ERDC/CHL Final Draft

Coastal and Hydraulics Laboratory

US Army Corps of Engineers®
Engineer Research and Development Center

Testing and Evaluation of a New Expedient Structure for Flood Fighting – Rapidly Deployed Fortification Wall (RDFW)

March 2001

George F. Turk

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Final report

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Preface

This investigation of the Rapidly Deployed Fortification Wall (RDFW), reported herein, was requested by A.M. Arellanes and Sons and Associates, and was conducted at the Coastal and Hydraulics Laboratory (CHL) of the U.S. Army Engineer Research and Development Center (ERDC). Authorization for CHL to perform the study was granted as part of a Cooperative Research and Development Agreement (CRDA) No. ERDC-GL-00-01 signed and dated 16 March 2000.

The study was conducted at CHL during a 1-month period in May and June 2000 by personnel of the Harbors and Entrances Branch (HEB) of CHL, under the direction of Dr. James R. Houston and Mr. Thomas W. Richardson, former Director and Acting Director of CHL, respectively; and the direct guidance of Mr. Dennis Markle, Chief of HEB. The physical model study was designed and conducted by Mr. George F. Turk, Research Hydraulic Engineer, HEB. Technical assistance was provided by the inventor of RDFW, Mr. Steve Webster, Civil Engineer. The experiment was conducted with the assistance of Mr. Raymond Reed, Civil Engineering Technican, HEB. Construction of the model was carried out by Mr. Al Arellanes, owner, A.M. Arellanes and Sons and Associates.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL James S. Weller, EN, was Commander.

Conversion Factors Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
degree	0.01745329	radians
feet	0.3048	meters
gallons	3.785	Liters

1 Introduction

Background

Historically, expedient flood fighting solutions to raising levees and impounding infrastructure against rising floodwaters have been primarily limited to those involving sandbagging operations. In a period between 1981 and 1985, the U.S. Army Corps of Engineers (USACE) conducted extensive research on a number of expedient levee-raising structures under the Improvements of Operation and Maintenance Techniques Civil Works Research Program. Markle and Taylor (1988) reported the results of this study. Structures such as sandbag levees, mud-boxes, "potato-ridges," and early versions of the Sand Confinement Grid (SCG) were evaluated for effectiveness. These concepts showed varying degrees of effectiveness.

Major flood events plagued the 1990's. The Great Flood of '93 was the largest and most devastating event. Flooding caused loss of life and extensive damage. In 1993, at the direct request of Dr. Robert B. Oswald, then USACE Director of Research and Development, Waterways Experiment Station (WES) fielded a two-man team, Dr. Victor H. Torrey III and Mr. George F. Turk to collect data on expedient methods used to fight floods (Turk and Torrey, 1993). The team was looking for innovative expedient methods, but found sandbag levees were still the primary flood-fighting tool. In many cases, labor-intensive sandbagging operations often yielded poorly constructed ineffective structures.

The Corps of Engineers encourages product innovation in the field of flood fighting. The U.S. Army Corps of Engineers entered a Cooperative Research and Development Agreement (CRDA) on 16 March 2000, with A. M. Arellanes and Sons and Associates, Inc. to test and evaluate one such innovative product. The expedient flood fight product is called Rapidly Deployed Fortification Wall (RDFW) (Figure 1). The RDFW is constructed from an assembly of Sand Confinement Grids (SCG). Steve Webster of ERDC's Structures and Geotechnical Laboratory originally invented and patented several variations of SCG for the U.S. Army Corps of Engineers.

Since its inception, the primary applications of SCG have been for roadway foundation stabilization and erosion control. The first tests of the use of SCG for flood fight applications were by Markle and Taylor(1988). The product

Figure 1. Rapidly Deployed Fortification Wall (RDFW)

was configured as a wall structure and subjected to both hydrostatic and wave action loading. Results were marginal. The original SCG module, where each cell was “onion-shaped” suffered from excessive sand loss when stacked, and failed plastic welds.

In 1996, WES licensed SCG to the Native-American 8-A company, A. M. Arellanes and Sons and Associates, of Mountain View, CA. Mr. Al Arellanes, Owner, working closely with WES engineers, produced a “second-generation” rectangular-cell SCG product. He called the product RDFW.

Purpose

The primary purpose of this investigation is to test and evaluate the effectiveness of RDFW when configured as a wall structure, and subjected to hydrostatic and wave-induced dynamic loads. Under the CRADA, the scope of work called for the RDFW to be rigorously tested. During the testing the wall was inspected for seepage, lateral deflection, sand loss, and material failure. A secondary purpose was to develop a protocol for testing and evaluating future expedient flood fight structures. By developing a testing protocol a variety of expedient structures can be evaluated under the same controlled conditions.

A crucial step in the further development of the RDFW is testing of the product in a controlled laboratory environment. The Coastal and Hydraulics Laboratory tested the RDFW at full scale. Limitations exist in the laboratory. For this particular test series, the RDFW wall was 50 ft in length and buttressed against an impermeable vertical concrete wall. It could be built no taller than 4 ft due to depth limitations in the basin. The structure was founded on a concrete floor, thus stability against foundation scour was not evaluated. No capability existed in the test basin to generate large steady-state currents along the face of the RDFW, thus their effects were not evaluated. This report serves to document testing of the RDFW structure, which was subjected to hydrostatic loads and wave action of increasing magnitude.

2 Study Plan for Rapidly Deployed Fortification Wall (RDFW)

RDFW Design

The RDFW is a rectangular-celled version of SCG. The primary building block of a RDFW wall is the RDFW grid. The grids are laid side-by-side and interlocked with each other. Subsequent lifts of connected grids are stacked on lower lifts until the desired height is reached. The grid forms the skeleton, which is then filled with sand. Sand, once confined in the cells exhibits a tremendous increase in compressive strength. The sand also provides the mass for the structure, which resists sliding forces and overturning moments. This research effort used a RDFW wall that had a base-to-height ratio of approximately one. The 4-ft wide, 4-ft tall, sand-filled RDFW wall has a dry weight of 1,800 lb/lin ft.

Facilities and Equipment

The tests were conducted in an L-shaped basin having the dimensions 250-ft long, 50-ft wide and 4.6-ft deep in the study area. The RDFW wall was constructed approximately 120 ft from the wave generator on a flat bottom portion of the basin (Figure 2). Waves were generated by a bottom hinge-type, hydraulically actuated, electronically controlled wave generator. In the pre-construction unobstructed basin the wave generator is capable of producing an incident wave height up to 2 ft for a model wave period of 2 sec. Only monochromatic waves were used for these RDFW tests. Waves were manually measured at 13 discrete locations using a wave staff. Closer measurements were not warranted for the purpose of this study.

Protocol Development

As previously mentioned, one goal of CHL was to develop a protocol for testing this and future innovative expedient structures acting as barriers against rising flood waters. The following forms the basis for the new testing protocol:

a. Construction sequence and duration.

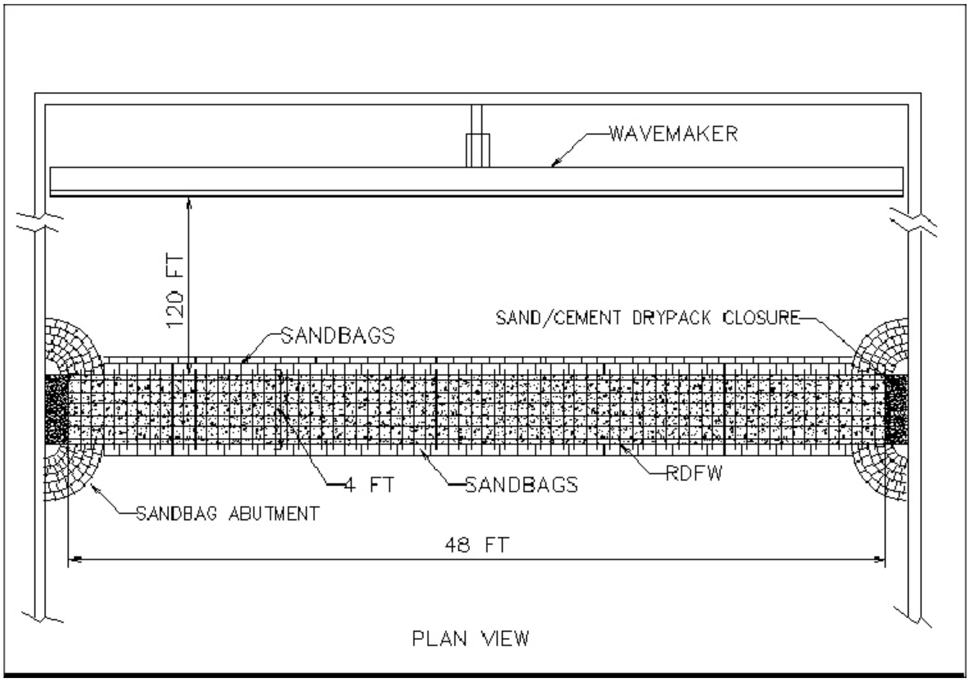

Figure 2. Plan view of RDFW wall in L-shaped basin

b. Structures are subjected to hydrostatic loads from incrementally increasing head pressure.

c. Structures are subjected to dynamic loads by applying incrementally increasing wave loads.

d. Measurements are taken, and observations made of wall deflection.

e. Measurements are taken, and observations made of damage, deterioration, and fill loss.

f. Up to three relatively small-scale repairs of documented damage are allowed during a test series.

g. Measurements are taken, and observations are made of under-seepage and through-seepage.

Static Head Testing Protocol

The testing protocol for the hydrostatic head test consisted of flooding the basin on the upstream side (or river side) of the wall and on the downstream, or "dry" side of the wall, taking measurements of the response (seepage and deflection) to rising waters. Three water levels were used for the testing, 50, 67, and 83 percent of the height of the wall. For the RDFW wall with a height of 48 in., this corresponds to 24, 32, and 40 in. (Figure 3). At each increment, the water level is held at this stage, at least once, for a minimum of 12 hr.

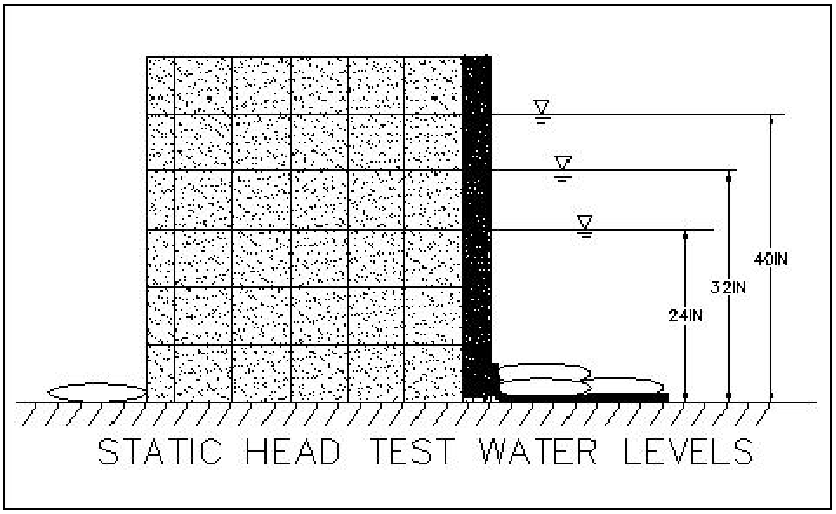

Figure 3. Static head test water levels, d = 2.0 ft, d = 2.67 ft, d = 3.33 ft

Wave-induced Dynamic Load Testing Protocol

The primary purpose of wave-induced dynamic load testing is to provide insight into the performance of the structure under extreme loading conditions. A secondary purpose is to observe its structural response to these loading conditions. Insight is gained into any failure mechanism, be it fill loss, material failure, wall sliding, overturning, or deforming. The protocol calls for running four 1-hour packets of monochromatic waves with a wave period of, $T = 2.0$ sec. With each successive set of 4 hr of wave action, barring failure or extensive damage, the wave heights are incrementally increased.

The test series was conducted for two different water depths. Observations of the wall's condition and response were made during the run. At the end of each run the basin was stilled for 10-15 min to allow the waves to dissipate. During this time the same observations and measurement made during the hydrostatic were recorded. The only addition is observations of the structural components subjected to impact and fatigue loading. In the case of RDFW, the polyethylene terephthalate glycerin (PETG) plastic grid was checked for signs of fatigue and breakage. At the end of each successive run if stability was maintained, then another run commenced. Four hours of wave action accumulated prior to advancing to the next water depth/ wave height combination.

Seepage, Losses, and Deflection

During each test stage, measurements were made of under and through seepage rates, and lateral or horizontal wall deflection. Observations and measurements were made of damage, such as material breakage and sand loss (fill) from the wall. Three repairs of the RDFW were allowed during the test series. At various increments of time, the volume of through or under seepage accumulated on the dry side (or backside) of the RDFW was quantified and reported in units of volume/time/unit length of wall.

Each 4-ft grid stack was assigned a number, and a cell numbering system established. A baseline of the pre-test condition (zero deflection) was first measured. Deflection measurements were taken at 13 locations (one at each transition against the basin wall, and 11 at even increments along the wall itself) at the end of each test run (Figure 4).

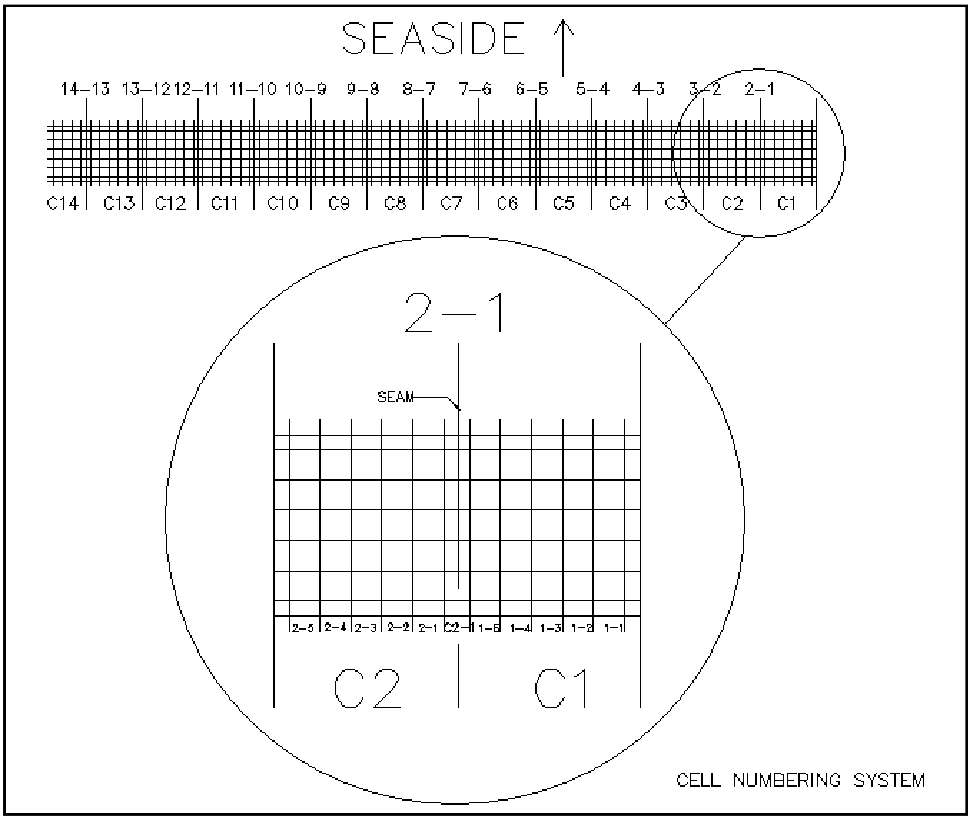

Figure 4. Cell numbering system and wave measurement locations

Wave Power Approach for Evaluating Performance

Any structure built across the basin reflects waves and traps wave energy. Installing the RDFW wall with its highly reflective face produced a partially standing wave in the basin. In addition the truncated conical abutment caused a cross-wave pattern further amplifying the wave height at various locations (Figure 5). The net result was an extremely complex wave field, much more energetic than would be encountered in a majority of inland flood situations.

For the hydrostatic test sequence a simple comparison of wall deflection and seepage rates suffices. With the complex conditions associated with wave loading, simply stating a mean wave height across the face of the wall and loading duration does not give the whole picture. In addition it may be difficult to duplicate wave conditions precisely with different types of structures. A new approach that may be more appropriate for cross comparison between different structures is a “wave power” approach.

