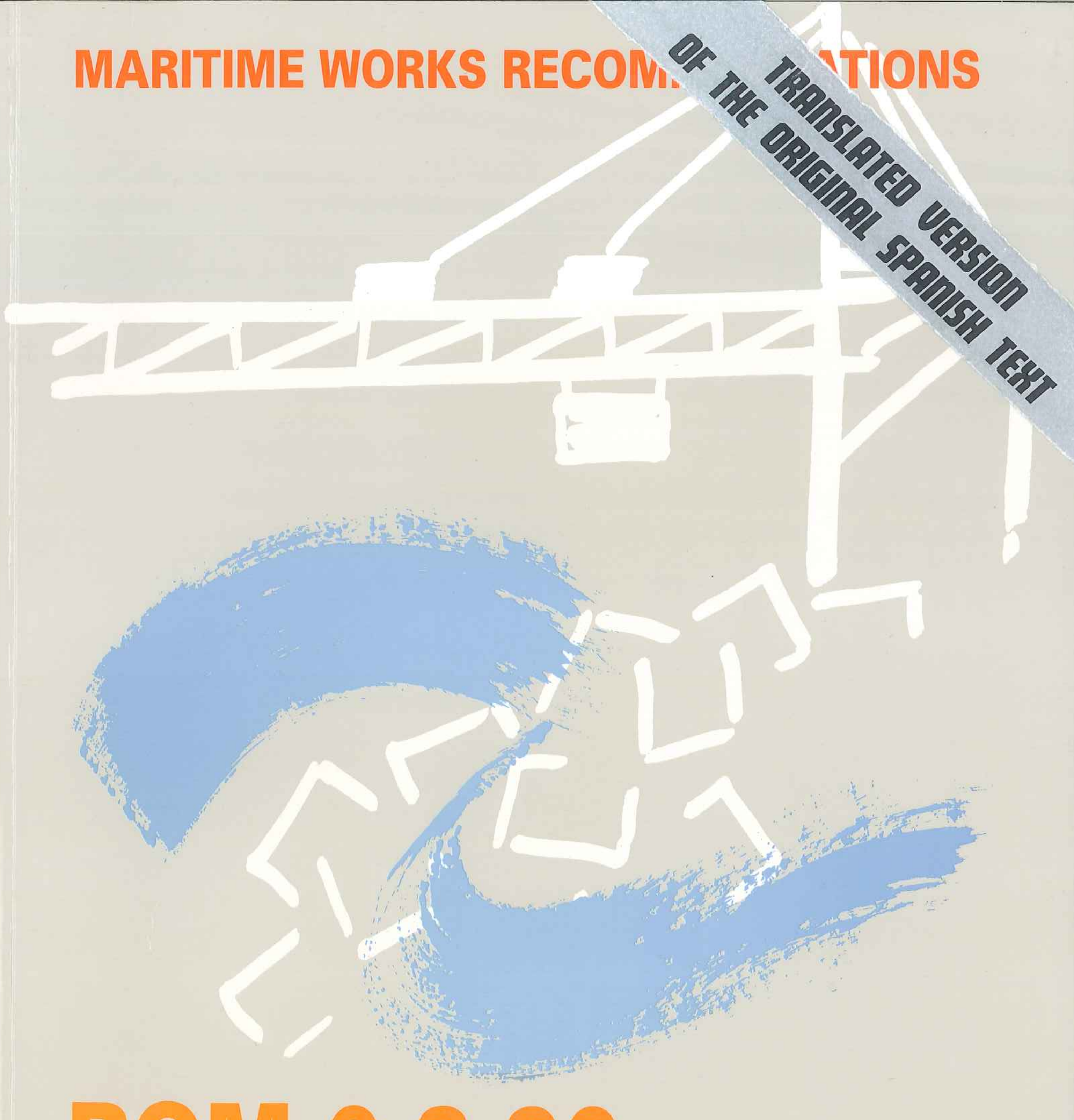
# MARITIME WORKS RECOMMENDATIONS

TRANSLATED VERSION
OF THE ORIGINAL SPANISH TEXT



# ROM 0.2-90

## ACTIONS IN THE DESIGN OF MARITIME AND HARBOUR WORKS

Puertos del Estado

# OBRAS MARITIMAS

*TECNOLOGIA*

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Puertos del Estado

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## ACTIONS IN THE DESIGN OF MARITIME AND HARBOUR WORKS

# PREFACE

The Ministry of Public Works and Urban Planning (Ministerio de Obras Públicas y Urbanismo, MOPU), through the General Direction of Ports and Coasts (Dirección General de Puertos y Costas) has begun a program of technological development in the area of Maritime and Port Construction whose objective is to establish Recommendations or 'Codes of Good Practice' for the planning and execution of Maritime and Port Construction, which would constitute the beginnings of a future Spanish School in this Engineering field.

These Maritime Works Recommendations (ROM) shall define an ordered set of technical criteria that, while not being for the moment compulsory or normative, will help to inform project engineers, directors or executives of maritime and port works and ensure the quality levels demanded by these works.

The lack of this type of Recommendations or Technical Codes in Spain in the field of Maritime Engineering introduces a significant dose of insecurity in the development of technical work. It often makes the criteria used by different project engineers and responsibilities of the supervision or execution of the maritime works differ notably or have contradictory results. In addition, Foreign Norms or Recommendations can only be applied in a limited way, as many of them only cover partial aspects of the general technical problems of these works.

These limitations underscore, on one hand, their technical complexity, and on the other, create a great opportunity for their development. All Engineers that have or have had a recognized voice in this field in Spain have participated in the development of this work, and so it is a true representation of the «State of the Artÿ in our country.

The purpose of this project has been to provide the project engineers, supervisors, and builders with a technical tool that systematically brings together the current technical criteria that must be applied to maritime work development, facilitating the work and responsibility of both the private and public concerns involved. It is the responsability of the Engineer to apply critical engineering judgement when refering to these Recommendations during the execution of a project. Therefore, we would be especially thankful for whatever commentary, suggestions or cautions that could contribute to the improvement of the technical efficiency and quality of the Recommendations. Only after a long period of technical maturation, and through their use by specialists in the treated material, will the Recommendations acquire the consolidation necessary to become compulsory Technical Codes for Administration projects.

I want to express my thanks to the different specialists that are participating in the development of these works, who understand their inherent responsibility, importance, and technical difficulty. I would especially like to mention two men who could not see this work published, and who contributed their exceptional talent - Victoriano Fernández Dupuy, an outstanding authority on port engineering, and Fernando Rodriguez Gómez, an exceptional collegue and collaborator. They have gone to the other side of the "Stygian Lagoon" where they are surely preparing an unequalled port of arrival for the "Barca de Caronte", when we all will be brought together again. For them, my admiration, remembrance and gratitude.

Madrid, April 1990

Fernando Palao Taboada
General Director of Ports and Coasts

# GENERAL INDEX

# INTRODUCTION

## THE ROM PROGRAM

The development of Maritime Works Recommendations (ROM Program) began in 1987, by order of the President of Ports of the State, at the time General Director of the Ports and Coasts of the Ministry of Public Works and Urban Planning, with the creation of a Technical Commission assigned to draw up a set of Recommendations or Technical Codes. These Recommendations or Codes would bring together the most advanced technology in the field of maritime and port engineering and become a technical instrument for project engineers, supervisors, and builders, allowing both public administrations and private companies with interest in maritime engineering easy access to specialized information necessary for carrying out their work.

Since that time, the Technical Commission, together with several specialists, and in collaboration with public and private institutions and organizations has been developing studies in different areas of maritime engineering, with the aim of covering all aspects of this field of technology.

The ROM program plans to complete the following Codes :

ROM 0. GENERAL RECOMMENDATIONS

- 0.1 Project Development Recommendations.
- 0.2 Action Consideration Recommendations.
- 0.3 Recommendations for the Consideration of Environmental Actions/I : Wave Action, Currents, Tides and other Water Level Variations.
- 0.4 Recommendations for the Consideration of Environmental Actions/II : Atmospheric and Seismic Conditions.
- 0.5 Geotechnical Recommendations.

ROM 1. BREAKWATER DESIGN AND CONSTRUCTION RECOMMENDATIONS

- 1.0 General Breakwater Design Recommendations.
- 1.1 Sloping Breakwater Design and Construction Recommendations.
- 1.2 Wall Breakwater Design and Construction Recommendations.
- 1.3 Composite Breakwater Design and Construction Recommendations.
- 1.4 Special Breakwater Design and Construction Recommendations.

ROM 2. BERTHING WORKS DESIGN AND CONSTRUCTION RECOMMENDATIONS

- 2.0 General Recommendations for Designing and Constructing Berthing Works.
- 2.1 Pier Design and Construction Recommendations.
- 2.2 Platform and Trestle Design and Construction Recommendations.
- 2.3 Dolphin Design and Construction Recommendations.

ROM 3. APPROACH AND WATERPLANE SURFACES DESIGN AND CONSTRUCTION RECOMMENDATIONS

- 3.0 General Recommendations for Designing and Constructing Waterplane Surfaces.
- 3.1 Recommendations for Designing and Constructing Access Channels.
- 3.2 Anchoring and Ship Maneuvering Area Design and Construction Recommendations.
- 3.3 Dredging Design and Execution Recommendations.

ROM 4. SUPERSTRUCTURE DESIGN AND CONSTRUCTION RECOMMENDATIONS

- 4.1 Port Pavement Design Recommendations. Pavement Standardizations.

This program only sets general guidelines and work priorities, allowing for their open and continuous development, as well as the possibility of their incorporation, segregation, or fragmentation while the ROM Program is developing, thereby easing its advancement and publication.

ROM 0.2. Action Consideration Recommendations has been written first, because of its importance in the design, because of the great gaps that exist in this field, and because it sets the basic criteria for the Design Life, Action Evaluation Criteria, and Calculation Bases, which are all necessary for the development of the ROM Program.

## DEVELOPMENT OF THE PROGRAM

The Program of drawing up the Recommendations is being developed by the following Expert Group, working as a Technical Commission, and under the direction and coordination of the Technical Direction of Ports of the State.

— D. Francisco Esteban Rodriguez-Sedano (Ports of the State)
Acting as the Commission President

— D. Victoriano Fernández Dupuy (†) (INTECSA)
— D. Carlos Sanchidrián Fernández (ALATEC, S.A.)
— D. Eduardo Arana Romero (IBERINSA)
— D. Eloy Pita Carpenter (Ports of the State)
— D. Javier Rodríguez Besné (Ports of the State)
— D. José Llorca Ortega (Ports of the State)
Acting as the Technical Commission Proposer and Secretary

based on a strict work methodology from the development of specific research and data analysis programs (especially environmental data), and on the summary, study, and comparative analysis of the existing bibliographical documentation, with special mention to the following :

— Codes, Manuals, Instructions and other international Normatives in the field of maritime engineering. Among others : "British Standard Code of Practice for Maritime Structures" (United Kingdom), "Recommendations of Committee for Waterfront Structures" (German Federal Republic), "U.S. Navy Manuals" (U.S.A.), "Danish Code of Practice" (Denmark), "Technical Standards for Port and Harbour Facilities in Japan" (Japan), "Construction Codes and Regulations" (U.S.R.R.), "Shore Protection Manual" (U.S.A.), 'UNCTAD Handbooks' (United Nations).

— Recommendations for companies or entities with interest in certain maritime engineering fields. Among others : American Petroleum Institute, "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms" (USA); Det Norske Veritas «Rules for the Design, Construction and Inspection of Offshore Structuresÿ (Norway); Oil Companies International Marine Forum (OCIMF), "Guidelines and Recommendations for the Safe Mooring of Large Ships at Ports and Sea Islands"; Bureau Veritas "Rules and Regulations for the Construction and Classification of Offshore Platforms" (France).

— Recommendations of International Associations in the field of maritime and port engineering, such as : Permanent International Association of Navigational Congress (PIANC); International Association of Ports and Harbors (IAPH); International Association of Hydraulics Reports (IAHR); The American Society of Civil Engineering's Coastal Engineering Research Center (CERC); and European Federation of Manutention (FEM).

— Codes and Recommendations of International Associations in the field of Civil Engineering, such as : Euro-International Concrete Committee (CEB); International Prestressed Federation (FIP); and European Committee on Metallic Construction (CECM).

— European Economic Community Normative : "Eurocodes".

— Spanish Normative in effect : NBE-AE-88, NBE-MV-103, EH-88 EP-80, PDS-1, Highway Instructions, Instructions Regarding the Actions to Consider in the Design of Highway and Railway Bridges, Codes for the Calculation of Electronic Portal Cranes for Port Services and the Regulation of Laws regarding Marinas.

— Work and studies carried out by the Ports and Coasts Laboratory, and by the Geotechnical Laboratory, of the Center for Studies and Experimentation of Public Works (CEDEX) and by the Maritime Climate Technical Department of Ports of the State (DTCM).

The writing of the Recommendations is carried out, taking into consideration the General Codes and Instructions in effect in Spain, expressly indicating those where the specific characteristics of maritime and port engineering require a complementary development of such Codes.

In any case, an attempt has been made to contribute to the gradual harmonizing of the Spanish Normatives with those of Europe especially with those of the European Economic Community, basically encompassed in the civil engineering field by the "Eurocodes".

The Technical Commission proposes to reunite and take into consideration the commentary, suggestions, and initiatives regarding each of the published Recommendations, in order to prepare revised versions that will then have the character of Instructions. Any such observations should be sent to :

Departamento Técnico de Tecnología y Normativa
Puertos del Estado
Avda. del Partenón, 10
Campo de las Naciones
28042 Madrid.

MAY, 1994

# SECTION 1 GENERAL

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# SECTION 1 TABLES

## Index

# SECTION 1 GENERAL

## 1.1 SCOPE

The Recommendations for the Consideration of Actions in the Design of Maritime and Port Works (ROM 0.2) shall be applicable to the design of all maritime and port works, whatever their class or use, or construction material. To this effect, those structures or structural elements located in maritime, port or fluvial zones, or in any other location within the coastal public properties, that always remain stationary in the service phase, shall be considered Maritime or Port Works. This shall be applicable to fixed as well as floating structures.

## 1.2 CONTENTS

The present Recommendations include all the information and criteria necessary for the complete definition of the actions that act upon maritime and port works, in the local and environmmental conditions of Spain. Nevertheless, the majority of their contents can be directly applied or extrapolated for use in any other place in the world with the necessary modifications to take into account the specific local conditions.

Recommendation 0.2 for the consideration of actions in the design of maritime and port works is structured in four parts :

Part 1. General. Includes all the general aspects necessary for the correct application and comprehension of the Recommendation; scope, general description of the contents, units used, notes and symbol explanations, and references.

Part 2. General Criteria of the Project. The different project phases and work hypotheses to be considered in the design of the maritime works are defined; where differences can occur in the evaluation of the acting loads, in the combination criteria of the actions, or in the strength of a part of or all of the structure. Likewise, minimum durations of the service phase, or design life are recommended for works of definitive character, which is highly important for the evaluation of the actions by means of statistical bases (risk criteria).

Part 3. Actions. The actions are classified in order to facilitate a system of application of loads in terms of their evaluation and combination. The general criteria are fixed for the evaluation of actions, focusing especially on the definition of their representative values and their determination by statistical means : with the concept of risk and recommendation of maximum admissible risks. The possibility of dynamic effects is analyzed, quantifying their importance and indicating when and how their consideration is feasible by a static analysis. Also included are the definitions and parameters of each one of the actions that act upon the maritime works, criteria for the determination and application of these actions, as well as typical and minimum recommended values. Environmental actions (basically wave action, currents, tides and wind) are excluded, because, due to their importance and specificity in maritime works, their very different parameters and necessary scope of treatment, they are covered in two Recommendations found in the general index: ROM 0.3. Recommendations for the Consideration of Environmental Variables/I: Wave action, currents, tides and other water level variations; and ROM 0.4 Recommendations for the Considerations of Environmental Variables/II: Atmospheric and Seismic Conditions.

Part 4. Calculation Bases. The general calculation process corresponding to the limit state methods is defined and described as the only one compatible with the action evaluation criteria and their representative values included in these Recommendations. General criteria for the evaluation of actions are fixed, together with the criteria for load combinations in each analyzed design phase and work hypothesis.

## 1.3 DEFINITIONS

For the present Recommendations, the following most commonly used fundamental terms are expressly defined. These and other terms shall generally be defined and explained in more detail in the sections of the Recommendations where they are introduced into the text.

- ACCESS TRACKS : A route that unites operation and storage areas with exterior areas of the port zone, or that serves zones without the handling of cargo. Generally are tracks used for conventional road traffic.
- ACCIDENTAL LOAD : Fortuitous or abnormal loads that happen as result of accidents, misuse, environmental conditions or overloads.
- ACTION OR LOAD : Any cause or acting agent capable of generating forces stresses, or strains in a structure or structural element.
- ALONGSIDE BERTHING : Berthing of a ship with its port or starboard to the berth.
- ANTROPIC FILL : Fill made up of products of human wastes (urban waste, rubbish, etc.).
- ARTESIAN WATER LEVEL : Hydrostatic pressure level higher than the ground level.
- ASHLAR MASONRY CONSTRUCTION : Construction of cut stone, of sizes that fit with prior set pieces, so as to unite in a regular, consistent way on the contact surface.
- ASTRONOMICAL TIDE : Tide due to gravitational attraction of the moon, sun and other astronomical bodies. Its intensity is closely related with the relative position of the sun and moon with respect to the earth.
- AVERAGE DAILY TRAFFIC (ADT) : Number of heavy vehicles that go by a section during the course of a year, divided by 365. It can be considered as the heavy vehicle traffic intensity that corresponds to the average day of the year in this section.
- AVERAGE RETURN PERIOD : Average time interval in which the extreme value exceeds a certain variable.
- BACK OF A STRUCTURE : Interior surface of a earth retaining structure.
- BALLAST DISPLACEMENT : Light displacement plus consumables, fresh water and saltwater, and the minimum ballast weight necessary for the ship to sail and maneuver safety.
- BASIN RESONANCE : Amplification and changes in the oscillation of a natural or artificial water area when waves with a period close to the natural oscillation period of the basin reach it. It results in long period steady or quasi-steady waves.
- BEAM : Maximum width of the cross section of the ship's hull.
- BERTHING WITH PREPONDERANT LONGITUDINAL APPROACH : The berthing maneuver is performed in a direction parallel to the ship's longitudinal axis.
- BERTHING WITH PREPONDERANT TRANSVERSE APPROACH : The berthing maneuver is performed in a direction perpendicular to the ship's longitudinal axis.
- BITT : Mooring device located ashore or fixed to a resistant structure consisting of a pair of short posts, generally cast iron, used for fixing a mooring line.
- BLOCKS : Elements which support the ship's hull in a dry dock or slipway
- BOLLARD : Mooring device located ashore or fixed to a resistant structure consisting of a short post, generally cast iron, adequately shaped in the upper part so as to fix the mooring lines in several directions.
- BOW : Front part of a ship.
- BREAST LINES : Mooring lines approximately perpendicular to the ship's longitudinal axis, which are usually set as far fore or aft as possible.
- BRICK CONSTRUCTION : Construction based upon bricks and set with mortar.

— BULKCARRIER : Ship used for transporting solid bulk.

— CHARACTERISTIC ACTION : The value of the action associated with an exceedance probability during the design life assigned to each phase or work hypothesis.

— CHARACTERISTIC VALUE OF THE MATERIAL'S PROPERTIES : The value of the material's properties associated with an exceedence probability for the statistical distribution obtained based on tests done under conditions preestablished in the corresponding Code.

— COHESIVE SOIL : Soil that exhibits cohesive properties.

— COMMERCIAL PORT : Port installation mainly used for handling and temporary storage of goods and acting as an interface between maritime and land transportation.

— COMPOSITE CONSTRUCTION : Construction made with fabricated elements and metallic elements.

— CONSOLIDATION PROCESS : Progressive decrease of the water content in the soil or a saturated cohesive landfill due to the action of a constant load.

— CONSTRUCTION PHASE : The period of time starting at the beginning of the construction of the structure, until its initial use.

— CONSTRUCTION WORK : Construction done with stone, brick, concrete, and other stone materials.

— CONTAINER : Paralelepipedic box with standardized dimensions, inside of which general cargo is put to be carried entirely from origin to destination.

— CONTAINER CRANE : Portal crane in whose seaward extreme there is a lowerable boom that allows the extension of a container hooking system, enabling direct loading and unloading to or from the evacuation or storage zone. It has the capability of longitudinal transfer upon rails in the direction perpendicular to the boom, and is unable to turn upon a vertical axis.

— CONTAINER SHIP : Ship devoted to transportation of unitized goods by means of containers.

— CONVENTIONAL ROAD TRAFFIC : Heavy vehicle traffic used for transporting cargo and people on highways : cargo trucks greater than 3 t., with more than four wheels and without trailers; trucks with one or more trailers; articulated vehicles, special vehicles; and vehicles with more than nine seats used for transporting people.

— CRANE : Cargo handling equipment used for raising, shifting and lowering.

— DEAD WEIGHT TONNAGE (DWT) : Measure of the loading capacity of a ship. It equals the weight of the cargo, fuel, fresh water and saltwater, consumables, passengers, luggage, crew and stores measured in tons.

— DESIGN ACTION : Or weighted value of an action, is that which results from the application of safety factors to their representative values. The effects produced by the actions shall be obtained, based on their design values.

— DESIGN LIFE : Period of time from the beginning of construction of the structure until it is dismantled, put out of service or used for another purpose.

— DESIGN LOAD : Effect produced by the actions, calculated based upon a load hypothesis.

— DESIGN PHASES : Differentiated stages in which the design life of a structure is normally divided.

— DESIGN VALUE OF THE MATERIAL'S PROPERTIES : Or decreased value, is the result of applying the appropriate safety factors to the characteristic value of the material's properties. The strength capacity of the structure or part of the structure is obtained based upon the design value of the construction material's properties.

— DISPLACEMENT : Total weight of a fully-loaded ship. It equals the weight of the volume of water displaced by the ship's hull.

— DOLPHIN : Free structure, usually built of waled piles, a reinforced concrete caisson or an enclosure of sheetpiles, with the following functions : resisting the impact of a berthing ship, resisting the tension of the mooring lines, serving as a guide during a ship's maneuver and/or protecting some other structure from the impact of ship.

— DRAUGHT : Height of the submerged cross section of a floating structure measured above the keel. It depends on the loading condition of the structure.

— DRY DOCK : A dock that can be kept dry for use during the construction or repairing of ships.

— DYNAMIC LOAD : Load whose application produces significant accelerations in the structure or structural elements.

— EFFECTIVE STRESS : Intergranular pressure in a soil or a landfill. Equivalent to the total pressure minus the pore pressure.

— EQUIPOTENTIAL LINE : Line of all points that have the same piezometric level.

— EXCEPTIONAL CONDITIONS : When the installation is subject to unusual or extraordinary actions as a result of accidents, misuse, or overloads, even if these actions are foreseen.

— EXTREME CONDITIONS : When the installation must cease or limit its activities during environmental actions beyond the operating condition's limits. This condition is associated with the most severe environmental conditions for which the structure is designed.

— EXTREME DISTRIBUTION : Relationship between the maximum foreseen values of a variable and their non-exceedence probability in the period of a year.

— EXTREME VALUE OF A VARIABLE : Maximum periodic value of the variable, determined by means of a statistical base.

— FENDER SYSTEM : Berthing structure and fendering elements capable of absorbing the impact energy of a berthing ship or resisting the push loads once it is moored.

— FILL OR BACKFILL : Artificial deposition of natural materials originating from the earth's shell (soils, rocks), of specially made artificial material (tetrapods, dolos), or of industrial or urban remains (rubbish, slags).

— FISHING HARBOUR : Harbour installations designated for fishing activities.

— FLEXIBLE STRUCTURES : Structures with low damping capacity and oscillation frequencies, corresponding to the fundamental mode in the direction of the acting load, that are low in relation to this load.

— FREEBOARD : Height of the emerged cross section of a floating structure, measured along the sides. It depends on the load of the floating structure.

— FREQUENTIAL LOAD : Actions that act upon the structure in a cyclical way, in regular time intervals, or in an irregular way as a combination of cyclical loads of different characteristics.

— FRONT OF A STRUCTURE : Exterior surface of an earth retaining structure.

— FRONTAL BERTHING : Berthing of a ship bow or stern to the berth.

— GENERAL CARGO : Products which are transported in a piled or packed way (inside boxes, sacks, barrels, in ingots or coils, etc.), handled in a discontinuous way individually (coils, equipment goods) or unitized (using pallets, nets, etc.).

— GENERAL CARGO MERCHANT SHIP : Ship used for the transportation of general cargo.

— GENERAL CARGO ON PALLETS : General cargo handled by means of pallets (platforms of standardized dimensions upon which goods are positioned, making a load and handling unit).

— GRANULAR FILL : Fill made up of gravel and/or sands extracted from terrestrial mines, with a low fine grain content.

— GRANULAR SOIL : Soil that does not exhibit any cohesion, or that does not have strength for simple compression without lateral pressure.

— GRAVITY EARTH RETAINING STRUCTURE : The structure that contains the soil fundamentally by its own weight.

— GROSS REGISTER TONNAGE (GRT) : Total volume capacity of a ship, measured in register tons.

— GROUNDWATER FLOW : Set formed by the flow lines and the equipotential lines.

— HEAD AND STERN LINES : Mooring lines positioned at the bow or stern of a ship. Usually set at a horizontal angle of 45∞± 15∞ with the longitudinal axis of the ship.

— HEAVE : Motion of a moored or free sailing ship, consisting of a translation along the vertical axis.

— HEAVY BULK : High density solid bulk, such as coal and iron ore.

— HIGH WATER : Maximum height reached by the sea level during an ascending tidal cycle.

— HIGHEST WATER SPRING LEVEL : Maximum theoretical high water level that can be produced under average meteorological conditions, in the case that all the astronomical conditions occur simultaneously.

— HYDRAULIC FILL : Fill deposited hydraulically, that is, by means of a settling process of soil particles contained in an effluent (material from dredging).

— HYDRAULIC GRADIENT : Relationship between the difference of the piezometric level and the distance covered.

— HYDROSTATIC PRESSURE : Hydraulic load obtained when the piezometric level coincides with the ground waterlogging level.

— IMPACT LOAD : Action that acts upon a structure, producing a response that reaches a maximum value in the initial moment; becoming smaller with each cycle until reaching the rest position.

— INDUSTRIAL PORT : Port installation exclusively used for serving an industrial area (ship-building, steel industry, chemical industry, refineries, etc.).

— KEEL : Main longitudinal frame in the bottom of a ship's hull.

— LARBOARD : Left side of the ship looking towards the bow.

— LIGHT DISPLACEMENT : Total weight of the ship when it leaves the shipyard, without any load, ballast or fuel.

— LIMIT STATES : The structure states that, when reached or exceeded, put the structure out of use due to lack of compliance with the pre-established stress or functional limits.

— LIQUID BULK : Liquid products which are transported in a homogeneous way and can be handled in a continuous way (oil products, liquified gases, water, oils, etc.).

— LNG CARRIER : (LIQUIFIED NATURAL GAS) : Ship devoted to transportation of liquified natural gas.

— LOAD : Set of effects produced by the actions upon the resistant structure such as stresses, tensions, strains, displacements, movements, etc.

— LOAD HYPOTHESIS : Design load combination.

— LOAD SAFETY FACTOR OR INCREASE FACTOR : Multiplication factor of the representative values of the actions to obtain the design value.

— LOAD TRAIN : Series of several concentrated and/or distributed loads acting simultaneously, perfectly defined in magnitude, geometric disposition, and application conditions.

— LONG WAVES : Long period waves (>20-30 sec.), generally with low amplitude in the open sea, usually generated by sudden changes in wind or air pressure, or by the existence of wave groups.

— LOWEST WATER SPRING LEVEL : Minimum theoretical low tide that can occur under average weather conditions, in the case that all astronomical conditions effecting the tides occur at the same time.

— LOW WATER : The minimum elevation reached by the sea level during a falling tide cycle.

— LPG CARRIER (LIQUIFIED PRESSURIZED GAS) : Ship used for the transportation of liquified oil-derived gases (propane, propylene, butane, etc.).

— MANEUVERING TRACKS : A route that unites operation areas with storage areas, primarily used for the circulation of cargo handling equipment.

— MARINA : Harbor installation mainly devoted to sport and recreational activities.

— MARINE GROWTHS : Any type of marine organism that grows on the surface of the maritime structure, such as algae, mollusks, etc.

— MASONRY CONSTRUCTION : Construction done using stone of any size, set with or without mortar.

— MATERIALS STRENGTH SAFETY FACTOR OR DECREASE FACTOR : Coefficient introduced in the calculations to decrease the characteristic values of the materials' properties to obtain their design values.

— MEAN DISTRIBUTION : Relationship between a variable and the probability that its values are not surpassed in a period of time equal to a year.

— MEAN HIGH WATER : Mean value of the high tide levels during all tidal cycles.

— MEAN LOW WATER : The average height of the low water during all falling tides.

— MEAN MAXIMUM WAVE HEIGHT : Maximum wave height foreseen for the mean duration of the extreme design conditions.

— MEAN SEA LEVEL : Average level of the surface of the sea, obtained from observations over many years (at least 18.6 years) that correspond to a lunar cycle. It could also be defined as the average sea level that would exist in the absence of tides. As an approximation, it can be obtained by taking the average of all the high and low tides during a lunation.

— MEAN ZERO CROSSING WAVE PERIOD : Average time interval between two consecutive ascending crossings of the mean sea level in a wave series.

— METALLIC CONSTRUCTION : Construction made of mostly metallic elements.

— METEOROLOGICAL TIDE : Changes in the sea level in coastal areas as a consequence of storms produced by strong barometric depressions.

— MULTIPURPOSE SHIP : Ship prepared for the transportation of different kinds of goods : containers, general cargo, solid or liquid bulk, etc.

— NAVAL PORT : Port installation mainly operating as a base for naval ships.

— NOMINAL VALUE OF A VARIABLE : Established theoretical value of a variable, determined taking into account all its reasonable foreseen variations.

— NON-CHANNELED TRAFFIC AREA : Zone where the movements of cargo handling equipment and conventional road traffic cannot be predetermined.

— NORMAL BULK : Low and medium density solid bulk. Some of the most common in maritime transport are cereals and other food products, chemicals, and cements.

— NORMAL LIQUID BULK : Non-combustible, non-toxic liquids such as water, wine, etc.

— NORMALLY CONSOLIDATED SOIL : Cohesive soil that has never undergone effective stresses greater than those existing at the current moment.

— NORMAL OPERATING CONDITION : Condition in which the maritime or port installation functions without limitations, unaffected by environmental conditions.

— NUMBER OF EQUIVALENT PAWLS : Used in calculations of pavement deterioration, number of accumulated standard loads (PAWL), equivalent to the circulation of the cargo handling equipment.

— OIL TANKER : Ship used for the transportation of crude oil or petrochemicals.

— OPERATION AREA : Zones designated for the transfer and handling of cargo, materials, or supplies, and where they are not heavily accumulated.

— OVERCONSOLIDATED SOIL : Cohesive soil that has undergone effective stresses greater than those existing at the current moment.

— PASSENGER SHIP : Ship whose main purpose is to serve as transportation means for passengers.

— PERMANENT LOAD : Loads that act during every moment of the analyzed project phase.

— PHREATIC OR SATURATION LEVEL : Upper surface of the outer water, or zero hydraulic load line.

— PIEZOMETRIC LEVEL OR PIEZOMETRIC HEIGHT : Height that the water level reaches in placing a piezometric pipe at a point. This level is usually different from the phreatic level whenever there is a groundwater flow.

— PILE : Element used for deep foundations, made of materials such as wood, steel, concrete and combinations of these three, which are prefabricated and positioned by means of driving, or executed on site after boring or excavation of a hole.

— PITCH : Motion of a moored or free sailing ship consisting of a rotation around the transverse axis. It results in the largest vertical oscillation in the bow and in the stern

— PORTAL OR QUAY CRANE : Crane capable of displacing longitudinally upon rails along the pier, in a direction parallel to the edge of the quay, with all four feet being supported on the pier, and able to turn upon a vertical axis.Consists of three parts : Portal, operating and motor cabin, and boom.

— PULLEY : Mooring device consisting of one or more pulleys fixed to a resistant structure, used to guide a mooring line towards a bitt or windlass.

— QUICK RELEASE HOOK : Mooring device located ashore or fixed to a resistant structure which permits an easy and fast release of the mooring lines by means of a simple manual operation or an electromechanical device. It is usually applied either in mooring points not accessible from the shore or with the aim of speeding up the ship's departure.

— RAIL : Basic structural element of the rolling track of a railway and restricted movement cargo handling equipment, that acts as track, wheel guide and electric core.

— REGISTER TON : Unit of volume equal to 100 cubic feet, that is, 2.83 cubic meters.

— REPRESENTATIVE VALUE OF AN ACTION : Value of an action associated with its level of variation in time.

— RESONANCE OF SHIP/MOORING/FENDERING SYSTEM : Amplification of the motion amplitudes of a moored ship when the incident wave period is close to the natural oscillation period of any of the ship motions. The consequence is a large increase of mooring forces, fender forces and bollard forces.

— RIGID STRUCTURES : Structures with high damping capacity and natural oscillation frequencies, corresponding to the fundamental mode in the direction of the acting load, that are high in relation to this load.

— RISK : Probability of occurence of an extreme value of the variable during a given period of time.

— ROCK : Natural aggregate of one or more minerals that do not undergo changes in the presence of water.

— ROLL : Motion of a moored or free sailing ship consisting of a rotation around the longitudinal axis. This motion is linked to the transverse stability of the ship.

— RO-RO SHIP (ROLL ON-ROLL OFF) : Ship especially designed for loading and unloading by means of direct wheeling vehicles.

— RUBBLE MOUNDS AND RIP RAPS OF CLOSED GRANULOMETRY : Fill where the larger elements are completely surrounded by the fine grained material, and big pores do not exist. The granular conditions is: 30% sifted through a 3/4" sieve; and 35% through a nº 200 sieve.

— RUBBLE MOUNDS AND RIP RAPS OF OPEN GRANULOMETRY : Fill basically created using rocky materials or specially made artificial materials, generally with low content of fine grained material (30% sifted trough a 3/4" sieve, and 10% through a nº 200 sieve). The coarse grains are in contact with one another, and the finer grains do not totally fill the voids.

— SAFETY LEVEL : Evaluation of the safety requirements of a structure, in terms of the consequences of loss of human life, environmental damage, and economic losses in case of its failure or lack of use.

— SERVICEABILITY OR USE LIMIT STATE : All states or structure situations in which the structure is out of service because of functional, durability or aesthetic reasons.

— SERVICE AREA : Zone excluded from cargo, material, or supply traffic. It shall basically be either a habitable, administrative, service or recreation area.

— SERVICE DESIGN LIFE : Duration of the service phase.

— SERVICE PHASE : The period of time starting with the complete installation of the structure until it is dismantled, used for another purpose, or is no longer usable.

— SHIP'S DEPTH : Maximum height of the cross section of a ship's hull, measured from the keel up to the upper part of the main deck.

— SHIP'S HULL : Floating receptacle or main body of the ship.

— SHIP'S LENGTH : Maximum length of a ship from bow to stern.

— SIGNIFICANT WAVE HEIGHT : The average height of the highest one-third of the waves.

— SIGNIFICANT WAVE PERIOD : Mean value of the periods of the highest one-third of the waves.

— SILO EFFECT : Secondary equilibrium that originates in a breaking soil mass, against deformation.

— SLIPWAY : Auxiliary facility used for ship building, maintenance and repair. It consists of an inclined plane with complementary elements, such as guide rails, travelling carriages, blocks, etc., aimed at launching or drydocking of ships by longitudinal or transverse hauling.

— SOIL : Portion of the earth's shell formed by broken or loose materials, easily separated into individual particles by agitating a dry sample in water. Included are: pebbles, gravel, sands, clays and organic materials.

— SOLID BULK : Products which are transported in a homogenous way with a loose appearance and can be handled in a continuous way.

— SPRINGS : Crossed mooring lines that are available at the ship's side and center, acting completely longitudinally to the mooring points.

— STARBOARD : Right side of the ship, looking towards the bow.

— STATIC LOAD : Load whose application does not produce the significant accelerations in the structure or structural elements.

— STERN : Back part of a ship.

— STORAGE AREA : Zones designated for prolonged stays of cargo, materials, or supplies, where their accumulation is permitted.

— STREAMLINES : Trajectory of the particles of fluid when it is flowing.

— STRUCTURE : The elements of a construction that form its resistant and supporting parts.

— STRUCTURE TYPE 1 : Structures where the loads act directly upon the structural elements.

— STRUCTURE TYPE 2 : Structures where the loads transmit their action to the structural elements through a load distribution.

— STRUCTURE TYPE 3 : Structures where the loads act upon a fill situated in back of the structure, which is indirectly loaded through an earth pressure increase.

— SURGE : Motion of a moored or free sailing ship, consisting of a translation along the longitudinal axis.

— SWAY : Motion of a moored or free sailing ship, consisting of a translation along the transverse axis.

— TANDEM AXIS : The two vehicle axes that support the chassis.

— TEU UNITS (TWENTY EQUIVALENT UNITS) : Measurement of the loading capacity of container ships, expressed in number of standard twenty foot containers.

— TIDE : Periodical movement of rising and lowering of the sea level.

— TIDAL RANGE : The difference in height between consecutive high and low waters.

— TRAFFIC TRACKS : A route designated exclusively for the transport of cargos, materials or supplies from the operation area to the storage area, and between the two to exterior areas of the port zone, both incoming and outgoing. Tracks designated for service traffic for the installations area are also considered traffic tracks.

— TRUCK CRANE : Crane upon wheels or treads that is capable of moving over any surface in an unrestricted way.

— TUG : Low displacement ship with high towing power at slow speed, used as an assistance during berthing maneuvers of larger ship by pushing or by towing with a wire towline.

— ULTIMATE LIMIT STATE : All those states corresponding to the structure's end of service by collapse, breakage, loss of stability or other forms of structure (or part of structure) failure.

— UNCONVENTIONAL FILL : Fill made up of products that originate from industrial or atypical residues (slags, flying ash, etc.).

— UNDERKEEL CLEARANCE : Vertical distance between the ship's keel and the bottom.

— VARIABLE LOAD : Loads whose magnitude and/or position is variable during the time of the work, and are of frequent or continuous form.

— WAVE HEIGHT : The vertical distance between a crest and the preceeding trough.

— WAVE PERIOD : Time interval between two consecutive wave crests in a monochromatic or regular wave train.

— WAVELENGTH : Horizontal distance between two consecutive wave crests in a monochromatic or regular wave train.

— WAVES : Superposition of monochromatic short period (<20-30 sec.) wave trains, generated mainly by the continuous action of wind.

— WORK HYPOTHESIS : Distinct work conditions of the resistant structure during the service phase, primarily taken into consideration in the calculation of the effects of action combinations.

— YAW : Motion of a moored or free sailing ship consisting of a rotation around the vertical axis.

## 1.4 UNIT SYSTEM

The unit system used in these Recommendations corresponds to the Legal Measurement Unit System in Spain, called the International Unit System (SI). The only exception being that used to express tonnage (t), due to its standard use in Spain in the measurement of loads and forces.

The basic unit system of the International System most commonly used in the civil engineering field are the following.

— Length : Meter (m)

— Mass : Kilogram (kg) or its multiple, the ton (t) (1t = 1000 Kg)

— Time : Seconds (s)

— Temperature : Degrees centigrade (∞C)

— Frequency : Hertz (Hz)

The tonnage-force relationship with the force unit in the International System (Newton-N-) is the following : 1t = 9.8kN.

## 1.5 NOTES

The conventional notes, abbreviations and symbols used in these Recommendations and their units are detailed in table 1.5.1.

## 1.6 REFERENCES

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*Calcul des effects du vent sur les constructions. Recommandations de la CECM.*
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**TABLE 1.5.1 THE BASIC CONVENTIONAL NOTES, ABREVIATIONS AND SYMBOLS USED IN THESE RECOMMENDATIONS**

I. LATIN CAPITAL LETTERS

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| A | Accidental load. | — |
| A | Area of the straight section of a cell. | $m^2$ |
| $A_k$ | Characteristic value of an accidental load. | — |
| $A_t$ | Longitudinal projected area of a ship exposed to wind. | $m^2$ |
| $A_{LC}$ | Submerged longitudinal area of a ship exposed to current. | $m^2$ |
| $A_T$ | Transverse projected area of a ship exposed to wind. | $m^2$ |
| $A_{TC}$ | Submerged transverse area of a ship exposed to current. | $m^2$ |
| $A_d$ | Design value of an accidental action. | — |
| $A_i$ | Typified load trains equivalent to restricted cargo handling equipment. | — |
| $A_t$ | Area equivalent to an isolated group of elements that form a non-continuous retaining earth structure. | $m^2$ |
| $A'_{LC}$ | Longitudinal wetted surface of a ship. | $m^2$ |
| $A'_{TC}$ | Transverse wetted surface of a ship. | $m^2$ |
| B | Transverse dimension of an isolated element in a non-continuous retaining structure, in the direction perpendicular to the action of earth pressures. | m |
| B | Beam of a ship. | m |
| $B_i$ | Typified load trains equivalent to unrestricted movement cargo handling equipment. | — |
| $B_t$ | Width equivalent to an isolated group of elements that form a non-continuous retaining earth structure. | m |
| C | Friction coefficient : travelling carriage/rails. | * |
| C | Construction Phase. | — |
| C1 | Construction Phase : manufacturing subphase. | — |
| C2 | Construction Phase : transport subphase. | — |
| C3 | Construction Phase : installation subphase. | — |
| C4 | Construction Phase : other subphase not expressly stated. | — |
| $C_v$ | Shape factor for the calculation of resultant wind force on a ship. | * |
| $C_{LC}$ | Shape factor for the calculation of longitudinal current force on a ship. | * |
| $C_{TC}$ | Shape factor for the calculation of transverse current force on a ship. | * |
| $C_b$ | Block coefficient of a ship. | * |
| $C_c$ | Configuration coefficient of a berth. | * |
| $C_d$ | Fixed nominal value for the checking of serviceability limit states. | — |
| $C_{dw}$ | Depth coefficient for the calculation of wave forces on a ship. | * |
| $C_e$ | Eccentricity coefficient of a berthing. | * |
| $C_{fw}$ | Waterplane coefficient for the calculation of wave forces on a ship. | * |
| $C_g$ | Geometric coefficient of a ship. | * |
| $C_m$ | Hydrodynamic mass coefficient of a ship. | * |
| $C_r$ | Friction coefficient for the calculation of current drag force on a ship. | * |
| $C_s$ | Stiffness coefficient of the berthing system. | * |
| D | Dynamic load. | — |
| D | Float design draught. | m |
| D | Draught of a ship. | m |
| D | Number of standard loads equivalent to a cargo handling equipment. | PAWLS |

TABLE 1.5.1 (Continued)

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| D' | Ship projection length in the direction of incident waves. | m |
| DWT | Dead weight tonnage | t |
| E | Modulus of elasticity or Young's modulus. | $t/m^2$ |
| E | Static Load. | — |
| E | Occurence probability or risk. | * |
| E | Kinetic energy released by the ship during berthing. | t . m |
| $E_1$ | Energy absorbed by fender system 1. | t . m |
| $E_2$ | Energy absorbed by fender system 2. | t . m |
| $E_d$ | Effect produced by the design loads corresponding to serviceability limit states. | — |
| $E_{d,\,dst}$ | Effect produced by the destabilizing design loads. | — |
| $E_{d,\,est}$ | Effect produced by the stabilizing design loads. | — |
| $E_f$ | Energy absorbed by berthing system. | t/m |
| F | Action applied to a structure. | — |
| F | Sliding safety factor. | * |
| F | Reaction of the soil to the active or passive earth pressure wedge. | t/m |
| $F_I$ | Soil's reaction to active wedge I. | t/m |
| $F_{II}$ | Soil's reaction to active wedge II. | t/m |
| $F_{LC}$ | Component of the resultant current force in the longitudinal direction of the ship. | t |
| $F_{LV}$ | Component of the resultant wind force in the longitudinal direction of the ship. | t |
| $F_{LW}$ | Component of the resultant wave force in the longitudinal direction of the ship. | t |
| $F_{TC}$ | Component of the resultant current force, in the transversal direction of the ship. | t |
| $F_{TV}$ | Component of the resultant wind force in the transversal direction of the ship. | t |
| $F_{TW}$ | Component of the resultant wave force in the transversal direction of the ship. | t |
| $F_d$ | Design value of an action. | — |
| $F_h$ | Horizontal load produced by helicopters in normal operating conditions. | t |
| $F_i$ | Soil reaction to the active or passive wedge in zone i. | t/m |
| $F_k$ | Characteristic value of an action. | — |
| $F_{kinf}$ | Minimum characteristic value of an action. | — |
| $F_{ksup}$ | Maximum characteristic value of an action. | — |
| $F_v$ | Vertical load produced by helicopters in normal operating conditions. | t |
| $F'_{LC}$ | Longitudinal current drag force on a ship. | t |
| $F'_{TC}$ | Transverse current drag force on a ship. | t |
| G | Freeboard of ship. | m |
| G | Shear modulus of elasticity. | m |
| G | Permanent Load. | — |
| $G_1$ | Self weight. | — |
| $G_{1k}$ | Nominal or characteristic value of the self weight of the structural elements. | — |
| $G_2$ | Dead load. | — |
| $G_{2k}$ | Nominal or characteristic value of the non-structural element's dead load. | — |
| $G_d$ | Design value of a permanent load. | — |
| $G_k$ | Characteristic value of a permanent load. | — |
| $G_{kinf}$ | Minimal characteristic value of a permanent load. | — |
| $G_{kinf.i}$ | Minimal characteristic value of the permanent load i. | — |
| $G_{ksup}$ | Maximum characteristic value of a permanent load. | — |

**TABLE 1.5.1 (Continued)**

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| $G_{ksup.\ i}$ | Maximum characteristic value of the permanent load i. | — |
| GRT | Gross register tonnage. | t Morson ($2.83\ m^3$) |
| H | Height of a retaining structure. | m |
| H | Point or lineal horizontal overload. | t or t/m |
| H | Anchoring block depth with respect to the soil surface. | m |
| $H_{1/3}$ | Significant wave height. | m |
| $H_A$ | Additional horizontal action. | t/m |
| $H_{VL}$ | Longitudinal horizontal load per linear meter, due to wind, corresponding to each foot of the crane. | t/m |
| $H_{VT}$ | Transverse horizontal load per linear meter, due to wind, corresponding to each foot of the crane. | t/m |
| $H_a$ | Maximum storage or stacking height of cargo in port zones. | m |
| $H_b$ | Significant wave height at the mouth or entrance of a protected zone. | m |
| $H_s$ | Significant wave height. | m |
| I | Impact factor. | * |
| I' | Reduced impact factor. | * |
| K | Coefficient of earth pressure. | * |
| $K_a$ | Coefficient of active earth pressure. | * |
| $K_{ac}$ | Cohesion term for the evaluation of active earth pressures. | $t/m^2$ |
| $K_{aci}$ | Cohesion term for the evaluation of active earth pressures in layer i. | $t/m^2$ |
| $K_{ah}$ | Earth pressure coefficient to obtain the active pressure's horizontal component. | * |
| $K_{ai}$ | Active earth pressure coefficient in layer i. | * |
| $K_c$ | In general, cohesion term. Earth pressure coefficient (cohesion term) to evaluate lateral earth pressures produced by potentially unstable soils. | $t/m^2$ or * |
| $K_c^o$ | Earth pressure coefficient (cohesion term) to evaluate lateral pressures produced by potentially unstable soils with $\phi = 0$. | * |
| $K_{ch}$ | Cohesion term to evaluate horizontal earth pressures. | $t/m^2$ |
| $K_{cv}$ | Cohesion term to evaluate vertical earth pressures. | $t/m^2$ |
| $K_e$ | Eccentricity factor for the calculation of the resultant wind moment on a ship. | * |
| $K_{ec}$ | Eccentricity factor for the calculation of the resultant current moment on a ship. | * |
| $K_h$ | Horizontal earth pressure coefficient. | |
| $K_o$ | Earth pressure at rest coefficient. | * |
| $K_p$ | Passive earth pressure coefficient. | * |
| $K_{pc}$ | Cohesion term to evaluate passive earth pressures | $t/m^2$ |
| $K_{pci}$ | Cohesion term to evaluate passive earth pressures in layer i. | $t/m^2$ |
| $K_{ph}$ | Earth pressure coefficient to obtain the passive earth pressure's horizontal component. | * |
| $K_{pi}$ | Passive earth pressure coefficient in layer i. | * |
| $K_p$ | Lateral earth pressure coefficient to evaluate lateral earth pressures produced by potentially unstable soils. | * |
| $K_v$ | Vertical earth pressure coefficient to evaluate vertical earth pressures. | * |
| L | Design life. | y. or m. |
| L | Spacing between axes of isolated elements. | m |
| L | Ship's total length. | m |
| L | Length of the soil failure line. | m |
| $L_I$ | Length of the soil failure line in wedge I. | m |
| $L_{II}$ | Length of the soil failure line in wedge II. | m |
| $L_c$ | Length of rails in a slipway. | m |

TABLE 1.5.1 (Continued)

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| $L_e$ | Width of the fictitious equivalent structure used to obtain earth pressures upon isolated narrow elements. | m |
| $L_f$ | Period of time assigned to each of the project's phases. | y. or m. |
| $L_i$ | Number of movements of equipment i in the load state corresponding to the critical deterioration. | * |
| $L_p$ | Keel length of a ship. | m |
| $L_{pp}$ | Length between perpendiculars of a ship. | m |
| $L_t$ | Length between the centers of extreme elements of non-continuous structures in the perpendicular direction to the earth pressure. | m |
| $L_w$ | Wave length at site's depth. | m |
| $M_{TC}$ | Resultant current moment on a ship relative to a vertical axis at the center of gravity. | t . m |
| $M_{TV}$ | Resultant wind moment on a ship, relative to a vertical axis at the center of gravity. | t . m |
| $M_d$ | Ship's mass. | t |
| $M_w$ | Water mass moved by the ship. | t |
| N | Number of applications accumulated from the standard load during the analyzed project phase equivalent to the entire spectrum of handling equipment. | PAWLS |
| N | Decrease Factor. | * |
| $N_i$ | Number of applications accumulated from the standard load during the project phase equivalent to equipment i. | PAWLS |
| $N_i$ | Number of actions of a determined group or number of accumulated standard loads necessary to produce critical deterioration of the structure. | — |
| $N_0$ | Stability number. | * |
| OCR | Overconsolidation ratio. | * |
| P | Point load, equivalent to the load per wheel of a restricted movement cargo handling equipment. | t |
| P | Wheel contact pressure. | $t/m^2$ |
| $P_a$ | Total active earth pressure. | t/m |
| $P_{aI}$ | Total active earth pressure in layer I. | t/m |
| $P_{aII}$ | Total active earth pressure in layer II. | t/m |
| $P_h$ | Maximum design helicopter weight. | t |
| $P_i$ | Slipway load. | t/m |
| $P_{k0}(x)$ | Maximum characteristic value of pretensioning force in the section x for t = 0. | t |
| $P_{k\infty}(x)$ | Mimimum characteristic value in the section x for $t = \infty$. | t |
| $P_0$ | Initial pretensioning forces (t = 0) in the origin (x = 0). | t |
| $P_0$ | Earth pressure at rest. | t/m |
| $P_p$ | Total passive earth pressures. | t/m |
| $P(x \leq x_i)$ | Non-exceedence probability for value $x = x_i$. | * |
| $P(x > x_i)$ | Exceedence probability for value $x = x_i$. | * |
| PAWLS | Standard port area wheel load. | 12 t |
| Q | Variable load. | — |
| $Q_C$ | Construction load. | — |
| $Q_{C1}$ | External load during fabrication. | — |
| $Q_{C2}$ | External load during transport. | — |
| $Q_{C3}$ | External load during installation. | — |
| $Q_{C4}$ | Other external load not expressly mentioned. | — |
| $Q_{Ck}$ or $Q_{Cik}$ | Characteristic value of a construction load. | — |
| $Q_D$ | Deformation load. | — |
| $Q_{D1}$ | Prestress load. | — |

TABLE 1.5.1 (Continued)

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| $Q_{D2}$ | Thermal load. | — |
| $Q_{D3}$ | Rheologic load. | — |
| $Q_{D4}$ | Imposed movement load. | — |
| $Q_{Dk}$ or $Q_{Dik}$ | Characteristic value of a deformation load. | — |
| $Q_H$ | Hydraulic load. | — |
| $Q_{Hk}$ | Characteristic value of a hydraulic load. | — |
| $Q_L$ | Lineal surcharge on the surface parallel to the top of a retaining wall. | t/m |
| $Q_M$ | Environmental load. | — |
| $Q_{M1}$ | Wave action. | — |
| $Q_{M2}$ | Current action. | — |
| $Q_{M3}$ | Action due to tides and other water level variations. | — |
| $Q_{M4}$ | Wind action. | — |
| $Q_{M5}$ | Action due to atmospheric pressure. | — |
| $Q_{M6}$ | Action due to air and water temperature. | — |
| $Q_{M7}$ | Action due to precipitation. | — |
| $Q_{M8}$ | Action due to snow and ice. | — |
| $Q_{M9}$ | Seismic actions. | — |
| $Q_{Mk}$ or $Q_{Mik}$ | Characteristic value of an environmental action. | — |
| $Q_P$ | Point surcharge on the surface, behind a retaining wall. | t |
| $Q_T$ | Earth load. | — |
| $Q_{Tk}$ | Characteristic value of an earth load. | — |
| $Q_V$ | Variable use load. | — |
| $Q_{V1}$ | Stage and storage overload. | — |
| $Q_{V2}$ | Cargo handling equipment and installations overload. | — |
| $Q_{V3}$ | Traffic overload. | — |
| $Q_{V4}$ | Overloads for the design of pavements and yards. | — |
| $Q_{V5}$ | Ship operation overloads. | — |
| $Q_{Vk}$ or $Q_{Vik}$ | Characteristic value of variable use load. | — |
| $Q_d$ | Design value of a variable load. | — |
| $Q_k$ | Characteristic value of a variable load. | — |
| $Q_{kinf}$ | Minimum characteristic value of a variable load. | — |
| $Q_{kinf,i}$ | Minimum characteristic value of the variable load i, distinguished from the predominant load in the combination of actions. | — |
| $Q_{ksup}$ | Maximum characteristic value of a variable load. | — |
| $Q_{ksup,1}$ | Maximum characteristic value of the variable load considered to have the predominant effect. | — |
| $Q_{ksup,i}$ | Maximum characteristic value of the variable load i, distinguished from the predominant load in the combination of actions. | — |
| R | Passive earth pressure coefficient's reduction factor for various $\delta/\phi$ ratios. | * |
| R | Impact load due to berthing ships. | t |
| $R_L$ | Component in the ship's longitudinal direction of the resultant of the exterior forces upon the ship. | t |
| $R_T$ | Component in the ship's transverse direction of the resultant of the exterior forces upon the ship. | t |
| $R_V$ | Resultant wind force on a ship. | t |
| R-a | Longitudinal metacentric radius. | m |
| $R_d$ | Structure's response capacity. | — |
| S | Mooring tension | t |
| S | Service phase. | — |
| S1 | Service phase in normal operating conditions. | — |
| S2 | Service phase in extreme conditions. | — |
| S3 | Service phase in exceptional conditions. | — |
| S4 | Service phase in repair. | — |
| $S_d$ | Effects produced by actions acting upon a structure. | — |
| $S_f$ | Displacement at failure of a soil in a shear strength test. | mm |

TABLE 1.5.1 (Continued)

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| $S_r$ | Residual displacement of a soil in a shear strength test. | mm |
| T | Mean return period years or months. | y. or m. |
| T | Resultant per linear meter of the lateral earth pressures produced by potentially unstable soils. | t/m |
| T | Ship's depth. | m |
| T | Friction load due to berthing ships. | t |
| T | Maximum necessary pull for hauling a ship in a slipway. | t |
| $T_a$ | Anchoring force. | t |
| $T_i$ | Typified load trains equivalent to rail traffic. | — |
| $T_m$ | Total quantity of cargo handled or foreseen handled in the zone served by a maneuvering track or access track, or moved in the analyzed area. | t/year |
| $T_s$ | Significant wave period. | S |
| TEU | Number of equivalent twenty foot containers. | — |
| T ($X_i$) | Mean return period for variable $X_i$ years or months. | y. or m. |
| U | Total water pressure. | t/m |
| V | Ship's berthing velocity. | m/s |
| $V_C$ | Design horizontal current velocity at a depth of 50% of the ship's draught. | m/s |
| $V_{C1min}$ | Average current velocity calculated during 1 minute intervals. | m/s |
| $V_V$ | Basic design wind velocity at a height of 10 meters. | m/s |
| $V_{V1min}$ | Average wind velocity for 1 minute long gusts. | m/s |
| $V_{V15s}$ | Average wind velocity for 15 second long gusts. | m/s |
| $V_{V3s}$ or $V_{3s}$ | Average wind velocity for 3 second long gusts. | m/s |
| $(V_{V3s})_T$ | Average wind velocity for 3 second long gusts with return period T | m/s |
| $V_a$ | Soil volume occupied by air. | $m^3$ |
| $V_b$ | Component of ship's velocity normal to the berthing surface at the moment of impact. | m/s |
| $V_s$ | Soil volume occupied by solid particles. | $m^3$ |
| $V_w$ | Soil volume occupied by water. | $m^3$ |
| W | Wheel load in cargo transport and handling equipment. | t |
| W | Maximum load handled by the most characteristic cargo handling equipment. | t |
| W | Mean load transported by each heavy load vehicle. | t/vehicles |
| W | Weight, in general terms. | t |
| W | Weight of the active or passive sliding wedge. | t/m |
| $W_I$ | Weight of the active or passive sliding wedge I. | t/m |
| $W_{II}$ | Weight of the active or passive sliding wedge II. | t/m |
| $W_i$ | Weight of zone i in the active or passive sliding wedge. | t/m |
| X or $X_i$ | Specific value of a variable. | — |
| Z | Height of the piezometric level at a point. | m |
| $Z_i$ | Height of the piezometric level at point i. | m |

**TABLE 1.5.1 (Continued)**

II. LATIN SMALL LETTERS

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| a or $a_i$ | Length measure. | m |
| a | Permissible draught tolerance for a float. | m |
| a | Distance between impact point and ship's center of gravity. | m |
| a | Geometric parameter. | m |
| $a_d$ | Design value of a geometric parameter. | m |
| $a_{nom}$ | Nominal value of a. | m |
| b | Length measure. | m |
| b | Width of a berthing structure in the impact direction. | m |
| c or $c_i$ | Length measure. | m |
| c | Cohesion. | $t/m^2$ |
| $c_i$ | Soil cohesion in the i layer. | $t/m^2$ |
| $c_u$ | Undrained cohesion. | $t/m^2$ |
| c' | Effective cohesion. | $t/m^2$ |
| $c'_f$ | Breaking effective cohesion. | $t/m^2$ |
| $c'_r$ | Residual effective cohesion. | $t/m^2$ |
| d or $d_i$ | Length measure. | m |
| $d_{1i}$ | Thickness of permeable layer i. | m |
| $d_{2i}$ | Thickness of low permeability layer i. | m |
| $d_a$ | Thickness of cohesive layer a. | m |
| $d_t$ | Depth of tension crack in cohesive landfills. | m |
| e | Void ratio. | * |
| e | Fictitious thickness. | m |
| f | Berthing coefficient. | * |
| f | Properties of the structure's construction material. | — |
| $f_c$ | Frequency of a repeating load. | Hz |
| $f_d$ | Design value of a property of a structure's construction materials. | — |
| $f_k$ | Characteristic value of a property of a structure's construction materials. | — |
| $f_n$ | Natural oscillation frequency of a structure. | Hz |
| $f_n$ | Maximum unitary negative skin friction. | $t/m^2$ |
| g | Gravity acceleration. | $m/s^2$ |
| h | Tidal range (astronomical). | m |
| h | Piezometric height. | m |
| h | Height of an anchoring plate. | m |
| h | Height measure, in general terms. | m |
| h | Thickness of the load distribution layer. | m |
| h | Water depth at the site. | m |
| h | Height of the travelling carriage. | m |
| $h_L$ | Mean height of the longitudinal projected area of a ship's superstructure above deck. | m |
| $h_T$ | Mean height of the transverse projected area of a ship's superstructure above deck. | m |
| $h_w$ | Artesian water piezometric height. | m |
| $h_\theta$ | Thickness of the layer $\theta$. | m |
| h' | Height measure, in general terms. | m |
| i | Hydraulic gradient. | * |

