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# Guide for the Design and Construction of Fixed Offshore Concrete Structures

## Reported by ACI Committee 357

Harvey H. Haynes, chairman    A.E. Fiorato, secretary

A. Leon Abolitz
Arthur R. Anderson
Jal N. Birdy
Irvin Boaz
Anthony D. Boyd
William J. Cichanski
George F Davenport
Joseph A. Dobrowolski
J.M. Duncan
Svein Fjeld
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Alan H. Mattock
Karl H. Runge
B.P. Malcolm Sharples
Charles E. Smith
Raymond J. Smith
Stanley G. Stiansen
Alfred A. Yee
Shu-Yin Yu

Associate Members

Nicholas Carino
John Clarke
Odd E. **Gjørv**
Eivind Hognestad
Masatane Kokubu
W. Frank Manning
T.H. Monnier

*The report provides a guide for the design and construction of fixed reinforced andlor prestressed concrete structures for service in a marine environment. Only fixed structures which are founded on the seabed and obtain their stability from the vertical forces of gravity are covered.*

*Contents include: materials and durability; dead, deformation, live, environmental, and accidental loads; design and analysis; foundations; construction and installation; and inspection and repair. Two appendixes discuss environmental loads such as wave, wind, and ice loads in detail, and the design of offshore concrete structures for earthquake resistance.*

**Keywords:** anchorage (structural); concrete construction; construction materials; cracking (fracturing); dynamic loads; earthquakes; earthquake resistant structures; foundations; grouting; harbor structures; inspection; loads (forces); ocean bottom; offshore structures; post-tensioning; prestressed concrete; prestressing steels; reinforced concrete; repairs; static loads; structural analysis; structural design; underwater construction.

## CONTENTS



ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be phrased in mandatory language and incorporated into the Project Documents.

Supersedes ACI 357-78 (Reaffirmed 1982).

## PREFACE

Concrete structures have been used in the North Sea and other offshore areas of the world. With the rapid expansion of knowledge of the behavior of concrete structures in the sea, and discoveries of hydrocarbons off North American shores, there will likely be an increased use of such structures. This report was developed to provide a guide for the design and construction of fixed offshore concrete structures. Reference to the following documents is acknowledged:

*API Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms,* API RP2A, American Petroleum Institute.

*Recommendations for the Design and Construction of Concrete Sea Structures,* Fédération International de la Précontrainte.

*Rules for the Design, Construction, and Inspection of Offshore Structures,* Det Norske Veritas.

Where adequate data were available, specific recommendations were made, while in less developed areas particular points were indicated for consideration by the designer. The design of offshore structures requires much creativity of the designer, and it is intended that this guide permit and encourage creativity and usage of continuing research advancements in the development of structures that are safe, serviceable and economical.

## NOTATION

$A$ = accidental load
$C_m$ = hydrodynamic coefficient
$D$ = dead load, diameter of structural member
$E$ = environmental load
$E_c$ = concrete modulus of elasticity
$E_i$ = initial modulus of elasticity
$E_o$ = frequently occurring environmental load
$E_{max}$ = extreme environmental load
$E_s$ = reinforcing steel modulus of elasticity
$L$ = live load
$L_{max}$ = maximum live load
$L_{min}$ = minimum live load
$T$ = deformation load
$W/C$ = water-cement ratio
$b$ = section width
$d_b$ = diameter of reinforcing bar
$d_e$ = effective tension zone
$f_c$ = stress in concrete
$f_{CD}$ = allowable design stress in concrete
$f_s$ = stress in reinforcing bar
$f_{SD}$ = allowable design stress in reinforcing bar
$f_t$ = mean tensile strength of concrete
$f_y$ = yield stress of reinforcing bars
$f'_c$ = 28-day strength of concrete (ACI 318)
$h$ = section thickness
$x$ = depth of compression zone
$\gamma_c$ = material factor for cohesive soils
$\gamma_f$ = material factor for friction type soils
$\gamma_L$ = load multiplier
$\gamma_m$ = material factor
$\gamma_{mc}$ = material factor for concrete
$\gamma_{ms}$ = material factor for reinforcing bars
$\Delta_{ps}$ = increase in tensile stress in prestressing steel with reference to the stress at zero strain in the concrete
$\varepsilon$ = strain
$\lambda$ = wave length
$\phi$ = strength reduction factor

# CHAPTER 1-GENERAL

## 1.1-Scope

This report is intended to be used as a guide for the design of fixed reinforced and/or prestressed concrete structures for service in a marine environment. Only fixed structures which are founded on the seabed and obtain their stability from the vertical forces of gravity are covered herein. Such structures may be floated utilizing their own positive buoyancy during construction and installation, however.

This report is not intended to cover maritime structures such as jetties or breakwaters, or those which are constructed primarily as ships or boats. ACI 318 should be used together with this report. Because of the nature of the marine environment, certain recommendations herein override the requirements of ACI 318.

## 1.2-Instrumentation

In regions of the structure or foundation where it is necessary to actively control conditions to insure an adequate margin of safety for the structure, instrumentation should be provided to monitor the conditions. Such conditions might be fluid level, temperature, soil pore water pressure, etc.

Adequate instrumentation should be provided to insure proper installation of the structure.

When new concepts and procedures that extend the frontier of engineering knowledge are used, instrumentation should be provided to enable measured behavior to be compared with predicted behavior.

## 1.3-Auxiliary systems and interfaces

Special consideration should be given to planning and designing auxiliary nonstructural systems and their interfaces with a concrete structure.

Auxiliary mechanical, electrical, hydraulic, and control systems have functional requirements that may have a significant impact on structural design. Special auxiliary systems may be required for different design phases of an installation, including construction, transportation, installation, operation, and relocation.

Unique operating characteristics of auxiliary systems should be considered in assessing structural load conditions. Suitable provisions should be made for embedments and penetrations to accommodate auxiliary equipment.

# CHAPTER 2-MATERIALS AND DURABILITY

## 2.1-General

All materials to be used in the construction of offshore concrete structures should have documentation demonstrating previous satisfactory performance under similar site conditions or have sufficient backup test data.

## 2.2-Testing

**2.2.1-** Tests of concrete and other materials should be performed in accordance with applicable standards of ASTM listed in the section of ACI 318 on standards cited. Complete records of these tests should be available for inspection during construction and should be preserved by the owner during the life of the structure.

**2.2.2-** Testing in addition to that normally carried out for concrete Structures, such as splitting or flexural tensile tests, may be necessary to determine compliance with specified durability and quality specifications.

## 2.3-Quality control

**2.3.1-** Quality control during construction of the concrete structure is normally the responsibility of the contractor. Supervision of quality control should be the responsibility of an experienced engineer who should report directly to top management of the construction firm. The owner may provide quality assurance verification independent of the construction firm.

## 2.4-Durability

**2.4.1-** Proper ingredients, mix proportioning, construction procedures, and quality control should produce durable concrete. Hard, dense aggregates combined with a low water-cement ratio and moist curing have produced concrete structures which have remained in satisfactory condition for 40 years or more in a marine environment.

**2.4.2-** The three zones of exposure to be considered on an offshore structure are:

(a) The submerged zone, which can be assumed to be continuously covered by the sea water.

(b) The splash zone, the area subject to continuous wetting and drying.

(c) The atmospheric zone, the portion of the structure above the splash zone.

**2.4.3-** Items to be considered in the three zones are:

(a) Submerged zone-Chemical deterioration of the concrete, corrosion of the reinforcement and hardware, and abrasion of the concrete.

(b) Splash zone-Freeze-thaw durability, corrosion of the reinforcement and hardware and the chemical deterioration of the concrete, and abrasion due to ice.

(c) Atmospheric zone-Freeze-thaw durability, corrosion of reinforcement and hardware, and fire hazards.

## 2.5-Cement

**2.5.1-** Cement should conform to Type I, II, or III portland cements in accordance with ASTM C 150 and blended hydraulic cements which meet the requirements of ASTM C 595.

**2.5.2-** The tricalcium aluminate content ($C_3A$) should not be less than 4 percent to provide protection for the reinforcement. Based on past experience, the maximum tricalcium aluminate content should generally be 10 percent to obtain concrete that is resistant to sulfate attack. The above limits apply to all exposure zones.

**2.5.3-** Where oil storage is expected, a reduction in the amount of tricalcium aluminate ($C_3A$) in the cement may be necessary if the oil contains soluble sulfates. If soluble sulfides are present in the oil, coatings or high cement contents should be considered.

**2.5.4-** Pozzolans conforming to ASTM C 618 may be used provided that tests are made using actual job materials to ascertain the relative advantages and disadvantages of the proposed mix with special consideration given to sulfate resistance, workability of the mix, and corrosion protection provided to the reinforcement.

## 2.6-Mixing water

**2.6.1-** Water used in mixing concrete should be clean and free from oils, acids, alkalis, salts, organic materials, or other substances that may be deleterious to concrete or reinforcement. Mixing water should not contain excessive amounts of chloride ion. (See Section 2.8.6).

## 2.7-Aggregates

**2.7.1-** Aggregates should conform to the requirements of ASTM C 33 or ASTM C 330 wherever applicable.

**2.7.2-** Marine aggregates may be used when conforming to ASTM C 33 provided that they have been washed by fresh water so that the total chloride and sulfate content of the concrete mix does not exceed the limits defined in Section 2.8.6.

## 2.8-Concrete

**2.8.1-** Recommended water-cement ratios and minimum 28-day compressive strengths of concrete for the three exposure zones are given in Table 2.1.

**2.8.2-** Measures to minimize cracking in thin sections and to prevent excessive thermal stresses in mass concrete are necessary if more than 700 pounds of cement per cubic yard of concrete are used (415 kg per cubic meter). A minimum cement content of 600 pounds per cubic yard (356 kg per cubic meter) is recommended to obtain high quality paste adjacent to the reinforcement for corrosion protection.

**2.8.3-** The rise of temperature in concrete because of cement heat of hydration requires strict control to prevent steep temperature stress gradients and possible thermal cracking of the concrete on subsequent cooling. Reducing the temperature rise may be difficult in the rich mixes and thick sections required in concrete sea structures.

The control of concrete temperatures includes selection of cements which have low heat of hydration, reduced rates of placement, precooling of aggregates, the use of ice to replace some or all of the mixing water and liquid nitrogen cooling, as described in ACI 207.4R. Pozzolans may be used to replace a portion of the cement to lower the heat of hydration.

**2.8.4-** When freeze-thaw durability is required, the concrete should contain entrained air as recommended by Table 1.4.3 of ACI 201.2R. Air entrainment is the most effective means of providing freeze-thaw resistance to the cement paste. Conventional guidelines, such as those contained in Table 1.4.3 generally apply to unsaturated concrete. Where concrete is exposed to frost action in a marine environment, care must be taken to insure that critical water absorption does not occur. Using a rich, air-entrained mix of low water-cement ratio, a pozzolan and an extended curing period are the most effective means of producing a concrete of low permeability, which is essential in such an environment. Lightweight aggregates behave differently from normal weight aggregates. The pores in lightweight aggregate particles are large and less likely to fill by capillary action than normal weight aggregates. However, care must be taken to prevent excessive moisture absorption in lightweight aggregates prior to mixing. Such absorption can result in critical saturation levels if sufficient curing and drying do not take place before the structure is subjected to severe exposures. High strength lightweight aggregates with sealed surfaces are effective in limiting water absorption.

**2.8.5-** Where severe surface degradation of the concrete is expected to occur, the minimum specified concrete strength should be 6000 psi (42 MPa). Additional protection can be achieved by using concrete aggregates having equal or higher hardness than the abrading material or by the provision of suitable coatings or surface treatments.

**2.8.6-** No chlorides should intentionally be added. Total water soluble chloride ion ($Cl^-$) content of the concrete prior to exposure should not exceed 0.10 percent by weight of the cement for normal reinforced concrete and 0.06 percent by weight of cement for prestressed concrete. A chloride ion ($Cl^-$) content of up to 0.15 percent may be acceptable in reinforced concrete but should only be used after evaluation of the potential for corrosion of the specific structure under the given environmental conditions.

**2.8.7-** Structural lightweight concrete should conform to ACI 213R. Where it will be exposed to a freeze-thaw environment, it should be air entrained, and additional measures contained in Section 2.8.4 should be followed.

TABLE 2.1--WATER-CEMENT RATIOS AND COMPRESSIVE STRENGTHS FOR THREE EXPOSURE ZONES

| Zone | Maximum W/C ratio | Minimum 28-day cylinder compressive strength |
|---|---|---|
| Submerged | 0.45 | 5000 psi (35 MPa) |
| Splash | 0.40 | 5000 psi (35 MPa) |
| Atmospheric | 0.40 | 5000 psi (35 MPa) |

## 2.9-Admixtures

**2.9.1-** Admixtures should conform to Section 3.6 of ACI 318. Limits given in this section for calcium chloride should not increase the total limits recommended for concrete as outlined in Section 2.8.6 of this report. When two or more admixtures are used, their compatibility should be documented.