Figure 5. Boundary effects from south lateral transition

Power can be derived from a work perspective and an energy perspective. Two methods were developed to determine how much wave power a given structure is exposed to during a given wave run. In a gravity system, potential energy is equal to the work performed in elevating the center of mass (center of gravity, c.g.) from one location to another. As a wave passes, a local change in the center of gravity occurs. Work is done on the system. Work and energy are essentially synonymous. Power, *P*, is derived from the change in Energy, *E*, or Work, *W*, with a change in time, *t*, expressed as:

$$P = \frac{\partial E(or\ W)}{\partial t} \tag{1}$$

This premise forms the basis for the two methods used.

The first method is called the Dynamic Overturning Moment (DOM) Method. Referring to Figure 6, the RDFW, like any structure, is subject to forces that may overturn it. In the case of a floodwall without any anchorage, the wall will overturn when external moments, M_{LOAD} (in this case water pressure acting about the center of pressure) exceed the resisting moments, M_{RESIST} (in this case, self-weight) of the structure. This is written as:

$$\sum M_{LOAD} - \sum M_{RESIST} > 0 \tag{2}$$

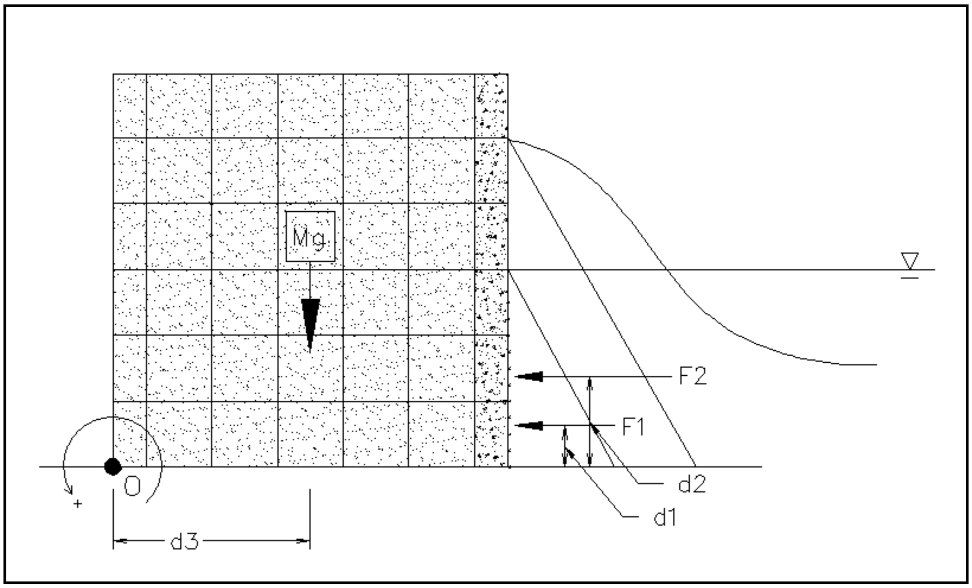

Figure 6. Work done by waves to overturn RDFW wall

In the case of hydrostatic loading the external moment is:

$$M_{LOAD1} = F_1 \bullet d_1 \tag{3}$$

Adding the wave loading increases the external moment to:

$$M_{LOAD2} = F_2 \bullet d_2 \tag{4}$$

In the case of wave loading, each passing wave does work on the wall by applying a dynamic pressure and subsequent alternating raising and lowering the center of pressure from that of the hydrostatic load alone. The force exerted on the RDFW wall due to hydrostatic loading is,

$$F_1 = \frac{1}{2}\gamma\, d\, A \tag{5}$$

where

F_1 = hydrostatic force

γ = specific weight of water

d = water depth

A = submerged area of wall (water depth, d, times unit width)

As the wave crest reaches a maximum at the wall the pressure reaches a maximum and the center of pressure moves upward from the hydrostatic center of pressure, d_1 (in this case, $d_1 = d/3$), to the combined hydrostatic and dynamic center of pressure, d_2. The length of the moment arm increases from d_1 to d_2.

The total force exerted on the RDFW wall, F_2, is comprised of both dynamic and hydrostatic components. For non-breaking wave loading, after the Miche-Rundgren method (SPM, p. 7-162, 1984) the total force is expressed as,

$$F_2 = \frac{1}{2}\gamma\left[d + \left\{\frac{\left(\frac{1+\chi}{2}\right)H_i}{\cosh\left(\frac{2\pi d}{L}\right)}\right\}\right]\left[d + h_o + \left(\frac{1+\chi}{2}\right)H_i\right] \tag{6}$$

where

F_2 = total force on Wall

χ = reflection coefficient, in this case $\chi = 1.0$

H_i = incident wave height

h_o = superelevation of wave mid-height (in this case using linear wave theory, $h_o = 0$)

L = wavelength

The location where F_2 is applied is,

$$d_2 = \frac{1}{3}\left[d + h_o + \left(\frac{1+\chi}{2}\right)H_i\right] \tag{7}$$

As a wave crest impinges on the wall not only does the force increase, but the length of the moment arm as well. Knowing the wave period, T, the average power or change in work with time, can be calculated by

$$\overline{P} = \frac{dW}{dt} = \frac{F_2 d_2 - F_1 d_1}{T} \tag{8}$$

The second method developed for the protocol and used to determine the power content in progressive waves is the Wave Energy Density (WED) Method. After Dean and Dalrymple (1984) and according to linear wave theory, the potential energy density, $\overline{PE}$, of a wave can be described as:

$$\overline{PE} = \frac{1}{L}\int_{x}^{x+L} d(PE_{d+\eta}) \tag{9}$$

where

η = elevation of the free surface

x = discrete location along the axis of the wavelength

The potential energy density of the total water column including contribution from the hydrostatic component, as well as the wave is:

$$\overline{PE_{d+\eta}} = \frac{\gamma d^2}{2} + \frac{\gamma H_i^{\ 2}}{16} \tag{10}$$

The kinetic energy associated with the wave field is defined as being equal to the local integral of the momentum flux or:

$$\begin{aligned} d(KE) &= \int local\ momentum \bullet\ d(velocity) \\ &= (dm)\left(\frac{u^2 + w^2}{2}\right) \end{aligned} \tag{11}$$

where

m = momentum

u,w = horizontal and vertical velocity components

dm = $\rho dxdz$

The total kinetic energy in one wavelength is obtained by double integration and yields:

$$KE_{d+\eta} = \int_{x}^{x+L} \int_{-h}^{0} d((KE) \tag{12}$$

The kinetic energy density of the total water column per unit of surface area is:

$$\overline{KE} = \frac{1}{L} \int_{x}^{x+L} \int_{-d}^{0} \rho \frac{(u^2 + w^2)}{2} dxdz \tag{13}$$

This reduces for a progressive wave simply to:

$$\overline{KE} = \frac{\gamma H_i^2}{16} \tag{14}$$

The kinetic energy density is proportional to the square of the wave height. It is exactly equal to the potential energy density, therefore the total energy density for a progressive wave is

$$\overline{E} = \overline{PE} + \overline{KE} = \frac{\gamma H_i^2}{16} + \frac{\gamma H_i^2}{16} = \frac{\gamma H_i^2}{8} \tag{15}$$

The energy content of a single progressive wave per unit width is

$$E = \frac{\gamma H_i^2 L}{8} \tag{16}$$

According to Dean and Dalrymple (1984), "The rate at which the energy is transferred is called the energy flux, $\Im$, and for linear wave theory it is the rate at which work is being done by the fluid on one side of a vertical section on the fluid on the other side." The "average energy flux",

$\overline{\Im}$, or average power, $\overline{P}$,is described by the well-known expression

$$\overline{\Im} = \overline{P} = \overline{E} \bullet C_g \tag{17}$$

where

C_g = wave group velocity

In shallow water, as is the case for these test series, $C_g = C = L/T$. Substituting this relationship and Eq. 15 into Eq. 17 yields and expression for the average power content of a single progressive wave as

$$\overline{P} = \frac{\gamma H_i^2 L}{8T} \tag{18}$$

For either the DOM or WED method, knowing the average power content of a single progressive wave allows the calculation of the cumulative amount of power that the wall has been exposed to at any point in the test series. Results are given in Chapter 3.

RDFW Wall Construction

The first step of the experiment was the construction of the RDFW wall. Three previously untrained laborers, an equipment operator, and a skilled supervisor constructed the 4-ft x 4-ft x 50-ft wall. The RDFW grids were unpacked and then each of the 72 grids were stacked and interlocked (Figure 7). Once all the grids were placed (Figure 8), the wall was then filled with "concrete" sand (Figure 9). This local concrete sand is typically used for laboratory experiments at ERDC. It is pit-run washed containing approximately 4-percent gravel sizes and 2-percent minus No. 200 U.S. standard sieve size. It is classified as poorly graded sand (SP). The sand was then compacted by hand, and additional sand added to the cells, as required, and again compacted. The top of the wall was leveled out with a screed (Figure 10). The entire process was completed in approximately 1 hr and 30 min.

The RDFW grid has a narrow 4-in. cell along the face of the wall. At first clean gravel was placed in these cells. But during the first "shake-down" waves, it was observed that the small gravel could escape from between the seams of adjacent seaward facing cells. In order to prevent gravel leakage, two special

Figure 7. Stacking and interlocking of RDFW grids

Figure 8. Final assembly of RDFW grids

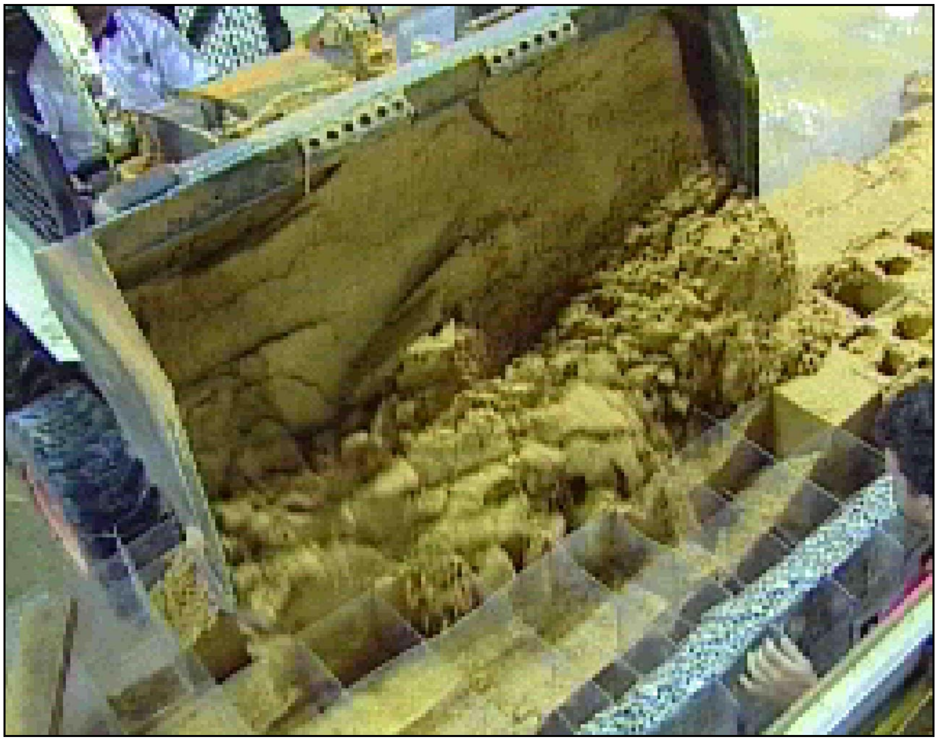

Figure 9. Filling cells with small loader

Figure 10. Compacting and screeing sand in RDFW cells

Figure 11. View of gravel-filled fabric and plastic inserts

treatments were used along the face of the wall. One treatment was to place an 8-in. x 96-in. strip of geosynthetic fabric, folded in half and placed in each cell face. This formed a sock that was then filled with ¾-in. minus gravel. The second face treatment used 8-in. x 48-in. strip of PETG plastic placed inside the cell to cover the horizontal gap between grid lifts. Gravel was then filled in behind the plastic strips (Figure 11).

At first the lateral transition and closure between the RDFW wall and the wave basin side walls posed a problem. Pure sandbag abutments were first tried but proved, to leak excessively. After a rigorous brainstorming session, Mr. Rey Rodriges, of A. M. Arellanes and Sons and Associates came up with the idea of closing the gaps by placing a dry pack mix of cement and sand, using sandbag abutments on each side of the wall as formworks (Figure 12). This proved highly successful. Very little leakage was attributed to this closure. It is easy to see how this dry pack closure has real world flood fighting applications when a situation calls for the RDFW to abut a vertical surface. With the construction of the RDFW complete (Figure 13) all the tests were ready to commence.

Figure 12. Riverside view of north abutment lateral transition

Figure 13. View along the alignment of the completed RDFW wall

3 Results

Hydrostatic Head Test and Underseepage Results

The hydrostatic tests consisted of flooding the basin on one side of the wall (riverside) and on the "dry" side of the wall, visually checking for wall deformation, fill loss, and measuring the amount of under seepage. The test was conducted at three water levels, 24 in. (2.0 ft), 32 in. (2.67 ft), and 40 in. (3.33 ft). Figure 14 shows a seaside view of the 2.67-ft (32-in.) head level, and Figure 15 shows the 3.33-ft (40-in.) head level. The original plan according to the aforementioned protocol called for maintaining these head levels for a minimum of 12 hr. To maximize utilization of the test facility, the tests were made for several time spans ranging from just over 3 hr to in excess of 42 hr. Figure 16 gives a dry-side view of the relatively small volume of water that leaked to the dry-side. For a cumulative test period of over 128 hr the wall showed no sign of deterioration, and no deflections or deformations were noted. While not precisely quantified, the total amount of sand that leeched out of the 4-in. dry side cell wall was approximately 4 cu ft. No sand loss was detected from the 8 in. cells.

Table 1 shows the under seepage rates calculated for the three water levels retained by the RDFW when placed on a concrete surface. The water level was brought up to the desired head level and a measurement of the depth of water impounded on the dry-side of the wall was taken. After an elapse time, a second measurement of the water depth in the impoundment was made. Knowing the basin geometry, the volume of the impounded water was calculated. Table 1 expresses these seepage rates in terms of Volume/Hour/LF of RDFW wall. Average values were calculated for each of the three water depths.

