CHAPTER 106

Dolos Design Using Reliability Methods

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Abstract

Historically, concrete armor unit design has not included conventional structural design methods. The primary reason is the lack of knowledge of the loads and the resulting structural response. Also, complex and random boundary conditions and wave loading made the engineering problem difficult. With recent advances in concrete armor unit stress prediction and measurement methods, we can begin to utilize conventional structural design methods in concrete armor design. This paper adapts conventional structural reliability design methods in the Load and Resistance Factor Design (LRFD) format to allow a unified approach to both reinforced and unreinforced dolos design. Stress prediction methods are validated for several well known structures. Then LRFD methods are described and applied generically for both unreinforced and reinforced dolos design. The reliability methods described herein are adaptable for general concrete armor unit design.

1 Reliability Methods

Conventional land-based concrete structures are typically designed using Load and Resistance Factor Design (LRFD) methodology. It has been argued that conventional structural design methods are not applicable to the dolos structural design problem because the random and highly variable wave loading and boundary conditions cannot yet be specified. With recent advances in concrete armor unit stress prediction and measurement methods, the reliability methods appear to be readily adaptable to concrete armor design. The primary advantages of the LRFD format are:

- follows conventional structural engineering practice
- provides measures of the uncertainty for both the loads and the strength
- permits robust, unified unreinforced and reinforced concrete design

The LRFD methods are particularly appropriate for breakwater armor design because the large uncertainties associated with breakwater armor hydraulic and structural response can be quantified and presented in a familiar format. Also, historically, breakwater armor designers often put structural steel reinforcement in armor without knowledge of the loads or internal response and without reasonable analysis methods. Typical concrete armor designs were therefore not economical when designed with reinforcement. But even with current dolos stress prediction methods, efficient reinforcement design

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could not previously be done because of the need to specify component forces. But conventional structural engineering techniques provide the means to distribute this armor layer design stress to the component forces. The existing armor layer stress prediction methods along with the component force relations enclosed within the LRFD framework, therefore, provide a complete procedure for concrete armor design.

The LRFD methods outlined herein are based on reliability methods as described in Ellingwood, et al (1980). The balanced LRFD design equation can be expressed as the equilibration of a factored load with a factored strength or

$$\gamma Q_n = \phi R_n \tag{1}$$

where γ and ϕ are the load and strength factors and Q_n and R_n are the nominal load and resistance, respectively. The strength coefficient and load factor take into account the appropriate uncertainties.

The loads on breakwater armor units are difficult to determine due to the complexity and randomness of both hydrodynamics and boundary conditions. Therefore, the methods currently used to describe the loads on concrete armor units are indirect, specifying the maximum armor layer stress as a function of environmental parameters such as wave height, structure slope, and armor weight. Previous publications show methods for determining the design stress level using probability of exceedance curves and expected levels of exceedance (Howell and Melby 1991). To couple these previously described methods and the LRFD method, the load factor is chosen so as to preserve the design probability of exceedance as follows.

$$E = \Phi(-\beta) \tag{2}$$

where Φ is the unit normal cumulative distribution and β is the reliability index. The load factor can be determined as

$$\gamma = \phi \frac{R_n Q_m}{R_m Q_n} \exp\left[-\Phi^{-1}(E)\sqrt{V_R^2 + V_Q^2}\right] \tag{3}$$

where Q_m and R_m are the mean load and resistance and V_Q and V_R are the load and resistance coefficients of variation, respectively. The limit states are given by ACI (1989) and Ellingwood (1980) as $\phi = 0.85$, $R_m/R_n = 1$, and $V_R = 0.2$ for torsion and $\phi = 0.90$, $R_m/R_n = 1.05$, and $V_R = 0.11$ for flexure (ACI 1989).

In this report, previously published dolos stress prediction methods are used to preliminarily define the loads. The stress is distributed to the component forces which are then used as the loads in the LRFD formulation. The result is a comprehensive design methodology for dolosse that includes strength enhancement specification and conforms with present structural engineering design practice. The methods allow the designer to compare the economies of different strength enhancement options including high strength concrete, shape modification, and steel bar reinforcement in a unified format to achieve the most efficient dolos design. The basic methods outlined herein are general and can be adapted to any armor unit shape.