## 2.10-Reinforcing and prestressing steel

**2.10.1-** Reinforcing and prestressing steel should conform to Section 3.5 of ACI 318. Low temperature or cold climate applications may require the use of special reinforcing and prestressing steel and assemblages to achieve adequate ductility. To facilitate future repairs that might be necessary, only weldable reinforcement should be used in the splash zone and other areas susceptible to physical damage. Weldable reinforcement should conform to the chemical composition of ASTM A706.

## 2.11-Post-tensioning ducts

**2.11.1-** Post-tensioning ducts should conform to Section 18.15 of ACI 318.

**2.11.2-** Post-tensioning ducts should be semi-rigid and watertight and have at least 1 mm of wall thickness. Ferrous metal ducts or galvanized metal ducts passivated by a chromate wash may be used. Plastic ducts are not recommended.

**2.11.3-** Bends in ducts should be preformed as necessary. Joints in ducts should be bell and spigot with the ends cut by sawing so as to be free from burrs and dents.

Joint sleeves should fit snugly and be taped with waterproofing tape. Splices should preferably be staggered but where this is impracticable, adequate space should be provided to insure that the concrete can be consolidated around each splice.

**2.11.4-** If flexible metal ducts must be used in special areas of congestion, etc., they should have a mandrel inserted during concrete placement. Bars for supporting and holding down such ducts should have a curved bearing plate against the duct to prevent local crushing.

## 2.12-Grout

**2.12.l-** Grout for bonded tendons should conform to Section 18.16 of ACI 318 and to applicable sections of this report. Suitable procedures and/or admixtures should be used to prevent pockets caused by bleeding when grouting of vertical tendons or tendons with substantial vertical components.

**2.12.2-** Recommendations for mixing water outlined in Section 2.6 also apply to grout mixes.

**2.12.3-** Admixtures may be used only after sufficient testing to indicate their use would be beneficial and that they are essentially free of chlorides, or any other material which has been shown to be detrimental to the steel or grout.

## 2.13-Concrete cover of reinforcement

**2.13.l-** Recommended nominal concrete covers for reinforcement in heavy concrete walls, 20 in. (50 cm) thick or greater are shown in Table 2.2.

Concrete covers of reinforcement should not be significantly greater than prescribed minimums to restrict the width of possible cracks. This would be more critical for those members in flexure.

Table 2.2-RECOMMENDED NOMINAL CONCRETE COVER OVER REINFORCEMENT

| Zone | Cover over reinforcing steel | Cover over post-tensioning ducts |
|---|---|---|
| Atmospheric zone not subject to salt spray | 2 in. (50 mm) | 3 in. (75 mm) |
| Splash and atmospheric zone subject to salt spray | 2.5 in. (65 mm) | 3.5 in. (90 mm) |
| Submerged | 2 in. (50 mm) | 3 in. (75 mm) |
| Cover of stirrups | ½ in. (13 mm) less than those listed above | |

**2.13.2-** If possible, structures with sections less than 20 in. (50 cm) thick should have covers as recommended in Section 2.13.1, but when clearances are restricted the following may be used with caution. Cover shall he determined by the maximum requirement listed below:

(a) 1.5 times the nominal maximum size of aggregate, or

(b) 1.5 times the maximum diameter of reinforcement, or

(c) ¾ in. (20 mm) cover to all steel including stirrups.

Note: Tendons and post-tensioning ducts should have 0.5 in. (13 mm) added to the above.

## 2.14-Details of reinforcement

**2.14.1-** Reinforcement details should conform to Chapters 7 and 12 of ACI 318.

**2.14.2-** Special consideration should be given to the detailing of splices used in areas subjected to significant cyclic loading. Staggered mechanical and welded splices should preferably be used in these instances. Lap splices, if used, should conform to the provisions of ACI 318. In general, noncontact lap splices should be avoided unless adequate justification can be developed for their use. Mechanical devices for positive connections should comply with the section of ACI 318 dealing with mechanical connections. Welded splices may be used where reinforcing steel meets the chemical requirements of ASTM A706.

**2.14.3-** Mechanical or welded connections should be used for load-carrying reinforcing bar splices located in regions of multiaxial tension, or uniaxial tension that is normal to the bar splices.

## 2.15-Physical and chemical damage

**2.15.1-** In those areas of the structure exposed to possible collision with ships, flotsam, or ice, additional steel reinforcement should be used for cracking control and concrete confinement. Particular consideration should be given to the use of additional tension reinforcement on both faces and additional shear reinforcement (transverse to walls) to reinforce for punching shear. Unstressed tendons and unbonded tendons are two techniques which can be used to increase the energy absorption of the section in the post-elastic stage.

**2.15.2-** The possibility of materials and equipment being dropped during handling on and off the platform should be considered. Impact resisting capacity may be provided as mentioned in Section 2.15.1. In addition, protective coverings may be installed such as steel or concrete grids and energy-absorbing materials such as lightweight concrete.

**2.15.3-** A polymer or other special coating may be used to control ice abrasion or adfreeze between an ice feature and a structure. Compatibility between a coating and the underlying concrete should be assessed to preclude problems with bond development, coating delamination caused by air or moisture migration, and freeze-thaw effects.

**2.15.4-** Exposed steel work and its anchor systems should be electrically isolated from the primary steel reinforcement by at least 2 in. (50 mm) of concrete. The use of cathodic protection systems is generally not required for reinforcing steel and prestressing steel embedded in concrete.

**2.15.5-** Exposed steel work should normally be painted or coated to reduce corrosion. Particular care should be taken to insure against corrosion on the edges and horizontal surfaces. Epoxy coatings are normally used for protection of carbon steel plates and fittings. Cathodic protection systems for externally exposed steel should be of the sacrificial anode type. Impressed current should not be used unless positive controls are instituted to prevent embrittlement of the reinforcing and prestressing steel.

## 2.16-Protection of prestressed anchorages

**2.16.1-** The anchorages of prestressed tendons should be protected from direct contact with seawater, which could lead to corrosion. A desirable method is to use recessed pockets so that the steel anchorage and tendon ends may be protected by concrete or grout fill in the pocket. The pocket surface should be thoroughly cleaned and the exposed steel should be coated with bonding epoxy just prior to placing the concrete or grout fill. Particular care should be taken to prevent shrinkage and the formation of bleed lenses. Alternative details are acceptable provided they are designed to limit the penetration of seawater and oxygen to the same degree as that provided to the tendon proper.

## 2.17-Anchorages for embedments and connections to steel work

**2.17.1-** Embedments may be anchored by studs, steel bars, or prestressing tendons. Welds should conform to AWS D1.l or AWS D1.4.

**2.17.2-** Prestressing tendons should normally he used to provide precompression in regions where connections or embedments are subject to cyclic or high dynamic loads.

**2.17.3-** The concrete section to which the embedment or connection is anchored should be adequately reinforced to prevent pullout shear and delamination.

**2.17.4-** Steel plates should have adequate properties to insure against delamination.

**2.17.5-** Where welding will subsequently be carried out on the embedment plate, the effect of heat must be considered. A wood or rubber chamfer strip may be placed around the plate while the concrete is formed to create a reveal and thus prevent spalling of the adjacent concrete. Where long-term protection of vital embedments must be assured, epoxy may be injected behind the plate which has been subjected to heat distortion.

## 2.18-Electrical ground

**2.18.1-** An electrical ground conductor should be provided to protect prestressing tendons and reinforcing steel from accidentally acting as a ground for lightning discharges and other sources of electrical current.

## 2.19-Durability of pipes containing pressure

**2.19.1-** Where long term operation of the platform requires continual pressure difference between the surrounding sea and contained fluids, pipes critical to the maintenance of this pressure and virtually inaccessible should be designed with excess corrosion resistance. Flanged connections and inaccessible valves should be avoided.

## 2.20-Epoxy resins

**2.20.1-** Epoxy resins may be used for waterproofing coatings, sealing construction joints, repairing cracks, and other similar usages. The epoxy resins should be carefully selected on the basis of the materials' suitability for the particular application Required strength, ability to cure and bond to wet concrete for the temperatures involved should all be considered. Refer to ACI Committee 503 recommendations and to manufacturers' instructions, along with specialist literature.

# CHAPTER 3-LOADS

## 3.1- Classifications

Loads may be classified as follows:
Dead loads *D*
Deformation loads *T*
Live loads *L*
Environmental loads *E*
Accidental loads *A*

**3.1.1** *Dead loads-* Dead loads are permanent static loads such as:

Weight of the structure
Permanent ballast and equipment that cannot be removed
External hydrostatic pressure

**3.1.2** *Deformation loads-* Deformation loads consider the effects of the following:

Temperature, including heat of hydration
Differential settlements and uneven seabed
Creep and shrinkage
Initial strains imposed by prestressing cables

**3.1.3** *Live loads-* Live loads may be static or dynamic and may vary in position and magnitude.
Live loads may also result from operation of the structure.
The following are representative examples:
Helicopters
Loads induced by the operation of equipment
Liquids stored internally
Equipment and supplies
Berthing, breasting, and mooring loads
Snow and accumulated ice

**3.1.4** *Environmental loads-* Environmental loads are due to natural phenomena and may result from the following (see Appendix A):

Waves
Wind
Current
Earthquake
Ice (sheet ice, first-year and multiyear ridges, icebergs, etc.)
Marine growth

**3.1.5** *Accidental loads*–Accidental loads result from accidents or misuse, such as:

Collision from service boats, barges, and ships
Dropped objects
Explosion
Loss of assumed pressure differential

## 3.2-Design phases

Loads listed above should be considered for each design phase, including:
Construction
Transportation
Installation
Operation
Retrieval

# CHAPTER 4-DESIGN AND ANALYSIS

## 4.1-General

In addition to the strength and serviceability requirements described below, the survivability of the structure should also be investigated to insure that the structure-foundation system will endure extreme environmental events without catastrophic failure. Appendix B includes survivability criteria for earthquake design of concrete structures.

The guidelines in this section are intended to provide a readily applied basis for practical analysis and design. However, nothing herein is intended to prevent the use of more detailed analytical methods.

Insofar as practicable, the designer should select structur-

TABLE 4.1-ALLOWABLE TENSILE STRESSES FOR PRESTRESS AND REINFORCING STEEL TO CONTROL CRACKING

| Stage | Loading | Allowable stress, ksi $\Delta_{ps}$ | Allowable stress, ksi $f_s$ |
|---|---|---|---|
| Construction: Where cracking during construction would be detrimental to the completed structure | All loads on the structure during construction | 18.5 (130 MPa) | 23.0 (160 MPa) |
| Construction: Where cracking during construction is not detrimental to the completed structure | All loads on the structure during construction | 18.5 (130 MPa) | 30 (210 MPa) or $0.6f_y$ whichever is less |
| Construction | All loads on the structure during transportation and installation | 18.5 (130 MPa) | 23.0 (160 MPa) |
| At offshore site | Dead and live loads plus monthly recurring environmental loads | 11.0 (75 MPa) | 17.0 (120 MPa) |
| At offshore site | Dead and live loads plus extreme environmental loads | | $0.8f_y$ |

al configurations and reinforcing details that will insure a ductile (nonbrittle) mode of failure and avoid progressive collapse.

## 4.2-Strength

The strength of the structure should be such that adequate safety exists against failure of the structure or its components. Among the modes of possible failure that should be considered are:

1. Loss of overall equilibrium
2. Failure of critical sections
3. Instability resulting from large deformations
4. Excessive plastic or creep deformation

## 4.3-Serviceability

The structure should be capable of operating according to its intended function under extreme imposed loading and frequently occurring environmental conditions. Among the conditions that could cause the structure to become unserviceable are:

1. Excessive cracking
2. Unacceptable deformations
3. Corrosion of reinforcement or deterioration of concrete
4. Undesirable vibrations
5. Excessive leakage

## 4.4-Design conditions

**4.4.1** *Strength requirements*-A structure should have adequate strength to resist extreme forces resulting from environment or man-made causes without sustaining permanent damage. It is assumed that these forces will occur at least once during the expected service life of a structure.

Extreme environmental and man-made conditions requiring strength design considerations are either of the following:

(a) Surface waves, currents, and winds with long period recurrence intervals (see Appendix A)

(b) Severe earthquake ground motions (see Appendix B for design earthquakes)

(c) Temporary submergence during construction and deck installation and installation of the structure on site

(d) Severe ice conditions

The recommended recurrence interval for all environmental events is generally 100 years, except that for temporary exposures such as during construction and towing the recurrence interval of the extreme environmental event may be reduced to be commensurate with the actual exposure period and season in which the operation takes place.