Wave-induced Dynamic Load Testing

A second phase of the testing involved subjecting the wall to dynamic loading by running waves against the wall at increasingly larger wave heights. Regular waves, each with a two second period ($T = 2$ sec) were selected. Six increments of wave heights, ranging from approximately 0.4 ft to over 1.5 ft were used during the experiment. At each wave height, a packet of 1,800 waves impacted the wall during the 1 hour duration of each test run. Two water depths were used, 2.0 ft and 2.67 ft. The magnitude of the wave action was

Figure 14. Static head test, d = 2.67 ft

Figure 15. Static head tests, d = 3.33 ft

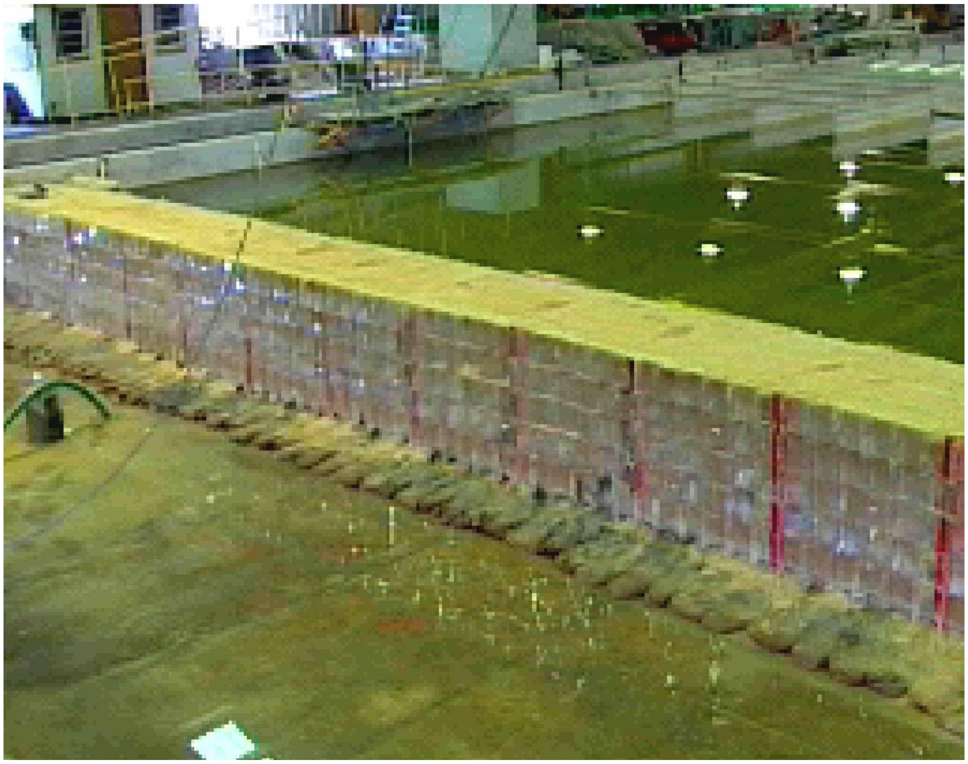

Figure 16. Backside of RDFW after 42.5 hrs of static head test, d = 3.33 ft

Table 1
Hydrostatic Head Tests and Underseepage Rates

Head (ft)	Elapsed Time (hr)	d_backside start (ft)	d_backside end (ft)	d_backside differential (ft)	Seepage Volume (ft^3)	Volume/LF (ft^3/ft)	Vol/LF/hr (ft^3/LF/hr)	Vol/LF/hr (gal/LF/hr)
2.00	4.25	0.07	0.16	0.09	96.2	1.9	0.5	3.4
2.00	42.50	0.17	0.70	0.53	1781.0	35.6	0.8	6.3
2.00	3.12	0.07	0.15	0.08	81.3	1.6	0.5	3.9
2.00	4.50	0.19	0.25	0.06	54.8	1.1	0.2	1.8
					HEAD, H = 2.0 ft, Average Rate -			**3.8**
2.67	5.75	0.12	0.30	0.18	276.8	5.5	1.0	7.2
2.67	14.30	0.30	0.60	0.30	648.8	13.0	0.9	6.8
2.67	18.00	0.30	0.50	0.20	328.3	6.6	0.4	2.7
2.67	12.00	0.07	0.50	0.43	1221.0	24.4	2.0	15.2
					HEAD, H = 2.67 ft, Average Rate -			**8.0**
3.33	12.00	0.08	0.50	0.42	1170.8	23.4	2.0	14.6
3.33	12.50	0.03	0.58	0.55	1905.5	38.1	3.0	22.8
					HEAD, H = 3.33 ft, Average Rate -			**18.7**

greater than anticipated. This was largely due to the highly reflective face of the RDFW and the cross -wave patterns produced from wave reflection off of the conical abutments. A high degree of non-linearity in the wave form was observed. Figures 17-22 show examples of some of the wave action encountered during the testing (Refer to Table 2 to relate percent gain to wave height).

In the case of the RDFW, the mean wave heights were measured at 13 discrete locations across the face of the wall (Table 2). These measurements represent the combined incident and reflected wave heights. From these values, assuming 100-percent reflection estimated incident wave heights were calculated for the 13 locations (Table 3). The first column of Table 4 expressed wave height in terms of percentage of wave board gain. It is a convenient measurement for the physical modeler, and is used throughout this report. The table gives mean incident wave heights for the 13 locations and the standard deviation. It also shows the duration for each run and the cumulative duration for the entire test series.

Wall Deterioration and Overtopping

As wave heights increased, the RDFW wall began to be overtopped. Figure 23 shows the onset of overtopping at 25 percent gain and d = 2.67 ft. Figures 24-27 show the severity of the overtopping, especially for the large waves. Table 5 shows measurements of impounded backside water levels, *d_backside*) that were taken before wave runs with overtopping, and

Figure 17. Wave field at 35 percent gain, H = 1.04 ft, d = 2.0 ft

Figure 18. Wave field at 35 percent gain, H = 1.04 ft, d = 2.0 ft

Figure 19. Wave field at 35 percent gain, H = 1.46 ft, d = 2.67 ft

Figure 20. Wave field at 35 percent gain, H = 1.46 ft, d = 2.67 ft

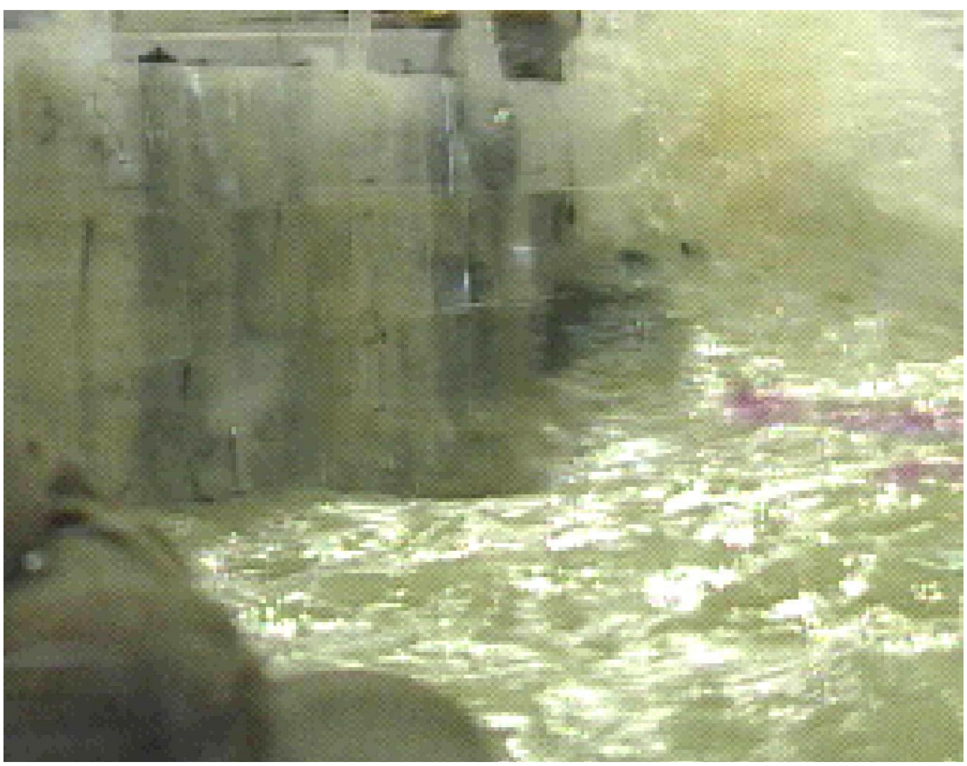

Figure 21. Wave field at 45 percent gain, H = 1.52 ft, d = 2.0 ft

Figure 22. Wave field at 45 percent gain, H = 1.52 ft, d = 2.67 ft

Table 2
Wave Height Measurements at Front Side Wall Face

Gain (%)	Dfront (ft)	STATIONS (*Measurement in ft												
		C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
10	2.00	1.00	0.80	0.15	0.30	0.35	0.50	0.60	0.20	0.40	0.50	0.60	0.75	1.50
15	2.00	1.30	1.10	0.60	0.40	0.70	1.00	0.70	0.70	0.60	0.50	0.80	1.30	1.30
20	2.00	1.50	1.50	1.00	1.00	1.00	1.03	1.03	1.04	1.50	1.40	1.40	1.60	1.60
25	2.00	1.60	1.60	1.10	1.10	1.30	1.40	1.40	1.40	1.40	1.30	1.40	1.60	1.60
35	2.00	2.10	2.10	1.40	1.50	1.75	2.30	2.30	2.00	1.60	1.50	2.00	2.30	2.30
45	2.00	2.40	2.40	2.00	2.40	3.00	3.20	3.50	3.50	3.20	3.00	2.60	3.20	3.20
15	2.67	3.10	3.10	2.20	0.90	1.15	1.20	1.70	1.50	1.50	1.25	1.40	2.50	2.50
20	2.67	3.60	3.60	2.50	1.10	1.25	1.40	2.00	1.80	1.70	1.50	1.60	2.70	2.60
25	2.67	3.50	3.50	2.40	2.00	2.00	1.80	2.20	2.10	2.10	2.20	2.20	2.80	2.60
35	2.67	3.10	3.10	2.20	2.30	2.60	2.40	2.80	3.10	2.80	2.30	2.50	3.10	3.10
45	2.67	2.90	2.90	2.30	2.80	3.30	3.07	2.80	2.65	3.08	2.65	2.80	3.00	3.00

Table 3
Estimated Incident Wave Height Across Front Side Wall Face

Gain (%)	Dfront (ft)	C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
10	2.00	0.50	0.40	0.08	0.15	0.18	0.25	0.30	0.10	0.20	0.25	0.30	0.38	0.75
15	2.00	0.65	0.55	0.30	0.20	0.35	0.50	0.35	0.35	0.30	0.25	0.40	0.65	0.65
20	2.00	0.75	0.75	0.50	0.50	0.50	0.52	0.52	0.52	0.75	0.70	0.70	0.80	0.80
25	2.00	0.80	0.80	0.55	0.55	0.65	0.70	0.70	0.70	0.70	0.65	0.70	0.80	0.80
35	2.00	1.05	1.05	0.70	0.75	0.88	1.15	1.15	1.00	0.80	0.75	1.00	1.15	1.15
45	2.00	1.20	1.20	1.00	1.20	1.50	1.60	1.75	1.75	1.60	1.50	1.30	1.60	1.60
15	2.67	1.55	1.55	1.10	0.45	0.58	0.60	0.85	0.75	0.75	0.63	0.70	1.25	1.25
20	2.67	1.80	1.80	1.25	0.55	0.63	0.70	1.00	0.90	0.85	0.75	0.80	1.35	1.30
25	2.67	1.75	1.75	1.20	1.00	1.00	0.90	1.10	1.05	1.05	1.10	1.10	1.40	1.30
35	2.67	1.55	1.55	1.10	1.15	1.30	1.20	1.40	1.55	1.40	1.15	1.25	1.55	1.55
45	2.67	1.45	1.45	1.15	1.40	1.65	1.54	1.40	1.33	1.54	1.33	1.40	1.50	1.50

Table 4
Mean Wave Heights, Standard Deviation, and Duration

Gain (%)	Water Depth(ft)	Mean Wave Height (ft)	Wave Height St. Dev (ft)	Duration at Gain (hr)	Cummulative Duration (hr)
10	2.00	0.42	0.18	1	1
15	2.00	0.54	0.16	4	5
20	2.00	0.74	0.13	4	9
25	2.00	0.79	0.09	4	13
35	2.00	1.04	0.17	4	17
45	2.00	1.49	0.24	4	21
15	2.67	1.05	0.37	4	25
20	2.67	1.17	0.42	5	30
25	2.67	1.31	0.27	4	34
35	2.67	1.46	0.18	3	37
45	2.67	1.52	0.12	3	40

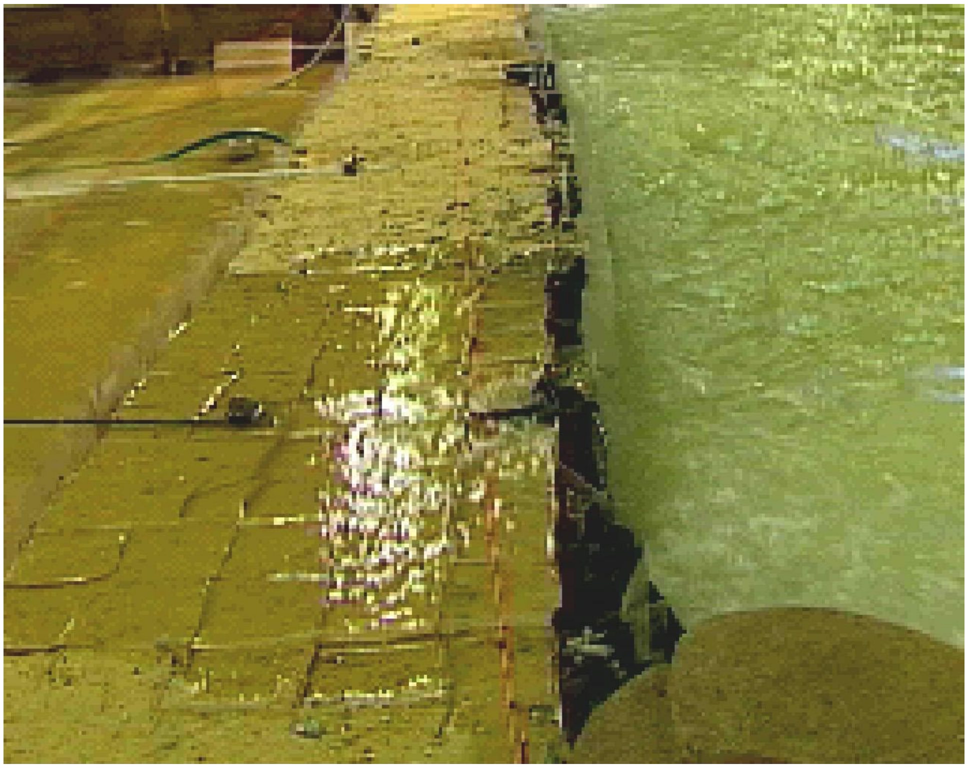

Figure 23. Onset of overtopping, 25 percent gain, H = 1.31 ft, d = 2.67 ft

Figure 24. Overtopping RDFW wall, 35 percent gain, H = 1.46 ft, d = 2.67 ft

Figure 25. RDFW wall being overtopped, 35 percent gain, H = 1.46 ft, d = 2.67 ft

Figure 26. Overtopping at 45 percent gain, H = 1.49 ft, d = 2.0 ft

Figure 27. Significant overtopping, 45 percent gain, H = 1.52 ft, d = 2.67 ft

measurements taken after the run. One hour overtopping rates ranged from 22-gal/LF/hr to almost 300-gal/LF/hr. No water was added to the basin during each test run to offset the overtopping losses. Therefore a potential exists that overtopping rates may have been larger if riverside water levels had been maintained. Concurrent with waves that were severe enough to cause overtopping was sand loss and damage to the wall.

The first onset of significant and measurable damage occurred at Hour 19. Against the south abutment, the waves caused extreme turbulence and the beginning of the plastic grids breaking and subsequent fill loss. By the end of Hour 23 a decision to stop the test was made. This damage is referred to as Damage Section 1 and is shown in Figure 28. The amount of fill loss was measured and, the location of the loss documented. This is depicted in Figure 29. The total sand loss from the damaged area was 28,740 cu in. or 0.62 cu yd.

With the water level on the riverside at 2.0 ft, the construction crew made the repair. Working in the wet, the crew removed the two broken RDFW grids and replaced them with two new ones (Figure 30). Gravel was replaced in the facing cells and sand was added and compacted (Figure 31). As a precaution against future damage the facing was reinforced with nylon tie-wraps to prevent the vertical facing edges of the RDFW wall from deflecting under severe wave loads. The total repair time was just over 2 hrs. The wave runs resumed.

