CECW-EW

DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

EM 1110-2-2104
Change 1

Engineer Manual
No. 1110-2-2104

20 Aug 03

Engineering and Design
STRENGTH DESIGN FOR
REINFORCED-CONCRETE HYDRAULIC STRUCTURES

1. This Change 1 to EM 1110-2-2104, 30 June 1992, revised Table of Content and Chapter 3. Also commentary for Chapter 3 is added

2. File this changes in front of the publication for reference purposes

FOR THE COMMANDER:

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CHAPTER 3

STRENGTH AND SERVICEABILITY

3-1. General

a. Nonhydraulic vs. hydraulic structures. All reinforced-concrete hydraulic structures must satisfy both strength and serviceability requirements. In the strength design method this is accomplished by multiplying the service loads by appropriate load factors and for hydraulic structures multiplying by an additional hydraulic factor, H_f. The hydraulic factor is applied to the overall load factor equations. This increased loading is then used for obtaining the required nominal strength for the hydraulic structures. The hydraulic factor is used in lieu of performing an additional serviceability analysis.

b. Single and modified ACI 318 load factor approaches. Two methods are available for determining the factored moments, shears, and thrusts for designing hydraulic structures using the strength design method. They are the single load factor method and a method based on slight modification of the ACI 318 Building Code requirements. Both methods are described herein.

c. Stability requirement. In addition to strength and serviceability requirements, many hydraulic structures must also satisfy stability requirements under various loading and foundation conditions.

d. Nonhydraulic structures. Reinforced concrete structures and structural members that are not classified as hydraulic shall be designed according with this guidance, except the hydraulic factor shall not be used.

3-2. Stability Analysis

a. Unfactored loads. The stability analysis of hydraulic structures must be performed using unfactored loads in accordance with EM 2101 Stability Analysis of Hydraulic Structures. The unfactored loads and the resulting reactions are then used to determine the unfactored moments, shears and thrusts at critical sections of the structure. The unfactored moments, shears and thrusts are then multiplied by the appropriate load factors, and hydraulic factor when appropriate, to determine the required nominal strengths to be used in establishing the section properties.

b. Soil structure interaction load factors. The single load factor method must be used when the loads on the structural component being analyzed include reactions from a soil-structure interaction (SSI) stability analysis, such as footings for a wall. For simplicity and ease of application, the single load factor method should generally be used for all elements of such structures. The load factor method based on the ACI 318 Building Code may be used for some non-SSI related elements of the structure, but must be used with caution to assure that the load combinations do not produce unconservative results.

3-3. **Required Strength**

a. General. Reinforced concrete hydraulic structures and hydraulic structural members shall be designed to have a required strength, U_h, to resist dead and live loads in accordance with the following provisions. The hydraulic factor is to be applied in the determination of the required nominal strength for all combinations of axial load, moments and shear (diagonal tension). In particular, the shear reinforcement should be designed for the excess shear, the difference between the hydraulic factored ultimate shear force, V_{uh}, and the shear strength provided by the concrete, ϕV_c, where ϕ is the concrete resistance factor for shear design. Therefore, the design shear for the reinforcement, V_s, is given by

$$V_s \geq \left(\frac{V_{uh} - 1.3\phi V_c}{\phi} \right) \tag{3.1}$$

b. Single Load Factor Method. In the single load factor method, both the dead and live loads are multiplied by the same load factor.

$$U = 1.7(D + L) \tag{3.2}$$

where

U = factored loads for a nonhydraulic structure
D = internal forces and moments from dead loads
L = internal forces and moments from live loads

$$U_h = H_f\left[1.7(D + L)\right] \tag{3.3}$$

where

U_h = factored loads for a hydraulic structure
H_f = hydraulic factor.

For hydraulic structures the basic load factor, 1.7, is multiplied by a hydraulic factor, H_f, where $H_f = 1.3$, except for members in direct tension. For members in direct tension, H_f = 1.65. Other values may be used subject to consultation with and approval from CECW-ED.