2 Design Stress Prediction

Concrete armor unit design has progressed a great deal during the last few years. This rapid progress is, in large part, due to the prototype dolos structural response data set and associated research accomplished under the Crescent City Prototype Dolos Study (CCPDS). Figure 1 shows 38-tonne dolosse being placed on the breakwater at Crescent City. In the prototype study, dolos structural response data and numerical models were

Figure 1: 38-tonne Dolosse at Crescent City, CA

used to link design parameters such as dolos size, shape, and material specifications to the measured stress statistical moments. Using these methods, the Crescent City stress distributions could be extended to other structure geometries (Howell and Melby 1991). The dolos small-scale-model load cell was verified as a tool to measure pulsating stresses in the physical model (Markle 1990). The CCPDS has been widely reported and other primary publications include Howell (1988), Melby and Howell (1989), Kendall and Melby (1990), and Rosati and Howell (1990).

The dolos stress prediction methodology is based on stochastic methods because of the random nature of both the loading and the boundary conditions. Separate distributions for static and pulsating stresses have been generated and are combined using the methods that follow. The result is a single maximum design stress for the armor layer for a specified probability of occurrence.

In general, the static stress will be much larger than the pulsating stress (Melby and Howell 1989). The static nondimensional stress log-normal distribution is given in Equation 4 with mean and standard deviation given in Equation 5 and 6 and shift parameter given in Equation 7 (Howell and Melby 1991). The original distribution based on measured stress statistics has been extended for the general design case by modifying the statistics for the dolos size, density, waist ratio, and stacking depth. The waist ratio is the ratio of the depth of the shank (center section) to the length of a fluke (end sections).

$$p(\sigma_s') = \frac{1}{\sigma_s'\beta\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{\ln\sigma_s' - \alpha}{\beta}\right)^2\right] \tag{4}$$

$$\alpha = \ln[k_r(\alpha_{cc} + a)] - \beta/2 \tag{5}$$

$$\beta = \sqrt{\ln\left(\frac{\beta_{cc}}{\alpha_{cc} + a}\right)^2 + 1} \tag{6}$$

$$a = S_L\left[\frac{1}{\gamma}(N_L - 1) - \frac{1}{\gamma_{cc}}\right] \tag{7}$$

In the above equations, $\alpha_{cc} = 26$ and $\beta_{cc} = 12$ are the Crescent City prototype nondimensional mean and standard deviation, being nondimensionalized by the product of fluke length and weight density, $\gamma_{cc} = 2456 kg/m^3$ is the Crescent City dolos weight density, and N_L is the number of armor layers. The waist ratio coefficient, given by

$$k_r = 5.14 - 28.74r + 66.07r^2 - 52.08r^3 \tag{8}$$

was determined using a fully deterministic FEM analysis with several representative boundary and loading conditions.

The maximum pulsating or wave-induced stress is a function of the design wave height, H_n and a wave stress constant, k_{ps}. The Rayleigh distribution of the form

$$p(\sigma_p) = \frac{\pi\sigma_p}{\bar{\sigma}^2_{pmax}} \exp\left[-\frac{\pi}{4}(\frac{\sigma_p}{\bar{\sigma}_{pmax}})^2\right] \tag{9}$$

best describes the dolos pulsating response. The mean of the maximum pulsating stress, which is linearly related to the average of the highest one-tenth of the waves can be expressed by the empirical relationship

$$\bar{\sigma}_{pmax} = k_{ps}H_{1/10} \tag{10}$$

where

$$k_{ps} = 0.036 MPa \tag{11}$$

per meter of wave height and $H_{1/10}$ is computed using the zero-downcrossing method of analysis, i.e., the difference between the maximum and the preceding minimum between two successive zero downcrossings in a time series. Note that during the prototype data acquisition period, the maximum pulsating stress was approximately $\sigma_1 = 1$ MPa, which occurred during a design event.

The modified static and pulsating distributions are convolved, assuming they are independent, to get a combined stress distribution which is integrated to get a stress exceedance distribution. This distribution is used along with a design probability of exceedance, E, to determine a design stress. This stress is interpreted as that which will be exceeded in E percent of the armor units. Note that this is a hydraulically stable design stress because the impact stress is not yet included in the calculations. We do not include impact stresses because of the unknown scale effects in the instrumented impact tests and because of the uncertainties associated with uninstrumented drop test results. All of the calculations for stress prediction are performed within a PC computer-based program called CAUDAID.