Table 5
Overtopping Rates for 4.0 ft Wall

Gain (%)	Mean Incident Wave Ht (ft)	Water Depth (ft)	d_backside start (ft)	d_backside end (ft)	d_backside differential (ft)	Total Volume (ft^3)	Volume/LF (ft^3/LF)	Elapsed Time(hr)	Vol/LF/hr ft^3/LF/hr	Vol/LF/hr gal/LF/hr
35	1.04	2.00	0.04	0.42	0.38	980.1	19.6	1.0	19.6	146.6
35	1.04	2.00	0.04	0.32	0.28	576.3	11.5	1.0	11.5	86.2
						HEAD, 35% Gain, H = 2.0 ft, Average Rate -				***116.4***
45	1.49	2.00	0.07	0.62	0.55	1905.5	38.1	1.0	38.1	285.1
45	1.49	2.00	0.62	0.93	0.31	686.5	13.7	1.0	13.7	102.7
						HEAD, 45% Gain, H = 2.0 ft, Average Rate -				***193.9***
25	1.31	2.67	0.11	0.22	0.11	129.0	2.6	1.0	2.6	19.3
25	1.31	2.67	0.52	0.65	0.13	166.0	3.3	1.0	3.3	24.8
						HEAD, 25% Gain, H = 2.67 ft, Average Rate -				***22.1***
35	1.46	2.67	0.08	0.43	0.35	848.0	17.0	1.0	17.0	126.9
35	1.46	2.67	0.43	0.61	0.18	276.8	5.5	1.0	5.5	41.4
35	1.46	2.67	0.51	0.85	0.34	806.1	16.1	1.0	16.1	120.6
						HEAD, 35% Gain, H = 2.67 ft, Average Rate -				***96.3***
45	1.52	2.67	0.08	0.56	0.48	1488.0	29.8	1.0	29.8	222.6
45	1.52	2.67	0.03	0.50	0.47	1432.5	28.7	1.0	28.7	214.3
						HEAD, 45% Gain, H = 2.67 ft, Average Rate -				***218.5***

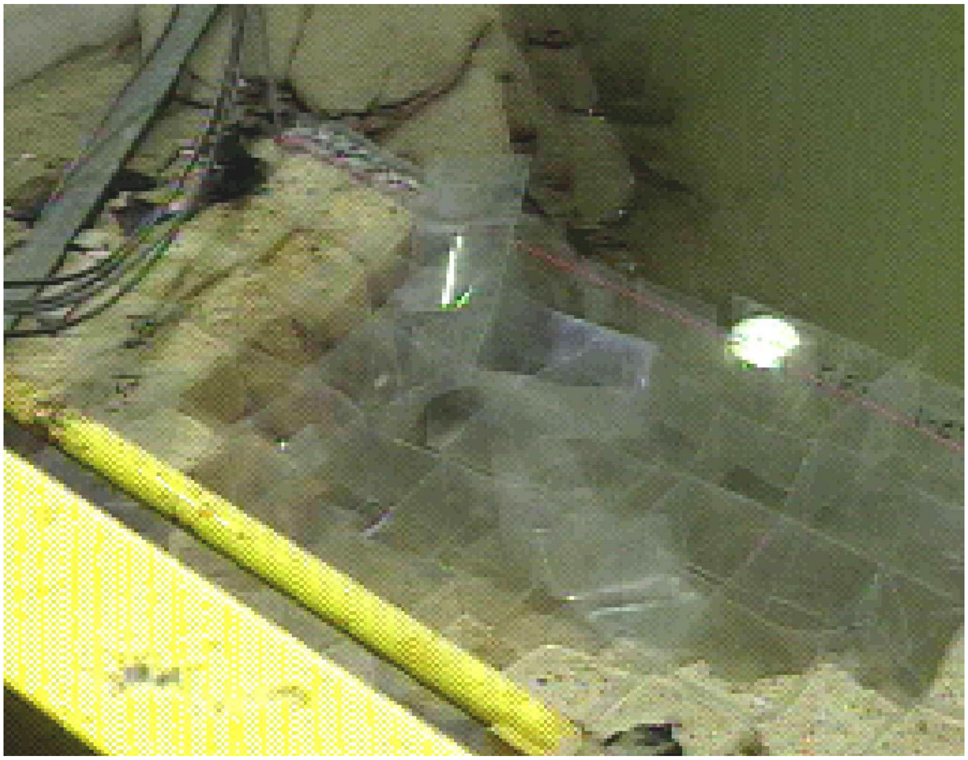

Figure 28. Damage at Section 1 after 23 hrs of wave action

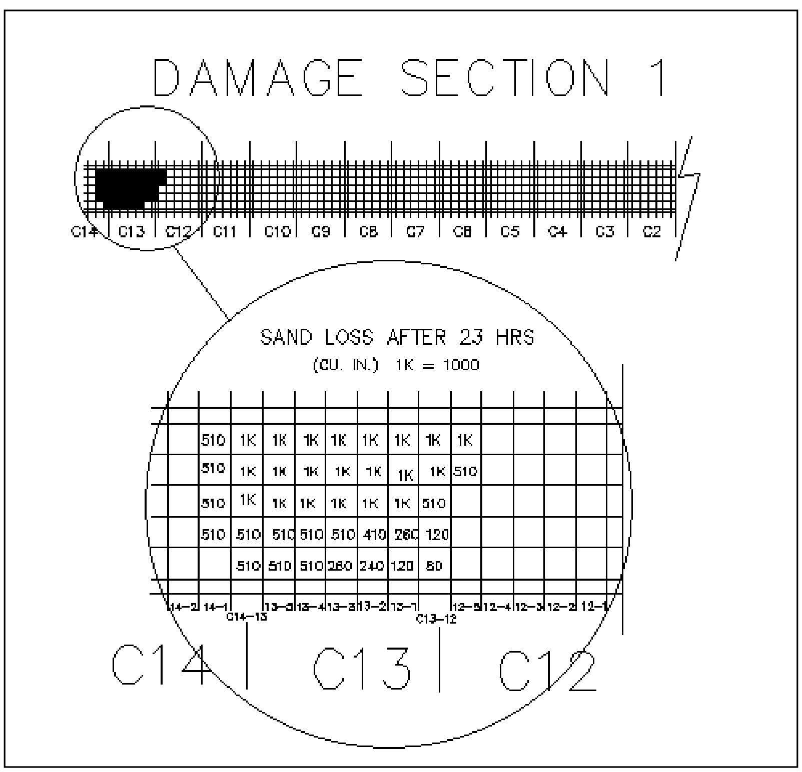

Figure 29. Damage to Section 1 after 23 hrs of wave action

Figure 30. Repair of damaged Section 1 in the wet, d = 24 in.

Figure 31. Repairing damaged section

At Hour 28 a second section of the wall was showing signs of sand loss and plastic breakage. This time the damage was to the area of the wall adjacent to the North Abutment. Figure 32 shows the onset of sand loss to Damage Section 2. At the end of Hour 28 the wall had lost 7,157 cu in. (0.15 cu yd) of sand (Figure 33). The wave runs continued. By the end of Hour 32, deterioration of Damage Section 2 had progressed to the point repair was needed, and the wave runs halted (Figure 34). Total sand loss after 32 hr of wave action was 12,287 cu in. (0.26 cu yd) (Figure 35). A similar repair as the one made to Damage Section 1 was made and the wave runs continued.

The third section of the RDFW wall to start to deteriorate was almost in the center of the 50-ft span. The onset of damage was at Hour 37 (Figure 36). At this point the sand loss was 4,457 cu in., or 0.10 cu yd (Figure 37). After the measurement was made the wall was allowed to deteriorate until the end of Hour 40 (Figure 38). By this time all the sand had emptied from four individual grids for a total volume loss of 59,000 cu in. (1.26 cu yd) (Figure 39), the tests were stopped.

RDFW Wall Deflection

Throughout the wave-induced dynamic load tests, measurements were taken of the amount of lateral deflection the RDFW exhibited. Table 6 shows the deflections measured after the corresponding wave runs. Figure 40 shows the deflection at the 13 discreet measurement locations. Values are given for Hour 0, Hour 5, Hour 14, Hour 26, Hour 30, Hour 37, and Hour 40. The maximum deflection was measured at the top of the wall, approximately mid-span. No measurement was made at the bottom of the wall. By Hour 30 the maximum deflection was 0.17 ft. As the wall sustained increased wave attack, this deflection grew to approximately 0.62 ft at Hour 40.

Wave Power Calculations

For each of the estimated incident wave heights measured across the face of the RDFW during the tests (Table 3), the wave power imparted on the wall was calculated. Two methods were used, Dynamic Overturning Moment (DOM) Method and Wave Energy Density (WED) Method. Tables 7-17 give the results for these two methods. The format and formulas are identical for each of the 11 tables. Only the input variables are changed. The input variables (wave heights, wave period, and water depths) are given at the top of the tables. Each row of calculations is marked with reference numbers from 1 to 19. No. 1 shows the