An exception to the above occurs when resistance to the effects of unusual or extreme loads such as wind, earthquake or other forces of short duration and low probability of occurrence are included in the design. For those cases, one of the following loading combinations should be used:

$$U = 0.75 U_{W\,or\,E} \tag{3.4}$$

for nonhydraulic structures and

$$U = H_f \left(0.75 U_{W\,or\,E} \right) \tag{3.5}$$

for hydraulic structures where

$U_{W\,or\,E}$ = nonhydraulic factored loads including wind or earthquake effects.

c. Modified ACI 318. The load factors prescribed in ACI 318 may be applied directly to hydraulic structures with two modifications. The load factor for lateral fluid pressure, *F*, should be taken as 1.7 instead of the ACI 318 prescribed value of 1.4. Also, for hydraulic structures, the factored load combination for total factored design load, *U*, as prescribed in ACI 318 shall be increased by the hydraulic factor $H_f = 1.3$, except for members in direct tension. For members in direct tension, $H_f = 1.65$.

$$U = 1.4D + 1.7L \tag{3.6}$$

for nonhydraulic structures and

$$U_h = H_f U = H_f \left(1.4D + 1.7L \right) \tag{3.7}$$

for hydraulic structures.

For certain hydraulic structures such as U-frame locks and channels, the live load can have a relieving effect on the factored load combination used to determine the total factored load effects. In this case, the combination of factored dead and live loads with a live load factor of unity

$$U_h = H_f \left(1.4D + 1.0L \right) \tag{3.8}$$

should be investigated and reported in the design documents.

d. Earthquake effects. If resistance to earthquake loads, *E*, are required, the following definitions and load combinations shall apply.

1) Unusual and extreme loads. Earthquake loads are considered either unusual or extreme due to their low probability of occurrence and short duration. Yet, their low probability of occurrence also allows them to be combined with normal operating loads, such as normal operating pool levels for hydraulic structures, when developing load combinations.

2) Design earthquake definitions. In developing earthquake loads, three different earthquakes may need to be considered. The Maximum Credible Earthquake, MCE, the Maximum Design Earthquake, MDE, and the Operating Basis Earthquake, OBE are all potential critical earthquakes when designing for strength and serviceability.

a) Maximum Credible Earthquake (MCE). The MCE, as defined in ER 1110-2-1806, is the greatest earthquake that can reasonably be expected to be generated by a specific source near the structural site on the basis of seismological and geological evidence. Multiple MCE's may be defined for a given site, each with their own individual characteristic ground motion parameters and spectral shapes.

b) Maximum Design Earthquake (MDE). The MDE, as defined in ER1110-2-1806, is the maximum level of ground motion for which a structure is designed or evaluated. The associated performance requirement is that the project performs without catastrophic failure although severe damage or economic loss may be tolerated. For critical features, the MDE is the same as the MCE. For all other features, the MDE shall be selected as a lesser earthquake than the MCE, which provides economical designs meeting appropriate safety standards. The MDE can be characterized as a deterministic or probabilistic event. (A reasonable earthquake in many cases is associated with a 10-percent probability of being exceeded in 100 years, i.e., a return period of 950 years.) The MDE load is considered an extreme load case due to the very low probability (potentially high magnitude) of occurrence and should be combined with usual loads, normally expected loads and pool levels.

c) Operational Basis Earthquake (OBE). The OBE, as defined in ER1110-2-1806, is an earthquake that can reasonably be expected to occur within the service life of the project, that is, with a 50-percent probability of being exceeded during the service life. (This corresponds to a return period of 144 years for a project with a service life of 100 years.) The associated performance requirement is that the project function with little or no damage, and without interruption of function. The purpose of the OBE is to protect against economic losses from damage or loss of service, and therefore alternative choices of return period for the OBE may be based on economic considerations. The OBE is considered an unusual load due to the low probability of occurrence and should be used with usual loads, normally expected loads and pool levels.

3) OBE load factors. There are separate load factors for OBE's that are generated using standard (nonsite-specific) and site-specific ground motions. The load factor for a site-specific developed OBE reflects the increased reliability of the load by reducing the load factor.

a) Standard ground motion analysis. The load factors given below are to be used when the seismic coefficient or standard design spectrum is used for developing equivalent static seismic forces for an OBE. The load combinations including a nonsite specific OBE analysis are

$$U_E = 1.4(D+L)+1.5E \tag{3.9}$$

and substituting into equation 3.5 gives

$$U_h = 0.75\left[H_f\left(1.4(D+L)+1.5E\right)\right] \tag{3.10}$$

for hydraulic structures. The dead and live loads in this equation are to be developed using normal operating conditions.