Extensive physical model tests utilizing the small-scale load cell instrumentation were used to validate the static log-normal distribution (Melby 1992). But, as noted by Melby, the mean for the prototype data is significantly greater than that of the scaled physical model data. There are several reasons for this difference including the flat slope of the prototype breakwater, surface friction scale effects, and overly stiff instrumented cross sections in the small-scale units. Utilization of the modified prototype static distribution, rather than the load cell measured results, is likely to be conservative. But as shown later in this paper, these modified prototype distributions appear to predict the design stresses well.

The data reported in Burcharth et al. (1991) and Anglin et al. (1990) are based, for the most part, on scale model results utilizing the load cell structural instrumentation scheme. The results published by these two authors were accomplished through careful laboratory examinations and their results appear to be very reliable. As noted above, this load cell instrumentation scheme has also been employed by the present authors but has only been validated for pulsating response. Scale effects in both static and impact load cell measured responses may introduce unconservativeness in the dolos load prediction process. Thus the load cell measured results are not included in this paper.

2.1 Application of Maximum Stress Prediction Methods

The stress predictions described in the previous section have been applied to several dolos armored breakwaters (Table 1). The LRFD methods were not used in this section so that actual computed stress levels could be shown clearly. Each example breakwater, with the exception of Cleveland and Sines was physically surveyed by the authors within the last 6 months. Cleveland and Sines breakwaters have been thoroughly studied by others and therefore provide excellent examples. The 1974 rehabilitation of the Crescent City breakwater was not used as an example because many of the dolosse were broken due to storms that occurred during construction. In Table 1, *Age* is the difference in years between original construction and the last survey; H, the wave height in meters; W, the weight in tonnes; N, the number of dolosse placed; S, the specific gravity; r, the waist ratio; $\cot(\alpha)$, the breakwater slope; E, the probability of exceedance used in the calculation of the design stress, which is the surveyed number of broken dolosse as a percentage of the total number of dolosse placed; $\sigma 1$, the maximum principal stress as computed in CAUDAID in MPa; and f_t, the concrete tensile strength in MPa.

Each structure has its own design peculiarities which effect the design stress as follows.

- Crescent City: Flat structure slope makes structure extremely stable and limits breakage; conservative stress estimates will always be high.
- Humboldt: Conventional reinforcement adds approximately 20% to resistive capacity as reflected in high f_t.
- Nawiliwili: No peculiarities; relatively simple application of stress prediction methods.
- Waianae: Wide fronting reef limiting wave energy; $E = 1$ does not include 170 construction related breaks.

Table 1: Application of Design Methods

SITE	Age	H	W	N	S	r	$cot(\alpha)$	E	σ_1	f_t
Crescent City	7	11	38	680	2.46	0.32	5	1	8.1	7.0
Humboldt	10	12	38	4772	2.46	0.32	2	1	8.5	8.4
Nawiliwili	15	7	10	485	2.30	0.32	2	8	3.6	3.8
Waianae	13	4	1.8	6633	2.30	0.32	2	1	3.5	3.8
Honolulu, H	16	8	5.5	4516	2.30	0.32	2	4	3.5	3.0
Honolulu, T	16	8	3.6	13790	2.30	0.32	1.5	4	3.0	3.0
Cleveland	5	4	1.8	29500	2.30	0.32	2.0	4	2.4	2.5
Sines	1	14	42	19000	2.55	0.35	1.5	5	6.2	5.0

- Honolulu: Wide fronting reef; estimated concrete strength is low by U.S. construction standards.
- Cleveland: Estimated concrete strength is very low.
- Sines: Damage primarily due to single storm, long slope, deep water.

Given no peculiarities, the stress prediction methods will reasonably predict long-term cumulative damage of a relatively stable structure (Nawiliwili) but will overpredict short term damage (Sines) and damage on a extraordinarily stable structure (Crescent City). Although the stress prediction methods do not include impact response, the conservativeness in the predicted stress appears to allow enough safety to account for all loading over the structure life. Because the design goal is to achieve an armor layer design that does not require periodic rehabilitation and has an extremely low probability of catastrophic failure during its design life, the stress prediction program is appropriate for conservative design load determination and it was used as input to the following LRFD methods.

