meters in prototype. Five breakwater configurations were modeled using rocks approximately scaled from the size specified in the prototype. Waves were directed at the site from three different directions, three different tide levels were studied, and six different significant wave heights were used.

The model bathymetry was constructed to model bathymetric survey data obtained in December 1983. Gravel was used to build up the model floor, and the final 0.025 meter finish layer was made of mortar. The prototype breakwater had a quarry run core, an underlayer of 900 kilogram (1 ton) stone, and an armor layer of 7,800 kilogram (8.6 ton) stone arranged in one and two layers. The model used a pea gravel core, 7 gram concrete aggregate for the underlayer, and 80 gram concrete aggregate for the armor layer. The stone size was modeled correctly, although the model armor was heavier than necessary to model the prototype rock. This was considered acceptable because the tests were conducted primarily to determine wave patterns and wave heights at the pier rather than armor unit stability.

The submerged breakwater had (prototype) crest elevations ranging from -1.8 meters MLLW to +1.8 meters MLLW and crest widths ranging from 11 meters to 24 meters. Waves were directed at the breakwater at an angle 13 degrees north of normal (235° azimuth). Water levels of 3.0 meters and 2.6 meters (prototype) above mean lower water were used in conjunction with tests of significant wave heights of 3.4, 3.9, 3.0, 1.9, 4.1, 3.2, and 1.9 meters. The waves were generated with periods having a frequency distribution matching the spectrum recorded on offshore wave recorders during the storm that damaged the pier.

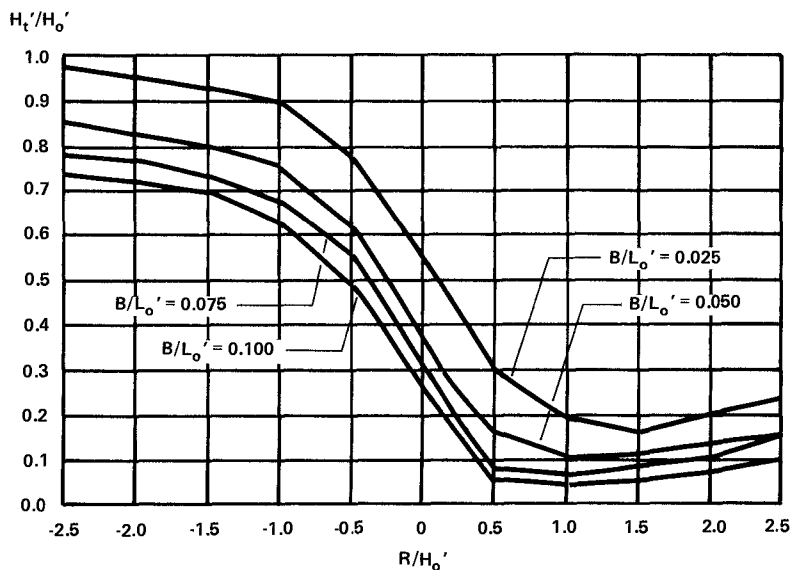
Wave heights in the model were measured using capacitance-type wave gauges. Wave gauge locations were concentrated in the vicinity of the pier, although a wave gauge immediately behind and another in front of the breakwater are of particular interest in this paper.

Previous Studies

Researchers have conducted investigations of wave transmission across permeable structures for over 30 years. Some of the earliest work may have been studies of wave filters used in front of laboratory wave

generators. Saville (1963) examined a number of structures with crest elevations near the still water level for a proposed breakwater at Point Loma, California. Goda (1969) tested impermeable breakwaters for transmission by overtopping and developed an empirical equation to predict transmission coefficients which was found to be a function of the breakwater freeboard. Cross and Sollitt (1971) presented a semiempirical treatment for overtopping of subaerial breakwaters in which transmission coefficients depend on the breakwater freeboard to wave runup ratio. Keulegan (1973) studied vertical faced permeable breakwaters. Sollitt and Cross (1976) developed an analytical-empirical model for transmission through permeable breakwaters. Madsen and White (1976) presented a model of wave transmission and reflection for subaerial rubble-mound breakwaters. Seelig (1980) combined the model for transmission through permeable breakwaters of Madsen and White (1976) with prior work on transmission by overtopping.

Tanaka (1976) found the wave transmission coefficient related to the relative submergence depth and the relative crest width. His work is unique in that it deals with a continuous spectrum of breakwater crests, including both negative and positive clearance, while at the same time considering a broad range of crest widths. The continuous spectrum of crest heights was particularly useful for our purpose because, depending on the restoration scenario, the breakwater might function alternatively as a partially submerged or partially emerged structure at different phases of the tide. The broad range of crest widths was likewise



SOURCE: TANAKA (1976)

Figure 5. Results of Tanaka Study

appealing because the scenario included both the existing deteriorated broad-crested breakwater and rebuilt highly emerged narrow-crested configurations. The crest width to wave length ratio ranged from 0.025 to 0.8.

As shown in Figure 5, Tanaka's criteria give the transmission coefficients H_t/H_o' as a function of B/L_o and R/H_o' , that is, as a function of crest width clearance, wave period, and wave height where B is the crest width, L_o is the deepwater wave length, R is the crest clearance, H_o' is the equivalent deepwater unreflected wave height, and H_t is the transmitted wave height. Tanaka found that for values of R/H_o' less than about -1.0, the breakwater produced little reduction in wave height and the transmission coefficient was not highly dependent on R/H_o' . For values of R/H_o' between -1.0 and +1.0, changes in R/H_o' produce large changes in transmission coefficient. For values of R/H_o' greater than +1.0, the breakwater was effective in reducing the wave height but the transmission coefficient was not highly dependent on R/H_o' and wave reflection was significant.

