COAST & HARBOR ENGINEERING

Technical Memorandum

Coastal Engineering Analysis and Assistance with Design
Boulevard Park Gravel Beach, Bellingham, Washington

Erosion and Sediment Transport Evaluation

1. Introduction

This Technical Memorandum summarizes the results of coastal modeling and analysis to determine the littoral impacts of the overwater walkway at Boulevard Park Gravel Beach in Bellingham, Washington. The objective of the study was to determine whether or not the walkway will affect erosion and sediment transport at the two landings, the shoreline reach between the landings, and the areas extending approximately 300 feet beyond each landing. This evaluation is required by the Mitigated Determination of Non-Significance, (MDNS), dated September 23, 2010 issued by the City of Bellingham Planning and Community Development Department.

2. Methodology and Input Data

The analysis of impacts from the overwater walkway was conducted based on the assumption that impacts would occur if at least one of two factors controlling sediment transport, erosion, and deposition in the project area changes: wave characteristics (wave period and wave height) and sediment transport potential in a swash zone[1]. Therefore, analysis and modeling was conducted to determine if construction of the overwater walkway can possibly change wave characteristics and sediment transport potential along the shoreline at the two landings, the shoreline reach between the landings, and the areas extending approximately 300 feet beyond each landing.

The analysis was conducted using a 2-Dimensional (2-D) wave refraction/diffraction numerical model SWAN ((Holthuijsen *et al.*, 2004). Modeling was conducted for existing (pre-project) and post-project conditions. Analysis of the potential impacts was based on comparison of wave orbital current velocities[2] and sediment transport potentials for existing and post-project conditions. A 25-year return period of occurrence wind-wave storm event was used as a criterion for the impact analysis. It is believed that the smaller (more

[1] Swash zone is the upper beach area where breaking of waves and dissipation of wave energy is observed. In this zone uprush and backwash, motions of waves mobilize and transport large quantities of sediment compared to other regions. Sediment transport potential characterizes a theoretical, maximum possible sediment movement by waves in a swash zone.

[2] In wave theory wave motion is described by orbital movement of water particles in a water column. When a wave interacts with the bottom slope, the orbital motion transforms to elliptical motion. The height of elliptical motion reduces with depth, and at the bottom layer this motion is presented by uprush and downrush motions. Bottom orbital velocity describes the maximum speed of water in this motion.

frequently recurring events) would not be as sensitive to the changes from the project; and therefore would be less likely to cause any changes or impacts.

Wind data to construct the 25-year return period storm event were obtained from the compilation and statistical analysis of long-term wind data. These data and the analysis are described in more detail in CHE's Technical Memorandum *Coastal Engineering Analysis and Assistance with Design Boulevard Park Gravel Beach, Bellingham, Washington* (CHE April 16, 2010). The results of the wind statistical and extremal analyses are shown in Table 1.

Table 1. Bellingham Bay, Return Periods of Wind Events from Wave-Forming Fetches

BELLINGHAM BAY, WA[1]
BOULEVARD-CORNWALL OVERWATER WALKWAY
RETURN PERIOD WIND SPEEDS (mph)
(1-min duration)

Return Period (yr)	Wind Direction (°T)															
	230	240	250	260	270	280	290	300	310	320	330	340	350	360	10	20
2	40.7	42.0	36.8	30.8	34.1	38.9	35.5	31.7	31.2	30.8	25.4	24.6	16.3	21.0	23.9	33.0
5	46.5	47.2	42.5	36.6	39.0	42.8	39.8	35.2	34.8	35.3	29.4	29.1	20.4	27.1	31.3	39.9
10	49.8	50.2	45.8	39.9	41.8	45.1	42.3	37.2	36.8	37.9	31.7	31.7	23.5	31.0	36.1	43.8
25	53.4	53.6	49.4	43.6	44.9	47.6	45.0	39.3	39.1	40.8	34.3	34.6	27.6	35.7	41.9	48.2
50	55.9	55.8	51.8	46.1	46.9	49.3	46.8	40.8	40.6	42.7	36.0	36.5	30.7	39.0	45.9	51.2
100	58.1	57.8	54.0	48.3	48.8	50.8	48.5	42.1	42.0	44.5	37.5	38.3	33.8	42.2	49.7	53.8

Notes:
[1]Period of record: 1973-2007

Based on previous analysis and sensitivity modeling, it was determined that the wind-wave fetch of 240°T produces the largest wave parameters at the project site.[3] Therefore, this direction (240°T) was used to generate the design storm events. All modeling was conducted at the MHHW tidal elevation.