**4.4.1.1** *Load combinations.* The required strength of the structure and each member should be equal to or greater than the maximum calculated by the following:

$$U = 1.2(D + T) + 1.6L_{max} + 1.3E_0 \quad (4\text{-}1)$$

$$U = 1.2(D + T) + 1.2L_{max} + \gamma_L E_{max} \quad (4\text{-}2)$$

$$U = 0.9(D + T) + 0.9L_{min} + \gamma_L E_{max} \quad (4\text{-}3)$$

$L_{max}$ = maximum live load

$L_{min}$ = minimum live load

$E_o$ = frequently occurring environmental load (e.g., monthly)

$E_{max}$ = Extreme environmental load

$\gamma_L$ = load multiplier and assumes following values: waves plus current plus wind, $\gamma_L$ = 1.3, earthquakes (see Appendix B), ice, $\gamma_L$ = (should be selected to be consistent with the method of analysis used to calculate the design ice load and should reflect the quality of the data available to describe the design ice feature).

For dead loads $D$, the load multiplier 1.2 should be replaced by 1.0 if it leads to a more unfavorable load combination. In Eq. (4-1) the multiplier 1.3 on $E_o$ should be reduced if a more unfavorable load combination results.

When the design is governed by earthquake, then other transient environmental loads are usually not assumed to act simultaneously. In certain special circumstances, when the design is not controlled by a single environmental load, it may be necessary to consider the simultaneous occurrence of environmental events. However, the overall probability of survival is not required to be any greater than that associated with a single event.

While it is assumed that the critical design loadings will be identified from the load combinations in Eq. (4-l), (4-2), and (4-3), the designer should be aware that there may be other simultaneously occurring load combinations that can cause critical load effects. This may be particularly evident during construction and installation phases.

**4.4.1.2** *Strength Reduction Factors:* The selection of strength reduction factors $\phi$ for concrete members should be based on ACI 318. The $\phi$-factors not only account for variability in stress-strain characteristics of concrete and reinforcement, but also reflect variations in the behavior of different types of concrete members, and variations in quality and construction tolerances.

Alternatively, the expected strength of concrete members can be determined by using idealized stress-strain curves such as shown in Fig. 4.4.1 and 4.4.2 for concrete and reinforcing steel, respectively, in conjunction with material factors $\gamma_m$. For prestressing steel, actual diagrams as supplied by the manufacturer should be used together with a material factor $\gamma_m$ = 1.15.

While the material factors are directly applied to the stress-strain curves to limit the maximum stress, it should be recognized that the intent of using materials factors is similar to using ACI 318 strength reduction factors, in that the use of these factors will achieve the desired reliability.

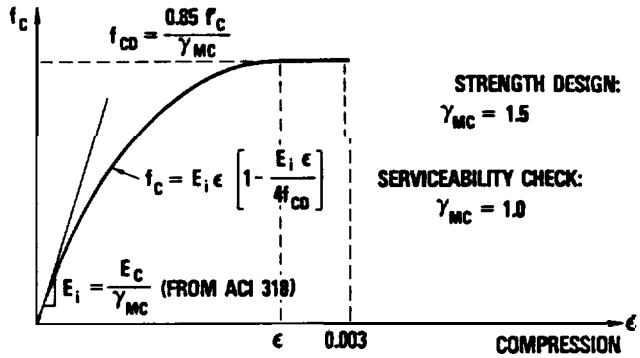
Under no circumstances should $\phi$ factors and $\gamma_m$ factors be used simultaneously.

**4.4.2** *Serviceability requirements*-The structure may be checked elastically (working stress method) or by the use of stress-strain diagrams (Fig. 4.4.1 and 4.4.2) with material factors, $\gamma_m$ = 1.0 to verify its serviceability. It is important that cracking in structural members be limited so that the durability of the concrete is not impaired. Control of cracking based on limiting reinforcing stresses is recommended. Table 4.1 is intended to serve as a guide for limiting such stresses.

Allowable stresses contained in Table 4.1 apply to reinforcing steel oriented within 10 deg of a principal stress direction. Allowable stresses should be reduced if the angular deviation between reinforcing steel and principal stress is more than 10 deg. Guidelines for reducing allowable stresses are contained in Chapter 19 of ACI 318.

For thin-walled, hollow structural cross sections the maximum permissible membrane strain across the walls should not cause cracking under any combination of D, L, T, and E using a load factor, $\gamma_L$ = 1.0. *E* shall be the probable value of environmental event or combination of events corresponding to the recurrence interval selected (usually 100 years).

For structures prestressed in one direction only, tensile stresses in reinforcement transverse to the prestressing steel shall be limited so that the strains at the plane of the prestressing steel do not exceed $\Delta_{ps}/E_s$. This is a supplementary requirement to control longitudinal splitting along prestressing tendons.




4.4.1-Concrete stress-strain curve

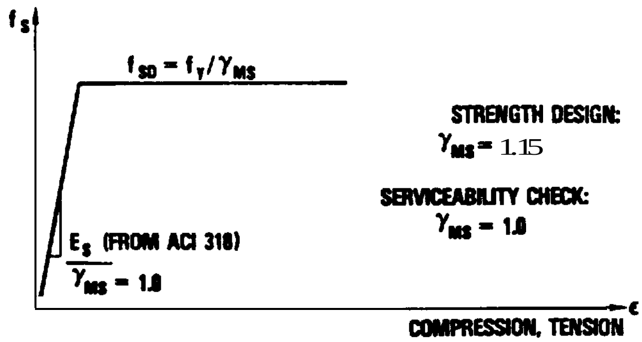



Fig. 4.4.2- Reinforcing steel stress-strain diagram

**4.4.2.1** *Load combinations.* Serviceability needs to be verified for the load combination of Eq. (4-l), except that loads should remain unfactored; i.e.,

$$U = D + T + L + E_o \tag{4-4}$$

where the live load *L* should represent the most unfavorable loading that is expected to prevail during the normal operating life of the structure.

**4.4.2.2** *Material properties.* In the absence of reliable test data for the materials to be used, values for the modulus of elasticity should be selected according to ACI 318.

## 4.5- Special requirements

During sequences of construction and submergence the strength of the structure as well as its serviceability requirement should be verified. Where the acting hydrostatic pressure is the differential between two fluid pressures the applicable load factor should be applied to the larger pressure and the load multiplier of 0.9 should be applied to the smaller. Where physical arrangements make such differential impossible, modification of this rule is permissible.

**4.5.1** *Implosion*-The walls of concrete shell and plate panels should be properly proportioned to prevent catastrophic collapse during periods of large hydrostatic pressure exposure. Potential failure modes to be considered should be material failure and structural instability. For more complex structures such as shell structures, stability should be checked on the basis of a rational analysis of the behavior of the structure, including the influence of loads and second order effects produced by deformations. The latter are to be evaluated by taking into account possible cracking, the effect of reinforcement on the rigidity of the member, creep effects, and the effects of possible geometrical imperfections. The design assumptions made as to geometrical imperfections should be checked by measurements during construction. To allow for observed differences between experimental tests and analytical predictions the safety factor against implosion for stability sensitive geometries should reflect this uncertainty.

**4.5.2** *Use of compressed air*- When internal air pressurization is employed for short term immersions, provision should be made for redundant sources in the event of equipment, power, or valving failures and for additional supplies in the event of leakage. The internal air pressure should not exceed a value equal to the external pressure less 2 atmospheres at any time or at any location, but should not be less than 1 atmosphere.

In any case, the structure should have a load multiplier of 1.05 times the external pressure, assuming loss of internal air pressurization.

Consideration should be given to changes in temperature of the internal air as a result of compression, expansion, and immersion.

**4.5.3** *Liquid containment*- Liquid containment structures should be considered adequately designed against leakage when the following requirements have been satisfied:

1. The reinforcing steel stresses are limited to those of Section 4.4.2
2. The compression zone extends over 25 percent of the wall thickness or 8 in., whichever is less
3. There are no membrane tensile stresses unless other construction arrangements are made such as special barriers to prevent leakage.

**4.5.4** *End closures*- End closures should be designed to obtain smooth stress flow. Shear capability at end closures junctures should be carefully verified with consideration given to the influence of normal forces and bending moment on the shear capacity of concrete sections.

**4.5.5** *Temperature load considerations*-Temperature loads may lead to severe cracking in regions of structural restraints. When investigating thermal effects, consideration should be given to:

1. Identifying the critical fluid storage pattern of a structure
2. Selecting a method that will reliably predict the temperature difference across the walls
3. Defining a realistic model of concrete material behavior to predict induced stresses

To reduce the severity of the effects of thermal strains it is recommended to use the drawdown method, i.e., to maintain a hydrostatic pressure external to the storage containment in excess of the internal fluid pressure.

**4.5.5.1** *Heat of hydration*. During construction of offshore concrete structures thermal strains from the concrete hydration process may result in significant cracking. While it is expected that such temperature increases can be controlled during the concreting process, the designer should check the sensitivity of crack formation due to local temperature rises especially when the structure under consideration consists of massive concrete components interacting through common walls. The designer should consider the effect of such cracking upon future performance of the structure under service and extreme environmental conditions.

**4.5.5.2** *Thermally induced creep.* Creep strain induced by temperature loadings may be a significant proportion of the total strain to which a structural component is subjected during its service life. To assess thermally induced creep the reduced modulus of elasticity method may only be used if all structural components are subjected to the same temperature change.

Where the storage process allows for nonuniform temperature distributions, the reduced modulus may lead to serious errors. In such cases a more refined methodology to assess the differential creep effects is essential to identify unfavorable force redistribution.

**4.5.6** *Minimum reinforcement*-The minimum requirements of ACI 318 should be satisfied. In addition, for loadings during construction, transportation, and operation (including extreme environmental loading), where tensile stresses occur on a face of the structure, satisfactory crack behavior should be insured by providing the following mini-. mum reinforcement on the face:

$$A_s = \frac{f_t}{f_y} b d_e \qquad (4\text{-}5)$$

where

$f_t$ = mean tensile strength of concrete
$f_y$ = yield stress of the reinforcing steel
$b$ = cross-sectional width
$d_e$ = effective tension zone, to be taken as $1.5c + 10\,d_b$, where $c$ = cover of reinforcement and $d_b$ = diameter of reinforcing bar

($d_e$ should be at least 0.2 times the depth of the section but not greater than 0.5 $(h - x)$ where $x$ is the depth of the compression zone prior to cracking and $h$ is the section thickness).

At intersections between structural elements, where transfer of shear forces is essential to the integrity of the structure, adequate transverse reinforcement should be provided.

**4.5.7** *Control of crack propagation*-At critical sections where cracking and consequent hydrostatic pressure in the crack will significantly change the structural loading and behavior (e.g., re-entrant corners), a special analysis should be made as follows:

As a general approach, a crack of depth $1.0c + 7d_b$ (see Section 4.5.6 for definition) should be assumed and the analysis (normally using the finite element approach) should demonstrate that sufficient reinforcement across the crack, anchored in compressive zones, is provided to prevent crack propagation.

**4.5.8** *Minimum deck elevation*-To establish the minimum elevation of decks the following items should be considered:

(a) Water depth related to some reference point such as LAT (lowest astronomical tide)
(b) Tolerances in water depth measurement
(c) Astronomical tide range
(d) Storm surge
(e) Crest elevation of the most probable highest wave (considering the statistical variation in crest heights for waves of similar heights and periods)

(f) Hydrodynamic interaction of structure and environment (caisson effect, run-up, reflected and refractive waves, spray, etc. )

(g) Initial penetration into seabed

(h) Long term and elastic settlement of structure foundation

(i) Inclination of structure

(j) Lowering of seabed due to pressure reduction of oil reservoir-subsidence (where applicable)

(k) Air gap

(1) Maximum ice rubble pileup

## 4.6-Other strength requirements

**4.6.1** *Accidental loads*- Accidental loads are caused by man-made events and are associated with significantly lower probability of occurrence than those events for which the structure and its components are designed. Examples of accidental loads are explosions, very large dropped objects, and collisions.

It is considered fundamental to good design practice to make adequate allowance for the occurrence of accidents. This is usually done through the concept of "alternative load paths" or structural redundancy to prevent the occurrence of progressive collapse.

**4.6.2** *Concrete ductility*- The reinforcing and prestressing steel in primary structural members (e.g., deck support towers) should be arranged and proportioned to provide ductility in regions of maximum bending moment and stress concentrations to insure a ductile mode of failure in the event of the rare natural or man-made event.