b) Site specific ground motion. Site-specific ground motions are typically used to perform time-history or response spectrum analyses. The site-specific response spectrum can be developed using a computer program such as DEQAS. The response spectrum must be developed using specific location data and the definition of an OBE. Similarly, a deterministic acceleration time history must reflect the specific dynamic characteristics of the site and the definition of an OBE. The load combinations including a site-specific OBE analysis are

$$U_E = 1.4(D+L)+1.4E \tag{3.11}$$

and substituting into equation 3.4 gives

$$U_h = 0.75\left[H_f\left(1.4(D+L)+1.4E\right)\right] \tag{3.12}$$

The dead and live loads in this equation are to be developed using normal operating conditions.

4) MDE load factors. There are separate load factors for MDE's that are generated using standard and site-specific ground motions. The load factor for a site-specific developed MDE reflects the increased reliability of the load by reducing the load factor.

a) Standard ground motion. The load factors given below are to be used when the seismic coefficient or standard design spectrum is used for developing equivalent static seismic forces. These seismic coefficients or design spectra can be found using the procedure outlined in EM6050. The load combinations including a nonsite specific MDE analysis are

$$U_E = 1.0(D+L)+1.25E \tag{3.13}$$

and substituting this equation into equation 3.4 gives

$$U_h = 0.75\left[H_f\left(1.0(D+L)+1.25E\right)\right] \tag{3.14}$$

The dead and live loads in this equation are to be developed using normal operating conditions.

b) Site specific ground motion. Site-specific ground motions are to be used to perform time-history or response spectrum analyses. The site-specific response spectrum can be developed using the procedure outlined in EM6050 using the NEHRP maps or a computer program such as DEQAS. The response spectrum must be developed using specific location data and the definition of an MDE. The load combinations including a site-specific MDE analysis are

$$U_E = 1.0(D+L) + 1.0E \tag{3.15}$$

and substituting this equation into equation 3.4 gives

$$U_h = 0.75\left[H_f\left(1.0(D+L) + 1.0E\right)\right] \tag{3.16}$$

The dead and live loads in this equation are to be developed using normal operating conditions.

3-4. Design Strength of Reinforcement

a. Design should normally be based on 60,000 psi, the yield strength of ASTM Grade 60 reinforcement. Other grades may be used, subject to the provisions of paragraphs 2-2 and 3-4.b. The yield strength used in the design shall be indicated on the drawings.

b. Reinforcement with a yield strength in excess of 60,000 psi shall not be used unless a detailed investigation of ductility and serviceability requirements is conducted in consultation with and approved by CECW-ED.

3-5. Maximum Tension Reinforcement

a. For singly reinforced flexural members, and for members subject to combined flexure and compressive axial load when the axial load strength ϕP_n is less than the smaller of $0.10 f_c' A_g$ or ϕP_b, the ratio of tension reinforcement ρ provided shall conform to the following:

1) Recommended limit = $0.25\rho_b$.

2) Maximum permitted upper limit not requiring special study or investigation = $0.375\rho_b$. Values above $0.375\rho_b$ will require consideration of serviceability, constructibility, and economy.

3) Maximum permitted upper limit when excessive deflections are not predicted when using the method specified in ACI 318 or other methods that predict deformations in substantial agreement with the results of comprehensive tests = $0.50\rho_b$.

4) Reinforcement ratios above $0.50\rho_b$ shall only be permitted if a detailed investigation of serviceability requirements, including computation of deflections, is conducted in consultation with and approved by CECW-ED. Under no circumstance shall the reinforcement ratio exceed $0.75\rho_b$.

b. Use of compression reinforcement shall be in accordance with provisions of ACI 318.

3-6. Control of Deflections and Cracking

a. Cracking and deflections due to service loads need not be investigated if the limits on the design strength and ratio of the reinforcement specified in paragraphs 3-4.a and 3-5.a(3) are not exceeded.

b. For design strengths and ratios of reinforcement exceeding the limits specified in paragraphs 3-4.a and 3-5.a(3), extensive investigations of deformations and cracking due to service loads should be made in consultation with CECW-ED. These investigations should include laboratory tests of materials and models, analytical studies, special construction procedures, possible measures for crack control, etc. Deflections and crack widths should be limited to levels which will not adversely affect the operation, maintenance, performance, or appearance of that particular structure.