An approach using two numerical modeling grids (large and nested) similar to that from a previous modeling study (CHE, April 16 2010) was used for the wave modeling and impact analysis. The large numerical modeling grid, which includes all of Bellingham Bay, is shown in Figure 1.

[3] The predominant wind direction for Bellingham Bay is from the south, with most winds in the 0-20 mph range up to and exceeding 30 mph. However, the project site is sheltered from southerly winds and direct southerly wind waves by the headland to the south, and the dominant remaining events are as shown in Table 1.

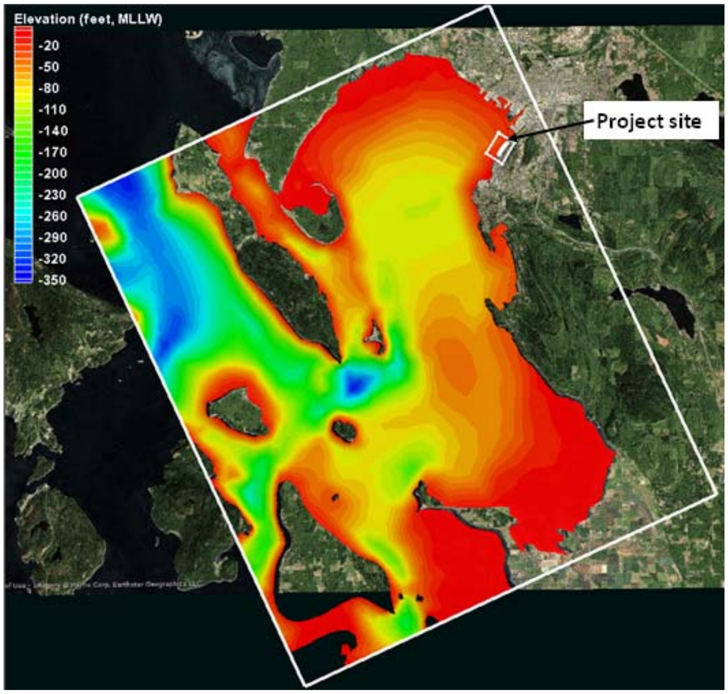

Figure 1. Large numerical modeling grid for Bellingham Bay

The nested modeling grid was modified to optimize the modeling effort and provide detailed information on wave parameters at the project shoreline, as well as along the overwater pedestrian walkway. A fine-detail nested numerical modeling grid was built at locations of overwater piles and along the shoreline. The nested modeling grid is shown in Figure 2.