TABLE 1.5.1 (Continued)

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| $k$ | Permeability coefficient. | cm/s |
| $k$ | Ship's gyration radius. | m |
| $k_a$ | Mean wave agitation coefficient. | * |
| $k_{1i}$ | Permeability coefficient of the permeable layer i. | cm/s |
| $k_{2i}$ | Permeability coefficient of the low-permeability layer i. | cm/s |
| $l$ or $l'$ | Length of contact between ship and structure or between ship and fenders. | m |
| $n$ | Number of years of observation of maximum annual values necessary for the determination of the extreme value distribution of a variable. | * |
| $n$ | Porosity, or percentage of soil volume occupied by voids. | % |
| $n_a$ | Percentage of soil volume occupied by voids filled with air. | % |
| $n_i$ | Number of data equal to or higher than $X_i$, when using a statistical determination model for a variable. | * |
| $n_i$ | Number of actions of a determined group that act during the analyzed phase, or number of equivalent fictitious actions accumulated during said phase. | — |
| $n_t$ | Number of acting load groups adopted in order to check the ultimate limit fatigue state. | — |
| $n_w$ | Percentage of soil volume occupied by voids filled with water. | % |
| $p_a$ | Active earth pressure. | t/m$^2$ |
| $p_{ah}$ | Horizontal component of the active earth pressure. | t/m$^2$ |
| $p_{ai}$ | Active earth pressure in layer i. | t/m$^2$ |
| $p_{av}$ | Vertical component of the active earth pressure. | t/m$^2$ |
| $p_h$ | Lateral earth pressure associated with soil deformations. | t/m$^2$ |
| $p_h$ | Unit horizontal earth pressure. | t/m$^2$ |
| $p_n$ | Perpendicular ensilation earth pressure upon a sloped plane. | t/m$^2$ |
| $p_p$ | Passive earth pressure. | t/m$^2$ |
| $p_{ph}$ | Horizontal component of the passive earth pressure. | t/m$^2$ |
| $p_{pi}$ | Passive earth pressure in layer i. | t/m$^2$ |
| $p_{pv}$ | Vertical component of the passive earth pressure. | t/m$^2$ |
| $p_t$ | Tangential ensilation earth pressure upon a sloped plane. | t/m$^2$ |
| $p'h$ | Horizontal ensilation earth pressure upon a sloped plane. | t/m$^2$ |
| $p'v$ | Vertical ensilation earth pressure upon a sloped plane. | t/m$^2$ |
| $p''v$ | Vertical ensilation earth pressure upon a horizontal plane. | t/m$^2$ |
| $q$ | Superficial or lineal uniform surcharge. | t/m$^2$ or t/m |
| $q_i$ | Lineal vertical overload transmitted by each foot of the crane or equivalent to a restricted mobility cargo handling equipment. | t/m |
| $r$ | Index : $\dfrac{\text{Cost of direct losses}}{\text{Investment}}$ | * |
| $r$ | Index : $\dfrac{\text{Width of the zone, route or truck route}}{\text{Width of the cargo handling equipment}}$ | * |
| $s$ | Length of the path of the water particle. | m |
| $s$ | Slope of a slipway plane. | * |
| $u$ | Unit hydraulic pressure. | t/m$^2$ |
| $u$ | Perimeter of a cell in contact with backfill material. | m |
| $v$ | Flow velocity. | m/s |
| $w$ | Humidity. | % |

TABLE 1.5.1 (Continued)

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| x | Horizontal movement on top of a retaining wall. | m |
| z | Distance from the top of soil behind a retaining structure to the point where the earth pressure is evaluated. | m |
| $z_i$ | Distance from the top of the i layer behind a retaining structure to the point where the earth pressures are evaluated. | m |
| $z_0$ | Critical ensilation depth. | m |
| $z_0$ | In a cohesive soil retaining wall, height at which tension stresses are produced. | m |
| $z_1$ | Distance from the top of a retaining earth wall to the phreatic level in the soil behind the wall. | m |

III. GREEKS LETTERS

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| $\alpha$ | Angle between the back of a retaining structure and the horizontal. | degrees |
| $\alpha$ | Quota of cargo handling on wheels, or quota of heavy vehicles. | % |
| $\alpha$ | Angle measure, as a general term. | degrees |
| $\alpha$ | Angle formed between the ship's longitudinal axis, considered from stern to bow, and the wind, current or wave actions upon the ship. | degrees |
| $\alpha$ | Coefficient used to calculate ship's light draught, using maximum draught. | * |
| $\alpha$ | Vertical angle of a mooring line with the horizontal plane. | degrees |
| $\alpha$ | Thermal expansion coefficient. | $°C^{-1}$ |
| $\beta$ | Ground surface angle with the horizontal. | degrees |
| $\beta$ | Coefficient used to calculate ship's light draught using maximum draught. | * |
| $\gamma$ | Individual or apparent specific weight. | $t/m^3$ |
| $\gamma$ | Angle between the line connecting the impact point of a berthing ship and the center of gravity and the speed vector of the ship (or the component of such vector transverse to the berth. | degrees |
| $\gamma_a$ | Decrease coefficient of the properties of the construction steel for checking limits states. | * |
| $\gamma_c$ | Decrease coefficient of the properties of the compressed concrete for checking limits states. | * |
| $\gamma_d$ | Dry specific weight. | $t/m^3$ |
| $\gamma_f$ | Safety or weighted factor of actions, taken to evaluate their design value based on their representative values. | * |
| $\gamma_{fa}$ | Safety factor for the characteristic values of the accidental actions. | * |
| $\gamma_{fg}$ | Safety factor for the characteristic values of permanent loads. | * |
| $\gamma_{fga}$ | Safety factor for the characteristic values of permanent loads for exceptional conditions. | * |
| $\gamma_{fga\ max}$ | Safety factor for the maximum characteristic values of permanent loads for exceptional conditions. | * |
| $\gamma_{fga\ min}$ | Safety factor for the minimum characteristic values of permanent loads for exceptional conditions. | * |

TABLE 1.5.1 (Continued)

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| $\gamma_{fg\,max}$ | Safety factor for the maximum characteristic values of permanent loads. | * |
| $\gamma_{fg\,min}$ | Safety factor for the minimum characteristic values of permanent loads. | * |
| $\gamma_{fmax}$ | Safety or weighted factor of actions with unfavorable effects. | * |
| $\gamma_{fmin}$ | Safety or weighted factor of actions with favorable effects. | * |
| $\gamma_{fq}$ | Safety factor for the representative values of variable loads. | * |
| $\gamma_{fqa}$ | Safety factor for the representative values of variable loads for accidental combinations or exceptional conditions. | * |
| $\gamma_{fq\,max,1}$ | Safety factor for the maximum characteristic values of the variable load considered to have predominant effect. | * |
| $\gamma_{fq\,max,j}$ | Safety factor for the maximum representative values of the j variable. | * |
| $\gamma_{fq\,min,j}$ | Safety factor for the minimum representative values of the j variable. | * |
| $\gamma_i$ | Apparent specific soil weight in the i layer. | $t/m^3$ |
| $\gamma_m$ | Decrease coefficient of the properties of the materials for checking limit states. | * |
| $\gamma_p$ | Decrease coefficient of the properties of active reinforcing steel for checking limit states. | * |
| $\gamma_s$ | Decrease coefficient of the properties of passive reinforcing steel for checking limit states. | * |
| $\gamma_{sat}$ | Specific saturated weight. | $t/m^3$ |
| $\gamma_{sat\,i}$ | Specific saturated weight in layer i. | $t/m^3$ |
| $\gamma_v$ | Decrease coefficient of the properties of the connectors in composite concrete-steel structures for checking limit states. | * |
| $\gamma_w$ | Specific water weight. | $t/m^3$ |
| $\gamma_\theta$ | Apparent specific soil weight in layer $\theta$. | $t/m^3$ |
| $\gamma'$ | Submerged specific weight. | $t/m^3$ |
| $\gamma'_{ar}$ | Modified apparent specific weight of the submerged layer in zones with constant groundwater flow and hydraulic gradient. | $t/m^3$ |
| $\gamma'_i$ | Submerged specific weight in layer i. | $t/m^3$ |
| $\gamma'_r$ | Modified apparent specific weight of submerged soil in zones with constant groundwater flow and hydraulic gradient. | $t/m^3$ |
| $\Delta$ | Weight of the design ship. Displacement of a ship. | t |
| $\Delta_1$ | In areas with astronomical tides and fluvial flows, tidal range that corresponds to the low water level + maximum rise in outer water in 24 hours, corresponding to 1/4 of the return period associated with maximum admissible risk. | m |
| $\Delta_2$ | In areas with astronomical tides and fluvial flows, tidal range that corresponds to the flood level + maximum drop in outer water in 24 hours, corresponding to 1/4 of the return period associated with maximum admissible risk. | m |
| $\Delta K_a$ | Corrector coefficient of active earth pressures for $\delta/\phi \neq 0$. | * |
| $\Delta K_p$ | Corrector coefficient of passive earth pressures for $\delta/\phi \neq 0$. | * |
| $\Delta P_0(x)$ | Instantaneous losses of the prestressing force, in section x. | t |
| $\Delta P_{t=\infty}(x)$ | Deferred losses of the prestressed forces for $t = \infty$. | t |
| $\Delta T_a$ | Maximum decrease or increase of the air temperature (average monthly temperature) referring to the atmospheric temperature at the moment of joint closing or during the construction phase. | °C |
| $\Delta T_w$ | Maximum decrease or increase of the water temperature (average monthly temperature) referring to the water temperature at the moment of joint closing or during the construction phase. | °C |
| $\Delta Z$ | Level differences in outer waters. | m |
| $\Delta a$ | Additional safety factor for geometric parameters. | — |
| $\Delta p$ | Variation of the piezometric water level, with respect to the hydrostatic level at the point analyzed. | m |

TABLE 1.5.1 (Continued)

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| $\Delta p_{2i}$ | Variation of the piezometric level in low-permeability layer i, with respect to the hydrostatic distribution. | m |
| $\Delta p_a$ | Additional active earth pressures (+ or -). | $t/m^2$ |
| $\Delta p_i$ | Variation of the piezometric level at the analyzed point, with respect to the hydrostatic level. | m |
| $\Delta p_p$ | Additional passive earth pressures (+ or -). | $t/m^2$ |
| $\Delta p_v$ | Increment of the unitary vertical load upon a cohesive layer. | $t/m^2$ |
| $\Delta\theta$ | Characteristic global thermal variation of the construction. | °C |
| $\Delta\theta_{steel}$ | Characteristic global temperature variation in the metallic element. | °C |
| $\Delta\theta_{concrete}$ | Characteristic global temperature variation in the concrete element. | °C |
| $\Delta\theta_{max}$ | Maximum virtual temperature increase of the construction used for the calculation of thermal loads in reference to the atmospheric temperature at the moment of joint closing or during the construction phase. | °C |
| $\Delta\theta_{min}$ | Minimum virtual temperature decrease of the construction used for the calculation of thermal loads in reference to the atmospheric temperature at the moment of joint closing or during the construction phase. | °C |
| $\Delta'_1$ | In fluvial currents not affected by tides, maximum level rise in 24 hours, corresponding to 1/4 of the return period associated with the maximum admissible risk. | m |
| $\Delta'_2$ | In fluvial currents not affected by tides, maximum level drop in 24 hours, corresponding to 1/4 of the return period associated with the maximum admissible risk. | m |
| $\delta$ | Soil-structure friction angle. | degrees |
| $\delta_{max}$ | Maximum deformation of a fendering system during the berthing of a ship. | m |
| $\varepsilon$ | Percentage of heavy loaded vehicles. | % |
| $\varepsilon_t$ | Shrinkage or shortening of concrete. | m |
| $\theta$ | Angular measure. | degrees |
| $\theta_i$ | Angle between the horizontal projection of a mooring line and the longitudinal axis of a ship, considered from bow to stern. | degrees |
| $\lambda$ | Earth pressure coefficient for the silo effect. | * |
| $\lambda(x_i)$ | Average annual recorded data exceeding $x_i$. | * |
| $\mu$ | Friction coefficient between the ship's hull and the fenders. | * |
| $\nu$ | Poisson's ratio. | * |
| $\zeta$ | Soil failure surface angle with the horizontal. | degrees |
| $\zeta_i$ | Soil failure surface angle with the horizontal in layer i. | degrees |
| $\varrho$ | Air specific weight. | $t/m^3$ |
| $\sigma$ | Stress perpendicular to a plane or total vertical stress. | $t/m^2$ |
| $\sigma'$ | Effective vertical stress. | $t/m^2$ |
| $\tau$ | Shear strength. Tangential stress or shear stress. | $t/m^2$ |
| $\tau_f$ | Breaking shear stress in a shear strength test. | $t/m^2$ |
| $\tau_r$ | Residual shear stress in a shear strength test. | $t/m^2$ |
| $\phi$ | Internal friction angle. | degrees |

TABLE 1.5.1 (Continued)

| SYMBOL | DEFINITION | UNIT |
|---|---|---|
| $\phi$ | Angle between the ship's longitudinal axis, considered from stern to bow, and the direction of the wind resultant on the ship. | degrees |
| $\phi$ | Angle of the slope of a slipway. | degrees |
| $\phi_i$ | Internal friction angle in layer i. | degrees |
| $\phi_i$ | Angle of mooring line i with the horizontal plane. | degrees |
| $\phi'$ | Effective internal friction angle. | degrees |
| $\phi'_f$ | Breaking effective internal friction angle. | degrees |
| $\phi'_r$ | Residual effective internal friction angle. | degrees |
| $\psi_0$ | Factor defining the combination value of an action obtained from its characteristic value. | * |
| $\psi_{0,j}$ | Coefficient to obtain the combination value of variable j. | * |
| $\psi_1$ | Determination factor of the frequent value of an action obtained from its characteristic value. | * |
| $\psi_{1,1}$ | Coefficient to obtain the frequent value of the variable action considered to have the predominant effect. | * |
| $\psi_2$ | Determination factor of the quasi-permanent value of an action obtained from its characteristic value. | * |
| $\psi_{2,j}$ | Coefficient to obtain the quasi-permanent value of variable action j. | * |
| $\psi_d$ | Static load increase factor equivalent to the dynamic effect. | * |
| $\psi_i$ | Determination factors of the representative values of an action obtained from its characteristic value. | * |

IV. ABREVIATIONS

| ABREVIATION | MEANING |
|---|---|
| ADT | Average daily traffic intensity of heavy vehicles. |
| CEB | Eurointernational Concrete Committee. |
| CECM | European Committee on Metallic Construction. |
| CEDEX | Public Works Research Center. |
| CERC | Coastal Engineering Research Center. |
| DWT | Dead weight tonnage. |
| EH-88 | Spanish Code for the design and construction of unreinforced and reinforced concrete structures. |
| EP-80 | Spanish Code for the design and construction of prestressed concrete works. |
| FEM | European Fedration of Manutention. |
| FIP | International Prestress Federation. |
| GRT | Gross register tonnage. |
| HWSL | Maximum high water spring level. |
| IARH | International Association for Hydraulic Research. |
| IAPH | International Association of Ports and Harbors. |
| IN | Insignificant. |
| LNG | Liquied natural gas carrier. |
| LPG | Liquified petroleum gas carrier. |
| LWSL | Lowest water spring level. |
| MaxFL | Maximum flood level in fluvial currents, corresponding to the return period associated with maximum risk. |
| MeanFL | Mean of maximum annual flood levels in fluvial currents. |
| MeanSL | Mean summer level in fluvial currents. |

**TABLE 1.5.1 (Continued)**

| ABREVIATION | MEANING |
|---|---|
| MHW | Mean high water level. |
| MHWS | Mean high water spring level. |
| MinSL | Minimum summer level in fluvial currents, corresponding to the return period associated with maximum admissible risk. |
| MLWS | Mean low water spring level. |
| MPL | Mean phreatic level. |
| MSL | Mean sea level referred to hydrographic zero on the navigation charts. |
| NBE-AE | Spanish Basic Code : Building loads. |
| NBE-MV103 | Spanish Basic Code : Structural steel design. |
| OWL | Characteristic free outer water level. |
| PAWL | Standard Port Area Wheel Load. |
| PIANC or AIPCN | Permanent International Association of Navigational Congresses. |
| PDS-1 | Spanish Seismic Code. |
| ROM | Maritime Works Recommendations. |
| SLS | Serviceability or use limit state. |
| SPT | Standard penetration test. |
| TEU | Number of equivalent twenty-foot containers. |
| ULS | Ultimate limit state. |
| UNCTAD | United Nations Committee for Trade and Development. |
| UNE | Spanish Codes. |
| max | Maximum. |
| min | Minimum. |

**LEGEND:**

* : Adimensional.

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# SECTION 2 GENERAL CRITERIA OF THE PROJECT

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# SECTION 2 GENERAL PROJECT CRITERIA

## 2.1 PROJECT PHASES

**2.1.1** Defines project life of a structure as the period of time beginning at the start of construction and ending when the structure is dismantled, used for another purpose, or is no longer usable.

**2.1.2** The project life is divided into the following phases :

a) CONSTRUCTION PHASE

This phase consists of the period of time starting at the beginning of the construction of the structure until its initial use.

The following subphases shall be differentiated as follows :

C1. Manufacture :
Includes the manufacturing of the structure on shore and in the sea, submerged or afloat.

C2. Transport :
Consists of the transport of the structure, or part of it, on land, from land to sea or from land to barge and on the open sea; including mooring operations in protected waters.

C3. Installation :
This subphase includes the process of installing the structure in its final location until its entrance into service (e.g. sinking, anchoring, deposit...).

C4. Others :
This subphase includes all the procedures or construction situations that do not directly affect the execution of the resistant structure (e.g. improving the existing soil for the foundation, dewatering, etc).

b) SERVICE PHASE (S)

This phase consists of the period starting with the complete installation of the structure until it is dismantled, used for another purpose, or is no longer usable. Said period shall also be designated as the "design life".

This phase shall consider the following work hypotheses :

S1. Normal Operating Conditions :
When the maritime or port installation functions without limitations, unaffected by environmental conditions.

S2. Extreme Conditions :
When the installation must cease or limit its activities during environmental actions beyond the operating condition's limits. This phase is associated with the most severe environmental conditions for which the structure is designed.

S3. Exceptional Conditions :
When the installation is subject to unusual or extraordinary actions as a result of accidents, misuse, or overload, even if these actions are foreseen.

S4. Repair :
Includes the repair of the structure or maritime installation.

**2.1.3** For the projects included within the scope of these Recommendations all the phases, subphases, and work hypotheses that affect the dimensioning of the project shall be taken into account; carrying out a detailed and individualized analysis for each structure as a whole and each of its resistant elements, in all phases.

**2.1.4** The project engineer shall set the maximum duration of each one of the project phases that affect the design, giving special significance to the evaluation of :

— Safety levels of the structure in terms of factors or states dependant upon time : fatigue, corrosion, marine growths, long-term soil resistance, etc.

— Action and probability levels associated with return periods and Limit state design.

— Economic feasibility of the project and its future development possibilities.

If, during the construction phase, the execution deadline or the construction process contemplated in the design is modified, the effects of these modifications on the design loads and therefore on the dimensioning, shall be taken into account.

## 2.2 DESIGN LIFE (L)

**2.2.1** The design life shall be determined for each project, according to the time the structure is projected to be in service.

The evaluation of the design life shall take into account the possibility, ease, and economic feasibility of repairs, the probability and possibility of changes in the circumstances and conditions of the project's foreseen use as a consequence of operations or port traffic variations, and the viability of reinforcements and readaptations to the new service needs.

Given the character of maritime construction, it is not realistic to apply the prior strict criteria to the construction of works with very short foreseeable lives. The values in table 2.2.1.1. may be accepted as minimums in works of a definitive character and without specific justification, as a function of the type of construction or installation and of the level of safety required.

When different design lives for parts of the same work or structure are admitted, each one of them shall be calculated separately in function of the corresponding evaluation of actions.

TABLE 2.2.1.1 **MINIMUM DESIGN LIVES FOR WORKS OR STRUCTURES OF DEFINITIVE CHARACTER (in years)**

| TYPE OF WORK OR INSTALLATION | REQUIRED SAFETY LEVEL | | |
|---|---|---|---|
| | LEVEL 1 | LEVEL 2 | LEVEL 3 |
| GENERAL USE INFRASTRUCTURE | 25 | 50 | 100 |
| SPECIFIC INDUSTRIAL INFRASTRUCTURE | 15 | 25 | 50 |

**LEGEND :**

GENERAL USE INFRASTRUCTURE :
General character works : not associated with the use of an industrial installation or of a deposit.

SPECIFIC INDUSTRIAL INFRASTRUCTURE :
Works in the service of a particular industrial installation or associated with the use of transitory natural deposits of resources (e.g. industry service port, loading platform of a mineral deposit, petroleum extraction platform, etc).

LEVEL 1 :
Works and installations of local or auxiliary interest. Small risk of loss of human life or environmental damage in case of failure.
(Coastal defense and regeneration works, works in minor ports or marinas, local outfalls, pavements, commercial installations, buildings, etc).

LEVEL 2 :
Works and installations of general interest.
Moderate risk of loss of human life or environmental damage in case of failure.
(Works in large ports, outfalls of large cities, etc).

LEVEL 3 :
Works and installations for protection against inundations or of international interest.
Elevated risk of loss of human life or environmental damage in case of failure.
(Defense of urban or industrial centers, etc).

# SECTION 3 ACTIONS

# Index

# SECTION 3 ACTIONS

## 3.1. ACTION CLASSIFICATIONS

### 3.1.1 GENERAL

Actions are classified by

a) *Their time variation :*

G - Permanent Loads
Q - Variable Loads
A - Accidental Loads

b) *Their spatial variation :*

— Fixed Loads : Distribution over the structure is clearly defined by only one parameter.
— Mobile Loads : Within given limits, they can be arbitrarily distributed over the structure.

c) *The structure's response :*

E - Static Loads : Application does not produce significant accelerations in the structure or structural elements.

D - Dynamic Loads : Application produces significant accelerations in the structure or structural elements.

For the organization and application of the present Recommendation the action classification a) shall be adopted.

In the pertinent sections, the influence of the resistant structure's response is taken into account in the evaluation of actions and in the analysis methods.

### 3.1.2 PERMANENT LOADS

Permanent loads are essentially gravity loads that are present at every moment during the analyzed project phase, having a consistent position and magnitude or, if not consistent, of slow or non-existent variation in comparison with their mean values. Likewise, those loads whose variation takes place in only one direction until reaching a certain limit value, are also considered permanent loads.

They are divided into the following groups :

$G_1$ - Self Weight
$G_2$ - Dead Loads

### 3.1.3 VARIABLE LOADS

Variable loads are external loads whose magnitude and/or position is variable during the time of the work, and have a frequent or continuous form, and whose variation is not negligable in comparison to their mean value.

They are divided into the following groups :

$Q_H$ - Hydraulic Loads
$Q_T$ - Earth Loads
$Q_V$ - Variable Use Loads
$C_M$ - Environmental Loads

$Q_D$ - Deformation Loads
$Q_C$ - Construction Loads

### 3.1.3.1 HYDRAULIC LOADS

These are loads associated with water level and liquid load or the phreatic layers of other types of loads. They can be differentiated in the following two ways : Hydrostatic pressures, and hydrodynamic pressures associated with hydraulic gradients (filtration networks, consolidation processes, etc).

### 3.1.3.2 EARTH LOADS

Earth loads are pressures due to either the direct action of natural soils or fills, or other loads that act indirectly through the soil.

### 3.1.3.3 VARIABLE USE LOADS

These can be defined as loads associated with the service and normal use of the resistant structure, which can vary in position and magnitude during the analyzed phase.

They are divided as follows :

$Q_{V1}$ - Stage and Storage Overloads
$Q_{V2}$ - Cargo Handling Equipment and Installation Overloads
$Q_{V3}$ - Traffic Overloads
$Q_{V4}$ - Pavement and Yard Design Overloads
$Q_{V5}$ - Ship Operation Overloads

### 3.1.3.4 ENVIRONMENTAL LOADS

Loads due to natural climatic or environmental phenomena that act upon the resistant structure or upon elements that act on the structure.

They are divided as follows :

$Q_{M1}$ - Wave Actions
$Q_{M2}$ - Current Actions
$Q_{M3}$ - Actions due to tides and other water level variations
$Q_{M4}$ - Wind Actions
$Q_{M5}$ - Actions due to atmospheric pressure
$Q_{M6}$ - Actions due to air and water temperature
$Q_{M7}$ - Actions due to precipitation
$Q_{M8}$ - Snow and ice actions
$Q_{M9}$ - Seismic actions

### 3.1.3.5 DEFORMATION LOADS

Loads produced by imposed deformations.

They are divided as follows :

$Q_{D1}$ - Prestress
$Q_{D2}$ - Thermal
$Q_{D3}$ - Rheologic
$Q_{D4}$ - By Imposed Movements

### 3.1.3.6 CONSTRUCTION LOADS

These are loads that are specifically associated with the process of execution and construction of the resistant element.

They can be divided as :

$Q_{C1}$ - External Loads during Fabrication
$Q_{C2}$ - External Loads during Transport
$Q_{C3}$ - External Loads during Installations
$Q_{C4}$ - Other External Loads

### 3.1.4. ACCIDENTAL LOADS

Loads of fortuitous or abnormal character that can result from accidents, misuse, environmental conditions or exceptional work conditions.

They can be considered variable character actions with a low occurrence probability, or that only occur for short periods during the structure's design life, but when they do occur, they can significantly affect the safety of the structure.

It is advised to include in the calculations the accidental actions quantified in these Recommendations, whenever these actions can occur and are compatible with and relevant to the analyzed structure. The Project Engineer, the Client or the Government Authority may set greater values of accidental actions if deemed necessary for the design of the project.

The following are a few examples of Accidental Loads :

- Load Tests.
- Inundations due to the rupture of canalizations or deposits.
- Drainage system or subpressure control failures.
- Elevations of the phreatic level of ballasted projects.
- Pressures due to soil instability (e.g. potentially unstable slopes).
- Deposits and overdredging.
- Scouring or erosions of the soil due to ship propellers or extraordinary currents.
- Collisions and exceptional local overloads.
- Impacts and overloads due to exceptional maneuvers or operating situations of conventional freight transportation.
- Impacts and overloads due to exceptional maneuvers or operating situations of design ships.
- Overloads due to ship operations in exceptional load conditions.
- Wave overtopping
- Actions and overloads produced by exceptional environmental conditions : seismic actions and extreme seismic actions, heavy storms, hurricanes, etc.
- Explosions.
- Fire.

## 3.2. ACTION EVALUATION CRITERIA

### 3.2.1. GENERAL

All the actions defined in these Recommendations should be distinguished as either Representative Values or Design Values.

■ REPRESENTATIVE VALUES

The principal Representative Value of an actions is the Characteristic Value ($F_k$). It is defined as the value of the action associated with accident probability, assigned to each one of the phases and work hypotheses during the life of the project.

If necessary, the Maximum Characteristic Values ($F_{Ksup}$) and the Minimum Values ($F_{Kinf}$) of the same action, which can be zero, could be distinguished, for the purpose of taking into account the favorable or unfavorable effect of its performance.

The Characteristic values or actions shall be determined for each one of the project phases, with the possibility of differentiation according to their performance regarding different structural typology.

The values of the actions specified in these Recommendations shall be considered Characteristic Values. In their absence, the values given in the Codes and Instructions in effect, specialized brochures given directly by manufacturers, or suppliers of the equipment and installations shall also be considered characteristic values.

The project Engineers, Clients, or Governments Authorities may set other values as long as the minimum values given in the distinct Codes, Instructions, or Recommendations are observed.

Other Representative Values of an action are adopted depending on time level variations. These values shall be expressed in terms of the Characteristic Value ($F_k$) multiplied by a factor of $\psi_i$.

The following factors shall be distinguished :

— Combination Value : $\psi_0 \cdot F_k$.
— Frequent Value : $\psi_1 \cdot F_k$.
— Quasi-Permanent Value : $\psi_2 \cdot F_k$.

These values shall be included in the different load combinations foreseen in the Design Calculations for each load hypothesis, with the objective of considering the reduced probability of the combination of the Characteristic Values of various actions.

Other supplementary Representative Values expressed in the identical terms shall also be admitted in dynamic or fatigue analysis.

The factors $\psi_i$ shall be applied and quantified according to prior sections of this Recommendation, or in their absence, according to the specified factors in other applicable Norms and Instructions. Project Engineers, Clients, or Government Authorities may specify other values as long as they meet the minimum requirements of the normative in effect.

■ DESIGN VALUE

The Design Value or Weighted Value of an action ($F_d$) is the result of applying the appropriate safety coefficients ($\gamma_f$) to the Representative values ($F_k$).

$$F_d = \gamma_f \cdot F_k$$

Safety coefficients are included in the calculation to allow for certain factors. The following are examples of these factors : Possible unfavorable deviations from the representative values of the respective loads due to abnormal or unforeseen processes; lack of precision in the execution of the works in a way that can affect the external loads; possibility of inadequate modelling of the action; uncertainly regarding the methods to determine forces and possibility of unforeseen stress redistribution in the structure; considered evaluation of the limited state; requirement or safety levels of the considered structure; and degree of statistical reliability of the initial data.

The safety factors specified in Section 4: Calculation Bases of these Recommendations, are adopted and differentiated according to the Limited State that is verified, the type of load, the favorable or unfavorable effect on the performance, the Project phase and hypothesis of the considered work, the structure's construction material, the control level of the projected work and the foreseen damage in case of failure.

### 3.2.2 REPRESENTATIVE VALUES OF PERMANENT LOADS

For permanent loads (G) that are known to have large value dispersions, or that are expected to vary throughout the project life, maximum and minimum characteristic values should obligatority be distinguished. Usually, the maximum and minimum characteristic values are assimilated with those obtained from the corresponding statistical data base to comparable structural or non-structural elements associated with an exceedence probability of 5% to 95%, respectively. In the absence of other statistical indications, a normal distribution shall be assumed.

These values shall be replaced by a unique nominal value equal to the mean value ($G_k$) when the minimum and maximum values do not differ more than 5% from this value.

If the permanent load's effects is favorable, a minimum characteristic value equal to the following shall be adopted :

$$G_{Kinf} = 0.90 \cdot G_k$$

Other Representative Values of the permanent loads shall be considered equal to the design value ($\psi_i = 1$).

— The weight of the structural elements ($G_{1k}$) shall be represented by a unique nominal value, calculated by the dimensions of the project and the mean specific weight of the different elements and materials.

— The weight of the non-structural elements ($G_{2k}$ - Dead Loads) shall be represented by two nominal values, one maximum and the other minimum. These two values shall be determined taking into account all the modifications that can reasonably be expected. Normally, the minimum value shall be equal to zero.

The present Recommendation assigns, in the section corresponding to these loads, the characteristic specific weight of common structural materials and elements, to be used when there is no existing specific data about the project.

### 3.2.3 REPRESENTATIVE VALUES OF VARIABLE LOADS

#### 3.2.3.1 CHARACTERISTIC VALUES

■ GENERAL

The representative values of the variable loads (Q) should preferably be obtained from statistical data. For the majority of these loads, this data shall be the maximum periodic data (Extreme values).

The extreme value of a load corresponding to a mean return period (T), or associated with a probability or risk (E) during the period assigned in the project to each one the phases ($L_f$) is defined as the maximum design value of a variable load ($Q_{ksup}$).

The return period or recurrence of the variable value $X = X_i$ is the mean time interval when the extreme value surpasses $X_i$ only once.

The relation between the risk and mean return period shall be :

— For $L_f \geq 10$ years $E = 1 - (1 - (1/T)^{L_f}$ (Model I)

— For $L_f \geq 1$ year $E = 1 - e^{-(L_f/T)}$ (Model II)

With $L_f$ and T in years.

■ STATISTICAL DETERMINATION MODELS

— *MODEL I (Peak value method : Series of Maximum annual data)*

In model I, the relation between maximum projected values and their occurrence probability (Extreme Distribution) shall be obtained based on a series of maximum annual values observed during a sufficiently long period (n) of years; the greater the number of years studied the more reliable the procedure.
The probability that a value ($X = X_i$) is not exceeded in the period of a year ($P(X \leq X_i)$ shall be calculated using the following formula :

$$P(X \leq X_i) = 1 - P(X > X_i) = 1 - (n_i/(n+1))$$

given that :

n = nº of observed years
$n_i$ = nº of recorded data points with value equal to or greater than $X_i$.

The relation between the annual non-exceedence probability, and the return period, in years, shall be :

$$T(X_i) = 1/(1 - P(X \leq X_i))$$

— *MODEL II (Peak value method : Series of maximum data at any point on the time scale).*

In model II, the relation between projected maximum values and their occurrence probability (Exceedence Distribution) shall be attained based on a series of maximum recorded values during a sufficient long period (n) of years, gathering all the observed maximum values that exceed a certain limit value, and not only the maximum annual value.

The probability that the value ($X = X_i$) is not exceeded in the period of a year ($P(X \leq X_i)$) shall be obtained by means of the following formula :

$$P(X \leq X_i) = 1 - P(X > X_i) = 1 - (n_i/n), \qquad n_i \leq n$$

given that :

n = nº of years observed
$n_i$ = nº of recorded data points with value greater than $X_i$.

The return period, in years, for the value $X_i$ shall be :

$$T(X_i) = 1/\lambda = 1/(n_i/n) = n/n_i$$

With $\lambda$ being the annual mean of data points that exceed $X_i$.

For project phase durations of less than a year, model II shall be complemented in order to take into account the nonuniform frequency of events throughout the year.

To extrapolate the available data beyond the data recording periods, it is necessary to fit them to classical statistical distribution functions. The selected distribution function shall be that which best fits the available data, from among those that best represent the analyzed phenomenon's behavior (best correlation factor). Once the distribution function is selected the estimation of its parameters can be done by several methods. The most often used methods are : the moment method, the maximum likelihood method, and the graphic method (by least squares or visual giving more weight to the zone of interest). In general, when the analyzed sample is sufficiently well fit to the selected distribution function, it is not customary to find significant differences in the fit obtained by the different methods. The most often used distributions for the fit of extreme variables are : Weibull, Log-Normal, Exponential, Gumbel or Frechet ; customarily using distorted probability scales in order to fit a straight line to the distribution and therefore ease the fit. In table 3.2.3.1.1. a typical example is developed and applied to wave action.

The project engineer must take into account, when determining the design value of the variable load, the uncertainties (confidence interval) in the extrapolated variable values, due to the limited amount of available recorded data. Corrections of the obtained values by means of the fit are permitted. Except where justified (e.g. for high return periods in relation to the data recording period), values less than the upper limit of the estimation with a confidence band of 90% shall not be admitted.

The models I and II of statistic determination coincide for the high values P $(X \leq X_i)$, that is, high return periods, but they diverge for low probabilities or low return periods, therefore in those cases the exceedence distribution must be used. In any case, model II reduces the confidence intervals and therefore the uncertainty of the variable, especially if a reduced amount of recorded data is available.

■ ADMISSIBLE RISKS

The admissible Risks shall be set for each structure or structural element as a function of its physical and economic characteristics, the direct and indirect economic repercussions in case of partial or total incapacitation, and the estimation of human loss in case of destruction or failure for each significant phase of the project and work hypothesis.

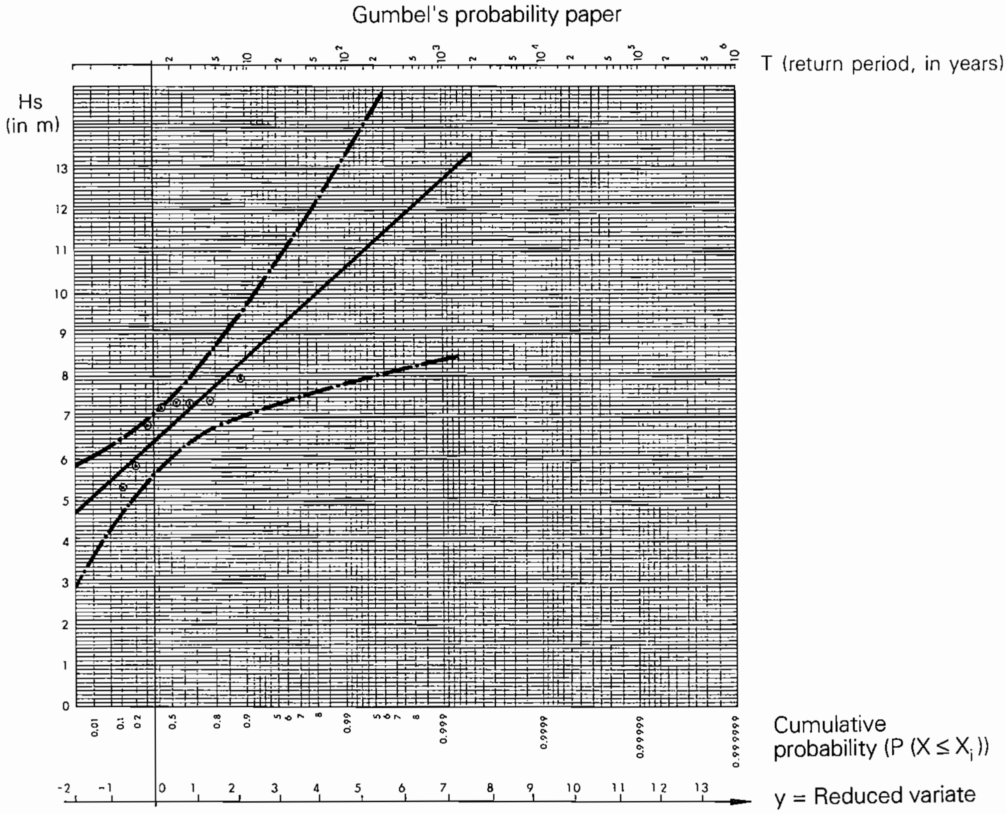
— *IN SERVICE PHASE (S)*

The maximum admissible risk for the Service Phase in extreme conditions, (Work hypothesis S2.) is given in table 3.2.3.1.2.

For normal operating conditions (S1), maximum admissible risks generally shall not be fixed, rather minimum operation levels for each installation, associated more with mean distributions than with extreme distributions, shall be fixed. These levels shall be used to define the use limit states of the installation. The characteristic value of the action in this work hypothesis shall be assimilated to that which results from the application of the servicability conditions and criteria.

Lacking specific studies on the project, the common serviceability limits given in these Recommendations shall be used for each action and port installation. Likewise, maximum admissible risks are given for statistically based actions that require normal operating conditions to be set with risk criteria (e.g. the berthing energy developed by the ship in normal operating conditions : Maximum admissible risk of 63%; that is, corresponding to a return period equal to the design life). Obviously, the normal operating conditions are only taken into account if the characteristic values of the actions in this hypothesis have return periods smaller than those adopted for extreme conditions.

TABLE 3.2.3.1.1 EXTRAPOLATION OF EXTREME VARIABLES BY MEANS OF CLASSIC STATISTICAL DISTRIBUTIONS.
MODEL I
EXAMPLE : FITTING OF AN EXTREME WAVE DISTRIBUTION (wave height) BY A GUMBEL DISTRIBUTION.




—— Extreme distribution fit
—·— Confidence distribution fit (95% confidence level)

PORT OF BILBAO (BOYA MORRO)
SEASONAL DISTRIBUTION

$H_s$: Significant wave height

| YEARS | $H_s$(m) | ORDER | PROBABILITY |
|---|---|---|---|
| 1976 | 7.90 | 8 | 0.8889 |
| 1978 | 7.50 | 7 | 0.7778 |
| 1980 | 7.35 | 6 | 0.6667 |
| 1982 | 7.30 | 5 | 0.5556 |
| 1979 | 7.25 | 4 | 0.4444 |
| 1984 | 6.87 | 3 | 0.3333 |
| 1981 | 5.85 | 2 | 0.2222 |
| 1977 | 5.35 | 1 | 0.1111 |

Generally, characteristic values of variable loads shall not be taken into account for the work hypothesis S3.- Exceptional Conditions. In this hypothesis, the variable action shall be represented by other representative values of the action (frequent or quasi-permanent values) according to those assigned in Part 4.- Calculation Bases. When characteristic values of variable loads in exceptional conditions are adopted, these actions shall be considered accidental loads.

Accidental loads shall be considered, therefore, those variable actions whose risk or occurrence probability during the design life is less than that adopted for the determination of the variable loads in extreme conditions.

For Repair (work hypothesis S4), risk criteria identical to that which is given in the Construction Phase shall be adopted.

— *IN CONSTRUCTION PHASE (C)*

Maximum admissible risks identical to those set for the Service Phase in extreme conditions can be adopted for the Construction Phase, considering medium (Cost/Investment) indexes and a reduced possibility of human loss.

| | Maximum Admissible Risk for Construction Phase |
|---|---|
| Damage initiation | 0.30 |
| Total destruction | 0.15 |

Either damage initiation or total destruction risk shall be applied according to the criteria set forth in table 3.2.3.1.2. for the Service Phase.

Lacking specific data regarding the duration of the Construction Phase or the different subphases, the data corresponding to a mean return period 50 times less than the admissible level in the project for the Service Phase shall be adopted as the characteristic value of the variable loads in the construction phase.

For construction phases or subphases of short duration, the characteristic values of the variable actions that are considered opportune may be justifiably adopted if adequate control of these values or their effects are established for construction works.

The level of adopted risk for the determination of characteristic values of variable loads may be reduced with respect to the maximum levels established as a function of economic optimization studies.

When the return period associated with the maximum admissible risk during the design life of the structure exceeds 1000 years, the prior criteria shall not be rigidly applied, as economic optimization studies would be necessary. In these cases, a return period of less than 1000 years shall not be adopted as the characteristic value.

The extreme distributions of vector type actions shall be differentiated according to each one of the possible directions, in order to consider in the calculations only those directions that can affect the resistant structure.
When an action is considered equally distributed in intensity and frequency in all directions; or when directional statistical data is not available, and it is not possible to determine a directional distribution by means of correlation with other statistical data, or with empirical forecasting and quantification models of the action, it shall be admissible to take the action distribution in a specific direction from the scalar distribution , multiplying the mean return period that corresponds to each value by a coefficient, dependant upon the physical properties of the action (e.g. for wind this coefficient is customarily taken as 2.50). Generally, this method covers the possible existence of dominant directions.

TABLE 3.2.3.1.2 MAXIMUM ADMISSIBLE RISKS TO DETERMINE CHARACTERISTIC VALUES OF VARIABLE LOADS IN THE SERVICE PHASE AND EXTREME CONDITIONS BASED ON STATISTICAL DATA

a) DAMAGE INITIATION RISK

| | | POSSIBILITY OF HUMAN LOSS | |
|---|---|---|---|
| | | REDUCED | EXPECTED |
| ECONOMIC REPERCUSSION IF WORK IS DISABLED<br>r Index = $\frac{\text{Cost of losses}}{\text{Investment}}$ | LOW | 0.50 | 0.30 |
| | MEDIUM | 0.30 | 0.20 |
| | HIGH | 0.25 | 0.15 |

b) TOTAL DESTRUCTION RISK

| | | POSSIBILITY OF HUMAN LOSS | |
|---|---|---|---|
| | | REDUCED | EXPECTED |
| ECONOMIC REPERCUSSION IF WORK IS DISABLED<br>r Index = $\frac{\text{Cost of losses}}{\text{Investment}}$ | LOW | 0.20 | 0.15 |
| | MEDIUM | 0.15 | 0.10 |
| | HIGH | 0.10 | 0.05 |

The damage initiation or total destruction risk shall be adopted as admissible risk according to the deformation characteristics and ease of repair of the resistant structure. For rigid or fragile works without possibility of repair, the total destruction risk shall be adopted.
For flexible, semi-rigid, or generally reparable works, (damages less than a preset level in function of the structure type) the damage initiation risk shall be adopted. In these types of works, the total destruction risk may also be adopted, defining the damage level to be considered as total destruction for each structure type. The resulting action shall be considered accidental.

**LEGEND :**

- POSSIBILITY OF HUMAN LOSS
  - — Reduced: When human loss is not expected in the case of failure or damage.
  - — Expected: When human loss is foreseeable in the case of failure or damage
- ECONOMIC REPERCUSSION IF WORK IS DISABLED

$$r \text{ Index} = \frac{\text{Cost of indirect or direct losses}}{\text{Investment}}$$

— LOW : $r \leq 5$
— MEDIUM : $5 < r \leq 20$
— HIGH : $r > 20$

■ OTHER DETERMINATION MODELS

In the absence of reliable or sufficient statistical data (minimum periods of data recording or 1/20 of the project's return period), the maximum characteristic values of the variable loads maybe determined based upon empirical methods of recognized validity, models developed for the forcasting and quantification of these actions, or they may be replaced by nominal values corresponding to the maximum permitted values established in the operation or use conditions for each one of the phases and work hypotheses established in the project. In the same way, if an action is limited by local physical conditions (e.g. wave braking due to depth limitations, geotechnical characteristics of the natural soil) or by design (e.g. fender stiffness, characteristics of a fill, initial prestressed loads) the design value of the action shall coincide with the nominal value of this limit.

The minimum characteristic values of the variable loads ($Q_{kinf}$) shall generally be taken as zero or negligible, except for hydraulic loads, earth loads and deformation loads, which shall obligatorily be differentiated by two characteristic values : one maximum and one minimum.

— Hydraulic loads ($Q_{Hk}$) shall normally be represented by two nominal values, one maximum and the other minimum, corresponding to the high and low water levels, or certain liquid ballast levels, in the different work phases and hypotheses.
Lacking more rigorous determinations, these Recommendations include the common characteristic levels of outer waters.

— Earth Loads ($Q_{Tk}$) shall be represented by unique nominal values, calculated based on the characteristic values of the geotechnical parameters of the soil. They shall usually correspond to the maximum values of active pressures, and to the minimum values of passive pressures. The minimum value of passive pressures shall be taken as zero in those cases where the development of passive pressures cannot be guaranteed. Lacking other data, the common characteristic values of the soil's geotechnical parameters included in these Recommendations shall be used in the indicated conditions.

— Generally, for the determination of characteristic values of variable use loads ($Q_{Vk}$), a sufficient or reliable statistical base will not be available. These actions shall be represented by nominal values corresponding to the maximum permitted values established in the project, or determinated by means of empirical methods or simulation models.

  The minimum values shall be considered zero, except for those loads that must obligatorily be maintained within certain limits.

  These Recommendations assign, in the corresponding sections, the minimum admissible nominal values or the theoretical methods of determination for these actions.

— The characteristic values of the environmental actions ($Q_{Mk}$) shall be determined, preferably based on statistical data that refers to the action's physical origin parameters.
Only in those cases where sufficient and reliable data is not available shall empirical methods or models developed for the projection or quantification of these actions be adopted.
Without disregarding other methods these Recommendations and subsequent Recommendations (ROM 0.3.- Consideration of Environmental Variables/I : Wave Action, Currents, Tides and other Water Level Variations; and ROM 0.4.- Consideration of Environmental Variables/II. Atmospheric and Seismic Conditions) include references to the more common methods and models. Likewise, for cases in which reliable and sufficiently elaborated statistical data is available, extreme distributions and maps of the origin parameters of the actions corresponding to preestablish periods of return are included.
In general, the minimum characteristic values of the environmental loads shall be considered zero.

— The loads produced by imposed deformations ($Q_{Dk}$) shall be represented by two unique nominal values : one maximum, and the other minimum, generally associated with the evolution of the action over the course of time.
The characteristic values of the deformation loads may be estimated, based on the mean values of the deformation parameters of the materials and on the methods of determination of the normative in effect. The deformation loads caused by differential settlement shall generally have a minimum characteristic value of zero.

— The characteristic values of the construction loads ($Q_{Ck}$) shall generally be represented for each constructive phase or subphase by a unique maximum value, applicable where their effect is unfavorable. The minimum values shall be zero.
In each case the importance and possibility of occurence of these actions as a function of the construction process shall be analyzed. The Project Engineer, Client or Government Authority shall then set its nominal value.

### 3.2.3.2 OTHER REPRESENTATIVE VALUES

The other representative values of the variable loads shall be chosen according to the following criteria :

— Combination Values, $\psi_0 \cdot Q_k$

Lacking other data, the values corresponding to a return period equal to 1/4 of the period fixed for the determination of the characteristic value may be taken as combination values of the action $Q_k$.

When the action is not fixed by means of a statistical base, the following $\psi_0$ factors may be adopted :

| ACTION | $\psi_0$ |
|---|---|
| $Q_H$ - Hydraulic Loads | 1.00* |
| $Q_T$ - Earth Loads | 1.00 |
| $Q_V$ - Variable Use Loads | 0.70 |
| $Q_M$ - Environmental Loads | 0.70 |
| $Q_D$ - Deformation Loads | 1.00 |
| $Q_C$ - Construction Loads | 1.00 |

* To obtain the combination values of Hydraulic Loads, the maximum and minimum values mean levels of the outer waters in tidal zones shall be taken as equal to 0.85 of the characteristic levels. This product shall be equivalent to the mean spring levels.

— Frequent Values, $\psi_1 \cdot Q_k$ :

Frequent values are those values that shall not be estimated according to their possibility of occurrence, but rather according to the frequency or duration of the occurrence.
Lacking other data, the following $\psi_1$ factors may be adopted :

| ACTION | $\psi_1$ |
|---|---|
| $Q_H$ - Hydraulic Loads | 1.00* |
| $Q_T$ - Earth Loads | 1.00 |
| $Q_V$ - Variable Use Loads | 0.60 |
| $Q_M$ - Environmental Loads | 0.30 |
| $Q_D$ - Deformation Loads | 1.00 |
| $Q_C$ - Construction Loads | 1.00 |

* To obtain frequent values of Hydraulic Loads, the maximum and minimum mean levels of the outer waters in tidal zones shall be taken as equal to 0.80 of the characteristic levels. This product shall be equivalent to the mean maximum and minimum water levels.

— Quasi-permanent values, $\psi_2 \cdot Q_k$ k :

Quasi-permanent values are those values that are generally determined as the mean value of the action during a period of time. Lacking other data, the following $\psi_2$ factors shall be adopted :

| ACTION | $\psi_2$ |
|---|---|
| $Q_H$ - Hydraulic Loads | 1.00* |
| $Q_T$ - Earth Loads | 1.00 |
| $Q_V$ - Variable Use Loads | 0.50 |
| $Q_M$ - Environmental Loads | 0.00 |
| $Q_D$ - Deformation Loads | 1.00 |
| $Q_C$ - Construction Loads | 1.00 |

* To obtain quasi-permanent values of Hydraulic Loads, the maximum and minimum mean levels of outer waters in tidal zones shall be taken as equal to 0.60 of the characteristic levels. This product shall be equivalent to the mean of all water levels.

### 3.2.4 REPRESENTATIVE VALUES OF ACCIDENTAL LOADS

Accidental loads (A) are variable actions whose risk or occurence probability during the design life is less than that adopted for the determination of the variable loads' characteristic values in extreme conditions ; or those that, independently of the evaluation criteria used, are applied for only short periods of time during the design life of the structure (e.g. seismic actions).

Project Engineers, Clients, or Government Authorities may choose the characteristic values of the accidental loads ($A_k$) as those values, above which, the survival of the structure cannot be guaranteed. These values shall not supercede the minimum values set by these Recommendations or other applicable specifications.

When sufficient and reliable statistical data is available, the extreme value whose exceedence probability in a year is equal to $10^{-4}$ (Mean return period T=1000 years) shall, lacking other criteria, be adopted as the characteristic value of the accidental load, unless the return period set to determine the characteristic value of the equivalent variable load in extreme conditions is greater.
In the absence of statistical data, these Recommendations set common nominal values for these actions.

The minimum characteristic values, as with the rest of the accidental loads' representative values, shall be considered zero.

## 3.3 DYNAMIC EFFECTS

### 3.3.1 GENERAL

The actions that act upon the Marine Works shall be considered dynamic loads if their application induces significant accelerations in the entire resistant structure or in individual structural elements.

The dynamic character of an action and its effects shall depend upon the magnitude of the load, its variation in time, and fundamentally, upon the response of the resistant structure, or movement of the structure caused by this action.

Two types of dynamic loads are distinguished :

— Frequencial Loads : These are actions that act upon the structure in a cyclical way, in regular time intervals, or in an irregular way such as a combination of cyclical loads of different characteristics (e.g. Loads due to wave action).

— Impact Loads : These are actions that act upon the structure, producing a response that reaches a maximum value in the initial moment; becoming smaller with each cycle until the rest position (e.g. Berthing loads).

The response of the structure shall be a function of its natural oscillation frequency in each direction and type (bending or torsion) and of its damping characteristics in relation to the type, direction and frequency of the acting load. In light of the above, two categories of structures and structural elements shall be considered :

— Rigid structures : Of high damping capacity and natural oscillation frequencies ($f_n$), corresponding to the fundamental mode in the same direction as the acting loads, that are high in relation to these loads ($f_c$) (e.g. vertical wall breakwaters).

  $f_n > 3f_c$ or

  $f_n > 4f_c$ if the damping capacity of the structure is low.

— Flexible Structures : With low damping capacity and low oscillation frequencies, corresponding to the fundamental modes. (e.g. Simple pile mooring dolphin)

$$f_n < 3f_c$$

### 3.3.2 FREQUENCIAL LOADS

The principle frequencial loads that act upon maritime works are :

— Actions due to waves
— Mooring forces induced by wave action or long waves upon the ship.
— Vortices generated by marine currents, wave action or wind.
— Vibrations due to traffic and fixed machinery and installations.
— Wind Forces.
— Seismic forces.

In general, dynamic effects produced by frequencial loads upon rigid structures shall not be considered.

Regarding flexible structures, dynamic effects shall be expected if the natural oscillation frequencies, corresponding to the fundamental mode in the direction of the acting load, are within the interval :

$$f_c/2 < f_n < 3f_c$$

The comparison of frequencies shall be carried out for the whole structure and for each of its individual structural elements in each of the significant project phases.

The natural oscillation frequency of a structure shall be determined by any of the following procedures :

— Tests on structures with characteristics equal or similar to those of the project.
— Testing on physical or mathematical models.
— Theoretical methods of mechanics and elasticity.
— Approximate or empirical formulas like Rayleigh, Vianello-Stodola, Dunkerley, ....

For the application of these procedures, the following shall be computed as the mass of the structure :

— The mass of the structure itself together with the marine growths that may be present.
— The mass of water contained inside the structure.
— The mass of water mobilized by the structure and its marine growths.

For the majority of the maritime structures, the consideration of dynamic effects due to frequencial loads may be simplified, assuming that the response of the structure can be treated by a static analysis, transforming the dynamic loads into an equivalent static system of loads.

Generally, the adopted equivalent static system shall consist of the application of the maximum value of the dynamic load (maximum amplitude in regular cyclical load ; and maximum amplitude of the equivalent cyclical load or value corresponding to the peak spectral period for irregular frequencial actions) multiplied by a safety factor, $\psi_d$.
Lacking other, more rigorous procedures, these Recommendations give equivalent static systems applicable for each load in those cases where the response of the structure allows such simplification. Likewise, these Recommendations include common frequency values, in the sections corresponding to the characteristic value of each frequencial action, in order to facilitate the consideration of dynamic effects for each structural type.

In those cases where the dynamic response of the structure is significant, or when the structure is complex or is designed using new technologies, the actual behavior of the structure can differ notably from the behavior anticipated from the action of the equivalent static system. In these cases, specific methods of dynamic analysis or model studies shall be used, introducing the frequencial loads from their spectral definitions (energy distribution of the action according to the different frequencies) (See ROM 0.3.- Environmental Variables/I.- Wave Action, Currents, Tides and Other Water Level Variations). The dynamic response of the structure may also be taken into account based on measurements on a prototype.

Generally, appreciable dynamic responses produced by frequencial actions will be present in thin structures ((height/width) ratio > 5 in the face perpendicular to the direction of the acting load), in structures located in open seas or in exposed environmental conditions, structures in sea waters of depth greater than 30 meters, or structures higher than 50 m on land, as well as floating structures in exposed environmental conditions.

### 3.3.3 IMPACT LOADS

The principal impact loads acting on maritime works are :

- Berthing loads
- Circulation, starting and braking of the freight equipment, truck traffic, and railway traffic.
- Accelerations and decelerations of the lifting movements of freight handling equipment (cranes).
- Pressures produced by breaking waves.
- Vertical impact forces due to waves acting upon horizontal elements close to the mean water level, produced by sudden immersion of an element (Wave slam).
- Forces induced by the freeing or breakage of loaded moorings.

In general, dynamic effects due to impact loads shall be considered for both rigid and flexible structures.

For most maritime structures, the consideration of dynamic effects due to impact loads may be simplified as long as the response of the structure, due to the load application, is susceptible to being analyzed in terms of static analysis through the simulation of these loads by an equivalent static system of loads.

Generally, equivalent static loads can be obtained by means of the simplification of the structure to a system with one mode of motion (approximation to a simple oscillator), and establishing an energy conservation equation between the actions before the impact, and the structure at the moment of maximum deformation (peak dynamic deformation). If other systems of energy absorption are incorporated in the resistant structure, (e.g. fenders in berthing structures) they shall also have to be included in the calculations, assuming that the lost energy is absorbed jointly by the resistant structure and the fender system. It may be assumed in the calculation that the maximum deformation of the fender system takes place at the same time as the maximum deformation of the resistant structure.

Rarely shall it be necessary to use more elaborate studies to take into account dynamic effects produced by impact loads. It shall be considered that the usual energy estimations of the impact shall cover the errors due to the simplification of the structure.

These simplifications shall be considered sufficient for structures where the impact coincides with its center of gravity, or for lineal structures where the point of impact is sufficiently separated from the extremes. For other types of structures, other methods of specific dynamic analysis (model studies, idealization as a multiple damper etc.) shall be adopted.

It is evident that the more rigid the structure, the more sensitive it is to impact loads, and therefore, the more severe the equivalent static load, as a consequence of the minor deformations of rigid structures for a given acting load.

Lacking other, more rigorous procedures, these Recommendations give applicable equivalent static systems for each load, and criteria for the formulation of the energy conservation equation (e.g. Berthing loads). In those cases where the application of this equation is not possible due to difficulties in quantifying the energy absorbed by the structure at the moment of maximum deformation, and in the absence of model or prototype studies, these Recommendations include theoretical-empirical methods of determination : Theoretical impact loads, additional loads, amplification factors, etc. (e.g. braking, starting and circulation of cranes, road and railway traffic).

## 3.4 CHARACTERISTIC VALUES OF ACTIONS

### 3.4.1 PERMANENT LOADS ($G_k$)

#### 3.4.1.1 SELF WEIGHT ($G_{1k}$)

■ DEFINITION

Self weight is defined as the load produced by the weight corresponding to the different resistant or structural elements.

■ DETERMINATION

The characteristic values of the action shall be deduced from the real dimensions (net transversal section) and individual or apparent specific weights ($\gamma$) corresponding to the different elements and materials in the conditions most unfavorable for the safety of the structure.

To determine the self weight, two types of elements or materials shall be distinguished:

— Simple elements (using individual specific weights).
— Compound elements (using apparent specific weights).

The element shall be considered compound when the porosity parameter (n) is relevant to the definition of its specific weight in every condition (e.g. natural soil, fills, rubble mounds, etc.).