Figure 32. Onset of damaged Section 2, Hour 28

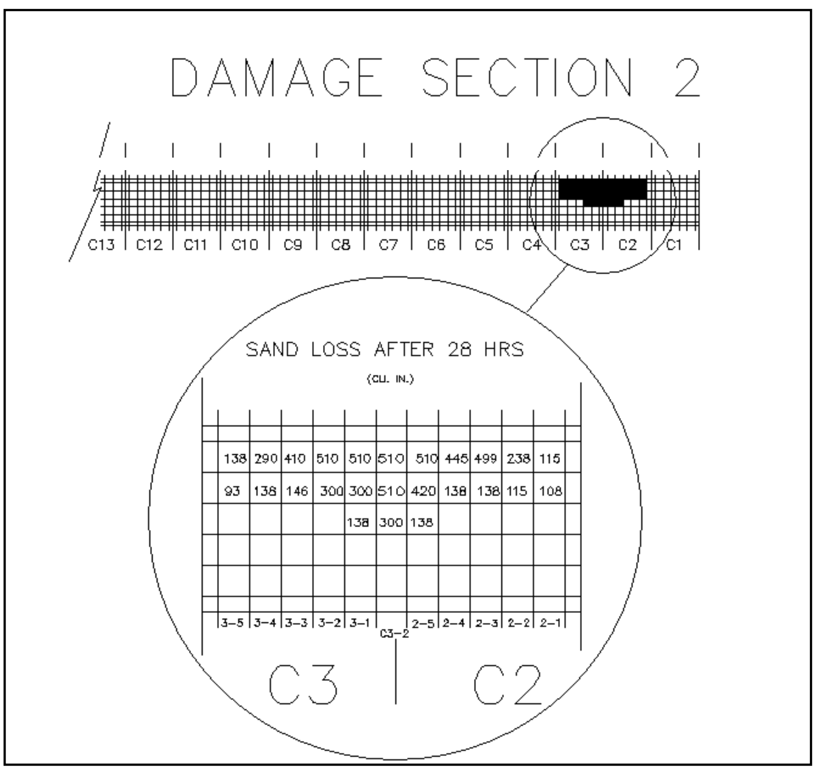

Figure 33. Onset of damage to Section 2 after 28 hrs

Figure 34. Final damage at Section 2, Hour 32

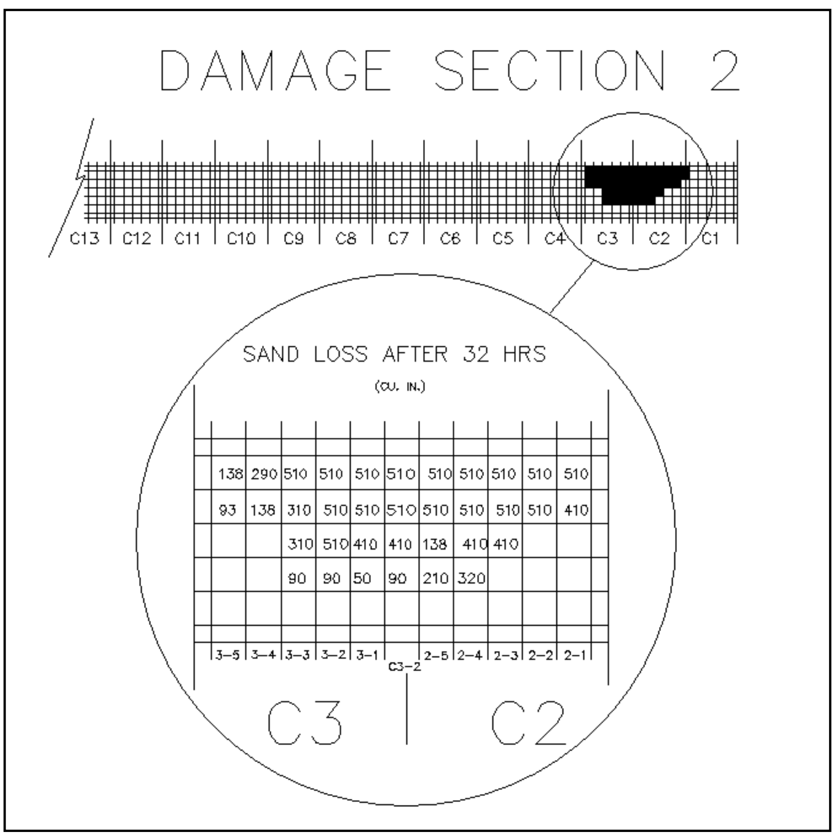

Figure 35. Final damage to Section 2 after 32 hrs

Figure 36. Onset of damage at Section 3, Hour 37

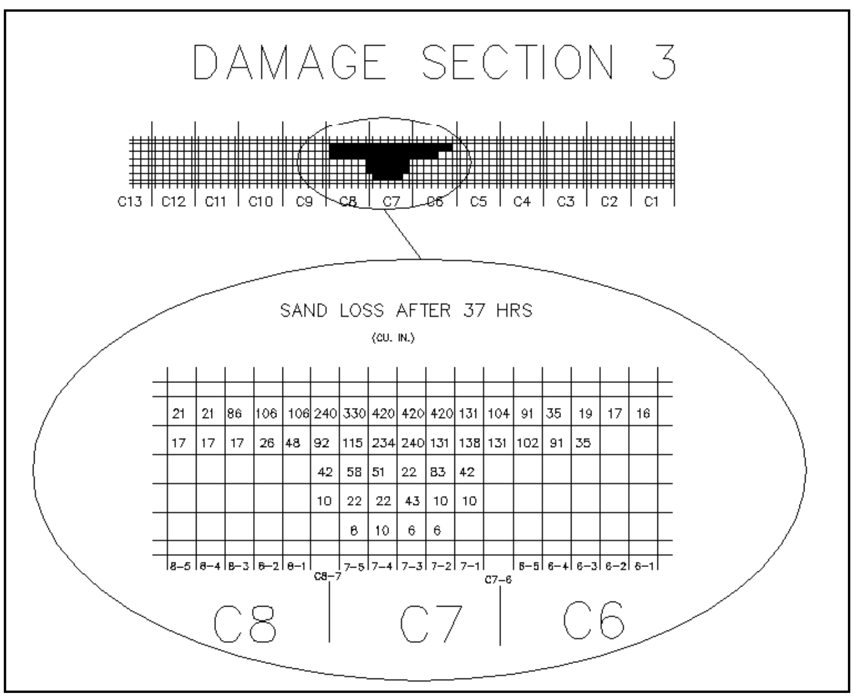

Figure 37. Onset of damage to Section 3 after 37 hrs

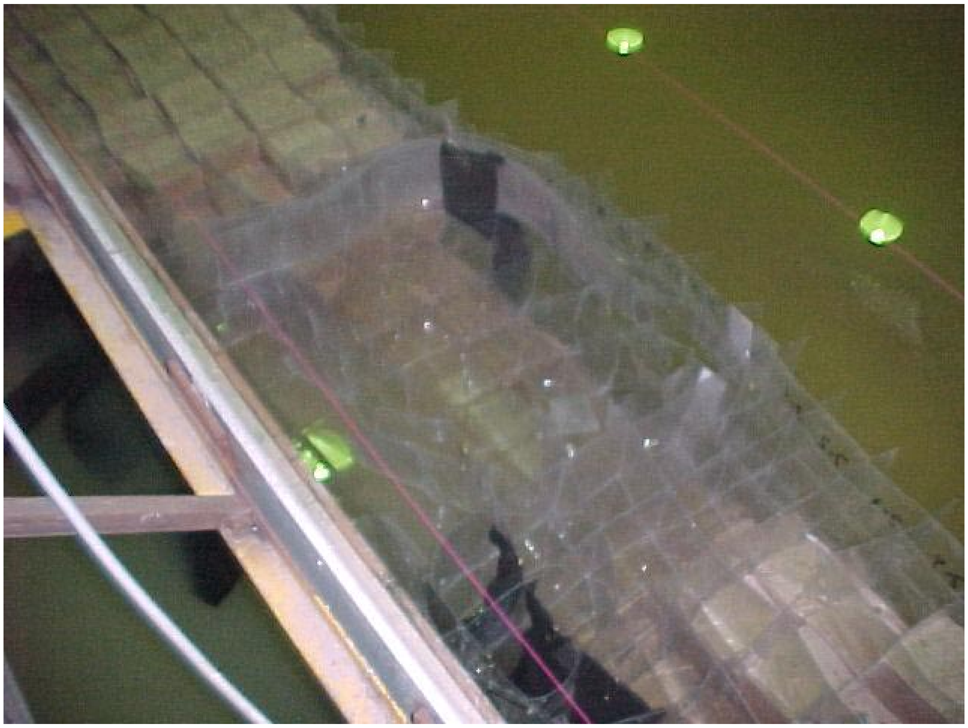

Figure 38. Final damage Section 3, Hour 40

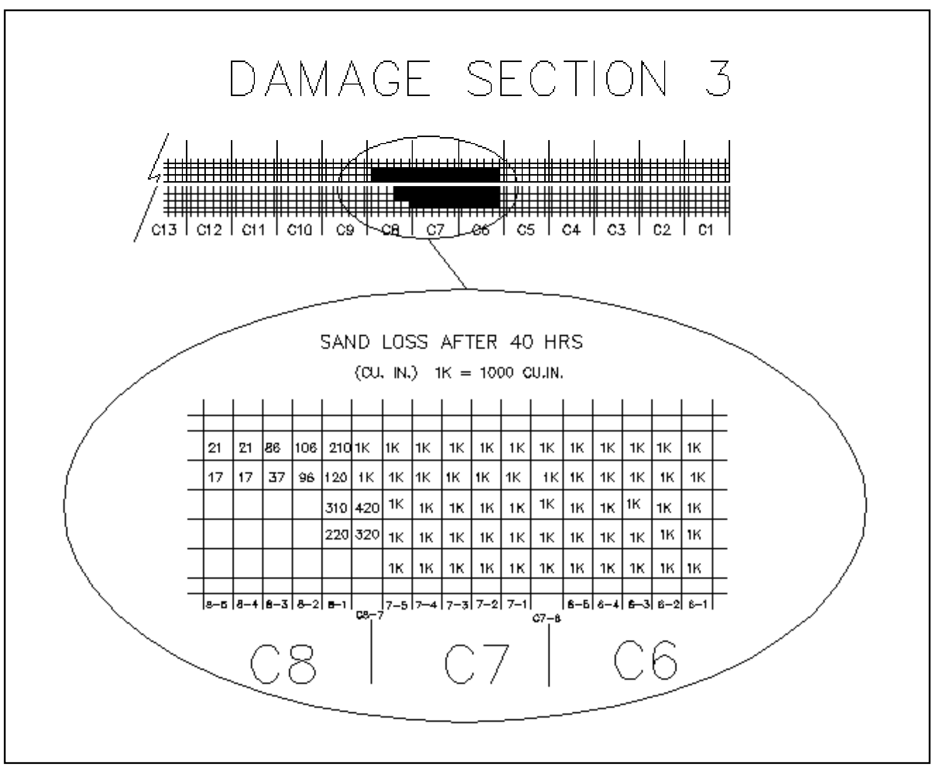

Figure 39. Final damage at Section 3 after 40 hrs

Table 6
Wall Deflection Measurements - Back(Dry) Side

RUN NO.	Gain (%)	Dfront (ft)	C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
								STATIONS		*measurements in FT					
Baseline Measurement			**3.11**	**3.34**	**3.57**	**3.66**	**3.69**	**3.79**	**3.62**	**3.80**	**3.75**	**3.89**	**4.03**	**3.96**	**3.91**
2	15	2.00	3.09	3.33	3.56	3.66	3.68	3.79	3.61	3.79	3.74	3.88	4.02	3.95	3.91
3	15	2.00	3.09	3.33	3.56	3.66	3.68	3.79	3.61	3.79	3.74	3.88	4.03	3.95	3.91
4	15	2.00	3.11	3.33	3.56	3.65	3.68	3.79	3.61	3.79	3.74	3.89	4.03	3.95	3.91
5	15	2.00	3.09	3.33	3.59	3.65	3.68	3.79	3.61	3.78	3.74	3.88	4.03	3.95	3.91
6	15	2.67	3.09	3.33	3.55	3.64	3.68	3.79	3.60	3.78	3.74	3.88	4.02	3.94	3.91
7	15	2.67	3.09	3.33	3.55	3.64	3.68	3.78	3.61	3.78	3.74	3.88	4.03	3.95	3.91
8	15	2.67	3.09	3.33	3.55	3.65	3.64	3.78	3.61	3.78	3.74	3.89	4.03	3.95	3.91
9	15	2.67	3.09	3.33	3.55	3.65	3.64	3.78	3.61	3.78	3.74	3.89	4.03	3.95	3.91
10	20	2.00	3.09	3.33	3.55	3.65	3.68	3.78	3.61	3.78	3.73	3.89	4.03	3.95	3.91
11	20	2.00	3.09	3.33	3.56	3.65	3.68	3.78	3.61	3.78	3.74	3.89	4.03	3.95	3.91
12	20	2.00	3.09	3.33	3.56	3.65	3.68	3.78	3.61	3.78	3.73	3.89	4.03	3.95	3.91
13	20	2.00	3.09	3.33	3.56	3.65	3.68	3.78	3.61	3.79	3.74	3.89	4.03	3.95	3.91
14	20	2.00	3.09	3.33	3.55	3.65	3.68	3.78	3.61	3.78	3.74	3.89	4.03	3.95	3.91
15	20	2.67	3.09	3.34	3.55	3.64	3.67	3.78	3.61	3.78	3.74	3.89	4.03	3.95	3.91
16	20	2.67	3.09	3.33	3.55	3.65	3.68	3.78	3.61	3.78	3.74	3.88	4.03	3.95	3.91
17	20	2.67	3.09	3.33	3.54	3.63	3.68	3.78	3.61	3.78	3.74	3.88	4.02	3.94	3.90
18	20	2.67	3.09	3.33	3.55	3.64	3.67	3.78	3.61	3.78	3.74	3.88	4.03	3.95	3.91
19	25	2.00	3.09	3.34	3.55	3.65	3.68	3.78	3.61	3.78	3.74	3.9	4.03	3.95	3.90
20	25	2.00	3.09	3.33	3.56	3.65	3.68	3.78	3.61	3.79	3.78	3.89	4.02	3.96	3.90
21	25	2.00	3.09	3.33	3.55	3.64	3.67	3.78	3.6	3.78	3.79	3.89	4.03	3.96	3.90
22	25	2.00	3.09	3.33	3.55	3.65	3.68	3.78	3.61	3.78	3.74	3.89	4.03	3.95	3.90
23	25	2.67	3.09	3.33	3.54	3.63	3.66	3.77	3.59	3.76	3.71	3.86	3.99	3.91	3.86
24	25	2.67	3.09	3.31	3.54	3.63	3.66	3.76	3.58	3.76	3.71	3.86	3.98	3.89	3.85
25	25	2.67	3.09	3.33	3.52	3.61	3.65	3.74	3.57	3.74	3.69	3.83	3.97	3.87	3.82
26	25	2.67	3.10	3.32	3.54	3.64	3.68	3.78	3.59	3.78	3.73	3.87	4.01	3.93	3.87
27	35	2.67	3.09	3.31	3.51	3.63	3.66	3.76	3.58	3.78	3.71	3.86	3.97	3.89	3.85
28	35	2.67	3.09	3.34	3.47	3.63	3.64	3.70	3.55	3.71	3.67	3.79	3.92	3.82	3.74
29	35	2.67	3.09	3.28	3.50	3.59	3.63	3.70	3.54	3.72	3.67	3.81	3.94	3.84	3.80
30	35	2.67	3.09	3.34	3.47	3.63	3.64	3.70	3.55	3.71	3.67	3.79	3.92	3.82	3.74
31	35	2.00	3.09	3.27	3.50	3.59	3.63	3.71	3.57	3.74	3.69	3.84	3.98	3.91	3.71
32	35	2.00	3.09	3.31	3.51	3.62	3.66	3.75	3.58	3.76	3.70	3.84	3.97	3.89	3.85
33	35	2.00	3.09	3.33	3.47	3.60	3.64	3.71	3.55	3.71	3.67	3.83	3.92	3.82	3.75
34	35	2.00	3.10	3.32	3.54	3.61	3.66	3.72	3.58	3.76	3.71	3.83	3.98	3.89	3.82
35	45	2.00	3.09	3.26	3.50	3.58	3.63	3.71	3.58	3.72	3.69	3.34	3.97	3.91	3.82
36	45	2.00	3.09	3.33	3.57	3.66	3.70	3.76	3.65	3.76	3.71	3.85	3.97	3.92	3.81
37	45	2.00	3.09	3.33	3.57	3.66	3.70	3.76	3.65	3.76	3.71	3.85	3.97	3.92	3.81
38	45	2.67	3.09	3.26	3.50	3.58	3.63	3.72	3.59	3.73	3.69	3.84	3.97	3.90	3.81
39	45	2.67	3.04	3.21	3.43	3.49	3.50	3.28	2.29	3.48	3.41	3.52	3.60	3.48	3.38
40	45	2.67	3.08	3.25	3.43	3.50	3.50	3.30	2.30	3.50	3.42	3.52	3.60	3.50	3.41

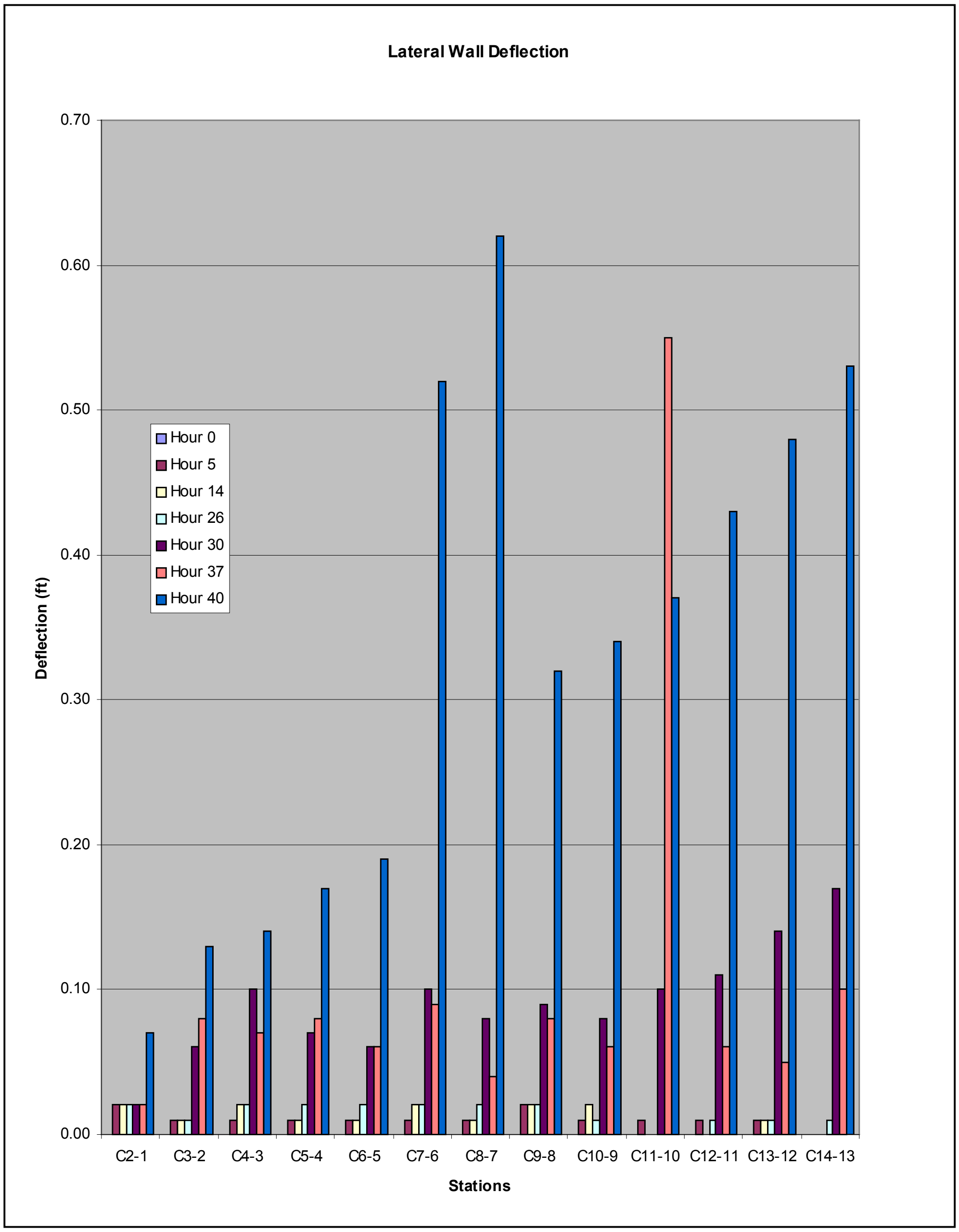

Figure 40. RDFW lateral deflections during dynamic loading

Table 7
Incident Wave Heights and Power Calculations for 10 Percent Gain and Dfront = 2.0 ft

				STATIONS		*measurement in FT							
Gain (%)	Dfront (ft)	T (sec)		C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
10	2.00	2	Incident Wave Ht (ft)	0.50	0.40	0.08	0.15	0.18	0.25	0.30	0.10	0.20	0.25	0.30	0.38	0.75

		RDFW GRIDS and ASSOCIATED WAVE HT PER GRID											
		C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
1	**Grid Incident Wave Ht, (ft)**	0.45	0.24	0.11	0.16	0.21	0.28	0.20	0.15	0.23	0.28	0.34	0.56
2	**Hydrostat Force (lb/LF), F1**	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80
3	**Total Force, (lb/LF), F2**	177.35	151.42	137.10	142.74	148.50	155.85	147.05	141.32	149.95	155.85	163.37	191.90
		METHOD 1 - DYNAMIC OVERTURNING MOMENT METHOD											
4	**Center of Pressure, hyd, d1**	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67
5	**Center of Pressure, total, d2**	0.82	0.75	0.70	0.72	0.74	0.76	0.73	0.72	0.74	0.76	0.78	0.85
6	**dW, F2d2 - F1d1**	61.58	29.70	13.32	19.67	26.29	34.95	24.61	18.06	27.99	34.95	44.05	80.63
7	**P=dW/dt, dt =T (ft-lb/s)**	30.79	14.85	6.66	9.84	13.14	17.47	12.30	9.03	13.99	17.47	22.02	40.32
8	**Power (hp/LF)**	0.06	0.03	0.01	0.02	0.02	0.03	0.02	0.02	0.03	0.03	0.04	0.07
9	**Power (kW/LF)**	0.08	0.04	0.02	0.02	0.03	0.04	0.03	0.02	0.03	0.04	0.05	0.10
10	**Power/4' section(hp/4 LF)**	0.22	0.11	0.05	0.07	0.10	0.13	0.09	0.07	0.10	0.13	0.16	0.29
11	**Power/4' section(kw/4 LF)**	0.30	0.14	0.06	0.10	0.13	0.17	0.12	0.09	0.14	0.17	0.21	0.39
		METHOD 2 - WAVE ENERGY DENSITY METHOD											
12	**Energy (ft-lb)/LF**	22.74	6.34	1.42	2.97	5.07	8.49	4.49	2.53	5.69	8.49	12.79	35.54
13	Power (ft-lb)/s/LF	11.37	3.17	0.71	1.48	2.54	4.25	2.25	1.26	2.84	4.25	6.40	17.77
14	**Power (hp/LF)**	0.02	0.01	0.00	0.00	0.00	0.01	0.00	0.00	0.01	0.01	0.01	0.03
15	**Power(KW/LF)**	0.03	0.01	0.00	0.00	0.01	0.01	0.01	0.00	0.01	0.01	0.02	0.04
16	**Power/4' section(hp/4 LF)**	0.08	0.02	0.01	0.01	0.02	0.03	0.02	0.01	0.02	0.03	0.05	0.13
17	**Power/4' section(kw/4 LF)**	0.11	0.03	0.01	0.01	0.02	0.04	0.02	0.01	0.03	0.04	0.06	0.17

	ONE HOUR TOTALS	**DOM METHOD**		**WED METHOD**		**Average of WED & DOM**	
18		**1.51**	**hph**	**0.42**	**hph**	**0.97**	**hph**
19		**2.03**	**kwh**	**0.57**	**kwh**	**1.30**	**kwh**

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Table 8
Incident Wave Heights and Power Calculations for 15 Percent Gain and Dfront = 2.0 ft

				STATIONS						*measurement in FT						
Gain (%)	Dfront (ft)	T (sec)		C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
15	2.00	2	Incident Wave Ht (ft)	0.65	0.55	0.30	0.20	0.35	0.50	0.35	0.35	0.30	0.25	0.40	0.65	0.65