3 LRFD - Optimizing Design Methods

3.1 Strength Enhancement Options

As shown in Table 1, the design stress for the large dolosse at Crescent City exceeds the tensile strength. Also, because these dolosse were built without significant reinforcement and because the average stress level is increasing over time (Kendall and Melby 1990), the dolos breakage is expected to continue and a rehabilitation will likely be required before the design life is reached. For the Crescent City breakwater, a strength enhancement of the design dolosse is required. Melby (1992) showed that, as a general rule, for dolosse exposed to design wave heights above 7m, the stress exceeds the commonly used concrete strength of 3.6 MPa and strength enhancement is required. The designer has several strengthening options including fiber reinforcement, increased concrete strength, modified shape, and steel reinforcement.

Metal fiber reinforcement was used in Crescent City and Humboldt, California dolosse. The Crescent City dolos tensile rupture strength was very high (Kendall and Melby 1990) using approximately 1 % fibers; but this high strength is likely attributable to the concrete mix characteristics and not the fiber. With approximately 50 kg of steel added per cubic meter of concrete, a mere seven percent tensile strength increase was reported from tests conducted during trial mix designs. One of the major problems encountered during prototype casting was the tendency for the steel fibers to congregate or "ball up" during concrete mixing. Recent evidence indicates that fiber reinforcement is not likely to increase the strength enough to make it economical.

Increasing the concrete tensile strength can be practical in the U.S. because high strength concrete is now commonly used. Also, recent tests of high strength silica fume concrete indicate that the ratio of tensile to compressive strength is maintained for compressive strengths up to about 107 MPa (Saucier 1984). But the compressive strength can only be increased economically to about 70 MPa at this time. The corresponding tensile strength of this concrete mix is approximately 7 MPa.

For armor unit design, the primary design input site parameters include directional wave energy, water depth, breakwater slope, and position on breakwater. The designer is free to optimize the design by varying the dolos weight, shape, density, packing density, and material strength. In practice though, the dolos shape and concrete properties are fixed, with a waist ratio of $r = 0.32$, a packing density of $k_{\Delta} = 0.94$, a specific weight of approximately 2.3, and a compressive strength of $f'_c = 36$ MPa. This leaves only the dolos weight as a variable. But in order to achieve an optimized design, none of these parameters should be fixed. Also, for some designs it may be more economical to reinforce slender dolosse than use unreinforced stout dolosse.

Utilizing the previous stress prediction methods, it is a simple process to minimize the stress level by varying the dolos shape, packing density, and material properties. For a given waist ratio, incorporating reinforcement in the optimizing process is straight forward, as will be shown. But our ability to fully optimize the dolos design by maximizing dolos strength and hydraulic stability is limited because knowledge of dolos stability and stress versus waist ratio is still needed. Also, no research has been done to determine the response of very slender dolosse ($r \leq 0.31$). Yet with efficient reinforcement schemes, slender reinforced dolosse may be a viable option because the amount of concrete is reduced and the wave energy dissipation of the armor layer is increased. Integrating reinforcement, shape modification, and stability analysis into a general optimizing design procedure using existing knowledge is therefore a great challenge.

In order to explore the advantages of shape modification on stability, previous research results were used. Zwamborn et al. (1988) provided stability results for dolos waist ratios of 0.33, 0.36, 0.38, and 0.40. In the following analyses, these stability data were highly simplified by averaging multiple curves for various surf similarity parameters and extrapolating to slender waist ratios. Figure 2 shows the resulting Hudson (1958) stability coefficient versus waist ratio curve. This curve is simply used in the general optimization process herein and is not intended for design purposes.

3.2 LRFD Formulation

The design stress is a result of combined bending and torsion loads. To resist combined loading, the strength due to torsion is generally different than the strength due to flexure.