3-7. **Minimum Thickness of Walls**

Walls with height greater than 10 feet shall be a minimum of 12 inches thick and shall contain reinforcement in both faces.

CHAPTER 3

STRENGTH AND SERVICEABILITY COMMENTARY

C.3-1 **General**

a. Nonhydraulic vs. hydraulic structures. For hydraulic structures, cracking, vibrations and stability are major serviceability concerns. In the past the allowable stress design methodology was used and the allowable stress for concrete members in these hydraulic structures was reduced from $0.45f_c'$ to $0.35f_c'$. This reduction in allowable stress produced deeper concrete members with lower stress levels and increased reinforcing requirements. This increase in concrete depth is beneficial for hydraulic structures, which often depend on mass for stability (more concrete), are lightly reinforced (no shear reinforcement) and often susceptible to vibrations (mass and damping). The increased reinforcing requirements considerably help in improving crack control. The hydraulic factor is used in the Load and Resistance Factor Design of hydraulic structures for the same purpose. Note that 0.45/0.35 is approximately 1.3 the value for H_f when a member is not in direct tension. The fact that the reduced allowable stress approach was sound and produced reasonable serviceability results led to incorporating this concept into the Load and Resistance Factor methodology. The use of a hydraulic factor is simple and eliminates the necessity for separate serviceability analyses.

b. Single and modified ACI 318 load factor approaches. The single load factor approach uses one load factor (1.7) for both dead and live load, whereas, ACI 318 uses different load factors for each of dead (1.4) and live load (1.7). USACE has chosen to use the ACI resistance factors which are higher than those found (derived) for USACE structures. Therefore, the load factors for the USACE structures are also slightly higher than they should be using conventional load and resistance factor approaches to develop USACE structural load factors.

In the case of hydraulic structures where fluid pressure is the primary live load, the ACI 318 method requires the use of a 1.4 load factor for fluid pressure. This ACI 318 requirement is overridden by this document stating that a load factor of 1.7 will be used for fluid pressure making the two procedures, nearly identical. The only difference being in the dead load factors of 1.7 versus 1.4 (ACI 318). For structures with large dead load components and limited fluid pressures, the ACI 318 procedure will provide lower strength requirements.

The need for load factors is best described through demand capacity relationships and safety. If structures were accurately analyzed, correctly designed, perfectly built and expectedly loaded, a capacity slightly beyond the demand would provide an adequate design. Unfortunately, most of these actions have uncertainties requiring the implementation of safety margins. Actual loads may be different in magnitude and distribution from the design loads. The loads may be instantaneous, daily, annual or once-in-a-service life. Modeling assumptions, limitations and simplifications may provide results that are slightly to radically different than the actual conditions. In addition to the relationship between demand and capacity, consequences of failure (importance) must be considered in the load factors. Loss of life and property or large economic losses due to a failure requires a larger margin of safety than minor disruption of service or minor inconveniences.

The need for resistance factors is based on the variability of member strength. The strength of a member must exceed the demand for all foreseeable loads without failure or significant distress. The actual strength of each member is different than the nominal calculated values. These differences are related to variations in as-built versus assumed (analysis/design) material properties, cross-sectional dimensions, reinforcement placement, and also the accuracy of the analytical procedures. In certain instances, these variations cause a reduction in the actual strength compared to the calculated values.

In order to guarantee safety with respect to a structure or individual member, the nominal strength must be reduced by a (resistance) factor and the loads must be increased by (load) factors. These resistance factors, ϕ, account for the variability in the strength, in particular a possible reduced member strength from the calculated value, and the load factors, γ, account for possible overloaded or inappropriately loaded system from the assumed loading. This translates into an equation such as:

$$\phi R_n \geq \sum_{i=1}^{l} \gamma_i Q_i \qquad \text{(C3.1)}$$

which is the basis for strength design where ϕ is the resistance factor (less than 1.0), R_n is the calculated nominal capacity of the member, γ_i is the load factor for the ith load, Q_i is the ith load type (dead, live, earthquake, etc.) and l is the number of load types.