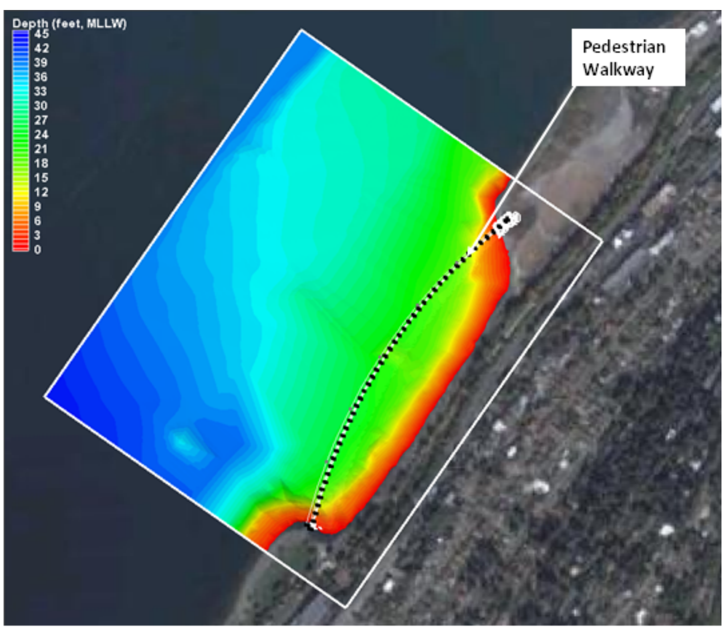

Figure 2. Zoomed-in nested numerical modeling grid for overwater pedestrian walkway

For the comparative analysis, 13 (thirteen) control stations were established on the modeling grid. Wave parameters (height and period) were obtained from the modeling results that were extracted from these stations and compared. The location of the controlling stations is shown in Figure 3.

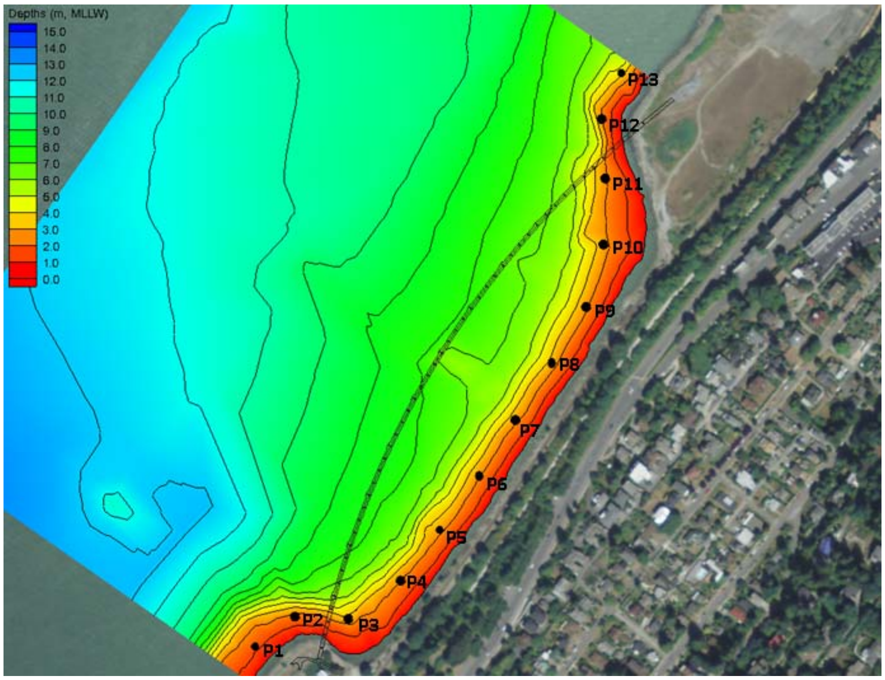

Figure 3. Controlling stations selected to extract the wave and bottom velocities

3. Modeling Results

Results of the modeling, wave heights and wave orbital velocities, are presented graphically in Figures 4a, 4b and 5a, 5b. Figures 4a and 4b show wave height distributions over the nested modeling grid for existing conditions (no overwater walkway) and for post-project conditions (with overwater walkway). Figures 5a and 5b show bed orbital velocities over the nested modeling grid for the same existing post-project conditions in color format. Figures 4b and 5b also show the alignment of the overwater pedestrian walkway.