When parts of the structure are completely, partially or intermittently submerged, it shall be preferable to consider the hydraulic loads (subpressures or buoyancy effect) as a system of loads applied independently of the Self Weight. The only exceptions being compound elements where the specific submerged weights ($\gamma'$) are used in the determination of the self weight, as long as significant hydraulic gradients do not exist, and therefore the existence of groundwater flow or consolidation processes are not considered in the calculations.

The relation between the distinct apparent specific weights, the porosity and the void ratio is given in table 3.4.1.1.1.

Lacking other data and specific information regarding the materials that are going to be used in the work, table 3.4.1.1.2 shall be used to obtain the most common unit and apparent specific weights. In this case the minimum characteristic value of the action shall be taken as :

$$G_{1kinf} = 0.9\ G_{1k}$$

For fills, the design specific weight values may also be obtained from this table, taking into account their dependence, especially in granular and unconventional fills on the way the material is deposited, subsequent degree of compaction, and possibility of material degradation over time :

— For the granular or unconventional fills placed in water, the specific weights corresponding to loose or slightly compacted states shall be taken.

— For hydraulic fills including drainage above the phreatic level, or for fills carried out by means of direct placement and subsequent compaction above the phreatic level, the values corresponding to the dense or compact state can be adopted.
For hydraulic fills, and as an accidental action, the portion situated above of the phreatic level shall be considered as a fluid with a high concentration of solid particles, during the time necessary for dissipation of the pore water pressure.

In this case, $\gamma = 1.2$ t/m$^3$ may be used.

— For rip raps and rubble mounds placed in water, the specific weights corresponding to the loose state shall be adopted.

— When the consolidation, compaction or improvement of the fill, above or below the phreatic level, is anticipated in the project, the specific weight used in calculations shall be in accordance with the construction procedures used.
Lacking more precise data, an increase of 7% in the specific weights given in the table for compact state shall be considered when the improvement of natural soils, and granular or unconventional fills using vibratory systems is foreseen in the project.

— In heavily compacted rip raps and rubble mounds, the possibility of their degradation over time shall be taken into account, with the consequent modification of their specific weights.

The self weight of a structural element, whose dimensions are to be determined in the calculations, shall be initially estimated, using previous dimensions, tables, empirical formulas or data of existing structures of similar characteristics. If the final design loads lead to dimensions of the resistant elements whose weights do not differ from those obtained in the prior estimates by more than 3%, a new calculation shall not be necessary, except in those cases where the self weight is critical for the analyzed element or structure.

When the self weight is critical for the element or structure, the Project's Technical Specifications shall include the corresponding clauses that impose the requirement that the referred weights are attained or are not surpassed during the execution of the work, as well as the different methods of control of these values in the work.

#### 3.4.1.2 DEAD LOADS ($G_{2k}$)

■ DEFINITION

A Dead Load is defined as the load produced by the weight of all the non-structural elements that are supported or included in the resistant structure as permanent loads, such as : construction elements, pavements, equipment, fixed installations, ballasts, fills and marine growths (algae, molluscs, crustaceans).

■ DETERMINATION

The characteristic values of the action shall be deduced from the real dimensions and the unit or apparent specific weights ($\gamma$) corresponding to the different elements and materials as well as the weights of the equipment and the fixed installations.

For the determination of the Dead Loads, all the criteria assigned in the section 3.4.1.1. referring to Self Weight shall be taken into account.

Likewise, lacking other data and specific knowledge of the materials that are going to be used or included in the project, the values of the most common specific weights shall be obtained from table 3.4.1.1.2. Weights of the equipment and the fixed installations shall preferably be obtained directly from manufacturers and suppliers, or through direct weighing of the corresponding elements.

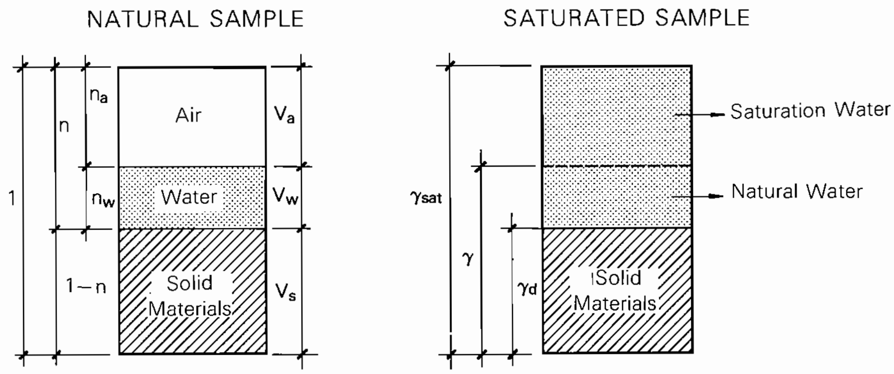
Given their small significance in relation to other calculation inaccuracy, the dead load due to the marine growths shall only be taken into account in those cases where their additional weight could be relevant to the structure (e.g. light structures such as light pontoons, floating piers, buoys, etc).
However, the existence of marine growths shall be considered to cause important modifications in the width and surface roughness, which must taken into account in the evaluation of some variable loads (e.g. drag and inertial loads caused by wave action upon

TABLE 3.4.1.1.1 RELATION BETWEEN THE DIFFERENT SPECIFIC WEIGHTS, POROSITY AND VOID RATIO

- Porosity: $n = n_a + n_w$
- Void ratio: $e = \dfrac{n}{1-n}$
- Apparent specific weight: $\gamma = (1-n) \cdot \gamma_s + n_w \cdot \gamma_w$
- Dry specific weight: $\gamma_d = (1-n) \cdot \gamma_s$
- Saturated specific weight: $\gamma_{sat} = (1-n) \cdot \gamma_s + n \cdot \gamma_w$
- Submerged specific weight: $\gamma' = \gamma_{sat} - \gamma_w$
- Water content: $w = \dfrac{n_w \cdot \gamma_w}{(1-n) \cdot \gamma_s}$

**LEGEND:**

$\gamma_s$ : Specific weight of the material that constitutes solid particles.
$\gamma_w$ : Specific water weight.




the piles). Marine growths in the construction phase shall not be considered, except if the length of the phase is longer than three years.

In the absence of more local and environmental information, the quantification of marine growths in Spanish coastal waters shall be made according to table 3.4.1.2.1.

In order to take into account the modifications that can reasonably occur in the work during the design life, two nominal values of the action shall be distinguished : Maximum Dead Loads and Minimum Dead Loads.

Given that, in general, the fixed equipment and installations are susceptible to withdrawals or location changes, the minimal characteristic value of the load ($G_{2kinf}$) shall usually be zero. In other cases, the minimal value shall be associated with the minimal levels or dimensions (e.g. minimal levels of landfills and ballasts). These values shall be applicable in those cases where their effect is favorable.

**TABLE 3.4.1.1.2 COMMON UNIT OR APPARENT SPECIFIC WEIGHTS AND POROSITIES OF CONSTRUCTION AND STRUCTURAL ELEMENTS**

| A-BASIC ELEMENTS | | $\gamma$(*) (t/m$^3$) | $\gamma_{sat}$ (t/m$^3$) | n (%) |
|---|---|---|---|---|
| A1-WATER | - Fresh | 1.00 | — | — |
| | - Salt | 1.03 | — | — |
| A2-BITUMINOUS BINDER (25°C) | | | | |
| | - Tar | 1.30 | — | — |
| | - Bitumens and Emulsions | 1.10 | — | — |
| A3-WOOD | | | | |
| | - Soft Dry | 0.60 | — | — |
| | - Hard Dry | 0.90 | — | — |
| | - Wet | 1.10 | — | — |
| A4-CERAMIC AND SIMILAR MATERIALS | | | | |
| | - Paving Tile | | | |
| | - Cement | 2.10 | — | — |
| | - Earthenware | 1.70 | — | — |
| | - Ceramic | 1.80 | — | — |
| | - Fibrecement | 2.00 | — | — |
| | - Brick | | | |
| | - Hollow ceramic | 1.10 | — | — |
| | - Perforated ceramic | 1.40 | — | — |
| | - Solid ceramic | 1.80 | — | — |
| | - Solid silicocalcium | 1.90 | — | — |
| A5-METAL | | | | |
| | - Steel | 7.85 | — | — |
| | - Aluminium | 2.70 | — | — |
| | - Bronze | 8.50 | — | — |
| | - Copper | 8.90 | — | — |
| | - Tin | 7.40 | — | — |
| | - Casting | 7.30 | — | — |
| | - Brass | 8.50 | — | — |
| | - Lead | 11.40 | — | — |
| | - Zinc | 7.20 | — | — |
| A6-ROCK | | | | |
| | - Sandstone | 2.60 | — | — |
| | - Basalt | 2.80 | — | — |
| | - Limestone | 2.80 | — | — |
| | - Chalk or Porous Limestone | 2.00 | — | — |
| | - Diorits | 2.80 | — | — |
| | - Gneiss | 3.00 | — | — |
| | - Granite | 2.80 | — | — |
| | - Lapilli (Volcanic) | 2.50 | — | — |
| | - Loam | 2.30 | — | — |
| | - Marble | 2.80 | — | — |
| | - Slate | 2.40 | — | — |

| B-CONSTRUCTIONS | $\gamma$(*) (t/m$^3$) | $\gamma_{sat}$ (t/m$^3$) | n (%) |
|---|---|---|---|
| B1-BLOCK WALLS (According to block type) | 1.3/1.6 | — | — |
| B2-BRICK WALLS (According to brick type) | 1.2/2.0 | — | — |

| TABLE 3.4.1.1.2 (Continued) | | | |
|---|---|---|---|
| B3-GABION WALLS | 2.00 | 2,30 | 30 |
| B4-CONCRETE | | | |
| - Normal | | | |
| - Unreinforced | 2.30 | — | — |
| - Reinforced or prestressed | 2.50 | — | — |
| - With fibers | 2.40 | — | — |
| - Lightweight | 1.80 | — | — |
| - Epoxy | 2.30 | — | — |
| - Cyclopean | 2.00 | — | — |
| - Heavyweight | 3.00 | — | — |
| B5-MASONRY WITH MORTAR (Snecked, polygonal rubble, range work) (According to specific rock weight) | 2.4/2.7 | — | — |
| B6-DRY MASONRY (According to specific rock weight) | 2.5/2.8 | — | — |
| B7-ASHLAR MASONRY (According to specific rock weight) | 2.6/3.0 | — | — |
| C-PAVEMENTS | $\gamma$(*) (t/m$^3$) | $\gamma_{sat}$ (t/m$^3$) | n (%) |
| C1-GRANULAR LAYERS | 2.30 | — | 20 |
| C2-STABILIZED SOIL | 2.10 | — | — |
| C3-BITUMINOUS MIXES | 2.50 | — | — |
| C4-CONCRETE PAVEMENTS | 2.40 | — | — |
| C5-ROCK PAVING BLOCK | 2.60 | — | — |
| C6-CONCRETE PAVING BLOCK | 2.20 | — | — |
| D-NATURAL SOILS | $\gamma$(*) (t/m$^3$) | $\gamma_{sat}$ (t/m$^3$) | n (%) |
| D1-COARSE-GRAIN SOIL | | | |
| - Gravel | | | |
| - Dense | 1.80 | 2.10 | 30 |
| - Loose | 1.60 | 2.00 | 40 |
| - Sandy Gravel | | | |
| - Dense | 2.10 | 2.20 | 20 |
| - Loose | 1.80 | 2.00 | 30 |
| - Sand | | | |
| - Dense | 1.90 | 2.20 | 35 |
| - Loose | 1.60 | 2.00 | 35 |
| D2-COHESIVE SOIL | | | |
| - Silts and sandy/silty clay | | | |
| - Firm | 2.10 | 2.10 | — |
| - Soft | 1.90 | 1.90 | — |
| - Clays | | | |
| - Stiff (over-consolidated) | 2.10 | 2.10 | 40 |
| - Soft (normally consolidated) | 1.80 | 1.80 | 55 |

TABLE 3.4.1.1.2 (Continued)

| | | | |
|---|---|---|---|
| - Organic Sediments | | | |
| - High clay content | 1.60 | 1.60 | 60 |
| - Low clay content | 1.40 | 1.40 | 75 |
| - Peat | 1.30 | 1.30 | — |
| - Mud | 1.50 | 1.50 | — |
| E-FILLS | $\gamma$(*) (t/m$^3$) | $\gamma_{sat}$ (t/m$^3$) | n (%) |
| E1-RUBBLE MOUNDS AND RIP RAPS | | | |
| - Of open granulometry | | | |
| - Natural rubble mounds (According to the specific rock weight) | 1.9/1.4 | 2.3/1.8 | 37/40 |
| - Modified Cube | 1.2/1.3 | 1.7/1.8 | 45/50 |
| - Tetrapods | 1.1/1.2 | 1.7 | 50/55 |
| - Dolos | 1.0/1.1 | 1.6/1.7 | 55/60 |
| - Acropodos | 1.1/1.2 | 1.7 | 50/55 |
| - Rip raps | 1.6/1.8 | 2.0/2.2 | 40 |
| - Ballast | 1.60 | 2.00 | 40 |
| - Of closed granulometry (mine run, quarry detritus and selected soils) | | | |
| - Dense | 2.00 | 2.20 | 25 |
| - Loose | 1.70 | 2.00 | 35 |
| E2-GRANULAR AND COHESIVE FILLS | | | |
| - Gravels | | | |
| - Dense | 1.90 | 2.20 | 30 |
| - Loose | 1.70 | 2.10 | 40 |
| - Sands | | | |
| - Dense | 2.00 | 2.30 | 30 |
| - Loose | 1.80 | 2.20 | 40 |
| - Silts | 2.00 | 2.00 | — |
| - Embankments | 1.70 | 2.10 | 40 |
| E3-ANTROPIC FILLS | | | |
| - Compact urban debris | 1.30 | 1.50 | 20 |
| E4-UNCONVENTIONAL FILLS | | | |
| - Blastfurnace Slag | | | |
| - Granular | | | |
| - Dense | 1.30 | 1.60 | 30 |
| - Loose | 1.10 | 1.50 | 40 |
| - In pieces | | | |
| - Dense | 1.80 | 2.10 | 30 |
| - Loose | 1.50 | 1.90 | 40 |
| - Lapillis | | | |
| - Dense | 1.80 | 2.10 | 30 |
| - Loose | 1.45 | 1.80 | 35 |
| - Flyash | | | |
| - Dense | 1.30 | 1.70 | 40 |
| - Loose | 0.90 | 1.50 | 60 |
| F-OTHERS | $\gamma$(*) (t/m$^3$) | $\gamma_{sat}$ (t/m$^3$) | n (%) |
| F1-MARINE GROWTHS | 1.00 | — | — |

| TABLE 3.4.1.1.2 (Continued) |
| --- |
| NOTE :<br>(*) SPECIFIC UNIT OR APPARANT EMERGED WEIGHT<br>Composite materials can very considerably according to local conditions, and in particular according to water content. |

■ DYNAMIC EFFECTS

The project engineer shall ascertain from manufacturers and suppliers the frequency and energy levels (spectrum) of the fixed equipment and installations included in the work, for the consideration of dynamic effects produced by vibration, rotation, or impact, from the fixed machinery or equipment according to the established criteria in section 3.3. Dynamic Effects.

Generally, it is not probable that their frequencies would be close to the natural oscillation frequencies of the structures, and therefore, dynamic effects are not to be excepted. The vibration frequencies of fixed machinery and equipment are usually in the range of 25Hz to 50 Hz.

### 3.4.2 VARIABLE LOADS ($Q_k$)

#### 3.4.2.1 HYDRAULIC LOADS ($Q_{Hk}$)

■ DEFINITION

Hydraulic loads are defined as the loads produced by water and other liquids acting preponderantly as free outer waters, phreatic waters in fills and natural soils and ballasts, and whose levels remain invariable, or almost invariable in relation to the response time of the resistant structure (e.g. tides and other variations of sea level; hydraulic regime of fluvial currents, and artificial water level variations).

They shall be differentiated as : Hydrostatic pressures, and hydrodynamic pressures associated with hydraulic gradients (e.g. in groundwater flow, consolidation processes, etc).
The actions due to waves or currents shall not be considered hydraulic loads, neither in terms of variations of water levels nor of the dynamic aspects of the water in movement. These actions shall be considered as Variable Environmental Loads.

■ DETERMINATION

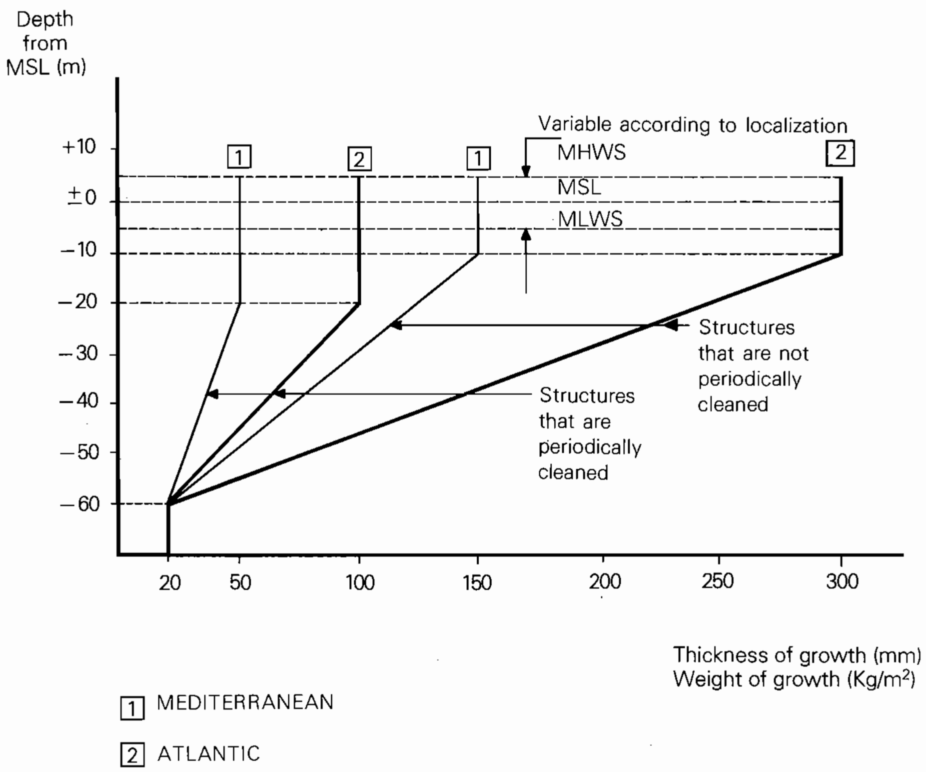
The hydraulic load, acting directly upon a superficial element of a construction, shall be a pressure in the direction normal to the surface that shall be considered as (u), with a value of :

$$u = \gamma_w \cdot Z$$

given that :

$\gamma_w$ = Specific weight of the water or other liquid.
When effects unfavorable to the safety of the structure are foreseen due to variations in the common specific weight of the water (differences in the salinity and temperature, high content of suspended solids), the project engineer shall investigate the causes and take them into consideration in the determination of hydraulic loads.
Lacking more information, in marine and fluvial zones affected by tides and storms, the specific weight of the marine water given in table 3.4.1.1.2, shall be adopted. For other liquids, the specific weights given in table 3.4.2.3.1.1, shall be adopted.

TABLE 3.4.1.2.1 QUANTIFICATION OF MARINE GROWTHS IN SPANISH COASTAL WATERS




MSL: Mean Sea Level
MHWS: Mean highest water spring level (0.83 HWSL)
MLWS: Mean lowest water spring level (0.83 LWSL)

**NOTE :**

Growths that are developed in fresh water shall be assumed to have weight and thicknesses 1/3 of that indicated for the Mediterranean, up to 2 m above than the Mean Summer Level.

Z = Height of the piezometric level in the point of determination.
Except for the cases where the existence of groundwater flow or consolidation processes are considered, the piezometric level shall coincide with the outer water level or the phreatic level.

The total hydraulic leads upon a construction shall be the resultant of the local loads upon the entire surface.
This generally leads to the determination of horizontal pressures and uplift (bouyancy) pressures as a result of the water level differences at the different structure surfaces.
The simplification of hydraulic load determination upon curved, brocken or irregular surfaces to regular surfaces with an average slope shall be admitted.

As indicated in section 3.4.1.1.- Self Weight, it is advisable to consider the hydraulics loads as external actions differentiated from the Self Weight and the Dead Loads. Determinations based on the reduction of the specific weight of submerged elements are only recommended in the case of compound materials (e.g. fills or natural soil), as long as no significant gradient pressures exist.

The levels used to obtain characteristic values of hydraulic loads shall be determined by the magnitude and frequency of free water oscillation; the existence of subterranean influx through the soil; the structural typology of the work; the permeability of foundations, natural soils, fills and structure; and the type and capacity of drainage systems and other forms of artificial variation of water levels foreseen in the project, as well as by the tolerances admitted for these cases in the Technical Specifications Dossier.
The Project Engineer shall fix the design levels, whenever possible, upon statistical data or experimentally, especially when significant groundwater flow, stratified soil, artesian pressures, or continued exposure to wave action exists.

— *FREE OUTER WATER LEVELS*

The extremes value associated with the maximum risk admissible for each phase of the project and work hypothesis shall be adopted as maximum and minimum free outer water levels.

The maximum and minimum free outer water levels in coastal zones are fundamentally due to the combination of astronomical tides, meteorological tides (storm-surge), seiches, wave-setup and hydraulic regime of fluvial currents in rivers, estuaries, river mouths, and fluvial ports.

Lacking sufficient or reliable statistical data, and given the unpredictability of all the causes and effects of variations in the outer water levels, the characteristic levels in table 3.4.2.1.1, may be adopted. These levels correspond to the combined effect of astronomical and meteorological tides in seas with astronomical tides; meteorological tides in seas without appreciable astronomical tides; astronomical tides and hydraulic regime of fluvial currents in zones with astronomical tides subjected to fluvial currents; and only hydraulic regime of fluvial currents in fluvial zones not affected by tides or storms. In the work hypothesis: Normal Operating Conditions, the Project Engineer, the Client or the Local Authorities shall set the characteristic levels as a function of the established use criteria. Lacking defined operating limits, those assigned in table 3.4.2.1.1, shall be used (e.g. for the definition of mooring loads in that hypothesis).

In confined natural areas (bays) or artificial areas (tidal basins), special care must be taken in checking for the possibility of resonant phenomenons due to the penetration of long waves. In these cases, there can be level alterations of up to 3.00 meters over those anticipated.

— *PHREATIC LEVEL IN FILLS AND NATURAL SOILS*

In general, in seas with astronomical tides, the phreatic level in fills and natural soils remains constant at the Mean Sea Level (MSL) + 0,3 m starting at a distance of approximately 20 m from the coastline.
In seas without significant astronomical tides, this level shall coincide with the mean sea level. For fluvial currents, whether affected by tides or not, or zones with tides subjected to fluvial currents, the phreatic level shall coincide with their mean summer or flood levels in function of the seasonal period. These levels can be subject to significant modifications when the following conditions exist :

— Subsurface flow due to internal currents or originating from direct rain, whose drainage is made difficult or impeded by extensive coastal structures.
— Artesian pressures.
— Low permeability layers behind retaining structures or in the foundation soil.
— Fills created by hydraulic methods.
— Continuous wave action upon the resistant structure, the fill or the natural soil.

Likewise, the use of artificial nourishment or drainage systems can modify the original projected water levels.

For the determination of hydraulic loads as a result of water level differences between the different structure surface, lacking more precise studies and as a simplification, horizontal saturation levels in fills and natural soils located behind these structures may be adopted. These levels for the most common cases, are given in table 3.4.2.1.2, as a function of the magnitude and frequency of free water oscillations and of the permeability of foundations, fills and structure; not considering the subsurface flow actions, wave action or other natural or artificial forms of level modification. For cases that are not given in the table, the determination of levels based on statistical or experimental data shall be necessary. In extreme cases, when this information is not available, sufficient or reliable it may be assumed that the phreatic level in the fill coincides with the maximum characteristic level of the free outer waters.

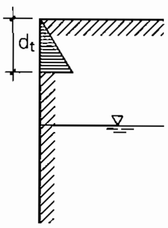
When the fill is placed by hydraulic methods, in order to consider that the drainage capacity during the placement may be exceeded, a saturation level that coincides with the lowest level at which the water can flow freely shall be taken for the construction phase.

The nourishment of water, exclusively originating from rainfall directly upon the exterior surface of the soil is taken into account in the unfavorable cases, assuming that it produce increases in the fill and natural soil saturation levels above the experimentally obtained levels or those from table 3.4.2.1.2. Lacking other data, only level increases due to direct rainfall upon highly permeable fills, located behind non-permeable structures, founded on low permeability soils or in stratified soils with layers of low permeability shall be considered relavant for the calculations. In this case, a level increase equal to the maximum intensity of precipitation in 24 hours expressed in terms of height/m$^2$ shall be adopted.

This intensity shall be determined for the specific site, adopting a return period equal to 1/4 of the return period selected for determining the characteristic values of the variable actions acting upon the project's structure.

In impermeable fills, increases in the phreatic level shall not be considered due to direct rainfall however, in the case of cohesive fills, an additional hydraulic load resulting from water penetration in the tension crack situated against the back of the structure, and above the phreatic level shall be considered. The depth of this crack shall be calculated according to the following formula :

$$d_t = \frac{2c_u}{\gamma_{sat}}$$




given that :

$c_u$ = Undrained landfill cohesion

$\gamma_{sat}$ = Specific saturated weight of the landfill.

In general, the horizontal levels adopted for the determination of the hydraulic loads shall also be valid for the determination of earth loads (pressures). Nevertheless, in those cases where the approximation to horizontal levels is not possible (e.g. structures with sloping fill, drained backfill and strong horizontal flow through the soil, the effects of the non-horizontal phreatic level shall be taken into account in the calculation of pressures following the trial-wedge procedure established in section 3.4.2.2. Earth loads.

— *ARTIFICIAL VARIATIONS OF OUTER WATER LEVELS AND PHREATIC LEVELS IN FILLS*

Where structures are subject to artificial variations of water level on their outer surfaces (e.g. dry docks, locks, etc.), the maximum and minimum water levels shall be determined, considering the specific use criteria established in the project. Also, the project shall fix the maximum water increase and decrease anticipated in 24 hours, that shall be necessary for the evaluation of hydraulic loads due to differences in phreatic level between structure surfaces.

## TABLE 3.4.2.1.1 CHARACTERISTIC WATER LEVELS OF FREE OUTER WATER IN SPANISH COASTAL ZONES

<table>
<tr><th colspan="2"></th><th>Sea with astronomical tides</th><th>Sea without significant astronomic tides</th><th>Zones with astronomical tides and fluvial currents</th><th>Fluvial currents unaffected by tides</th></tr>
<tr><td rowspan="2">Normal operating conditions</td><td>Max. level</td><td>HWSL</td><td>MSL + 0.5m</td><td>HWSL and MeanFL</td><td>MeanFL</td></tr>
<tr><td>Min. level</td><td>LWSL</td><td>MSL – 0.3m</td><td>LWSL and MeanSL</td><td>MeanSL</td></tr>
<tr><td rowspan="2">Extreme conditions</td><td>Max. level</td><td>HWSL + 0.5m</td><td>MSL + 0.8m</td><td>HWSL and MaxFL</td><td>MaxFL</td></tr>
<tr><td>Min. level</td><td>LWSL – 0.5m</td><td>MSL – 0.8m</td><td>LWSL and MinSL</td><td>MinSL</td></tr>
</table>

**LEGEND :**

HWSL : Highest water spring level.
LWSL : Lowest water spring level.
MSL : Mean sea level referred to hydrographic zero on navigational charts.

$$MSL = \frac{HWSL + LWSL}{2}$$

TIDAL RANGE (Astronomical) : h = HWSL - LWSL
MeanFL : Mean of maximum annual flood levels in fluvial currents.
MeanFL : Mean summer level in fluvial currents.
MaxFL : Maximum flood level corresponding to the return period associated with maximum admissible risk.
MinSL : Minimum summer level corresponding to the return period associated with maximum admissible risk.

Lacking more precise data, the following MSL and Tidal Range approximations can be adopted :

<table>
<tr><th>Maritime Front</th><th>Port</th><th>MSL (in m)</th><th>Tidal Range (in m)</th><th>Maritime Front</th><th>Port</th><th>MSL (in m)</th><th>Tidal Range (in m)</th></tr>
<tr><td rowspan="8">North</td><td>Pasajes</td><td>2.30</td><td>4.60</td><td rowspan="8">Galicia</td><td>Burela</td><td>2.15</td><td>4.50</td></tr>
<tr><td>Bilbao</td><td>2.25</td><td>4.60</td><td>Ferrol</td><td>2.10</td><td>4.50</td></tr>
<tr><td>Castro Urdiales</td><td>2.25</td><td>5.30</td><td>La Coruña</td><td>2.05</td><td>4.50</td></tr>
<tr><td>Santander</td><td>2.30</td><td>5.40</td><td>Malpica</td><td>2.05</td><td>4.00</td></tr>
<tr><td>San Vicente de la Barquera</td><td>2.30</td><td>5.20</td><td>Villagarcía</td><td>2.05</td><td>4.00</td></tr>
<tr><td>Gijón</td><td>2.30</td><td>4.60</td><td>Marín</td><td>1.90</td><td>4.00</td></tr>
<tr><td>Avilés</td><td>2.20</td><td>4.60</td><td>Vigo</td><td>1.95</td><td>4.00</td></tr>
<tr><td>Luarca</td><td>2.40</td><td>4.70</td><td></td><td></td><td></td></tr>
</table>

TABLE 3.4.2.1.1 (Continued)

| Maritime Front | Port | MSL (in m) | Tidal Range (in m) | Maritime Front | Port | MSL (in m) | Tidal Range (in m) |
|---|---|---|---|---|---|---|---|
| South Atlantic | Ayamonte | 1.75 | 3.60 | Canarias | San Sebastián de la Gomera | 1.15 | 2.40 |
| | Huelva | 1.85 | 3.70 | | La Estaca | 1.70 | 3.00 |
| | Sevilla (*) | 0.65 | 2.50 | | Pto. Luz | 1.50 | 3.00 |
| | Chipiona | 1.80 | 3.50 | | Mogán | 1.25 | 2.60 |
| | Rota | 1.80 | 4.00 | | Arrecife de Lanzarote | 1.50 | 3.00 |
| | Cádiz | 1.80 | 4.00 | | Pto. Rosario | 1.45 | 2.90 |
| | Barbate | 1.40 | 3.20 | Sta. Cruz Tenerife | Algeciras | 0.60 | 1.30 |
| | Tarifa | 0.70 | 1.60 | | Ceuta | 0.60 | 1.40 |
| Canarias | Sta. Cruz de Tenerife | 1.30 | 2.70 | | Málaga | 0.50 | 0.80 |
| | Los Cristianos | 1.20 | 2.50 | | Melilla | 0.30 | 0.60 |
| | Sta. Cruz de La Palma | 1.25 | 2.60 | | Almería | 0.30 | 0.60 |

NOTE :

(*) Approximately, the river flow raises the high water level by 0.6 mm and the low water level by 1.0 mm for each $m^3/s$. In extreme overflows, the river flow has reached 8000 $m^3/s$ (estimate). The summer river flow can go as low as 10 $m^3/s$.

Assuming that there are artificial water movements in the soil, such as in the case when artificial water drainage systems are adopted, the maximum and minimum water oscillation levels in the soil shall be determined, paying attention to the specific characteristics in each case.

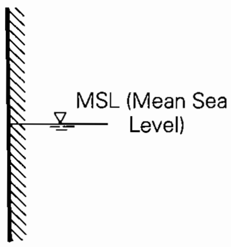
If the cause of the outer water oscillations is artificial, and only this water contributes to the water in the fill or natural soil, lacking other data, it shall be considered that the phreatic level in the soil in the long-term coincides with the level of the outer water, following their oscillations, with a limit lag in function of the permeability of the structure, the foundation and the fill, as follows :

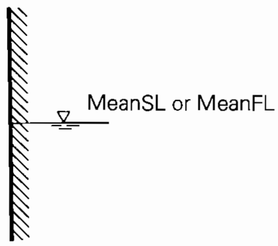
TABLE 3.4.2.1.2 PHREATIC LEVELS IN FILLS AND NATURAL SOILS TO DETERMINE HYDRAULIC LOADS ON RETAINING STRUCTURES

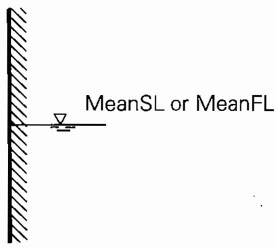
| Sea with or without astronomical tide | Zones with astronomical tides subject to fluvial currents | Fluvial currents unaffected by tides |
|---|---|---|

I. PERMEABLE FILLS

NON-PERMEABLE STRUCTURES FOUNDED IN LOW PERMEABILITY SOILS

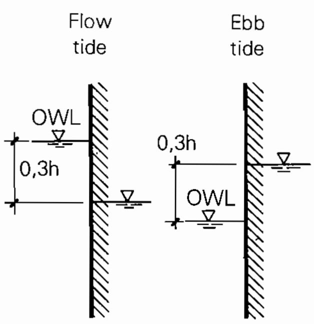


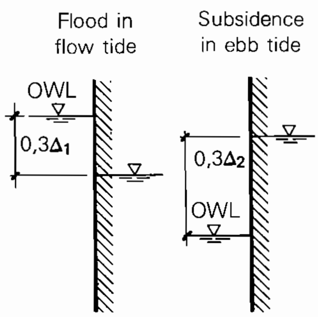



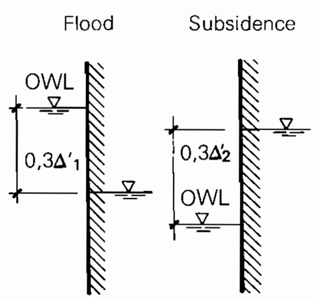




II. PERMEABLE FILLS

NON-PERMEABLE STRUCTURES FOUNDED IN PERMEABLE SOILS OR PERMEABLE STRUCTURES WITHOUT INTERRUPTIONS FOUNDED IN LOW PERMEABILITY SOILS

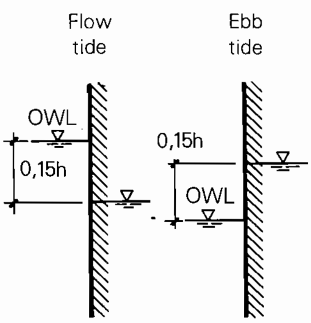


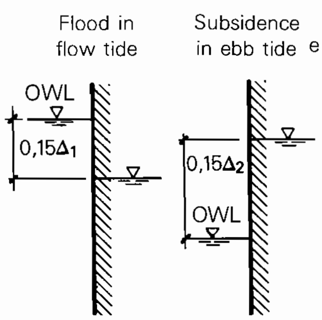



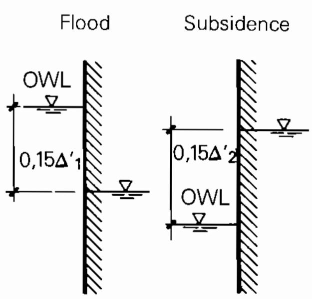




III. PERMEABLE FILLS

PERMEABLE STRUCTURES FOUNDED IN PERMEABLE SOILS



| TABLE 3.4.2.1.2 (Continued) |
|---|
| IV. LOW PERMEABILITY FILLS<br>NON-PERMEABILITY STRUCTURES FOUNDED IN HIGH OR LOW PERMEABILITY SOILS |
| THE SAME AS I |
| V. LOW PERMEABILITY FILLS<br>PERMEABLE STRUCTURES FOUNDED IN LOW PERMEABILITY SOIL |
| THE SAME AS II |

**LEGEND :**

OWL = Characteristic of free outer waters levels
h = Tidal range (Astronomical).
$\Delta_1$ = Tidal range that corresponds to the summer level + Maximum rise in outer water in 24 hours corresponding to 1/4 of the return period associated with the maximum admissible risk.
$\Delta_2$ = Tidal range that corresponds to the flood level + Maximum drop in outer water in 24 hours corresponding to 1/4 of the return period associated with the maximum admissible risk.
$\Delta'_1$ = Maximum rise in outer water in 24 hours corresponding to 1/4 of the return period associated with the maximum admissible risk.
$\Delta'_2$ = Maximum drop in outer water in 24 hours corresponding to 1/4 of the return period associated with the maximum admissible risk.

**NOTES :**

1. This table is intended for :
   - Low permeability fills or natural soils with a permeability coefficient of $k < 10^{-3}$ cm/s.
   - Permeable fills or natural soils with a permeability coefficient of $k \geq 10^{-3}$ cm/s.
2. When the permeability coefficient of a fill or natural soil is less than $10^{-7}$ cm/s, the soil shall be considered practically impermeable. When the material can be considered unsaturated (in the short term in compound elements, or in simple elements where the porosity is not a relevant parameter : unfissured rock) the hydrostatic actions in the retaining structure zones that come in contact with said material shall not be considered.
3. Permeable structures without interruptions are those whose permeability does not present physical interruptions that cut the flow of water upon reaching certain heights (e.g. Walls with weepholes).

*High Permeability Fill*

— Non-permeable retaining structures founded in low permeability soil: The phreatic level in the fill or natural soil does not vary when the outer water level changes suddenly.

— Non-permeable structures with foundations in high permeability soils or permeable structures without interruption founded in low permeability soil: Lag equal to 0.30 times the maximum foreseen outer water level variation in a 24 hour period.
— Permeable structures with foundations in high permeability soil: Lag equal to 0.15 times the maximum foreseen outer water level variation in a 24 hour period.

*Low Permeability Fill*

— The phreatic water level in fills or natural soils does not vary when the outer water level changes suddenly.

Possible water level reductions due to the establishment of drainage systems in a fill or natural soil shall only be taken into consideration if the drainage system's functioning can be checked and if cleaning and corrective measures of the system can be carried out at any time. Without these conditions, the hydraulic loads shall be determined as though no drainage system existed.

Assuming that the drainage is confined to small elements such as weep holes or flap valves, it shall be considered that the interruption height is situated 0.30 m above the real level. This value corresponds to the minimal water load necessary for these elements to function.
In these cases, differences of at least 1.00 m between outer water levels and the fill or natural soil's phreatic level shall be considered.

The possibility of drainage system failure shall be considered as an accidental load.

— *BALLAST LEVELS*

Assuming that the structure is subject to hydraulic actions produced by the existence, in its interior, of liquid ballast or a saturated fill, the project engineer shall set the performance levels, together with the admissible tolerances, which shall obligatorily be included in the project's Technical Specifications Dossier.

Usual tolerances of 10% in relation to the theoretic levels shall be considered as a consequence of the construction procedure or irregularities during the ballasting or unballasting phases.

When the structure is separated into different cells, the project engineer shall set maximum acceptable level differences between cells for each phase of the project. Usually, for construction of floating celled caissons, maximum differential levels between cells in the order of 1.00 m are set.

When the ballasting procedure is performed by hydraulic fill methods, it is reasonable to expect drainage exceedences during the fill placement, in which case, a phreatic level coinciding with the top of cell, or the lowest level at which the water can flow freely shall be adopted.

The elevation of the liquid ballast level, up to the top of the compartment, or to the lowest level at which the water can flow freely shall be considered as an accidental action.

In those cases where the outer waters may be in contact with the ballast (e.g. due to wave action), the project engineer shall take it into account and set the acting water levels accordingly.

For the determination of design water levels in structures that have a fill in their interior whose phreatic level is fed by outer waters or subject to artificial variations (e.g. to drainage systems), the criteria and simplifications established for retaining structures may be applied.

— *PIEZOMETRIC LEVELS IN FIXED STRUCTURES*

The determination of piezometric levels at each point on the structural element shall be carried out as a function of the performance levels of the free waters, ballast, and phreatic levels of fills and natural soils (as defined in prior sections); and as a function of the permeability characteristics of the structure, the fill and the foundation soil.

In each case, the combinations of levels in outer waters, fills and ballast, that are compatible with each other, and are most unfavorable for the resistant structure in the considered phase and work hypothesis shall be analyzed.

Once the water levels are fixed, the determination of piezometric levels at each point shall dictate if a groundwater flow is established or not, due to the water level differences on the structure's surfaces. The configuration of the groundwater flow net largely depends upon the permeability of the structure, fill and foundation soil, and as a result, is highly affected by the interposition of layers of differing permeability.

In the most usual cases, the established simplifications in table 3.4.2.1.3, may be adopted. This table is based on the assumption that water levels are horizontal and constant, and that variations in piezometric height in homogenous soils take place along the foundation base in massive structures (e.g. gravity piers) or along the penetration depth in narrow structures (e.g. sheet piling) (Case I of table 3.4.2.1.3), and in the low permeability layers in structures located in horizontally stratified soil with permeability varying greatly between alternate layers (Cases II and III of table 3.4.2.1.3).

For cases not given in table 3.4.2.1.3 (e.g. permeable structures, fills or natural soils that are clearly non-horizontally stratified, non-horizontal sea floor, sloped fills behind retaining wall structures, non-horizontal phreatic level in fills and natural soils, strong horizontal groundwater flows in soil, etc), the specific analysis of the groundwater flow net in function of the project characteristics shall be essential in evaluating piezometric heights at each point.

The analysis of the groundwater flow shall be carried out through the resolution of the Laplace equation in the filtration :

$$\nabla^2 h = 0$$

for the given boundary conditions.

This equation comes from the Darcy law :

$$v = -k \cdot i = -k \cdot (dh/ds)$$

and the continuity equations, given :

$h$ = Piezometric height.
$v$ = Flow velocity.
$k$ = Permeability coefficient.
$i$ = Hydraulic gradient.
$s$ = Length of the path of the water particle.

In a two dimensional system, the solution of the Laplace equation is represented by two families of orthogonal curves that constitute the groundwater flow (flow lines and equipotential or constant h lines). The resolution may be graphic, by drawing a groundwater flow net, analitycal, or through a physical or mathematical model, applying, among others, the following boundary conditions :

*Flow lines :*

— Impermeable stratum limit.
— Impermeable structure surfaces.
— Non-horizontal phreatic level.

*Equipotential lines :*

— Horizontal saturation or phreatic level.
— Sea or fluvial bottom.
— Submerged slope.
— Low permeability layer limits.

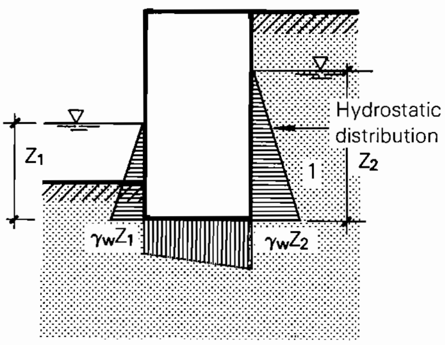
The most unfavorable piezometric levels for the calculation in fills or natural soils during the consolidation process (e.g. improvement of fills or natural soils behind retaining structures, using surchage techniques), may be obtained for each phase of the project by means of the application of the appropriate Soil Mechanics theories, especially those of Terzaghi-Fröhlich for primary consolidation.

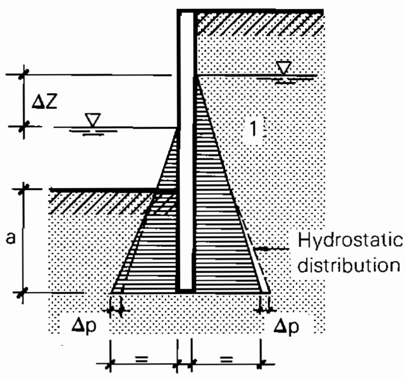
*— PIEZOMETRIC LEVELS IN FLOATING STRUCTURES OR IN FLOTATION*

The determination of piezometric levels in floating structures or structures in flotation in sheltered waters shall be carried out as a function of the float design draught (D) in the phase being considered, and of the acceptable draught tolerances for the same (a) given obligatorily in the Technical Specifications Dossier. These tolerances shall be included in the project in order to cover possible deviations with respect to the design draught, as well as alterations of the draught caused by the oscillations in the flotation of the structure in repose or during its launching and transport.

**TABLE 3.4.2.1.3 SIMPLIFIED PIEZOMETRIC LEVELS TO DETERMINE HYDRAULIC LOADS ON FIXED IMPERMEABLE STRUCTURES**

**I.** MASSIVE STRUCTURES NARROW STRUCTURES

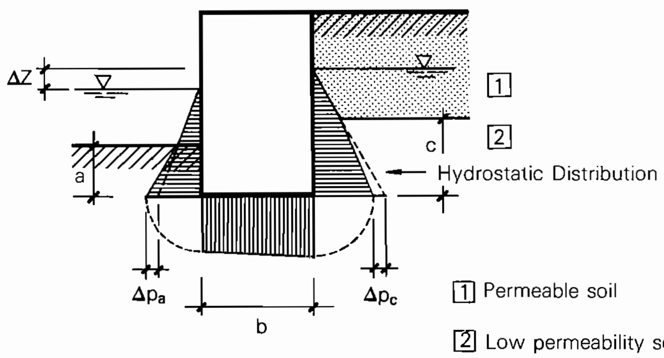



[1] Permeable soil

$$\Delta p = \gamma_w \cdot \frac{\Delta Z}{2}$$

**II.** MASSIVE AND NARROW STRUCTURES




[1] Permeable soil

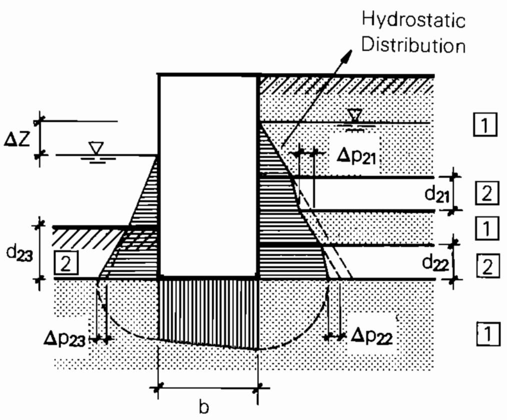
[2] Low permeability soil

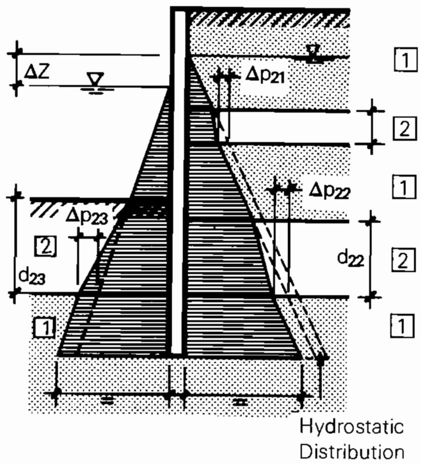
$$\Delta p_a = + \frac{\gamma_w \cdot a}{a+b+c} \cdot \Delta Z$$

$$\Delta p_c = - \frac{\gamma_w \cdot c}{a+b+c} \cdot \Delta Z$$

## TABLE 3.4.2.1.3 (Continued)

**III.** MASSIVE STRUCTURES — NARROW STRUCTURES




$$\Delta p_{2i} = \pm \Delta Z \cdot \gamma_w \cdot \frac{d_{2i}}{k_{2i}} \cdot \frac{1}{\sum \frac{d_{2\theta}}{k_{2\theta}} + \frac{b}{k_1}}$$

$$\Delta p_{2i} = \pm \Delta Z . \gamma_w \cdot \frac{d_{2i}}{k_{2i}} \cdot \frac{1}{\sum \frac{d_{2\theta}}{k_{2\theta}}}$$

given that:

[1] Permeable soil.

[2] Low permeability soil.

$k_{2i}$: permeability coefficient of low permeability layer i.

$k_1$: permeability coefficient of the permeable soil.

+ : For rising flow.
– : For descending flow.

**IV.** MASSIVE STRUCTURES

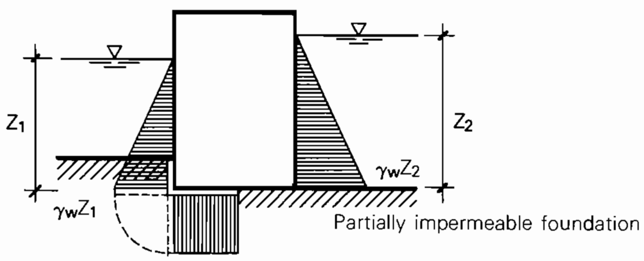
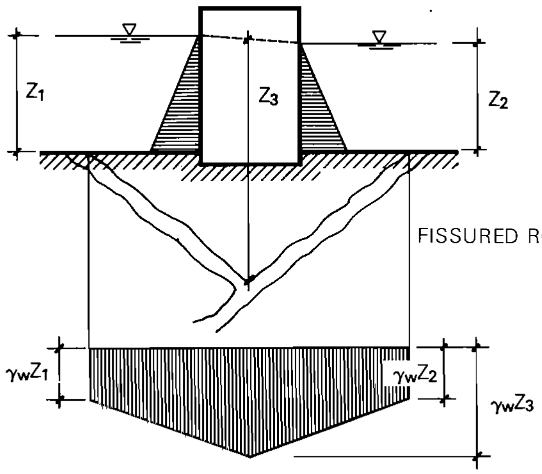
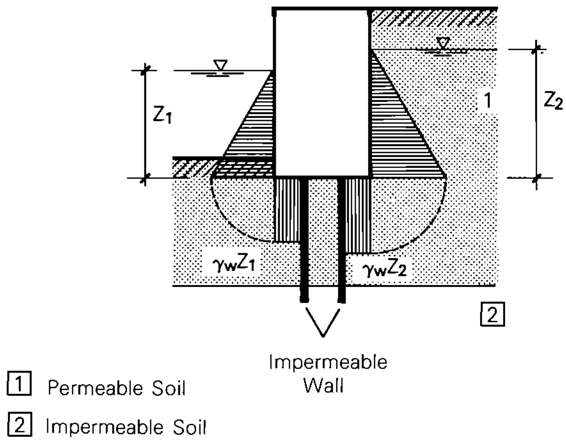


**TABLE 3.4.2.1.3 (Continued)**

V. MASSIVE STRUCTURES




VI. MASSIVE STRUCTURES




Usually a tolerance of 10% with respect to the theoretic draught, shall be considered.

For prefabricated caissons being launched or transported in sheltered waters, the usual tolerance is (+/-) 1.00 meter.

The exterior piezometric levels for the determination of hydraulic loads on floating structures are summarized in table 3.4.2.1.4.

For floating structures in open or exposed seas, the variations in the piezometric level produced by the oscillations of the float due to wave action shall be considered. These effects shall be quantified according to section 3.4.2.4.- Environmental Loads, and in ROM 0.3.- Environmental Variable Considerations /I : Wave action, currents, tides and other water level variations.

### 3.4.2.2 EARTH LOADS ($Q_{Tk}$)

■ DEFINITION

Earth loads are defined as the pressures, thrusts, and other forces created by a fill or natural soil upon the different elements of a resistant structure; or the reactions that such structures can cause in the soil to reach its equilibrium.

These forces shall be due to :

— The direct action of the soil.
— The indirect action of other loads through the soil.
— Actions induced by movements of the resistant structure.
— Actions induced by movements of the soil, independent of the analyzed structure (e.g. lateral pressures originating from asymmetrical loads upon soft layers, horizontal pressures due to instability phenomena, negative skin friction, etc).

■ DETERMINATION

The actions exercised by the soil upon a structure shall be determined fundamentally as a function of the following factors :

— Type of structure.
— Behavior of the structure : deformability.
— Soil-structure interaction : possibility of soil movement.
— Characteristic of the soil : physical characteristics of the soil (shear strength, deformability), geometry of the soil mass, stratification, degree of compactation and saturation and others.
— Indirect application of other loads through the soil or the structure.
— Phreatic levels.
— Modifications or variations of the pore water pressures with respect to the hydrostatic state : consolidation processes, existence of a ground water flow, drainage, pumping, ground water artesian pressures and others.
— Construction methods : construction process, compaction or densification process of fills or natural soils, hydraulic fills, etc.
— Possible modifications in the geometry of the soil mass and the conditions of use of the analyzed structure during its design life, over-dredging, scouring due to propellers, etc.

Due to the complexity of quantification of the aforementioned factors and their interrelation, there exists a large undetermined area in the evaluation of earth loads and their distribution along the resistant structure. For the structures included in the scope of these Recommendations, the following approximations may be adopted :

a) OUTER LOADS : EARTH PRESSURES

$a_1$) GENERAL

The denomination of the total earth pressure shall correspond to the action (or reaction) of the soil upon the surface of a resistant structure. For their determination, four types of resistant structures shall be distinguished :

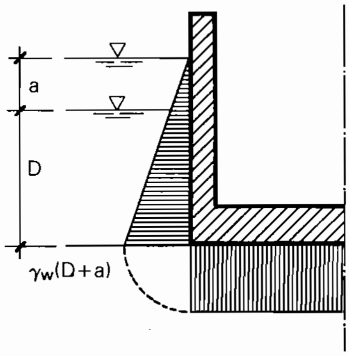
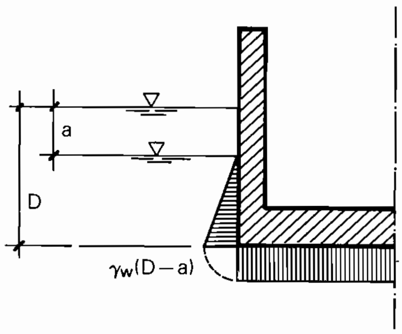
— Rigid wall structures, of indefinite length with respect to their transversal dimension, and whose lateral displacements are not restrained by external supports.
— Rigid wall structures of indefinite length, whose displacements are restrained by external supports.
— Flexible wall structures of indefinite length.
— Non-continuous structures.

$a_2$) INDEFINITE RIGID WALL STRUCTURES WITH LATERAL DISPLACEMENTS NOT RESTRICTED BY EXTERIOR SUPPORTS

If a soil acts upon an indefinite rigid wall without possibility of lateral displacement, the earth pressure at rest ($P_o$) shall be considered as acting.

If the wall moves in the direction of the earth pressure by the action of the soil weight, the earth pressure shall decrease as the displacement increases until the soil fails according to a curved surface that goes through the toe of the wall, in accordance with the shear equation or the Coulomb equation ($\tau = c + \sigma \tan \phi$); the progressive development

TABLE 3.4.2.1.4 EXTERIOR PIEZOMETRIC LEVELS FOR THE DETERMINATION OF HYDRAULIC LOADS ON FLOATING STRUCTURES

| | |
|---|---|
| MAXIMUM |   |
| MINIMUM |   |

**LEGEND :**

D : Float Design Draught.
a : Acceptable float draught tolerance given in the Technical Specifications Dossier.
$\gamma_w$ : Specific water weight.

of the shear strength ($\tau$) along this curve allows the reduction of the pressure. Beyond a certain value of displacement the pressure does not decrease more, since the soil shear strength reaches its maximum value. The value of the pressure in this moment shall be called "active pressure", and corresponds to the final state of equilibrium possible before a shear stress failure of the soil.
If, on the contrary, the wall is displaced in the opposite direction, landward, the soil shall fail according to another curve that also passes through the toe of the wall. The progressive mobilization of the shear strength along this curve shall allow an earth pressure increase as the displacement increases. Beyond a certain value of displacement the earth pressure does not grow any more, whatever its amplitude. The value of the earth pressure at this point shall be called the "passive pressure". In this state, the shear strength along the failure line is at its maximum.

For the calculation of the earth pressures in this type of structure, only the two limit pressures (active and passive) shall be considered, besides the earth pressure at rest.

It is pointed out that the earth pressures, and the consequent displacement of the structures necessary for its complete mobilization, are much higher in the case of passive pressures than in the case of active pressures. The amplitude of the displacements necessary to mobilize these earth pressures are given in table 3.4.2.2.1.

For the determination of earth pressures in rigid wall structures, it is assumed that the failure line goes through the toe of the structural wall, which is only possible when the structure and the backfill rest on a rigid base, or when the earth load in front of the wall is sufficient to impede movement below the toe of the wall. In those cases where doubts exist about the validity of this condition, it shall be expressly verified.

In order to facilitate the earth pressure calculations upon indefinite rigid wall structures, the horizontal or vertical earth pressures that soils produce upon a wall or surface element of a structure may be expressed as :

$$p_h = K_h \cdot \sigma' \pm K_{ch}, \text{ in t/m}^2$$

$$p_v = K_v \cdot \sigma' \pm K_{cv}, \text{ in t/m}^2$$

given that :

$K_h$ : Coefficient of horizontal earth pressure (dimensionless)

$K_v$ : Coefficient of vertical earth pressure (dimensionless)

$\sigma'$ : Effective vertical stress of the soil at the point where the earth pressures are evaluated, in $t/m^2$.

$K_c$ : Cohesion terms, in $t/m^2$

Generally these are considered zero due to the lack of apparent cohesion in granular soil, and the relaxation over time of the cohesive component of the shear strength in cohesive soils. The cohesion term shall only be taken into account in the determination of short-term earth pressures, and in the determination of long-term earth pressures produced by highly overconsolidated, undisturbed clays or by very dense fills of cohesive material whenever these soils are permanently protected from drying and frost, and are of low sensitivity (shear strength of an undisturbed sample/shear strength of a remolded sample) < 2).
It is recommended, in any case, to consider the soil without cohesion in a zone of approximately 0.5 to 1.00 m thickness from the surface.

The earth pressure coefficients may vary between the limit values corresponding to the failure states of the soil.

— *ACTIVE EARTH PRESSURES*

The active earth pressures correspond to the condition of soil failure at which the shear strength in the soil is fully mobilized in resisting gravity forces. The breaking wedge, produced by the lateral expansion of the soil, advances toward the structure and lowers, producing the settlement of the free surface of the soil. These pressures are the lowest that a soil mass acting upon a rigid wall can exert.

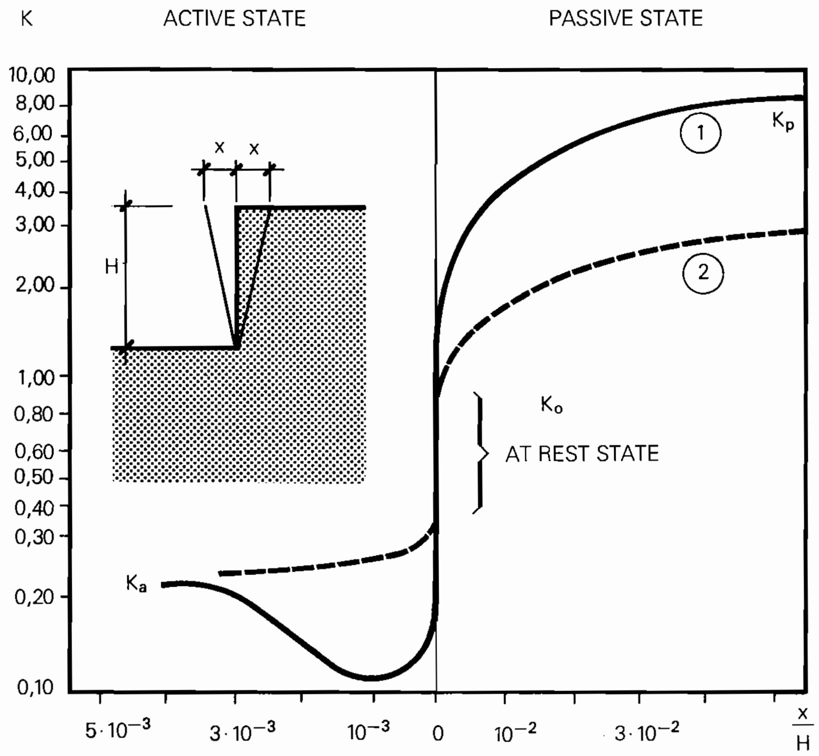
The action of active earth pressures on rigid wall structures with foundations in deformable soils shall be considered, if there is sufficient lateral displacement capacity (translation or rotation) in the direction of the pressures. The minimum displacements necessary for the actuation of active earth pressures are given in table 3.4.2.2.1.