Note: It is extremely important to prevent sudden, catastrophic failure due to inadequate shear capacity. Careful consideration is required where shear forces are transmitted through plates, slabs, shearwalls, or curved panels. Use of confining steel in the form of closed stirrups or spirals can significantly increase the apparent ultimate strain capacity, particularly for cyclic loads.

**4.6.3** *Fatigue strength*- The resistance of a structure to fatigue is considered to be adequate if the following stress limitations can be satisfied for frequently recurring environmental loads at sections subjected to significant cyclic stresses:

1. For reinforcing or prestressing steel maximum stress range 20,000 psi (140 Mpa); where reinforcement is bent or welded, 10,000 psi (70 MPa).

2. For concrete $0.5f_c'$, and in addition no membrane tensile stress and no more than 200 psi (1.4 MPa) flexural tension.

3. Where maximum shear exceeds the allowable shear on the concrete alone, and where the cyclic range is more than half the maximum allowable shear in the concrete alone, then all shear should be taken by stirrups. In determining the allowable shear on the concrete alone, the influence of permanent compressive stress on the section may be taken into account.

4. In situations where fatigue stress ranges allow greater latitude than those under the serviceability requirement, Table 4.1, the latter condition shah assume precedence.

In lieu of the stress limitation fatigue check or where fatigue resistance is likely to be a serious problem a more complete analysis based on the principle of cumulative damage should be substituted. This analysis should also consider low-cycle, high amplitude fatigue.

**4.6.4** *Shear in reinforced and prestressed concrete*

**4.6.4.1** *General*. The design and detailing of sections in shear should follow the recommendations of ACI 318.

**4.6.4.2** *Total shear capacity*. The total shear force that can be resisted at a section may be taken as the sum of the component forces contributed from the concrete, reinforcing steel and prestressing steel. The favorable effects of axial compression may be taken into account in assessing shear strength; however, consideration should be given to the shear-compression mode of failure and to the effects of prior cracking under different loading combinations. For cyclic shear loads refer to Section 4.6.3.

## 4.7- Structural analysis

**4.7.1** *Load distribution*- For purposes of determining the distribution of forces and moments throughout a structure when subjected to various external loadings the structure may be assumed to behave elastically with member stiffnesses based on uncracked section properties.

**4.7.2** *Second order effects*-To calculate second order effects on shell structures due to unintentional construction out-of-roundness, the use of stress-strain diagrams for concrete (Fig. 4.4.l), and reinforcing steel (Fig. 4.4.2) is recommended unless diagrams from actual field data are available. For prestressing steel actual diagrams as supplied by the manufacturer should be used. Second order effects (deformations) should be evaluated with a material factor, $\gamma_m = 1.0$.

**4.7.3** *Dynamic amplifications*- The increase in load effects due to dynamic amplification should be considered. The dynamic response should be determined by an established method that includes the effects of the foundation-structure interaction, and the effective mass of the surrounding water.

**4.7.4** *Impact load analysis*- In analyzing impact loads from ice features, dropped objects, boat collisions, etc., the response of the entire system should be considered, including the structure, foundation, and impacting object, if applicable. Material nonlinearities and other dissipative effects should be accounted for in components of the system that exhibit inelastic behavior. The methodology for partitioning energy absorption among system components should be justified. For purposes of structural design, the amount of energy dissipated by the structure should be maximized.

**4.7.5** *Earthquake analysis*- see Appendix B.

# CHAPTER 5- FOUNDATIONS

## 5.1- Site investigation

**5.1.1** *General*- Comprehensive knowledge of the soil conditions existing at the site of construction of any sizeable structure is necessary to permit a safe and economical design. Using various geophysical and geotechnical techniques, subsurface investigations should identify soil strata and soil properties over an area two or more times as wide as the structure and to the full depth that will be affected by anticipated foundation loads. These data should be combined with an understanding of the geology of the region to develop the required foundation design parameters.

The bearing capacity of a mat foundation is largely determined by the strength of the soils close to the sea floor. Consequently, particular attention should be given to developing detailed information on these soils.

A semi-permanent horizontal control system, for example, one employing sea floor transponders, should be established for the site investigation and maintained until installation is accomplished to assure that the structure is placed where subsurface conditions are known.

**5.1.2** *Bottom topography*-A survey of the sea bottom topography should be carried out for all structures. The extent and accuracy of the survey depends on the type of structure, foundation design, and soil conditions. Boulders, debris, and other obstructions should be located and their positions properly recorded if such obstructions would interfere with the installation or operation of the structure.

**5.1.3** *Site geology*- To aid and guide the physical tests of the soil, a preliminary geological study at the location of the structure should be made. This study should be based on the available information on geology, soil conditions, bottom topography, etc., in the general area.

After specific subsurface data are acquired during site investigation, additional geologic studies should be made to aid in identifying conditions that might constitute a hazard to the structure if not adequately considered in design.

**5.1.4** *Stratification*- The site investigation should be sufficiently extensive to reveal all soil layers of importance to the foundation of the structure. In general, soil borings should extend at least to a depth where the existence of a weak soil layer will not significantly influence the performance of the structure. The lateral extent of borings and in situ tests should be sufficient to guide selection of the final position of the structure and to determine what latitude exists with respect to final placement during installation.

Soil conditions may be investigated using the following methods:

(a) Geophysical methods such as high-resolution acoustic profiling and side-scan sonar.

(b) Soil boring and sampling.

(c) In situ tests (e.g., vane shear and cone penetration tests).

Geophysical methods are used for a general investigation of the stratification and the continuity of soil conditions. Geophysical methods alone should not be used to obtain soil properties used in foundation design.

In situ tests may be used to measure certain geotechnical parameters. Such methods may also serve as an independent check on laboratory test results.

At least one boring with sampling and laboratory testing of the samples should be done at the site of each structure.

Sampling should be as continuous as feasible to a depth of 40 ft (12 m) below the mudline. Thereafter, samples should be taken at significant changes in strata, at approximately 10 ft (3 m) intervals to a depth of 200 ft (60 m) below the mudline, and then at approximately 20 ft (6 m) intervals to a depth where a weak soil layer would not significantly affect the performance of the structure.

**5.1.5** *Geotechnical properties*- Tests sufficient to define the soil-structure interaction necessary to determine the safety and deflection behavior of the structure should be made. The number of parameters to be obtained from tests and the required number of tests of each type depend on soil conditions, foundation design, type of structural loading, etc.

**5.1.5.1** *Field tests.* As a guide, the field tests should include at least the following:

(a) Perform at least one miniature vane test on each recovered cohesive sample, and perform unconsolidated-undrained triaxial compression tests or unconfined compression tests on selected typical samples

(b) Perform field water content tests, or record the total weight of sealed disturbed samples to permit subsequent water content measurements to be corrected for water lost during transportation and storage

(c) When possible, in situ testing such as cone penetration tests and field vane tests should also be performed. The piezometer probe, developed for measuring pore pressures during penetration, can be helpful in defining stratigraphy and may also be considered.

All samples should be placed in adequately labeled containers. The containers should be properly sealed and carefully packaged for subsequent laboratory testing.

**5.1.5.2** *Laboratory tests.* In general, the additional testing in the laboratory should include at least the following:

(a) Perform unconsolidated-undrained triaxial compression tests and consolidated-undrained triaxial compression tests with pore pressure measurements on representative samples of cohesive strata to supplement field data and to develop stress-strain relationships. Tests that address strength anisotropy of the soil may be considered if justified by the type of imposed loads on the structure

(b) Determine the water content and Atterberg limits on all cohesive samples

(c) Determine the unit weight of all samples

(d) Investigate the behavior of selected samples under dynamic loading using undrained cyclic triaxial tests

(e) Perform grain size sieve analysis on all coarse grained samples and hydrometer analysis on selected clay and silt samples

(f) Perform consolidation tests on selected undisturbed cohesive samples

## 5.2- Stability of the sea floor

**5.2.1** *Slope stability*- The stability of the sea floor in the vicinity of the structure should be investigated. The study should include the effects of the structure on the soil during and after installation. The effects on the stability of the soil of possible future construction or natural movement of the sea floor materials should also be considered.

The effect of wave loads on the sea floor should be included in the analysis when necessary.

If the structure is located in a seismic region, the effects of seismic loads on the stability of the soil should be considered (see Appendix B).

## 5.3- Scour

When wave action and normal currents at the sea floor may combine to produce water velocities around the structure of such a magnitude that scouring of the sea floor will take place, the effect of this scour around or in the vicinity of the foundation should be considered and, where necessary, steps taken to prevent or check its occurrence.

## 5.4- Design of mat foundations

**5.4.1** *General-* The mat foundation of a gravity structure resting directly on the sea floor should be designed for adequate strength and deflections which are not excessive for the operation of the structure. The effects of the cyclic nature of wave loads and seismic loads and the potential liquefaction or softening that could accompany such loads should be considered in the design.

**5.4.2** *Bearing*

**5.4.2.1** *Loading combinations.* The load combinations in Section 4.4.1.1 with recommended multipliers should be investigated to identify critical forces acting on the foundation.

**5.4.2.2** *Safety factors.* The foundation must provide an adequate margin of safety against bearing capacity failure and sliding under the most critical combination of loads. When stability is analyzed in terms of effective stresses, the cohesive component of soil shear strength should be divided by a material factor $\gamma_c$, and the frictional component should be divided by a material factor $\gamma_f$. When stability is analyzed in terms of total stresses, the undrained shear strength should be divided by the factor $\gamma_c$. Selecting coefficients that will achieve a desirable margin of safety depends upon the uniformity of the soil conditions and the consistency of the measured strength values. For good quality data and relatively uniform soil conditions, it is recommended that $\gamma_c$ be taken at 1.4 and $\gamma_f$ be taken at 1.2. If the soil conditions have been evaluated with a lower degree of certainty, the value of these coefficients should be higher. The effects of repeated loading should be included in the evaluation of stability by using reduced undrained soil strength values in total stress analysis, or by using increased pore pressures in effective stress analyses, as indicated by data from tests with repeated cyclic loads.

**5.4.2.3** *Conditions to be considered.* The design should be based on fully drained, or undrained conditions, depending on which analysis best represents the actual conditions. An analysis for the undrained condition may be carried out on a total stress basis using undrained shear strengths, or on an effective stress basis using pore pressure parameters obtained from appropriate tests.

If the shear strength of the soil in the undrained condition is shown to be higher than the corresponding strength in the drained condition, the latter may be used in lieu of a more realistic analysis.

For clays, repeated shear stress applications during a storm may cause a reduction of the shear strength. Consolidation or swelling between storms may also change the shear strength properties of the clay. Based on test results or adequate previous experience, these effects are to be included in the design.

For frictional soils, repeated shear stress applications during a storm may lead to a gradual increase in pore water pressure which causes a reduction or possibly a complete loss of shear strength (liquefaction). On the basis of tests on the actual soil or relevant experience with similar soils, such effects should be accounted for in the design.

If the geometry of the structure or the soil conditions are complex, alternative failure modes should be investigated either by means of theoretical analysis or by model tests.

For structures where penetrating walls or skirts transfer loads to the foundation soil, additional analyses of the bearing capacity and resistance to lateral loading of the walls or skirts should be made.

**5.4.3** *Hydraulic stability--* For the conditions during both installation and operation of the structure there should be no undue risk of hydraulic instability. The following conditions should be investigated, including the effects of repeated loadings:

(a) Softening of the soil and reduction of bearing capacity due to seepage forces

(b) Formation of piping channels with accompanying internal erosion of the soil

(c) Surface erosion in local open areas under the foundation due to deformations caused by environmental loads

**5.4.4** *Foundation deformation and vibrations-* Movements and settlements of the structure caused by deformations of the foundations should not limit normal operation of the structure.

The elastic and inelastic strains of the soil under loads should be considered and the nonlinear properties of the soil taken into account.

**5.4.5** *Soil reaction on base of structure* -The reaction of the soil against all structural members seated on or penetrating into the sea floor should be included in the design load for the members.

The distribution of soil reactions should be in accordance with the results of the sea floor survey, considering the deviations from a plane surface, the force-deformation properties of the soil, and the geometry of the base of the structure.

The possibility of hard points produced by sand or gravel deposits should be considered in the design of the foundation. Ice gouges filled in by weak material can affect global soil-structure behavior and should also be considered in design.

Both installation and operating conditions should be considered.