		RDFW GRIDS and ASSOCIATED WAVE HT PER GRID											
		C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
1	Grid Incident Wave Ht, (ft)	0.60	0.43	0.25	0.28	0.43	0.43	0.35	0.33	0.28	0.33	0.53	0.65
2	Hydrostat Force (lb/LF), F1	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80
3	Total Force, (lb/LF), F2	196.87	174.20	152.89	155.85	174.20	174.20	164.90	161.85	155.85	161.85	186.99	203.60
		METHOD 1 - DYNAMIC OVERTURNING MOMENT METHOD											
4	Center of Pressure, hyd, d1	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67
5	Center of Pressure, total, d2	0.87	0.81	0.75	0.76	0.81	0.81	0.78	0.77	0.76	0.77	0.84	0.88
6	dW, F2d2 - F1d1	87.33	57.55	31.43	34.95	57.55	57.55	45.92	42.19	34.95	42.19	74.11	96.55
7	P=dW/dt, dt =T (ft-lb/s)	43.67	28.78	15.72	17.47	28.78	28.78	22.96	21.10	17.47	21.10	37.05	48.27
8	Power (hp/LF)	0.08	0.05	0.03	0.03	0.05	0.05	0.04	0.04	0.03	0.04	0.07	0.09
9	Power (kW/LF)	0.11	0.07	0.04	0.04	0.07	0.07	0.06	0.05	0.04	0.05	0.09	0.12
10	Power/4' section(hp/4 LF)	0.32	0.21	0.11	0.13	0.21	0.21	0.17	0.15	0.13	0.15	0.27	0.35
11	Power/4' section(kw/4 LF)	0.43	0.28	0.15	0.17	0.28	0.28	0.22	0.21	0.17	0.21	0.36	0.47
		METHOD 2 - WAVE ENERGY DENSITY METHOD											
12	Energy (ft-lb)/LF	40.44	20.29	7.02	8.49	20.29	20.29	13.76	11.86	8.49	11.86	30.96	47.46
13	Power (ft-lb)/s/LF	20.22	10.14	3.51	4.25	10.14	10.14	6.88	5.93	4.25	5.93	15.48	23.73
14	Power (hp/LF)	0.04	0.02	0.01	0.01	0.02	0.02	0.01	0.01	0.01	0.01	0.03	0.04
15	Power(KW/LF)	0.05	0.02	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0.04	0.06
16	Power/4' section(hp/4 LF)	0.15	0.07	0.03	0.03	0.07	0.07	0.05	0.04	0.03	0.04	0.11	0.17
17	Power/4' section(kw/4 LF)	0.20	0.10	0.03	0.04	0.10	0.10	0.07	0.06	0.04	0.06	0.15	0.23

	ONE HOUR TOTALS	DOM METHOD		WED METHOD		Average of WED & DOM	
18		2.41	hph	0.88	hph	1.64	hph
19		3.23	kwh	1.18	kwh	2.20	kwh

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Table 9
Incident Wave Heights and Power Calculations for 20 Percent Gain and Dfront = 2.0 ft

Gain (%)	Dfront (ft)	T (sec)		C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
									STATIONS		*measurement in FT					
20	2.00	2	Incident Wave Ht (ft)	0.75	0.75	0.50	0.50	0.50	0.52	0.52	0.52	0.75	0.70	0.70	0.80	0.80

		C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
		RDFW GRIDS and ASSOCIATED WAVE HT PER GRID											
1	Grid Incident Wave Ht, (ft)	0.75	0.63	0.50	0.50	0.51	0.52	0.52	0.64	0.73	0.70	0.75	0.80
2	Hydrostat Force (lb/LF), F1	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80
3	Total Force, (lb/LF), F2	217.38	200.22	183.75	183.75	184.72	185.69	186.01	201.57	213.89	210.43	217.38	224.44
		METHOD 1 - DYNAMIC OVERTURNING MOMENT METHOD											
4	Center of Pressure, hyd, d1	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67
5	Center of Pressure, total, d2	0.92	0.87	0.83	0.83	0.83	0.84	0.84	0.88	0.91	0.90	0.92	0.93
6	dW, F2d2 - F1d1	115.95	91.90	69.85	69.85	71.12	72.40	72.82	93.75	110.98	106.08	115.95	126.15
7	P=dW/dt, dt =T (ft-lb/s)	57.98	45.95	34.93	34.93	35.56	36.20	36.41	46.87	55.49	53.04	57.98	63.08
8	Power (hp/LF)	0.11	0.08	0.06	0.06	0.06	0.07	0.07	0.09	0.10	0.10	0.11	0.11
9	Power (kW/LF)	0.14	0.11	0.09	0.09	0.09	0.09	0.09	0.11	0.14	0.13	0.14	0.15
10	Power/4' section(hp/4 LF)	0.42	0.33	0.25	0.25	0.26	0.26	0.26	0.34	0.40	0.39	0.42	0.46
11	Power/4' section(kw/4 LF)	0.57	0.45	0.34	0.34	0.35	0.35	0.36	0.46	0.54	0.52	0.57	0.62
		METHOD 2 - WAVE ENERGY DENSITY METHOD											
12	Energy (ft-lb)/LF	63.18	43.88	28.08	28.08	28.93	29.79	30.08	45.29	59.04	55.04	63.18	71.88
13	Power (ft-lb)/s/LF	31.59	21.94	14.04	14.04	14.46	14.90	15.04	22.65	29.52	27.52	31.59	35.94
14	Power (hp/LF)	0.06	0.04	0.03	0.03	0.03	0.03	0.03	0.04	0.05	0.05	0.06	0.07
15	Power(KW/LF)	0.08	0.05	0.03	0.03	0.04	0.04	0.04	0.06	0.07	0.07	0.08	0.09
16	Power/4' section(hp/4 LF)	0.23	0.16	0.10	0.10	0.11	0.11	0.11	0.16	0.21	0.20	0.23	0.26
17	Power/4' section(kw/4 LF)	0.31	0.21	0.14	0.14	0.14	0.15	0.15	0.22	0.29	0.27	0.31	0.35

	ONE HOUR TOTALS	DOM METHOD		WED METHOD		Average of WED & DOM	
18		4.06	hph	1.99	hph	3.02	hph
19		5.45	kwh	2.66	kwh	4.06	kwh

Table 10
Incident Wave Heights and Power Calculations for 25 Percent Gain and Dfront = 2.0 ft

STATIONS *measurement in FT

Gain (%)	Dfront (ft)	T (sec)		C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
25	2.00	2	Incident Wave Ht (ft)	0.80	0.80	0.55	0.55	0.65	0.70	0.70	0.70	0.70	0.65	0.70	0.80	0.80

RDFW GRIDS and ASSOCIATED WAVE HT PER GRID

		C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
1	Grid Incident Wave Ht, (ft)	0.80	0.68	0.55	0.60	0.68	0.70	0.70	0.70	0.68	0.68	0.75	0.80
2	Hydrostat Force (lb/LF), F1	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80
3	Total Force, (lb/LF), F2	224.44	207.00	190.25	196.87	207.00	210.43	210.43	210.43	207.00	207.00	217.38	224.44
		METHOD 1 - DYNAMIC OVERTURNING MOMENT METHOD											
4	Center of Pressure, hyd, d1	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67
5	Center of Pressure, total, d2	0.93	0.89	0.85	0.87	0.89	0.90	0.90	0.90	0.89	0.89	0.92	0.93
6	dW, F2d2 - F1d1	126.15	101.27	78.44	87.33	101.27	106.08	106.08	106.08	101.27	101.27	115.95	126.15
7	P=dW/dt, dt =T (ft-lb/s)	63.08	50.64	39.22	43.67	50.64	53.04	53.04	53.04	50.64	50.64	57.98	63.08
8	Power (hp/LF)	0.11	0.09	0.07	0.08	0.09	0.10	0.10	0.10	0.09	0.09	0.11	0.11
9	Power (kW/LF)	0.15	0.12	0.10	0.11	0.12	0.13	0.13	0.13	0.12	0.12	0.14	0.15
10	Power/4' section(hp/4 LF)	0.46	0.37	0.29	0.32	0.37	0.39	0.39	0.39	0.37	0.37	0.42	0.46
11	Power/4' section(kw/4 LF)	0.62	0.49	0.38	0.43	0.49	0.52	0.52	0.52	0.49	0.49	0.57	0.62
		METHOD 2 - WAVE ENERGY DENSITY METHOD											
12	Energy (ft-lb)/LF	71.88	51.18	33.98	40.44	51.18	55.04	55.04	55.04	51.18	51.18	63.18	71.88
13	Power (ft-lb)/s/LF	35.94	25.59	16.99	20.22	25.59	27.52	27.52	27.52	25.59	25.59	31.59	35.94
14	Power (hp/LF)	0.07	0.05	0.03	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.07
15	Power(KW/LF)	0.09	0.06	0.04	0.05	0.06	0.07	0.07	0.07	0.06	0.06	0.08	0.09
16	Power/4' section(hp/4 LF)	0.26	0.19	0.12	0.15	0.19	0.20	0.20	0.20	0.19	0.19	0.23	0.26
17	Power/4' section(kw/4 LF)	0.35	0.25	0.17	0.20	0.25	0.27	0.27	0.27	0.25	0.25	0.31	0.35

	ONE HOUR TOTALS	DOM METHOD		WED METHOD		Average of WED & DOM	
18		4.57	hph	2.37	hph	3.47	hph
19		6.13	kwh	3.18	kwh	4.65	kwh

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Table 11
Incident Wave Heights and Power Calculations for 35 Percent Gain and Dfront = 2.0 ft

STATIONS *measurement in FT

Gain (%)	Dfront (ft)	T (sec)		C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
35	2.00	2	Incident Wave Ht (ft)	1.05	1.05	0.70	0.75	0.88	1.15	1.15	1.00	0.80	0.75	1.00	1.15	1.15

		RDFW GRIDS and ASSOCIATED WAVE HT PER GRID											
		C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
1	Grid Incident Wave Ht, (ft)	1.05	0.88	0.73	0.81	1.01	1.15	1.08	0.90	0.78	0.88	1.08	1.15
2	Hydrostat Force (lb/LF), F1	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80
3	Total Force, (lb/LF), F2	261.41	235.24	213.89	226.22	255.69	276.97	265.26	238.90	220.90	235.24	265.26	276.97
		METHOD 1 - DYNAMIC OVERTURNING MOMENT METHOD											
4	Center of Pressure, hyd, d1	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67
5	Center of Pressure, total, d2	1.02	0.96	0.91	0.94	1.00	1.05	1.02	0.97	0.92	0.96	1.02	1.05
6	dW, F2d2 - F1d1	182.38	142.10	110.98	128.76	173.38	207.41	188.50	147.58	121.01	142.10	188.50	207.41
7	P=dW/dt, dt =T (ft-lb/s)	91.19	71.05	55.49	64.38	86.69	103.71	94.25	73.79	60.51	71.05	94.25	103.71
8	Power (hp/LF)	0.17	0.13	0.10	0.12	0.16	0.19	0.17	0.13	0.11	0.13	0.17	0.19
9	Power (kW/LF)	0.22	0.17	0.14	0.16	0.21	0.25	0.23	0.18	0.15	0.17	0.23	0.25
10	Power/4' section(hp/4 LF)	0.66	0.52	0.40	0.47	0.63	0.75	0.69	0.54	0.44	0.52	0.69	0.75
11	Power/4' section(kw/4 LF)	0.89	0.69	0.54	0.63	0.85	1.01	0.92	0.72	0.59	0.69	0.92	1.01
		METHOD 2 - WAVE ENERGY DENSITY METHOD											
12	Energy (ft-lb)/LF	123.83	86.00	59.04	74.15	115.15	148.54	129.80	90.98	67.46	86.00	129.80	148.54
13	Power (ft-lb)/s/LF	61.92	43.00	29.52	37.07	57.57	74.27	64.90	45.49	33.73	43.00	64.90	74.27
14	Power (hp/LF)	0.11	0.08	0.05	0.07	0.10	0.14	0.12	0.08	0.06	0.08	0.12	0.14
15	Power(KW/LF)	0.15	0.10	0.07	0.09	0.14	0.18	0.16	0.11	0.08	0.10	0.16	0.18
16	Power/4' section(hp/4 LF)	0.45	0.31	0.21	0.27	0.42	0.54	0.47	0.33	0.25	0.31	0.47	0.54
17	Power/4' section(kw/4 LF)	0.60	0.42	0.29	0.36	0.56	0.72	0.63	0.44	0.33	0.42	0.63	0.72

	ONE HOUR TOTALS	DOM METHOD		WED METHOD		Average of WED & DOM	
18		7.05	hph	4.58	hph	5.82	hph
19		9.46	kwh	6.14	kwh	7.80	kwh

Table 12
Incident Wave Heights and Power Calculations for 45 Percent Gain and Dfront = 2.0 ft

				STATIONS						*measurement in FT						
Gain (%)	Dfront (ft)	T (sec)		C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
45	2.00	2	Incident Wave Ht (ft)	1.2	1.20	1.00	1.20	1.50	1.60	1.75	1.75	1.60	1.50	1.30	1.60	1.60

		RDFW GRIDS and ASSOCIATED WAVE HT PER GRID											
		C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
1	Grid Incident Wave Ht, (ft)	1.20	1.10	1.10	1.35	1.55	1.68	1.75	1.68	1.55	1.40	1.45	1.60
2	Hydrostat Force (lb/LF), F1	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80	124.80
3	Total Force, (lb/LF), F2	284.92	269.13	269.13	309.43	343.66	365.96	379.67	365.96	343.66	317.82	326.32	352.50
		METHOD 1 - DYNAMIC OVERTURNING MOMENT METHOD											
4	Center of Pressure, hyd, d1	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67
5	Center of Pressure, total, d2	1.07	1.03	1.03	1.12	1.18	1.22	1.25	1.22	1.18	1.13	1.15	1.20
6	dW, F2d2 - F1d1	220.49	194.71	194.71	262.07	323.14	364.74	391.00	364.74	323.14	276.72	291.78	339.46
7	P=dW/dt, dt =T (ft-lb/s)	110.25	97.35	97.35	131.03	161.57	182.37	195.50	182.37	161.57	138.36	145.89	169.73
8	Power (hp/LF)	0.20	0.18	0.18	0.24	0.29	0.33	0.36	0.33	0.29	0.25	0.27	0.31
9	Power (kW/LF)	0.27	0.24	0.24	0.32	0.39	0.44	0.48	0.44	0.39	0.34	0.36	0.41
10	Power/4' section(hp/4 LF)	0.80	0.71	0.71	0.95	1.18	1.33	1.42	1.33	1.18	1.01	1.06	1.23
11	Power/4' section(kw/4 LF)	1.08	0.95	0.95	1.28	1.58	1.78	1.91	1.78	1.58	1.35	1.42	1.66
		METHOD 2 - WAVE ENERGY DENSITY METHOD											
12	Energy (ft-lb)/LF	161.74	135.91	135.91	204.70	269.85	315.13	343.98	315.13	269.85	220.15	236.15	287.54
13	Power (ft-lb)/s/LF	80.87	67.95	67.95	102.35	134.92	157.56	171.99	157.56	134.92	110.07	118.08	143.77
14	Power (hp/LF)	0.15	0.12	0.12	0.19	0.25	0.29	0.31	0.29	0.25	0.20	0.21	0.26
15	Power(KW/LF)	0.20	0.17	0.17	0.25	0.33	0.38	0.42	0.38	0.33	0.27	0.29	0.35
16	Power/4' section(hp/4 LF)	0.59	0.49	0.49	0.74	0.98	1.15	1.25	1.15	0.98	0.80	0.86	1.05
17	Power/4' section(kw/4 LF)	0.79	0.66	0.66	1.00	1.32	1.54	1.68	1.54	1.32	1.07	1.15	1.40

	ONE HOUR TOTALS	DOM METHOD		WED METHOD		Average of WED & DOM	
18		12.90	hph	10.53	hph	11.71	hph
19		17.30	kwh	14.12	kwh	15.71	kwh

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Table 13
Incident Wave Heights and Power Calculations for 15 Percent Gain and Dfront = 2.7 ft

STATIONS *measurement in FT

Gain (%)	Dfront (ft)	T (sec)		C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
15	2.67	2	Incident Wave Ht (ft)	1.55	1.55	1.10	0.45	0.58	0.60	0.85	0.75	0.75	0.63	0.70	1.25	1.25