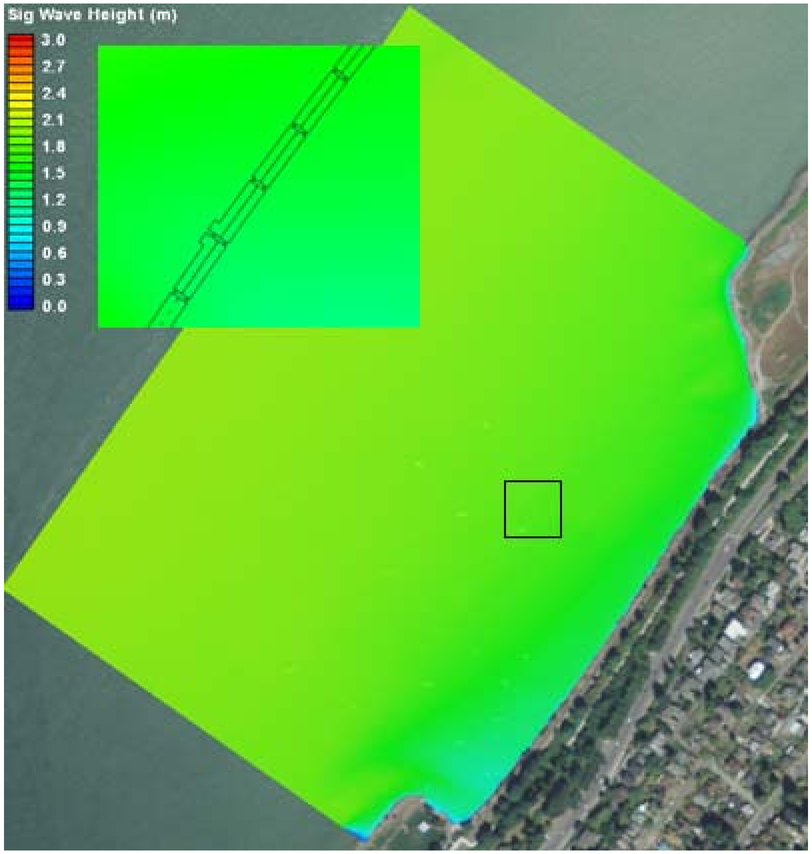

Figure 4a. Modeled Wave Heights for Existing Conditions

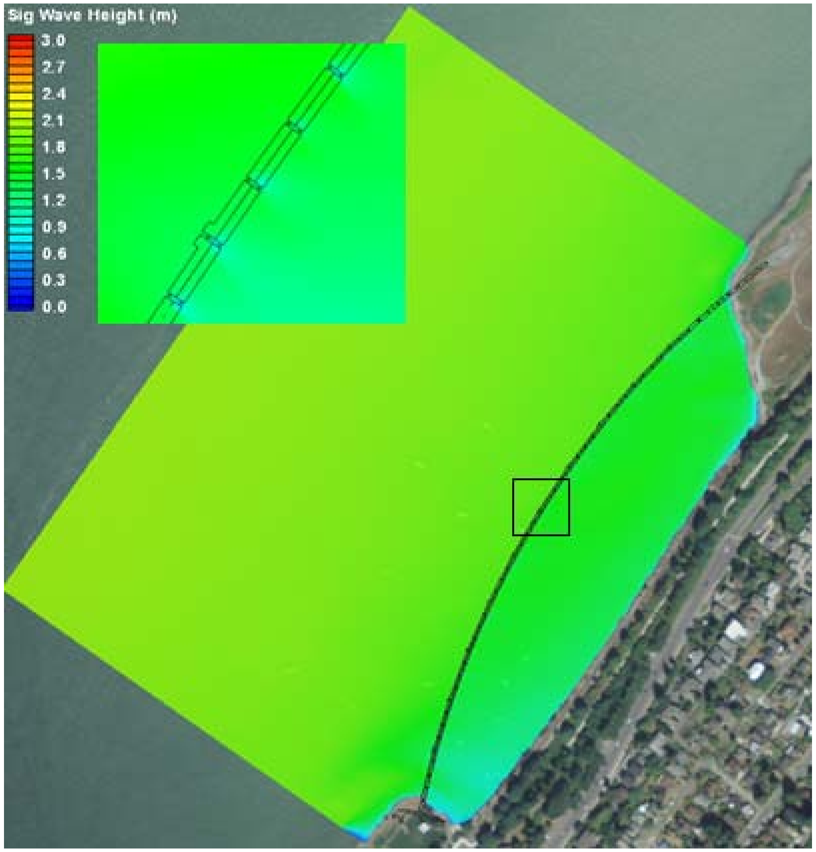

Figure 4b. Model Wave Heights for Post-Project Conditions

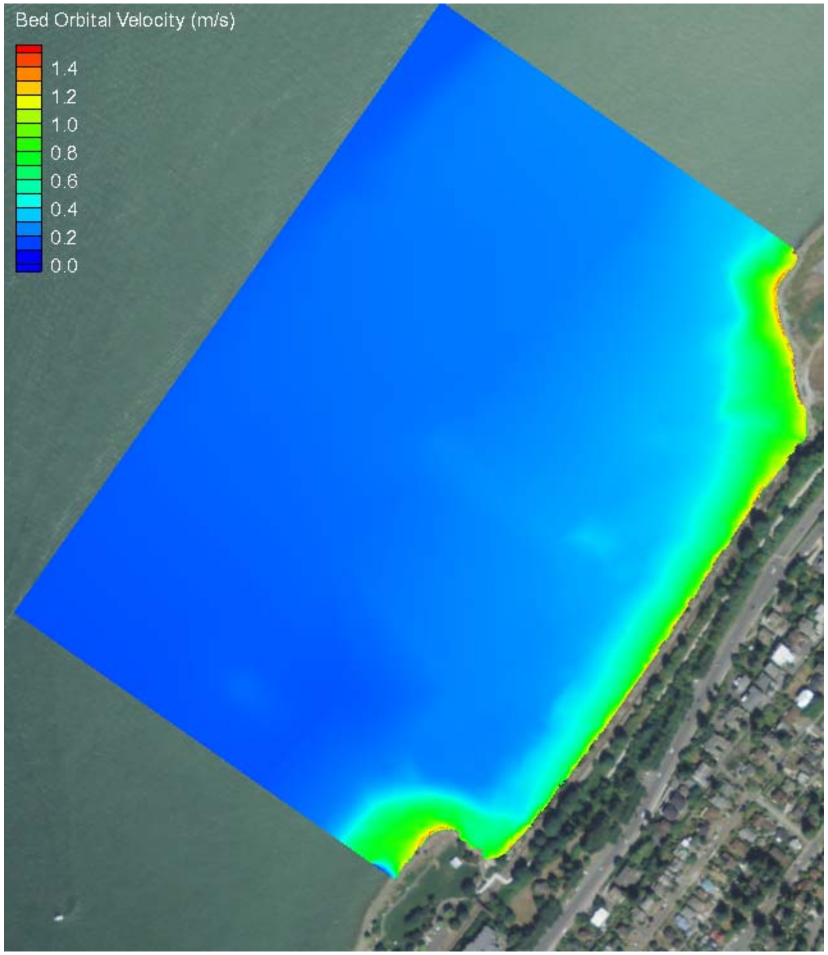

Figure 5a. Bed Orbital Velocities for Existing Conditions

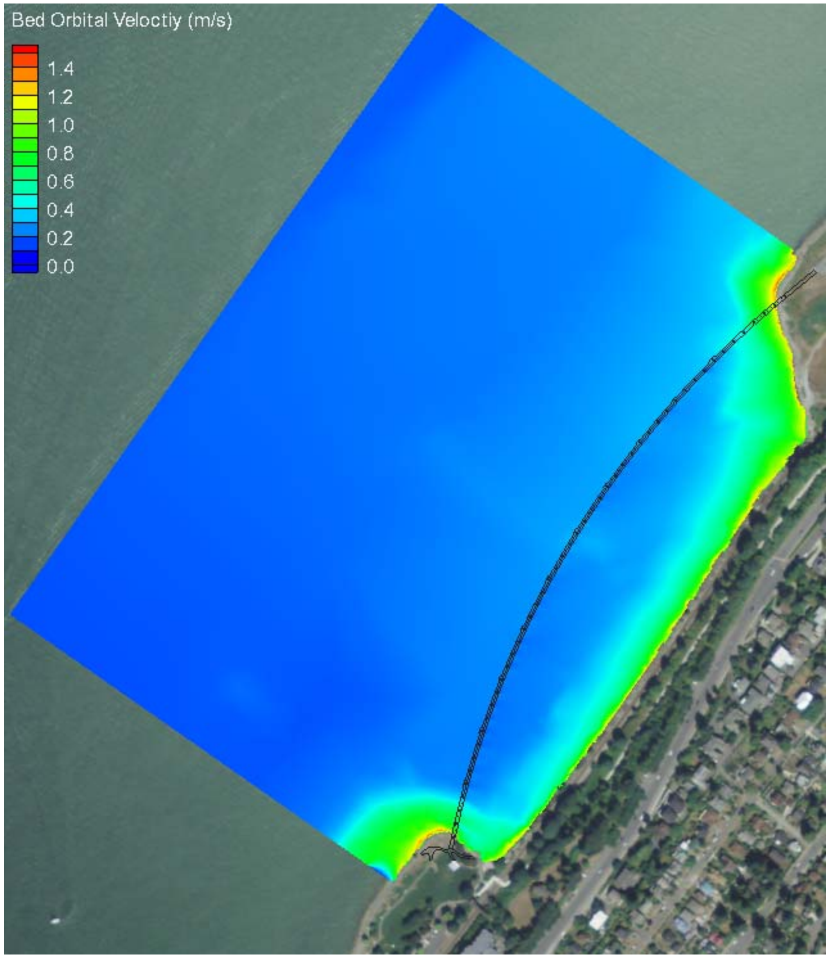

Figure 5b. Bed Orbital Velocities for Post-Project Conditions

Wave heights and orbital velocities were extracted from the modeling results for existing and post-project conditions at each of the 13 controlling stations (See Figure 3 above). Table 2 shows the extracted wave parameters (significant wave heights) for both, existing and post-project conditions. The table also computes possible changes of wave heights between existing and post-project conditions.

Table 2. Significant Wave Heights and percent of wave height reduction/increase at the controlling stations