# CHAPTER 6- CONSTRUCTION, INSTALLATION, AND RELOCATION

## 6.1-General

**6.1.1** *Construction stages*-For the types of concrete structures covered by this report, as much as possible of the construction work is normally performed away from the permanent site in a protected location or near the shore. For the purposes of this document, construction is assumed to take place in the following stages:

(a) First-stage construction in a fabrication area with the structure, initially at least, in the dry

(b) Initial flotation of the partially completed structure and towing offshore. Alternatively, the structure may be lifted by heavy marine lifts or floating cranes and towed offshore on barges or suspended from the floating cranes

(c) Further stages of construction with the structure afloat, or temporarily grounded, in a protected location near the shore

(d) Towing of the structure to its permanent location

(e) Installation

(f) Final construction in situ to complete the structure

**6.1.2** *Construction methods and workmanship*- Construction methods and workmanship should follow accepted practices as described in ACI 318, API RP2A, and the specialist literature. In general, only additional recommendations specially relevant to concrete sea structures are included here. At no time should the procedures or methods adopted decrease the safety of the structure or lead to difficulties during later stages of construction and installation. The design should be checked to insure that bollards, areas of outer walls which will be pushed by tugs and parts of the structure which will be exposed to severe dynamic forces during later stages of construction, are strong enough for their intended purpose.

**6.1.3** *Solid ballast*- Solid ballast in the form of rock, sand, or iron ore may be used to lower the center of gravity during construction and tow, and to provide greater weight for stability on the seafloor in service. Effects of temperature and moisture content should be considered when using solid ballast subjected to freezing conditions.

**6.1.4** *Construction and installation manual*--A construction and installation manual should describe all critical operations during construction, towing, and installation.

## 6.2-- Buoyancy and floating stability

**6.2.1** *Tolerances and control*- Tolerances for the buoyancy and the stability of the structure afloat should be set with due regard to the safety of the structure during all stages of construction and installation. In setting these tolerances, attention should be given to the following factors which might affect the center of gravity, draft and metacenter of the structure.

(a) The unit weight of the concrete in the dry

(b) The variation with time of the absorption of water by the concrete, with due allowance for pressure gradients which could occur during all stages of construction

(c) Accuracy of dimensions, in particular the thicknesses of walls and slabs

(d) Control of overall configuration, particularly radii of curvature of cylinders and domes and the prevention of distortion during casting

(e) The weight and weight distribution of any permanent or temporary ballast construction equipment and material

(f) The proper functioning of the system provided to vary the ballast when floating and sinking, including the control of effective free water planes inside the structure

(g) Any loads added during construction

(h) Specific gravity of water, including variations caused by tidal and tributary sources, at construction and installation sites

**6.2.2** *Temporary buoyancy tanks*

**6.2.2.1** Buoyancy tanks with an atmospheric pressure interior should be designed to withstand the maximum credible external pressure. The maximum credible external pressure should include accident conditions where the structure might sink deeper than actually planned. Provisions for internal pressurization can be made to increase safety against collapse (see Section 4.5.2).

**6.2.2.2** Net buoyancy is influenced by many parameters that need to be considered in the design. These parameters should include the following:

(a) Change in volume of pressure-resistant structure with depth

(b) Change in volume of structure with depth due to the bulk modulus effect

(c) Change in the specific gravity of seawater with depth

(d) Change in the specific gravity of buoyancy fluid or gas with depth

(e) Changes in the structural volume and the specific gravity of the buoyancy fluid or gas due to temperature changes

**6.2.2.3** Temporary tanks must be connected to the structure with adequate strength and support so as to remain in the proper attitude, resist low cycle fatigue, and withstand construction impacts. The release of temporary tanks must be carefully planned, and should preferably be done at a slightly negative buoyancy.

## 6.3- Constructlon joints

**6.3.1** *Preparation*-Construction joints should be prepared with extra care wherever the structure is to remain watertight or is designed to contain oil. This applies whether the watertightness is required permanently or only temporarily, such as during towing and installation. Suggested precautions to be taken when watertight construction joints are required include the following:

(a) Careful preparation of the surface by heavy wet abrasive blasting or high-pressure water jet to remove laitance and to expose the coarse aggregate. The maximum size aggregate should be exposed to about 25 percent of its normal diameter

(b) Use of an epoxide-resin bonding compound sprayed on just before concreting

(c) Increasing the cement content of the concrete at the start of the next placement

## 6.4- Concreting in hot or cold weather

Concreting in hot or cold weather should follow the guidance of "Cold Weather Concreting"-ACI 306R or "Hot Weather Concreting"-ACI 305R except that the use of calcium chloride as an accelerating admixture for cold weather is prohibited.

## 6.5- Curing of concrete

Special attention should be given to the curing of concrete to insure maximum durability and to minimize cracking. Seawater should not be used for curing reinforced or prestressed concrete although, if demanded by the construction program, concrete may be submerged in seawater provided that it has gained sufficient strength to avoid physical damage from waves, etc. When there is doubt about the ability to keep concrete surfaces permanently wet for the whole of the curing period, a heavy-duty membrane curing compound or curing mat cover should be used.

Heat generation caused by hydration of the cement should be evaluated for thick concrete sections to control cracking under various conditions of volume change and restraint. ACI 207.lR, "Mass Concrete for Dams and Other Massive Structures," contains guidance on materials and practices employed in proportioning, mixing, placing, and curing mass concrete. Guidance on effects of restraint and volume change is contained in ACI 207.2R. "Effect of Restraint, Volume Change, and Reinforcement on Cracking of Massive Concrete."

Thermal gradients may be minimized by either insulating formwork and concrete surfaces to control heat loss from the section or by uniformly extracting heat from the section with cooling water conduits. Either method should be used until internal temperatures have stabilized at acceptable levels.

## 6.6- Reinforcement

The reinforcement should be free from loose rust, grease, oil, deposits of salt or any other material likely to affect the durability or bond of the reinforcement. The specified cover to the reinforcement should be maintained accurately. Special care should be taken in the cutting, bending, and fixing of reinforcement, to insure that it is correctly positioned and rigidly held, so as to prevent displacement during concreting. The reinforcement should be protected against weld spatter and arcs due to strikes or current drainage.

## 6.7- PrestressIng tendons, ducts, and grouting

**6.7.1** *General-* This section deals, in the main, only with those requirements of prestressed concrete which are special to sea structures. Further guidance on prestressing steels, sheathing, grouts, and the procedure to be taken when storing, making up, positioning, tensioning, and grouting tendons will be found in relevant sections of ACI 318, of the Prestressed Concrete Institute and the Post-Tensioning Institute publications, FIP recommended practices, and the specialist literature.

**6.7.2** *Tendons-* All steel for prestressing tendons should be clean and free from grease, insoluble oil, deposits of salt, or any other material likely to affect the durability or bond of the tendons. However, protection by water-soluble oil is the preferred method.

**6.7.2.1** During storage, prestressing tendons should be kept clear of the ground and protected from weather, moisture from the ground, sea spray, and mist. No welding, flame cutting, or similar operations should be carried out on or adjacent to prestressing tendons under any circumstances where the temperature of the tendons could be raised or weld splatter could reach them.

**6.7.2.2** Where protective wrappings or coatings are used on prestressing tendons, these should be chemically neutral and should not produce chemical or electro-chemical corrosive attack on the tendons.

**6.7.3** *Ducts-* Metal post-tensioning ducts should be stored clear of the ground and protected from the weather, moisture from the ground, sea spray, and mist.

**6.7.3.1** All ducts should be watertight and all splices carefully taped to prevent the ingress of water, grout, or concrete. During construction, the ends of ducts should be capped and sealed to prevent the entry of seawater. Ducts may be protected from excessive rust by the use of chemically neutral protective agents such as vapor phase inhibitor powder.

**6.7.3.2** Where ducts are to be grouted, all oil or similar material used for internal protection of the sheathing should be removed before grouting. However, water-soluble oil used internally in the ducts or on the tendons may be left on, to be removed by the initial portion of the grout.

**6.7.3.3** Air vents should be provided at all crests in the duct profile. Threaded grout entries, which permit the use of a screwed connector from the grout pump, may be used.

**6.7.4** *Grouting-* For long vertical tendons, the grout mixes, admixtures, and grouting procedures should be checked to insure that no water is trapped at the upper end of the tendon due to excessive bleeding or other causes. Suitable admixtures known to have no injurious effects on the metal or concrete may be used for grouting to increase workability and to reduce bleeding and shrinkage. General guidance on grouting will be found in specialist literature. Holes left by unused ducts, or by removal of climbing rods of slipforms should be grouted in the same manner as described above.

Air entrainment should be considered for freeze-thaw resistance of grout used in low temperature or cold climate applications.

## 6.8- Initial flotation

Launching of the structure should be carried out in such a way that the structure is not subjected to excessive forces, taking into account the position of the ballast at the time of the flotation and that, at this stage, the structure may be incomplete and the concrete still young.

**6.8.l-** If the structure is to be lifted by lifting devices, the stresses in the concrete in the vicinity of the lifting points should not exceed the permissible limits. Analysis should be performed for the lifting mode of operation to assure that tensile stresses in the concrete, prestressed or otherwise, do not exceed the cracking limit. Lifting accelerations should be minimized to limit dynamic tensile stresses in the lifting lines and in concrete. Lifting attachments or embedments should be designed for at least 100 percent dynamic amplification.

**6.8.2-** When an air cushion is used beneath the structure to reduce draft during early construction and towing stages, the effects of bending forces and accidental loss of air should be considered. Suitable instrumentation should be installed to permit control of the air pressure and also to indicate the water level in each underbase compartment.

## 6.9- Construction while afloat or temporarily

**6.9.1-** When further construction takes place while the structure is afloat, the rate of concreting should be adequately coordinated with the rate at which the structure is submerged below water level so as to avoid overstressing of the concrete. Calculations should be made of stress changes during concreting and submergence so as to avoid excessive bending stresses in the horizontal plane, and local overstresses from bending and circumferential compression due to the increasing hydrostatic pressure.

**6.9.2-** If the structure is temporarily grounded, the shape of the seabed on which it is placed should be within acceptable tolerances having regard for the strength of the concrete at the time. When grounding and subsequently lifting the structure, considerations similar to those given in Chapter 7 will apply.

## 6.10- Towing

**6.10.1** *Strength of the structure*- All aspects of towing should be designed and planned to insure that the structure is not exposed to loadings greater than those for which it was designed (see Chapter 3). Hydrostatic loading of the structure should be given particular attention because this loading condition is severe and can produce castastrophic failure.

**6.10.1.1** Fatigue of permanent or temporary steelwork, even after the relatively few stress cycles occurring during a tow, may be a serious consideration in a corrosive environment.

**6.10.2** *Response to motion*- The response of the structure to motion in all directions of freedom should be determined for the structure in the towing condition. These responses should be verified by all possible means. Model tests may be desirable, particularly for unusual structures. Where the shape of the structure makes it sensitive to dynamic uplift, nosediving, yaw, etc., the hydrodynamic control of the structure under tow should be adequate to minimize these effects. Checks should be made to insure that the motions of the structure in the maximum extreme environmental conditions during the tow do not result in unacceptable stresses or increase in draft.

The effect of structure accelerations during tow on equipment must be evaluated. Tie downs of equipment, etc., should be adequate to resist these forces, taking into account elongation and fatigue in a corrosive environment.

**6.10.3** *Towing connections and attachments*- An adequate number of towing connections, suitably placed, should be fitted to the structure.

**6.10.3.1** Towing line attachments should be suitably designed so as to insure that any possible failure will occur in the line. Consideration should be given to factors such as maximum static breaking strength and strain rate strength increases in the tow line. Traditionally, attachments have been designed to develop at least twice the breaking strength of the line.

**6.10.4** *Damage stability* - Compartmentation and damage control should be considered to insure stability and buoyancy in event of accidental flooding.

## 6.11—Installation

**6.11.1** *General*- All aspects of the installation of the structure, including its sinking and placing on the seabed, should be planned and carried out with the greatest care. The arrangements made for installation should insure that the structure is placed in position within the given tolerances.

**6.11.2** *Condition of the seabed*- Planning of the installation should take into account the conditions of the soil at the site, including its hardness and its susceptibility to scour and suction or breakout effects. The topography of the seabed should be checked, attention being given to its slope, unevenness, and the occurrence of boulders. The topography should be tied-in accurately to horizontal survey controls by seabed transponders or other means. Planning should also take into account the possibility of erosion of the seabed due to the horizontal flow of water from beneath the structure as it nears the bottom.

**6.11.3** *Preparation of the seabed*- It may be necessary to prepare the seabed prior to platform installation. Such preparation may consist of removal of rocks, dredging, leveling and trimming, provision of a screeded rock base, prior overload by surcharging, or other means. Screeded rock base should be placed with adequate control of position and tolerances, They should be protected from erosion or silt deposits during the period before the structure is placed or provision made for removal of silt or sand deposits.