The failure surfaces of the soil in the active state shall have continuous or discontinuous curvature in function of the conditions that limit the wall's movement; as has been determined by Caquot and Kerisel.
For the calculations of active earth pressure coefficients, simplified plane failure surfaces may be adopted according to the Coulomb approximation, given that these surfaces do not differ from the real sliding surfaces except for off-plum walls or for high wall-soil friction angles. These simplifications yield admissible non-conservative differences on the order of 4% for $\phi = 30°$, $\delta = \phi$, and vertical wall.

The practical development of the Coulomb Theory is shown in tables 3.4.2.2.2 and 3.4.2.2.3.

For the particular case of active earth pressure applied to vertical surfaces, produced by homogenous soils without groundwater or with horizontal phreatic level, with horizontal ground surface, and excluding the soil/wall friction, the calculation may be simplified by the application of the Rankine theory developed in table 3.4.2.2.4. Rankine's theory shall also be valid for homogenous soil without groundwater, with ground slope surface coinciding with the wall-soil friction angle.

TABLE 3.4.2.2.1 NECESSARY AMPLITUDE OF DISPLACEMENT IN RIGID WALL STRUCTURES FOR THE MOBILIZATION OF EARTH PRESSURES




1. Dense cohesionless soil.
2. Loose cohesionless soil.

| TYPE OF SOIL | ROTATION $\frac{x}{H}$ | |
|---|---|---|
| | ACTIVE P | PASSIVE P |
| DENSE GRANULAR | $10^{-3}$ | $2 . 10^{-2}$ |
| LOOSE GRANULAR | $4 . 10^{-3}$ | $6 . 10^{-2}$ |
| STIFF COHESIVE | $10^{-2}$ | $2 . 10^{-2}$ |
| SOFT COHESIVE | $2 . 10^{-2}$ | $4 . 10^{-2}$ |

— *PASSIVE EARTH PRESSURES*

The passive pressures correspond to the soil failure condition in which the shear strength of the soil is fully mobilized in resisting the lateral forces. The breaking wedge produced by lateral compression of the soil rises, forced by the resistant structures. These pressures are the largest that a soil mass can exert upon a structure that is displaced against it.

The action of passive earth pressures shall be considered in rigid wall structures founded in deformable soils, with appreciable lateral displacement capacity. The minimum deformations for the complete actuation of passive earth pressures are given in table 3.4.2.2.1. It is necessary, therefore, to proceed with considerable prudence when estimating the stabilizing action of the passive earth pressures, not taking them in consideration unless it is proved that the type and function of the retaining structure is compatible with the necessary movements of the soil, and being sure that the soil's characteristics remain unaltered.

In the passive state, the soil's real failure curves clearly depart from the plane surface (Coulomb's theory), and more so when the internal friction angle of the soil ($\phi$) and the wall-soil friction angle (near $\phi$) are high.

The differences are accentuated when the wall is not vertical, and when the ground surface slopes upwardly.

To calculate the passive earth pressure coefficients, various simplified methods may be adopted, assimilating the surface of the failure to a logarithmic spiral, or other complex curves formed by a surface in circular arc and tangential plane (circular friction method) or logarithmic spiral and Rankine plane (logarithmic spiral method).

Generally, applying the logarithmic spiral method is recommended to determine the passive earth pressures, nevertheless, the Coulomb approximation shall be acceptable when the values of $\phi$ and $\delta$ are not high ($\phi < 30°$ and $\delta < 10°$), $\beta$ is in the interval $\pm$ 10°, and the wall is vertical or quasi-vertical. The practical development of these approximations is given in tables 3.4.2.2.5 to 3.4.2.2.7.

The calculation shall be simplified by the application of the Rankine case, developed in table 3.4.2.2.8, in the particular case of passive earth pressures upon vertical surfaces produced by homogenous soils without groundwater, or with horizontal phreatic level, with horizontal ground surface and not considering the soil/wall friction. The validity of this approximation shall be limited to internal friction angles of the soil less than or equal to 30°.

— *PRESSURES AT REST*

Earth pressures at rest can be found in the range between the largest pressures that a soil mass acting upon a structure can exert(passive earth pressures) and the smallest (active earth pressures).

The At Rest Earth Pressure is defined as the initial pressure that a homogenous natural soil with horizontal ground surface produces, in an undisturbed state, upon a rigid wall structure with a vertical back, before the structure undergoes any movement, as long as no lateral expansions of the soil have been produced during the construction process.

The relation between horizontal earth pressures and effective vertical stress ($\sigma'$) shall be called the earth pressure at rest coefficient ($K_0$).

$$P_h = K_0 \cdot \sigma' = \frac{\upsilon}{1-\upsilon} \cdot \sigma'$$

with $\upsilon$ being Poisson's ratio.

Earth pressures at rest shall be considered in all rigid wall structures whose movement is impeded with respect to the adjacent soil, or with foundations upon rigid soil, and in general in all cases where there are no significant lateral deformations of the soil.

To calculate the stability of rigid wall structures, the action of the earth pressures at rest shall not be used, rather the active or passive pressures according to the case shall be used, since for both the rotation or sliding state, significant lateral deformations of the soil will generally occur.

## TABLE 3.4.2.2.2 ACTIVE EARTH PRESSURE DETERMINATION. COULOMB THEORY

### I. GENERAL CASE : HOMOGENOUS SOIL WITHOUT GROUNDWATER

ANALYTIC SOLUTION

$$P_a = K_a \cdot \sigma' - K_{ac} \qquad (2)$$

$$P_{ah} = K_a \cdot \sin(\alpha - \delta) \cdot \sigma' - K_{ac} \cdot \sin(\alpha - \delta)$$

$$P_{av} = K_a \cdot \cos(\alpha - \delta) \cdot \sigma' - K_{ac} \cdot \cos(\alpha - \delta) \qquad (1)$$

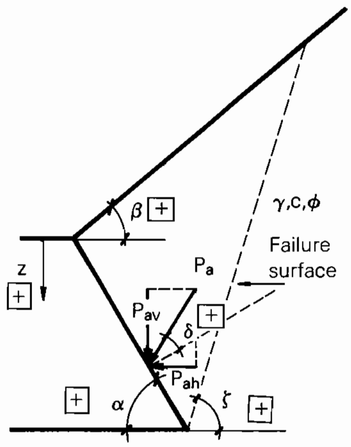
$$K_a = \frac{\sin^2(\alpha + \phi)}{\sin(\alpha - \delta) \cdot \sin^2\alpha \cdot \left[1 + \sqrt{\dfrac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\sin(\alpha - \delta) \cdot \sin(\alpha + \beta)}}\right]^2}$$

$$K_{ac} = 2c\sqrt{K_a}$$

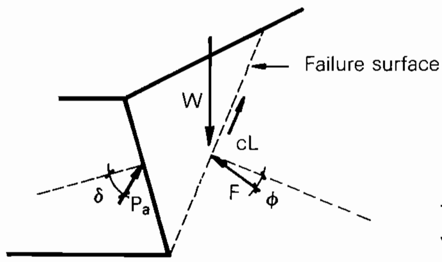
$$\sigma' = \gamma \cdot z \qquad z > 0$$

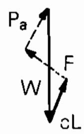
Failure surface in soils where c = 0

$$\cot(\zeta - \beta) = \cot(\phi + \gamma - \alpha - \beta) - \operatorname{cosec}(\phi + \delta - \alpha - \beta) \cdot \sqrt{\frac{\sin(\alpha - \delta)\sin(\phi + \delta)}{\sin(\alpha + \beta)\sin(\alpha - \beta)}}$$




GRAPHIC SOLUTION




The calculation shall proceed by successive active wedge trials drawn from the lowest point of the active zone on the retaining structure until obtaining a maximum $P_a$ value

EARTH PRESSURE DISTRIBUTION

Granular Soil

Cohesive Soil

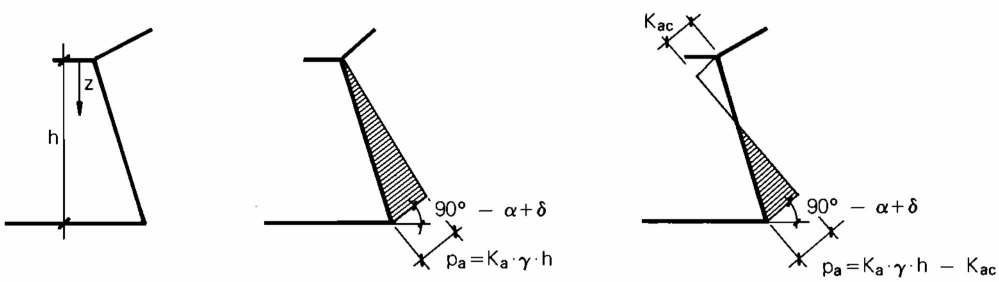


TABLE 3.4.2.2.2 (Continued)

II. STRATIFIED SOIL WITHOUT GROUNDWATER (Simplified type section)
(Soil layers parallel to the ground surface)

Each layer shall be considered as a homogenous soil upon whose upper surface a load equal to the sum of the weights of the higher layers acts.

$$p_{ai} = K_{ai} \cdot \sigma' - K_{aci}$$

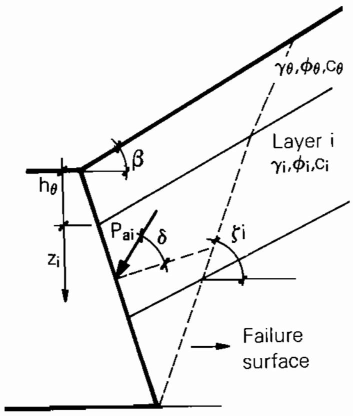
$$K_{ai} = \frac{\sin^2 (\alpha + \phi_i)}{\sin (\alpha - \delta) \cdot \sin^2 \alpha \cdot \left[ 1 + \sqrt{\frac{\sin (\phi_i + \delta) \cdot \sin (\phi_i - \beta)}{\sin (\alpha - \delta) \cdot \sin (\alpha + \beta)}} \right]^2}$$

$$K_{aci} = + 2 \cdot c \cdot \sqrt{K_{ai}}$$

$$\sigma' = \sum_{\theta = 1}^{\theta = i-1} \gamma_\theta \, h_\theta + \gamma_i \cdot z_i \qquad z_i > 0$$

Failure surface in soils, given that c = 0

$$\cot (\zeta_i - \beta) = \cot (\phi_i + \delta - \alpha - \beta) - \operatorname{cosec} (\phi_i + \delta - \alpha - \beta) \cdot \sqrt{\frac{\sin (\alpha - \delta) \sin (\phi_i + \delta)}{\sin (\alpha + \beta) \sin (\phi_i - \beta)}}$$




III. HOMOGENOUS SOIL WITH HORIZONTAL GROUNDWATER (Horizontal ground surface)

The calculation of active earth pressure shall be carried out, as in the case of stratified soil, with the following soil characteristics : $\gamma$, $\phi$ and c for the non-saturated zone; and $\gamma_i'$, $\phi_i$ and $c_i$ for the saturated zone.
When calculating total pressures, the introduction of the hydrostatic action, as defined in article 3.4.2.1- must not be forgotten.

IV. STRATIFIED SOIL WITH HORIZONTAL GROUNDWATER (Simplified Type Section)
(Horizontal soil layers and ground surfaces)

The calculation of active earth pressures shall be done as in the case of stratified terrains with the following soil characteristics : $\gamma_\theta$, $\phi_\theta$ and $c_\theta$, for the non-saturated zone and $\gamma'_i$, $\phi_i$ and $c_i$ for the calculation of earth pressures in each layer of the saturated zone, using $\gamma_{sati}$ to determine the weight of the upper saturated layers. The layer where the phreatic level is located shall be considered as two layers.
When calculating total pressures, the effects of the hydrostatic action must not be forgotten.

**TABLE 3.4.2.2.2 (Continued)**

V. STRATIFIED SOIL WITH HORIZONTAL OR NON-HORIZONTAL PHREATIC LEVEL
(Complex type section)

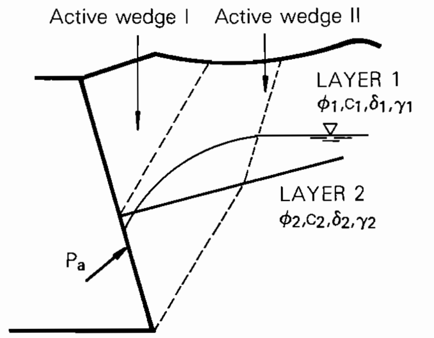
The calculation of active earth pressures shall proceed by successive active wedge trials, drawn from the lowest point of the active zone on the retaining structure, until obtaining a maximum value of $P_a$.

The active wedge shall be limited by straight failure surfaces, with different inclinations for each layer.

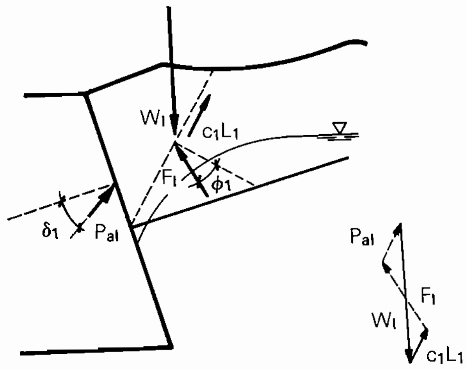
For complex type sections, wedges situated above the active zone shall be analyzed, in order to determine the earth pressure distribution in more detail.

The trials shall begin with the active wedges obtained from the simplified type section case.

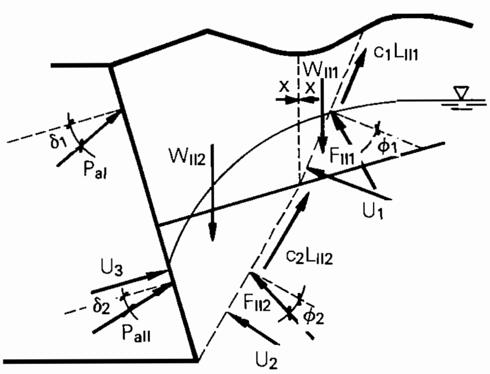
To determine the localization of the total earth pressure, moments in each analyzed wedge shall be statically balanced.

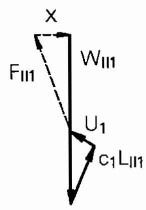



ACTIVE WEDGE I

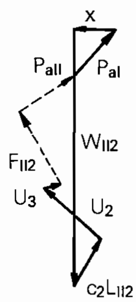



ACTIVE WEDGE II




In layer I

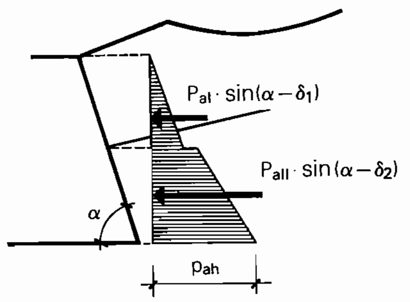



In layer I and II

— $P_{aI}$ obtained from wedge I.
— $U_1$ and $U_2$ = Total water pressures referred to the failure plane.
— $U_3$ = Total water pressures behind the structure.

TABLE 3.4.2.2.2 (Continued)

HORIZONTAL PRESSURE DISTRIBUTION, KNOWING THE MAGNITUDE AND APPLICATION POINT OF THE TOTAL FORCES




LEGEND :

$P_a$ : Total active pressure, in t/m.
$p_a$ : Active earth pressure, in t/m$^2$
$p_{ai}$ : Active earth pressure in the i layer, in t/m$^2$
$p_{ah}$ : Horizontal component of the active earth pressure, in t/m$^2$
$p_{av}$ : Vertical component of the active earth pressure, in t/m$^2$
$\sigma'$ : Effective vertical stress of the soil at the point where the earth pressure is evaluated, in t/m$^2$
$K_a$ : Coefficient of the active earth pressure (dimensionless)
$K_{ai}$ : Coefficient of the active earth pressure in the i layer (dimensionless)
$K_{ac}$ : Cohesion term, in t/m$^2$
$K_{aci}$ : Cohesion term in the i layer, in t/m$^2$
$\phi$ : Internal friction angle of the soil, in degrees
$\phi_i$ : Internal friction angle of the soil, in the i layer, in degrees
c : Soil cohesion, in t/m$^2$
$c_i$ : Soil cohesion in the i layer, in t/m$^2$
$\gamma$ : Apparent specific soil weight, in t/m$^2$
$\gamma_i$ : Apparent specific soil weight of the i layer, in t/m$^3$
$\gamma_\theta$ : Apparent specific soil weight of the $\theta$ layer, in t/m$^3$
$\gamma'$ : Submerged specific weight, in t/m$^3$
$\gamma'_i$ : Submerged specific weight of the i layer, in t/m$^3$
$\gamma_{sati}$ : Saturated specific weight in the i layer, in t/m$^3$
z : Distance from the top of the soil in the back of the structure to the point where the earth pressures are evaluated, in m
$z_i$ : Distance from the top of the i layer in the back of the structure to the point where the earth pressures are evaluated, in m
$\alpha$ : Back of the structure's angle with the horizontal, in degrees
$\beta$ : Ground surface angle with the horizontal, in degrees
$\delta$ : Soil-structure friction angle
$\zeta$ : Failure surface angle with the horizontal, in degrees
$\zeta_i$ : Failure surface angle with the horizontal, in the i layer, in degrees
W : Weight of the active failure wedge, in t/m
$W_I$ : Weight of the active wedge I, in t/m
$W_I$ : Weight of the active wedge II, in t/m
L : Length of the failure line, in m
$L_I$ : Length of the failure line, in the wedge I, in m
$L_{II}$ : Length of the failure line, in the wedge II, in m
F : Reaction of the soil to the active earth pressure wedge, in t/m
$F_I$ : Reaction of the soil to the active wedge I, in t/m
$F_{II}$ : Reaction of the soil to the active wedge II, in t/m
$h_\theta$ : Thickness of the $\theta$ layer
U : Total water pressures, in t/m

| TABLE 3.4.2.2.2 (Continued) |
|---|
| **NOTES :**<br>(1) The values of $K_a$ are supplied in table 3.4.2.2.3 for $\alpha$ = 90° fixed, and for $\beta$ = 0° fixed, with relationships $\delta/\phi = 0$ and $\delta/\phi = +1$<br>(2) At the height $z_0$ where tensions could be produced the pressure shall be considered zero; that is to say, the soil remains vertically stable without collapsing. |

Lacking specific test, $K_0$ shall be taken as a constant, and may be calculated for natural homogenous soil with horizontal ground surface by the following equations :

| Granular soils<br>Normally consolidated cohesive soils | Over-consolidated<br>cohesive soils |
|---|---|
| $K_0 = 1 - \sin\phi$ | $K_0 = (1 - \sin\phi)\ (OCR)^{0.5}$ |

given that :

$\phi$ = Internal friction angle of the soil

OCR = Overconsolidation rate : existing ratio between the preconsolidation pressure and the current effective stress of the soil.

For fills, the prior formulas may be used except when soils are highly compacted or exposed to vibrations, in which case the coefficient $K_0$ may take values higher than 1, including values close to those adopted for the passive earth pressure coefficient.

In weakly compacted or non-compacted fills, the earth pressure at rest state shall not be reproduced. One closer to the active earth pressure shall be produced. Nevertheless, the at rest earth pressure shall be designed for.

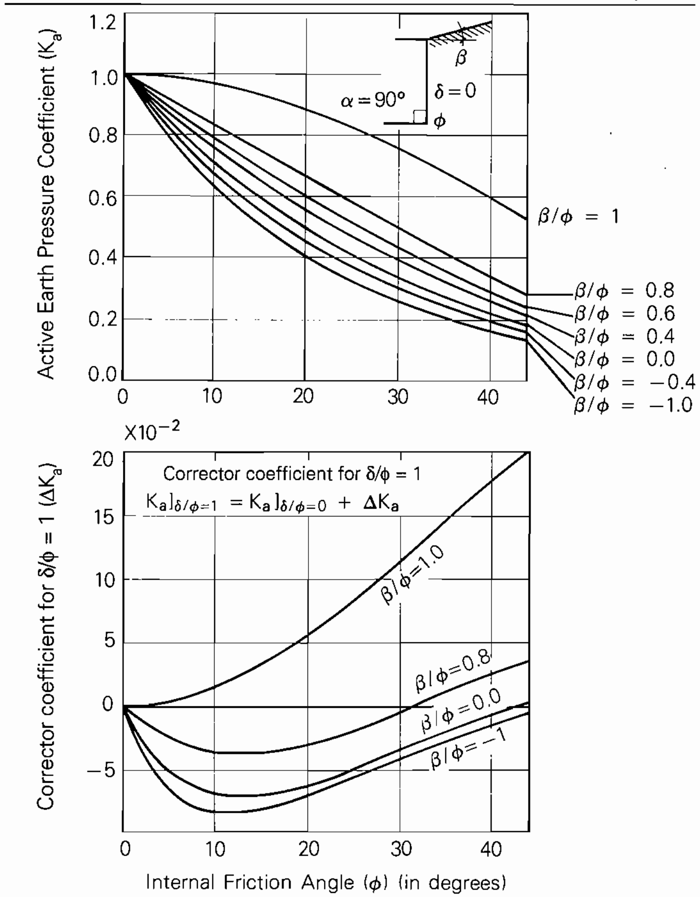
To calculate the at rest earth pressure in the general cases of rigid wall structures and non-vertical backs, with non-horizontal ground surface, or stratified soil, the Coulomb theory for the determination of active earth pressures shall be adopted, assuming $\delta = 0$, and an internal friction angle value ($\phi$) obtained from the equation:

$$K_0 = \tan^2 \left(45 - \frac{\phi}{2}\right)$$

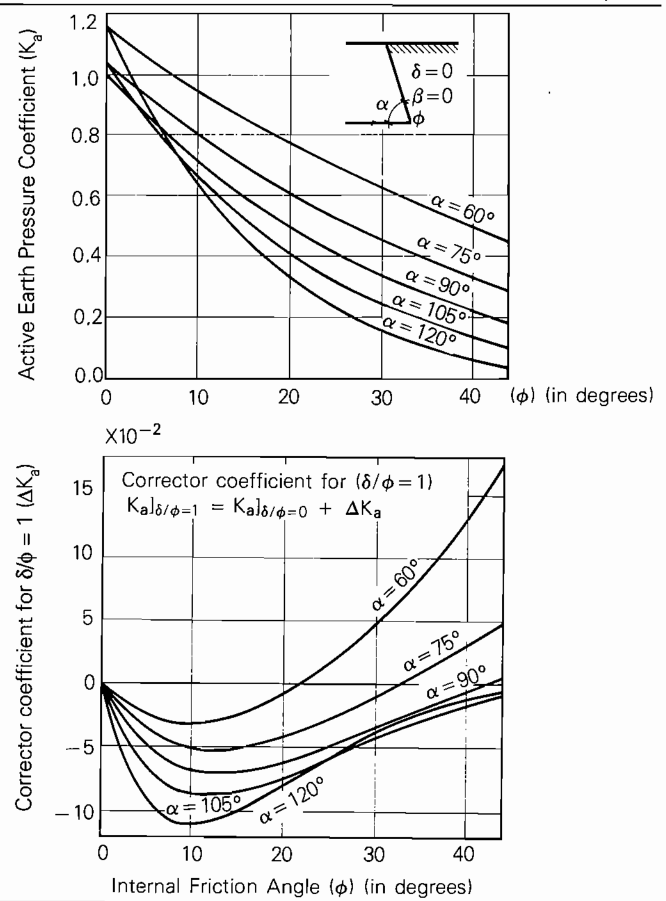
with $K_0$ being the at rest earth pressure coefficient corresponding to the homogenous soil, vertical back and horizontal ground surface.

## TABLE 3.4.2.2.3 COEFFICIENTS OF ACTIVE EARTH PRESSURE ACCORDING TO COULOMB'S THEORY

### ACTIVE EARTH PRESSURE DETERMINATION, CASE 1 ($\delta/\phi = 0$)




### ACTIVE EARTH PRESSURE DETERMINATION, CASE II ($\delta/\phi = 0$)



## TABLE 3.4.2.2.4 ACTIVE EARTH PRESSURE DETERMINATION. RANKINE'S THEORY (1)

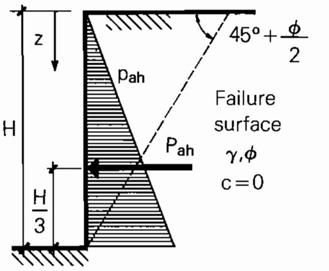
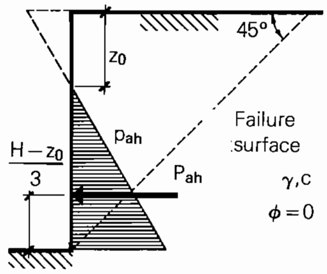
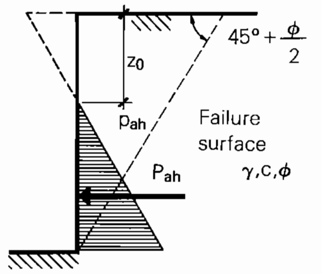
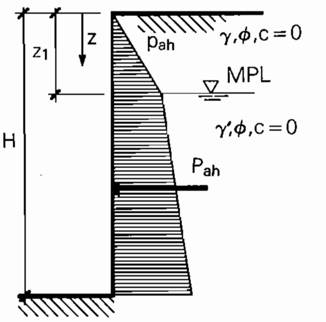
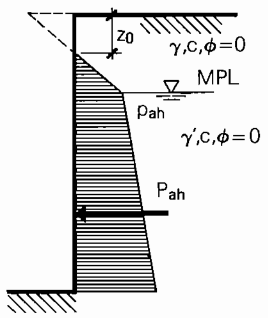
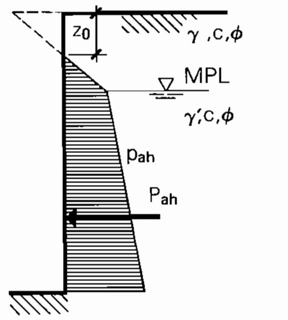
| GRANULAR SOILS | SATURATED OR QUASI-SATURATED COHESIVE SOILS IN UNDRAINED CONDITIONS (short-term earth pressures) | COHESIVE SOILS IN DRAINED CONDITIONS (long-term earth pressures) (2) |
|---|---|---|
| **WITHOUT INTERMEDIATE PHREATIC LEVEL** | | |
|  |  |  |
| $K_{ah} = \tan^2(45° - \frac{\phi}{2})$<br>$p_{ah} = K_{ah}\cdot\gamma\cdot z$<br>$P_{ah} = \frac{K_{ah}\cdot\gamma\cdot H^2}{2}$<br>$p_{av} = 0$ | $z_0$ (Tension crack depth) = $2c/\gamma$<br>$p_{ah} = \gamma\cdot z - 2c,\quad z > z_0$<br>$P_{ah} = \gamma\cdot\frac{H^2}{2} - 2\cdot c\cdot H + \frac{2c^2}{\gamma}$<br>$p_{av} = 0$ | $z_0 = \frac{2c}{\gamma}\cdot\tan(45° + \frac{\phi}{2})$<br>$p_{ah} = \gamma\cdot z\cdot\tan^2(45° - \frac{\phi}{2}) - 2c\cdot\tan(45° - \frac{\phi}{2})$<br>$P_{ah} = \frac{\gamma\cdot H^2}{2}\cdot\tan^2(45° - \frac{\phi}{2}) - 2cH\cdot\tan(45° - \frac{\phi}{2}) + \frac{2c^2}{\gamma}$<br>$p_{av} = 0$ |
| **WITH INTERMEDIATE HORIZONTAL PHREATIC LEVEL** | | |
|  |  |  |
| $K_{ah} = \tan^2(45° - \frac{\phi}{2})$<br>$p_{ah} = K_{ah}\cdot\gamma\cdot z,\quad z < z_1$<br>$p_{ah} = K_{ah}\cdot(\gamma'\cdot(z - z_1) + \gamma\cdot z_1),\ z > z_1$<br>$p_{av} = 0$ | $z_0 = \frac{2c}{\gamma}$<br>$p_{ah} = \gamma z - 2c,\quad z_0 < z < z_1$<br>$p_{ah} = \gamma z_1 + \gamma'(z - z_1) - 2c,\ z > z_1$<br>$p_{av} = 0$<br>(3) | $z_0 = \frac{2c}{\gamma}\cdot\tan(45° + \frac{\phi}{2})$<br>$p_{ah} = \gamma\cdot z\cdot\tan^2(45° - \frac{\phi}{2}) - 2c\cdot\tan(45° - \frac{\phi}{2}),\ z_0<z<z_1$<br>$p_{ah} = [\gamma\cdot z_1 + \gamma'(z - z_1)]\ \tan^2(45° - \frac{\phi}{2}) - 2c\cdot\tan(45° - \frac{\phi}{2}),\ z > z_1$<br>$p_{av} = 0$ (3) |

TABLE 3.4.2.2.4 (Continued)

NOTES :

(1) Only applicable to the determination of active earth pressures applied to vertical surfaces produced by homogenous soils without groundwater, or with horizontal phreatic level, with horizontal ground surface, and neglecting the soil/wall friction, or by homogenous soils without groundwater, with ground surface slope coinciding with the soil-wall friction angle.

(2) Only for undisturbed overconsolidated clays or very dense fills of cohesive material, as long as they are permanently protected from freezing and drying and are of low sensitivity.

(3) In the case of $z_0 > z_1$ the earth pressure diagram shall correspond to the case of a soil without intermediate phreatic level with :

$$z_0 = \frac{2c}{\gamma} \text{ or } z_0 = \frac{2c}{\gamma} \cdot \tan\left(45° + \frac{\phi}{2}\right),$$

respectively.

The total pressure of the soil upon a structure is the resultant of the local earth pressures exerted upon all its surfaces.

The determination of earth pressures on rigid wall structures with backs at different angles shall begin by calculating the total pressures upon the first segment of wall according to the methods described above. The resulting pressure upon the second segment shall be obtained be means of the trial-wedge method developed in tables 3.4.2.2.2., 3.4.2.2.5. and 3.4.2.2.6. The earth pressure upon curved wall structures may be approximated as that obtained for an equivalent segmented wall. As a simplification, in general, the determination of pressures upon segmented, curved or totally irregular wall structures shall be admitted, approximating the wall as a regular wall with one mean inclination.

In the assumption that in some of the surfaces active earth pressures can be developed, and in others, passive earth pressures can be developed, the compatibility of both types of pressures generated simultaneously shall be analyzed.

$a_{21}$) *CHARACTERISTIC VALUES OF GEOTECHNICAL PARAMETERS FOR THE DETERMINATION OF EARTH PRESSURES*

The geotechnical parameters that fundamentally are used in the determination of earth pressures are :

— Apparent specific weight ($\gamma$)
— Shear strength parameters :

- internal friction angle ($\phi$)
- cohesion (c)

— *NATURAL SOILS*

The geotechnical parameters of the project for natural soils shall be experimentally obtained by geotechnical investigation and laboratory or field tests, according to the evaluation criteria of the actions assigned in section 3.2. For their determination, the geology and geomorphology of the zone, the historical stress of the soil, the possibility of changes in the properties due to the construction process, the action of dynamic loads, and dependency on factors such as time or the state of deformation, shall be taken into account.

In order to reduce the number of samples or field tests necessary for the complete statistical definition of the geotechnical parameters of a soil, those obtained according to the following process shall be assimilated to characteristic values :

**TABLE 3.4.2.2.5** **DETERMINATION OF PASSIVE EARTH PRESSURES. COULOMB'S THEORY, VALIDITY RANGE : VALUES OF $\phi \leq 30°$ AND $\delta \leq 10°$, $\beta \leq \pm 10°$ (vertical or quasi-vertical wall).**

## I. GENERAL CASE : HOMOGENOUS SOIL WITHOUT GROUNDWATER

### ANALYTIC SOLUTION

$$P_p = K_p \cdot \sigma' - K_{pc}$$

$$P_{ph} = K_p \cdot \sin(\alpha - \delta) \cdot \gamma' + K_{pc} \cdot \sin(\alpha - \delta)$$

$$P_{pv} = K_{pv} \cdot \cos(\alpha - \delta) \cdot \gamma' + K_{pc} \cdot \cos(\alpha - \delta) \qquad (1)$$

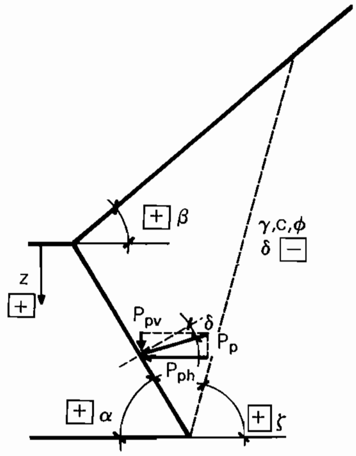
$$K_p = \frac{\sin^2(\alpha - \phi)}{\sin(\alpha - \delta) \cdot \sin^2\alpha \cdot \left[1 - \sqrt{\dfrac{\sin(\phi - \delta) \cdot \sin(\phi + \beta)}{\sin(\alpha + \beta) \cdot \sin(\phi + \beta)}}\right]^2}$$

$$K_{pc} = 2c\sqrt{K_p}$$

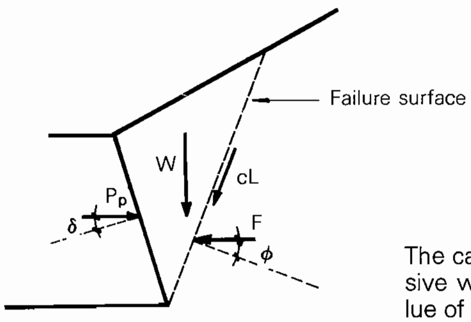
$$\gamma' = \gamma \cdot z \qquad z > 0$$

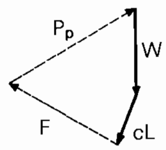
Failure surface in soils where c = 0

$$\cot(\zeta - \beta) = -\cot(\phi - \delta + \alpha + \beta) + \operatorname{cosec}(\phi - \delta + \alpha + \beta) \cdot \sqrt{\frac{\sin(\alpha - \delta)\sin(\phi - \delta)}{\sin(\alpha + \beta)\sin(\phi + \beta)}}$$




### GRAPHIC SOLUTION




The calculation shall proceed by successive passive wedge trials until obtaining a minimum value of $P_p$.

### PRESSURE DISTRIBUTION

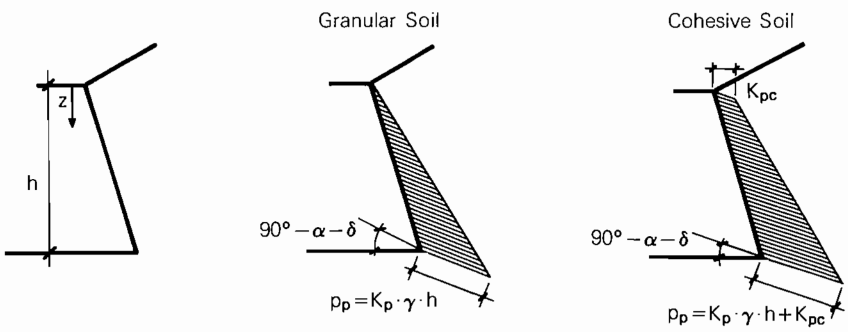


**TABLE 3.4.2.2.5 (Continued)**

II. STRATIFIED SOIL WITH FREATED LEVEL (Simplified type section)
(Layers parallel to the ground surface)

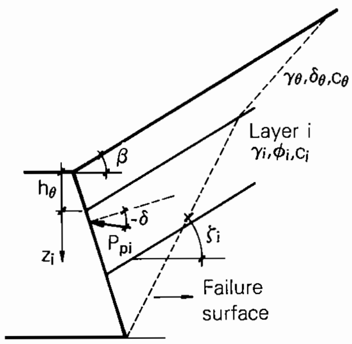
Each layer shall be considered as a homogenous soil, upon whose upper surface a load equal to the total of the weights of the upper layers acts. The above shall be valid only for the cases where the displacements of the structure are sufficient to mobilize the passive earth pressures in every layer.

$$P_{pi} = K_{pi} \cdot \sigma' + K_{pci}$$

$$K_{pi} = \frac{\sin^2(\alpha - \phi_i)}{\sin \cdot (\alpha - \delta) \cdot \sin^2 \alpha \cdot \left[1 - \sqrt{\dfrac{\sin(\phi_i - \delta) \cdot \sin(\phi_i + \beta)}{\sin(\alpha - \delta) \cdot \sin(\alpha + \beta)}}\right]^2}$$

$$K_{pci} = 2c\sqrt{K_{pci}}$$

$$\sigma' = \sum_{\theta=1}^{\theta=i-1} \gamma_\theta h_\theta + \gamma_i \cdot z_i \qquad z_i > 0$$




Failure surface in soils where c = 0

$$\cot(\zeta_i - \beta) = -\cot(\phi_i - \delta + \alpha + \delta) + \operatorname{cosec}(\phi_i - \delta + \alpha + \delta) \cdot$$

$$\cdot \sqrt{\frac{\sin(\alpha - \delta)\sin(\phi_i - \delta)}{\sin(\alpha + \beta)\sin(\phi_i + \beta)}}$$

III. HOMOGENOUS SOIL WITH HORIZONTAL PHEATIC LEVEL (Horizontal ground surface)

The calculation of passive earth pressure shall proceed as in the case of stratified soils with the following soil characteristics : $\gamma$, $\phi$ and c, for the non-saturated zone and $\gamma$, $\phi$ and c for the saturated zone.
When calculating the total pressures, the effects of the hydrostatic action according to the definition in section 3.4.2.1, must not be forgotten.

IV. STRATIFIED SOIL WITH HORIZONTAL PHEATIC LEVEL (Simplified Type Section)
(Horizontal soil layers and ground surfaces)

The calculation of passive earth pressure shall proceed as in the case of stratified terrains with the following soil characteristics : $\gamma_\theta$, $\phi_\theta$ and $c_\theta$, for the non-saturated zone and $\gamma'_i$, $\phi_i$ and $c_i$ for the calculation of earth pressures in each layer of the saturated zone, taking $\gamma_{sati}$ for the determation of the weight of the upper saturated layers.

V. STRATIFIED SOIL WITH HORIZONTAL OR NON-HORIZONTAL PHREATIC LEVEL
(Complicated Type Section)

For this case, it shall not be possible to apply the Coulomb approximation method. Curved sliding wedges shall be applied according to table 3.4.2.2.6.

TABLE 3.4.2.2.5 (Continued)

LEGEND : SEE TABLE 3.4.2.2.2

$P_p$ : Total passive earth pressures, in t/m
$p_p$ : Passive earth pressure, in t/m$^2$
$p_{pi}$ : Passive earth pressure in the i layer, in t/m$^2$
$p_{ph}$ : Horizontal component of the passive earth pressure, in t/m$^2$
$p_{pv}$ : Vertical component of the passive earth pressure, in tm$^2$
$K_p$ : Coefficient of the passive earth pressure (dimensionless)
$K_{pi}$ : Coefficient of the passive earth pressure in the i layer (dimensionless)
$K_{pc}$ : Cohesion term for passive earth pressures, in t/m$^2$
$K_{pci}$ : Cohesion term for passive earth pressures in the i layer, in t/m$^2$

NOTES :

(1) The values of $K_p$ are supplied in table 3.4.2.2.7 for $\alpha = 90°$ fixed, and for $\beta = 0°$ fixed, with the relationships $\delta/\phi = 0$ and $\delta/\phi = -1$.

TABLE 3.4.2.2.6 DETERMINATION OF PASSIVE EARTH PRESSURES LOGARITHMIC SPIRAL AND FRICTION CIRCLE METHODS.
VALIDITY RANGE : GENERAL

I. GENERAL CASE : HOMOGENOUS OR LAYERED SOILS OF SIMPLIFIED TYPE SECTION

The logarithmic spiral method is of general application for the determination of passive earth pressures, and is specially recommended in those cases where high internal friction angle ($\phi$) values are taken, combined with large soil-wall friction angle ($\delta$) values, for which the application of the Coulomb method is not considered valid.

The logarithmic spiral method admits a failure mechanism of the soil mass whose failure surface is composed of a logarithmic spiral (BC) that passes through the toe of the wall, with its origin 0 situated at the top of the wall combined with a Rankine zone (OCD). This mechanism assumes a rotation around O and sliding along CD. The spiral layout is performed by successive trials.
The friction circle methods substitutes the logarithmic spiral with a circular arc.

In homogeneous or stratified non-cohesive soils of simplified section for retaining structures with vertical walls and variable angle of the ground surface with respect to the horizontal, or retaining structures with a variable backwall angle and with horizontal ground surface, the passive earth pressure coefficients given in table 3.4.2.2.7. (bis) can be adopted.

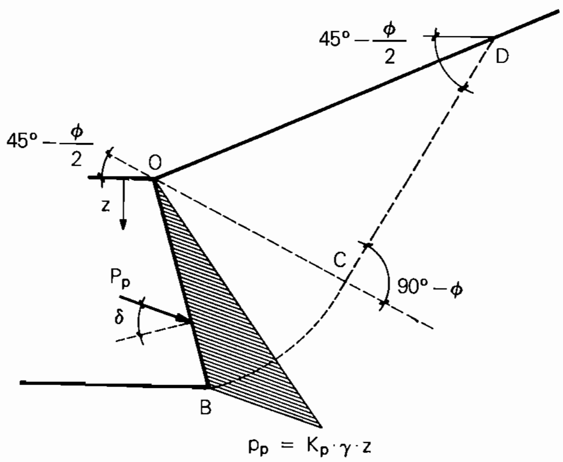
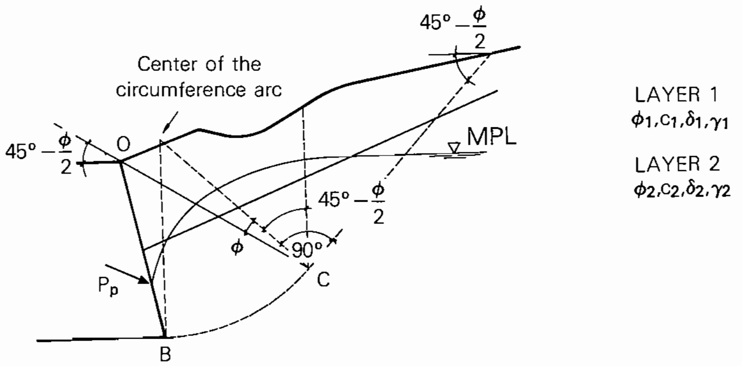


TABLE 3.4.2.2.6 (Continued)

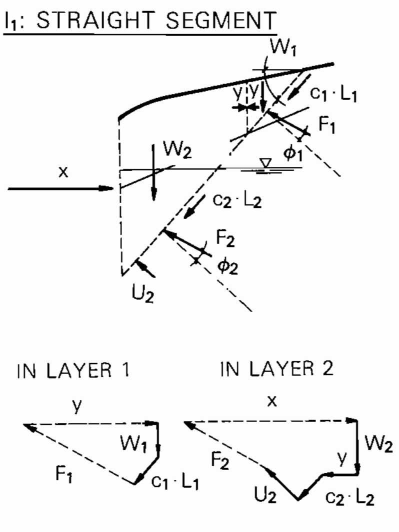
II. STRATIFIED SOIL WITH HORIZONTAL OR NON-HORIZONTAL PHREATIC LEVEL (Simplified Type Section)

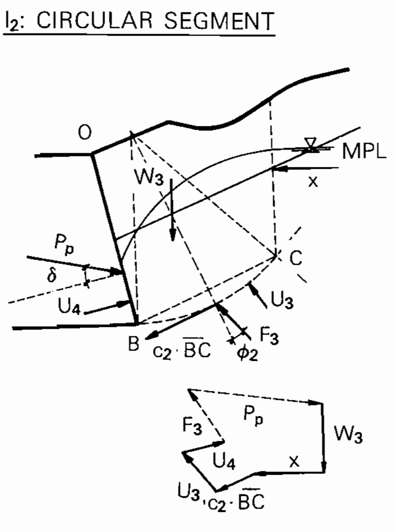



The calculation of passive earth pressures shall proceed by successive trials of passive wedges plotted according to the criteria shown in the above diagram, until obtaining a minimum value of $P_p$.

The passive wedges shall be limited by idealization of the logarithmic spiral method, by a slip plane formed by a straight line and a circumference arc (friction circle method) except when soil-wall friction is not considered in which case wedges defined by straight lines will be tried.

To determine the localization of the total earth pressure, moments in the passive wedge $I_2$ shall be balanced.
For very complicated sections, wedges situated at intermediate heights shall be tried, in order to determine the pressure distribution in more detail.

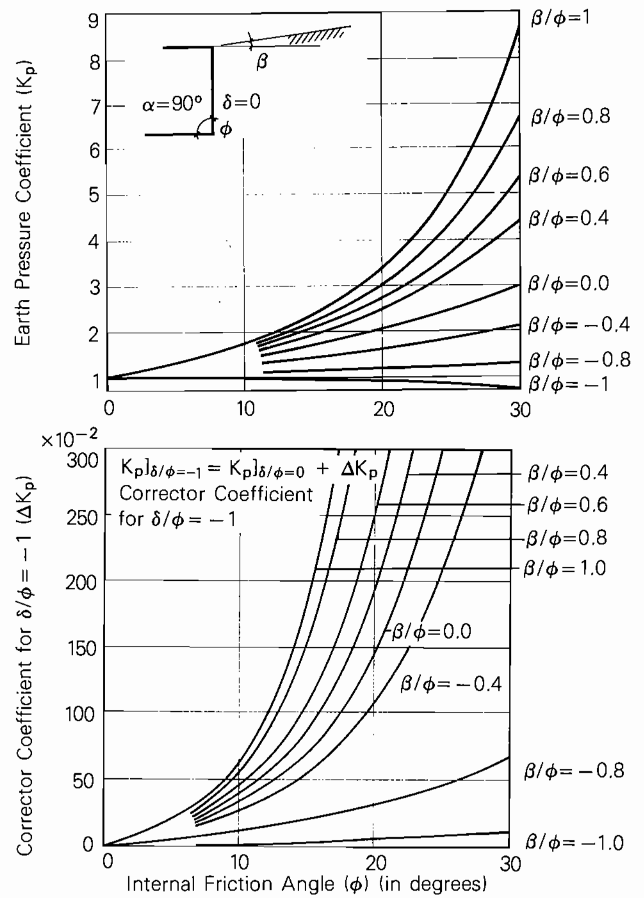


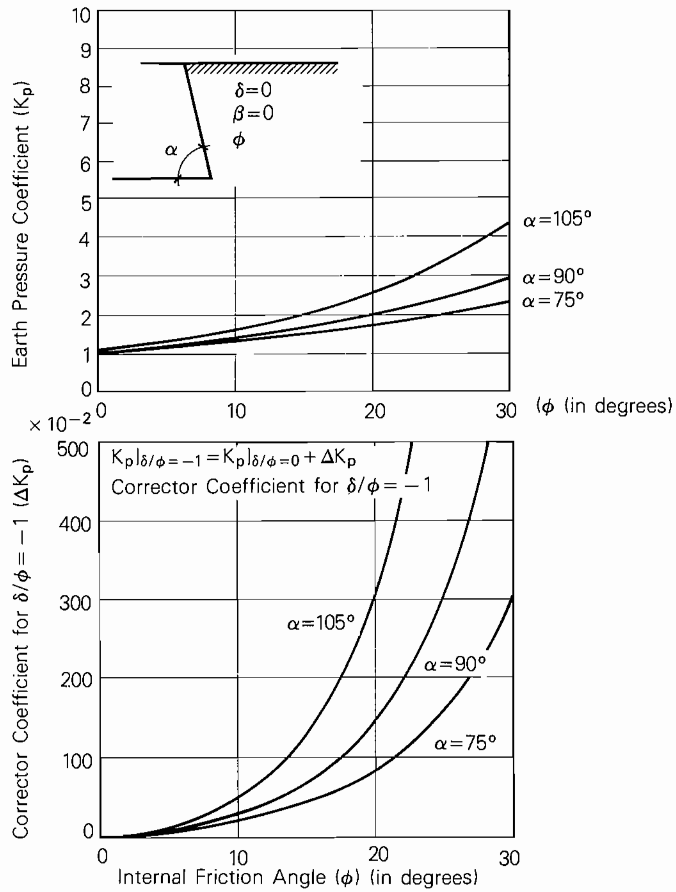

**LEGEND :** See Tables 3.4.2.2.2 and 3.4.2.2.5

## TABLE 3.4.2.2.7 PASSIVE EARTH PRESSURE COEFFICIENTS ACCORDING TO COULOMB'S THEORY

PASSIVE EARTH PRESSURE DETERMINATION. CASE 1 ($\delta/\phi = 0$)



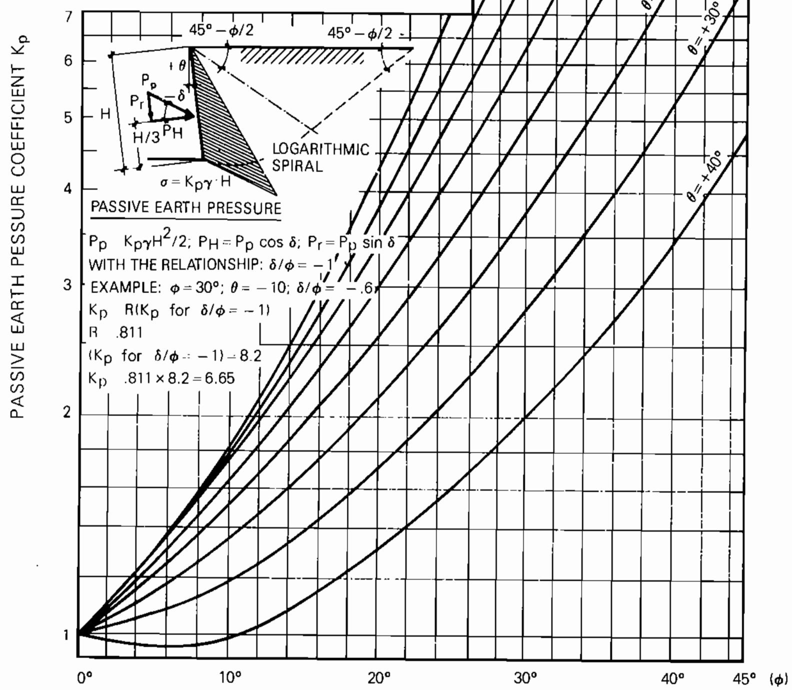

PASSIVE EARTH PRESSURE DETERMINATION. CASE 2 ($\delta/\phi = 0$)



# TABLE 3.4.2.2.7 (bis) PASSIVE EARTH PRESSURE COEFFICIENTS ACCORDING TO COULOMB'S THEORY

FACTOR R OF $K_p$ FOR DIFFERENT VALUES OF $\delta/\phi$

| $\phi$ \ $\delta/\phi$ | 0 7 | 0 6 | 0 5 | 0 4 | 0 3 | 0 2 | 0 1 | 0 0 |
|---|---|---|---|---|---|---|---|---|
| 10° | 973 | 962 | 945 | 923 | 912 | 898 | 881 | 864 |
| 15° | 981 | 934 | 907 | 881 | 854 | 830 | 803 | 775 |
| 20° | 909 | 901 | 882 | 824 | 787 | 752 | 716 | 673 |
| 25° | 912 | 860 | 808 | 759 | 711 | 668 | 620 | 574 |
| 30° | 873 | 811 | 785 | 636 | 627 | 574 | 520 | 467 |
| 35° | 836 | 752 | 674 | 603 | 536 | 475 | 417 | 352 |
| 40° | 783 | 682 | 592 | 512 | 439 | 375 | 316 | 282 |
| 45° | 718 | 600 | 500 | 414 | 339 | 275 | 221 | 174 |




FACTOR R OF $K_p$ FOR DIFFERENT VALUES OF $\delta/\phi$

| $\phi$ \ $\delta/\phi$ | 0 7 | 0 6 | 0 5 | 0 4 | 0 3 | 0 2 | 0 1 | 0 0 |
|---|---|---|---|---|---|---|---|---|
| 10° | 978 | 0 982 | 0 945 | 0 929 | 0 912 | 0 898 | 0 881 | 0 864 |
| 15° | 961 | 924 | 0 907 | 891 | 0 864 | 0 830 | 0 800 | 0 775 |
| 20° | 909 | 901 | 862 | 824 | 0 787 | 752 | 0 716 | 0 678 |
| 25° | 912 | 860 | 803 | 759 | 711 | 0 668 | 0 620 | 0 574 |
| 30° | 878 | 811 | 745 | 686 | 627 | 574 | 520 | 0 457 |
| 35° | 836 | 752 | 674 | 603 | 536 | 475 | 417 | 0 387 |
| 40° | 783 | 682 | 592 | 512 | 439 | 375 | 316 | 0 262 |
| 45° | 718 | 600 | 500 | 414 | 338 | 216 | 221 | 0 174 |

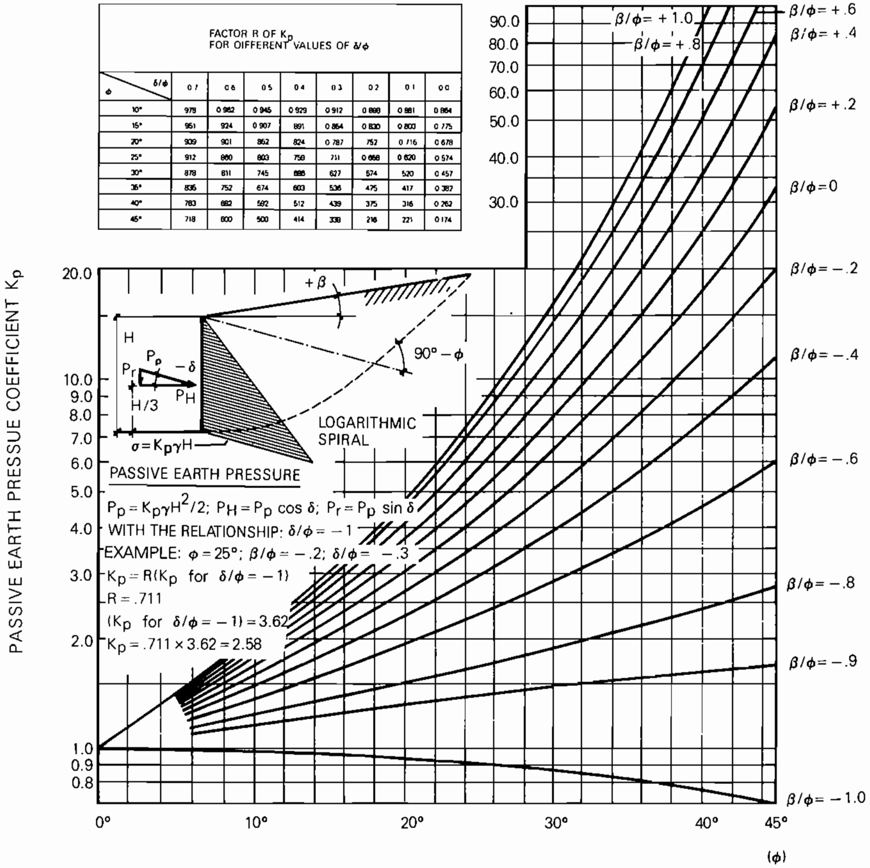
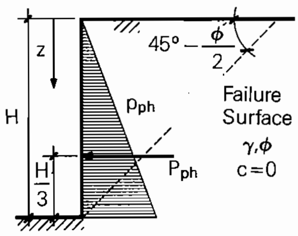
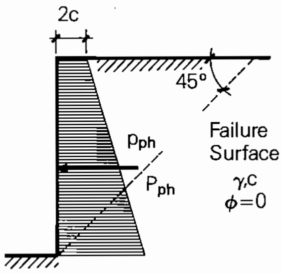
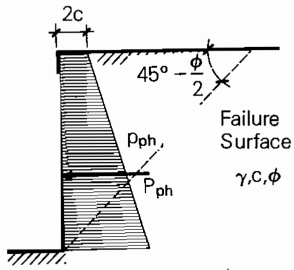
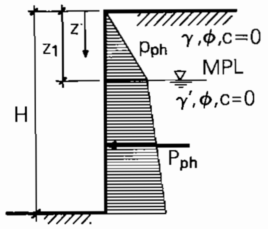
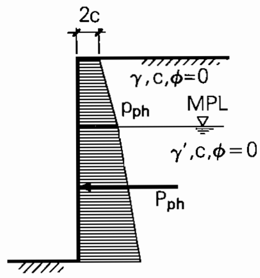
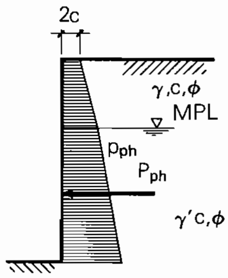


**TABLE 3.4.2.2.8 DETERMINATION OF PASSIVE EARTH PRESSURES. RANKINE'S THEORY (1)**
**VALIDITY RANGE : Values of $\phi \leq 30°$**

| GRANULAR SOILS | SATURATED OR QUASI-SATURATED COHESIVE SOILS IN UNDRAINED CONDITIONS (Short-term earth pressures) | COHESIVE SOILS IN DRAINED CONDITIONS (long-term earth pressures) (2) |
|---|---|---|
| **WITHOUT INTERMEDIATE PHREATIC LEVEL** | | |
|   |   |   |
| $K_{ph} = \tan^2 (45° + \frac{\phi}{2})$<br>$p_{ph} = K_{ah} \cdot \gamma \cdot z$<br>$P_{ah} = \frac{K_{ph} \cdot \gamma \cdot H^2}{2}$<br>$P_{pv} = 0$ | $p_{ph} = \gamma \cdot z + 2c$<br>$P_{ph} = \gamma \cdot \frac{H^2}{2} + 2 \cdot c \cdot H$<br>$P_{pv} = 0$ | $p_{ph} = \gamma \cdot z \cdot \tan^2 (45° + \frac{\phi}{2}) -$<br>$+ 2c \cdot \tan (45° + \frac{\phi}{2}]$<br>$P_{ph} = \frac{\gamma \cdot H^2}{2} \cdot \tan (45° + \frac{\phi}{2}) +$<br>$+ 2 \cdot c \cdot H \cdot \tan (45° + \frac{\phi}{2})$ |
| **WITH INTERMEDIATE HORIZONTAL PHREATIC LEVEL** | | |
|   |   |   |
| $K_{ph} = \tan^2 (45° + \frac{\phi}{2})$<br>$p_{ph} = K_{ah} \cdot \gamma \cdot z, \quad z < z_1$<br>$p_{ph} = K_{ah} \cdot (\gamma' \cdot (z - z_1) + \gamma \cdot z_1),$<br>$z > z_1$<br>$p_{pv} = 0$ | $p_{ph} = \gamma z + 2c, \quad z < z_1$<br>$p_{ph} = \gamma \cdot z_1 + \gamma' (z - z_1) + 2c,$<br>$z > z_1$<br>$p_{pv} = 0$ | $p_{ph} = \gamma \cdot z \cdot \tan^2 (45° + \frac{\phi}{2}) +$<br>$+ 2 \cdot c \cdot \tan (45° + \frac{\phi}{2}), z < z_1$<br>$p_{ph} = [\gamma \cdot z_1 + \gamma' (z - z_1)]$<br>$\cdot \tan^2 (45° + \frac{\phi}{2}) +$<br>$- 2c \cdot \tan (45° - \frac{\phi}{2}), z > z_1$<br>$p_{pv} = 0$ |

**NOTES :**

(1) Only applicable for the determination of passive earth pressures upon vertical surfaces produced by homogenous soils without groundwater or with a horizontal phreatic level, with a horizontal ground surface and neglecting the soil/wall friction.

(2) Only for undisturbed, overconsolidated clays or very dense fills of cohesive materials, as long as they are permanently protected from freezing or drying, and are of low sensitivity.