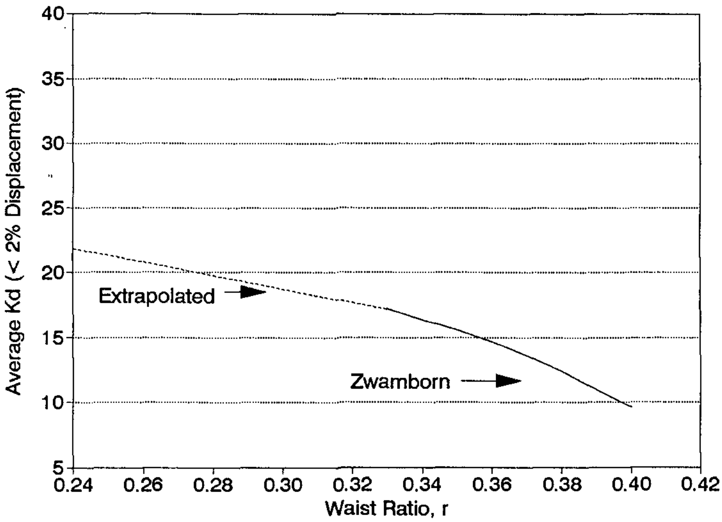

Figure 2: Hydrodynamic Stability from Zwamborn et al.(1988)

Also, reinforcement schemes must be designed for torsion and flexure separately in order to maximize efficiency. Therefore, a strength failure criterion in the form of a moment-torque circular interaction curve has been adopted for both unreinforced and reinforced dolos design, i.e.,

$$\left(\frac{T_c}{T_{cr}}\right)^2 + \left(\frac{M_c}{M_{cr}}\right)^2 = 1 \tag{12}$$

$M_{cr} = S_M f_{ct}$ is the flexural cracking moment in the absence of torsion and $T_{cr} = S_T f_{ct}$ is the torsional cracking moment in the absence of flexure. Here S_M and S_T are the flexural and torsional section moduli and f_{ct} is the tensile splitting strength. M_c and T_c are the respective moments at failure in the presence of combined bending.

The principal stress as computed by CAUDAID can be used as the loading criterion if the statistical variability of the torsional and flexural contributions to this principal stress are known. For this analysis, we assumed $M = S_M k_M \sigma_1$ and $T = S_T k_T \sigma_1$, where the torsional and flexural section moduli are given by $S_T = 0.2105(rC)^3$ and $S_M = 0.10526(rC)^3$, respectively. Here C is the fluke length and r the waist ratio. The stress contribution factors are taken from Crescent City prototype data as $k_T = 0.6$ and $k_M = 0.6$. These values require further refinement and will be addressed in the future.

Using Equation 12, the torsional and flexural concrete strength in combined loading can be expressed as

$$T_c = \frac{T_{cr}}{\sqrt{1 + 4\left(\frac{M_c}{T_c}\right)^2}} \tag{13}$$

$$M_c = \frac{M_{cr}}{\sqrt{1 + 0.25\left(\frac{T_c}{M_c}\right)^2}} \tag{14}$$

With k_T and k_M equal to 0.6, $Mc/Tc = 0.5$, and the combined loading torsional and flexural strengths are 70 % of the respective pure torsional and flexural strengths. Also, design fluke stress levels are conservatively estimated to be equivalent to those in the shank, although they are assumed to be created from pure flexural loading. This is perhaps overly conservative but is done here in order to illustrate the general analysis procedure.

The load factor in Equation 1 has been determined using Equation 3 and the values given in Section 1 for the limit states. The load factor was found to range from 1.0 to 1.2 over a range of typical values of the exceedance probability. A value of $\gamma = 1.0$ was used herein because of the conservativeness inherent in the calculation of the design stress. With the strength and loading defined, the LRFD balanced equation, $\gamma Q = \phi R$, becomes for torque

$$\gamma S_T k_T \sigma_1 = \phi 0.7 T_{cr} \tag{15}$$
$$1.0(0.6)\sigma_1 = 0.85(0.7) f_{ct} \tag{16}$$
$$\sigma_1 \approx f_{ct} \tag{17}$$

and for moment

$$\gamma S_M k_M \sigma_1 = \phi 0.7 M_{cr} \tag{18}$$
$$1.0(0.6)\sigma_1 = 0.9(0.7) f_{ct} \tag{19}$$
$$\sigma_1 \approx f_{ct} \tag{20}$$

The approximation is done in order to show the basic methodology as simply as possible in this brief format.