Safety margins can be defined as the difference between the strength and the load effects as shown in Figure C3.1. The margin of safety is a random variable that gives rise to a probability distribution with the characteristics shown in Figure C3.2

The typical probability distribution used for defining the probability of failure is that for ln(*R/Q*), shown in Figure C3.3. Whenever the probability distribution is below zero in Figures C3.2 and C3.3, failure occurs. Therefore, the area (probability of failure) to the left of zero must be minimized to an acceptable value. This is typically achieved by forcing the mean value of the margin of safety to be a specified number of standard deviations from the origin. This multiplier is referred to as the safety index, β, and is typically taken as a value between 3 and 4. (AISC uses 2.5 for members with wind, 3.0 and 4.5 for connections for members with dead plus live and 1.75 for members with earthquake loads.) Historical data and back calculation of the safety indices from successful designs, has led to these values. The safety index is highly dependent upon the variability of the loading and member resistance. The value of the safety index is approximated by this formula:

$$\beta \cong \frac{\ln(R_m / Q_m)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(C3.2)}$$

where the subscript *m* stands for mean values of resistance and loads and, *V*, is the variability of the resistance, subscript *R*, and the load, subscript *Q*. A safety index between 3 and 4 provides a probability of failure on the order of one in one hundred thousand. Unfortunately, loads and strengths are rarely given as mean values, rather as nominal strengths and actual loads. Loads are usually provided as expected upper limits, not average values and the load factors need to be adjusted accordingly. The load and resistance factors can be selected based on a specified safety index and the (actual or perceived) variability of the loads and the strengths. ACI 318 resistance (φ) factors are based on a perceived value of R_m/R_n that is different than the ratio for USACE designed structures and the USACE design documents. Since the mean to nominal ratio for USACE designed members is larger than that for ACI 318, the USACE resistance factors should be lower, but in order to remain consistent with ACI 318, this difference is reflected in the increased load factors used by USACE. The increase in load factors also reflects the higher variability in the loadings seen by these structures.

C.3-2 **Stability Analysis**

a. Unfactored loads. Although there must be some inherent safety margin related to the stability of the structure, the stability of a structure is determined using the service loads. The stability of a structure is heavily dependent upon the actual ratio of loads such as dead to live loads. If unequal load factors are applied to the dead and live loads, the stability calculations no longer reflect the actual phenomenon and the potential for instability can be over exaggerated. (This typically occurs since the live load factor is typically larger than the dead load factor. Since the dead load (mass) is often used in hydraulic structures to counteract the live loads relative to stability, a smaller dead load factor would underestimate the structure's ability to resist the live load effects.) The components of the unfactored internal forces can then be multiplied by the appropriate load and hydraulic factors and combined in order to design the component members.

b. Soil structure interaction load factors. Soil-structure interaction requires the direct interaction of the soil, structure and loads. It is not advisable to separate the loads into their components, analyze the loads separately and then add multiples of the resulting forces as soil-structure systems typically are nonlinear systems and superposition of loads does not apply. Changing each component of the load in proportion to its load factor and performing an SSI analysis with this factored loading will not produce the same results as performing a series of analyses, one for each load type, and summing factored results for each component load. (Simply stated a factored load used in an SSI analysis typically will not produce the same result as taking the unfactored results and multiplying by the load factor. The results are dependent upon the stress-strain history of the soil which is typically non linear.) Therefore, it is best to perform the SSI analysis using the service loads and then apply one single load factor on the resulting internal force components. More detailed information can be found in EM 6051 (Time-History Dynamic Analysis of Concrete Hydraulic Structures) and in EM 2906 (Pile Foundations). Most SSI analyses performed to date are static P-Y curve analyses. As computing power increases and more nonlinear SSI software packages become available, each type of analysis and its interaction with load components must be carefully examined to ensure reasonable, accurate and realistic results.

C.3-3 **Required Strength**

a. General. The hydraulic factor is included within the V_{uh} term. This increase in V_{uh} will require an increase in the shear reinforcement or a deeper section to account for the increased design shear. This additional shear reinforcement and/or deeper sections are used to control cracking of the concrete, an important serviceability requirement for hydraulic structures. Historically, this method has produced structures with limited cracking and serviceability issues and reducing the need for extensive serviceability analyses in many instances.

b. Single Load Factor Method. The increase in H_f from 1.3 to 1.65 for members in direct tension is directly related to the historically lower allowable stress for direct tension members in the Allowable Stress Design code. The reason for increasing the hydraulic factor or decreasing the allowable stress for members in direct tension is to reduce the potential for crack propagation. Due to the difficulty in predicting crack propagation, this simpler approach of increasing the hydraulic factor coupled with successful experience led to this factor of 1.65.