— The determination of parameters shall be done for each type of distinct soil that is present at the site, unless the statistical treatment of the properties of the soil allows the homogenization, into only one type, of those soils of different nature that are found at the site. Soils shall be considered as different types if they are of different geological origin, non-similar granulometry, or show significant dispersions in their shear strength parameters (5° for $\phi$, 2 t/m$^2$ for c and $c_u$) or in their specific weights (0.2 t/m$^3$).

— For each type of soil, the minimum number of samples to test or «in situ» tests to be carried out shall be 3, each taken at a different depth.

— Having tested n samples, or carried out n «in situ» tests the basic value of a geotechnical parameter shall be defined as the one obtained from the sample or test whose value is just below the average of all those obtained.
If only three samples or three «in situ» tests have been taken, the least of those obtained shall be considered as a basic value, as long as these values do not differ very much between themselves. If the three values do differ substantially, the testing program will have to be continued on a larger number of samples.
The basic value of the geotechnical parameters, using disturbed samples (especially in granular soils), shall be obtained by recompacting the samples to relative densities close to those that are calculated from the on site test (e.g. penetrometer). The possibility of the estimation of the basic value of geotechnical parameters, using indirect procedures such as empirical correlations with field test results, may be considered (e.g. internal friction angle/no. of blows in the SPT). (See ROM 0.5. Geotechnical Recommendations for Marine Works).

— In order to take into account the lack of precision in the tests and the heterogeneity of soil masses, the characteristic values of the geotechnical parameters shall be obtained by reducing of the basic values by the following coefficients :

- For specific weight : 1.00
- For internal friction angle : 1.1 ($\tan\phi / 1.1$)
- For the cohesion : 1.3

Only with significant and proven experience of adjacent zones, and with structures of comparable structural elements, shall geotechnical tests be omitted.

For preliminary studies and preliminary designs, and in the absence of other information, the approximated values given in tables in these Recommendations (Table 3.4.1.1.2. for specific weights and Table 3.4.2.2.9. for shear strength parameters) shall be used.

Likewise, this data may be used in regards to secondary structures where the foreseen damage in case of accidents is small and confined to materials.

— *FILLS*

The project's geotechnical parameters for fills shall be obtained from the tables included in these Recommendations, taking into account their dependency, especially for granular fills, on the method of placement, degree of compaction and on degradation of the materials over time.

Therefore, fills shall be considered disturbed and completely remolded soils. It shall be assumed that their cohesion is zero.

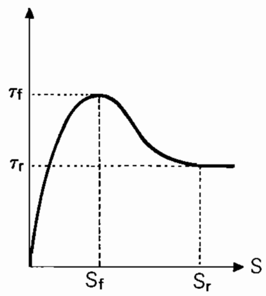
Lacking other data, for the consideration of loose or compact fills as a function of their method of placement, the instructions given in section 3.4.1.1- Self Weight shall be followed.

When the consolidation, compaction or improvement of the fill above or below the phreatic level, is anticipated in the project, the geotechnical parameters used in the calculations shall reflect these conditions. The Project's Technical Specifications shall include the required conditions of compliance for soil improvement and the methods of verification to be employed.
Lacking other more precise data, increases of 7% in the specific weights and 5%

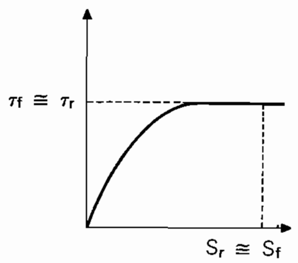
in the friction angles given in the tables shall be considered, when the compaction of natural soils or granular fills by vibration is foreseen.

To determine soil loads, the shear strength parameters for granular soils and cohesive soils in drained conditions (long-term earth pressures) shall be obtained in terms of effective stresses, based on the shear stress/strain diagrams of shear strength tests.

In earth pressure formulas, the effective internal friction angle and the effective cohesion shall be used. These values may be breaking (peak) values ($\phi'_f$, $c'_f$) or residual values ($\phi'_r$, $c'_r$), evaluated from the breaking shear strength or residual shear strength, according to the earth pressures to be calculated :




Shear stress/strain diagram for dense granular soils and hard cohesive soils (overconsolidated)




Shear stress/strain diagram for loose granular soil and soft cohesive soils.

ACTIVE EARTH PRESSURES :

$\phi_f$ y $c'_f$

PASSIVE EARTH PRESSURES :

- Granular soils :

  $\phi'_f$ y $c'_f$

- Dense cohesive soils (overconsolidated)

  Intermediate shear strength between the breaking and the residual values in cases where the structures can absorb large deformations.

- Soft cohesive soils :

  Shear strength lower than the residual in those cases where the structure does not absorb large deformations.

For cohesive, saturated soils in undrained conditions (short-term earth pressures) in the earth pressure formulas a zero internal friction angle ($\phi = 0$) and a cohesion equal to the undrained cohesion ($c_u$) shall be adopted. This parameter coincides with the undrained shear strength obtained in the lab tests.

The other factors being equal, it can be considered that the shear parameters (especially the internal friction angle) are the same above and below the phreatic level.

$a_{22}$) *SOIL-STRUCTURE FRICTION* ($\delta$)

In the determination of earth pressures, the friction between the soil and the wall prevents the sliding of the active wedge and therefore reduces the active earth pressures ($\delta$ positive); on the contrary the friction eases the sliding of the passive wedge and therefore increases the passive earth pressures ($\delta$ negative).

The effect of the soil-wall friction on the active earth pressure coefficient is small, and can be neglected under normal conditions.

TABLE 3.4.2.2.9 COMMON CHARACTERISTIC SHEAR STRENGTH PARAMETERS FOR THE DETERMINATION OF EARTH PRESSURES

| SOIL TYPE | Long Term Strength (1) | | | Short Term Strength (2) |
|---|---|---|---|---|
| | $\phi'_f$ (degrees) | $\phi'_r$ (degrees) | $c'$ ($t/m^2$) | $c_u$ ($t/m^2$) |
| NATURAL SOILS | | | | |
| GRANULAR SOILS | | | | |
| - Gravels | | | | |
| - Dense | 45 | 35 | — | — |
| - Loose | 35 | 35 | — | — |
| - Sandy Gravel | | | | |
| - Dense | 43 | 33 | — | — |
| - Loose | 33 | 33 | — | — |
| - Sand | | | | |
| - Dense | 40 | 30 | — | — |
| - Loose | 30 | 30 | — | — |
| COHESIVE SOILS | | | | |
| - Silts and sandy/silty clay | 27 | 25 | 0.5-2.00 | 1.00-5.00 |
| - Clay | | | | |
| - Hard (overconsolidated) | 20 | 10 | 2.00 | 2.50-5.00 |
| - Soft (normally consolidated) | 17 | 10 | | 1.00-2.50 |
| - Organic sediments | | | | |
| - High clay content | 15 | 12 | 1.50 | 1.00-2.00 |
| - Low clay content | 20 | 15 | 1.00 | 1.00-2.50 |
| - Peat | 15 | — | 0.5 | — |
| - Mud | 20 | — | 0.5 | 1.00-2.00 |
| FILLS | | | | |
| RUBBLE MOUNDS AND RIP RAPS | | | | |
| - Of open granulometry | 40-45 | — | | — |
| - Of closed granulometry (mine run) | | | | |
| - Dense | 35-40 | — | — | — |
| - Loose | 30-35 | — | — | — |
| GRANULAR AND COHESIVE FILLS | | | | |
| - Gravel | | | | |
| - Dense | 40 | — | — | — |
| - Loose | 35 | — | — | — |
| - Sand | | | | |
| - Dense | 35 | — | — | — |
| - Loose | 30 | — | — | — |
| - Silt | 25 | — | — | — |
| - Embankments | 30 | — | — | — |
| ATROPHIC FILLS | | | | |
| - Urban debris and demolition wastes | 35 | — | — | — |
| UNCONVENTIONAL FILLS | | | | |
| - Blastfurnace slag | | | | |
| - Granular | 30 | — | — | — |
| - In pieces | 40 | — | — | — |
| - Lapillis | 35 | — | — | — |
| - Fly ash | 25 | — | — | — |

TABLE 3.4.2.2.9 (Continued)

NOTES :

(1) Drained shear strength parameters.

(2) Undrained shear strength parameters.

TABLE 3.4.2.2.10. COMMON VALUES OF SOIL-STRUCTURE FRICTION ANGLES TO DETERMINE EARTH PRESSURES.

| STRUCTURAL MATERIAL | SOIL TYPE | | δ |
|---|---|---|---|
| Rubble mounds/Construction and Masonry/Concrete/Wood | Cohesionless | | 2/3 φ' |
| | Cohesive | Dry | 2/3 φ' |
| | | Saturated | 1/3 φ' |
| Steel | Cohesionless | | 1/3 φ' |
| | Cohesive | | 0 |
| Structure surfaces covered with asphalt, tar, bitumen, etc. | Any soil | | 0 |

NOTES :

(1) The effective internal friction angle ($\phi'$) to be used when determining angle $\delta$ shall be the breaking ($\phi'_f$) or the residual ($\phi'_r$) angle, in function of the earth pressure being calculated.

(2) If the structure or backfill is subject to important vibrations $\delta = 0$ shall be considered.
If the vibrations are caused by conventional, railroad, or loading equipment traffic, it shall be sufficient to reduce the angle $\delta$ derived from the table 5°.

The soil-wall friction has a big effect upon the determination of passive earth pressures, as large relative movements between the structure and the soil are necessary for the complete mobilization of these pressures. When the effect of the passive earth pressure is favorable for the strength or the stability of the analyzed structure, and the possibility of significant deformations between the soil and the structure is not guaranteed, $\delta = 0$ shall be considered in the calculation.

The friction angle ($\delta$) between the soil and the structure shall depend primarily upon:

— The characteristics of the soil (shear strength, consistency, humidity and the existence of groundwater flow).

— The roughness of the structure's surfaces.

— The soil-wall movements.

The value of $\delta$ shall never exceed the value of the internal friction angle of the soil ($\phi$), with a maximum value of 2/3 $\phi$ being accepted for the calculation, except where specially justified or where special precautions are taken.

In the absence of experimental data, the $\delta$ values given in table 3.4.2.2.10.- shall be adopted.

$a_{23}$) *INFLUENCE OF PORE PRESSURE VARIATIONS, WITH RESPECT TO THE HYDROSTATIC STATE IN THE DETERMINATION OF EARTH PRESSURES*

The influence of pore pressure variations, with respect to the hydrodynamic state, is taken into account in the determination of the earth pressures by introducing the resultant pore pressures along the sliding surfaces and along the back of the resistant structure in the trial-wedge methodology described in tables 3.4.2.2.2. (V) for active earth pressures and 3.4.2.2.6 (II) for passive earth pressures.

— *ZONES WITH GROUNDWATER FLOW*

The resultants of the pore pressures upon each surface shall be obtained by solving the filtration equation according to the criteria in section 3.4.2.1.- Hydraulic Loads, for the cases where circulation of water is established between different levels situated at different surfaces of the structural element, or through a drainage system.

For fundamentally vertical flow direction (vertical flow lines), usual in lineal maritime works when circulation of water between two invariable levels is established, the following corrections in the determined earth pressures without taking into account the existence of groundwater flow shall be accepted.

— *General*

The following shall be considered at each point as an increase or as a decrease in the values of the soil pressures.

- For active earth pressures : $\Delta p_a = \pm \Delta p \cdot K_a$
- For passive earth pressures : $\Delta p_p = \pm \Delta p \cdot K_p$

given that :

$\Delta p$ = variation of the pore pressure, with respect to the hydrostatic state in the analyzed point.
The sign + shall correspond to decrease (descending flow) and the sign - to increases (ascending flow).

$K_a$ = coefficient of the active earth pressure

$K_p$ = coefficient of the passive earth pressure

— *Constant Hydraulic Gradient Zones*

In those layers in which hydraulic gradients can be considered constant (see simplifications in the table 3.4.2.1.3) the earth pressures shall be calculated with a modified apparent specific weight ($\gamma'_r$) of the submerged soil in this layer, as follows :

$$\gamma'_r = \gamma' + i \cdot \gamma_w$$

given that :

$\gamma'$ = apparent specific submerged weight.
i = hydraulic gradient. Shall be considered positive for descending vertical flow and negative for ascending flow.
$\gamma_w$ = specific water weight.

— *ARTESIAN OVER-PRESSURE ZONES*

If a artesian water over-pressure ($h_w \cdot \gamma_w$) exists in a cohesionless layer, situated under a superficial layer of cohesive soil of a thickness $d_a$ and submerged specific weight of $\gamma'_a$, the determination of earth pressures shall be affected by this over-pressure.
The following simplifications in the calculations may be adopted in this case :
— *For active earth pressures :*

The influence of the artesian water over-pressure on the active earth pressures is generally of little significance, and can be neglected.

— *For passive earth pressures :*

- If $\gamma'_a \cdot d_a > \gamma'_w \cdot h_w$

  - A lineal reduction of the artesian water over-pressure in the cohesive layer is assumed. The passive earth pressure shall be calculated according to that anticipated for constant hydraulic gradient zones.

$$\gamma'_{ar} = \gamma'_a - \frac{h_w \cdot \gamma_w}{d_a}$$

The cohesion term of the passive earth pressure is not modified by the artesian over-pressure.

To calculate the passive earth pressure below the surface layer, the following surchage level is assumed :

$$\gamma'_{ar} \cdot d_a$$

- If $\gamma'_a \cdot d_a < h_w \cdot \gamma_w$

  - $\delta = 0$ in the surface layer shall be considered.

The cohesion term in the surface layer shall not be considered, in order to take into account possible heaving of the surface layer.

The passive earth pressure below the surface layer shall be calculated assuming a zero surcharge in this layer.

— *CONSOLIDATION PROCESS ZONES*

When the pore water pressures vary, with respect to hydrostatic pressures, due to consolidation processes in cohesive saturated soils, the resultant of these pressures can be obtained by the application of the Terzaghi-Frölich Theory for primary consolidation.

$a_{24}$) *EXISTENCE OF LOADS UPON THE GROUND*

Permanent loads and surcharge loads acting indirectly through the soil produce additional earth pressures upon the resisting structures.

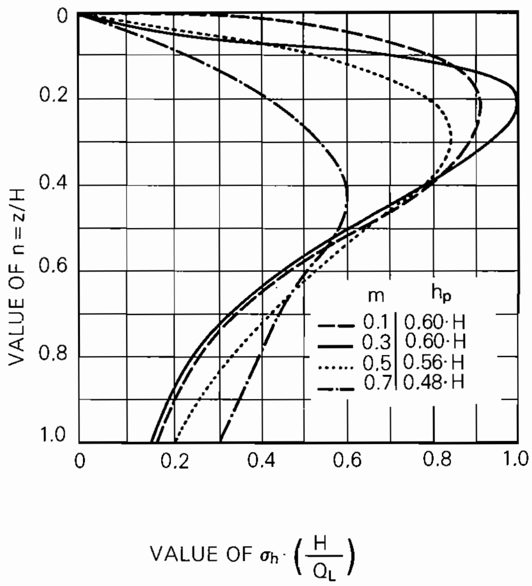
The evaluation of the additional earth pressures shall be carried out according to the following criteria, depending upon the type of acting load.

— *UNIFORMLY DISTRIBUTED VERTICAL LOAD*

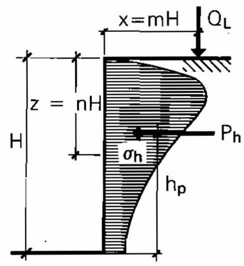
For an indefinite vertical load of value q per surface unit, situated upon a fill or homogenous natural soil, or stratified soil with layers parallel to the ground surface, located behind a resistant structure, the additional earth pressure produced shall be :

**TABLE 3.4.2.2.11** **ADDITIONAL EARTH PRESSURES UPON RETAINING STRUCTURES PRODUCED BY THE ACTION OF VERTICAL POINT OR LINE LOADS THROUGH HOMOGENOUS SOILS**
**(Vertical structure back and horizontal ground surface)**

LINE LOADS




LINE SURCHARGE $Q_L$




FOR $m \leqq 0.4$:

$$\sigma_h \cdot \left(\frac{H}{Q_L}\right) = \frac{0.20 \cdot n}{(0.16 + n^2)^2}$$

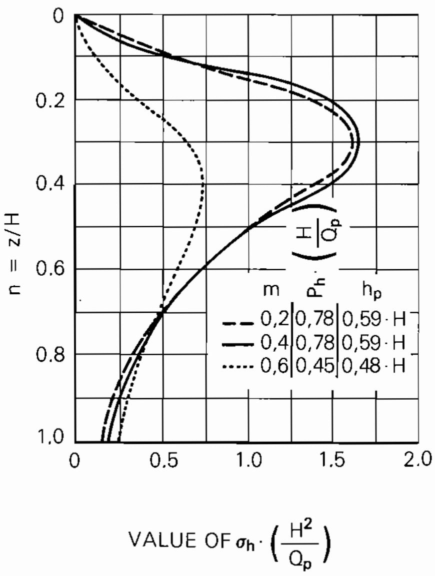
$$P_h = 0.55 \cdot Q_L$$

FOR $m > 0.4$

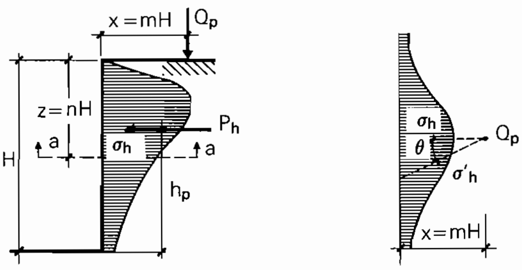
$$\sigma_h \cdot \left(\frac{H}{Q_L}\right) = \frac{1.28 \cdot m^2 \cdot n}{(m^2 + n^2)^2}$$

$$\text{TOTAL: } P_h = \frac{0.64 \cdot Q_L}{(m^2 + 1)}$$

POINT LOADS




POINT SURCHARGE $Q_p$




SECTION a—a

FOR $m > 0.4$:

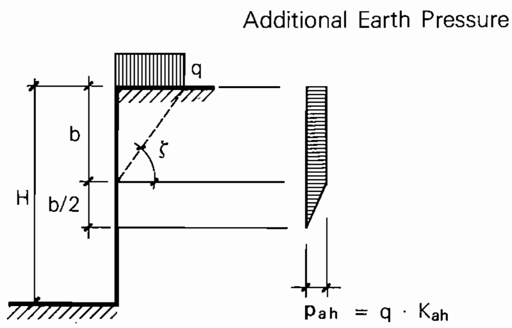
$$\sigma_h \left(\frac{H^2}{Q_p}\right) = \frac{1.77 \cdot m^2 \cdot n^2}{(m^2 + n^2)^3}$$

FOR $m \leq 0.4$:

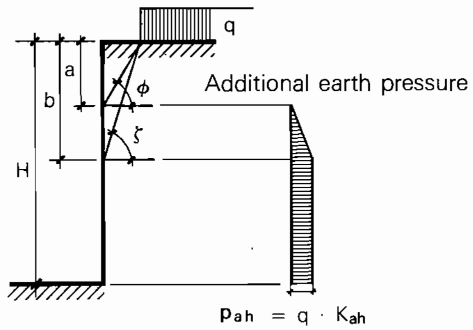
$$\sigma_h \left(\frac{H^2}{Q_p}\right) = \frac{0.28 \cdot n^2}{(0.16 + n^2)^3}$$

$$\sigma'_h = \sigma_h \cdot \cos^2 (1{,}1\ \theta)$$

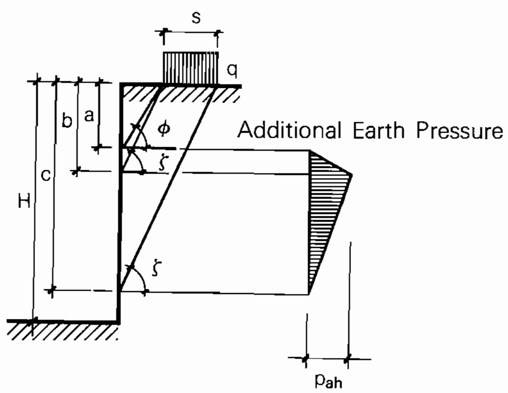
TABLE 3.4.2.2.12 ADDITIONAL ACTIVE EARTH PRESSURES, IN THE CASE OF LIMITED UNIFORM VERTICAL SURCHARGES, ON STRUCTURES WITH VERTICAL BACKS, HOMOGENOUS SOIL AND HORIZONTAL GROUND SURFACE




ζ: Failure surface angle corresponding to the active earth pressure according to the Coulomb theory. Can be obtained from the table 3.4.2.2.2. Active Earth pressure.




ϕ: Internal friction angle




$$p_{ah} = \frac{2 \cdot q \cdot s \cdot \theta}{c - a} \quad \text{for } \theta = \frac{\sin(\zeta - \phi) \cdot \cos\delta}{\cos(\zeta - \phi - \delta)}$$

TABLE 3.4.2.2.12 (Continued)

PLAN DISTRIBUTION OF ADDITIONAL EARTH PRESSURES FOR SURCHARGES ACTING ON LIMITED AREAS

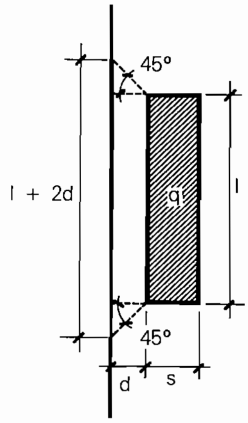
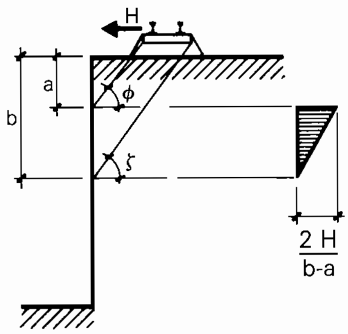



TABLE 3.4.2.2.13 ADDITIONAL EARTH PRESSURES, FOR INDEFINITE OR LIMITED HORIZONTAL LINE SURCHARGES, ON STRUCTURES WITH VERTICAL BACKS, HOMOGENOUS SOIL AND HORIZONTAL GROUND SURFACE




NOTES :

— $\phi$ and $\zeta$ are equal, as in table 3.4.2.2.12

— For limited horizontal line surcharges, the plan distribution shall be admitted with criteria equal to that for limited vertical loads (See Table 3.4.2.2.12)

$$\Delta p_a = K_a \cdot \frac{q \cdot \sin \alpha}{\sin(\alpha + \beta)}$$

$$\Delta p_p = K_p \cdot \frac{q \cdot \sin \alpha}{\sin(\alpha + \beta)}$$

The parameters have been defined in tables 3.4.2.2.2. and 3.4.2.2.5.

In purely cohesive soils, ($\phi = 0$), especially if the surcharge load does not have a permanent character, it shall be assumed that the active earth pressure cannot be less than $\Delta P_a = (q \cdot \sin \alpha / (\sin \alpha + \beta))$, which results in a correction in the upper part of the active earth pressure diagram.

— *VERTICAL POINT OR LINE LOADS PARALLEL TO THE TOP OF THE WALL*

When the point or line loads are small in relation to the total earth pressure produced by the soil (<0.30 x total active earth pressure), the additional earth pressure due to these loads can be calculated according to the methodology given in table 3.4.2.2.11 for vertical wall structures and homogenous soil with horizontal ground surface.

The earth pressures calculated in this way approximately equal those obtained by the theory of elasticity (Boussinesq). Boussinesq's theory is not applicable for rigid walls, given that it assumes a semi-infinite, elastic solid, with the possibility of expansion of the soil in contact with the wall; an evidentially false hypothesis for rigid walls.

For surcharge loads acting in strips, indefinite or not, the additional earth pressures may be calculated by the superposition of pressures produced by point or line loads. Likewise, the simplifications included in the table 3.4.2.2.12 for structures with vertical backs, homogeneous soil and horizontal ground surface can be applied.

For complicated sections, or for acting loads that don't meet the above conditions, the calculation of earth pressures shall be carried out including these loads in the analysis of sliding wedges according to the indications in tables 3.4.2.2.2., 3.4.2.2.5. and 3.4.2.2.6.

— *HORIZONTAL LINE LOADS PARALLEL TO THE TOP OF THE WALL*

For the action of horizontal line surcharges parallel to the top of the wall, the additional simplified earth pressures given in table 3.4.2.2.13. may be considered valid for structures with vertical walls, homogeneous soil and horizontal ground surface. Likewise, for complicated sections, the earth pressures may be obtained by the trial-wedge method already described.

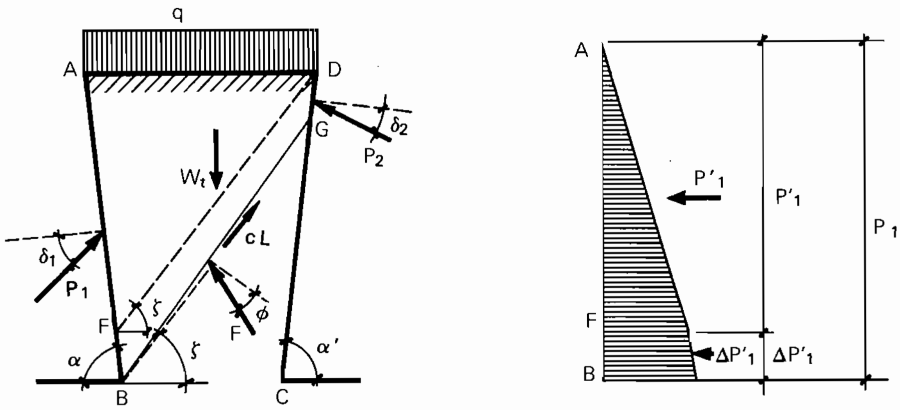
$a_{25}$) *SOIL PRESSURES IN SPECIAL CASES*

— *MODIFICATION OF EARTH PRESSURES ON CLOSE AND FACING STRUCTURAL ELEMENTS.- SILO EFFECT*

Modifications in anticipated earth pressures can occur in cases where structures are close to one another and facing each other (e.g. parallel walls, caisson cells, cellular cofferdams, etc) thus impeding the complete development of the sliding wedges as treated by the general theory.

These modifications can be resolved; according to Coulomb's theory shown in tables 3.4.2.2.2. and 3.4.2.2.5, by the following trial-wedge analysis :

FOR ACTIVE EARTH PRESSURES UPON AB




ABDG wedges shall be tried according to the general theory, calculating the pressures upon AF ($P'_1$) without taking into account the existence of the facing structure. The distance BF shall be obtained drawing a line from D parallel to the theoretical failure surface assuming the existence of a unique structure ($\zeta$).
Once $P_1$ is obtained; the earth pressure upon BF shall be equal to the increment with respect to $P'_1$ ($\Delta P'_1$), distributed lineally between F and B.

If point G, that corresponds to the real sliding wedge, is not located above the depth F, the prior methodology shown shall not be valid because of silo effects.

In this case, the following shall be adopted as ensilation pressures :

- Upon a vertical surface :

$$p'_h = \gamma \cdot z_0 \cdot (1 - e^{-z/z_0}) \cdot \lambda$$

$$p'_v = \gamma \cdot z_0 \cdot (1 - e^{-z/z_0}) \cdot \lambda \cdot \tan\delta$$

- Upon a horizontal surface

$$p''_v = \gamma \cdot z_0 \cdot (1 - e^{z/z_0})$$

- Upon an inclined surface :

$$p_h = p'_h \cdot \sin^2\alpha + p''_v \cdot \cos^2\alpha + p'_v \cdot \cos^2\alpha$$

$$p_t = \sin\alpha \cdot \cos\alpha \cdot (p''_v + p'_v - p'_h)$$

given that :

$p'_h$ : Horizontal earth pressure or a vertical surface, in t/m$^2$

$p'_v$ : Vertical earth pressure on a vertical surface, in t/m$^2$

$p''_v$ : Vertical earth pressure on a horizontal surface, in t/m$^2$

$p_n$ : Perpendicular earth pressure on a surface that forms an angle $\alpha$ with the horizontal, in t/m$^2$

$p_t$ : Tangential earth pressure upon a surface that forms an angle $\alpha$ with the horizontal, in t/m$^2$

$z_0$ : (critical depth) = $A/(u \cdot \lambda \cdot \tan\delta)$, in m z : Distance from the top of the ensilaged soil to the point where the earth pressures are evaluated, in m

A : Area of the straight section of the cell, in m

u : Perimeter of the cell in contact with the material, in m

A/u : (mean cell thickness)/2, in m

$\lambda$ : Earth pressure coefficient = $K_{ah}$ (dimensionless)

Nevertheless, the following shall be taken for granular fills, except rip raps and rubble mounds :

$\lambda$ = 0.5, when filling
$\lambda$ = 1.0, when emptying

$\delta$ : Soil-structure friction angle (in degrees) (see table 3.4.2.2.10)
Nevertheless, the following shall be taken for granular fills, except rip raps and rubble mounds :

$\lambda$ = 0.75 $\phi'$, when filling
$\lambda$ = 0.60 $\phi'$, when emptying
$\lambda$ = $\phi'$, when filling and emptying pulverized material
with $\phi'$ = Internal friction angle of the soil or fill

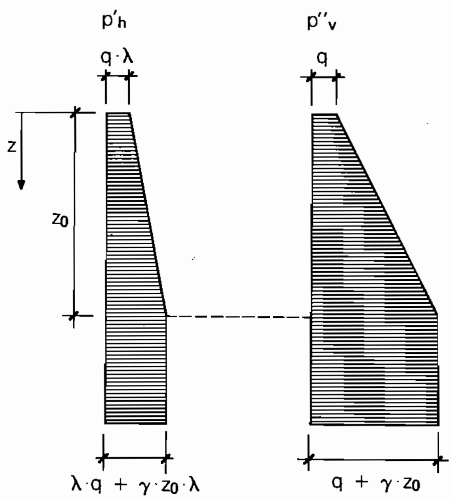
$\gamma$ : apparent specific soil weight, in t/m$^3$

$\alpha$ : angle that the inclined surface forms with the horizontal, (in degrees)

To take into account the dynamic effects that are produced during the emptying, the earth pressures obtained by the prior formulas shall be increased by 1.50.

If a uniformly distributed surcharge (q) is applied an additional uniform horizontal earth pressure equal to $q \cdot \lambda$ upon vertical planes and an additional uniform vertical earth pressure equal to q upon horizontal planes shall be considered.

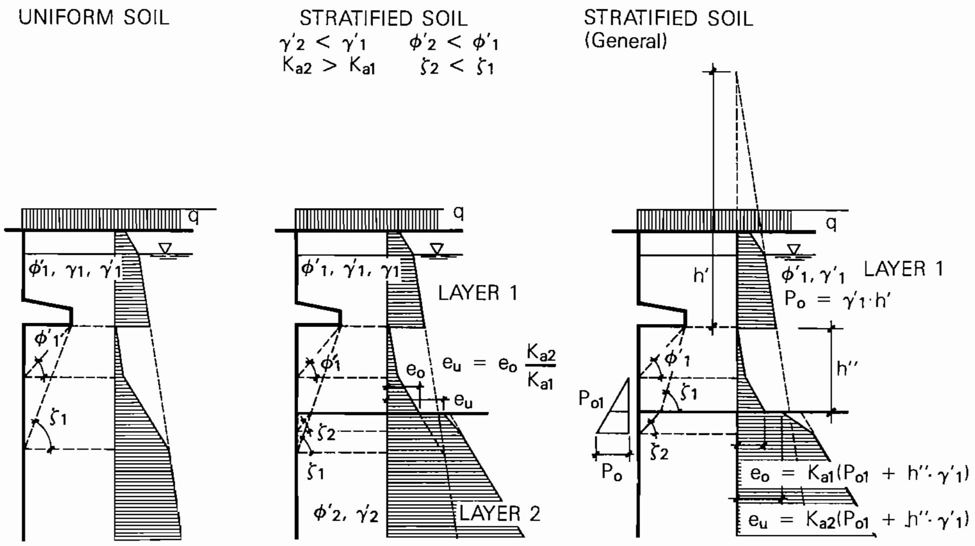
The theoretical earth pressure distributions may be simplified in the calculation, adopting conservative values.




— *MODIFICATIONS OF THE ACTIVE EARTH PRESSURES ON STRUCTURES WITH SHELVES ON THE BACK SIDE*

The modification in the distribution of active pressures produced by the existence of shelves behind retaining structures (e.g. slab on piles behind a diaphragm wall), shall depend on the localization and width of the cantilevered shelf and the shear strength parameters of the soil.
This type of structure is subject to reduced earth pressures due to the silo effect.

For cohesionless soils, the following distributions can be adopted :




$\phi_i$ = Effective internal friction angle in layer i.
$\zeta_i$ = Surface failure angle corresponding to active earth pressure in layer i.
$K_{ai}$ = Active earth pressure coefficient in layer i.

If the soil is cohesive, with cohesion c', the earth pressure distribution shall be carried out in the same way, by superimposing the cohesion term on the earth pressure distribution in the zone without ensilation effect :

$$K_{ac} = -2 \cdot c' \cdot \sqrt{K_a}$$

This procedure is valid if the cohesion term is small in relation to the total earth pressure.

For more complicated sections, or when various shelves exist, one above the other, successive active wedge trials shall be performed according to the methodology in table 3.4.2.2.2.

— *EARTH PRESSURE MODIFICATION DUE TO THE ACTION OF POINT OR LINE LOADS IN THE MASS OF THE SOIL*

When point or line loads act in the interior of the soil mass (e.g. loads due to pile foundations, footing foundations, anchors, etc.), the distribution of earth pressures upon the analyzed structure shall be modified by the presence of these loads starting from their application point (e.g. pile tip, bottom of footing, etc).
The analysis of earth pressures shall be carried out by the trial-wedge method, including in the force polygon the line loads whose application point is inside the sliding wedge. For point loads, the additional earth pressures shall be calculated by means of the application of the elasticity theory.
For the most simple cases, the simplifications in $a_{24}$. Existence of Loads upon the Ground can be applied starting from the application point of the load.

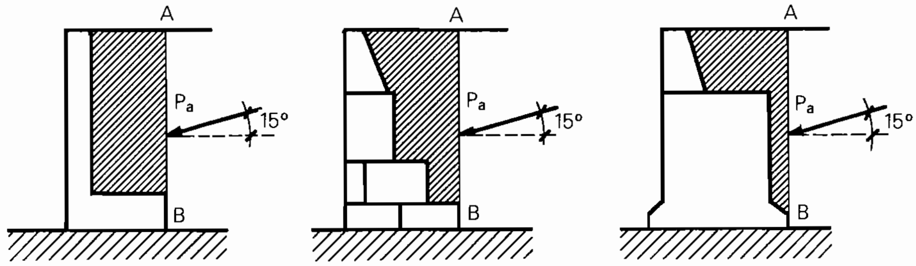
$a_{26}$) *PARTICULAR CASES*

— *SIMPLIFICATION FOR THE CALCULATION OF ACTIVE EARTH PRESSURES ON GRAVITY RETAINING STRUCTURES*

In gravity retaining structures, the following simplifications can be admitted :

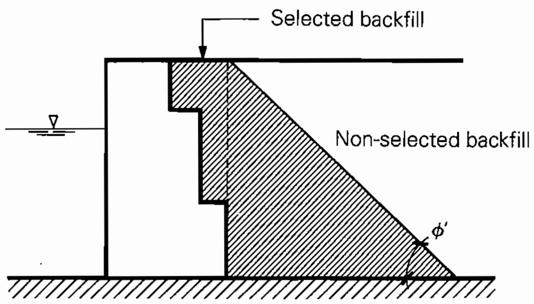
— For the overturning and sliding stability calculations of the retaining structure as a rigid body, the total earth pressure can be determined according to the stated methodology, but acting upon a virtual vertical back AB plotted from the toe of the structure's back, and taking a value of the soil/virtual back friction angle (δ) equal to 15°, independent of the structural type.

This consideration is based on the assumption that the soil mass that weighs on the analyzed structure moves, rotates or slids together with the structure.

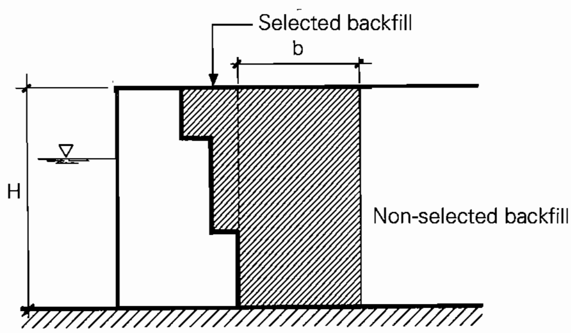



— When a good quality fill (selected backfill : internal friction angle equal to or higher than 30°) is used in the back of a gravity retaining structure, and has a horizontal surface, the following simplifications may be adopted in the calculations :

- If the section of the selected backfill is triangular, starting from the point of intersection between the vertical line that passes through the back toe of the resistant structure and the ground surface, forming an angle with the horizontal equal to or less than the internal friction angle of the selected backfill, it may be considered in the earth pressure calculations that all the soil behind the resistant structure has the identical characteristics as the backfill.
  If the non-selected soil is cohesive, or granular, with geotechnical characteristics different from the selected backfill, to take into account the reduction in earth pressure due to the good quality backfill; the impossibility of contamination or interpenetration of the second by the first soil shall be verified, otherwise, a material with filtering characteristics shall be placed in an intermediate phase (e.g. a classic granular filter or a geotextile).




- If the section of the backfill is rectangular, which is unusual due to construction difficulties, the following may be adopted :
  - When the width of the backfill (b), measured from the point of intersection between the vertical line that passes through the toe of the back of the structure, and the ground surface is greater than the height of the retaining structure (H), all the material situated in the back shall be considered to have the identical characteristics as the selected backfill.
  - If the width (b) is equal to 1/2 of the height, an average earth pressure distribution between that corresponding to the selected backfill and the rest of the material shall be adopted.
  - If the width (b) is equal to or less than 1/5 of the height, the effect of the selected backfill shall be considered negligible.




- If the section of the backfill is irregular, the effect of this backfill in the earth pressure distribution shall be equal to that caused by a regular section of equivalent slope or that caused by a rectangle of equal area.

— *EARTH PRESSURE ON SUPERFICIAL AND/OR DISCONTINUOUS ANCHORING BLOCKS AND ANCHORING DIAPHRAGM WALLS ($H/h \leq 2$)*

In the unloaded state, the at rest earth pressure acts on the two faces of the continuous diaphragm wall or continuous anchorage block. In the limit, it shall be considered that the effect of the anchoring force completely mobilizes the soil's passive earth pressure, assuming that the necessary displacement take place for the development of these pressures. Simultaneously, the mobilization of active earth pressures in the back of the anchoring wall by the soil expansion in this zone shall be assumed.
The sliding surfaces and the forces on the anchoring diaphragm wall are given in table 3.4.2.2.14. The difference between the total passive and active earth pressures shall be the usable anchoring pressure.

In order that the hypotheses that are carried out on the sliding surfaces can be considered valid, the failure plane of the anchor wall's passive wedge must not intersect, below the ground surface plane, with the failure plane of the anchored wall's active wedge. (See table 3.4.2.2.14).
Likewise, to guarantee the stability of the anchored wall/anchoring structure as a whole, the most unfavorable sliding surface that generally passes through the toe of the anchored wall and the toe of the anchoring structure shall be checked.

In the case of non-continuous and separated anchorage blocks or diaphragm walls, the effect of the set of anchors may be analyzed as an equivalent continuous wall, layed out according to the criteria in table 3.4.2.2.14, whenever the separation between isolated anchors does not exceed a critical length. This critical length (crit a) shall depend upon the height of the anchor (h), and upon the relationship between the anchor depth with respect to the ground surface (H) and h. Lacking other data, the following shall be admitted for granular soils :

$$\text{crit } a = h \cdot \beta \qquad \text{for } H/h \leq 5.5$$

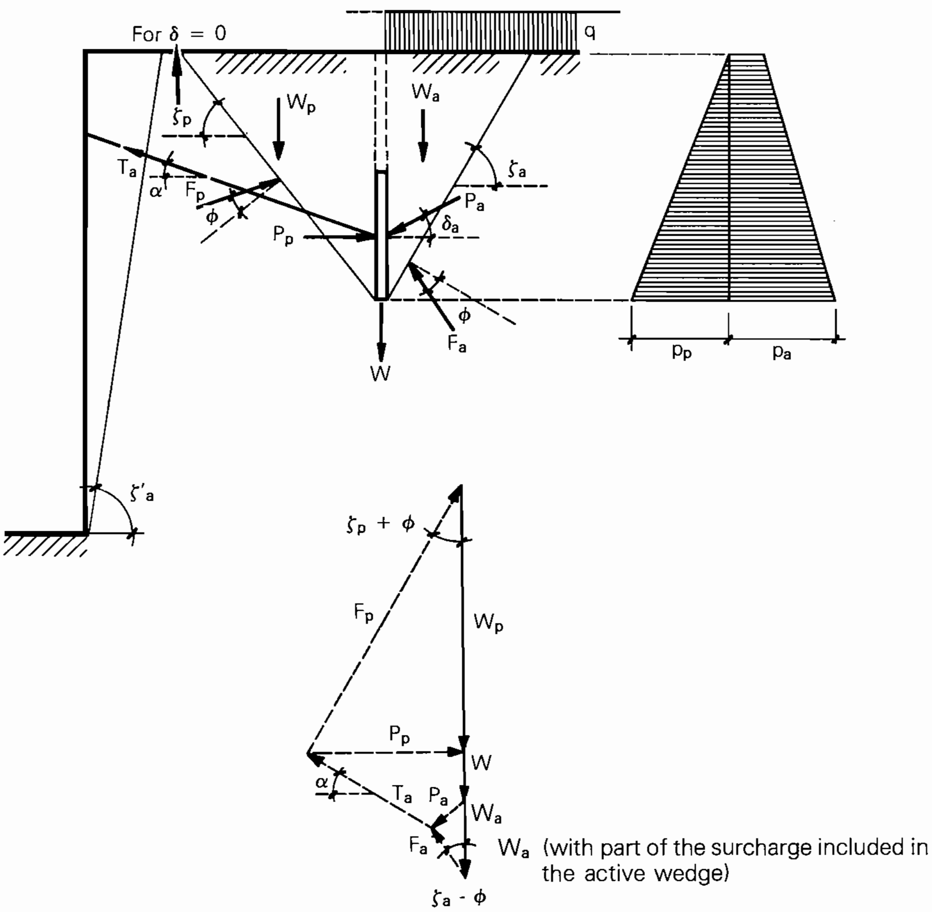
Obtaining $\beta$ from :

| H/h | 1 | 2 | 3 | 4 | 5 | 5.5 |
|---|---|---|---|---|---|---|
| $\beta$ | 2.1 | 2.3 | 2.5 | 2.8 | 3.1 | 3.3 |

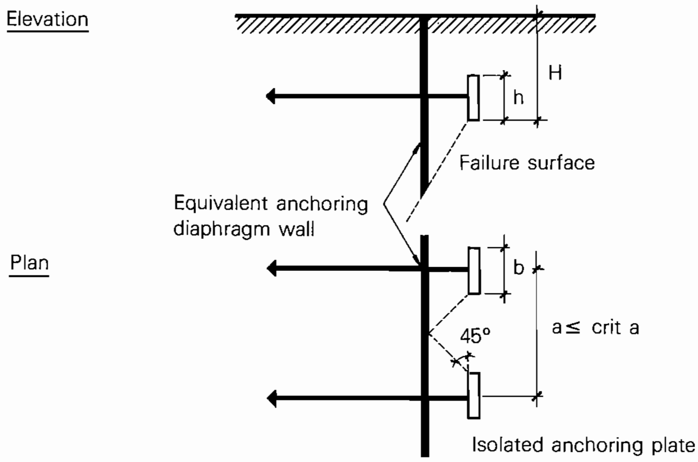
Valid for square plate or for rectangular plate taking h equal to the side of the equivalent surface area.

TABLE 3.4.2.2.14 FAILURE SURFACES, FORCES AND EARTH PRESSURES UPON AN ANCHORING DIAPHRAGM WALL

CONTINUOUS ANCHORING PLATE




EQUIVALENT CONTINUOUS DIAPHRAGM WALL PLAN FOR ISOLATED ANCHORING

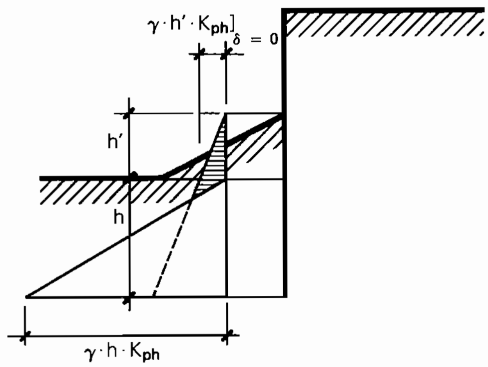



**FOR LEGEND :** SEE TABLES 3.4.2.2.2 AND 3.4.2.2.5.

— *PASSIVE EARTH PRESSURES IN SUPPORTING SLOPES*

The passive earth pressure distribution in supporting slopes can be precisely determined by the trial-wedge method shown in tables 3.4.2.2.5 and 3.4.2.2.6. However, the simplified procedure shown in table 3.4.2.2.15, valid for homogeneous soils and vertical wall structures, may be used.

**TABLE 3.4.2.2.15 ADDITIONAL PASSIVE EARTH PRESSURES IN SOILS WITH SUPPORTING SLOPES AND VERTICAL WALL STRUCTURES**




a$_3$) INDEFINITE RIGID WALL STRUCTURES WITH LATERAL DISPLACEMENTS RESTRICTED BY EXTERIOR SUPPORTS

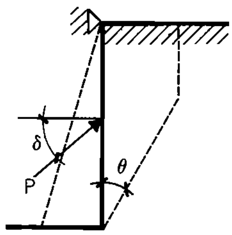
When a soil mass includes zones whose displacement is limited by exterior unions (e.g. by an anchor), at the moment of failure local equilibriums, analogous to the vaults that form between the interior walls of a silo, shall be formed in these zones, resulting in modifications with respect to the active state.
The modification of the internal equilibrium conditions shall produce modifications in the active earth pressure distribution due to a reduction of the soil capacity in transmitting vertical loads to the lower layers of the breaking soil mass.

In rigid wall structures with lateral displacements restricted by exterior supports the following shall be considered, with respect to the magnitude and distribution of active soil pressures :

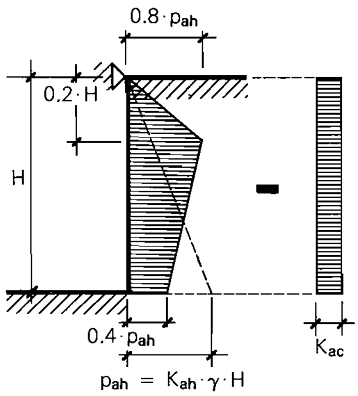
— Increase of the total value of the pressure, with positive inclination of the resultant force due to mobilization of the soil-wall friction.
— Increase in the unit earth pressures in zones near the fixed points.
— Reduction of the unit earth pressures in zones away from the fixed points.

The earth pressure distributions for these cases are of very difficult systemization and generalization, though some theoretical simplifications in concrete cases based upon experimentation and model measurement have been obtained.

In the particular case of rigid wall structures founded on rigid soil, and with displacement restricted at the top of the wall, the failure surface shall be assimilated to a segmented surface, formed by a vertical surface in the upper part and by an inclined surface, with slope angle θ with respect to the wall, that passes through its toe.




According to Terzaghi, the earth pressure distribution is approximately parabolic, enabling simplification to a distribution formed by two straight lines.
For vertical wall structures and homogeneous soil with horizontal ground surface, the following approximation may be applied :



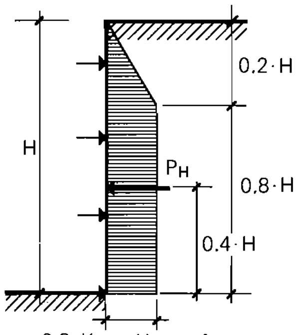
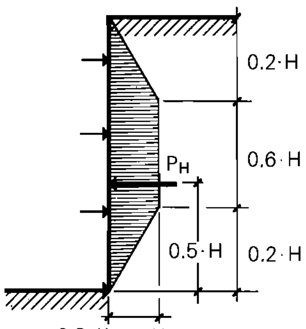
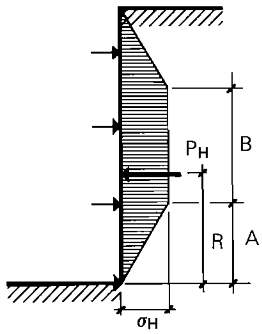

$a_4$) INDEFINITE FLEXIBLE WALL STRUCTURES

The possibility of deformation of flexible wall structures (e.g. anchored or unanchored sheet-piling) also modifies the failure conditions of a soil acting upon this structure, with respect to those defined for active and passive states.
The flexibility of the wall permits the decompression of the soil in its immediate proximity, resulting in increases in the unit earth pressures in the proximity of the supports and decreases in the intermediate zones, with respect to the cited states.

The theoretical calculation of the failure states and their associated earth pressures is not simple, and specific studies (e.g. mathematical model studies) are necessary when the deformation capacity of the wall is taken into account together with the soil's deformability. For the calculation of the magnitude and distribution of earth pressures, empirical methods of recognized validity shall be admitted for some structural types (Blum's method, bearing capacity methods of Brinch-Hansen, Danés, Rowe, Tschebotarioff, etc.).

In diaphragm walls or excavation shore walls with multiple struts or anchors the non-deformable behavior of these elements, together with the possible deflection of the wall between anchors or struts disallow the distribution and magnitude of the earth pressures to approximate those corresponding to the active state. For this particular case, the empirical method given in Table 3.4.2.2.16, valid for homogenous soil with horizontal surface, shall be adopted. The additional earth pressure caused by surcharges shall be taken into account in the same way as the active earth pressure.
The total earth pressures with the proposed distribution result in increases of 25% for dense granular soils and 50% for loose granular soils, with respect to the active pressures.

**TABLE 3.4.2.2.16 EARTH PRESSURES IN DIAPHRAGM WALLS AND EXCAVATION SHORE WALLS WITH MULTIPLE STRUTS OR ANCHORS**

| LOOSE GRANULAR SOILS | DENSE GRANULAR SOILS | COHESIVE SOILS |
|---|---|---|
| <br><br>$P_H = 0.72 \cdot K_a \cdot \gamma H^2 \cdot \cos\delta$ | <br><br>$P_H = 0.64 \cdot K_a \cdot \gamma H^2 \cdot \cos\delta$ | <br><br>$2 < N_0 < 5$<br>$P_H = 0.78 \cdot H \cdot \sigma_H$<br>$\sigma_H = \gamma \cdot H - 1.5 \cdot (1+N_0) \cdot c$<br>$A = 0.15H;\ B = 0.55H;\ R = 0.46H$<br><br>$5 < N_0 < 10$<br>$P_H = 0.78 \cdot H \cdot \sigma_H$<br>$\sigma_H = \gamma \cdot H - 4c$<br>$A = 0.15H;\ B = 0.55H;\ R = 0.46H$<br><br>$10 < N_0 < 20$<br>$P_H = (21 - 0.055N_0)H\sigma_H$<br>$\sigma_H = \gamma \cdot H - (8 - 4N_0) \cdot c$<br>$A = (3 - 0.015N_0) \cdot c$<br>$B = (1.1 - 0.055N_0) \cdot H$<br>$R = 0.38H$<br><br>$N_0 > 20$<br>$P_H = 0.5 \cdot H \cdot \sigma_H$<br>$\sigma_H = \gamma \cdot H$<br>$A = B = 0$<br>$R = 0.33H$ |

**LEGEND :**

$N_o$ (stability number) $= \dfrac{\gamma \cdot H}{c}$

$\gamma$ : Soil's apparent specific weight
H : Height of the retaining element upon the excavation base
c : Cohesion
$K_a$ : Active earth pressure coefficient
$\delta$ : Soil-wall friction angle

$a_5$) DISCONTINUOUS STRUCTURES

— *EARTH PRESSURES ON ISOLATED ELEMENTS*

In general and without very detailed studies, in narrow structural elements (e.g. piles, pillars, etc) the unit earth pressures per linear meter shall be calculated as those corresponding to indefinite structures, multiplied by 3 times the width of the isolated element in the direction perpendicular to the earth pressures, to take into account the formation of prismatic failure wedges.

For alignments of narrow isolated structural elements (e.g. non-continuous walls formed by piles) the corresponding earth pressures may be approximated considering an equivalent apparent structure of width ($L_e$) equal to :

$$L_e = L_t + 2B_t$$

given that :

$L_t$ : Length between the centers of extreme elements in the direction perpendicular to the earth pressure.

$B_t$ : Width of the equivalent area to the group in the direction of the earth pressure ($A_t = B_t \cdot L_t$).

as long as the earth pressure, that is obtained in this way, does not exceed the sum of the pressures calculated on the isolated elements.

— *SHIELDING EFFECT OF AN ALIGNMENT OF NARROW STRUCTURAL ELEMENTS*

When one or several narrow isolated structural element alignments (e.g. piles) are situated near a wall, interfering with the soil failure wedge, they shall produce reductions in the earth pressures acting upon the protected wall in such a way that the total theoretical earth pressure shall be divided among the alignments of isolated elements (n %) and the wall ((1 – n)%).

The shielding effect shall be a function of the spacing between elements in the alignment, the width of the elements in the direction of the alignment, and the characteristics of the soil. The shielding effect shall increase, for a constant spacing, with the width of the elements, with the internal friction angle and with the soil cohesion.

If we define B as the width of the isolated element in the direction of the alignment (perpendicular to the direction of pressure) and L as the center to center spacing of elements, it may be assumed that for fine granular soils the shielding effect is completely effective when $(B/L) \geq 0.5$.

Lacking tests or other experimental data, for fine granular soils (sandy soils), the following n values may be adopted :

| B/L | | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
|---|---|---|---|---|---|---|
| n | 1 Alignment | 0.46 | 0.68 | 0.84 | 0.92 | 1.00 |
| | 2 Alignments | 0.52 | 0.80 | 0.92 | 0.96 | 1.00 |

If a diaphragm wall effect is produced, the pressures upon the wall and upon the alignment of isolated elements shall be :

— *Unit earth pressure upon the wall.*

The unit earth pressure upon each point of the wall shall be equal to the larger of the two following values :

- Earth pressure transmitted through the isolated element.

  For active earth pressure :

$$p_a = K_a \cdot \sigma' \cdot (1 - n)$$

- Fictitious silo earth pressure, with $z_0$ = (wall-isolated element separation)/($2 \cdot K_a \cdot \tan \delta$) (See $a_{25}$ of section 3.4.2.2.).

— *Unit earth pressure upon the isolated elements.*

The unit earth pressure upon each point shall be equal to the difference between the earth pressures that are exerted on the two faces of the alignment :

## $b_4$) LATERAL EARTH PRESSURES DUE TO THE PHENOMENON OF INSTABILITY OF THE SOIL OR THE STRUCTURE

In those cases where underground structures retain or pass through masses of potentially unstable soils (e.g. progressive sliding slope, expansive soil, etc) the lateral earth pressures caused by the soil at the moment of instability shall be taken into account in the calculations. These loads shall be considered accidental loads.

— *ALIGNMENTS OF ISOLATED NARROW STRUCTURES THAT RUN THROUGH SOIL MASSES THAT ARE POTENTIALLY SLIDING (E.G. PILES)*

Potentially sliding masses shall be considered those that have a sliding safety factor (F) < 1.3 in the service phase, except in exceptional work conditions (accidental hypothesis). In this hypothesis or in the construction phase, potentially sliding masses shall be considered if (F) < 1.1. The coefficient shall be obtained through the Bishop Method without considering the stabilizing effect of the isolated structure that goes through it.

Lacking other more detailed studies, it can be assumed that breaking soil masses, sliding through the alignment of isolated narrow underground structures produce approximately triangular distributions of lateral earth pressures from the exterior surface of the sliding mass to the failure surface.

The isolated structure, below the potentially sliding zone, shall be subject to earth pressures calculated by empirical methods of recognized validity for this structural type (e.g. Blum's method).

Lacking more detailed studies, and without disregarding alternative formulations (Ito, Lorenzo de No, Matsui, etc), the magnitude of the maximum earth pressures may be estimated according to the criteria in table 3.4.2.2.17, obtained from the formula corresponding to a failure load of a shallow foundation (Brinch Hanson load factors). This formula shall be valid whenever the resultant forces resisted by the piles are not greater than those necessary for the equilibrium of the soil mass (earth pressure per linear meter less than the active pressure upon the back of the equivalent wall with its base at the failure line). Therefore, it shall only be valid beyond a certain center to center element spacing in function of the geometry of the slope, the transversal dimension of the isolated element, the depth of the sliding surface, and the properties of the soil ($\gamma$, $\phi$, and c). This relationship shall increase with $\phi$, c and B.
In the case of large overloads occuring in the work, specific studies shall be done.

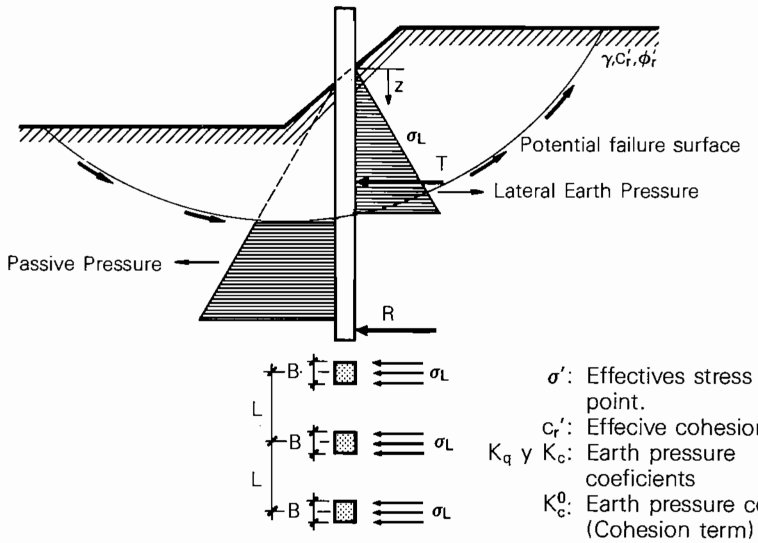
When various parallel or close structural alignments are included, to calculate the lateral earth pressure that acts upon each alignment, the shielding effect of one upon the other may be taken into account, based upon specific studies or experimentation. Given the difficulties of evaluation, the defined earth pressure acting upon each alignment according to table 3.4.2.2.17, shall be admitted.

The stabilizing effect due to the inclusion of alignments of isolated structures in a potential sliding zone, may be taken into account in the safety factor against sliding of the soil mass, introducing as a stabilizing force the resultant, per unit length, of the lateral earth pressures upon the structure in the interior of the failure circle (T / L of table 3.4.2.2.17). In general it is not adequate to attempt to resist sliding by means of piles due to the enormous forces that can be generated; nevertheless, due to the necessary presence of piles for supporting certain maritime structures it may be worthwhile to consider thier contribution to slope stability.

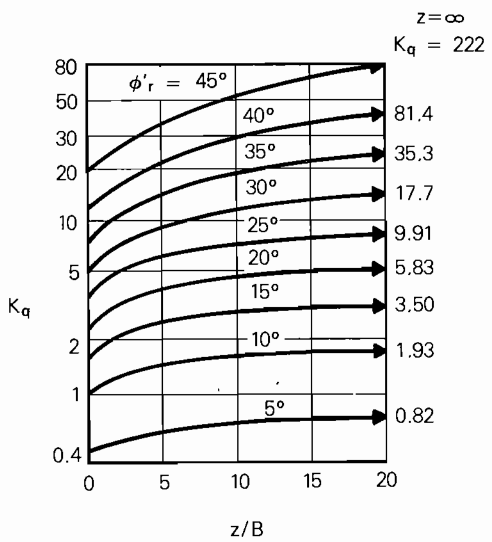
Likewise, when the structure is subject to structural instability problems (e.g. substantial unsupported pile lengths, or long underground piles in soft soil), it may be considered that the soil exerts a lateral stabilizing action. The performance of this action shall fundamentally depend upon the type of soil and upon the instability phenomenon analyzed, determining if more detailed studies should be considered.