3.3 Unreinforced LRFD

The preceding stress prediction methods, shape modification and high-strength concrete, waist ratio stability and combined loading strength reduction have all been incorporated into the LRFD design formulation above to determine the optimal unreinforced dolos weight for a given design wave height. For the unreinforced dolos analysis, stable dolos weights were computed for several waist ratios using the Hudson equation with a given wave height, structure slope of $\cot \alpha = 2$, specific gravity of $S = 2.34$, packing density of $K_\Delta = 0.94$, and K_D of half the value shown in Figure 2 (i.e., no-rocking). These stability coefficients were chosen to be conservative. Using the Hudson stable weight, the design maximum principal stress, which is the load side of the LRFD Equations 17 and 20, was computed using CAUDAID with $E = 5\%$. On the resistance side of the LRFD formulation, concrete splitting tensile strengths were estimated using compressive strengths of 35 MPa and 70 MPa and the ACI splitting strength recommendation of $f_{ct} = 6\sqrt{f'_c}$. Note that the strengths and loading moments were reduced for combined loading in Equations 13, 14, 15, and 18 but the final LRFD equations reduced to equating the design stress to the splitting tensile strength (Equations 17 and 20).

Figure 3 shows the hydraulically and structurally stable weight for unreinforced dolosse as a function of wave height for the three waist ratios and two concrete strengths. Note that the maximum wave height that can be successfully resisted for unreinforced dolosse over the design life of the structure is approximately 7.5m using high strength concrete. For larger wave heights it is necessary to reinforce with structural steel. The effects of waist ratio can be clearly seen in this figure. The more slender the dolos, the less the required weight. But the figure shows that, for a given wave height, high strength concrete might be required for a more slender design dolos while not for the stouter dolos. It is clear that dolos design optimization could save a considerable amount of money through minimizing the concrete costs.

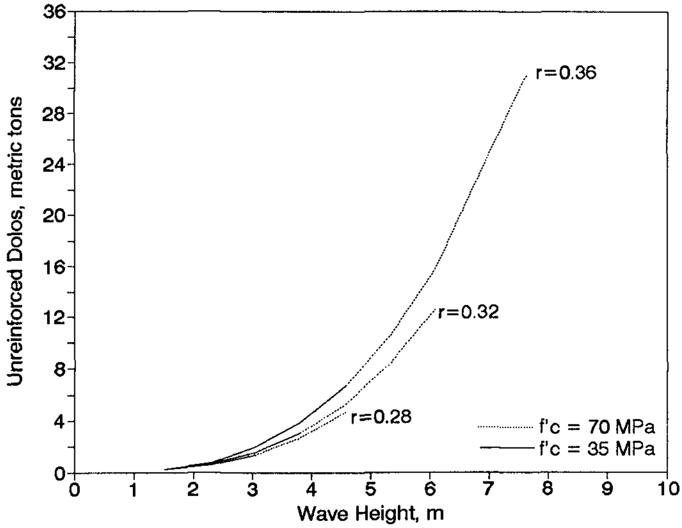

Figure 3: Unreinforced Structurally and Hydraulically Stable Dolos

3.4 Steel Reinforced LRFD

Two options exist for the steel reinforcement; deformed bars and prestressed tendons. In the U.S., the Corps of Engineers has placed conventionally reinforced dolosse on the Humboldt, Ca. Jetties; Manasquan Inlet, N.J. Jetties; and on several Hawaiian breakwaters. In most cases, no information was available about the magnitude of flexural and torsional loading. This resulted in inadequate hoop steel to resist torsional moments and improperly sized longitudinal reinforcing. With appropriate information about the nature of the loading, proper amounts of steel can be used to greatly strengthen dolosse.

Conventional reinforcement corrosion can be a significant problem. As a unit is loaded, tensile stresses within the concrete are transferred to the steel dowels along their development length only after the concrete cracks. Depending on the nature and severity of the loading condition, these cracks often extend through the concrete cover layer. Also, if the steel bars deform outward during concrete pouring then the amount of bar cover can be reduced. In the marine environment, cracks provide a conduit for seawater intrusion and subsequent chloride ion attack. This results in corrosion of the steel, and ultimately the eventual failure of the unit.

For the moderate to severe wave climate, prestressed concrete offers a solution for strengthening dolosse that would otherwise crack. Prestressed concrete also has an enhanced ability to resist impact loads and fatigue.