**6.11.4** *Installation techniques*- Depending on the soil conditions, one or more of the following techniques may be used to achieve satisfactory installation of the structure on the seabed:

(a) If penetration is desired, overload may be provided by adding ballast. Reducing the water pressure directly underneath the structure may also assist penetration, but soil pore pressures should be controlled with extreme care to prevent the development of a "quick" condition or "boil" in the foundation soil.

(b) Underbase grout may be used to fill voids between the base of the structure and the sea floor after the structure is installed. Provision shall be made for venting the trapped water. The grout should have satisfactory properties of long term stability, strength, modulus of elasticity, minimum bleed, acceptable temperature rise during hydration, and flowability and cohesion (lack of segregation) when placed in seawater. These properties should be verified by test.

(c) Sand may be used under the structure as a means of providing uniform bearing under the base. The periphery must be adequately sealed or cutoff to prevent erosion of the sand.

**6.11.4.1** The effects of methods used for installation on hydraulic stability of the soil are to be investigated as stated in Section 5.4.3.

## 6.12- Construction on site

Work on site should be executed according to accepted engineering practices and as required by relevant portions of ACI 318 and API RP2A.

## 6.13- Connection of adjoining structures

Where structures are to be positioned in close proximity to each other, impact should be prevented by suitable fenders. Consideration should be given to uncontrolled surge due to trapping of water under the caisson base. Connections should be designed to accommodate mismatch due to displacements and rotations in the six degrees of freedom, while preventing any undesirable damage.

## 6.14- Prevention of damage due to freezing

Structural members, materials, equipment, and components should be protected from damage due to internal freezing of contained water in cold climates. In general, ballast systems and fluid containment compartments should not be allowed to freeze unless means are incorporated into the design to accommodate the effects of freezing. Openings capable of trapping water, such as small construction holes, cracks, construction joints, and prestress tendon ducts, should be grouted or sealed to prevent damage from the expansive force of freezing water.

## 6.15- Relocation

**6.15.1** *General*- The relocation of a structure requires careful consideration during the structure's design to insure safety and structure integrity during the relocation operation. The initial phase of the operation involves breaking of the suction bond of the base on the soil, extraction of any penetrating skirts and dowels, removal of any solid ballast, and refloatation while maintaining stability. Subsequent phases involving transportation and reinstallation are provided for in Sections 6.10 and 6.11.

**6.15.2** *Suction bond*- The suction bond may generally best be broken by sustained waterflooding underneath the structure, at a pressure less than the shear strength of the soil. Pressures higher than the shear strength of the soil will cause piping channels to the outside sea and should therefore be avoided. The suction force may also be broken by eccentric deballasting. Care must be exercised to prevent damage to skirts and other protruding elements if the structure is to be relocated.

**6.15.3** *Skirt extraction*- The extraction of skirts and dowels will usually be accomplished by deballasting of the structure, using the buoyant force to overcome the long-term shear adhesion. Where physical arrangements or access permits, this adhesion may be reduced by jetting, vibration, or jacking free of individual elements.

**6.15.4** *Solid ballast removal*- Removal of solid ballast may be by sand pump or airlift. Rock may be removed mechanically or jettisoned by release of retaining walls.

**6.15.5** *Stability*- Refloatation is accomplished by deballasting. However, the extraction of the dowels and skirts and final breaking loose may cause a sudden rise to a lesser draft and may endanger stability. To insure stability, topside equipment may first be removed. The structure should be deballasted slowly to prevent sudden uncontrolled rise and dynamic heave amplification.

# CHAPTER 7- INSPECTION AND REPAIR

## 7.1- General

**7.1.1** *Concrete*- Concrete is a durable structural material. Where concrete structures have been well designed and then constructed to a high standard of workmanship with good materials, little repair work has been required. Nevertheless, repairs sometimes have to be carried out for a variety of reasons. Faulty construction or poor materials may lead to deterioration of the concrete or corrosion of the reinforcement and the structure may be damaged by overload, impact, abrasion, or fire.

**7.1.2** *Damage*- If the structure is to be subjected to repairs of any significance, or if the weight or position of the equipment supported by the structure is altered necessitating a reinforcement of the structure, or if the structure is otherwise to be converted or altered influencing structural integrity, then all construction should be carried out in accordance with the recommendations for new construction as far as is practicable.

## 7.2- Surveys

The structure should be surveyed annually for damage or deterioration. Particular attention should be given to those parts of the structure exposed or subjected to fatigue loading, alternate wetting and drying, and previous repairs or modifications of the structure. Surveys should be carefully reviewed every 5 years and should cover the following:

(a) Visual inspection of the general conditions
(b) Concrete deterioration or cracking
(c) Condition and function of corrosion protection system, if any
(d) Condition of exposed metal components (fixing plates, risers, pipelines, etc.)
(e) Condition of foundation and of scour protection system
(f) Amount of marine growth and the presence of debris

In the event of accident, discovery of damages of deterioration, modification or any other noted or possible change in the condition, or operation of the structure that may affect its short term safety, a special survey may be required.

**7.2.1-** The design and equipment of the structure should include provision for periodic inspections. Such provision should include, as appropriate, permanent reference points or marks for underwater locations to facilitate proper location of structural elements.

**7.2.2** -Visual signs which indicate the need for future surveillance or repair are the appearance of rust strains on the concrete surface, cracking or splitting of the concrete, spalling or erosion, or damage due to accidental impact. It may then be possible to prevent or reduce further corrosion by applying a sealing coating to the concrete surface. Corrosion of the reinforcement will then be controlled by the efficiency of the sealing coat and its maintenance.

## 7.3- Repalr of concrete

**7.3.1** *General*- Methods of repair of concrete sea structures should follow generally accepted practices (such as ACI 201.2R, API RP2A, etc.) and the instructions provided by the manufacturers of the materials being used. Other methods may be used on approval. Sea structures pose special problems of access and working conditions created by the environment. The methods chosen should enable adequate protection to be given to the work and the workmen so that a high standard of workmanship may be achieved. Materials should be carefully chosen to be compatible with the conditions prevailing during and after their application. The following recommendations cite standard methods and materials for repairs. It is not intended that the use of other methods and materials should be inhibited, provided that they can be shown to be satisfactory.

**7.3.2** *Resins*- Where resin materials are used, they should be of a moisture resistant type which does not lose its efficiency in a damp or wet environment. They should be of a formulation suitable for the particular application (e.g., damp concrete, low temperature, etc.) and the manufacturer's instructions should be strictly followed.

Epoxy and polyester resins generally will be found to be most suitable for the repair of sea structures. Apart from the manufacturer's instructions, general guidance on their use may be found in specialist literature.

**7.3.3** *Cement*-The cement for repair shall conform to ASTM C 150 and to the additional requirements of Section 2.5, and shall be similar to that used in the original construction.

### 7.4- Repairs of cracks

**7.4.1-** Before a crack is repaired, its cause should be determined so that the appropriate method of repair may be chosen. The chosen method will also depend on the zone in which the crack occurs (see Section 2.4.2).

**7.4.2-** Where corrosion of the reinforcement has spalled the surrounding concrete, the damaged concrete should be removed and the repair made.

**7.4.3-** If a narrow [crack less than 0.01 in. (0.25 mm)] has only to be sealed against the ingress of moisture and no further movement of the crack is expected, a low-viscosity epoxide resin should be used. The crack is then filled by repeatedly painting the crack until it absorbs no further resin, by running the resin into the crack under the action of gravity or by injecting the resin into the crack under pressure.

**7.4.4-** For wider cracks and when continuing movement of the crack is expected, a chase should be cut along the line of the crack and this sealed with an elastic material such as a polysulphide rubber, or by the insertion of a prepared neoprene or rubber-bitumen sealing strip or by application of an epoxy sealant. Alternatively, a flexible cover strip may be fixed to the surface of the concrete. Then the crack should be injected with an epoxy specially formulated for the purpose and moisture conditions. The epoxy should be injected successively through closely spaced ports so as to force out any water ahead of the epoxy. Care must be taken to thoroughly flush the initial emulsified interface. Injection pressures must be kept sufficiently low to avoid hydraulic "fracturing" extension of the crack.

## CHAPTER 8- REFERENCES

### 8.1- Standards and reports

The documents of the various standards-producing organizations referred to in this document are listed below with their serial designation, including year of adoption or revisions. The documents listed were the latest effort at the time this document was revised. Since some of these documents are revised frequently, generally in minor detail only, the user of this document should check directly with the sponsoring group if it is desired to refer to the latest revision.


*American Concrete Institute*

| | |
|---|---|
| 201.2R-77 (Reaffirmed 1982) | Guide to Durable Concrete |
| 207.1R-70 (Reaffirmed 1980) | Mass Concrete for Dams and Other Massive Structures |
| 207.4R-80 | Cooling and Insulating Systems for Mass Concrete |
| 213R-79 (Reaffirmed 1984) | Guide for Structural Lightweight Aggregate Concrete |
| 305R-77 (Reaffirmed 1982) | Hot Weather Concreting |
| 306R-78 (Reaffirmed 1983) | Cold Weather Concreting |
| 318-83 | Building Code Requirements for Reinforced Concrete |
| 408.1R-79 (Reaffirmed 1984) | Suggested Development, Splice, and Standard Hook Provisions for Deformed Bars in Tension |
| 503R-80 | Use of Epoxy Compounds with Concrete |

*American Petroleum Institute*

RP 2A, "API Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms," January, 1982, 13th Edition.

*ASTM*

| | |
|---|---|
| A 706-82a | Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement |
| C 33-82 | Specification for Concrete Aggregates |
| C 150-83a | Specification for Portland Cement |
| C 330-82a | Specification for Lightweight Aggregates for Structural Concrete |
| C 595-83 | Specifications for Blended Hydraulic Cements |
| C 618-83 | Specification for Fly Ash and Raw or Calcined Natural Pozzolan for use as a Mineral Admixture in Portland Cement Concrete |
| C 88l-78(83) | Specification for Epoxy-Resin-Base Bonding Systems for Concrete |
| D 512-81 | Test Methods for Chloride Ion in Water and Waste Water |

*American Welding Society*

| | |
|---|---|
| D1.l-83 | Structural Welding Code-Steel |
| D1.4-79 | Structural Welding Code-Reinforcing Steel |

*Det Norske Veritas*

"Rules for the Design, Construction, and Inspection of Offshore Structures," 1977.

***Fédération Internationale de la Précontrainte***

"Recommendations for The Design and Construction of Concrete Sea Structures," 1977, 3rd Edition.


The above publications may be obtained from the following organizations:

American Concrete Institute
P. O. Box 19150
Detroit, MI 48219

American Petroleum Institute
2101 L. Street, N. W.
Washington, DC 20037

ASTM
1916 Race Street
Philadelphia, PA 19103

American Welding Society
2501 N.W. 7th Street
Miami, FL 33125

Det Norske Veritas
Veritasveien 1
1322 Høvik
NORWAY

Federation Intemationale de la Précontrainte
Wexham Springs
Slough, SL3 6PL
ENGLAND

## APPENDIX A-ENVIRONMENTAL LOADS

### A.1- Introduction

The purpose of this appendix is to identify the present state-of-the-art evaluation of environmental loads applicable to offshore concrete structures. No attempt has been made to explain the theoretical or empirical derivations of concepts discussed. However, relevant references have been provided wherever possible. It is strongly suggested that the user of this report familiarize himself with the appropriate references before applying the concepts suggested herein.

### A.2- Wave loads

Loads on offshore structures are generally predicted by a semi-empirical approach. The derivatives of a theoretically derived flow potential function are combined with empirical drag and inertial coefficients to predict wave forces on structural components relative to the position of the wave. By considering the proper phase angles between the two force components, the maximum wave load may be predicted.

A background on the selection of applicable flow potential functions is provided in Reference 1. Water depth, wave height, and wave period in a specific geographic region are the most important parameters for the selection of an appropriate flow potential function.

Drag and inertial coefficients are experimentally derived. A range of values is suggested in Reference 2. Morison's equation is most frequently used to determine wave forces on cylindrical, structural members when the diameter to wave length ratio is sufficiently small so as not to cause significant wave scatter. A suggested range of applicability of Morison's equation is $D/\lambda \leq 0.2$, where $D$ is the member's diameter and $\lambda$ is the wave length.

With increasing member sizes the inertial forces become dominant. This also influences the selection of the inertial coefficient $C_m$. Reference 3, Fig. 12-47 provides $C_m$ values as a function of $D/\lambda$. Rapidly decreasing values for $C_m$ reflect that wave scatter is becoming significant.

### A.3- Wave diffraction

Wave diffraction occurs when the size of an immersed structure presents a major obstacle in the path of a propagation wave. For large concrete gravity structures the effects of wave diffraction must be an essential element of wave force calculations.