Point #	Existing Condition Wave Height (m)	Post-project Condition Wave Height (m)	Percent of Reduction
1	1.76	1.76	0.0
2	1.57	1.57	0.0
3	1.19	1.08	8.9
4	1.19	1.13	5.0
5	1.35	1.27	6.1
6	1.41	1.32	6.3
7	1.44	1.35	6.4
8	1.50	1.40	6.6
9	1.55	1.43	7.9
10	1.58	1.42	10.1
11	1.68	1.37	18.2
12	1.69	1.69	0.0
13	1.64	1.64	0.0

The table shows that no or insignificant (less than 10 percent[4]) change in wave heights would occur at most of the controlling stations after construction of the project. A small reduction of wave heights, 10-18 percent, may occur at Stations 10 and 11. These stations are located in close proximity to the walkway and are likely in a shading area of the adjacent piles.

Table 3 depicts the extracted wave parameters (wave orbital velocities) for both existing and post-project conditions. The table also computes shear velocities at the bottom generated by wave orbital velocities.

[4] Changes less than 10 percent may be due to the vicinity of model accuracy and should be disregarded.

Table 3. Wave Orbital Velocities at the controlling stations

Point #	Existing Bottom Orbital Velocities (m/s)	Existing Shear Velocities (m/s)	Post-project Bed Orbital Velocities (m/s)	Post Project Shear Velocities (m/s)
1	0.71	0.114	0.71	0.114
2	0.53	0.093	0.53	0.093
3	0.31	0.064	0.28	0.060
4	0.35	0.071	0.32	0.069
5	0.39	0.077	0.36	0.073
6	0.44	0.082	0.40	0.078
7	0.43	0.081	0.39	0.077
8	0.43	0.082	0.39	0.077
9	0.48	0.087	0.42	0.081
10	0.49	0.089	0.42	0.082
11	0.54	0.094	0.42	0.080
12	0.59	0.101	0.59	0.101
13	0.43	0.081	0.43	0.081

The data in the table (similar to Table 2) show no or insignificant change in orbital and shear velocities at most of the controlling stations after construction of the project. A small reduction of shear stress velocities 0.006-0.014 m/s (0.6 – 1.4 cm/second) may occur at Stations 9 through 11 due to close proximity of these stations to the walkway alignment.