3.4.1 Conventional Steel Bar Reinforcement Design

With conventional steel bar design, the torsional steel is specified first, and then the flexural steel. As per ACI (1989), resistance to torsional loading is computed as $T_n = T_c + T_s$, where T_n is the nominal torsional strength, and T_c and T_s are the nominal concrete and steel torsional strengths, respectively. Thus, the torsional resistance provided by the hoop steel is computed by

$$T_s = \frac{\gamma k_T T_u - \phi T_c}{\phi} \tag{21}$$

and the area of steel by $A_s = T_s / R_h f_y$ where R_h is the distance from the center of section and f_y is the steel yield strength. To offset pure torsional forces, an equivalent amount of longitudinal steel is placed in the spacing between hoops as is contained in a single hoop.

Contrary to the preceding torsional reinforcement design, resistance offered by the concrete tensile strength is not considered in flexure. Nominal strength is reached when a crushing strain at the extreme fiber occurs as the tension steel yields. Strains in the steel and concrete are assumed directly proportional to the distance from the neutral axis. Unlike conventional structures where ductile failure indicates imminent collapse, armor unit design benefits from a brittle failure where a high steel-to-concrete ratio reduces crack width and formation under service loading. Although the stress-strain distribution across the section is nonlinear, a rectangular distribution is used to facilitate design. A concrete stress intensity of $0.85 f_c'$ is assumed to be uniformly distributed across an equivalent compressive zone. With the compressive force defined, an equivalent tension force comprised of the sum of forces generated by symmetrically placed steel is assumed. These forces become the components of a moment couple dependent on an unknown neutral axis location, requiring an iterative solution. The procedure to specify flexural steel is straight forward and the reader is referred to ACI (1989).

Using the aforementioned conventional reinforcement analysis methods, it was found that the reinforcement scheme of 12 equally spaced #6 bars in 16-ton dolosse at Manasquan, NJ provided approximately a 20% increase in flexural capacity and little increase in torsional capacity.

Figure 4 shows the results of a more general analysis with the weight of reinforcing steel required within a stable dolosse versus design wave height. f_s is the steel yield strength. Note that while the packing density is held constant, the porosity of the armor layer varies with waist ratio. Therefore to permit comparison of differing strengthening schemes, the amount of steel is given per $100 m^2$ of breakwater surface. Again, the effects of waist ratio can be clearly seen. The more slender the dolos, the less the number required to armor the breakwater. This significantly offsets the increase in steel required for slender units. Also note that, although high strength concrete was analyzed, the resulting curves are not shown because they do not differ significantly from those of normal strength concrete. This is because most of the resistance in a conventionally reinforced dolos comes from the steel. It must be noted that even though a substantial amount of steel is specified, conventionally reinforced dolosse are still susceptible to cracking and, hence, a possible reduction in service life.

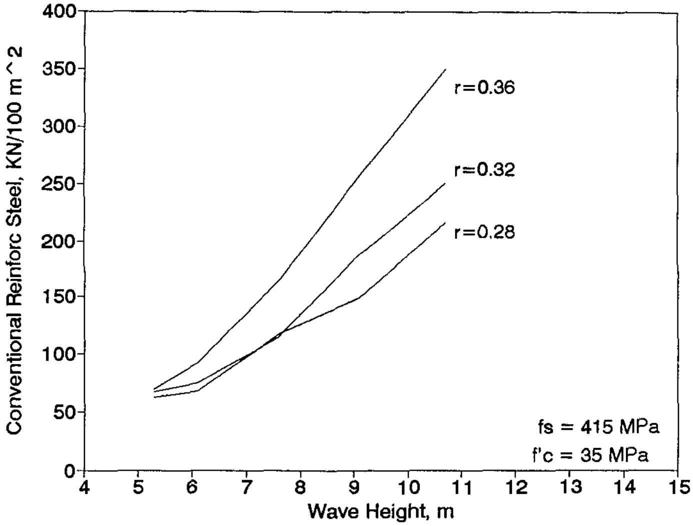

Figure 4: Conventional Reinforcing Steel Required

3.4.2 Prestressed Steel Reinforcement Design

Prestressing is a means of applying a precompression load to a structural member regardless of the dead or live loads acting on the structure. For the plane stress assumption, as precompression is added, the failure plane becomes more vertical and the section less susceptible to pure shear related inclined cracking.