Numerical methods have been developed by Garrison[4] and Hogben[5] to calculate pressure distributions for caissons of arbitrary geometry. For circular, cylindrical structures approximate methods by Gran[6] and McCamy[7] may be applicable.

Under extreme design wave conditions, a long wave may cause the relative size $(D/\lambda)$ of large cylindrical columns to be sufficiently small so that diffraction may not be significant. In such cases wave forces may be adequately determined by Morison's equation. The same columns may, however, experience wave diffraction for milder sea states. A general guideline for the regime in which diffraction becomes significant is $D/\lambda > 0.2$.

The fluid motion field in the vicinity of caisson-column junctions may require modification to account for the caisson blockage. Presently, the blockage effect can best be evaluated by numerical techniques.[6]

Linear diffraction analysis methods are not generally applicable for calculating wave forces on shallow-water structures. Problems arise in the analysis of near-breaking waves and structures with rapid variations in geometry near the water line. Model tests may be used, pending development and verification of suitable nonlinear diffraction analysis methods.

### A-4- Currents

Since concrete structures are expected to have large members, inertial forces will be dominant. These forces are proportional to the fluid particle acceleration and are not significantly affected by a constant current. However, currents may be important for the design of auxiliary equipment, such as exposed risers and well conductors. Such components should also be investigated for vortex shedding. An excellent treatment of this phenomenon may be found in Reference 8.

The guidelines presented in Reference 2, Paragraph 2.7b and c, are helpful in defining a current profile in the absence of detailed oceanographic data.

### A.5- Design wave analysis

Static analysis generally provides adequate accuracy when designing a structure against extreme waves of 50 or 100 year recurrence periods, provided that the guidelines of Section A.7 are satisfied. Maximum forces on caisson structures are very sensitive to the wave period selected. Generally, the extreme wave period is hindcast and then adjusted to maximize the wave force for a given structure.

### A.6- Wave response spectrum analysis

The inertial dominance of wave forces on concrete structures makes the wave spectrum approach a suitable method for predicting maximum wave force responses. Kinsman[9] discusses the principles of wave spectra and Crandall[10] develops the requirements for a spectrum analysis.

Examples of wave spectra are the Bretschneider,[9] Scott-Wiegel, and Jonswap spectra. A competent oceanographer should properly define the wave spectrum for a given location.

A linear relationship between wave heights and wave forces for a given period is essential for the valid application of the spectral method. Linear analysis applied to an offshore platform has been reported in Reference 11.

A serious shortcoming of calculating wave forces by spectrum analysis is the loss of phase relationships between the various frequency components. Approximate phase relationships have been suggested in Reference 12. Directional wave spectra may be used.

### A.7- Dynamic response analysis

Before selecting an appropriate wave response analysis model for a given structure, a preliminary dynamic assessment of the structure should be performed.[13] If significant wave force amplifications are expected, a dynamic analysis should be considered. Such an analysis may either be deterministic or spectral. Both techniques have been discussed extensively in the literature.[14,15] If the supporting soil is modeled by springs and dashpots, as opposed to more rigorous modeling techniques, the sensitivity of the soil stiffness must be carefully investigated as it affects the dynamic response of the structure. As a general guideline, a dynamic response analysis should be considered if the fundamental period of the structure exceeds 2.5 sec. This guideline may not be sufficient if a fatigue analysis is under consideration.

The dynamic analysis should be based on an accurate model of the deck when it contributes significantly to the structure stiffness. The deck design itself must be coordinated with the substructure design since the deck stresses are very dependent on substructure interaction.

### A.8- Wind loads

Reference 2, Paragraph 2.6, provides minimum design guidance for wind loading. An extensive investigation of wind forces on structures may be found in Reference 16.

### A.9- Ice loads

The process of predicting ice loads on fixed bottom offshore structures requires meteorological data on the presence of ice for a specific region coupled with information identifying the different modes of ice formation to be expected (i.e., first year ice, multiyear ice, consolidated and unconsolidated ice ridges, ice islands, and icebergs). From this, the extreme environmental event may be estimated which an offshore structure must resist.

Reference 17, Paragraph 4.3, provides useful design guidance for ice loading. Reference 17 also contains an extensive bibliography of relevant literature sources that deal with major aspects of design considerations of structures intended for arctic applications.

### A.10- Earthquakes

See Appendix B.

### A.11- References


1. Ippen, A. T., *Estuary and Coastline Hydrodynamics,* McGraw-Hill Book Co., New York, 1968, 650 pp.

2. *API Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms,* API RP2A, 8th Edition, American Petroleum Institute, Washington, D. C., 1977, 49 pp.

3. Myers, John J., Editor, *Handbook of Ocean and Underwater Engineering,* McGraw-Hill Book Co., New York, 1969, 1972 pp.

4. Garrison, C. J., et al., "Wave Forces on Large Volume Structures-A Comparison Between Theory and Model Tests," *OTC Paper No.* 2137, Offshore Technology Conference, Houston, May 1974, pp. 1061-1070.

5. Hogben, N., and Standing, R. G., "Wave Load on Large Bodies," International Symposium on the Dynamics of Marine Vehicles and Structures in Waves (University College, London, Apr. l-5, 1974), Mechanical Engineering Publications Ltd., London, 1975, pp. 258-227.

6. Gran, S., "Wave Forces on Submerged Cylinders," *OTC Paper* No. 1817, Offshore Technology Conference, Houston, May 1973, pp. 1801-1812.

7. McCamy, R. C., and Fuchs, R. A., "Wave Forces on Piles: A Diffraction Theory," *Technical Memo NO.* 69, Beach Erosion Board, U. S. Army Corps of Engineers, Washington, D. C., Dec. 1954.

8. "Oscillation of Piles in Marine Structures," *Technical Note* No. 40, Construction Industry Research and Information Association, London, 1972.

9. Kinsman, Blair, *Wind Waves,* Prentice-Hall, Inc., Englewood Cliffs, 1965, 676 pp.

10. Crandall, S. H., and Mark, W. D., *Random Vibrations in Mechanical Systems,* Academic Press, New York, 1963, 166 pp.

11. Malhotra, Anil, and Penzien, Joseph, "Nondeterministic Analysis of Offshore Structures," *Proceedings,* ASCE, V. 96, EM6, Dec. 1970, p. 985.

12. *Hydrodynamic Forces on Gravity-type Platforms,* Delft Hydraulics Laboratory, Mar. 1974.

13. Wirsching, Paul H., and Prasthofer, Peter H., "Preliminary Dynamic Assessment of Deepwater Platforms," *Proceedings,* ASCE, V. 102, ST7, July 1976, pp. 1447-1462.

14. Clough, Ray W., and Penzien, Joseph, *Dynamics of Structures,* McGraw-Hill Book Co., New York, 1975, 634 pp.

15. Maddox, N. R., "Fatigue Analysis for Deepwater Fixed-Bottom Platforms," *OTC Paper* No. 2051, Offshore Technology Conference, Houston, May 1974, pp. 191-203.

16. "Wind Forces on Structures," *Transactions,* ASCE, V. 126, Part II, 1961, pp. 1124-1198.

17. "Bulletin on Planning, Designing, and Constructing Fixed Offshore Structures in Ice Environments," *API Bulletin* 2N, American Petroleum Institute, Washington, D.C., 1982, 49 pp.


## APPENDIX B-DESIGN FOR EARTHQUAKES

### B.1- Introduction

Experience has shown that reinforced concrete structures can be designed to withstand severe earthquakes. The development of major buildings and nuclear power plants has resulted in the creation of substantial aseismic analysis and design methodology for structures with massive components. With this background it is reasonable to expect that suitable offshore concrete structures can be designed for seismically active regions. Research into the subject has suggested that feasible structures can be designed by borrowing from existing design methodology but that there are significant differences between the global and local characteristics of typical offshore and onshore structures. The earthquake resistant behavior of concrete sea structures has not as yet received the close attention which has been paid to typical offshore steel lattice structures. Consequently, it is considered premature to define procedures and criteria which are as detailed as the API RP2A provisions for steel structures.

This appendix has thus been prepared in the nature of a checklist to advise the designer of those problem areas which need to be addressed without making specific recommendations as to how this should be done. Several alternative methods are available for calculating earthquake loads and there is no general agreement as to which is best. The experience and judgment of the designer are considered to be as important as the particular method chosen.

Earthquakes cause ground shaking, submarine landslides, tsunamis, acoustic water waves and other hazards for offshore installations. Ground shaking is usually the most important phenomenon and it will be discussed in most detail. The loads induced by ground shaking are usually significantly different from those caused by other environmental factors such as waves, currents, or ice. In cases where the primary design constraints are imposed by these other factors, the designer must nevertheless check the effects of earthquakes on local parts of the structure-foundation system.

The most important difference between designing for earthquakes and for other natural phenomena such as waves is the greater uncertainty in predicting the characteristics of the earthquake event. In the case of phenomena such as wind and waves their extreme events usually have physical upper bounds likely to be adequately allowed for by the load and material factors chosen for the design event. This is not the situation for earthquakes and it is conceivable that the extreme earthquake could excite ground motions much larger than those for the design event. In the interests of achieving a safe but economic design, the recommended approach is to design the structure to respond without damage to a selected design event and then to check that this design will enable the structure to survive the conditions of the extreme earthquake, albeit with some damage. For certain types of structure whose general form and components are fairly standardized, sufficient work has been done to enable the survivability check to be replaced by general ductility requirements. The novelty and variety of concrete sea structures does not permit this simplification at this time, and this appendix therefore proposes a two stage design process, namely:

1. Design the structure and foundation for the design level earthquake, using a standard limit state approach.
2. Check that the structure-foundation system will endure the *survivability level* earthquake without collapse.

## B.2- Overall design procedure

The overall design procedure to insure a safe seismic structure can be considered in the following steps:

1. Seismicity study
2. Site response study
3. Selection of design criteria
4. Dynamic analysis
5. Stress analysis
6. Evaluation of failure modes
7. Satisfying ductility requirements
8. Development of aseismic design details

Steps 4 through 8 will usually go through several cycles in a typical project.

## B.3- Seismicity study

The characteristics of the design and survivability level earthquakes are specific to the planned installation location. They should be established from a study of the regional seismicity and of the local geology. The seismicity study should include an evaluation of the history of seismic events for the region, including the epicentral and focal distances from the installation location. An evaluation should be made of the regional and local geology to determine the location and alignment of faults, the source mechanism for energy release (thrust or slip), and the source to site attenuation characteristics.

The seismicity study should ideally result in rock motion parameters and magnitude recurrence statistics which permit the ground shaking characteristics for both the design and survivability level earthquakes to be selected.

## B.4- Site response studies

Allowance must be made for the effects of local soil conditions in amplifying or attenuating ground motions and in altering the frequency characteristics of the ground shaking. Free field response studies should be conducted to insure that the ground motions to be used as input for the dynamic interaction analyses are compatible with the soil conditions on the site. These analyses usually assume that the earthquake energy is transmitted vertically in the form of shear or body waves with in-phase excitation at all plan positions across the site. Shear beam or finite element models can be used, as appropriate. The nonlinear nature of the soil stress-strain behavior must be accounted for. This can be achieved by using a linear iterative technique in conjunction with strain dependent shear moduli and damping ratios, or through the use of a hyperbolic, Ramberg-Osgood or other nonlinear constitutive formulation.

Real ground shaking is truly three-dimensional with out-of-phase behavior across the site. The engineer should allow for this in interpreting the results of these simpler models.

## B.5- Selection of design criteria

**B.5.1** *General*- Earthquake design criteria should ideally be selected on a probabilistic basis. Unfortunately, there are no strong ground motion records for offshore locations and relatively few from onshore sites for high magnitude earthquakes. A semiprobabilistic approach is usually resorted to in which a design earthquake is selected and used in conjunction with appropriate partial safety factors on loads and materials. In view of the greater uncertainty associated with earthquake ground motions than with waves, it is sound practice to use a two tier criterion as follows:

1. *Design earthquake*- The structure and its foundation must be designed to withstand the effects of a design earthquake with no significant damage and be able to continue operation virtually without interruption
2. *Survivability level earthquake*- The structure-foundation system must be capable of *surviving* the conditions of a survivability level earthquake. The survivability of the combined structure-foundation system is a measure of its ability to endure the extreme event without the collapse of the platform structure or its foundation or any primary structural component. The structure may endure damage in one or more components which may or may not render it unfit for repair and operation after the survivability level earthquake.