Analysis of sediment on sediment transport potential in the nearshore zone was conducted to determine the importance of small changes in shear velocities due to construction of the overwater structure. The analysis was conducted using results of the study from the Naval Research Laboratory, Stennis Space Center, Mississippi, USA (Phaphitis, 2001).

Figure 6 shows that shear velocities at Stations 9 through 11 during the design storm will be able to move beach sand and gravel sediment up to 0.5" size. Reduction of shear velocities to 0.6-1.4 cm/second at Stations 9-11 will not change the ability of waves to move beach sediment by any significant amount. Reduction of shear velocity will not result in shoreline erosion. Quite the opposite could occur, resulting in small localized accumulations of coarser sediment particles at the shoreline close to the walkway. However, the amount of this accumulation most likely would be small, and may not be detected by available measurement (survey) techniques.

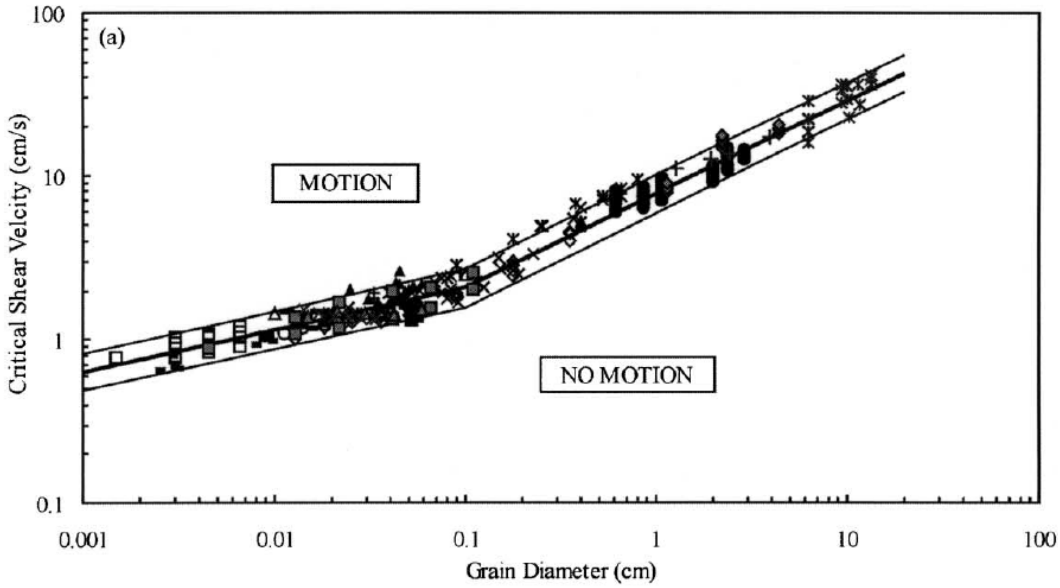

Figure 6. Critical shear velocity as a function of grain diameter

4. Conclusions

The modeling results show that the changes in wave climate affected by the structure are barely discernible for the 25-year event. Changes for smaller wave events should be even lower.

The results of the analysis show that the shoreline will not be impacted negatively after construction of the pedestrian walkway. No shoreline erosion is expected to occur at the two landings, the shoreline reach between the landings, and in the areas extending approximately 300 feet beyond each landing

5. References

Holthuijsen, L.H., Booij, N., Ris, R.C., Haagsma, IJ.G., Kieftenburg, A.T.M.M., Kriezi, E.E.,Zijlema, M. and A.J. van der Westhuysen. February 5, 2004. "SWAN Cycle III Version 40.31 User Manual."

Phaphitis, D., 2001. Sediment movement under unidirectional flows: an assessment of empirical threshold curves. Coastal Engineering 43 (2001) 227-245.