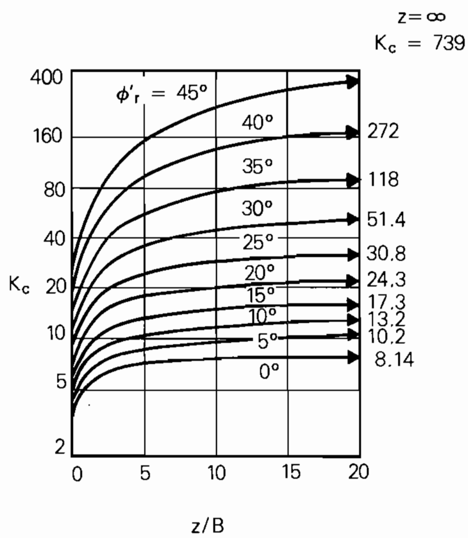
## TABLE 3.4.2.2.17 LATERAL EARTH PRESSURES UPON NARROW ISOLATED UNDERGROUND STRUCTURES IN POTENTIALLY SLIDING MASSES

- Soils with $\phi$ and c: $\sigma_L = K_q \sigma' - K_c c'_r$
- Soils with c only ($\phi = 0$): $\sigma_L = \sigma' - K_c^0 c'_r$




$\sigma'$: Effectives stress at considered point.
$c_r'$: Effecive cohesion.
$K_q$ y $K_c$: Earth pressure coeficients
$K_c^0$: Earth pressure coefficient (Cohesion term) for $\phi = 0$



■ DYNAMIC EFFECTS

Independently of the modifications produced in the earth pressures during seismic load actions, analyzed in the ROM 0.4.- Consideration of Environmental Actions/II : Atmospheric and Seismic Conditions, the following dynamic effects must be taken into account in the calculation :

— If the structure or the soil is subject to the action of significant repeating loads (e.g. wave loads, mooring loads, equipment and machinery vibrations), lacking specific studies, $\delta = 0$ shall to be taken.
  If the performance of these loads gives rise to densifications or compactions of the natural soil or fill, increases of 5% in the internal friction angle and of 7% in its apparent specific weight may be considered.

— If the structure of the soil is subject to the vibrations produced by conventional or railway traffic loads, the values of $\delta$, suggested for the static state shall be reduced 5°.

— To take into account dynamic effects that are produced during the filling and emptying of structures affected by ensilation phenomena, earth pressure amplification factors, earth pressure coefficient modifications ($\delta$) and soil-structure friction angles shall be adopted according to section $a_{25}$. Earth Pressures in Special Cases.- Modifications of earth pressures upon close and facing structural elements.- Silo Effect.

■ ACTION DIFFERENTIATION

a) BY PROJECT PHASES

For certain types of structures such as sheet piling, anchored walls or excavation shore walls, the earth pressures produced during construction (e.g. pressures prior to the loading of the anchors or struts; transitory thrust pressures during the excavation) can be critical for dimensioning and shall taken into account in the calculations. Some construction methods (flooding and filling of cellular structures, hydraulic fills or construction sequences) can give rise to significant variations in the earth pressure distributions with respect to the service state, and shall be checked.
When the project specifically includes construction procedures that involve distributions, types and values of differentiated pressures (e.g. anchoring phases, tensioning, piling and subsequent excavation, dredging after construction, etc) during the construction phase, these states shall be taken into account in the calculations.
In particular, for hydraulic fills, it shall be taken into account that the flooding velocity can exceed the drainage capacity. In the calculations, a phreatic level coinciding with the lowest level at which the water can freely flow shall be taken for the construction phase. Likewise, in pumped hydraulic fills above the phreatic level, the variation in time of the fill's specific weight and pore pressure shall be considered in the construction phase. The soil shall primarily behave as a fluid with suspended solids until its sedimentation and subsequent compaction by dissipation of the pore pressures.

When the project includes compaction, consolidation or soil improvement processes, the effects produced by these processes during the construction phase shall be considered in the calculations. Generally, they can cause, among other things; modifications in the soil's apparent specific weights and in the internal friction angles, cancellations of the soil/structure friction angle, variations in the pore pressures with respect to the hydrostatic state, increases in the loads acting upon the soil and the possibility of negative friction and lateral pressures.

For service phases, the distinction between soil loads in normal conditions and extreme operating conditions shall basically be a function of phreatic levels (See section 3.4.2.1. Hydraulic loads) and of the existence of loads upon the soil or upon the structure associated with each one of these conditions.

In exceptional conditions, the possible independent and non-simultaneous occurance of the following accidental loads, whose quantification is included in section 3.4.2.4. Environmental Loads and 3.4.3. Accidental Loads shall be considered :

— Environmental actions in extraordinary conditions
— Inundations due to breakage of installations, canals and deposits

— Drainage system failures
— Unforeseen overtopping of waves
— Scouring produced by ship propellers or currents
— Collisions and local exceptional overloads.
— Failure or sliding of slopes
— Overdredging

These accidental actions shall cause additional earth pressures, that shall be considered accidental soil loads.

b) BY STRUCTURAL TYPES

The distribution and magnitude of the earth pressures that a soil produces upon a resistant element is a function of, among other parameters, the typology and deformation capacity of the structure and its foundation. Because of this, the systemization in the determination of soil loads is based upon four structural types. (See a1.- of this section).

### 3.4.2.3 VARIABLE USE LOADS ($Q_{kv}$)

#### 3.4.2.3.1 STAGE AND STORAGE OVERLOADS ($Q_{V1k}$)

■ DEFINITION

Stage and Storage Overloads are defined as loads of a variable nature fundamentally due to the weight of materials, supplies or cargo, stored or stacked directly inside the specific installations, such as silos, warehouses, or sheds, or inside auxiliary installations for their transport and handling such as containers, trailers, etc. Their action and distribution is constant during relatively long periods of time.

■ DETERMINATION

The value of the action shall be determined, taking into account the designated use of the area and the way that the action effects the resistant structure (directly, through load distributing layers or upon a fill); taking into consideration :

— The nature of the stored or stacked material (bulk cargo, general cargo, containers, etc)
— The form and maximum dimensions of the stored or stacked material
— The maximum quantity that can be handled
— The handling methods and equipment
— The nature and characteristics of the storage or support structure (containers, trailers, sleepers, etc).

The stage and storage overloads shall be treated as distributed or concentrated vertical loads according to the nature of the materials, their manner of application or support and whether or not elements of distribution exist. The overloads shall act along the entire surface, or on part of it, according to what is most unfavorable for the analyzed structural element, or their distribution shall be limited by the compatibility with the cargo handling installation overloads or traffic loads.

Generally, regardless of the structural or support element characteristics, the stage and storage overloads shall only be treated as distributed loads, since this distribution is critical for the design of the majority of common structures. Nevertheless, especially with structures that have short spans, slabs, or other types of elements highly sensitive to local effects, and whenever there is the possibility of large concentrated loads acting directly upon the resistant structure (e.g. container quays, metal product yards, or large parts in shipyards), the structural shall also be checked alternatively with concentrated loads.
In these cases, only for stability calculations, shall the large concentrated loads be converted into equivalent uniform loads.

— AREA DIFFERENTIATION ACCORDING TO USE

In order to systemize the evaluation of variable use overloads, the following areas shall be differentiated for maritime works :

— *Operation Areas*

Zones designated for the transfer and handling of cargo, materials or supplies, and where these are not accumulated.

— *Storage Areas*

Zones designated for prolonged stays of cargo, materials or supplies, where their accumulation is permitted.

— *Service Areas*

Zones excluded from cargo, material or supply traffic. They shall basically be habitable, administrative service or recreation areas.

— *Traffic Tracks*

Zones designated exclusively for the transit of cargo, materials or supplies between the Operation Areas and the Storage Areas, and between these two and Exterior Areas. Also included are those zones designated for carrying the construction traffic. These tracks shall be differentiated as Maneuvering Tracks and Access Tracks.

The project engineer shall geometrically fix and include in the project documents the distribution of areas that affect the maritime works, according to the divisions above, as a function of the prior given general port planning criteria.
For this purpose, if prior planning or specific criteria from the Client or Government Authority does not exist, the following may be adopted :

— Operation Area : Band parallel to the edge of a berth that extends from this edge up to 5.00 m behind the center line of the innermost crane foot tracks, with a width of not less than 15.00 m.
Lacking a definition in the project of the cargo handling installations, the narrowest band shall be considered the operation area.
The existence of this area is conditional, based upon the verification that the cargo transfer process compels its exclusive use for transitory parking. Otherwise, the operation and storage areas shall not be differentiated.

— Storage Area : Remaining areas not included in the Operation Areas.

To consider stage and storage overloads, only operation and storage areas shall be distinguished, with the object of taking into account that traffic tracks and service areas can be reconverted during the design life of the work. Only those works specifically designated for services or traffic tracks (e.g. office building, living area on a petroleum platform, access bridge to a port, etc) shall be considered as service areas or traffic tracks, respectively. In this last area, no parking or storage overloads shall be considered.

Once the area or areas that have an influence on the conception and loading of the resistant structure are determined, the following shall be taken as values of the stage and storage overloads :

a) DISTRIBUTED LOADS

The stage and storage overloads considered as distributed loads shall be equal to the maximum weight of the cargo, material or supplies per unit area, temporarily parked in the operation area or stored in the storage area, in the projected stowage conditions.

The following may be taken for each elemental area .

$$Q_{V1} = \gamma \cdot H_a, \quad \text{in t/m}^2$$

given that :

$\gamma$ = Apparent specific weight of the staged or stored material in the most unfavorable environmental conditions, in $t/m^3$.

For materials with little drainage capacity subjected to environmental actions or constant irrigation, the saturated specific weight shall be used.

For the determination of apparent specific weights, the project engineer shall ascertain in agreement with preestablished general and port planning criteria, or lacking that, according to specific Client or Government Authority criteria, the nature of the cargo, materials and supplies to be handled and stored (solid bulk cargo, general cargo, petroleum products, etc) as well as the type and characteristics of the staging or stacking (outside or inside auxiliary installations; bulk, boxed, bagged, coiled, in ingots, etc).

Lacking other data, the values of apparent specific weights given in table 3.4.2.3.1.1.- shall be taken for the most common materials in port zones during storage.

$H_a$ = Maximum storage or stacking height of the considered material, in m.

The maximum height shall basically depend upon :

- Considered area (operation or storage) and its use (commercial, fishing, industrial, etc)
- Nature and type of cargo
- Form of packing or storage (Bulk, contained or packaged) (prismatic, conical, pyramidal, etc)
- Installations and handling methods
- Storage place (outer yards or specific installations such as sheds, silos, deposits, retention walls, etc)

being limited by the following parameters :

- Available plan space
- Angle of repose of solid bulk cargos (tan $\phi$, given that $\phi$ is the internal friction angle of the material)
  Lacking other data, the internal friction angles given in table 3.4.2.3.1.1- shall be used for the most common solid bulk cargos in port areas
  It shall be considered that the internal friction angle is constantly maintained independent of the degree of saturation of the material
- Free height in the interior of the storage structures
- Height reachable with the handling machinery
- Strength of the containers, packages or other auxiliary elements
- Commercial regulations and customary practices.

Lacking other specific Client or Government Authority prescriptions, the Project Engineer shall, with justification, set the maximum stacking and storage height, based on the use conditions established for the zone and analyzed area, taking into consideration the parameters listed in the prior paragraph.

Independent of the indicated limitations, the common values of $H_a$ in open yards are given in table 3.4.2.3.1.2.

In general, the distributed loads shall be considered uniformly distributed with a value :

$$0.8 \cdot Q_{V1}]_{peak}$$

with $Q_{V1}]_{peak}$ being the maximum overload per a unit area ($Q_{V1}$ for maximum $H_a$). The correction factor shall be introduced in order to take into account the low probability that the operation or storage areas are loaded to maximum height in 100% of the surfaces that produce the most unfavorable effects.

In general, this correction shall be applicable for all cases (especially in open yards), but shall not be applied in installations that are exclusively for bulk material (e.g. silos or deposits).

**TABLE 3.4.2.3.1.1. APPARENT SPECIFIC WEIGHTS AND INTERNAL FRICTION ANGLES OF COMMON CARGOS STORED IN PORT ZONES**

| BULK MATERIALS | $\gamma$ (t/m$^3$) | $\phi$ (°) | STACKED MATERIALS | $\gamma$ (t/m$^3$) |
|---|---|---|---|---|
| A) SOLID BULK MATERIALS | | | — Ores | |
| | | | Bauxite (bagged) | 0.90 |
| — Ores | | | Chromium (boxed) | 2.50 |
| Alumina | 1.70 | 35° | Magnesium (bagged) | 1.50 |
| Aluminium (bauxite) | 1.50 | 50° (h) | Nickel (bagged) | 1.65 |
| | | 28° (d) | Nickel (in barrels) | 1.45 |
| Chromium | 2.60 | 40° | | |
| Cooper (Pyrites) | 2.60 | 45° | — Metal. Steel and Iron Products | |
| Iron (limonite and Magnetite) | 3.00 | 40° | Aluminium (in ingots) | 1.25 |
| Lead (Galena) | 2.80 | 40° | Cooper (coiled) | 1.10 |
| Magnesium | 1.50 | 35° | Cooper (in ingots) | 3.50 |
| Manganese | 2.40 | 45° | Cooper (in plates) | 3.50 |
| Roasted pyrite | 1.40 | 45° | Steel (coiled) | 2.80 |
| Tin (Cassiterite) | 2.00 | 38° | Steel (in bars) | 3.00 |
| Zinc (Zincblende) | 1.80 | 38° | Steel (in ingots) | 3.60 |
| | | | Steel (in plates) | 3.50 |
| — Chemical Products | | | Tin (in ingots) | 3.40 |
| | | | Zinc (in ingots) | 2.50 |
| Artificial fertilizers | 1.20 | 40° | | |
| Carbide | 0.90 | 30° | — Chemical Products | |
| Mineral fertilizer | 1.20 | 30° | Fertilizers (bagged) | 0.90 |
| Phosphates | 1.10 | 35° | Potash (bagged) | 1.00 |
| Potash | 1.10 | 35° | Sulphur (bagged) | 1.00 |
| Sulphur | 1.20 | 40° | Sulphur (in barrels) | 0.75 |
| — Solid Fuels | | | — Solid Fuels | |
| Charcoal (crushed) | 0.40 | 45° | Lignite bricks (stacked) | 1.30 |
| Coal coke | 0.50 | 40° | | |
| Lignite | 0.70 | 35° | — Construction Materials | |
| Lignite bricks (stacked) | 0.80 | 30° | Cement Bagged | 1.00 |
| Other forms of coal | 0.85 | 30° | In barrels | 0.90 |
| Pulverized coal | 0.70 | 25° | Gypsum (bagged) | 0.83 |
| Residual washing coal | 1.20 | 0° | Kaolin (bagged) | 0.77 |
| Rough coal (moist) | 1.00 | 45° | Sand (in boxes) | 0.60 |
| Sawdust (loose) | 0.15 | 45° | | |
| Sawdust (settled) | 0.25 | 45° | — Wood and Wood Products | |
| Spill wood | 0.20 | 45° | Cork | 0.24 |
| Split firewood | 0.40 | 45° | Paper (in rolls) | 0.40 |
| — Construction Materials | | | Paper (parcels) | 0.80 |
| Blastfurnace Slag | | | Paper paste (compressed bales) | 0.60 |
| Granular | 1.10 | 25° | Plywood | 0.65 |
| Crushed | 1.50 | 40° | Rubber (in bales, bags or boxes) | 0.50 |
| Brick rubble or powder | 1.30 | 35° | Rubber (in plates) | 0.60 |
| Cement clinker | 1.50 | 30° | Timber ties | 0.77 |
| Cement powder | 1.20 | 25° | Wood Soft | 0.70 |
| Coke ash | 0.70 | 25° | Hard | 1.00 |
| Crushed stone | 1.80 | 40° | — Food Products | |
| Granite (worked) | 1.30 | 35° | | |
| Gravel Dry | 1.60 | 40° | Bananas (boxed) | 0.26 |
| Saturated | 2.00 | 40° | Barley (bagged) | 0.60 |
| Gypsum and Plaster | 1.25 | 25° | Beverages (in barrels) | 0.60 |
| Kaolin | 0.95 | 35° | Bones (bagged) | 0.60 |

**TABLE 3.4.2.3.1.1. (Continued)**

| BULK MATERIALS | $\gamma$ (t/m$^3$) | $\phi$ (°) |
|---|---|---|
| A) SOLID BULK MATERIALS | | |
| Lime powder | 1.00 | 25° |
| Limestone | 1.70 | 35° |
| Lump lime | 1.00 | 45° |
| Marble (worked) | 1.30 | 35° |
| Pumice Sand | 0.70 | 35° |
| Sand — Dry | 1.70 | 30° |
| Sand — Saturated | 2.00 | 30° |
| — Waste Products | | |
| Compressed manure | 1.80 | 45° |
| Demolition waste | 1.30 | 35° |
| Heavy scrap | 1.60 | 35° |
| Light scrap | 1.20 | 30° |
| Loose manure | 1.20 | 45° |
| Urban debris | 0.60 | — |
| — Food Products | | |
| Bones | 0.40 | — |
| Cereal or soy flour | 0.50 | 45° |
| Cereals — Rice | 0.60 | 25° |
| Cereals — Oats | 0.45 | 30° |
| Cereals — Barley | 0.65 | 25° |
| Cereals — Rye | 0.80 | 35° |
| Cereals — Corn | 0.75 | 25° |
| Cereals — Millet | 0.70 | 25° |
| Cereals — Wheat | 0.75 | 25° |
| Common salt | 0.90 | 45° |
| Crushed malt | 0.40 | 45° |
| Dried sugar beets | 0.30 | 40° |
| Feed | 0.50 | 45° |
| Fish meal | 0.80 | 45° |
| Fodder | 0.17 | — |
| Frozen meat | 0.35 | — |
| Fruits and Vegetables | 0.75 | 30° |
| Ice | 0.90 | 30° |
| Legumes | 0.80 | 30° |
| Rape | 0.70 | 25° |
| Semolina | 0.55 | 30° |
| Soybeans | 0.85 | 60° |
| Sugar | 0.75 | 35° |
| Sunflower seeds | 0.55 | — |
| Tubers | 0.75 | 30° |
| — Plants | | |
| Flax | 0.60 | 25° |
| B) LIQUID BULK MATERIALS | | |
| — Petroleum products | | |
| Crude oil | 0.80 | — |
| Fuel oil | 0.80 | — |
| Gas oil | 0.80 | — |
| Gasoline | 0.75 | — |

| STACKED MATERIALS | $\gamma$ (t/m$^3$) |
|---|---|
| Butter (in barrels or boxed) | 0.60 |
| Canned meat (boxed) | 0.60 |
| Cheese (boxed) | 0.70 |
| Citrus fruits (boxed) | 0.40 |
| Coconuts (bagged) | 0.53 |
| Coconuts (boxed) | 0.40 |
| Coffee (bagged) | 0.55 |
| Condensed milk (boxed) | 0.50 |
| Condensed milk (in barrels) | 0.60 |
| Corn (bagged) | 0.65 |
| Dehydrated milk (bagged) | 0.53 |
| Dehydrated milk (boxed) | 0.50 |
| Flour (bagged) | 0.85 |
| Flour (in barrels) | 0.66 |
| Fresh or frozen fish (boxed) | 0.50 |
| Frozen meat (bagged) | 0.44 |
| Frozen meat (boxed) | 0.48 |
| Grapes (boxed) | 0.25 |
| Oats (bagged) | 0.43 |
| Rice (bagged) | 0.70 |
| Rice (in barrels) | 0.53 |
| Rye (bagged) | 0.63 |
| Salt (bagged) | 0.90 |
| Salt (boxed) | 0.70 |
| Soybeans (bagged) | 0.72 |
| Sugar (bagged) | 0.80 |
| Sunflower seeds (bagged) | 0.48 |
| Sunflower seeds (boxed) | 0.50 |
| Tapioca (bagged) | 0.65 |
| Tea (in bags) | 0.35 |
| Tubers (bagged) | 0.60 |
| Tubers (boxed) | 0.40 |
| Vegetables (bagged) | 0.50 |
| Vegetables (boxed) | 0.60 |
| Wheat (bagged) | 0.65 |
| — Animal and vegetable products | |
| Cotton (baled) | 0.37 |
| Dried skins (baled) | 0.20 |
| Dried skins (compressed bales) | 0.24 |
| Esparto grass (baled) | 0.25 |
| Moist skins (baled) | 0.55 |
| Wool (compressed bales) | 0.60 |
| — Petroleum products (in barrels) | 0.50 |
| — Oils | |
| Fish (in barrels) | 0.60 |
| Latex (in barrels) | 0.70 |
| Molasses (in barrels) | 0.55 |
| Vegetable (in barrels) | 0.55 |
| — Containers | 0.50-0.70 |

TABLE 3.4.2.3.1.1. **(Continued)**

| BULK MATERIALS | $\gamma$ (t/m$^3$) | $\phi$ (°) | STACKED MATERIALS | $\gamma$ (t/m$^3$) |
|---|---|---|---|---|
| B) LIQUID BULK MATERIALS | | | — Vehicles | |
| Liquified gases (natural gas. methane. butane, etc) | (*) | — | Motor vehicles (as scrap iron in cages) | 1.00 |
| | | | Motor vehicles (empty) | 0.25 |
| — Chemical Products | | | | |
| Acetone | 0.80 | — | | |
| Aniline | 1.00 | — | | |
| Benzine | 0.70 | — | | |
| Carbon sulphide | 1.30 | — | | |
| Chlorydric acid Up to 40% | 1.20 | — | | |
| Ethylic alcohol | 0.80 | — | | |
| Nitric acid up to 40% | 1.25 | — | | |
| Sulphuric acid up to 50% | 1.40 | — | | |
| — Oils | | | | |
| Castor oil | 0.97 | — | | |
| Creosote | 1.10 | — | | |
| Fish | 0.90 | — | | |
| Latex | 1.00 | — | | |
| Linseed | 0.95 | — | | |
| Mineral | 0.93 | — | | |
| Molasses | 1.25 | — | | |
| Vegetable | 0.92 | — | | |
| — Wines and Beverages | | | | |
| Beer | 1.03 | — | | |
| Milk | 1.03 | — | | |
| Water Fresh | 1.00 | — | | |
| Water Salt | 1.03 | — | | |
| Wine | 1.00 | — | | |

NOTES :

(*) When determining loads transmitted by the storage of liquid gasses, the specific weight and height of storage shall not be significant parameters.
The significant parameter shall be the pressure used by the storage installation to maintain the gas in a liquid state.

(h) humid.

(d) dry.

In installations with four or more levels, upon which the storage overloads act, the following additional reductions in the total overload shall be considered in the calculation of all the resistant elements that receive the load of various levels :

— For 1, 2 or 3 levels : 0%
— For 4 levels : 10%
— For 5 levels : 20%
— For 6 or more levels : 30%

Only in those cases where specific cargo handling installations, that limit the distribution and form of the staged or stored materials, are included in the project (e.g. cranes, loading bridges, mobile stackers, etc) , shall non-uniform distributions of distributed loads (e.g. zones unusable for cargo storage or distributions of trapezoidal, triangular, pyramidal or conical section) be considered in function of the designed handling installation. These distributions shall act on zones specifically assigned in the project.

| TABLE 3.4.2.3.1.2 COMMON MAXIMUM STORAGE AND STACKED HEIGHTS OF CARGOS IN PORT AREAS | | | |
|---|---|---|---|
| USE | NATURE OF CARGOS | $H_a$ (in m) | |
| | | In operation Area | In Storage Area |
| COMMERCIAL | - Soil bulk cargos without special installation | | |
| | - Ordinary or pulverulent | 2.50 | 5.00 |
| | - Heavy (ores) | 5.00 | 15.00 |
| | - Liquid bulk cargos without special installation (in barrels) | 2.00 | 5.00 |
| | - General cargos on pallets | | |
| | - In zones > 30 m wide | 2.00 | 5.00 |
| | - In zones ≤ 30 m wide | 2.00 | 3.00 |
| | - General cargo | | |
| | - Ores (bagged or boxed) | 3.00 | 7.00 |
| | - Manufactured products : Metallurgic, iron and steel products (in bars, coiled, in plates, etc) prefabricated concrete, large pieces, etc | 2.00 | 3.00 |
| | - Chemical products (bagged or boxed) | 2.00 | 5.00 |
| | - Solid Fuels | 3.00 | 7.00 |
| | - Construction materials (packaged) | 3.00 | 7.00 |
| | - Wood and wood products | 3.00 | 7.00 |
| | - Food products (boxed or bagged) | 2.00 | 4.00 |
| | - Animal or vegetable products | 2.00 | 5.00 |
| | - Containers (according to handling system) | | |
| | - In zones > 30 m wide | 5.00 | 5.00 to 12.00 |
| | - In zones ≤ 30 m wide | 5.00 | 5.00 to 7.50 |
| | - Vehicles | 1.50 | 1.50 |
| INDUSTRIAL (ship-yard repair) | - Metallurgic, iron and steel products | 3.00 | 3.00 |
| FISHING | - Fish (boxed) | 2.00 | 2.50 |
| MARINAS | - General Cargos | 1.00 | 2.00 |

In the absence of detailed information regarding the staged or stored materials or cargo, the nature of storage, and the type and distribution of the handling and storage installation and foreseeing possible changes in the use conditions and criteria during the design life of the project, the stage and storage overloads given in table 3.4.2.3.1.3, shall be adopted as minimums for open yards in function of the generic use assigned to the project.
This load shall be considered uniformly distributed over the surfaces that produce the most unfavorable effects on the resistant structure.

The calculation of the resistant structure with the minimum load obviously shall not eliminate the need to check the structure with the specific storage overloads that arise in the project's conception.

b) CONCENTRATED LOADS

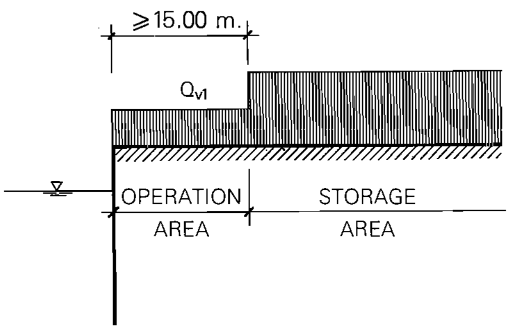
The stage and storage overloads, as concentrated loads, shall be equal to the largest loads transmitted to the resistant structure by the distinct support elements and systems of cargo, or of auxiliary transport installations (e.g. the containers). These elements and systems are used to isolate the cargo from the floor and aid in its handling.

The largest concentrated loads due to cargo stacked and stored in open yards in port zones shall generally be produced by :

— Manufactured products (metal, iron or steel products, prefabricated concrete products and large parts) supported on sleepers.
  Contact pressures up to 250 t/$m^2$.

— Containers, isolated or situated in rows or blocks, supported on four plates of approximately 0.15 x 0.15 $m^2$.
  Contact pressure up to 800 t/$m^2$ in an area 0.30 x 0.30 $m^2$, for containers stacked 5 high.

**TABLE 3.4.2.3.1.3 MINIMUM UNIFORMLY DISTRIBUTED STAGE AND STORAGE OVERLOADS**




| USE | $Q_{v1}$ (in t/$m^2$) | |
|---|---|---|
| Areas accessible only to pedestrians | | |
| - General | 0.50 | |
| - Operation or work zones in industrial installations in open seas (e.g. petroleum extraction platforms). | 0.85 | |
| - Housing or service zones | According to NBE-AE-88 | |
| Areas not only accessible to pedestrians | Operation Area | Storage Area |
| - Commercial Use | | |
| - Normal bulk cargos, conventional general cargos and containers. | 3.00 | 6.00 |
| - Heavy bulk cargos and heavy general cargos. | 5.00 | 10.00 |
| - Industrial Use (shipyard and ship repair) | 5.00 | 10.00 |
| - Military Use | 3.50 | 5.00 |
| - Fishing Use | 1.50 | 1.50 |
| - Marinas | 1.50 | 1.50 |

— Trailers (released from trucks), with forward supports upon wheels or plates. Contact pressures up to 200 t/m$^2$ in an area approximately 0.25 x 0.15 m$^2$, or up to 4000 t/m$^2$ in 0.01 x 0.09 m$^2$.

Lacking other specific data established by the project or by the port use criteria, and foreseeing possible changes in the use conditions, the concentrated loads given in table 3.4.2.3.1.4. shall be considered as minimums, in function of the generic uses assigned to the project.

In the calculation, a unique concentrated load shall be situated where it produces the most unfavorable effects on the resistant structure. This load shall be the largest possible. Its effects shall never be superimposed on the effects produced by the distributed load.

The calculation of the resistant structure with the minimum concentrated loads given in the table shall not eliminate the need to check the structure with the specific concentrated overloads that arise in the project's conception.

The stacking and storage overloads can be increased, and their distribution altered in magnitude and direction, due to environmental variables (wind, snow, temperature) and by the dynamic effects produced by impact loads primarily associated with :

— Blows due to falls, or positioning of cargos on surfaces from the handling installations.
— Loading and unloading in installations that are exclusively used for storage (e.g. in silos)

These effects shall be taken into consideration for the determination of the characteristics values of the stage and storage overloads assuming the most unfavorable disposition for the element under consideration.

In general, the influence of the environmental actions in the stage and storage overloads can be neglected, except in those storage installations or structures specially sensitive to horizontal forces.

TABLE 3.4.2.3.1.4 MINIMAL CONCENTRATED STAGE AND STORAGE OVERLOADS IN OPEN YARDS

| USES | LOADS (in t) | | SURFACE AREA (in m$^2$) |
|---|---|---|---|
| | In operation area | In storage area | |
| - Commercial Use | | | |
| - Conventional general loads and containers | 40 | 70 | 0.30 × 0.30 |
| - Heavy general cargo | 90 | 120 | 0.80 × 0.80 |
| - Industrial Use (shipyards and ship repair factories) | 90 | 120 | 0.80 × 0.80 |

**NOTE :**

For the remaining uses not included in the table (military, fishing or marinas), it shall not be obligatory to consider concentrated stage and storage overloads.

In any case, a more detailed analysis of these effects can be carried out according to sections 3.4.2.2. Soil Loads ($a_{25}$.- Silo Effect), 3.4.2.4. Environmental Loads, ROM 0.4.- Consideration of Environmental Variables/II : Atmospheric and seismic conditions, and the paragraph referring to dynamic effects in this section.

When minimum overloads are designed for, it shall be considered that they cover the existence of dynamic effects and the influence of environmental actions.

The lateral pressures that solid and liquid bulk exert on containment installations (e.g. deposits, silos) shall also be considered as stage and storage overloads. The evaluation of these pressures can be carried out in the same way as for earth pressures loads (See section 3.4.2.2.), or for hydraulic loads (See section 3.4.2.1.), by simply substituting the characteristic parameters of the soil or of the water for those of the bulk cargo.
Lacking other experimental data, the characteristic parameters of bulk cargos ($\gamma,\phi$) can be obtained from table 3.4.2.3.1.1.

■ DYNAMIC EFFECTS

The possibility of impact loads associated with blows, falls or positioning of cargos on the surface from the handling equipment, shall be taken into account in the calculations in terms of an additional point load equal to the maximum possible weight manipulated. This additional load shall never surpass 10 t.

As for the increase of the loads and load combinations, the impact loads shall be considered as accidental loads. Impact loads shall be included in the calculation only for structural elements whose failure could affect over-all safety or normal service of the analyzed structure, and the possibility of progressive collapse exists. It shall be admitted, therefore, that as a consequence of this load, localized damage occur, particularly in auxiliary installations, pavements and traffic tracks.

■ ACTION DIFFERENTIATION

a) BY PROJECT PHASES

The fixed stage and storage overloads indicated for the conditions in this section shall only be applicable to the Service Phase (S), in the two hypotheses of Normal Operating Conditions (S1) and of Extreme Conditions (S2), with the exception of including the additional impact load in exceptional conditions (S3).
Also, and as an additional exceptional condition, all the structural elements shall be designed to resist overloads caused by tests and trials required by the code in effect, that are applicable to each installation. In particular, for installations designated for liquid bulk cargos with a specific weight of less than 1 t/m$^3$, the overloads produced by the flow of water during their hydraulic tests must be considered. In terms of load combination and load increase, this action shall be considered accidental.

For the Construction Phase (C), the project engineer shall adopt storage overloads compatible with the storage or material accumulation necessary during the construction process. In areas where the accumulation of materials is possible, overloads less than 1 t/m$^3$ shall not be considered.

b) BY STRUCTURAL TYPES

Related to the actions of use overloads, structures shall be classified into the following structural types :

— *TYPE 1* : The loads act directly upon the structural elements
— *TYPE 2* : The loads transmit their action to the structural elements through a load distribution layer.
— *TYPE 3* : The loads act upon a fill situated in back of the structure, which is indirectly loaded through an earth pressure increase.

The stage and storage overloads given in this section shall be completely applicable to Type 1 structures.

For Type 3 structures and Type 2 structures with a load distribution layer greater than 1.50 m, the concentrated load's effects, or the existence of dynamic effects shall not be considered.

For Type 2 structures with a load distribution layer less than or equal to 1.50 m, the concentrated loads and the additional load equivalent to the dynamic effect shall be considered to be distributed through the distribution layer in two orthogonal directions, with slope 2 (vertical)/1 (horizontal) or 1/1 according to what is most unfavorable.

### 3.4.2.3.2. CARGO HANDLING EQUIPMENT AND INSTALLATION OVERLOADS ($Q_{V2k}$)

■ DEFINITION

Cargo Handling Equipment and Installation Overloads are defined as loads of variable nature transmitted to the resistant structure by the cargo, material or supply handling systems and equipment included in the following list :

— Discontinuous Handling Systems

- Fixed equipment.
- Equipment on rails (e.g. portal crane).
- Rubber tired equipment (e.g. container fork lift trucks).
- Treaded equipment (e.g. mobile cranes)

— Continuous Handling Equipment

- Pipelines.
- Conveyors belts.
- Gravity inclined planes.

The actions produced by fixed equipment and by continuous handling systems are included as cargo handling installation overloads, and not as dead loads, due to the fact that the magnitude and direction of the loads transmitted by this equipment are not constant.

■ DETERMINATION

Generally, the cargo handling installation overloads shall be considered as a function of the specific character of the projected installation and the zone where it is located.
The Project Engineer shall ascertain, in agreement with preestablished port planning criteria and the Client's or Government Authority's specifications, the characteristics of the cargo handling equipment that operates in the zone, its location, and the way in which it acts upon the resistant structure.

In the absence of specific criteria, the Project Engineer may include in the calculation the loads transmitted by the handling equipment that is considered necessary in function of the generic use of the work area, taking into consideration :

— The cargo, materials and supplies being handled.

— The most appropriate handling methods.

— The environmental characteristics of the area, most importantly the wind.

In this case, it shall be considered that the overloads caused by the handling installations shall only be significant in operation areas and in tracks designated and designed specifically as maneuvering tracks, and therefore it shall only be necessary to include them in the calculations of these zones.

The cargo handling equipment that affects the designed works will be listed in the project, including in the calculations the equipment's principle characteristics and the loads transmitted by each type of equipment in each work condition. The position of the

equipment and the load that produce the most unfavorable conditions shall also be indicated.
The principle characteristics of the equipment shall be referred to as :

— Generic definition of the handling system (type, maximum load, maximum reach, etc)

— Geometric definition of the performance band.

The cargo handling installation overloads shall be treated as mobile load trains made up of surface, line or point loads; acting, within the performance bands defined in the project, on part or all of the resistant structure, in the way that produces the most unfavorable effects in the analyzed elements. Its distribution shall be limited by its compatibility with other use overloads.
The characteristic values of the loads transmitted to the resistant structure shall be directly furnished by the equipment manufacturers and suppliers, and will have been obtained through the application of the loads and load combinations, required by the normative in effect for the calculation of the cargo handling systems, but without including the global safety factor. The following norms shall be applied, particularly for lifting systems (cranes) :

— UNE 58-102-74 : Heavy Lifting Apparatus.

— Norms for the Calculation of Electric Portal Cranes for Port Services. (Order of the Ministry of Public Works on August 11, 1964)

— Norms of the F.E.M. (European Manutention Federation) for the Design of Lifting Equipment.

In any case, in the calculation and verification of the resistant structure in each one of the project phases and work hypotheses the Engineer shall differentiate the overloads transmitted by the handling installations according to the detailed load hypotheses, for the position and magnitude of the handled load that produces the maximum actions :

— *In Normal Operating Conditions*

- Equipment in Service without Wind : Self Weight + Service load.

- Equipment in Service with Maximum Operating Wind : Self Weight + Service load + Maximum Operating Wind.
  Lacking other defined operating criteria, the mean velocity of 22 m/s (80 Km/h) in the interval (gust) of 3 s. ($V_{3s}$ = 22 m/s) shall be adopted as the maximum operating wind for lifting equipment, acting in the direction that produces the most unfavorable effects for a determined position of the service load.

— *In Extreme Conditions*

- In fixed equipment or in equipment with restricted movement (on rails or on fixed tracks) : It will be considered that once the environmental conditions surpass the maximum operating limits, the equipment shall cease operating, adopting a retracted position with the least exposed surface, and be secured against sliding by means of anchoring devices (e.g. chains, hooks or bolts).
  In this case, the load hypothesis shall be : Self Weight + Wind in Extreme Conditions. The calculation wind velocity in extreme conditions shall be the $V_{3s}$ that corresponds to the return period associated with the greatest admissible risk. In general, for cargo handling equipment, return periods (T) of 100 years shall be used, which is equivalent to the maximum admissible risk of approximately 20% for equipment design lives of 25 years (See table 2.2.1.1. Minimum design lives for works or installations of definitive character, and table 3.2.3.1.2. Maximum admissible risks to determine characteristic values of variable loads in the service phase and extreme conditions based on statistical data).

- For equipment without restricted movement (e.g. rubber tired equipment), it shall be considered that when the limit environmental conditions are reached the equipment is stored in protected or auxiliary zones, thus it is unnecesssary to consider in the calculation the work hypothesis : in extreme conditions.

— *In Exceptional Conditions*

The loads transmitted by the following work hypotheses shall be considered accidental actions in the calculations :

- Equipment in Service under the effect of a collision (with the ship, with other equipment, etc) : Self Weight + Service Load + Collision

- Equipment out of Service in Exceptional Environmental Conditions : Self Weight + Wind in Exceptional Conditions (Return Period T higher than that corresponding to extreme conditions. Lacking other data, T = 1000 years)
  This hypothesis shall only be considered for fixed or restricted mobility equipment.

For continuous cargo handling systems, in addition to the described actions, loads produced by direction changes in plan or elevation, by pressures or temperatures of the transported fluid with respect to the exterior conditions, or by any internal action of the system itself shall be included.
All these actions shall be superimposed on each one of the compatible load hypotheses.

The project engineer shall include in the project the source of the loads transmitted by the cargo handling installations used in the calculations, indicating whether or not these loads include the quantification of dynamic effects.

Values indicative of the order of magnitude of loads transmitted by cargo handling equipment in port zones are given in tables 3.4.2.3.2.1 and 3.4.2.3.2.2 for equipment upon tracks, and 3.4.2.3.2.3 for equipment with rubber tires or treads.
Likewise, for continuous cargo handling equipment, a load of 0.5 t/m$^2$ for each level of pipelines, conveyors, belts, etc. can be adopted, with a minimum of 2 t/m$^2$.

— *USE OVERLOAD COMPATIBILITY*

In order to simplify the many possible hypotheses of use overload combinations it shall be admitted that only the effects produced upon the resistant structure due to :

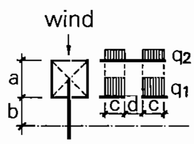
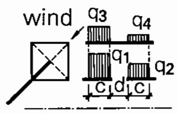
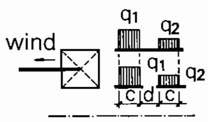
- Cargo handling equipment or installations upon rails (e.g. portal cranes or container cranes).
- Movement-restricted rubber-tired cargo handling equipment or installations (e.g. storage portal cranes).
- Continuous handling systems (e.g. pipelines)

are superimposed on each other and with the effects produced by the stage and storage overloads.

On the contrary, the effects produced by unrestricted (discontinuous) mobility systems (e.g. fork lift trucks) are not superimposed with effects due to other use overloads acting simultaneously upon the same area.
Therefore, for stage and storage overloads ($Q_{V1}$), combined with cargo handling equipment and installation overloads ($Q_{V2}$), it shall only be necessary to take into account the cases shown in table 3.4.2.3.2.4.

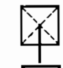
For the design and calculation of the resistant structure, the Project Engineer shall choose the overload combination that produces the most unfavorable effects in the analyzed element.
Generally, the combinations I and III in table 3.4.2.3.2.4. shall be critical for the majority of common structures in port zones; even though they must be alternatively checked with combination II. This combination shall be especially critical for structures with short span, slabs, or other elements that may be sensitive to local effects.

TABLE 3.4.2.3.2.1 LOADS TRANSMITTED BY COMMON CARGO HANDLING EQUIPMENT IN PORT ZONES. RAIL EQUIPMENT: PORTAL CRANES.

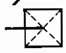
| EQUIPMENT | GEOMETRIC DEFINITION (in m.) | WORK HYPOTHESIS | | LOAD TRAIN (in t/m) | | |
|---|---|---|---|---|---|---|
| | | | |   |   |   |
| PORTAL CRANE OF 6 t and 25 m reach. (self weight ≃86 t) | a=6<br>b=2a3<br>c=1<br>d=5 | In normal operating conditions | Without wind | $q_1 = 36$<br>$q_2 = 10$ | $q_1 = 40$ $q_3 = 22$<br>$q_2 = 22$ $q_4 = 8$ | $q_1 = 36$<br>$q_2 = 10$ |
| | | | With wind | $q_1 = 40$ $H_{VT} = 2.5$<br>$q_2 = 6$ $H_{VL} = 0$ | $q_1=44$ $q_3=22$ $H_{VT}=1.8$<br>$q_2=22$ $q_4=4$ $H_{VL}=1.8$ | $q_1 = 40$ $H_{VT} = 0$<br>$q_2 = 6$ $H_{VL} = 2.5$ |
| | | In extreme conditions | | $q_1 = 37$ $H_{VT} = 8$<br>$q_2 = 6$ $H_{VL} = 0$ | – | $q_1 = 37$ $H_{VT} = 0$<br>$q_2 = 6$ $H_{VL} = 8$ |
| PORTAL CRANE OF 12 t and 25 m reach. (self weight ≃200 t) | a=10<br>b=2a3<br>c=3<br>d=6 | In normal operating conditions | Without wind | $q_1 = 25$<br>$q_2 = 10$ | $q_1 = 28$ $q_3 = 17$<br>$q_2 = 17$ $q_4 = 8$ | $q_1 = 25$<br>$q_2 = 10$ |
| | | | With wind | $q_1 = 27$ $H_{VT} = 1$<br>$q_2 = 8$ $H_{VL} = 0$ | $q_1=30$ $q_3=17$ $H_{VT}=0.75$<br>$q_2=17$ $q_4=6$ $H_{VL}=0.75$ | $q_1 = 27$ $H_{VT} = 0$<br>$q_2 = 8$ $H_{VL} = 1$ |
| | | In extreme conditions | | $q_1 = 24$ $H_{VT} = 3$<br>$q_2 = 9$ $H_{VL} = 0$ | – | $q_1 = 24$ $H_{VT} = 0$<br>$q_2 = 9$ $H_{VL} = 3$ |
| PORTAL CRANE OF 16 t and 35 m reach. (self weight ≃264 t) | a=10<br>b=2a3<br>c=4<br>d=6 | In normal operating conditions | Without wind | $q_1 = 25$<br>$q_2 = 10$ | $q_1 = 28$ $q_3 = 18$<br>$q_2 = 18$ $q_4 = 6$ | $q_1 = 25$<br>$q_2 = 10$ |
| | | | With wind | $q_1 = 27$ $H_{VT} = 0.7$<br>$q_2 = 8$ $H_{VL} = 0$ | $q_1=32$ $q_3=18$ $H_{VT}=0.5$<br>$q_2=18$ $q_4=2$ $H_{VL}=0.5$ | $q_1 = 27$ $H_{VT} = 0$<br>$q_2 = 8$ $H_{VL} = 0.7$ |
| | | In extreme conditions | | $q_1 = 24$ $H_{VT} = 2.5$<br>$q_2 = 9$ $H_{VL} = 0$ | – | $q_1 = 24$ $H_{VT} = 0$<br>$q_2 = 9$ $H_{VL} = 2.5$ |
| PORTAL CRANE OF 25 t and 30 m reach. (self weight ≃312 t) | a=10<br>b=2.5 a 3<br>c=4<br>d=5 | In normal operating conditions | Without wind | $q_1 = 32$<br>$q_2 = 10$ | $q_1 = 38$ $q_3 = 21$<br>$q_2 = 21$ $q_4 = 5$ | $q_1 = 32$<br>$q_2 = 10$ |
| | | | With wind | $q_1 = 35$ $H_{VT} = 0.75$<br>$q_2 = 7$ $H_{VL} = 0$ | $q_1=40$ $q_3=21$ $H_{VT}=0.55$<br>$q_2=21$ $q_4=3$ $H_{VL}=0.55$ | $q_1 = 35$ $H_{VT} = 0$<br>$q_2 = 7$ $H_{VL} = 0.75$ |
| | | In extreme conditions | | $q_1 = 29$ $H_{VT} = 3$<br>$q_2 = 10$ $H_{VL} = 0$ | – | $q_1 = 29$ $H_{VT} = 0$<br>$q_2 = 10$ $H_{VL} = 3$ |
| PORTAL CRANE OF 35 t and 30 m reach. (self weight ≃410 t) | a=10<br>b=2.5 a 3<br>c=6<br>d=3 | In normal operating conditions | Without wind | $q_1 = 27$<br>$q_2 = 10$ | $q_1 = 34$ $q_3 = 19$<br>$q_2 = 19$ $q_4 = 2$ | $q_1 = 27$<br>$q_2 = 10$ |
| | | | With wind | $q_1 = 30$ $H_{VT} = 0.6$<br>$q_2 = 7$ $H_{VL} = 0$ | $q_1=36$ $q_3=19$ $H_{VT}=0.45$<br>$q_2=19$ $q_4=0$ $H_{VL}=0.45$ | $q_1 = 30$ $H_{VT} = 0$<br>$q_2 = 7$ $H_{VL} = 0.6$ |
| | | In extreme conditions | | $q_1 = 25$ $H_{VT} = 2.2$<br>$q_2 = 9$ $H_{VL} = 0$ | – | $q_1 = 25$ $H_{VT} = 0$<br>$q_2 = 9$ $H_{VL} = 2.2$ |
| PORTAL CRANE OF 50 t and 35 m reach. (self weight ≃622 t) | a=10<br>b=3<br>c=8<br>d=1.5 | In normal operating conditions | Without wind | $q_1 = 32$<br>$q_2 = 10$ | $q_1 = 38$ $q_3 = 20$<br>$q_2 = 20$ $q_4 = 6$ | $q_1 = 32$<br>$q_2 = 10$ |
| | | | With wind | $q_1 = 34$ $H_{VT} = 0.5$<br>$q_2 = 8$ $H_{VL} = 0$ | $q_1=40$ $q_3=20$ $H_{VT}=0.35$<br>$q_2=20$ $q_4=4$ $H_{VL}=0.35$ | $q_1 = 34$ $H_{VT} = 0$<br>$q_2 = 8$ $H_{VL} = 0.5$ |
| | | In extreme conditions | | $q_1 = 27$ $H_{VT} = 2$<br>$q_2 = 12$ $H_{VL} = 0$ | – | $q_1 = 27$ $H_{VT} = 0$<br>$q_2 = 12$ $H_{VL} = 2$ |

## TABLE 3.4.2.3.2.1 (Continued)

**LEGEND :**

 Portal Crane with boom in centered lateral position, perpendicular to the rails.

 Portal crane with boom at an angle.

 Portal crane with boom in centered lateral position parallel to the rails.

$q_i$ : Vertical load per lineal meter corresponding to each foot of the crane

$H_{VT}$ : Horizontal wind load per linear meter, perpendicular to the rails, corresponding to each foot of the crane.

$H_{VL}$ : Horizontal wind load per linear meter, parallel to the rails, corresponding to each foot of the crane.

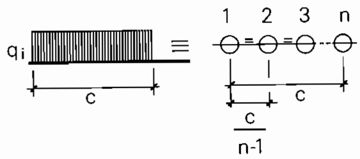
—·—·— Symbol that indicates the exterior limit of the operation zone (edge of the pier)

**COMMENTS :**

- The values given in this table do not include dynamic effects produced by inertial forces (acceleration and deceleration in elevation, translation, rotation, and reach change movements). These effects shall be evaluated according to the criteria in the "Dynamic Effects" section.
- The order of application of the equivalent load trains on each foot can be inverted, in order to take into account all the possible boom positions. For the same reason, they can be applied to any rail.
- For structural elements that are especially sensitive to concentrated loads, each linear load group corresponding to each foot of the crane can be brocken down into point loads, adopting the following number of wheels per foot, in function of the admissible load upon the rail.

| | |
|---|---|
| 6 t Portal Crane | 2 |
| 12 t Portal Crane | 4 |
| 16 t Portal Crane | 4 or 6 |
| 25 t Portal Crane | 6 |
| 35 t Portal Crane | 6 |
| 50 t Portal Crane | 8 |

It shall be considered that the separation between the wheels is constant, and that they are distributed along the entire length assigned to each linear load.

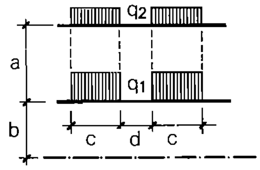


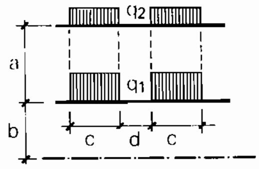

- $V_{3s}$ = 40 m/s has been taken as the wind velocity in extreme conditions. For wind velocity corresponding to the project's return period, corrections shall be admitted in the additional load values due to wind (total load in table upon each foot - uniform distribution of the crane's weight upon each foot) by means of the factor :

$$\frac{((V_{3s})_T)^2}{40^2}$$

given that $(V_{3s})_T$ is the 3 s gust velocity corresponding to the project's return period, expressed in m/s.
In normal operating conditions, the wind velocity's operating limit is 22 m/s.

TABLE 3.4.2.3.2.2. LOADS TRANSMITTED BY COMMON CARGO HANDLING EQUIPMENT IN PORT ZONES. RAIL EQUIPMENT : CONTAINER CRANES

| EQUIPMENT | Geometric Definition (in m) | WORK HYPOTHESIS | | LOAD TRAIN (in t/m)<br><br> |
|---|---|---|---|---|
| CONTAINER CRANE of 38 t load capacity (used for 40' containers). Seaward outreach 30 m. (Self weight ≈ 500 t) | a = 15<br>b = 3<br>c = 7.5<br>d = 9 | IN NORMAL OPERATING CONDITIONS | | |
| | | Maximum load with maximum outreach | Without wind | $q_1 = 30$<br>$q_2 = 5$ |
| | | | With transversal or longitudinal wind. Operating wind limit : $V_{3s} = 22$ m/s | $q_1 = 32 \pm 2$<br>$q_2 = 5 \pm 2$<br>$H_{VT}$ o $H_{VL} = 0.55$ |
| | | Maximum load with maximum backreach | Without wind | $q_1 = 15$<br>$q_2 = 20$ |
| | | | With transversal or longitudinal wind. Operating wind limit : $V_{3s} = 22$ m/s | $q_1 = 15 \pm 2$<br>$q_2 = 18 \pm 2$<br>$H_{VT}$ o $H_{VL} = 0.55$ |
| | | IN EXTREME CONDITIONS | | |
| | | Boom in Carriage and cabin in stowed position | With wind transversal to the rails $V_{3s} = 40$ m/s | $q_1 = 15 \pm 6$<br>$q_2 = 18 \pm 6$<br>$H_{VT} = 2$ |
| | | | With wind longitudinal $V_{3s} = 40$ m/s | $q_1 = 15 \pm 6$<br>$q_2 = 18 \pm 6$<br>$H_{VT} = 2$ |

| TABLE 3.4.2.3.2.2. (Continued) | | | | |
|---|---|---|---|---|
| EQUIPMENT | Geometric Definition (in m) | WORK HYPOTHESIS | | LOAD TRAIN (in t/m)<br><br> |
| CONTAINER CRANE of 53 t load capacity (used for 40' or 20 x 20 containers). Seaward outreach 35 m. (Self weight ≈ 800 t) | a = 18<br>b = 3<br>c = 10.5<br>d = 6 | IN NORMAL OPERATING CONDITIONS | | |
| | | Maximum load with maximum outreach | Without wind | $q_1 = 30$<br>$q_2 = 10$ |
| | | | With transversal or longitudinal wind. Operating wind limit : $V_{3s} = 22$ m/s | $q_1 = 30 \pm 2$<br>$q_2 = 10 \pm 2$<br>$H_{VT}$ o $H_{VL} = 0.50$ |
| | | Maximum load with maximum backreach | Without wind | $q_1 = 15$<br>$q_2 = 25$ |
| | | | With transversal or longitudinal wind. Operating wind limit : $V_{3s} = 22$ m/s | $q_1 = 15 \pm 2$<br>$q_2 = 25 \pm 2$<br>$H_{VT}$ o $H_{VL} = 0.50$ |
| | | IN EXTREME CONDITIONS | | |
| | | Boom in Carriage and cabin in stowed position | With wind transversal to the rails $V_{3s} = 40$ m/s | $q_1 = 15 \pm 6$<br>$q_2 = 22 \pm 6$<br>$H_{VT} = 2$ |
| | | | With wind longitudinal $V_{3s} = 40$ m/s | $q_1 = 15 \pm 6$<br>$q_2 = 22 \pm 6$<br>$H_{VT} = 2$ |

**LEGEND :**

$q_i$ : Vertical load per linear meter corresponding to each foot of the crane

$H_{VT}$ or $H_{VL}$ : Horizontal wind load (longitudinal or transversal) per linear meter, corresponding to each foot. Its distribution is coincident with the vertical linear load.
The direction in which the load acts shall be compatible with the wind direction considered and therefore with the vertical loads associated with that direction.

—·—·— : Symbol indicating the exterior limit of the operation zone (edge of the pier)

TABLE 3.4.2.3.2.2. (Continued)

COMMENTS :

— The values given in this table do not include dynamic effects produced by inertial forces (accelerations and decelerations in elevation and translation movements). These effects shall be evaluated according to the criteria in the "Dynamic Effects" section.

— Load Trains equivalent to container cranes shall be strictly applied in the given order with respect to the seaward limit of the operation zone, and not applied indiscriminately to each rail, due to the load distribution asymmetry between the landward and seaward rails that is produced in this type of cranes.

— For structures or structural elements especially sensitive to concentrated loads, each lineal load group that corresponds to each foot of the crane shall be brocken down into point loads, adopting 6 or 8 wheels for each foot in function of the maximum admissible load upon the rail. The separation between wheels shall be considered uniform and they should be distributed along the entire length assigned to the linear load.

— $V_{3s}$ = 40 m/s has been taken as the wind velocity in extreme conditions. For wind velocities corresponding to the project's return period, corrections shall be admitted in the values of the additional loads due to wind by the factor :

$$\frac{[(V_{3s})_T]^2}{40^2}$$

given that $(V_{3s})_T$ is the three second gust velocity corresponding to the project's return period in m/s.