Prestressed concrete design methodology differs from that of conventional reinforcement because both the torsional and flexural tensile strength of the concrete must be considered. Prestressing acts to directly apply an axial compressive force. The magnitude of the prestressing force is governed by the loading mode. The principal stress reduction factor as a function of a given precompression stress is

$$\xi = 0.5\left(k_M - \lambda + \sqrt{(k_M - \lambda)^2 + 4k_T^2}\right) \tag{22}$$

where λ is the ratio of applied precompressive stress to design principal stress, as computed by CAUDAID, and k_M and k_T are 0.6. Substituting the moment-torque interaction relation into Equation 1 yields

$$\gamma\xi k_T\sigma_1 = 0.5\phi\sqrt{\frac{f'_c}{1 + 4\left(\frac{k_M S_M}{k_T S_T}\right)^2}} \tag{23}$$

for torsion with a similar relation for flexure. Again, ACI (1989) standard design practice methods were used to determine the amount of steel required.

Figure 5 shows the amount of steel required for the given design wave for the same $100m^2$ of breakwater surface as used in the conventional reinforcement design. f_{pu} is the steel strength. Again, the slender reinforced dolos appears to be more efficient than the stouter one. Comparing Figures 4 and 5 shows that the prestressed dolos made with normal strength concrete requires slightly less steel than the conventionally reinforced dolos. But, using high-strength concrete, it is clear that the combination of prestressing and high-strength concrete is a much more efficient than conventional reinforcement.

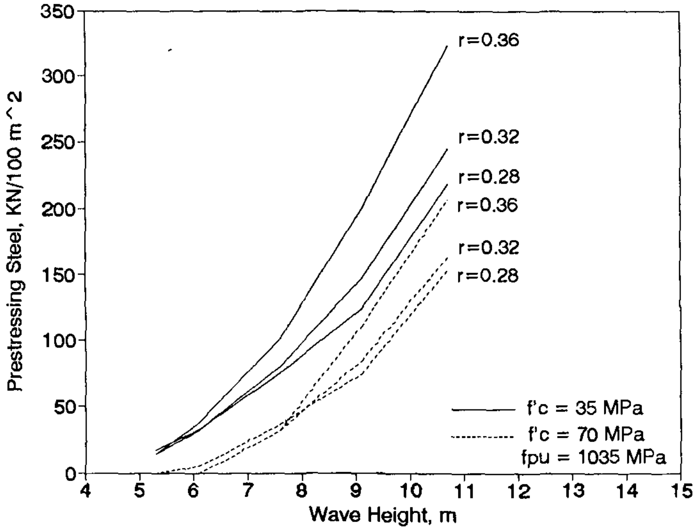

Figure 5: Prestressed Steel Required

4 CONCLUSIONS

In this paper conventional structural reliability methods have been adapted to the design of dolos. The loading was specified using a PC-based stochastic stress prediction algorithm incorporating Crescent City prototype dolos stress distributions. These site specific static and pulsating stress distributions are modified in the program according the user specified dolos weight, armor unit density, number of layers, waist ratio, and wave height. The static and pulsating stress distributions are combined to get a design distribution which is used along with a design probability of exceedance to get a maximum probable stress level for the design armor layer. The LRFD methods are formulated so as to preserve this exceedance probability or the expected amount of breakage on the breakwater over the design life. It is shown that these methods predict the stress levels in dolos armored breakwaters well. The LRFD methods are further employed to allow the design of steel bar reinforcement.

The new design methodology is used to compare various strength enhancement options for dolosse including shape modification, high strength concrete, conventional steel rebar reinforcement, and high strength prestressed reinforcement. It is shown that for wave heights below about 7.5m stouter dolosse and high strength concrete can be used effectively. Steel bar reinforcement may be required in dolosse for wave heights above 7.5m. It is shown that in some cases it may be economical to use slender dolosse with reinforcement than to use stouter dolosse without reinforcement. Finally, prestressing was found to be slightly more efficient than conventional steel bar reinforcement when used with normal strength concrete but superior when used with high strength concrete.

5 ACKNOWLEDGEMENT

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