RDFW GRIDS and ASSOCIATED WAVE HT PER GRID

		C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
1	Grid Incident Wave Ht, (ft)	1.55	1.33	0.78	0.51	0.59	0.73	0.80	0.75	0.69	0.66	0.98	1.25
2	Hydrostat Force (lb/LF), F1	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42
3	Total Force, (lb/LF), F2	467.57	426.69	334.34	294.05	305.31	326.48	338.31	330.40	320.64	316.77	366.68	413.47
	METHOD 1 - DYNAMIC OVERTURNING MOMENT METHOD												
4	Center of Pressure, hyd, d1	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89
5	Center of Pressure, total, d2	1.41	1.33	1.15	1.06	1.08	1.13	1.16	1.14	1.12	1.11	1.21	1.31
6	dW, F2d2 - F1d1	459.30	369.89	185.79	113.86	133.43	171.34	193.16	178.52	160.73	153.77	247.32	341.97
7	P=dW/dt, dt =T (ft-lb/s)	229.65	184.94	92.90	56.93	66.71	85.67	96.58	89.26	80.37	76.89	123.66	170.98
8	Power (hp/LF)	0.42	0.34	0.17	0.10	0.12	0.16	0.18	0.16	0.15	0.14	0.22	0.31
9	Power (kW/LF)	0.56	0.45	0.23	0.14	0.16	0.21	0.24	0.22	0.20	0.19	0.30	0.42
10	Power/4' section(hp/4 LF)	1.67	1.35	0.68	0.41	0.49	0.62	0.70	0.65	0.58	0.56	0.90	1.24
11	Power/4' section(kw/4 LF)	2.24	1.80	0.91	0.56	0.65	0.84	0.94	0.87	0.78	0.75	1.21	1.67
	METHOD 2 - WAVE ENERGY DENSITY METHOD												
12	Energy (ft-lb)/LF	269.85	197.19	67.46	29.50	38.77	59.04	71.88	63.18	53.09	49.30	106.77	175.50
13	Power (ft-lb)/s/LF	134.92	98.60	33.73	14.75	19.38	29.52	35.94	31.59	26.54	24.65	53.39	87.75
14	Power (hp/LF)	0.25	0.18	0.06	0.03	0.04	0.05	0.07	0.06	0.05	0.04	0.10	0.16
15	Power(KW/LF)	0.33	0.24	0.08	0.04	0.05	0.07	0.09	0.08	0.06	0.06	0.13	0.21
16	Power/4' section(hp/4 LF)	0.98	0.72	0.25	0.11	0.14	0.21	0.26	0.23	0.19	0.18	0.39	0.64
17	Power/4' section(kw/4 LF)	1.32	0.96	0.33	0.14	0.19	0.29	0.35	0.31	0.26	0.24	0.52	0.86

	ONE HOUR TOTALS	DOM METHOD		WED METHOD		Average of WED & DOM	
18		9.85	hph	4.30	hph	7.07	hph
19		13.21	kwh	5.76	kwh	9.49	kwh

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Table 14
Incident Wave Heights and Power Calculations for 20 Percent Gain and Dfront = 2.7 ft

				C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
									STATIONS		*measurement in FT					
Gain (%)	Dfront (ft)	T (sec)		C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
20	2.67	2	Incident Wave Ht (ft)	1.80	1.80	1.25	0.55	0.63	0.70	1.00	0.90	0.85	0.75	0.80	1.35	1.30

		RDFW GRIDS and ASSOCIATED WAVE HT PER GRID											
		C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
1	Grid Incident Wave Ht, (ft)	1.80	1.53	0.90	0.59	0.66	0.85	0.95	0.88	0.80	0.78	1.08	1.33
2	Hydrostat Force (lb/LF), F1	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42
3	Total Force, (lb/LF), F2	515.09	462.94	354.39	305.31	316.77	346.30	362.56	350.33	338.31	334.34	383.38	426.69
		METHOD 1 - DYNAMIC OVERTURNING MOMENT METHOD											
4	Center of Pressure, hyd, d1	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89
5	Center of Pressure, total, d2	1.49	1.40	1.19	1.08	1.11	1.17	1.21	1.18	1.16	1.15	1.25	1.33
6	dW, F2d2 - F1d1	568.96	448.94	223.54	133.43	153.77	208.16	239.30	215.81	193.16	185.79	280.36	369.89
7	P=dW/dt, dt =T (ft-lb/s)	284.48	224.47	111.77	66.71	76.89	104.08	119.65	107.90	96.58	92.90	140.18	184.94
8	Power (hp/LF)	0.52	0.41	0.20	0.12	0.14	0.19	0.22	0.20	0.18	0.17	0.25	0.34
9	Power (kW/LF)	0.69	0.55	0.27	0.16	0.19	0.25	0.29	0.26	0.24	0.23	0.34	0.45
10	Power/4' section(hp/4 LF)	2.07	1.63	0.81	0.49	0.56	0.76	0.87	0.78	0.70	0.68	1.02	1.35
11	Power/4' section(kw/4 LF)	2.77	2.19	1.09	0.65	0.75	1.02	1.17	1.05	0.94	0.91	1.37	1.80
		METHOD 2 - WAVE ENERGY DENSITY METHOD											
12	Energy (ft-lb)/LF	363.92	261.21	90.98	38.77	49.30	81.15	101.37	86.00	71.88	67.46	129.80	197.19
13	Power (ft-lb)/s/LF	181.96	130.61	45.49	19.38	24.65	40.58	50.68	43.00	35.94	33.73	64.90	98.60
14	Power (hp/LF)	0.33	0.24	0.08	0.04	0.04	0.07	0.09	0.08	0.07	0.06	0.12	0.18
15	Power(KW/LF)	0.44	0.32	0.11	0.05	0.06	0.10	0.12	0.10	0.09	0.08	0.16	0.24
16	Power/4' section(hp/4 LF)	1.32	0.95	0.33	0.14	0.18	0.30	0.37	0.31	0.26	0.25	0.47	0.72
17	Power/4' section(kw/4 LF)	1.77	1.27	0.44	0.19	0.24	0.40	0.49	0.42	0.35	0.33	0.63	0.96

	ONE HOUR TOTALS	DOM METHOD		WED METHOD		Average of WED & DOM	
18		11.71	hph	5.60	hph	8.65	hph
19		15.71	kwh	7.50	kwh	11.61	kwh

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Table 15
Incident Wave Heights and Power Calculations for 25 Percent Gain and Dfront = 2.7 ft

Gain (%)	Dfront (ft)	T (sec)		C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
									STATIONS		*measurement in FT					
25	**2.67**	**2**	**Incident Wave Ht (ft)**	**1.75**	**1.75**	**1.20**	**1.00**	**1.00**	**0.90**	**1.10**	**1.05**	**1.05**	**1.10**	**1.10**	**1.40**	**1.30**

		C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
		RDFW GRIDS and ASSOCIATED WAVE HT PER GRID											
1	Grid Incident Wave Ht, (ft)	1.75	1.48	1.10	1.00	0.95	1.00	1.08	1.05	1.08	1.10	1.25	1.35
2	Hydrostat Force (lb/LF), F1	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42
3	Total Force, (lb/LF), F2	505.41	453.74	387.62	370.82	362.56	370.82	383.38	379.18	383.38	387.62	413.47	431.15
		METHOD 1 - DYNAMIC OVERTURNING MOMENT METHOD											
4	Center of Pressure, hyd, d1	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89
5	Center of Pressure, total, d2	1.47	1.38	1.26	1.22	1.21	1.22	1.25	1.24	1.25	1.26	1.31	1.34
6	dW, F2d2 - F1d1	546.13	428.54	288.86	255.43	239.30	255.43	280.36	271.95	280.36	288.86	341.97	379.40
7	P=dW/dt, dt =T (ft-lb/s)	273.07	214.27	144.43	127.72	119.65	127.72	140.18	135.98	140.18	144.43	170.98	189.70
8	Power (hp/LF)	0.50	0.39	0.26	0.23	0.22	0.23	0.25	0.25	0.25	0.26	0.31	0.34
9	Power (kW/LF)	0.67	0.52	0.35	0.31	0.29	0.31	0.34	0.33	0.34	0.35	0.42	0.46
10	Power/4' section(hp/4 LF)	1.99	1.56	1.05	0.93	0.87	0.93	1.02	0.99	1.02	1.05	1.24	1.38
11	Power/4' section(kw/4 LF)	2.66	2.09	1.41	1.25	1.17	1.25	1.37	1.33	1.37	1.41	1.67	1.85
		METHOD 2 - WAVE ENERGY DENSITY METHOD											
12	Energy (ft-lb)/LF	343.98	244.37	135.91	112.32	101.37	112.32	129.80	123.83	129.80	135.91	175.50	204.70
13	Power (ft-lb)/s/LF	171.99	122.18	67.95	56.16	50.68	56.16	64.90	61.92	64.90	67.95	87.75	102.35
14	Power (hp/LF)	0.31	0.22	0.12	0.10	0.09	0.10	0.12	0.11	0.12	0.12	0.16	0.19
15	Power(KW/LF)	0.42	0.30	0.17	0.14	0.12	0.14	0.16	0.15	0.16	0.17	0.21	0.25
16	Power/4' section(hp/4 LF)	1.25	0.89	0.49	0.41	0.37	0.41	0.47	0.45	0.47	0.49	0.64	0.74
17	Power/4' section(kw/4 LF)	1.68	1.19	0.66	0.55	0.49	0.55	0.63	0.60	0.63	0.66	0.86	1.00

	ONE HOUR TOTALS	DOM METHOD	WED METHOD	Average of WED & DOM
18		14.02 hph	7.09 hph	10.56 hph
19		18.81 kwh	9.51 kwh	14.16 kwh

Table 16
Incident Wave Heights and Power Calculations for 35 Percent Gain and Dfront = 2.7 ft

				STATIONS	*measurement in FT											
Gain (%)	Dfront (ft)	T (sec)		C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
35	2.67	2	Incident Wave Ht (ft)	1.55	1.55	1.10	1.15	1.30	1.20	1.40	1.55	1.40	1.15	1.25	1.55	1.55

		RDFW GRIDS and ASSOCIATED WAVE HT PER GRID											
		C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
1	Grid Incident Wave Ht, (ft)	1.55	1.33	1.13	1.23	1.25	1.30	1.48	1.48	1.28	1.20	1.40	1.55
2	Hydrostat Force (lb/LF), F1	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42
3	Total Force, (lb/LF), F2	467.57	426.69	391.87	409.10	413.47	422.26	453.74	453.74	417.85	404.76	440.12	467.57
		METHOD 1 - DYNAMIC OVERTURNING MOMENT METHOD											
4	Center of Pressure, hyd, d1	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89
5	Center of Pressure, total, d2	1.41	1.33	1.26	1.30	1.31	1.32	1.38	1.38	1.31	1.29	1.36	1.41
6	dW, F2d2 - F1d1	459.30	369.89	297.46	332.86	341.97	360.48	428.54	428.54	351.17	323.86	398.74	459.30
7	P=dW/dt, dt =T (ft-lb/s)	229.65	184.94	148.73	166.43	170.98	180.24	214.27	214.27	175.59	161.93	199.37	229.65
8	Power (hp/LF)	0.42	0.34	0.27	0.30	0.31	0.33	0.39	0.39	0.32	0.29	0.36	0.42
9	Power (kW/LF)	0.56	0.45	0.36	0.41	0.42	0.44	0.52	0.52	0.43	0.39	0.49	0.56
10	Power/4' section(hp/4 LF)	1.67	1.35	1.08	1.21	1.24	1.31	1.56	1.56	1.28	1.18	1.45	1.67
11	Power/4' section(kw/4 LF)	2.24	1.80	1.45	1.62	1.67	1.76	2.09	2.09	1.71	1.58	1.94	2.24
		METHOD 2 - WAVE ENERGY DENSITY METHOD											
12	Energy (ft-lb)/LF	269.85	197.19	142.16	168.55	175.50	189.82	244.37	244.37	182.59	161.74	220.15	269.85
13	Power (ft-lb)/s/LF	134.92	98.60	71.08	84.28	87.75	94.91	122.18	122.18	91.30	80.87	110.07	134.92
14	Power (hp/LF)	0.25	0.18	0.13	0.15	0.16	0.17	0.22	0.22	0.17	0.15	0.20	0.25
15	Power(KW/LF)	0.33	0.24	0.17	0.21	0.21	0.23	0.30	0.30	0.22	0.20	0.27	0.33
16	Power/4' section(hp/4 LF)	0.98	0.72	0.52	0.61	0.64	0.69	0.89	0.89	0.66	0.59	0.80	0.98
17	Power/4' section(kw/4 LF)	1.32	0.96	0.69	0.82	0.86	0.93	1.19	1.19	0.89	0.79	1.07	1.32
	ONE HOUR TOTALS	DOM METHOD		WED METHOD		Average of WED & DOM							
18		16.55	hph	8.97	hph	12.76 hph							
19		22.20	kwh	12.03	kwh	17.11 kwh							

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Table 17
Incident Wave Heights and Power Calculations for 45 Percent Gain and Dfront = 2.7 ft

STATIONS *measurement in FT

Gain (%)	Dfront (ft)	T (sec)		C2-1	C3-2	C4-3	C5-4	C6-5	C7-6	C8-7	C9-8	C10-9	C11-10	C12-11	C13-12	C14-13
45	2.67	2	Incident Wave Ht (ft)	1.45	1.45	1.15	1.40	1.65	1.54	1.40	1.33	1.54	1.33	1.40	1.50	1.50

RDFW GRIDS and ASSOCIATED WAVE HT PER GRID

		C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13
1	Grid Incident Wave Ht, (ft)	1.45	1.30	1.28	1.53	1.59	1.47	1.36	1.43	1.43	1.36	1.45	1.50
2	Hydrostat Force (lb/LF), F1	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42	222.42
3	Total Force, (lb/LF), F2	449.18	422.26	417.85	462.94	475.49	452.37	433.38	446.00	446.00	433.38	449.18	458.33
	METHOD 1 - DYNAMIC OVERTURNING MOMENT METHOD												
4	Center of Pressure, hyd, d1	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89
5	Center of Pressure, total, d2	1.37	1.32	1.31	1.40	1.42	1.38	1.34	1.37	1.37	1.34	1.37	1.39
6	dW, F2d2 - F1d1	418.50	360.48	351.17	448.94	477.16	425.51	384.20	411.54	411.54	384.20	418.50	438.68
7	P=dW/dt, dt =T (ft-lb/s)	209.25	180.24	175.59	224.47	238.58	212.76	192.10	205.77	205.77	192.10	209.25	219.34
8	Power (hp/LF)	0.38	0.33	0.32	0.41	0.43	0.39	0.35	0.37	0.37	0.35	0.38	0.40
9	Power (kW/LF)	0.51	0.44	0.43	0.55	0.58	0.52	0.47	0.50	0.50	0.47	0.51	0.53
10	Power/4' section(hp/4 LF)	1.52	1.31	1.28	1.63	1.74	1.55	1.40	1.50	1.50	1.40	1.52	1.60
11	Power/4' section(kw/4 LF)	2.04	1.76	1.71	2.19	2.33	2.07	1.87	2.01	2.01	1.87	2.04	2.14
	METHOD 2 - WAVE ENERGY DENSITY METHOD												
12	Energy (ft-lb)/LF	236.15	189.82	182.59	261.21	284.85	241.89	208.51	230.49	230.49	208.51	236.15	252.72
13	Power (ft-lb)/s/LF	118.08	94.91	91.30	130.61	142.42	120.94	104.26	115.24	115.24	104.26	118.08	126.36
14	Power (hp/LF)	0.21	0.17	0.17	0.24	0.26	0.22	0.19	0.21	0.21	0.19	0.21	0.23
15	Power(KW/LF)	0.29	0.23	0.22	0.32	0.35	0.29	0.25	0.28	0.28	0.25	0.29	0.31
16	Power/4' section(hp/4 LF)	0.86	0.69	0.66	0.95	1.04	0.88	0.76	0.84	0.84	0.76	0.86	0.92
17	Power/4' section(kw/4 LF)	1.15	0.93	0.89	1.27	1.39	1.18	1.02	1.12	1.12	1.02	1.15	1.23

	ONE HOUR TOTALS	DOM METHOD	WED METHOD	Average of WED & DOM
18		17.93 hph	10.05 hph	13.99 hph
19		24.04 kwh	13.48 kwh	18.76 kwh

mean incident wave heights for each of the 12 stacks of four grids. This is the average wave height for those particular 4 lin ft of wall. No. 2 shows the hydrostatic force, F_1, exerted on the wall based on Eq. 5 for the given water depth. No. 3 gives the total force, F_2, based on Eq. 6. No. 4 gives the hydrostatic center of pressure, d_1, from the triangular pressure distribution as measured from the base of the structure (refer to Figure 6). No. 5 is the vertical distance from the base to the center of pressure, d_2, as calculated with Eq. 7 as a result of the larger triangular pressure distribution from the total force load. No. 6 is the change in work on the wall resulting from waves. No. 7 is the average power for one wavelength as shown in Eq. 8. No. 8 expresses power in terms of horsepower. No. 9 expresses power in terms of kilowatts. No. 10 and No. 11 give the power for each 4-ft section of wall in terms of horsepower and kilowatt per 4 lin ft of wall, respectively. Calculations for the WED Method begins with No. 12. Here the energy content per unit width is calculated as per Eq. 16. No. 13 shows the average power per unit width based on Eq. 18. Nos. 14-17 provides power calculations in various units. Finally No. 18 and No. 19 show the summed results for the DOM and WED methods, and the average of the two results. The preferred units to express the power expended on the wall are kilowatt-hours (kwh) which most people are familiar with from reading electric meters. A kilowatt-hour is a sustained kilowatt for a 1 hr duration.