Comparison of Data

The general approach used in this paper is to compare the results from the Santa Monica model study to the results presented by Tanaka to see if his criteria predict the Santa Monica results. Figure 6 shows the Santa Monica results and the values that were obtained by applying Tanaka's criteria superimposed on the graph shown in Figure 5.

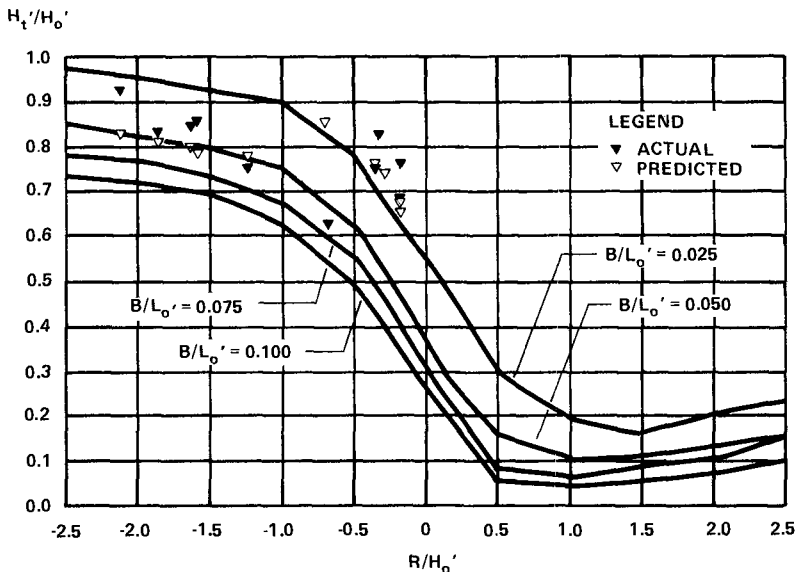


Figure 6. Santa Monica Data Comparison

Because Tanaka's criteria are based on monochromatic waves, our wave data were reduced to comparable parameters. Wave data in the Santa Monica tests were processed by the zero downcrossing method to obtain a frequency spectrum at each gauge. After several trials, it became apparent that parameters consisting of a significant wave height defined by

$$H_s = 4.0 (m_0)^{1/2}$$

$$\text{where } m_n = \int_0^\infty f^n S(f) df$$

$$S(f) = (1/2) a^2(f)$$

a = amplitude

f = circular frequency

and a peak period provided the best agreement with Tanaka's results.

It is interesting to note that with a crest elevation of 1.5 meters below MLLW, the breakwater offers little attenuation for waves under 4.6 meters in height.

Figure 7 shows the predicted values of the transmission coefficient plotted against the values of the transmission coefficient measured in the Santa Monica model study. The figure shows the degree to which

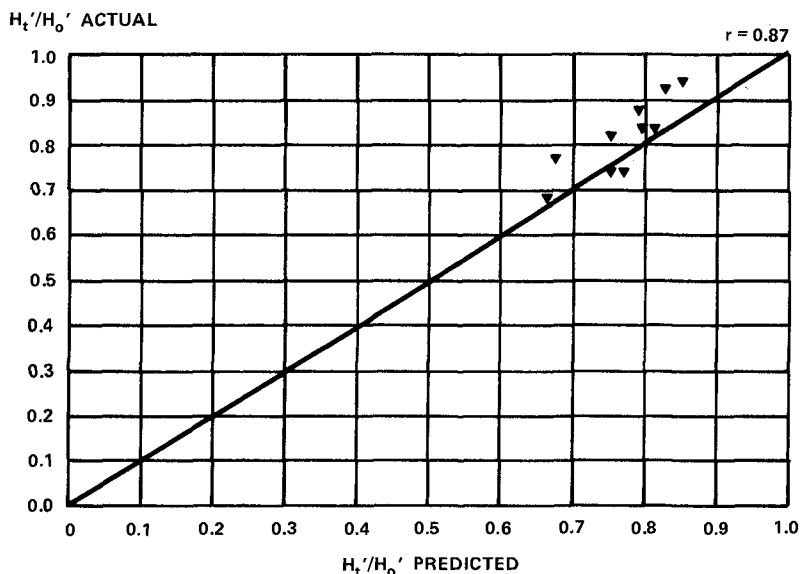


Figure 7. Actual vs Predicted Transmission Coefficients

Tanaka's criteria predict actual results. A perfect correlation would occur if all the data points fell on the diagonal line. Here we found r , the Pearson Product Moment Correlation Coefficient, to be equal to +0.87, essentially corroborating the criteria by Tanaka.

The figure indicates that the predicted transmission coefficients underestimate the actual results observed in the model test. This is demonstrated by the fact that most of the plotted points fall above the 45° diagonal line rather than being distributed evenly on either side. An interesting future study would be to determine if diffraction around the end of the breakwater in the Santa Monica model was the cause of this result.

Conclusions

Our conclusions are that the Santa Monica model data agree with the trend of Tanaka's findings. The correlation coefficient, a measure of this agreement, is +0.87. This conclusion lends credence to the use of Tanaka's curves as design tools when investigating submerged breakwaters. Tanaka's curves underpredict the transmission coefficients found in the Santa Monica model tests, so some care must be used in applying the curves. We also determined that except for very large waves, the existing breakwater at Santa Monica is not effective in attenuating wave height.

ACKNOWLEDGMENTS

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APPENDIX I, REFERENCES

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