The 0.75 factor accounts for the low probability of the maximum wind, earthquake or other short duration load occurring simultaneously with the maximum dead and live loads. These loads would be considered unusual or extreme loads such as an OBE, operating basis earthquake or the MCE, maximum credible earthquake.

c. Modified ACI 318. The first modification of changing the ACI 318 prescribed lateral fluid pressure factor from 1.4 to 1.7 is due to the high variability and importance of the USACE lateral fluid pressure loads. These loads are considered by the USACE to be similar to live loads for typical reinforced concrete structures. The second modification is to apply the hydraulic factor H_f as described in C.3-3.b.

It is also suggested that a load case using load factors of normal dead load factors and unity for the live load be used for certain hydraulic structures. For some internal forces, the use of different load factors for the dead and live loads can cause a relieving effect on the internal forces, similar to the discussion for stability in C.3-2.a.

d. Earthquake effects.

1) Unusual and extreme loads. Unusual or extreme loads are typically low probability and/or short duration loads. (Floods, high winds, and earthquakes are examples of these types of loads.) The likelihood of maximum combinations of these low probability/short duration loads is very small, therefore the combination of these low probability/short duration events are not required. Similarly, it is unlikely that the maximum earthquake will occur with the maximum live load; therefore the live load factors can be adjusted to reflect the low probability of this combination occurring. Also, for low probability/short duration loads, the expected stress levels and serviceability requirements can be modified to reflect their infrequent occurrence and short-lived effects on the structure.

2) Design earthquake definitions.

a) Maximum Credible Earthquake (MCE). A MCE is the greatest earthquake that can be reasonably generated by a specific source for the structure's region, a once in the lifetime of the source event. It is anticipated that inelastic action within the structure will take place during this type of event. The structure may not be operational after an MCE, but the structure should still be adequate in protecting lives and large economic consequences.

b) Maximum Design Earthquake (MDE). A MDE can be defined as the level of earthquake for which the structure could reasonably see once in its service life. Therefore, 10% probability in 100 years is reasonable for most cases, but may need to be redefined if large loss of life or large economic consequences would result from a once in a service life event, essentially an MDE. For critical structures and features, the MDE can be the maximum credible earthquake (MCE).

c) Operational Basis Earthquake (OBE). An OBE can be defined as an earthquake that the structure is likely to see in its service life. Therefore, 50% probability in 100 years is reasonable for most cases, but may need to be modified if a longer service life is expected or the need for a longer potential return period is chosen based on probable economic losses. In an OBE, the structure is to behave elastically with minimal amount of nonlinear behavior, therefore causing minor damage. The structure is to remain in operation or be operational in a short period of time with minor repairs after an OBE.

3) OBE load factors. Different load factors are used for OBE's generated using site-specific and standard procedures. The techniques used to generate site-specific earthquakes are assumed to produce more reliable (less variability) earthquakes, which produces a lower load factor. The standard (non-site specific) earthquakes are based on a generalization of the earthquake potential within the region, and they are considered less reliable (more variability) than a site-specific generated earthquake. Therefore, they will have higher load factors to reflect this increased variability.

a) Standard ground motion analysis. The reduction in the load factors for the dead and live loads is due to the unlikely probability of the maximum dead and live loads occurring with the OBE event. The higher load factor for the earthquake load is a direct result of the high variability (uncertainty) in this standard earthquake load. This uncertainty is a function of the nature of earthquakes coupled with the development and use of seismic coefficients without regards to site-specific seismic characteristics.

b) Site-specific ground motion. The earthquake load factor is reduced due to the determination of a site-specific earthquake that incorporates the potential sites and local attenuation characteristics in the earthquake load. Performing a site-specific seismic analysis to define a site-specific response spectrum or time history reduces the variability in the seismic load and therefore reduces the seismic load factor. The reduction is limited since earthquake load predictions are still uncertain with a fairly high variability and the likelihood of the structure seeing this earthquake during it lifetime is quite high.