**TABLE 3.4.2.3.2.3 LOADS TRANSMITTED BY COMMON PORT ZONE CARGO HANDLING EQUIPMENT. RUBBER TIRED OR TREADED EQUIPMENT**

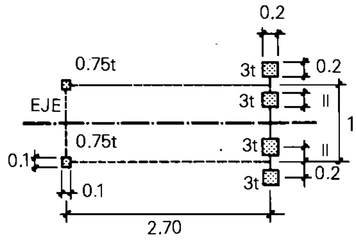
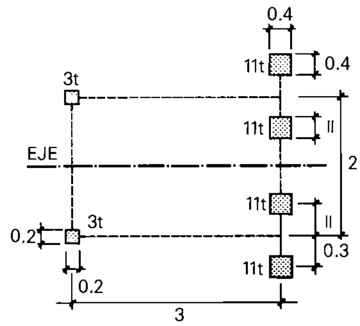
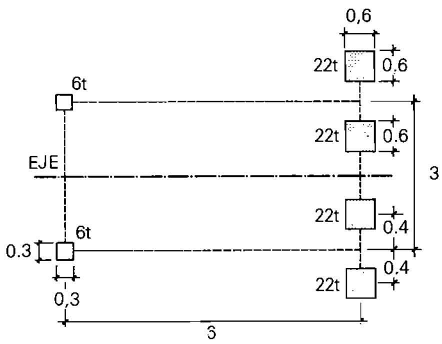
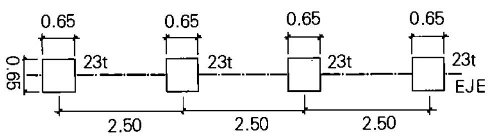
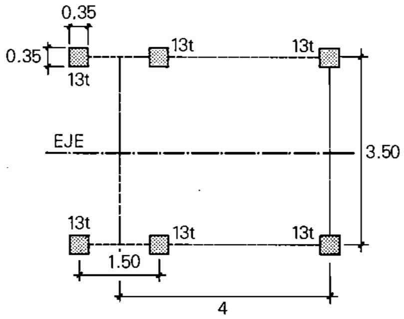
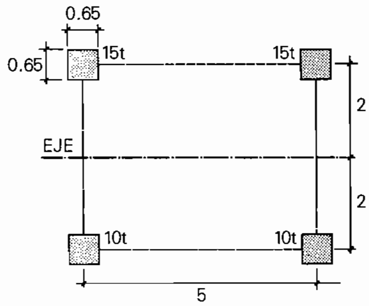
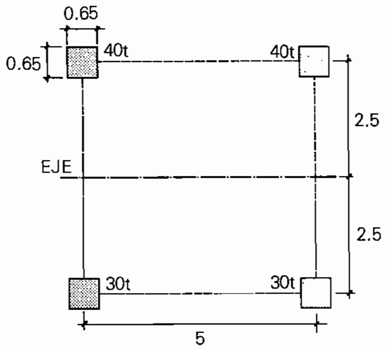
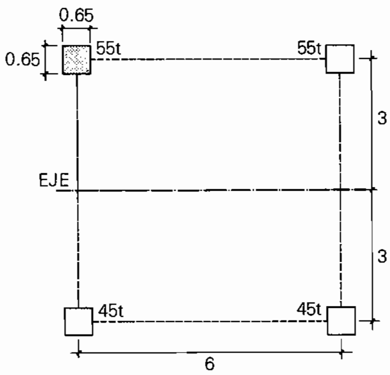
| LOAD TRAIN<br>- Length units in m.<br>- Loads in tons. | Equivalent uniform overload ($t/m^2$) | PHYSICAL EQUIVALENCE |
|---|---|---|
|  | 1.50 | Fork lift trucks with 5 t nominal load capacity<br><br>Self weight = 8t |
|  | 2.50 | Fork lift trucks with 20 t nominal load capacity<br><br>Self weight = 30t |
|  | 4 | Fork lift trucks with 40 t nominal load capacity<br><br>Self weight = 80t |
|  | 1.50 | Sideloader trucks with 40 t load capacity with outriggers extended.<br><br>Self weight = 50t |

**TABLE 3.4.2.3.2.3 (Continued)**

| LOAD TRAIN<br>- Length units in m.<br>- Loads in tons. | Equivalent uniform overload ($t/m^2$) | PHYSICAL EQUIVALENCE |
|---|---|---|
|  | 1.50 | Straddle carrier for containers 40' long and 40 t.<br><br>Self weight = 30t |
|  | 3.00 | Truck crane with 10 t lifting capacity (working short radius) and 12 m boom. Loads are for over side and over rear lifts, with outriggers extended.<br><br>Self weight = 40t |
|  | 6.00 | Truck crane with 30 t lifting capacity (working short radius) and 12 m boom. Loads are for over side and over rear lifts, with outriggers extended.<br><br>Self weight = 110t |
|  | 9.00 | Truck crane with 50 t lifting capacity (working short radius) and 12 m boom. Loads are for over side and over rear lifts, with outriggers extended.<br><br>Self weight = 150t |

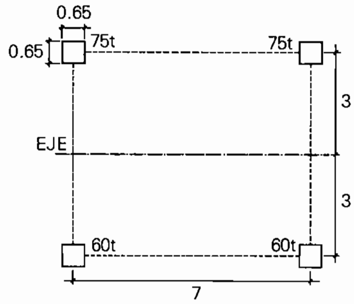
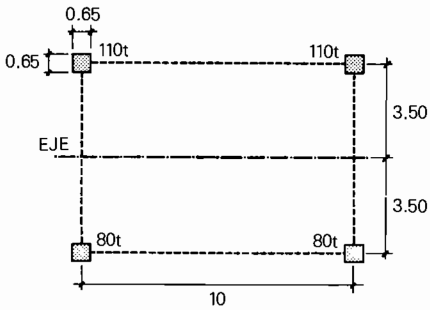
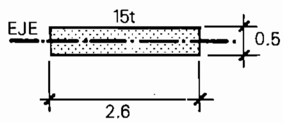
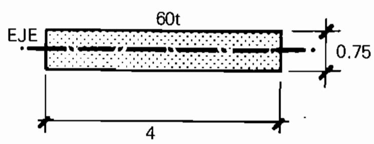
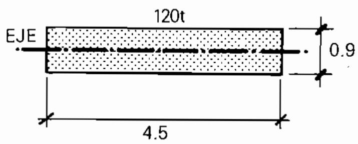
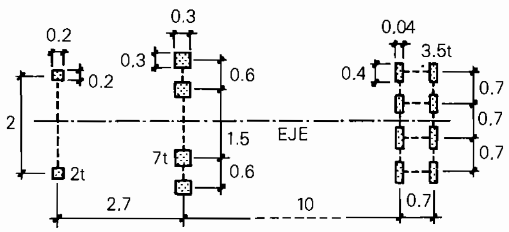
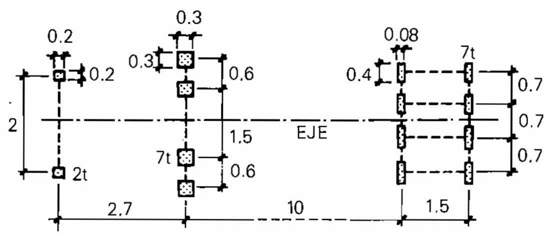
| TABLE 3.4.2.3.2.3 (Continued) | | |
|---|---|---|
| LOAD TRAIN TYPE<br>- Length units in m.<br>- Loads in tons. | Equivalent uniform overload ($t/m^2$) | PHYSICAL EQUIVALENCE |
|  | 12.00 | Truck crane with 70 t lifting capacity (working short radius) and 12 m boom. Loads are for over side and over rear lifts, with outriggers extended.<br><br>Self weight = 200t |
|  | 14,00 | Truck crane with 140 t lifting capacity (working short radius, 3.6 m) and 12 m boom. Loads are for over side and over rear lifts, with outriggers extended.<br><br>Self weight = 240t |
|  | – | Treaded crane with 6 t lifting capacity (working with short radius, 3 m), and 6 m. boom. Loads are for over side lifts.<br><br>Self weight = 12t |
|  | – | Treaded crane with 30 t lifting capacity (working with short radius, 3 m), and 10 m. boom. Loads are for over side lifts.<br><br>Self weight = 40t |

**TABLE 3.4.2.3.2.3 (Continued)**

| LOAD TRAIN<br>- Length units in m.<br>- Loads in tons. | Equivalent uniform overload ($t/m^2$) | PHYSICAL EQUIVALENCE |
|---|---|---|
|  | — | Treaded crane with 50 t lifting capacity (working with short radius, 3.6 m), and 12 m. boom. Loads are for over side lifts.<br><br>Self weight = 60t |
|  | 1.00 | Tractor and trailer with 40 t maximum load capacity |
|  | 1.50 | Tractor and trailer with 80 t maximum load capacity |

NOTES :

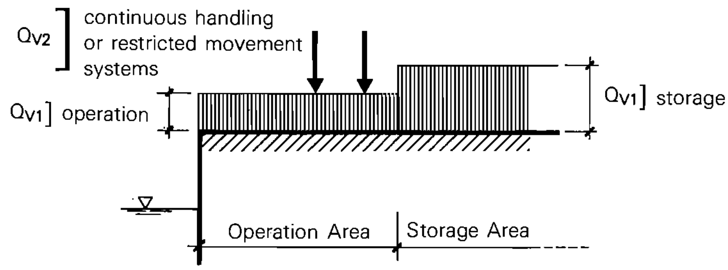
- The equivalent uniform overload is defined as the distributed load which is equivalent to the mobile load train, extended over an area approximately coinciding with the vehicle's equivalent area.
  Example :
  For a 20 t fork lift truck.
  Total loaded weight : 30 + 20 = 50 t.
  Equivalent overload : 2.5 t / m²

  $$\text{Equivalent area} : \frac{50\ t}{2.5\ t/m^2} = 20\ m^2\ (4.5 \times 4.5\ m^2)$$

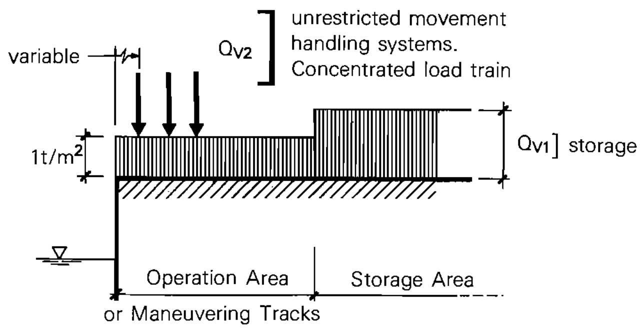
- The load trains reflected in these tables do not take into account factors such as the performance of environmental actions upon the machinery or load, sloped ground surfaces, operation speeds or other dynamic effects.
  In general, the environmental loads in normal operating conditions can be disregarded.

**TABLE 3.4.2.3.2.4 STAGE AND STORAGE OVERLOAD ($Q_{V1}$) COMPATIBLE WITH CARGO HANDLING EQUIPMENT AND INSTALLATION OVERLOADS ($Q_{V2}$)**

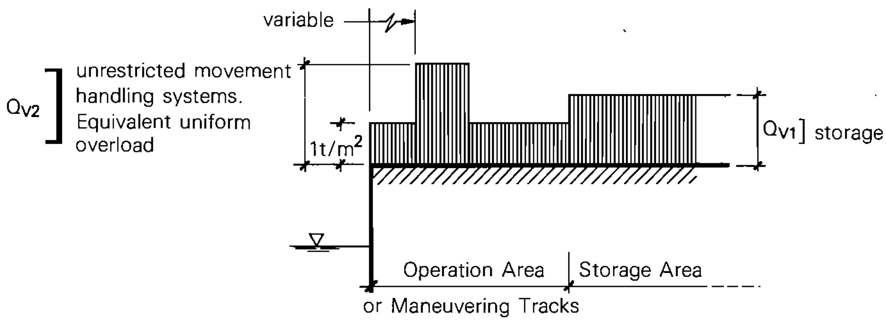
I




II




III




— MINIMUM LOAD TRAINS

Foreseeing possible changes during the design life of the project in the conditions of use or in the use criteria considered in the design, the resistant structure shall be calculated alternatively with the load trains, equivalent to the handling equipment overloads included in table 3.4.2.3.2.5. The load trains that could affect the work and that produce the most unfavorable effects shall be applied individually, according to the assigned generic use and the considered area.
The application criteria shall be the following :

These load trains shall be combined with the minimum stage and storage overloads established in table 3.4.2.3.1.3., according to the prior given criteria. (See table 3.4.2.3.2.4).

| USES | AREAS | | |
|---|---|---|---|
| | Operation Area | Storage Area | Maneuvering Tracks |
| COMMERCIAL | | | |
| - Normal bulk cargos and conventional general cargos | A1, A2, A4, B2, B5 | — | B5 |
| - Containers | A3, A5, B2, B4 | — | B4, B7 |
| - Heavy bulk cargos and heavy general cargos | A1, A2, A5, B1, B4 | — | B4 |
| INDUSTRIAL | A1, A2, A5, B1, B4 | — | B4 |
| MILITARY | A1, A2, A4, B2, B5 | — | B5 |
| FISHING | B3, B6 | — | B6 |
| MARINAS | B3, B6 | — | B6 |

The minimum load trains equivalent to restricted movement equipment shall be applicable at any distance (b) between the first rail and the seaward operation zone's exterior limit (edge of the pier or berth). Unless this distance is determined in the project due to structural or functional considerations, the following values are recommended :

— For A1 $2 \leq b \leq 3$
— For A2 $2 \leq b \leq 3$
— For A3 $b = 3$
— For A4 $b \geq 2$
— For A5 $b \geq 3$

When the project engineer does not have sufficient information about the cargo or materials to be handled, or about the most appropriate equipment for their handling, only the minimum load trains may be adopted, without the need to list the cargo handling equipment that could effect the project, nor to define their type and geometry as was previously stated.

■ DYNAMIC EFFECTS

The existence of frequential or impact loads associated with cargo handling equipment and installation overloads in normal operating conditions shall be taken into account in the calculation. These loads shall be considered to be superimposed on the self weight of the handling system, the service load and the environmental actions. The dynamic loads are caused, generically and without distinguishing between handling system, by the following effects:

— Vertical Movements : Vertical inertial forces and impacts.

- Lifting or lowering of the service load.
- Acceleration or deceleration of the elevation movement.
- Irregularities in the surface of equipment tracks.

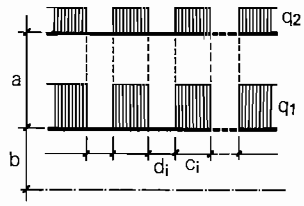
## TABLE 3.4.2.3.2.5 MINIMUM LOAD TRAINS, EQUIVALENT TO CARGO HANDLING EQUIPMENT AND INSTALLATION OVERLOADS

### A. RESTRICTED MOVEMENT EQUIPMENT (CRANES ON RAILS)

**A1**

| | Normal operating conditions | Extreme conditions |
|---|---|---|
| $q_1$ | 25 t/m | 22 t/m |
| $q_2$ | 8 t/m | 10 t/m |
| $H_{VT}$ | 0,8 t/m | 2,5 t/m |
| $H_{VL}$ | 0 t/m | 0 t/m |

(length in meters)

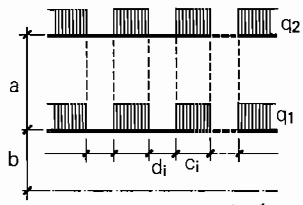



$4{,}5 \leqslant a \leqslant 10$ $c_i \geqslant 1$ $d_i \geqslant 0$

**A2**

| | Normal operating conditions | Extreme Conditions |
|---|---|---|
| $q_1$ | 15 t/m | 14 t/m |
| $q_2$ | 15 t/m | 14 t/m |
| $H_{VT}$ | 0 t/m | 0 t/m |
| $H_{VL}$ | 0,8 t/m | 2,5 t/m |

(length in meters)

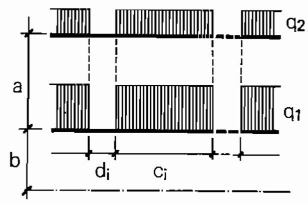



$4{,}5 \leqslant a \leqslant 10$ $c_i \geqslant 1$ $d_i \geqslant 0$

**A3**

| | Normal operating conditions | Extreme conditions |
|---|---|---|
| $q_1$ | 25 t/m | 15 t/m |
| $q_2$ | 12 t/m | 20 t/m |
| $H_{VT}$ o $H_{VL}$ | 0.5 t/m | 1.5 t/m |

(length in meters)

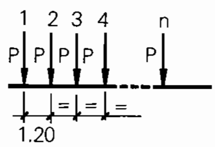



$15 \leqslant a \leqslant 18$ $c_i \geqslant 5$ $d_i \geqslant 0$

**A4**

| | Normal operating conditions | Extreme conditions |
|---|---|---|
| P | 25 t | 18 t |

(length in meters)

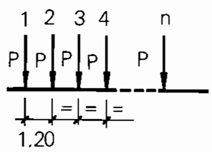



$2 \leqslant n \leqslant 6$

**A5**

| | Normal operating conditions | Extreme conditions |
|---|---|---|
| P | 40 t | 30 t |

(length in meters)




$6 \leqslant n \leqslant 8$

**TABLE 3.4.2.3.2.5 (Continued)**

B. UNRESTRICTED MOVEMENT EQUIPMENT

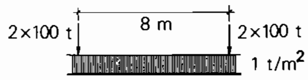
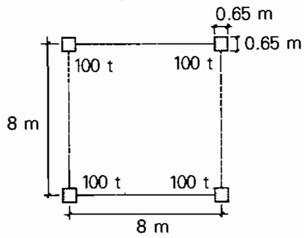
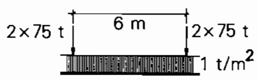
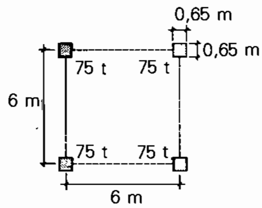
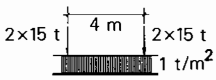
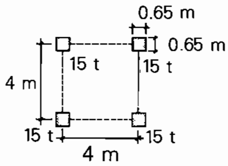
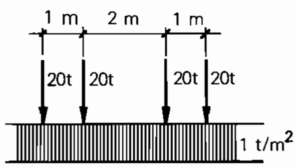
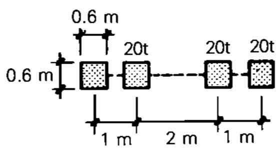
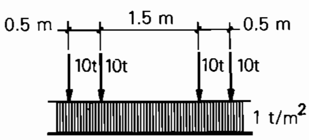
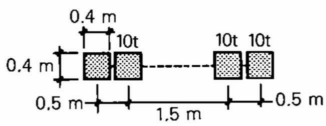
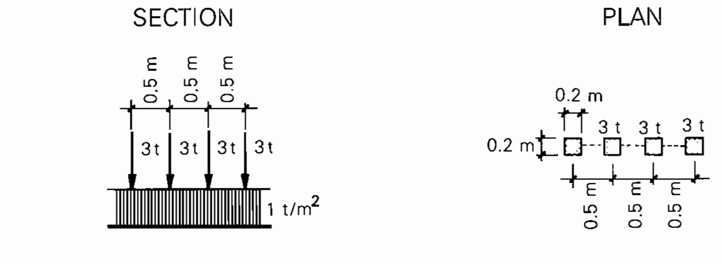
| | |
|---|---|
| B1 | SECTION   PLAN  <br>Uniform overload equivalent to the mobile train: 14 t/m$^2$ in 5.5 × 5.5 m$^2$ |
| B2 | SECTION   PLAN  <br>Uniform overload equivalent to the mobile train: 12 t/m$^2$ in 5 × 5 m$^2$ |
| B3 | SECTION   PLAN  <br>Uniform overload equivalent to the mobile train: 3 t/m$^2$ in 4 × 4 m$^2$ |
| B4 | SECTION   PLAN  <br>Uniform overload equivalent to the mobile train: 4 t/m$^2$ in 5 × 5 m$^2$ |
| B5 | SECTION   PLAN  <br>Uniform overload equivalent to the mobile train: 2.5 t/m$^2$ in 4.5 × 4.5 m$^2$ |

TABLE 3.4.2.3.2.5 (Continued)

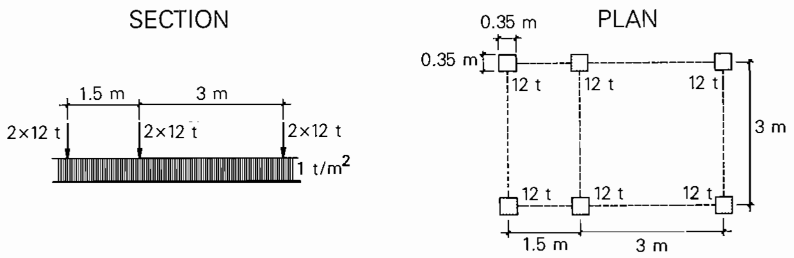
B. UNRESTRICTED MOVEMENT EQUIPMENT (Continued)

B6




Uniform overload equivalent to the mobile train: 1.5 t/m² in 3 x 3 m²

B7




Uniform overload equivalent to the mobile train: 1.5 t/m² in 7 x 7 m²

**LEGEND :**

$q_i$ : Vertical load per linear meter corresponding to the rail i.

$H_{VT}$ : Horizontal load per linear meter perpendicular to the rail, corresponding to each one of the rails. Its distribution shall be identical to the linear vertical load.

$H_{VL}$ : Horizontal load per linear meter, parallel to the rail, corresponding to each one of the rails. Its distribution shall be identical to the linear vertical load.

P : Point load, equivalent to a load per wheel, corresponding to circulation along one rail.

—·—·— : Seaside exterior limit of the operation zone(e.g. pier's edge).

**APPLICATION CONDITIONS :**

- Load Trains A1 to A3 are uniformly distributed lineal overloads, of indefinite length, with values a, b, $c_i$ and $d_i$ that within the indicated limits produce the most unfavorable effects. Their order can be inverted, alternately applying each load train to each alignment (rail). These load trains correspond to circulation on one set of tracks.
- Load Trains A4 and A5 are point loads, of limited length. They shall be applied individually where they produce the most unfavorable results. These loads correspond to circulation on one rail.
- B Load Trains are uniformly distributed surface overloads, acting simultaneously with a mobile train of point loads.
  The distributed overloads have an indefinite character, being partially or completely extended over the application surface, according to what is most unfavorable for the studied element.
  The point Load Train of limited character, shall be displaced either longitudinally or transversally, or change direction in a manner that would produce the most unfavorable effects. This load train shall be applied individually.
- The given values do not include dynamic effects produced fundamentally by inertial forces.
  These effects shall be evaluated according to the criteria in the "Dynamic Effects" paragraph.

— Horizontal Movements : Horizontal inertial forces.

- Translation movements.- Starting, breaking and speed changes.
- Rotations and reach changes.
- Centrifugal force in works with a curved plan alignment.
- Shifting of tracks.

— Vibrations induced by bending or torsion

The induced dynamic effects shall be taken into account in terms of Vertical Load Amplification Factors (I: Impact Factor), and Additional Horizontal Actions ($H_A$), whose values shall depend upon the type of cargo handling equipment or installations used, and the distribution adopted for the load trains.

The impact Factor (I) shall take into account the Dynamic effects produced by the vertical inertial forces. The additional action ($H_A$) shall take into account the effects produced by the horizontal inertial forces such as breaking and starting, centrifugal forces or shifting of tracks.

Due to the low translation and elevation velocities at which the cargo handling equipment operates, it is not probable that dynamic effects due to vibrations will occur. Lacking other data, common frequencies of between 25 and 50 Hz shall be considered.

In the absence of more precise data, the Impact Factors and Additional Horizontal Actions from table 3.4.2.3.2.6. may be used, and shall be valid for the service phase and normal operating conditions.

These factors shall also be applicable to the minimum load trains given in tables, considering that the type A load trains refer to equipment on rails, and type B loads refer to equipment on rubber tires, outriggers or treads.

It shall be considered that the impact factors and the additional horizontal loads take into account the simultaneous occurrence of these effects.

In extreme conditions, dynamic effects shall not be considered.

■ ACTION DIFFERENTIATION

a) BY PROJECT PHASES

The overloads due to cargo or material handling equipment and installations applicable to each resistant structure shall be distinguished in function of the project phase being studied.

For the construction phase, the Project Engineer shall consider overloads in function of the necessary or specified construction procedures and work equipment foreseen in the project. The environmental and work conditions compatible with these overloads shall be indicated.
Lacking other data, when mobile cranes or other equipment on treads are considered in the construction procedure, the loads transmitted by the equipment types given in table 3.4.2.3.2.3. may be applied, as long as they are equivalent to the equipment anticipated.

For the service phase, the handling equipment and installations in normal operating conditions, extreme conditions and exceptional conditions shall be distinguished. The last two work hypotheses shall only be taken into consideration when the overloads are created by continuous handling equipment, discontinuous restricted mobility equipment or by equipment that can not be withdrawn once the limit operating conditions are surpassed. At this moment, it shall be considered that the equipment ceases activity, adopting a storm position with the least possible surface exposed to the environmental actions.

According to Section 3.2.3.- Representative Values of Variable Loads, the limit operating conditions shall be set by the Project Engineer based on the application of the use conditions and criteria of the port installation, taking into account the limit functioning conditions established in the design of the handling equipment.
In the absence of specific criteria, a mean wind velocity in 3 s. of 22 m/s shall be adopted as the limit operating condition.

**TABLE 3.4.2.3.2.6 IMPACT FACTORS AND ADDITIONAL HORIZONTAL ACTIONS FOR THE CONSIDERATION OF DYNAMIC EFFECTS OF CARGO HANDLING EQUIPMENT AND INSTALLATION OVERLOADS UNDER NORMAL OPERATING CONDITIONS**

| SYSTEMS | VERTICAL LOAD AMPLIFICATION FACTOR (I) | | ADDITIONAL HORIZONTAL ACTIONS ($H_A$) |
|---|---|---|---|
| | Surface Loads | Lineal or concentrated loads | |
| - CONTINUOUS SYSTEMS | 1.00 | 1.20 | 0.00 |
| - DISCONTINUOUS SYSTEMS | | | |
| - Fixed equipment | 1.00 | 1.20 | 0.00 |
| - Restricted mobility equipment (on rails) | — | 1.20 | In the direction of the rail:1/7 of the vertical loads. (1)(2)(3)(6). In the transverse direction:1/10 of the vertical loads. (2)(3)(6). |
| - Unrestricted mobility equipment (on rubber tires or treads) | 1.00 | 1.15 | 1/20 of the vertical load. (4)(5)(6). |

**NOTES :**

(1) The additional horizontal actions in the direction of the track ($H_A$) to be considered in equipment on rails, shall correspond to 1/7 of the vertical load transmitted by the driving wheels. Generally, the driving wheels reach 2/3 to 3/4 of the total existing wheels, reaching in some cases the total wheel load.
To take into account the most unfavorable case, all of the wheels must be considered driving wheels.
When the project engineer knows precisely the principle characteristics of the projected equipment, he may reduce $H_A$ in proportion to the number of driving wheels, to no less than 2/3 of the additional horizontal load given in this table.

(2) This action shall be applied acting at the height of the top of the rails in the most unfavorable position and direction for the analyzed element, with the identical disposition and distribution as the compatible vertical overload.

(3) When load trains equivalent to various pieces of equipment working together are adopted, horizontal actions in a single piece of equipment shall be considered, in a way that results in the most unfavorable effect for the studied element. When applying minimum load trains, the whole load shall be considered, except in load trains equivalent to restricted movement equipment in which the equivalent horizontal action shall be distributed over a maximum length of 15 m; independently of the adopted compatible vertical lineal overload distribution.

(4) This action is assumed to act at the height of the pavement surface according to the longitudinal direction defined by the load trains with identical disposition and distribution as the compatible vertical load (load trains).

| NOTES TO TABLE 3.4.2.3.2.6 (Continued) |
|---|
| (5) To obtain additional horizontal actions based on uniformly distributed surface overloads, the following shall be taken into account :<br><br>- the possibility of larger horizontal forces associated with small surfaces.<br>- the slight possibility of all the actions acting simultaneously over large surfaces.<br><br>For this purpose, the additional action's maximum and minimum values shall be limited to the range :<br><br>$8 \text{ t} \leq H_A \leq 25 \text{ t}$<br><br>(6) To obtain an additional horizontal action, the vertical loads shall be considered without amplifying for dynamic effects. |

For extreme conditions, environmental action values corresponding to return periods associated with maximum admissible risk shall be used. According to Sections 2.2. Design Life and 3.2.- Action Evaluation Criteria, the maximum admissible risk shall be 20% for 25 year equipment design lives, which yields return periods of approximately 100 years.

When the analyzed structure is subjected to extreme conditions, in which environmental actions act with return periods greater than those adopted for the evaluation of handling equipment overloads in those conditions, the verification of the resistant structure in this hypothesis shall consider the loads transmitted by the equipment, in the extreme condition of the structure, as accidental (exceptional condition for the equipment), being included in the calculations with the safety coefficients that correspond to those types of actions.

Likewise, and as a differentiated exceptional condition, all the structural elements shall be designed to resist the overloads transmitted by the handling equipment and installations during the tests and trials required by the codes in effect applicable to each installation. In particular, for heavy lifting equipment, the load tests specified in the Code UNE 58-107-72, primarily the test that consists of suspending a load equal to 150% of the nominal load maintaining the boom at maximum reach in calm wind conditions will be taken into account.

The minimum load trains in tables give different values of actions for normal operating conditions and extreme conditions, and are only valid for the service phase.

b) BY STRUCTURAL TYPES

For each structural type defined in section 3.4.2.3.1. Stage and Storage Overloads. Differentiation of the action by structural type, the following simplifications in the handling equipment and installation overloads may be admitted :

— *STRUCTURE TYPE 1*

The cargo handling equipment and installation overloads in the general conditions shown in this section shall be completely applicable to structures where the loads act directly upon the analyzed structural elements. Nevertheless, the impact factor (I) shall not be applied in the calculation of those structural elements that receive the loads indirectly through other elements with a capacity to buffer impact loads. The impact factor shall be considered for the calculation of slab and decks, pavement, support beams for crane rails, bents, etc, but not for the calculation of pillars, pilings or foundations.

When the distance between the application surfaces of the loads and the axis of the analyzed resistant structure is significant, the distribution of the acting loads over this surface shall be done according to the local load distribution criteria defined for Type 2 structures, without neglecting other types of forces that could be caused because the load application point and the axis of the analyzed element do not coincide.

— *STRUCTURE TYPE 2*

— *Structures with load distribution layer less than or equal to 1.50 m*

For Type 2 structures with a load distribution layer less than or equal to 1.50 m, It shall be considered that the vertical load trains applied on the surface, formed by lineal or concentrated loads, are uniformly distributed in a zone of the resistant structure conforming to the following :

- As the overload's application surface, those adopted for type 1 structures can be taken; being for rail equipment that which corresponds to the rail base support.
- The loads shall be assumed to be transmitted through the thickness of the load distribution layer, extending planes from the edges of the application surface of the load at a slope of 2 (vertical)/1 (horizontal) or 1/1 according to which is more unfavorable.
  The most favorable distributions may be adopted whenever they are justified, by means of theoretical models of recognized validity.

Likewise, for the evaluation of local effects, horizontal loads or load trains may be assumed to be uniformly distributed over the same area and with identical criteria as the associated vertical loads or load trains.

Reductions in the impact coefficients adopted for Type 1 structures shall be admitted, in function of the thickness of the load distribution layer. This coefficient shall be considered to vary linearly from the application surface of the loads, reducing to zero when the load distribution layer is greater than or equal to 1.50 m. Therefore, for Type 2 structures with a load distribution layer of thickness h, the impact coefficient (I') to consider shall be :

$$I' = I - \frac{2}{3} \cdot I \cdot h, \qquad \text{for} \leq 1{,}50 \text{ m.}$$

(h in meters)

given that I is the Impact coefficient on the surface

— *Structures with load distribution layer greater than 1.50 m*

If the load distribution layer is greater than 1.50 m, neither the application of concentrated loads nor dynamic effects shall be considered.

The load trains due to equipment on rails or restricted mobility equipment shall be taken into account in the calculations with equal local distribution as for structures with load distribution layers of less than 1.50 m, with simplifications being admitted in the distribution and variability of the loads acting on the surface.

The load trains due to unrestricted mobility equipment shall only be taken into account as uniform overloads in the equivalent area, being distributed locally through the load distribution layer in the previously shown manner.

For Type 2 structures the additional forces produced when the load's application point and the axis of the analyzed element do not coincide shall be taken into account.

— *STRUCTURE TYPE 3*

Given the great load distribution capacity of these types of structures, the overloads adopted for Type 1 structures shall be simplified and homogenized. It will not be necessary to consider the concentrated loads or load trains and impact factors in the calculations.

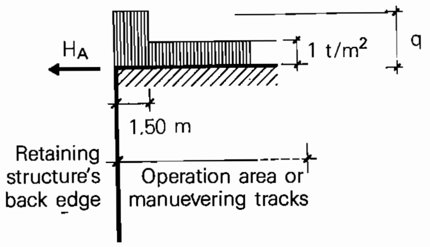
The minimum load trains assigned in table 3.4.2.3.2.5. may be simplified according to the following criteria :

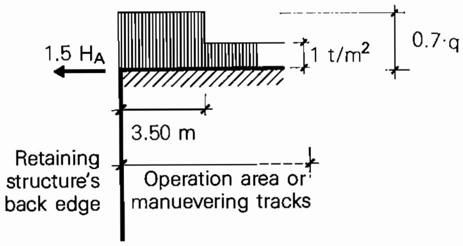
— *A Load Trains*

- Load Trains A4 and A5 shall not be considered
- Load Trains A1, A2 and A3 shall be considered to have an indefinite length ($d_i$ = 0).
- Additional horizontal loads compatible with the applied vertical overload shall be considered.

— *B Load Trains*

- Load Trains B1 and B7 shall be substituted by :




| | B1 | B2 | B3 | B4 | B5 | B6 | B7 |
|---|---|---|---|---|---|---|---|
| q (t/m$^2$) | 14 | 12 | 3 | 4 | 2.5 | 1.5 | 1.5 |
| $H_A$ (t/m) | 1 | 0.9 | 0.25 | 0.3 | 0.2 | 0.12 | 0.12 |

The overloads of 1 t/m$^2$, q, and $H_A$ shall be considered to be of indefinite length.

Lacking other data, these overloads shall also be considered equivalent to excavators or other construction equipment.

### 3.4.2.3.3. TRAFFIC OVERLOADS ($Q_{v3k}$)

■ DEFINITION

Traffic Overloads are defined as variable nature loads due to the different conventional means of transporting cargo, materials or supplies.

Loads produced by the following shall be included as traffic overloads :

— Conventional Road Traffic (Heavy vehicles).
— Rail Traffic
— Helicopters (On offshore industrial platforms)

■ DETERMINATION

The value of overloads transmitted by conventional heavy road vehicles and rail traffic shall generally be determined from the normative in effect relative to the actions to be considered in bridge projects :

— Instrucción Relativa a las Acciones a Considerar en el Proyecto de Puentes de Carreteras: MOPU - February 1972.
— Instrucción Relativa a las Acciones a Considerar en el Proyecto de Puentes de Ferrocarril: MOPU - June 1975.

although with some limitations and modifications in the loads and application condition, in order to take into account the different characteristics and performance conditions of the conventional traffic in port areas. These modifications shall especially effect the rail traffic overloads, analyzed in this section.

Articles of the cited Codes that contradict the general criteria of action evaluation and calculation bases introduced in these Recommendations shall not be applicable, especially in reference to the evaluation of environmental actions by means of risk criteria. In any case, that which can be found in Section 3.2. Action Evaluation Criteria, Section 3.4.2.4. Environmental Loads and in Part 4. Calculation Bases, will be followed.

The characteristics values of the actions transmitted by helicopters shall be directly provided by the helicopter manufacturers and suppliers for the maximum helicopter foreseen in the installation use plan or if these values are not avilable, values shall be set by the Client or Government Authority, in the following conditions :

— *Normal Operating Conditions :*

- Landing helicopter : Helicopter with Maximum load. Landing conditions upon two wheels of least spacing, or a landing skid, independent of the total number with which it is equipped. Operating limit wind.
- Parked helicopter : Helicopter with maximum loads. Without wind

— *In Extreme Conditions :*

- Helicopter parked without load, with or without storm moorings. Extreme condition wind.

Lacking greater definition, in normal operating conditions (landing) and without considering dynamic effects, the following shall be adopted as the traffic overload produced by helicopters :

— Vertical load $F_v = 0.75 \cdot P_h$
— Horizontal load $F_h = 0.50\ F_v$

acting upon a surface of 0.30 x 0.30 $m^2$, and given that $P_h$ is the maximum helicopter weight landing and with maximum load. The direction of the horizontal force shall be considered to coincide with the direction of the design wind; which is the maximum wind compatible with the performance of this load and corresponds to the operating limit of landing helicopters. This load shall be applied in a way that produces the most unfavorable effects on the analyzed structure.

— *APPLICATION CONDITIONS OF TRAFFIC OVERLOADS*

The traffic overloads shall be applied with the following conditions and limitations :

— The overloads due to conventional road traffic shall be taken into consideration in all those areas accessible to this type of traffic.
For the purposes of simplification, it shall be admitted that these loads shall not act simultaneously with other use overloads in the same area, except with the rail traffic overloads. Therefore, it shall be checked alternatively if the Load Trains given in the Highway Bridge Design Codes, slightly modified (see modifications

in the values foreseen in Bridge Codes), are more unfavorable for the design than the combinations of overloads in table 3.4.2.3.2.4.

Generally, these overloads shall not be critical in operation, storage or maneuvering track areas, except for fishing docks or marinas.
They shall be especially significant in works specifically designed as Access tracks.

— The rail traffic overloads shall be applied in those areas where railways are specifically included in the project, and in those cases where the functioning or the use conditions of the installation demand it. Likewise, foreseeing possible changes in the use conditions during the design life of the project, rail traffic overloads inside the performance band of the cargo handling equipment on rails in operation areas shall always be considered, whenever there is the possibility, however remote, of the existence of this type of traffic in the zone (if the city where the project is located has a railway).

For the purpose of simplification, it shall be admitted that these loads can simultaneously be applied with other use overloads in the same area, although not superimposed within the circulation Band of the railway. In these conditions, rail overloads shall be incorporated into cases I, II, and III of table 3.4.2.3.2.4. Overload Capability, or combined with the load corresponding to conventional road traffic, according to that which is more unfavorable for the analyzed structure.

Lacking other data, the zone limited by lines parallel to the tracks, situated on both sides of one or various railways and at 3.00 m from the axis of the extremes shall be considered the Circulation Band of the railway. Railways whose separation between axes surpass 6.00 m shall not be considered multiple railways.

If the circulation band of the railway is included inside the performance band of a restricted mobility cargo handling equipment (e.g. portal crane), the circulation band of the first shall be considered to coincide with the performance of the second. The number of railways included in each performance band of this equipment shall depend upon their width. The following ralationships may be used :

- Performance Band of 6 m : 1 railway
- Performance Band of 10 m : 2 railway
- Performance Band of 15 m : 3 railway
- Performance Band of 18 m : 3 railway

The separation between railways shall be taken as homogeneous, situating the axis of the extremes at 3.00 m from the rail of the cranes.

Generally, the rail traffic overloads shall not be critical in storage areas, except for fishing docks or marinas. In these facilities, it shall not be necessary to take rail traffic overloads into consideration. They will be especially significant in operation areas and in works designed specifically as Traffic Tracks.

— The overloads due to helicopters shall be applied only in those areas considered in the use plans, and therefore in the design, as heliports or helisurfaces. These areas shall generally exist in industrial facilities or other types of installations offshore.

With helicopter traffic overloads, the simultaneous action of other use overloads shall not be considered, except those produced by pedestrians (0.5 t/m$^2$ in stationary conditions and 0.2 t/m$^2$ in landing or takeoff conditions).

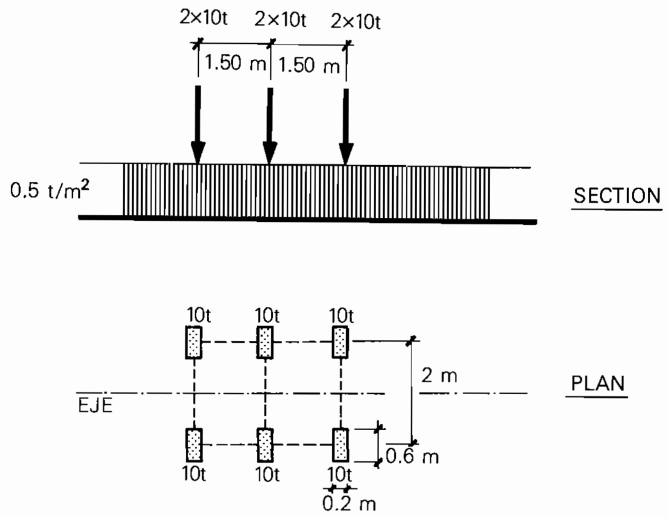
If the use criteria, the Client or the Government Authority do not guarantee the exclusive use of these zones for helicopter traffic, and their permanent exemption from the effects of staging and storage of cargo, materials or supplies, and foreseeing unplanned transitory stacking, the project engineer shall alternatively check the analyzed structures with the minimum staging overloads in operation areas.

— *MODIFICATIONS IN THE LOAD TRAINS FORESEEN IN THE ROAD AND RAILWAY BRIDGE CODES FOR THEIR USE AS TRAFFIC OVERLOADS IN PORT ZONES*

— *Load Train corresponding to Conventional Road Traffic*

For coherence with the use overloads set in these Recommendations for zones accessible only to pedestrians, and given the higher percentage of heavy vehicles in port zones and the lesser degree of channelling of the traffic, the uniform overload of 0.4 t/m$^2$ that forms part of the load train of the Highway Bridge Code shall be substituted for an overload of 0.5 t/m$^2$.

Therefore, the Load Train corresponding to conventional road traffic applicable in port zones shall be :




Uniform overload equivalent to the mobile train: 3 t / m$^2$ in 6 x 3 m$^2$.

In those areas where the conventional traffic cannot be considered channelled, the vehicle (concentrated load train) could be situated in any orientation and displaced in any direction.

— *Load Trains corresponding to Railway Traffic*

Load Trains corresponding to the railways included in the Railway Bridge Loading Codes shall not be applicable to the design of port works with railway traffic exclusively servicing these zones; nevertheless these trains can be alternatively taken into account as accidental loads.

The substitution of load trains is due to the railway traffic characteristics in port areas substantially differing from the conventional railway traffic, fundamentally because of the following causes :

- Exclusively cargo trains.
- Maneuvering locomotives of less weight, and restricted circulation.
- Lack of continuity in the train composition and wagon placement during loading and unloading.
- Reduced circulation velocities, with frequent stopping and starting.
- Wagons subject to considerable impacts during the loading process.
- Normal or accidental rail disleveling due to the multiplicity of uses for the platform.
- Rail alignment with numerous and small radius curves.

The load trains to consider in the design of port works are given in table 3.4.2.3.3.1 and are differentiated according to railway width. These load trains correspond to circulation along one railway, with half of the load being applied to each rail. The train type that produces the most unfavorable effects together with the compatible use overloads shall be adopted for each structural element analyzed.

The load trains T1 and T3 are formed by uniformly distributed linear overloads of indefinite length, and with values $a_1$, $a_2$,...$a_i$ that produce the most unfavorable effects.
The load trains T2 and T4 are created by point overloads of limited length, being uniquely applied in such a way as to produce the most unfavorable effects.
For multiple railways, the combinations of the prior cases that have the most unfavorable results shall be designed for with the reductions considered in the Bridge Code.

The uniform overload equivalent to the railway load trains shall be 5 t/m$^2$, extended along the length of the circulation band.

■ DYNAMIC EFFECTS

The existence of frequential actions, primarily impact actions (circulation, starting and braking) associated with the traffic loads shall be taken into account in the calculations, in terms of a static analysis by means of amplification factors and additional actions.

— The evaluation of dynamic effects associated with the conventional road traffic shall be carried out according to the criteria and limitations adopted by the Highway Bridge Loading Codes, summarized as :

- Load amplifications due to vertical inertial forces and impacts (Fundamentally caused by superficial irregularities) shall not be taken into account, nevertheless, the possibility of induced vibrations must be analyzed.

- The braking, starting and velocity change force shall be estimated as a horizontal force equal to 1/20 of the acting load train.

Nevertheless, the following modifications shall be considered with respect to the cited Code :

- In order to quantify the starting, braking , velocity change and centrifugal forces, the uniform load that acts simultaneously with the vehicle shall be 0.5 t/m$^2$; coinciding with the modified load train corresponding to conventional road traffic applicable in port zones.

- In those areas where the conventional traffic cannot be considered channelled (e.g. in operation area), the starting, braking, velocity change and centrifugal forces shall be assumed to act at the height of the pavement surface, in to the longitudinal direction defined by the vehicle (starting, braking, and velocity change) or the perpendicular direction (centrifugal force in projects with a curved alignment) with identical disposition and distribution as the associated vertical load.

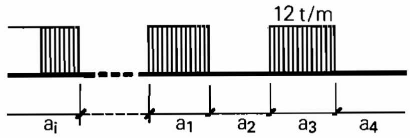
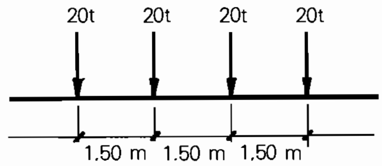
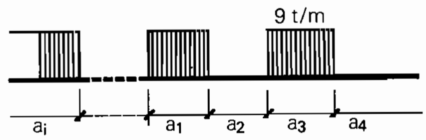
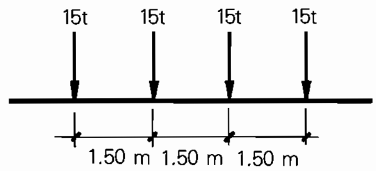
— For railway traffic, the consideration of forces associated with impacts, starting, braking, centrifugal force, losso effect and shifting of tracks shall be carried out according to the criteria assigned in the Railway Bridge Loading Codes, for load trains applicable to port works.

Nevertheless, the following modifications shall be considered, with respect to the cited Code :

- A load increase factor of 1.20 shall be adopted, independently of the velocity of the train, for the consideration of Impact.
The criteria modification with respect to the Bridge Code is due to the underestimating of the impact effect in this Norm for low train velocities, which are very common in port zones.

- When the design train is considered as an equivalent uniform overload, the impact effect will not be taken into account.
To evaluate the additional horizontal forces (starting, braking, etc), it shall be assumed that the uniform overload is equivalent to the weight of the design train.

— The dynamic effects due to helicopter impact upon a helisurface at the moment of landing shall be taken into account with an increase factor of 1.50 of the load transmitted by the helicopter in normal operating conditions, according to prior defined criteria.

| TABLE 3.4.2.3.3.1 RAILWAY TRAFFIC OVERLOADS. TYPE LOADS TRAIN | | |
|---|---|---|
| RAILWAY TYPE | DESIGN LOAD TRAIN | VERTICAL OVERLOAD (1) |
| RENFE (SPANISH) OR EUROPEAN (2) | T1 |   $a_1, a_2, a_3..., a_i \geqslant 0$ |
| | T2 |   |
| METRIC (2) | T3 |   $a_1, a_2..., a_i \geqslant 0$ |
| | T4 |   |

**NOTES :**

(1) Circulation along one set of tracks
(2) Regarding load application, the following shall be considered the railway widths : Renfe = 1.70 m, European = 1.50 m, Metric = 1.00 m; with half of the design train acting on each rail.

■ ACTION DIFFERENTIATION

a) BY PROJECT PHASES

The general criteria assigned in Section 3.4.2.3.2.- Cargo handling equipment and installation overloads, where the differentiation of the action by project phases is referred to, shall be applicable to traffic overloads.

For the Service Phase, The traffic overloads shall be distinguished for normal operating conditions, extreme conditions and exceptional conditions.
The extreme conditions hypothesis shall not be taken into account for the evaluation of overloads due to conventional road traffic, assuming their withdrawal once the operating limit conditions are surpassed.
The values assigned in this section to the conventional road traffic overloads, basically coinciding with the specifications in the Bridge Codes, amply cover the variations due to environmental actions in normal operating conditions.
For exceptional conditions and as an accidental action, the possibility of vehicle collision shall be taken into account according to the Highway Bridge Code.

The performance of environmental actions, in normal operating conditions as well as in extreme or exceptional conditions, shall be taken into account in the evaluation of railway traffic overloads, according to Section 3.4.2.4. Environmental Loads, and the ROM 0.4. Consideration of Environmental Variables/II : Atmospheric and seismic conditions, whenever their effect is significant in the calculations. In such cases, it shall be considered that the real train cross section, in the loaded span hypothesis, is equal to a rectangle 3.50 m in height from the top of the rail, by 3.00 m wide.
To evaluate the wind action, the mean velocity for 3s ($V_{3s}$) shall be applied.
In the absence of specific criteria, the operating limit wind shall coincide with that established for cargo handling equipment. In regards to the combination of design load trains equivalent to railway traffic with environmental actions, and in order to account for unfavourably loaded structural elements when extreme environmental loads act simultaneously with minimum weight design trains (unloaded), design trains formed by 1 t/m distributed overloads shall be considered whenever their application is more unfavorable.
The concentrated load trains given in the Bridge Code can be considered as accidental actions (exceptional conditions), in order to take into account the remote possibility of locomotives of greater tonnage than usual.

The helicopter traffic loads according to the evaluation criteria contained in this section, have been obtained for normal operating conditions and extreme conditions. The operating limits shall be set by the Project Engineer, the Client or the Government Authority according to the operating criteria for landing and take off defined by the manufacturers of the design helicopter, and will be incorporated in the installation use conditions.
When the operating limits are surpassed, it shall be assumed that the helicopter remains inoperative, being anchored or not on the helisurface. In those cases where it remains anchored, the pull produced by the anchors in extreme environmental conditions shall be taken into account.
Lacking additional information, a load equal to 2.5 times that corresponding to normal operating conditions, equivalent to emergency landing due to failure of the landing gear shall be considered as an accidental action.

Likewise, and as a seaparate exceptional condition, all the structural elements shall be designed to resist the overloads transmitted by the load tests required by the Technical Specifications Dossier.

b) BY STRUCTURAL TYPES

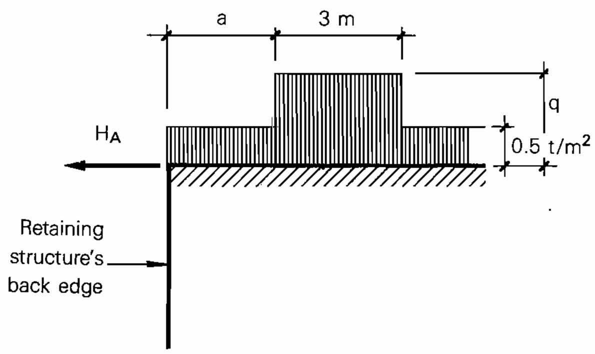
The traffic overloads in the conditions reflected in this section shall be completely applied to Type 1 structures (See related paragraph in section 3.4.2.3.1) with the limitations referring to the consideration of dynamic effects, load distribution, and the existence of forces due to the lack of coincidence between the application point of the load and the axis of the analyzed element, set in the related paragraph in section 3.4.2.3.2. Cargo handling equipment and installation overloads.

Likewise, for Type 2 structures with a load distribution layer less than or equal to 1.50 m, it shall be assumed that the loads acting on the surface are distributed, and the impact factors are modified according to the criteria given in the cited section.

In Type 2 structures with a load distribution layer greater than 1.50 m, the existence of dynamic effects and the performance of loads or load trains, lineal or concentrated

(heavy vehicles for conventional road traffic, load trains for railway traffic, helicopter loads) shall not be considered.
The load trains corresponding to conventional road traffic may be simplified as uniform overloads in the equivalent area (3 t / m$^2$ in 6 x 3 m$^2$). They are distributed locally through the distribution layer in the previously shown manner. Therefore, these loads shall only be significant in operation areas, storage areas and maneuvering tracks in fishing ports or marinas, and access tracks .
The railway traffic may be taken into account in terms of a uniform overload of 5 t/m$^2$ in a band of 3.00 m wide per railway, centered along the axis of the tracks. The railway traffic overload simultaneousness criteria shall be applied in the case of design trains operating on multiple sets of tracks. The distribution of loads through the distribution layer shall be the same as for structures with a distribution layer less than 1.50 m. In the case of multiple railways, where distribution zones are superimposed, it shall be considered that the total overload is distributed from the extremes of the circulation band.

The simplifications of vertical loads, admitted for Type 2 structures with a load distribution layer greater than 1.50 m shall be applied to Type 3 structures. Nevertheless, the additional horizontal loads compatible with the applied vertical loads shall be considered to act at the top of the analyzed retaining structure.
In order to ease the application of overloads corresponding to conventional road traffic acting upon retaining structures, the load trains shown in this section together with the applicable simplifications, may be substituted by the following indefinite length overloads :




| | a ≥ 2 m | 1 m ≤ a < 2 m | 0.5 m ≤ a < 1 m |
|---|---|---|---|
| q (t/m$^2$) | 2 | 3 | 4 |
| $H_A$ (t/m) | 0.4 | 0.5 | 0.6 |

These overloads shall not be significant in operation and storage areas, except for fishing ports or marinas, or in maneuvering tracks; since the minimum use overloads corresponding to stored cargo or cargo handling equipment are greater in these areas.

### 1.3.4.2.3.4 OVERLOADS FOR THE DESIGN OF PAVEMENTS AND YARDS ($Q_{V4k}$)

- DEFINITION

Overloads for the Design of Pavements and Yards are defined as the fictitious actions equivalent, in terms of failure or deterioration of pavements, to the loads produced by the different cargo transport and handling equipment on rubber tires or treads, circulating in a restricted (e.g. storage portal crane) or unrestricted (e.g. fork lift trucks), and repeated manners upon pavements and yards.
These loads shall simultaneously take into account the different types of acting vehicles (transmitted loads and their effects) and the frequency of use of each one of them during the design life of the pavement (15 or 25 year minimum for definitive projects according to table 2.2.1.1).

These overloads shall only be applicable for the design of pavements and yards through analytical or empirical methods based on the accumulated effects caused by the repeated passing of a load or load train.
These methods shall be complemented by the calculation of the pavement by means of static analysis, for the largest loads that can act on the pavement. The largest loads transmitted to the pavement shall generally be due to cargo handling and transport equipment (See section 3.4.2.3.2. Cargo Handling Equipment and Installation Overloads ($Q_{V2k}$)) and to support elements and systems of the staged cargo (See section 3.4.2.3.1. Stage and Storage Overloads ($Q_{V1k}$).

- DETERMINATION

The typology of the use overloads to consider for the design of pavements and yards situated in port zones shall fundamentally depend upon the use areas where they are located, since this determines the type of cargo handling and transport equipment and the possibility of its staging and storage, and therefore the loads that can act and the total number of applications.

a) IN OPERATION AREAS AND MANEUVERING WAYS

Zones fundamentally subjected to mobile loads produced by the circulation and aggression of a wide rande of handling and transport equipment, whose traffic is randomly distributed in areas that are generally not very defined or channelled.

The parameters, methods and design factors usually used for highway pavement projects and adopted by the normative in effect (Highway Instructions, Code 6.1 and 2-1C Pavement Sections) shall not be applicable to pavement projects in these zones due to the following reasons :

— The wheel loads that the cargo handling and transport equipment transmit are much greater than those transmitted by heavy conventional traffic vehicles, easily surpassing 25 t.
— The range of types and sizes of equipment, and therefore of transmitted loads, pressure and wheel separations, differ substantially from the conventional traffic for which the equivalencies given in the Highway Code have been determined.
— The dynamic forces are quite significant due to impacts, turns, braking, superficial irregularities, etc.
— The circulation areas are not very defined or channelled.
— The difficulty in knowing beforehand the typology of the traffic and its evolution, and the lack of accumulated experience in port zones.

The design of pavements in operation areas and maneuvering tracks in port zones shall require the consideration of the cargo handling and transport equipment that effects the projected work, as well as its principle characteristics and the loads transmitted by each type of equipment in each work condition. Furthermore, specific studies shall be necessary to determine the performance frequency of each type of equipment during the analyzed project phase.

In order to ease the design of pavements in these areas, given all the difficulties in the consideration of diverse actions together with their frequency of application, to evaluate the accumulated traffic during the design life of the project, the following overload evaluation method may, homogeneous with the method used in the Highway Code, may be adopted :

— As a simplification of the actions, pavement deterioration shall be due to a standard Design Load, such that the action of the real load is equivalent to a number of repeated applications of the standard load.
The standard load is defined as a vertical load of 12 t. and contact pressure of 80 t/m2 distributed in a circular area. This load is internationally denominated PAWL (Port Area Wheel Load).

— Establishment of equivalency factors for different equipment and load states, taking into account dynamic effects, wheel proximity, etc.
For each wheel, the equivalency factor between loads shall be :

$$D = \left(\frac{W}{12}\right)^{3.75} \cdot \left(\frac{P}{80}\right)^{1.25}$$

with :

D = Number of equivalent PAWLs.
W = Wheel load in t.
P = Contact pressure in $t/m^2$

For each equipment and load state, the sum of PAWLs equivalent to each wheel of the most loaded side of the equipment shall be taken as the number of PAWLs equivalent to the equipment, increasing first the wheel loads by 1.50, to take into account the existence of dynamic forces.
It shall not be necessary to consider dynamic effects in zones where the movements of the equipment can be predetermined (channelled circulation) and these effects are not foreseen.
For axes with more than one wheel on each side, the calculation of the number of equivalent PAWLs shall be carried out considering the load transmitted by all the wheels on a side as if it were a simple wheel.
For axes in tandem, to take into account the amplification of stresses produced by the proximity of two or more wheels, the loads for each wheel shall be increased for separations between axes less than 4.50 m according to the following factors :

| Separation between wheels (mm) | 500 | 1.000 | 1.500 | 2.000 | 2.500 | 3.000 | 3.500 | 4.000 | 4.500 |
|---|---|---|---|---|---|---|---|---|---|
| Amplification Factor | 1.95 | 1.90 | 1.70 | 1.60 | 1.40 | 1.30 | 1.20 | 1.10 | 1.00 |

Intermediate separations, may be interpolated linearly.

— Estimation of the number N of accumulated applications of the standard load during the analyzed project phase equivalent to the entire equipment spectrum.
For this, the following states and parameters are defined :

- *Load State corresponding to Equipment Critical Deterioration* : An equipment load state where the number of equivalent PAWLs multiplied by their frequency during the analyzed period gives the maximum value. This corresponds to the weight of greatest contribution to the total pavement deterioration.

- *Load State corresponding to Equipment Mean Deterioration* : An equipment load state where the number of PAWLs is equal to the sum of all the equivalent PAWLs, corresponding to each one of the equipment load states, multiplied by their frequency during the analyzed period and divided by 100.

The number of applications ($L_i$), that shall be used in the fatigue equations, shall correspond to the number of movements of the equipment in the load state corresponding to the Critical Deterioration, assuming that its effect is equivalent to the total equipment movements.

The following equation shall be adopted for each equipment type :

$$N_i = L_i \times (\text{n}^{\text{o}} \text{ of PAWLs equivalent to the equipment's critical state})$$

given that :

$$L_i = \frac{(\text{total n}^{\text{o}} \text{ of equipment movements}) \times (\text{n}^{\text{o}} \text{ of PAWLs equivalent to the mean state})}{(\text{n}^{\text{o}} \text{ of PAWLs equivalent to the critical state})}$$

with :

$$N = \Sigma N_i$$

to take into account the combined effect of all the equipment.

Lacking more detailed studies of the cargo handling and transport equipment spectrum that is anticipated to act in a specific zone, their different load states and the frequency with which they loads are repeated in the analyzed zone, or of survey information in equivalent port areas, the values of the number of equivalent PAWLs for each characteristic equipment given in table 3.4.2.3.4.1. may be adopted in order to calculate the pavements in operation areas and maneuvering tracks.

In these cases, for the calculations, the following value of N may be adopted:

$$N = T_m \cdot L \cdot \frac{\alpha}{0.8 \cdot W} \cdot \mathrm{PAWLS}$$

given that :

$T_m$ : Total quantity of cargo handled or foreseen handled in the zones served by the maneuvering tracks, or moved in the analyzed area, in t/year.

L : Design life of the work, in years

$\alpha$ : Cargo handling quota by wheeling ((Total load handled by wheeling)/$T_m$).

W : Maximum load handled by the equipment foreseen as the most characteristic.

PAWLs : Number of PAWLs equivalent to the handling equipment considered the most characteristic. (See table 3.4.2.3.4.1).

— Normally the operation areas and the maneuvering tracks or rails in port zones are quite wide, and therefore, the traffic in these areas is not channeled or delimited.
To take into account this effect in the quantification of accumulated traffic, the number N of accumulated standard loads, obtained by any of the procedures indicated, shall be reduced by means of a reduction factor in function of the track width/equipment width ratio. These reductions are based on the observation of the lateral distribution of handling equipment movements on lanes of different widths.

| $\frac{\text{Zone, track or lane maneuvering width}}{\text{Equipment width}} = r$ | Reduction factor of N |
|---|---|
| $r > 5.50$ | 1/3 |
| $3.00 \leq r \leq 5.50$ | 1/2 |
| $r < 3.00$ | 1 |

| TABLE 3.4.2.3.4.1 NUMBER OF EQUIVALENT STANDARD LOADS FOR EACH CHARACTERISTIC CARGO HANDLING EQUIPMENT IN OPERATION AREAS AND MANEUVERING TRACKS | | | |
|---|---|---|---|
| EQUIPMENT | Nº OF STANDARD LOADS (in PAWLs) | | EQUIPMENT WIDTH (in meters) |
| | Normal Wheeling | With Dynamic Effects | |
| Conventional Heavy Traffic | 0.30 | 1.00 | 3.00 |
| 5 t Fork lift truck | 1.00 | 4.00 | 1.60 |
| 20 t Fork lift truck | 15.00 | 50.00 | 3.00 |
| 40 t Fork lift truck | 40.00 | 120.00 | 4.50 |
| 40 t Sideloader truck | 4.00 | 10.00 | 3.30 |
| Straddlecarrier for 40' containers | 2.50 | 15.00 | 4.00 |
| Mobile cranes (unloaded) | 5.00 | 10.00 | 2.80 |
| Tractor and trailer | 4.50 | 10.00 | 3.20 |
| Portal Crane on wheels | 200.00 | — | — |

b) IN STORAGE AREAS

Zones fundamentally subjected to the actions of static overloads produced by the support elements and systems of the stored cargo.
The design of the pavement shall normally have to be carried out in such a way as to avoid puncturing by permanent deformation of the constituent materials.

Lacking other specific data set by the project engineer or by the port use criteria, the minimum concentrated stage and storage overloads in exterior yard given in table 3.4.2.3.1.4. may be considered as values of the overloads acting upon the pavement.

c) IN SERVICE AREAS

Zones intended for the internal, unchannelled circulation of conventional road traffic (usually not heavy). The pavements can be designed according to urban pavement design criteria.

d) IN ACCESS TRACKS

Zones subject to mostly heavy, channelled, conventional road traffic. The overloads and other design factors shall coincide with those adopted for the design of highway pavement. (See Code 6.1 and 2-1C Pavement Section).

The principal design factor of the pavement shall be, in these cases, the average daily traffic of heavy vehicles (ATD) that is anticipated for the design lane in the year it begins service.

Given that the design life of a pavement considered in the Highway Code is 20 or 30 years, in order to take into account the possibility of other design lives in the pavement design in port zones (Minimum design lives of 25 or 15 years according to table 2.2.1.1),The Heavy Traffic Categories considered in this Code (Table 1 of 6.1 and 2-1C) shall be corrected, multiplying the limits assigned to each traffic category by the factor (20 or 30 years) / (Project design life in years).