Table 18 provides summary comments of significant events encountered during the dynamic-load tests. Information is given for each wave run. Table 19 summarizes the results of the wave-induced dynamic load tests in terms of duration, number of cycles and cumulative power. For the eleven combinations of wave height and water depth the table shows both the duration of each individual combination and the cumulative duration for the entire series. It also shows the cumulative number of individual waves (at a 2 sec wave period there are 1,800 waves per hour). Finally the table shows the cumulative amount of power the wall was exposed to.

In order to provide some guidance to the field the power results from Table 7-17 and Table 19 can be used as a baseline against some arbitrarily selected standard wave heights. The results are based on a 48-ft wall length and wall height of 4-ft. Using the aforementioned damage interval of 23, 32, and 40 hours (the three times when the wall was repaired), and the associated cumulative power at that time in the test series, several preliminary design standards can be extrapolated.

Table 20 gives the estimated durability of RDFW for a wave period, T = 2 sec, and certain combinations of idealized wave height and water depth. The table gives eight sets of combinations of wave height and water depth for each of the three damage intervals of 23, 32, and 40 hours. For each of the eight wave height and water depth combinations, an estimated 1-hr power content of the wave packet is calculated. For example, an H = 0.5 ft, and d = 2.0 ft the 1-hr power content is 2.87 kwh. In another case, for H = 2.0 ft and d = 2.67 ft the 1-hr power content is 32.60 kwh. Dividing these values into the RDFW test cumulative power results give an estimate of the duration the RDFW can sustain wave action with that amount of damage. For the two examples, based on the 23-hour damage duration, the wall would warrant repair after 55 hours (99,094 waves) for H = 0.5 ft, d = 2.0 ft , and after 6 hours of wave action (10,395 waves) for H = 2.0 ft and d = 2.67 ft.

Table 18
Summary Comments on Wave-induced Dynamic Load Testing of RDFW

DATE	TIME	RUN NO.	Gain (%)	Mean Wave Height (ft)	Water Depth (ft)	Comments
21-Apr	1600	1	10	0.42	2.00	Minimal wave action moved on to 15% Gain
22-Apr	1245	2	15	0.54	2.00	Wave action more intense than anticipated
22-Apr	1400	3	15	0.54	2.00	Conical abutments causing cross wave
22-Apr	1510	4	15	0.54	2.00	
22-Apr	1630	5	15	0.54	2.00	
24-Apr	1445	6	15	1.05	2.67	
24-Apr	1545	7	15	1.05	2.67	
24-Apr	1700	8	15	1.05	2.67	
24-Apr	1800	9	15	1.05	2.67	
25-Apr	820	10	20	0.74	2.00	
25-Apr	1010	11	20	0.74	2.00	
25-Apr	1105	12	20	0.74	2.00	
25-Apr	1200	13	20	0.74	2.00	
25-Apr	1400	14	20	0.74	2.00	Unintentional extra run at Hour 14
25-Apr	1559	15	20	1.17	2.67	
25-Apr	1659	16	20	1.17	2.67	
26-Apr	845	17	20	1.17	2.67	
26-Apr	1009	18	20	1.17	2.67	
26-Apr	1256	19	25	0.79	2.00	Onset of damage to Section 1
26-Apr	1407	20	25	0.79	2.00	
26-Apr	1530	21	25	0.79	2.00	
26-Apr	1638	22	25	0.79	2.00	
27-Apr	1210	23	25	1.31	2.67	Damage to Section 1 repaired after this run
27-Apr	1305	24	25	1.31	2.67	Onset of wall overtopping
27-Apr	1430	25	25	1.31	2.67	
27-Apr	1600	26	25	1.31	2.67	
28-Apr	1020	27	35	1.46	2.67	Heavy overtopping
28-Apr	1140	28	35	1.46	2.67	Onset of Damage to Section 2
28-Apr	1402	29	35	1.46	2.67	
28-Apr	1650	30	35	1.46	2.67	
1-May	1504	31	35	1.04	2.00	
	1606	32	35	1.04	2.00	Damage to Section 2 repaired after this run
2-May	938	33	35	1.04	2.00	
	1048	34	35	1.04	2.00	
2-May	1345	35	45	1.49	2.00	Only 3 hours @ 45%, d = 2 ft due to
	1449	36	45	1.49	2.00	budget and time constraints
	1551	37	45	1.49	2.00	Onset of damage to Section 3
3-May	1009	38	45	1.52	2.67	Only 3 hours @ 45%, d = 2.67 ft due to
	1517	39	45	1.52	2.67	budget and time constraints
4-May	950	40	45	1.52	2.67	Final damage to Section 3 ends test series

Table 19
Summary of RDFW Test Series

Gain (%)	Water Depth (ft)	Mean Wave Height (ft)	Wave Height St. Dev (ft)	Duration at Gain (hr)	Cummulative Duration (hr)	Cummulative No. of Cycles	Cummulative Avg. Power (kwh)
10	2.00	0.42	0.18	1	1	1,800	1
15	2.00	0.54	0.16	4	5	9,000	10
20	2.00	0.74	0.13	4	9	16,200	26
25	2.00	0.79	0.09	4	13	23,400	45
35	2.00	1.04	0.17	4	17	30,600	76
45	2.00	1.49	0.24	4	21	37,800	139
15	2.67	1.05	0.37	4	25	45,000	177
20	2.67	1.17	0.42	5	30	54,000	235
25	2.67	1.31	0.27	4	34	61,200	292
35	2.67	1.46	0.18	3	37	66,600	343
45	2.67	1.52	0.12	3	40	72,000	399

Table 20
Estimated Durability of RDFW for Idealized Wave and Water Depth Conditions

Water Depth (ft)	Mean Wave Height (ft)	Estimated 1-hr Power (kwh)	RDFW Test Duration (hr)	RDFW Test Cumm. Power (kwh)	Equivilent Duration (hr)	Equivilent No. of Cycles
2.00	0.50	2.87	23	158	55	99,094
2.00	1.00	8.27	23	158	19	34,389
2.00	1.50	16.83	23	158	9	16,898
2.00	2.00	27.36	23	158	6	10,395
2.67	0.50	4.98	23	158	32	57,108
2.67	1.00	11.55	23	158	14	24,623
2.67	1.50	20.71	23	158	8	13,732
2.67	2.00	32.60	23	158	5	8,724
2.00	0.50	2.87	32	263	92	164,948
2.00	1.00	8.27	32	263	32	57,243
2.00	1.50	16.83	32	263	16	28,128
2.00	2.00	27.36	32	263	10	17,303
2.67	0.50	4.98	32	263	53	95,060
2.67	1.00	11.55	32	263	23	40,987
2.67	1.50	20.71	32	263	13	22,859
2.67	2.00	32.60	32	263	8	14,521
2.00	0.50	2.87	40	399	139	250,244
2.00	1.00	8.27	40	399	48	86,844
2.00	1.50	16.83	40	399	24	42,674
2.00	2.00	27.36	40	399	15	26,250
2.67	0.50	4.98	40	399	80	144,217
2.67	1.00	11.55	40	399	35	62,182
2.67	1.50	20.71	40	399	19	34,679
2.67	2.00	32.60	40	399	12	22,031

4 Discussion of Results

Hydrostatic Head Tests and Underseepage

The RDFW wall was subject to over 128 hours of hydrostatic head levels between 2.0 ft and 3.33 ft. During this time a minimal amount of sand fill was lost from the 50-ft wall. There was no lateral deflection associated with these loads and no damage was observed. The wall essentially remained unchanged for the duration of the test series. There was no observable through seepage, but there was underseepage. The highest head level was d = 3.33 ft or 83 percent of the total wall height. Under this condition, the maximum-recorded underseepage rate was 22.8 gal of water per linear foot of wall per hour. While the RDFW wall was founded on a concrete surface, this is typical of a foundation condition that typically exists in an urban flood fight scenario. Placement of the RDFW wall on a soil foundation may yield different results. On a soil foundation, placing and keying a RDFW wall in a shallow trench may eliminate underseepage. From the underseepage rates given in Table 1, a field engineer has some guidance on anticipated amount of water that may become impounded on the dry side of the wall due to underseepage, as a function of time and wall length. The engineer can then select the appropriate pump size for draining impounded water. For example, under idea field conditions with proper drainage gradients for impounded water for the greatest recorded rate of 18.7 gal/LF/hr, a small a 3-hp, 150 GPM gasoline-powered water pump typically used in flood fights should be able to drain more than 400 lin ft of wall.

Wave-induced Dynamic Loads

The RDFW was subjected to incident wave heights, which were estimated to range from 0.42 ft to 1.52 ft. Waves were run at two water depths, d = 2.0 ft and d = 2.67 ft. The wall was exposed to a total of 72,000 waves within the 40-hour duration of wave action. While precise measurement of the wave height at the wall was near impossible due to the extremely complex and energetic wave field, visual observation of the waves indicate the estimates of wave heights were well within an order of magnitude. From observations and photographic documentation (Figures 17-22), there is no doubt, that in much of the wave runs the wave action was severe.

Overtopping

The onset of overtopping of the RDFW wall occurred at Hour 24. All of the wave runs after this time exhibited various degrees of wave overtopping, with the most severe cases at the larger wave heights. From the onset of overtopping, average rates ranged from 22.1 to 218.5 gallons per linear foot of wall per hour. The maximum overtopping rate recorded was 285 gallons per linear foot of wall per hour. This was for a wave height of 1.49 ft and a water depth of 2.0 ft. As with underseepage rates, the overtopping rates given in Table 5, give the field engineer some guidance of what can be expected if a 4 ft RDFW wall experiences severe wave action.

Wall Deterioration and Fill Loss

The wall did sustain damage in three locations along the wall. The use of conical abutments, and associated acute wave conditions at the abutment seemed to accelerate damage at these locations. Repairs were made to the three sections, but not until the damage was deemed excessive. Even with “excessive” damage, the total amount of fill required to make the repair was just over 2 cu yd of sand or approximately eight percent of the total fill in the entire wall. In the field, under flood conditions, proper maintenance could occur at the onset of damage and most likely could have prevented the degree of deterioration that occurred.

The face of the RDFW wall did show a significant amount of plastic breakage (Figure 41). Apparently the PETG plastic used in the in the grids is susceptible to breakage under repeated impact loading. Reinforcing the face with nylon tie wraps reduced the breakage. Refinements need to be made in the formulation of the plastic to reduce brittleness.

Figure 41. PETG plastic damage due to wave impacts

Throughout the test series measurements were made of lateral wall deflection. Measurements were only taken along the top edge of the wall. Measurements were considered along the base but proved too difficult with the row of sandbags placed along the dry-side toe of the wall. Observations of the dry-side of the base did not indicate any movement up until the last wave run at Hour 40, where several waves produced big impacts against the damaged wall. It did appear the wall base might have moved approximately 1 in. From Table 6 and Figure 40, it can be seen the lateral wall deflection was almost none existent up until Hour 37, when the top of the wall began to deflect landward. The deflection was directly a result of sand loss to upper grids at mid-span, allowing the empty grids to deform and deflect.

Wave Power

A new approach to evaluating structural performance was developed in conjunction with this test series. Cross comparison between different types of expedient structures tested in laboratory facilities pose problems when trying to evaluate based on wave height and duration alone. Cumulative effects become hard to quantify. From the power calculations (Tables 7-17) the two power calculation methods, DOM and WED, yielded slightly different results. Therefore the final results shown in Table 19 were the average value found by the two methods. In the future, other structures tested in the wave basins can be compared to RDFW results, as long as incident wave heights across the basin can be measured.

Table 20 indicates how durable an RDFW wall would be under idealized wave height and water depth conditions. It must be realized that this table is only for preliminary planning and design use. The experimental data, which supports these findings, is limited. The unsupported length of the wall was approximately 50 ft. It was founded on concrete base, and had large truncated-conical abutments. The effects of current scour on the foundation were not a part of these test series. Many factors may influence how durable an actual field installation of RDFW will be, and only documented monitoring of actual RDFW installations will increase confidence in its use.

5 Summary and Conclusions

This report documents the testing and evaluation of a new type of expedient flood fight structure, RDFW. The tests were funded under a Cooperative Research and Development Agreement between A. M. Arellanes and Sons and Associates and the U.S. Army Corps of Engineers. Tests were conducted 1 month beginning in April 2000.

The tests were conducted at full-scale in a large L-shaped basin. Five men and one piece of earth moving equipment constructed the RDFW wall in the basin. The completed RDFW wall was approximately 4-ft tall by 4-ft wide and 50-ft long. The structure was founded on the basin's concrete floor, and used rows of sandbags along both the riderside and dry side bases to prevent leakage. Conical sandbag abutments terminated the wall at lateral transitions. The wall was sealed to the vertical basin walls with a dry-pack mix of cement and sand to prevent leakage. The wall was constructed in the basin, in 1 hr 30 min.

One significant observation of the wall construction process pertains to consistency and repeatability. Sandbag levees are the most used of any type of expedient structure. However, it takes training and diligence to build a sandbag levee with consistency. Rotation in crews, fatigue, and adverse conditions often lead to substandard levee construction. From observations of the construction process in building the RDFW wall, it appears the wall is relatively easy to construct. It seemed the untrained laborers quickly understood the construction process. And the rate of construction increased. As long as the walls are properly interlocked, filled and compacted if is anticipated there will be very little deviation in the integrity between construction of various RDFW walls.

The premise for the testing of RDFW was developed into a new testing protocol for evaluating expedient structures in the L-shaped basin. Tests consisted of subjecting the wall to both hydrostatic and wave-induced dynamic loads. The wall was exposed to hydrostatic loads for a total duration of 128 hours. The structure was exposed to wave action for 40 hr. Under these loads and durations, the wall was evaluated for structural and hydraulic performance. Measurements were made of wave height, wall deflection, water seepage, wave overtopping, and fill loss. All measurements seemed within reason based on visual observation.

The structure was exposed to incrementally increasingly wave attack. Damage to the structure occurred at three locations along the wall. When damage at one location was deemed excessive, the wall was repaired, and wave runs continued. At the two damage locations adjacent to abutments, it appeared the abutments concentrated wave loads and accelerated damage progression. For a typical structure used in the field, the dry-side may have abutments to support the wall but it is doubtful abutments would be needed on the riverside, except at a closure. So abutment-related wave load concentrations will not occur in the field. However, in a flood there is the possibility that debris may become trapped against the wall and similar concentrated loads may occur.

Damage at the wall mid-span did not occur until Hour 37. By Hour 40 the wall had sustained significant damage and the tests terminated. In observations of the deterioration of the RDFW wall, it appears the wall has more of a ductile failure mode rather than a brittle failure mode. That is, the failure comes on slowly, rather than quickly and catastrophically. In an actual flood fight under similar conditions, it appears there could be adequate time to make repairs before significant damage occurs, if manpower were available and trained.

The wave field generated in the basin was extremely complex and energetic. New methods were developed to quantify performance in terms of power exposure, as opposed to wave height and wave attack duration alone. Test results were used to generate estimated durability of RDFW for idealized wave height and water depth combinations. These estimates should be used with discretion, for preliminary planning and maintenance schedules until they are validated with actual field experience.

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