4) MDE load factors. Different load factors are used for MDE's generated using site-specific and standard procedures. The techniques used to generate site-specific earthquakes are assumed to produce more reliable (less variability) earthquakes, which produces a lower load factor. The standard (non-site specific) earthquakes are based on a generalization of the earthquake potential within the region, and they are considered less reliable (more variability) than a site-specific generated earthquake. Therefore, they will have higher load factors to reflect this increased variability.

a) Standard ground motion analysis. The unity load factor on the dead and live loads is due to the very low probability of the maximum dead and live load occurring with the maximum design earthquake. Also, the unity load factors directly reflect the fact that the MDE is considered a once in a lifetime event and that limited inelastic behavior will be tolerated for this event. The lower load factor on the MDE earthquake load versus that for the OBE is due to the lower probability of occurrence of an MDE than an OBE.

b) Site-specific ground motion. The unity load factor on the dead and live loads is due to the very low probability of the maximum dead and live load occurring with the maximum design earthquake. The earthquake load factor is reduced to unity due to the site-specific procedure used to define the MDE. Performing a site-specific seismic analysis to define the site-specific response spectrum or time history reduces the variability in the seismic load and therefore reduces the seismic load factor. The reduction to unity for the earthquake load factor also reflects the once in the lifetime definition of the MDE and that limited inelastic behavior will be tolerated for this event.