**B.5.2** *Design earthquake*- The selection of the design earthquake must be based on an acceptable risk level for the operating life of the structure. For example, in the case of a 25-year operating life, the design earthquake might be associated with a return interval of about 100 years. The basic means of expressing the ground shaking is in the form of an accelerogram. Each earthquake possesses particular frequency, amplitude, and duration characteristics, all of which are influenced by the numerous factors involved in the seismicity and ground response studies described in Sections B.3 and B.4. It is essential to make allowance for variations in selecting accelerograms for the planned installation location. The resulting accelerograms can be expressed in the form of response spectra.

The differences between the shaking characteristics of events with the same nominal intensity are such as to demand a realistic treatment of these differences when selecting the design earthquake. Usually there is not enough information to treat the problem statistically. As a consequence, the designer customarily resorts to the use of a smoothed design spectrum. This obviates the problem of structural frequencies at or between the peaks and troughs in the response spectra of actual accelerograms. Such a design spectrum is thus a reflection of the seismicity and the site response as well as their respective variabilities. The so-called "design earthquake" must provide an adequate representation of this spectrum.

Representation of the design earthquake by scaling measured accelerograms will require the use of several records to insure that the structural behavior is adequately evaluated over the relevant frequency range of the smoothed spectrum. Alternatively, an artificial record can be generated from the design spectrum with a close fit over the required frequency band. The latter is a useful device when used with linear systems. However, when the problem is nonlinear (as is the case for the foundation, even for the design earthquake), the nonunique nature of the artificial record means that several artificial records may have to be generated to represent the range of duration and amplitude characteristics. In general therefore, while we use the term "design earthquake," what is really meant is the set of conditions typical of the design earthquake, with the understanding that these will frequently require several records in order to define them adequately.

**B.5.3** *Load and material factors for the design earthquake*-The values of the load and material factors must relate to the methods used to select the design earthquake and to calculate the structural or foundation loads, as well as the consequences of failure. The following are offered as initial guidelines, assuming that adequate seismicity, site response, and dynamic analyses are conducted. The smoothed undamped response spectrum should correspond to the mean plus one standard deviation over the relevant frequency band. The load and material factors for the design environmental event in Section 4.4.1.1 may be used and the $\gamma_L$ factor for the earthquake load should be 1.4. In the case where the design is governed by earthquakes, the selection of appropriate factors should be studied in some depth.

**B.5.4** *Survivability level earthquake*-The upper level criterion is applied to insure that the designed system can survive the extreme conditions of a very low probability event. Rather than require the system to be designed to respond without damage to such an event, it has become accepted practice to recognize that the criterion in this case should be survivability as defined in Section B.5.1. This definition assumes that the system may well be unserviceable after the event.

Recognizing that earthquake responses in the structure and foundation are displacement, and not load-controlled, the survivability can best be provided for by building sufficient ductility into the system. The amount required depends on the extreme ground motions assumed. There are no easy procedures for determining these. As a general guideline, it is recommended that the structure-foundation system be capable of withstanding distortions at least twice those caused by the design earthquake without risk of collapse. (The emphasis is on distortions rather than displacements since the latter implies the inclusion of rigid body motions which do not cause structural distress.) If this is associated with significant yielding in the structure, the survivability of the structure should be tested by means of a nonlinear dynamic analysis. The selection of appropriate inputs for this analysis should ideally form a part of the seismicity study in Section B.3. Where this is not done, base rock inputs having spectral ordinates at least twice those for the design earthquake should be used. The nonlinear structure and foundation should be simulated in this analysis and an adequate number of accelerograms must be used to assess the consequences of a realistic range of amplitude and duration characteristics.

## B.6- Dynamic analysis

The most important aspects of the dynamic analysis are, first, selection of a model and analysis method which describe the physics of the problem adequately and second, interpretation of the results by a person experienced in the analysis method and its limitations.

**B.6.1** *Structure-fluid interaction*- The dynamic interaction of structure and water can be described quite adequately for many structures by employing the added mass concept together with an allowance for fluid damping where appropriate. The importance of frequency dependent effects associated with the generation of free surface gravity waves is dependent on the problem geometry. For structures comprising a large submerged caisson with a limited number of slender legs, the free surface effects are usually quite small and are typically associated with a water depth equal to at most two or three leg diameters. For this situation, it is reasonable to treat the added mass contribution as constant, but allow some reduction in the coefficient near the free surface. The damping due to radiated energy is frequently ignored. This will not be appropriate in the case of large surface piercing structures, for which the analysis method should provide an adequate treatment of the free surface effects.

Added mass coefficients may be determined experimentally (Froude scaling) or analytically. Either the source distribution method or a finite element technique may be used for the analysis, provided that the latter gives an adequate simulation of the three dimensional nature of the problem and the farfield boundary conditions.

Drag damping is usually of secondary importance for large displacement structures. Where the designer desires to allow for it, the values of the drag coefficients used must be justified for the ranges of Reynolds and Keulegan-Carpenter numbers appropriate to the problem.

**B.6.2** *Soil-structure interaction*-A complete interaction analysis should (1) account for the variation of soil properties with depth, and possibly in plan, (2) give appropriate consideration to the nonlinear behavior of soil, (3) consider the three-dimensional nature of the problem, (4) consider the effect of nearby structures upon each other, and (5) consider the complex nature of wave propagation which produces the ground motions. A report in preparation by the Ad Hoc

Group on Soil-Structure Interaction, Nuclear Structures, and Materials Committee of the Structural Division of ASCE states that the relative importance of these problem features is still being investigated and that clear advice as to their importance cannot be given as yet. However, the virtues and weaknesses of the various methods can be delineated to enable the engineer to select a method appropriate to the problem at hand. It is emphasized that there is no single agreed best method.

There are two basic approaches to the problem, namely the direct analysis method and the superposition method.

**B.6.2.1** *Direct analysis method.* This is a one-step procedure in which the complete structure-foundation system is modelled, usually by means of finite elements. The soil nonlinearity may be represented by the use of strain-dependent modulii and damping coefficients in combination with an iterative analysis procedure. Such analyses are almost invariably done in two dimensions with some form of energy absorbing boundary used to simulate the far-field boundary condition.

**B.6.2.2** *Superposition method.* This is a two-step procedure in which the interactive characteristics between structure and foundation are evaluated and then incorporated in a dynamic analysis of the structure as a set of frequency dependent foundation impedances.

**B.6.2.3** *Limitations.* Either method can be used subject to some of the following limitations. The *2D* finite element direct method requires considerable care in the selection of the element mesh to preserve the required frequency characteristics as well as in the handling of the absorbing boundaries. Its chief limitations in the present context are that the procedure is neither truly nonlinear, nor is it three-dimensional. It has been shown that it is theoretically not possible to reproduce simultaneously the stiffness and damping characteristics of the *3D* problem by a *2D* model.

The superposition method can overcome the two-dimensional limitation by using three-dimensional impedances together with a two or three-dimensional model of the structure. The chief difficulty with the superposition approach is to derive impedance functions which adequately represent the nonlinear behavior of the soil when used in conjunction with a linear analysis method. Many other factors are involved and it is important to emphasize that the full limitations of applying the methods to the case of concrete offshore structures have been subjected to far less scrutiny than for nuclear reactors and no clear preference can be made at this time.

**B.6.2.4** *Modal analysis of spring-dashpot models.* Constant valued springs and dashpots are a special case of the frequency dependent impedances used in the superposition method. Due to its simplicity, this foundation characteristic is frequently used in conjunction with a modal analysis technique. This approach must be used with great caution for offshore structures in view of the problem of developing appropriate modal damping coefficients which provide an adequate representation of the very different levels of damping in structure and foundation. Also, the engineer should be aware that the response of typical sea structures is not necessarily dominated by first mode behavior as is the case of the structural systems which have been mainly used to justify the technique.

**B.6.3** *Dynamic analysis for survivability level earthquake*-- It is considered advisable to attempt to quantify the extent of damage under the survivability level earthquake, unless exceptionally large safety factors are used. As a guideline, this should be done if twice the distortions from the design earthquake are sufficient to initiate significant yielding.

Damage in the form of irrecoverable distortions can only be determined by a truly nonlinear analysis method which keeps track of the instantaneous and cumulative distortions. Ideally, the constitutive laws employed should permit incorporation of the progressive degradation characteristics of reinforced concrete and soil. Such analyses have been attempted for various systems but are far from being standard techniques.

**B.6.4** *Structural damping*- The selection of damping contributions from the structure itself should allow for the major differences between offshore structures and typical reinforced concrete buildings on land. In particular, the existence of so-called nonstructure such as partition walls, brick panels and the like in onshore buildings can make significant damping contributions which would not necessarily be present in a typical offshore structure. Relevant structural damping values will generally be smaller than for conventional buildings.

For the design level earthquake an upper limit of 2 percent of critical should be used unless a higher level can be justified from laboratory or field measurement data. A higher value can be used for the survivability event but the structural damping chosen must be consistent with the applicable damage levels associated with this case.

## B.7- Stress analysis

The idealization of the structure for the dynamic analysis is frequently not sufficiently detailed to provide adequate stress output for the structural components. Further post processing of the results to obtain stresses is fairly straightforward and may involve quasistatic or dynamic analysis of substructures. Care must be taken to allow for fluid interaction forces in calculating factors such as panel shear. Pressures required to accelerate both the contained and added fluid masses must be incorporated.

## B.8- Failure modes

The results of the stress analysis for the design earthquake should be used to conduct an audit of the structural system to assess the likely failure modes. Both local and global behavior must be considered and particular care must be devoted to identifying and eliminating possible progressive collapse situations. The structural design must be executed to avoid wherever possible the initiation of failure in compression or shear of primary components.

## B.9- Ductility requirements

The importance of ductility in preventing structural collapse has been clearly established. General ductility requirements have been developed for buildings but these are not directly applicable to most offshore structures due to the large differences in the structural systems employed. However, the same basic principles should be employed. The overall structural system must be designed such that yielding does not initiate in a member governed by compressive loads. Plastic hinges should be allowed to initiate only in flexural members not subject to large thrusts. The system must possess suitable redundancy characteristics such that progressive or immediate collapse does not occur. The ductility requirements must be considered in an overall sense for the structure as well as for the design of individual sections. Excessive use of prestressing should be avoided.

Attention must be paid to insuring adequate ductility even in the case where the initial survivability check indicates that yielding will not occur. The designer must guard against the apparent security provided by the use of "adequate" safety factors, it is essential to remember that in the case of earthquakes, a stronger but brittle structure is usually less desirable than a weaker one with adequate ductility.

## B.10- Aseismic design details

The successful performance of a concrete structure in an earthquake is very dependent on proper reinforcement detailing. The structural designer must not relegate the development and execution of the design details to another organization over which he has no supervisory authority and control. He must insure that these details are developed in accordance with the overall design philosophy.

Some assistance may be obtained from the detailing principles developed for typical building components but these should be applied with care to offshore structural components in which both the structural shapes and the disposition of reinforcement are different. Special care should be given to the use of lateral ties to insure adequate confinement of the concrete in wall elements, joints, and intersections, and the development of suitable splice details to avoid bond degradation. Detailing should be such that yielding will typically commence in the reinforcement in tension and not in compression of the concrete, either direct or associated with flexure or shear mechanism. The load and material factors for shear and flexure must be selected such that failure will be forced to initiate in flexure. As an approximate guide, the ratio of the equivalent safety factors in shear and flexure should be at least 1.5. Members subjected to dead load compressive stress and/or significant prestress may be given a significant increase in ductility by heavy confining steel in the form of spirals or tied stirrups.

## B.11- Other factors

**B.11.1** *Site stability*- The stability of the site itself during earthquakes should be established before carrying out the above analyses. Factors such as liquefaction and slope instability must be checked. In addition to evaluating conditions on the site itself, the possibility of slides which may affect the location from unstable portions further up and down the slope should be checked

**B.11.2** *Tsunamis*- Very long period waves can be created by earthquake and/or marine landslides. In deep water, these can cause a rise (and fall) of water surface of several meters, depending upon the geographical location. This increases (and decreases) the hydrostatic head throughout the full water column. In addition, the long period wave may cause a significant lateral force on the structure. Near shore and in shallow water, the tsunami wave buildup and translatory motion can be destructive; therefore, areas at entrances to harbors and inlets need to be examined on the basis of historical records.

**B.11.3** *Compressive shock waves*- Compression waves are generated in the water due to earthquake action. The pressure magnitude can be quite large at short epicentral distances but the effect will usually only be significant for the structures which are not fully flooded.

**B.11.4** *Cyclic degradation*- Both structure and foundation may be subject to degradation of stiffness and strength properties during cyclic loading at high stress levels. The implications should be investigated for high stress level cycling in critical parts of the system

This report was submitted to letter ballot of the committee which consists of23 members; 16 voted affirmatively and 7 ballots were not returned.