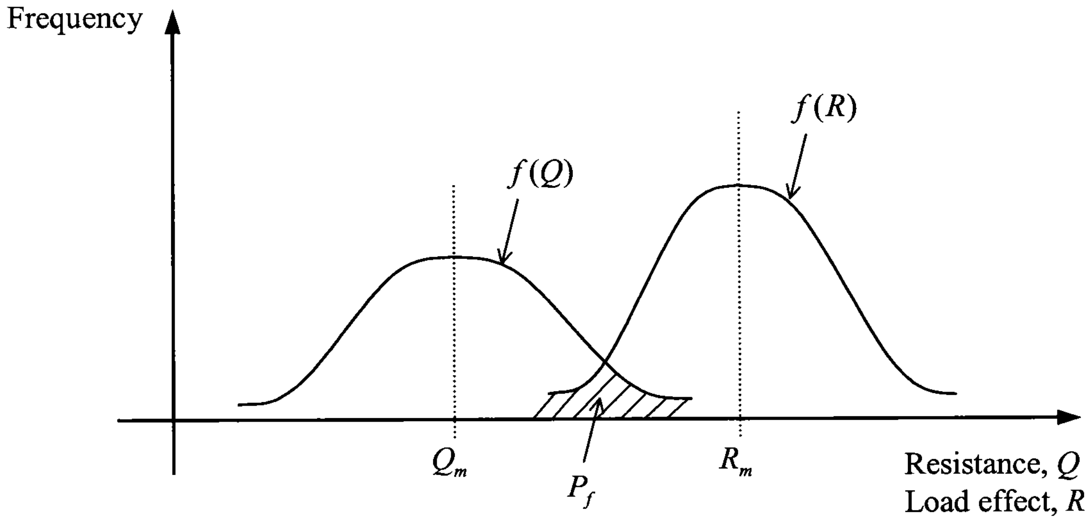

Figure C3.1. Probability density functions for strength and load effect

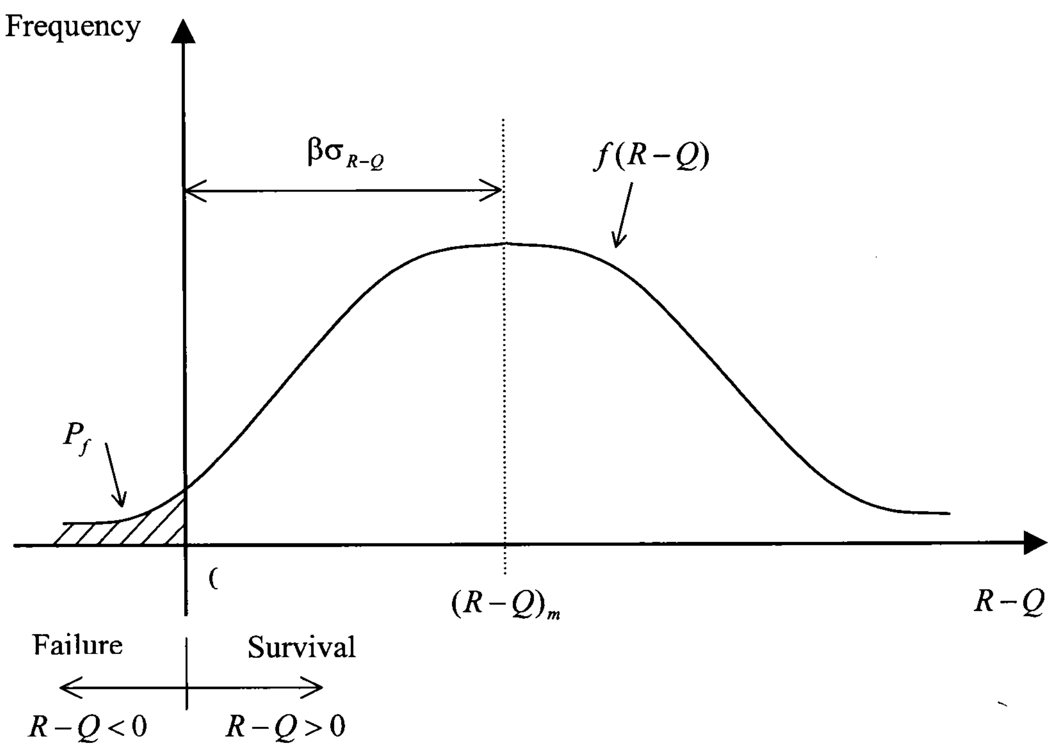

Figure C3.2. Characteristics of *R-Q*

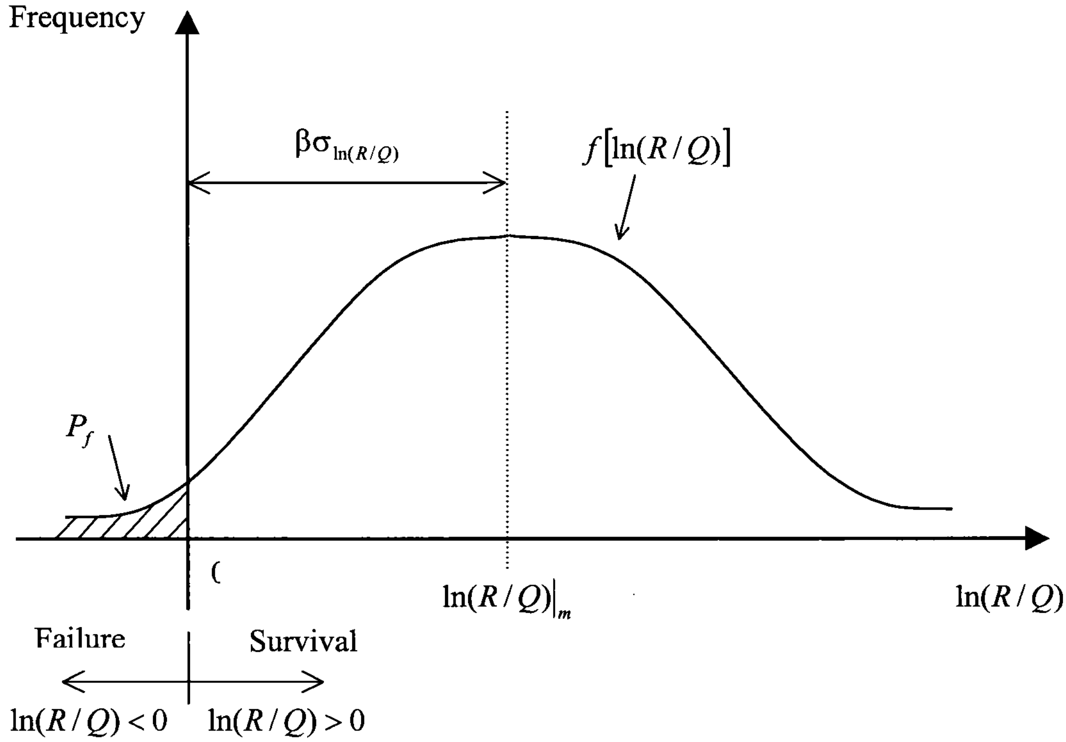

Figure C3.3. Characteristics of ln(*R/Q*)