Lacking concrete studies of traffic characteristics and evolution, with intensity measurements, loads per axis, and data regarding projected evolution, in access tracks to port zones, the following may be adopted as design traffic per roadway :

$$ADT = \frac{1}{365} \cdot T_m \cdot \frac{\alpha}{W} \cdot \frac{1}{\varepsilon}$$

given that :

ADT : Average daily traffic intensity of heavy vehicles for the roadway of the project in the year service begins.

$T_m$ : Total quantity of cargo handled , or foreseen handled, in the service areas serviced by the access track, in t/year. For this purpose, the cargo moved to and from the interior port areas serviced by the Access tracks shall be considered as the total quantity of cargo handled or foreseen handled. The cargo handled as maritime transit shall not be included.
In some cases, generally regarding access tracks that serve industrial areas, marinas, fishing ports, and passenger areas, the cargo transported by land can be significant in the evaluation of the total quantity of handled cargo.

$\alpha$ : Heavy vehicle quota = (Total Load transported by heavy conventional traffic) / / ($T_m$)).

W : Mean load transported by each heavy loaded vehicle, in t/vehicle.

$\varepsilon$ : Proportion of heavy loaded vehicles = ((Loaded heavy vehicles) / (Total heavy vehicles)

### 3.4.2.3.5 SHIP OPERATION OVERLOADS ($Q_{V5k}$)

■ DEFINITION

Ship operation overloads are the external loads produced by the direct or indirect action of ships upon port structures or installations. They are divided into the following groups:

— Berthing Loads
— Mooring Loads
— Hull Loads
— Slipway and Shipbuilding Berth Loads

■ DETERMINATION

The loads produced by the direct or indirect action of ships upon port structures or installations shall be determined taking into account the following factors :

— Ship dimensions, structural characteristics and movements
— Physical characteristics of the installation : location, accessibility, protection, etc.
— Operational factors : approach conditions to the installation and operation and maneuvering methods, arrival frequency, etc.
— Nature and characteristics of the resistant structure, including compatibility with different types of equipment such as fenders, moorings, bollards, mooring buoys, blocks, slipway carriages, etc.
— Tides, sea level variations, and possibility of modifications in the ship's freeboard.
— Environmental conditions : wind, waves, currents and ice.

a) BERTHING LOADS

Loads generated between a ship and a berthing structure from the first moment of contact until the ship finally comes to a resting position.

The berthing loads transmitted to the resistant structure are divided into the following two groups:

— Impact Loads (R) (Perpendicular to the berthing surface)

— Friction Loads (T) (Parallel to the berthing surface)

$a_1$) IMPACT LOADS (R)

Impact loads, perpendicular to the berthing surface, depend upon the following parameters :

— The kinetic energy developed by the ship during berthing

— The eccentricity of the berthing

— The geometry of the ship

— The geometric configuration of the berthing

— The stress/deformation relationship of the ship, the resistant structure and the fendering system.

The berthing structure, and eventually the fendering system, shall be subjected to impact forces that coincide with the reactions corresponding to the kinetic energy that is transferred or transmitted to the resistant structure or to the complete berthing system, in the eccentricity hypothesis established for the calculation.

— KINETIC ENERGY DEVELOPED BY THE SHIP DURING BERTHING (E)

In cases in which there are reliable and sufficient records of berthing energy correlated to ship displacements, in berths with identical characteristics and similar location as the project (similar wind and current conditions), the design kinetic energy may be determined by statistical procedures. If possible, each record shall be associated with specific environmental and operational conditions.
The observed data shall be fitted to classical statistical distribution functions, in order to extrapolate the available information beyond the data recording period (usually the log-normal distribution). (See section 3.2.3.1). Generally, data corresponding to a range of ship displacements which coincide with those foreseen in the project shall be fitted. Before fitting the data, those values that show significant dispersions shall be eliminated from the calculation (extraordinary impacts due to human error, loss of control, etc).

Taking only normal impacts, an "Impact Distribution" shall be obtained from extreme values which will permit the estimation of Kinetic Energy or Impact of the Characteristic Berthing, by means of extrapolation.
The impact value whose return period equals the design life of the projected structure shall be adopted as Characteristic Impact in Normal Operating Conditions. That is, a maximum admissible Risk of 63% is specified.
In Exceptional Conditions, a lower risk of 30% shall be adopted.

Likewise, physical or mathematical model studies may be carried out to determine the energy developed in berthing, complemented with maneuverability models, taking into account factors such as ship characteristics, the effect of problems due to maritime climate, the influence of local conditions (depth and bathymetry), tug action, and other human factors involved with the control of the ship.

These types of records generally are nonexistent or difficult to obtain and a high degree of occuracy is needed for their extrapolation and validity. As a result, usually, lacking model studies, the kinetic energy developed by the ship during berthing may be calculated assuming that the movement of the ship is a translation without rotation, in a direction practically perpendicular to the berthing surface (this simplification is valid in the majority of practical cases, particularly regarding large ships that berth with the aid of tugs), by means of the equation :

$$E = (1/2 \cdot g) \cdot C_m \cdot \Delta \cdot (V_b)^2$$

given that :

E : Characteristic kinetic energy, in t . m.

$\Delta$ : Design ship displacement (generally full load displacement, in t)

$V_b$ : Perpendicular component of the approach speed of the ship at the moment of impact with the berthing surface in m/s.

$C_m$ : Hydrodynamic mass coefficient (dimensionless)

g : Acceleration of gravity (9.8 $m/s^2$)

— *Design Ship*

The berthing energy shall be determined for the largest displacement ship that could operate in the installation according to its use conditions, assuming that it is fully laden.
Lacking specific use conditions, the project engineer shall take the largest displacement ship, compatible with the local conditions and generic use assigned to the projected work, as the design ship.

Displacement ($\Delta$) is defined as the total weight of the fully laden ship, equal to the weight of the volume of water displaced.

When ships of higher displacement than the original design ship use the installations in exceptional cases, checking of the existing structures for the actions induced by the new ships shall be required. The limiting conditions in which these ships can operate shall be determined so as not to surpass the design actions.

Assuming the installation is designed exclusively for loading operations and that ships arrive in lighted displacement - hypotheses only admissible for solid or liquid bulk, a displacement corresponding to a minimum ballast of 40% of the Dead Weight Tonnage of the ship (DWT) shall be adopted.
Foreseeing possible changes in the use of the port installation and the possibility that the loaded ship might have to return to the berth, the fully loaded ship shall be adopted as the design ship in exceptional conditions.

The most common parameters used to define a ship and specify its size and load capacity are :

- Dead Weight Tonnage (DWT) : Weight corresponding to the maximum load, plus the full fuel weight, in metric tons.
- Gross Register Tonnage (GRT) : Total interior volume or capacity of a ship, measured in Morson tons or register tons. One Morson ton is equal to 100 cubic feet, that is, 2.83 cubic meters.
  This parameter is also denominated as the ship rating.

Some specific types of ships are commonly designated by other parameters. This is the case in liquified natural gas (LNG) and liquid pressurized gas (LPG) carriers, which are designated by their load capacity in cubic meter, or container ships that are designated by their TEU capacity (Number of equivalent twenty foot containers).

Lacking more precise data, using these parameters, the fully loaded displacement ($\Delta$) in tons can be estimated according to the following relationships :

- Bulk Carriers and Multipurpose Ships :
  DWT x (1.20 to 1.30)
  GRT x (2.00)

- Oil Tankers and LNG Carriers :
  DWT x (1.20 to 1.50)

- Liquified Pressurized Gas Carriers (LPG) :
  DWT x (1.60 to 1.80)

- General Cargos :
  DWT x (1.40 to 1.60)
  GRT x (2.00)

- Container Ships :
  DWT x (1.40)

- Ro-ro Ships :
  DWT x (1.80 to 2.20)

- Passenger Ships :
  Liners : GRT x (1.00 to 1.10)
  Ferries : GRT x (1.20 to 2.00)

- Fishing Vessels :
  Coastal : GRT x (2.00 to 2.50)
  Seagoing : GRT x (1.20 to 2.00)

The highest factors correspond to the ships of lowest displacement for each type of ship.

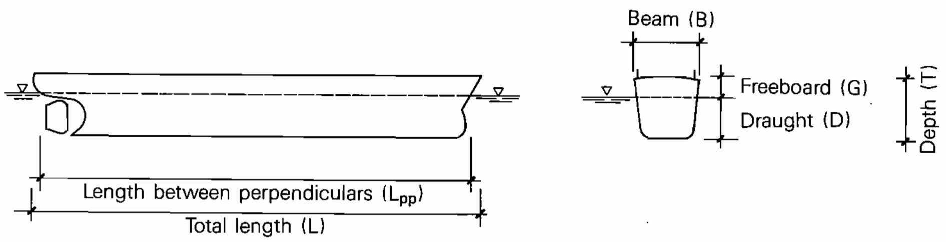
Likewise, the ballast displacement (weight of the ship when it leaves the shipyard, unloaded and without water ballast or fuel) shall be estimated as 15 to 25% of the Fully Loaded Displacement, and the Light Displacement (ballast displacement plus consumables, water and the minimum ballast weight necessary for the ship to navigate and maneuver safely) varies from 30 to 50% of the Fully Loaded Displacement, depending on the environmental conditions.

The dimensions and characteristics of the design ship shall be supplied to the project engineer by the authorities or the owners of the installation in accordance with the projected use. When the ship's dimensions are not clearly known, and lacking more precise information, (e.g. Lloyd's Register) the average dimensions for fully loaded ships given in table 3.4.2.3.5.1 may be used for maritime and port work projects.In the same table, the most characteristic geometric parameters of the ship are given. When the ships are in a partially loaded condition, the project engineer shall use curves or specific tables to obtain the draught and displacement in this condition, or use empirical formulas of recognized validity, or assume that under any load condition the block coefficient of the ship remains constant (displacement / (length between perpendiculars x beams x draught x $\gamma_w$)).

— Berthing Velocity ($V_b$)

The perpendicular component of the approximation velocity of a ship to the berthing surface is the most determining factor for the evaluation of the kinetic energy of the ship during berthing. Its magnitude depends upon, among other parameters, the size of the ship, its load conditions and frequency of arrivals; the local and environmental conditions (location of the structure, wave action, prevailing winds, currents and tides); operation and maneuvering strategies; and the approximation conditions at the installation.

## TABLE 3.4.2.3.5.1 AVERAGE DIMENSIONS OF FULLY LOADED SHIPS




| TYPE OF SHIP | Tonnage | Total Length (m) | Beam (m) | Depth (m) | Maximum draught (m) |
|---|---|---|---|---|---|
| BULK CARRIERS AND MULTIPURPOSE | DWT | | | | |
| | 300.000 | 356 | 57,0 | 28,8 | 22,0 |
| | 250.000 | 348 | 51,8 | 27,0 | 20,4 |
| | 200.000 | 325 | 47,2 | 26,0 | 19,2 |
| | 150.000 | 313 | 44,5 | 24,7 | 18,0 |
| | 100.000 | 275 | 42,0 | 20,3 | 15,1 |
| | 90.000 | 260 | 39,7 | 19,7 | 14,6 |
| | 70.000 | 244 | 37,8 | 18,7 | 13,3 |
| | 50.000 | 222 | 32,6 | 16,8 | 11,9 |
| | 40.000 | 208 | 30,2 | 15,9 | 11,4 |
| | 30.000 | 192 | 27,3 | 14,5 | 10,6 |
| | 20.000 | 170 | 23,7 | 12,9 | 9,6 |
| | 15.000 | 157 | 21,5 | 11,9 | 9,0 |
| | 10.000 | 140 | 18,7 | 10,5 | 8,1 |
| TANKERS | DWT | | | | |
| | 500.000 | 416 | 69,2 | 32,2 | 25,5 |
| | 400.000 | 390 | 65,5 | 28,8 | 22,8 |
| | 300.000 | 368 | 57,0 | 28,4 | 22,4 |
| | 250.000 | 348 | 51,8 | 26,0 | 20,4 |
| | 200.000 | 325 | 47,2 | 24,7 | 19,2 |
| | 150.000 | 291 | 44,2 | 23,0 | 17,9 |
| | 120.000 | 280 | 41,0 | 21,0 | 15,0 |
| | 100.000 | 270 | 39,0 | 19,2 | 14,6 |
| | 80.000 | 255 | 37,5 | 18,7 | 14,0 |
| | 70.000 | 250 | 35,9 | 18,4 | 13,6 |
| | 60.000 | 230 | 34,0 | 17,0 | 13,0 |
| | 50.000 | 226 | 32,1 | 16,1 | 12,5 |
| | 40.000 | 211 | 29,9 | 15,4 | 11,7 |
| | 30.000 | 194 | 27,2 | 14,1 | 10,9 |
| | 20.000 | 171 | 23,8 | 12,4 | 9,8 |
| | 15.000 | 157 | 21,7 | 11,3 | 9,0 |
| | 10.000 | 139 | 19,0 | 9,9 | 8,1 |
| | 5.000 | 102 | 14,7 | 7,6 | 6,9 |
| | 3.000 | 85 | 12,8 | 6,4 | 5,8 |
| | 2.000 | 73 | 11,4 | 5,6 | 5,1 |
| | 1.000 | 57 | 9,4 | 4,5 | 4,2 |
| | 700 | 50 | 8,5 | 4,0 | 3,7 |
| LPG CARRIERS | DWT | | | | |
| | 75.000 | 294 | 43,4 | 26,1 | 12,9 |
| | 50.000 | 257 | 34,8 | 20,7 | 11,5 |
| | 20.000 | 182 | 29,0 | 16,5 | 9,0 |
| | 4.000 | 107 | 17,4 | 9,4 | 6,1 |

| TYPE OF SHIP | Tonnage | Total Length (m) | Beam (m) | Depth (m) | Maximum draught (m) |
|---|---|---|---|---|---|
| LNG CARRIERS | DWT | | | | |
| | 60.000 | 256 | 35,5 | 23,5 | 13,6 |
| | 47.000 | 229 | 36,0 | 21,0 | 12,1 |
| | 40.000 | 206 | 31,4 | 18,6 | 11,3 |
| | 18.000 | 157 | 25,3 | 16,0 | 10,1 |
| | 16.000 | 151 | 25,0 | 14,3 | 9,6 |
| | 5.000 | 106 | 17,0 | 10,0 | 7,4 |
| | 3.000 | 75 | 14,0 | 7,9 | 6,8 |
| GENERAL CARGO SHIPS | DWT | | | | |
| | 50.000 | 232 | 30,0 | 18,4 | 12,7 |
| | 40.000 | 217 | 28,3 | 17,2 | 11,9 |
| | 30.000 | 199 | 26,1 | 15,7 | 11,0 |
| | 20.000 | 177 | 23,4 | 13,8 | 10,0 |
| | 15.000 | 162 | 21,7 | 12,7 | 9,1 |
| | 10.000 | 144 | 19,4 | 11,2 | 8,2 |
| | 9.000 | 139 | 18,9 | 10,8 | 8,0 |
| | 8.000 | 135 | 18,3 | 10,4 | 7,8 |
| | 7.000 | 129 | 17,6 | 10,0 | 7,5 |
| | 6.000 | 124 | 16,9 | 9,5 | 7,2 |
| | 5.000 | 103 | 15,4 | 8,4 | 6,8 |
| | 4.000 | 95 | 14,4 | 7,8 | 6,4 |
| | 3.000 | 86 | 13,2 | 10,5 | 8,1 |
| | 2.000 | 74 | 11,7 | 6,3 | 5,1 |
| | 1.000 | 58 | 9,5 | 5,1 | 4,2 |
| | 700 | 51 | 8,5 | 4,6 | 3,8 |
| CONTAINER SHIPS | DWT | | | | |
| | 50.000 | 290 | 32,4 | 24,2 | 13,0 |
| | 42.000 | 285 | 32,3 | 22,4 | 12,0 |
| | 36.000 | 270 | 31,8 | 21,4 | 11,7 |
| | 30.000 | 228 | 31,0 | 20,3 | 11,3 |
| | 25.000 | 212 | 30,0 | 19,2 | 10,7 |
| | 20.000 | 198 | 28,7 | 17,5 | 10,0 |
| | 15.000 | 180 | 26,5 | 15,6 | 9,0 |
| | 10.000 | 159 | 23,5 | 13,6 | 8,0 |
| | 7.000 | 143 | 19,0 | 11,0 | 6,5 |
| RO-RO SHIPS | DWT | | | | |
| | 20.000 | 205 | 30,0 | – | 9,5 |
| | 15.000 | 190 | 27,0 | – | 8,3 |
| | 10.000 | 170 | 23,0 | – | 7,0 |
| | 7.500 | 155 | 21,5 | – | 6,4 |
| | 5.000 | 135 | 20,0 | – | 5,5 |
| | 2.500 | 105 | 18,0 | – | 5,0 |

## TABLE 3.4.2.3.5.1 (Continued)

| TYPE OF SHIP | | Tonnage | Total Length (m) | Beam (m) | Depth (m) | Maximum draught (m) |
|---|---|---|---|---|---|---|
| PASSENGER SHIPS | LINERS | GRT | | | | |
| PASSENGER SHIPS | LINERS | 50.000 | 291 | 31,2 | 18,0 | 10,5 |
| PASSENGER SHIPS | LINERS | 40.000 | 260 | 29,7 | 17,5 | 10,2 |
| PASSENGER SHIPS | LINERS | 30.000 | 223 | 28,2 | 17,0 | 10,0 |
| PASSENGER SHIPS | LINERS | 20.000 | 197 | 25,1 | 15,1 | 9,2 |
| PASSENGER SHIPS | LINERS | 15.000 | 181 | 23,1 | 13,9 | 8,8 |
| PASSENGER SHIPS | LINERS | 10.000 | 160 | 20,6 | 12,3 | 8,2 |
| PASSENGER SHIPS | LINERS | 9.000 | 155 | 20,0 | 12,0 | 8,0 |
| PASSENGER SHIPS | LINERS | 8.000 | 150 | 19,3 | 11,6 | 7,8 |
| PASSENGER SHIPS | LINERS | 7.000 | 144 | 18,6 | 11,1 | 7,7 |
| PASSENGER SHIPS | LINERS | 6.000 | 138 | 17,8 | 10,6 | 7,4 |
| PASSENGER SHIPS | LINERS | 5.000 | 135 | 17,2 | 8,4 | 6,0 |
| PASSENGER SHIPS | LINERS | 4.000 | 123 | 16,3 | 7,8 | 5,6 |
| PASSENGER SHIPS | LINERS | 3.000 | 109 | 15,3 | 7,1 | 5,1 |
| PASSENGER SHIPS | LINERS | 2.000 | 92 | 13,9 | 6,2 | 4,5 |
| PASSENGER SHIPS | LINERS | 1.000 | 68 | 11,9 | 5,0 | 3,6 |
| PASSENGER SHIPS | LINERS | 500 | 51 | 10,2 | 4,0 | 2,9 |
| PASSENGER SHIPS | FERRIES | GRT | | | | |
| PASSENGER SHIPS | FERRIES | 13.000 | 195 | 24,0 | 16,1 | 6,7 |
| PASSENGER SHIPS | FERRIES | 10.000 | 168 | 24,0 | 14,7 | 6,5 |
| PASSENGER SHIPS | FERRIES | 8.000 | 155 | 21,8 | 13,2 | 6,1 |
| PASSENGER SHIPS | FERRIES | 6.000 | 138 | 21,4 | 12,7 | 5,9 |
| PASSENGER SHIPS | FERRIES | 4.000 | 122 | 20,0 | 11,2 | 5,3 |
| PASSENGER SHIPS | FERRIES | 3.000 | 105 | 17,7 | 10,5 | 5,0 |
| PASSENGER SHIPS | FERRIES | 2.000 | 90 | 16,2 | 9,8 | 4,3 |
| PASSENGER SHIPS | FERRIES | 1.000 | 75 | 13,4 | 5,0 | 4,0 |
| FISHING VESSELS | | GRT | | | | |
| FISHING VESSELS | | 2.500 | 90 | 14,0 | 6,8 | 5,9 |
| FISHING VESSELS | | 2.000 | 85 | 13,0 | 6,4 | 5,6 |
| FISHING VESSELS | | 1.500 | 80 | 12,0 | 6,0 | 5,3 |
| FISHING VESSELS | | 1.000 | 75 | 11,0 | 5,7 | 5,0 |
| FISHING VESSELS | | 800 | 70 | 10,5 | 5,4 | 4,8 |
| FISHING VESSELS | | 600 | 65 | 10,0 | 5,1 | 4,5 |
| FISHING VESSELS | | 400 | 55 | 8,5 | 4,5 | 4,0 |
| FISHING VESSELS | | 200 | 40 | 7,0 | 4,0 | 3,5 |

| TYPE OF SHIP | | Tonnage | Total Length (m) | Beam (m) | Depth (m) | Maximum draught (m) |
|---|---|---|---|---|---|---|
| NAVAL SHIPS | | DWT | | | | |
| NAVAL SHIPS | a | 16.000 | 172 | 23,0 | – | 8,2 |
| NAVAL SHIPS | b | 15.000 | 195 | 24,0 | – | 9,0 |
| NAVAL SHIPS | c | 6.000 | 117 | 16,8 | – | 3,7 |
| NAVAL SHIPS | d | 4.000 | 134 | 14,3 | – | 7,9 |
| NAVAL SHIPS | e | 3.500 | 120 | 12,5 | – | 5,5 |
| NAVAL SHIPS | f | 1.500 | 90 | 9,3 | – | 5,2 |
| NAVAL SHIPS | g | 1.500 | 68 | 6,8 | – | 5,4 |
| NAVAL SHIPS | h | 1.400 | 89 | 10,5 | – | 3,5 |
| NAVAL SHIPS | i | 750 | 52,3 | 10,4 | – | 4,2 |
| NAVAL SHIPS | j | 400 | 58 | 7,6 | – | 2,6 |
| NAVAL SHIPS | (*) | 130 | 36 | 5,8 | – | 2,5 |
| NAVAL SHIPS | | 85 | 30 | 5,3 | – | 1,5 |
| PLEASURE CRAFT | MOTOR CRAFT | t | | | | |
| PLEASURE CRAFT | MOTOR CRAFT | 50,0 | 24,0 | 5,5 | – | 3,3 |
| PLEASURE CRAFT | MOTOR CRAFT | 35,0 | 21,0 | 5,0 | – | 3,0 |
| PLEASURE CRAFT | MOTOR CRAFT | 27,0 | 18,0 | 4,4 | – | 2,7 |
| PLEASURE CRAFT | MOTOR CRAFT | 16,5 | 15,0 | 4,0 | – | 2,3 |
| PLEASURE CRAFT | MOTOR CRAFT | 6,5 | 12,0 | 3,4 | – | 1,8 |
| PLEASURE CRAFT | MOTOR CRAFT | 4,0 | 9,0 | 2,7 | – | 1,5 |
| PLEASURE CRAFT | MOTOR CRAFT | 1,3 | 6,0 | 2,1 | – | 1,0 |
| PLEASURE CRAFT | SAILING CRAFT | t | | | | |
| PLEASURE CRAFT | SAILING CRAFT | 60,0 | 24,0 | 4,6 | – | 3,6 |
| PLEASURE CRAFT | SAILING CRAFT | 40,0 | 21,0 | 4,3 | – | 3,0 |
| PLEASURE CRAFT | SAILING CRAFT | 22,0 | 18,0 | 4,0 | – | 2,7 |
| PLEASURE CRAFT | SAILING CRAFT | 13,0 | 15,0 | 3,7 | – | 2,4 |
| PLEASURE CRAFT | SAILING CRAFT | 10,0 | 12,0 | 3,5 | – | 2,1 |
| PLEASURE CRAFT | SAILING CRAFT | 3,5 | 9,0 | 3,3 | – | 1,8 |
| PLEASURE CRAFT | SAILING CRAFT | 1,5 | 6,0 | 2,4 | – | 1,5 |

NOTES :

(*)

a : Attack carrier
b : Aircraft carrier
c : Landing ship
d : Guided missile frigate
e : Destroyer
f : Fast frigate
g : Submarine
h : Corvette
i : Mine sweeper
j : Patrol craft

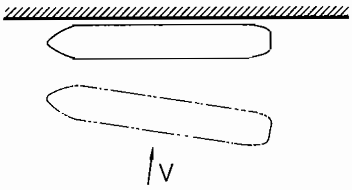
(1) The common dimensions of the ships given in tables shall vary, depending on the country of origin and the shipyard. These dimensions shall vary by a total of ± 10% in extreme cases.

(2) The length between perpendiculars may be approximated as 95% of the total length.

(3) Lacking other data, the ship displacement shall be calculated as the product of the length between perpendiculars, the beam, the maximum draught, the specific weight of the water, and the block coefficient.
This coefficient shall vary from 0.8 to 0.6 for general cargo ships and bulk carriers; 0.85 for tankers; 0.55 to 0.65 for ro-ro ships and ferries; 0.3 to 0.5 for naval ships and 0.3 to 0.4 for fishing vessels.

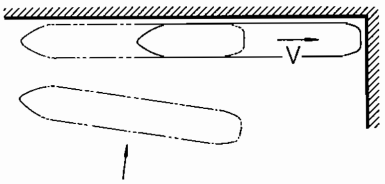
The consideration of the operation and maneuvering methods, and in general, of the applicable approximation conditions of the ship to the port installation shall be analyzed for each location. This analysis shall be made with the involvement of the Client or maritime and port Authorities, taking into account the local practice and experience for each type of ship and environmental condition, or on the basis of specific maneuverability studies or models.
The use or availability of tugs, shall be specially emphasized since when an insufficient number of tugs is used or they are not used at all, the berthing velocities shall be increased.

Normally, large displacement ships (>10.000 t) are stopped some 10 or 20 m from the berth in a parallel position, making the berthing maneuver in the direction practically perpendicular to the berthing line with the help of tugs.
This method produces velocities of approximately 0.10 to 0.40 m/s in normal operating conditions.
For Ro-ro ships, ferries, and in general for ships with low displacement, (<10,000 t) the berthing maneuver is usually executed by the ship using her own means. In these cases, in normal operating conditions, the velocity in the direction of the movement is approximately:



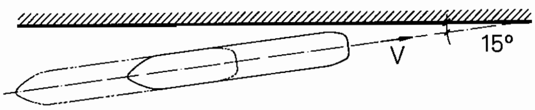

ALONGSIDE BERTHING WITH TRANSVERSE APPROACH.

V = 0.4 a 0.9 m/s.



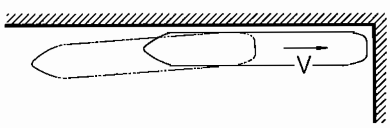

FRONTAL BERTHING. STEADY SHIP IN FRONT OF THE BERTHING PLACE

V = 0.15 m/s




ALONGSIDE BERTHING WITH PREPONDERANT LONGITUDINAL APPROACH

V = 2 a 3 m/s




FRONTAL BERTHING WITH PREPONDERANT LONGITUDINAL APPROACH

V = =.5 a 1 m/s

For preponderant longitudinal approximation and alongside berthing, the velocity vector shall be considered to form an angle of 15° with the berth front. Therefore, in the kinetic energy equation $V \cdot \sin 15°$ shall be considered as $V_b$.
For frontal, bow or stern berthing with longitudinal approximation, $V_b = V \cdot \cos 15°$ shall be adopted.
Lower approximation angles shall be admitted if the geometry of the berthing restricts them.

The design berthing velocity shall be set, preferably using statistical data obtained in berths with similar characteristics and similar environmental conditions, for the operating limit environmental conditions defined in the use criteria of the port installation. In those cases in which the established use criteria permits ship berthing at all times and in all conditions, the extreme values of the

environmental actions associated with the maximum admissible risk shall be adopted as operating condition limits. The projected reductions for the simultaneous consideration of several environmental actions ($\psi_0 . Q_{Mk}$) shall be taken into account, as long as the presence of the moored ship is considered in those conditions. In the opposite case, the operating condition limits for the berthing shall coincide with the mooring operating limits (see paragraph b of this section).The berthing operation conditions shall never be more unfavorable than the adopted mooring conditions.
Lacking specific operating criteria, the Project Engineer shall adopt that which corresponds to ship berthing, at all times and in all conditions, as operating condition limits.

When there are no available records, it is recommended to use the values in tables 3.4.2.3.5.2 and 3.4.2.3.5.3 as design berthing velocities. These values are valid for alongside berthing with transverse approximation in a direction practically perpendicular to the berthing line and for fully loaded ships, and vary according to the design ship, the use or not of tugs, the approximation conditions and the environmental conditions adopted as operating limits.
For frontal bow or stern berthing, or for longitudinal approximation, lacking records, the values of $V_b$ for low-displacement ships mentioned above may be applied. In the event that the design ship does not correspond to the fully loaded condition, berthing velocities not lower than 120% of those given in the tables for the same ship, fully loaded, shall be adopted.
Whenever high ship arrival frequencies are foreseen, (e.g. in a ferry terminal) berthing velocities higher than those projected in the tables (increases of 15 to 20%) shall be considered.

— *Hydrodynamic Mass Coefficient ($C_m$)*

The Hydrodynamic mass coefficient takes into account the effect produced by the mass of water that is moved, together with the ship during berthing, which produces an effective increase in the mass to be considered in the evaluation of the berthing energy.

The hydrodynamic mass coefficient is defined as the relation between the total mass of the system (Ship mass + Moved water mass) and the ship mass.

$$C_m = (M_d + M_w) / M_d$$

For ship berthing with low-velocity transverse approximation in shallow or limited depth areas, the coefficient $C_m$ basically depends on the ship dimensions and configuration under the surface of the water (mainly draught/beam ratio), the underkeel clearance, the current direction in the berthing area, the berthing velocity and the influence of the type and stiffness of the berthing structure in the deceleration of the ship.

Except for very small underkeel clearances, ($<0.1 \cdot D$) or for extremely low berthing velocities ($<0.08$ m/s) the coefficient $C_m$ may be estimated using the formula :

$$C_m = 1 + 2 \cdot (D / B)$$

(Vasco Costa 1964)

given that :

D : Design ship's draught

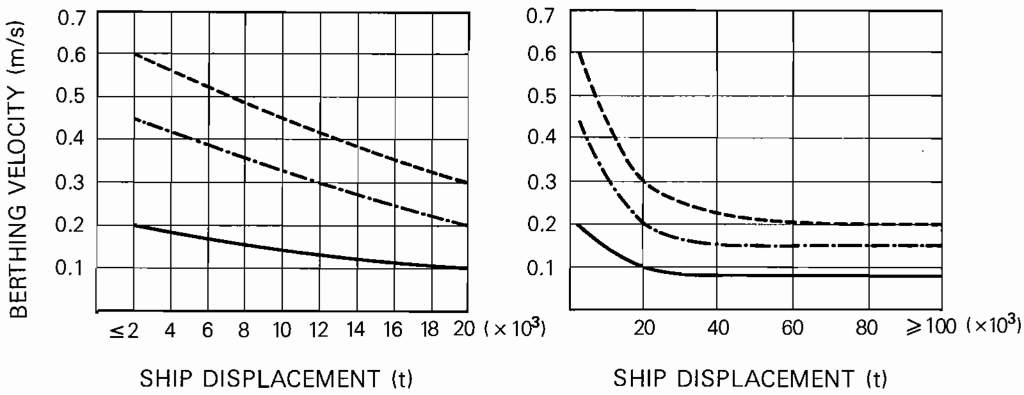
B : Design ship's beam

other internationally recognized formulas such as Saurin (1963), Rupert (1976), Giraudet (1966) and Ueda (1981) can also be used.

Typical values of $C_m$ shall be between 1.30 and 2.00

When the depth in a berthing area is high, with underkeel clearances approximately equal to or greater than $1.5 \cdot D$, $C_m = 1.5$ may be used if other data is not available.

TABLE 3.4.2.3.5.2 **BERTHING VELOCITIES FOR ALONGSIDE BERTHING WITH PREPONDERANT TRANSVERSE APPROACH IN A DIRECTION PRACTICALLY PERPENDICULAR TO THE BERTH. WITH TUG ASSISTANCE**




**LEGEND :**

ENVIRONMENTAL OPERATING CONDITIONS :

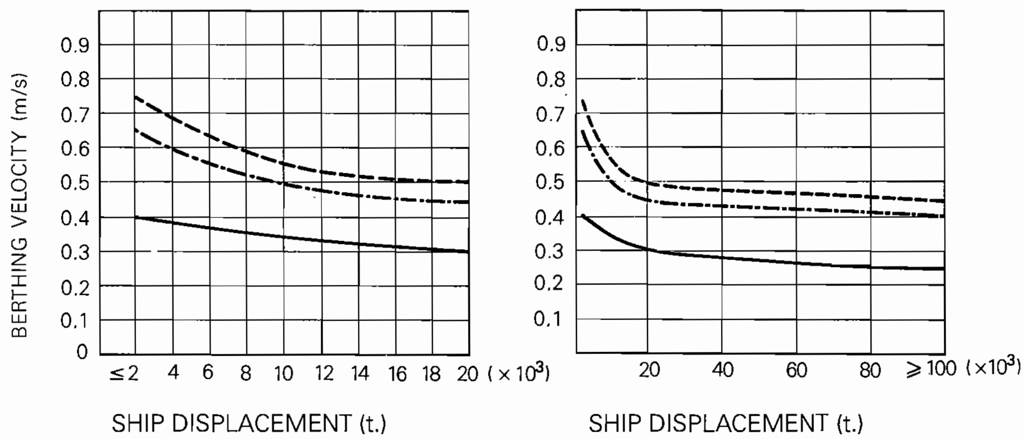
| | | |
|---|---|---|
| ---- | Very unfavorable conditions : wind, waves, and/or strong currents. | $V_{V1min}$ ≥ 17 m/s (60 Km/h)<br>$H_s$ ≥ 2 m for Δ ≥ 3.000 t.<br>≥ 1 m for Δ < 3.000 t.<br>$V_{C1min}$ ≥ 1 m/s (2 knots) |
| —·— | Intermediate conditions : strong wind, and moderate waves and currents. | $V_{V1min}$ ≥ 17 m/s (60 Km/h)<br>$H_s$ ≥ 2 m for Δ ≥ 3.000 t.<br>≥ 1 m for Δ < 3.000 t.<br>$V_{C1min}$ ≥ 1 m/s (2 knots) |
| —— | Favorable conditions : moderate wind, waves and currents. | $V_{V1min}$ ≥ 17 m/s (60 Km/h)<br>$H_s$ ≥ 2 m for Δ ≥ 3.000 t.<br>≥ 1 m for Δ < 3.000 t.<br>$V_{C1min}$ ≥ 1 m/s (2 knots) |

$V_{V1min}$ : Mean wind velocity at a height of 10 meters for one minute long gusts.
$H_s$ : Significant wave height at the depth of the project location
$V_{C1min}$ : Mean horizontal current velocity at a depth of 50% of the ship's draught, in a 1 minute interval.

**NOTES :**

The values assigned in the tables are valid for normal approach conditions (current practically parallel to the berth front). For difficult conditions (currents in a direction different from the berth front) increases of 25% may be adopted for equal environmental conditions.

| TABLE 3.4.2.3.5.3 | BERTHING VELOCITIES FOR ALONGSIDE BERTHING WITH PREPONDERANT TRANSVERSE APPROACH IN A DIRECTION PRACTICALLY PERPENDICULAR TO THE BERTH. WITHOUT TUG ASSISTANCE |
|---|---|




**LEGEND AND NOTES :** Same as in 3.4.2.3.5.2

For ship berthing with longitudinal approximation, and lacking specific studies, $C_m = 1$ shall be taken for bow or stern berthing. For lateral berthing, the same formula as for transversal approximation shall be used.

— *ENERGY ABSORBED BY THE BERTHING SYSTEM ($E_f$)*

The kinetic energy developed by the ship (E) during berthing shall not be completely transferred to the berthing system (structure + fenders). Only a part of the total developed energy will be absorbed by the berthing system.
The energy absorbed by the berthing system from the moment of initial contact with the ship until the system reaches maximum deformation shall equal the difference of the kinetic energies of the ship at both moments. To obtain this value, the following simplifications shall be admitted :

- The approximation movement of the ship to the port installation is a simple translation, without rotation.
- At the moment of maximum deformation, there is no relative sliding at the point of berth/ship contact. Only a ship rotation around the contact point is produced.
- The actions produced by tugs, wind, currents, etc. are negligible in comparison with the reaction of the berthing system (high impacts).

With these hypotheses, the energy absorbed by the berthing system shall depend upon the following factors : the kinetic energy developed by the ship, the eccentricity of berthing, the ship's geometry, the geometric configuration of the berth and the stress/deformation relationships of the ship, the resistant structure and the fender system. The following equation applies :

$$E_f = f \cdot E$$

(Saurin and Risselada 1963)

given that :

$E_f$ : Kinetic energy absorbed by the berthing system
$E$ : Kinetic energy developed by the ship during berthing
$f$ : $C_e \cdot C_g \cdot C_c \cdot C_s$
$C_e$ : Eccentricity coefficient
$C_g$ : Ship's geometric coefficient
$C_c$ : Berth's configuration coefficient
$C_s$ : Berthing system's stiffness coefficient

Each one of the factors is analyzed here.

— *Berthing Eccentricity*

When the point of impact of a ship upon a berthing system does not coincide with the center of gravity of the ship, the kinetic energy developed by the ship is not completely transmitted to the berthing system. If the effects due to the ship's and berth's configuration and characteristics are not taken into account, the fraction of kinetic energy transferred to the berthing system shall be defined by the eccentricity coefficient ($C_e$).
This coefficient shall basically be a function of the ship's geometric characteristics and of the berthing approximation conditions.

For ship berthing by preponderant transversal approximation, the eccentricity coefficient may be determined according to table 3.4.2.3.5.4. assuming that the simplified hypotheses relative to the ship movement before and after impact (given in this paragraph) are valid.

For ship berthing by direct longitudinal approximation, the eccentricity coefficient shall be determined according to table 3.4.2.3.5.5.

— *The geometric configuration of the ship*

The curvature of the ship and of the fendering system at the point of contact influence the energy absorbed by fendering system. The proportion of energy absorbed is determined by the ship's geometric coefficient ($C_g$).

A value of $C_g = 0.95$ is recommended when the point of impact is located at the curved part of the ship and $C_g = 1$ when it is located at the straight part.

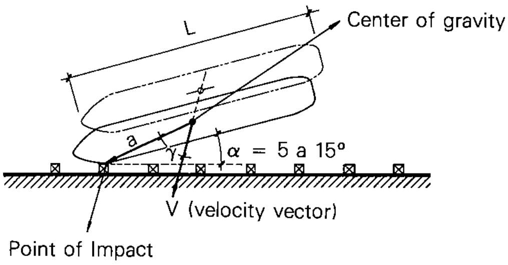
— *The Berthing configuration*

The geometric configuration of the berthing has a significant influence on the consideration of the proportion of kinetic energy developed by the ship that is absorbed by the cushion of water between the ship's hull and the berthing structure. The fraction of kinetic energy absorbed by the berthing system is obtained by the berthing configuration coefficient ($C_c$).

The value of $C_c$ shall depend upon the type of berthing structure : diaphanous (discontinuous) (e.g. pile platform) or solid (continuous) (e.g. vertical quay made of caissons) and also upon the free distance between the ship's hull and the fenders; the angle and method of approximation; the geometric configuration of the ship's hull; and the underkeel clearance.

Lacking other data, it is recommended that the values given in table 3.4.2.3.5.6 be taken as $C_c$ values.

TABLE 3.4.2.3.5.4 ECCENTRICITY COEFFICIENT FOR BERTHING MANEUVER WITH TRANSVERSE APPROACH




$$C_e = \frac{k^2 + a^2 \cdot \cos^2\gamma}{k^2 + a^2}$$

given that :

— k: Ship gyration radius (m)
It may be approximated by the function $k = (0.19 \cdot C_b + 0.11) \cdot L$, for :

- $C_b$ = Ship block coefficient.

$$- C_b = \frac{\text{Displacement}}{\text{Length between perp.} \cdot \text{Beam} \cdot \text{Draught} \cdot \gamma_w} = \frac{\Delta}{L_{pp} \cdot B \cdot D \cdot \gamma_w}$$

Values of $C_b$ between 0.3 and 0.9 are customary

- L = Ship's length (m)

Generally, k shall have values between 0.20 and 0.25 L.

— a : Distance between impact point and ship's center of gravity (m)
It may be assumed that the center of gravity of the ship coincides with the center of the length.
The place where the point of impact is produced shall basically depend upon the approach method and the local conditions where it is produced. To make the impacts as soft as possible, the ship should approach in a way that creates the greatest values for the distance (a) and angle (γ).
Generally, the point of contact is produced in the proximities of the bow or stern, depending on the approach angle α (generally reaching 5° to 15° for berthing without tugs and 7 to 10° for berthing with tugs), and the shape of the ship's hull and the fender.

The following approximate values of "a" may be used in the calculations :

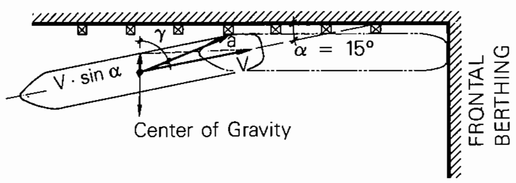
- For berthing on continuous fender system : a = 0.25 L.
- For berthing on isolated fender structures (e.g. berthing dolphin) : it shall be assumed that the ship berths with an eccentricity (x) with respect to the center of the berth in the direction of the point of impact with a value of 0.10 L, with a minimum value of 10 m and maximum of 15 m (measured parallel to the berthing line)

—γ : Angle between the line connecting the point of impact and the ship's center of gravity and the vector velocity.

In normal conditions, γ = 70° – 80° shall be adopted.

Generally, values of $C_e$ between 0.55 and 0.60 for berthing with continuous fender systems and between 0.70 and 0.80 for berthing on isolated fender structures shall be obtained.

TABLE 3.4.2.3.5.5 ECCENTRICITY COEFFICIENT FOR BERTHING MANEUVER WITH DIRECT LONGITUDINAL APPROACH (RO-RO SHIPS AND FERRIES)




— ALONGSIDE OR GUIDED BERTHING LINE

$C_e$ shall be determined according to the formula given in table 3.4.2.3.5.4, considering $\alpha$ = 15° as a minimum approach angle (maximum 20°), except if the geometric disposition of the berth imposes a lower one. The angle $\gamma$ shall be considered as the one formed between the line connecting the impact point and the ship's center of gravity and the component of the velocity vector normal to the berth. $\gamma$ shall depend on the geometric configuration of the hull of the ship in the impact zone. Lacking other data, $\gamma$ = 70° shall be used.

— FRONTAL BERTHING

$C_e$ = 1.00 shall be adopted.

NOTE :

The excentricity coefficient for the ship berthing by direct longitudinal approach to a frontal berth after coming to rest in an alongside or guided berthing shall likewise be taken as equal to 1.00

— *The stiffness of the berthing system*

The relationship between the berthing system (structure + fender) stiffness and the ship stiffness is of fundamental importance in the determination of the fraction of kinetic energy transferred to the berthing system and that absorbed by the deformations of the ship's hull.
Generally, the energy transferred to the berthing system must by large, since if it were small, the large deformations in the ship's hull could lead to its failure.

The proportion of kinetic energy absorbed by the berthing system is determined by the berthing systems's rigidity coefficient ($C_s$).

In the case of very stiff berthing systems, such as those made of wooden fenders fixed along the length of the stiff structure, values as low as $C_s$ = 0.50 have been adopted. For very flexible berthing structures, $C_s$ = 1.00 is customarily adopted.

Generally, in order to guarantee the ship's safety, berthing systems capable of absorbing 90% of the berthing energy are designed.In these conditions, and in the absence of better information, $C_s$ = 1.00 is recommended for flexible berthing systems and between 0.90 and 1.00 for stiff systems.

| TABLE 3.4.2.3.5.6 SUGGESTED VALUES FOR THE CONFIGURATION COEFFICIENT OF A BERTH ($C_c$) | | | | |
|---|---|---|---|---|
| SHIP'S APPROACH METHOD | | STRUCTURE TYPE | | |
| | | Diaphanous | Semi-solid | Solid |
| Preponderantly Transverse | | 1.00 | 0.90 | 0.80 |
| Preponderantly Longitudinal | By Bow or Stern | 1.00 | 1.00 | 1.00 |
| | Alongside Berthing | 1.00 | 0.90 | 0.80 |

**NOTES :**

Berthing in angular zones or in corners shall be considered as diaphanous structure type (e.g. in extremes of solid gravity pier)

Systems in which the design ship's impact produces deformations greater than 0.15 m in usual berthing conditions shall be considered flexible fendering systems. On the contrary, if the deformations are less than 0.15 m, the fendering systems shall be considered stiff.

— *IMPACT LOAD (R)*

The impact load or reactions to which a berthing system is subjected shall be a function of the kinetic energy absorbed by the berthing system ($E_f$) and of its deformation characteristics.

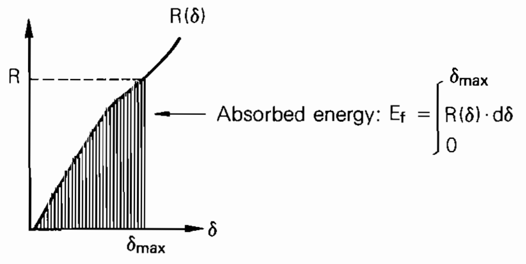
Because of the large variety of possible berthing systems, the following types shall be distinguished for the determination of berthing loads :

— *Fixed rigid structure with a compressible fender (e.g. Gravity dock with flexible fenders)*

The fender shall absorb all the energy absorbed by the berthing system. The energy absorbed by the structure shall be considered negligible.

The berthing force to introduce in the calculation shall be obtained from the fender's behavior curves ( Reaction/ deformation curve, or absorbed energy/ maximum deformation curve).

The energy absorbed by the fender shall coincide with the area between the reaction/ deformation curve and the deformation axis from zero deformation up to maximum deformation.



— *Fixed flexible structure without fendering or with very rigid fendering. (e.g. Berthing dolphin with simple wooden fenders)*

In this case, all the kinetic energy transferred to the berthing system shall be absorbed by the deformation of the resistant structure (translation + rotation).

In the usual case when the deformation of the structure is a simple translation, proportional to the applied load, and the impact is applied in the center of gravity of the structure, the energy absorbed by the flexible structure shall be:

$$E_f = (1/2) \cdot \delta \cdot R$$

given that :

$\delta$ : Maximum deformation of the structure.
R : The impact force that produces the maximum deformation.

If the reaction is not proportional to the deformation, the absorbed energy shall be expressed by the area between the deformation axis and the reaction/deformation curve.

For lineal structures that can be considered indefinite, the hypothesis that a simple translation occurs may be adopted; with the impact load resisted by a length of structure equal to :

- Without fenders or with continuous fenders :

$$l + 2b$$

given that :

l : Length of contact between ship and structure or ship and fenders. Lacking other data, the following is recommended :
$l = 0.25 \cdot L \not> 7$ m, for ships $\leq$ 10,000 DWT.
$l = 14$ m for ships > 10,000 DWT.
b : Width of the structure.

- With isolated fenders :

$$l' + 2b$$

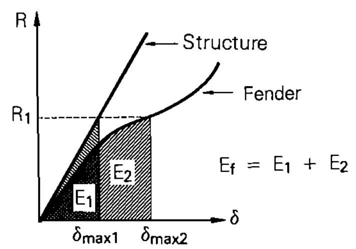
given that :

l' : Length of contact between fender and structure.
b . Width of the structure

If the lineal structure is narrow, (e.g. sheet piling), other specific distribution conditions shall be defined depending on the structural type and characteristics.

— *Fixed flexible structure with compressible fendering (e.g. Berthing dolphin with flexible fenders)*

The total energy transferred to the berthing system shall be absorbed by both the fender and the resistant structure.
The impact load or reaction shall be taken as that for which the sum of the energy absorbed by the fenders and the structure corresponding to this reaction is equal to the total kinetic energy transferred. This hypothesis is valid when considering that the maximum deformation of the fender occurs at the same time as the maximum deformation of the structure.



This simplification shall be considered sufficient for structures where the impact practically coincides with its center of gravity or for lineal structures where the point of impact is sufficiently removed from the ends. (See in the prior paragraph the length of lineal structure that resists the impact load). For other types of structures, other specific methods of dynamic analysis (multiple dampers, etc) shall be adopted.

If the energy absorption capacity of the fender is used up, the resistant structure shall continue absorbing energy, behaving at this point like a flexible structure without fendering.

— *Floating structures*

The impact load produced by the ship berthing on a floating structure shall basically depend upon the horizontal movements of the ship relative to the structure, and the design ship displacement.

In these cases, given the different movement and deformation conditions of the resistant structure, the proposed methodology for the determination of berthing loads on fixed structures shall not be applicable. In the absence of a more precise analysis, the berthing force shall be estimated by the following equation:

$$R = (26 \cdot (H_s/(g \cdot T_s^2)) + 0.05) \cdot \Delta$$

given that :

R : Impact load, in t.
$H_s$ : Operating limit significant wave height, in m.
$T_s$ : Operating limit significant wave period, in s.
g : Acceleration of gravity (9.8 m/s$^2$).
$\Delta$ : Design ship displacement, in t.

— *IMPACT LOAD DISTRIBUTION CRITERIA*

The impact forces shall be distributed taking into account the following

— The contact pressures on the ship's hull shall be maintained within admissible limits.
The maximum admissible contact pressure between the ship's hull and the fendering system depend on several factors : ship's type and size,nature of the fendering system (rigid or flexible) and position of the contact area relative to the ship's structure.
Lacking other data, as a general rule, a value equal to the depth of the fully loaded ship (expressed in m) shall be adopted as the maximum admissible contact pressure (expressed in t/m$^2$) for the design ship. For large ships, usual values shall be between 15 and 25 t/m$^2$.

— If possible, direct contact between the berthing structure and the ship's hull shall be avoided.

— The area of contact between the hull and the fendering system basically depends on the geometry of the hull in the contact zone, the berthing angle and the fender system's characteristics and type (hinged fenders, etc).

For continuous fendering systems, and lacking more precise information, it shall be adopted that the impact load is distributed in a rectangular area whose larger dimension coincides with the ship/fender system contact length (l). This length shall be approximated to :

| | |
|---|---|
| $l = 0.25 \cdot L \not> 7$ m, | for ships ≤ 10,000 TPM |
| $l = 14$ m, | for ships > 10,000 TPM |

Contact areas greater than 5 m$^2$ shall never be considered.

— For the consideration of local effects in the berthing structure, the impact load shall be distributed in the fender/structure contact area.

— *WORK HYPOTHESIS*

Two levels of energy transferred to the berthing system shall be considered for the design of fender systems and berthing structures :

— *Energy transferred in Normal Operating Conditions*

Based in the berthing conditions foreseen in the project and quantified according to the stated methodology.

— *Energy transferred in Exceptional Conditions*

Accidental situations : ship or tugs mechanical failures, mooring line breakage, sudden environmental condition changes, human error, etc. can give rise to abnormal impacts. The energy transferred to the berthing system in these conditions shall be twice that considered for normal operating condition calculations.

In order to guarantee the safety of the resistant structure, the fender systems shall be designed such that their ultimate energy absorption capacity before breaking coincides with the energy considered for exceptional conditions.

Due to the non-linearity of the relationship (transferred energy/deformation-reaction) of the berthing system the resistant structure shall be designed to resist the impact reaction or force produced in both considered work hypotheses.

$a_2$) FRICTION LOADS (T)

Loads parallel to the fender system's surface that act vertically and horizontally in the contact zone between the ship's hull and the berthing system.
These forces are the tangential components induced by the obliquity of the impact and the geometry of the ship in the contact zone. For the determination of these forces, it shall be considered that at the moment of maximum deformation, at the berthing/ship point of contact, there is no relative sliding. Therefore, only a rotation of the ship around the point of contact occurs. Moreover, it shall be considered that the fendering system is subject to deformation only in the direction perpendicular to the berthing surface, assuming that the fenders and the structure are rigid in the transversal direction.

In these conditions, the maximum value, either vertical or horizontal, shall be :

$$T = \mu \cdot R$$

given that :

R : Design Impact Load

$\mu$ : Friction coefficient between the fender system's surface and the ship's hull in the contact area.

In the absence of better information, the values given in table 3.4.2.3.5.7 shall be adopted for steel/other material contact, for flat, smooth, unrusted surfaces. In other cases, the project engineer shall adopt higher values.

b) MOORING LOADS

Mooring loads are those loads imposed on a structure by a moored ship through contact between the ship and the structure or the fendering system or through the tensioned mooring lines. Loads due to moored ship maneuvers shall be considered mooring loads, especially the release or breakage of loaded mooring ropes and their pretensioning during the berthing maneuver.

Mooring loads are caused by external actions, mainly environmental, that are exerted upon a moored ship. Their magnitude and distribution shall primarily depend upon the ship/mooring/fender system's geometric and physical characteristics. The mooring system is designed to resist the forces produced by the external actions in such a way that permits the performance of moored ship operations within admissible movement limits (depending on the type of ship and operation to carry out) and maximum mooring, fender and hull load limits.

| TABLE 3.4.2.3.5.7 FRICTION COEFFICIENTS BETWEEN STEEL AND OTHER MATERIALS IN DRY CONDITIONS | |
|---|---|
| MATERIAL | $\mu$ |
| Wood | 0.3 |
| Rubber | 0.5 |
| Nylon | 0.2 |
| Polyethylene | 0.2 |
| Steel | 0.2 |

The main exterior forces are :

- Wind
- Currents
- Waves
- Resonances due to long wave phenomena
- Tides
- Mooring location in zones with significant water flow or backflow
- Passage of other ships
- Loading and unloading of the ship
- Ice

The determination of the load upon each mooring point can be rigorously carried out by studying the ship/mooring/fender system as a solid with elastic links, subject to variable external actions and dynamic response. The influence of many parameters shall be taken into account, such as : direction, magnitude and variation in time of the external forces; location and orientation of the berth and the ship; the number, location, and separation between the mooring points and between the fenders; the characteristics of fenders and mooring lines; the strength and elasticity of the moorings and fenders; and the load condition of the ship (ballasted, lighted, loaded and fully loaded).

Mooring forces are, therefore, difficult to analyze, and generally must be evaluated by means of mathematical or physical model tests, or by field measurements on prototypes extrapolated from berths with similar characteristics.

Nevertheless, in some particular cases, the following simplifications can be adopted :

$b_1$) FOR DESIGN SHIPS UP TO 20,000 t. DISPLACEMENT

For design ships up to 20,000 t. displacement, it shall generally not be necessary to apply the general mooring load calculation methodology, taking into account the action of external forces upon the ship.

In these cases, each mooring point shall be designed so that it resists the following minimum loads, regardless of the physical and local environmental conditions and the conditions adopted as operating limits of the installation :

— Horizontal pull according to the value given in table 3.4.2.3.5.8
— Vertical pull with a value of 1/2 the horizontal pull, acting at the same time

Pull load shall be produced shipwards at any angle with the berthing front. That which produces the most unfavorable effects in the analyzed structure shall be chosen. Onshore pull loads shall not be considered, unless the mooring point serves a berth in that direction or is especially designed as a corner mooring point.

| TABLE 3.4.2.3.5.8 | MOORING LOADS FOR SHIPS OF UP TO 20,000 DISPLACEMENT | | |
|---|---|---|---|
| DISPLACEMENT (tons) | MOORING LOADS (tons) | | |
| | Bollard or quick release hook | Bitt | Pulley (*) |
| Up to 2,000 | 10 | 10 | 20 |
| 2,000 ~ 10,000 | 30 | 15 | 50 |
| 10,000 ~ 20,000 | 60 | 30 | 100 |

(*) A pulley is understood to mean the pulley system that returns the mooring to the ship from land for its anchoring or tensioning.

For the analysis of the resistant structure, it shall be considered that the minimum pull load occurs simultaneously in all the mooring points.

For the application of these values, and to obtain the response of the structure through static analysis methods it is recommended to space the mooring points between 15.00 and 30.00 m. In the case of continuous berthing works, this length shall usually coincide with the distance between expansion joints, locating the mooring point centered in the section between joints. If there is more than one mooring point per section they will be distributed as symetrically as possible with respect to the center of the berth.

Mooring points compatible with these minimum pull loads can be designed as simple or double, assuming that they can simultaneously receive several mooring lines. For the calculation, the real level of application of the horizontal loads with respect to the structure shall be taken into account.

For structures where the pull load is applied near the center of gravity, or symetrically, with respect to the center of gravity, or for linear structures where the application point is sufficiently distanced from the ends, the point load shall be substituted with a linear load equal to the quotient pull/distance between mooring points.

Lacking other data, in order to evaluate the local effects in indefinite lineal structures, it can be assumed that the point mooring load is resisted by a length of structure equal to 2b, given that b is the width of the resistant structure. The exception is with narrow structures, where specific distribution conditions shall have to be defined in terms of their structural type and characteristics (e.g. in anchored metal sheet piling, it is customary to admit that the pull load affects a length equal to 4 times the distance between anchors).

Likewise, the moored ship pressures upon the fenders or the structure can be estimated for the cases indicated by : Horizontal pull load (according to table 3.4.2.3.5.8) times (separation between fenders/separation between bollards). The pressures shall be distributed according to paragraph a.- Berthing loads, and shall act perpendicularly to the berth front. If the moored ship acts directly upon the resistant structure or a continuous fendering systems, a unit load equal to the minimum horizontal pull load divided by the distance between mooring points shall be assumed.

For specialized ships with large superstructures (e.g. ferries, LNG carriers, or LPG carriers), ships with less than 400 t. displacement, specially exposed berthing locations (very unfaborable conditions according to table 3.4.2.3.5.2.) or structures for which the mooring loads are predominant shall be calculated according to the methodology used for ships with displacement greater than 20,000t.

$b_2$) SHIPS GREATER THAN 20,000 t. DISPLACEMENT

When determining the most probable maximum loads upon the mooring points and fenders for design ships greater than 20,000 t. displacement, the action of external forces upon the ships shall be taken into account (especially wind, currents and waves, given that the combination of these three forces generally covers the effects produced by all the external forces of simultaneous action upon the moored ship). These forces shall be taken into account together with the location, number, layout and specific characteristics of the mooring points, mooring lines and fenders, as well as the type and characteristics of the berthing structure.

The calculations shall be done not only for the largest fully loaded displacement ship (design ship,) but also for all the most unfavorable load conditions, including those between the maximum loaded ships and the minimum lighted ships. The only exception shall be projects in shipyards where the hypothesis of the ship in ballast has to be considered. The calculation shall be done for any combination of external actions under the limit conditions defined in the installation use criteria.
Likewise, the range of ship sizes that will probably use the berth shall be taken into account.

In order to minimize the effects of the external forces upon the moored ship, it is recommended to locate the berthing works in sheltered waters and in such a way that the longitudinal axis of the moored ship goes in the direction of the prevailing distribution of winds, currents and waves.

When the simultaneous action upon the ship of several external actions is considered in order to determine the mooring loads, the load combination criteria defined in Part 4. Calculation Bases shall be taken into account, in order to consider the reduced probability of various actions (mainly environmental) occuring simultaneously with their characteristic values, especially those for extreme conditions (See Section 4.2.) In this case, to obtain the characteristic value of the mooring load, the external action with predominant effect shall be considered with its characteristic value and the rest with their combination values ($\psi_0 \cdot Q_k$)

The forces and moments resulting from the action of external forces upon the ship shall be transmitted to the moorings and the mooring points, or to fenders depending on the direction of the resultant force. The determination of these forces and moments upon the moored ship, as well as their tranfer to the mooring points and fenders in a way that creates the most unfavorable effects upon the structure, can be carried out according to the following criteria :

— *LOADS DUE TO THE PERFORMANCE OF EXTREME FORCES ON THE MOORED SHIP*

— *Wind*

The load resulting from wind pressures upon ship shall be simplified in one horizontal force in the longitudinal direction of the ship, another in the transversal direction, and a vertical axis moment, all of them applied at the ship's center of gravity. They shall be determined by means of the formula in table 3.4.2.3.5.9, but other existing methods of recognised validity for specific ships can also be applied.

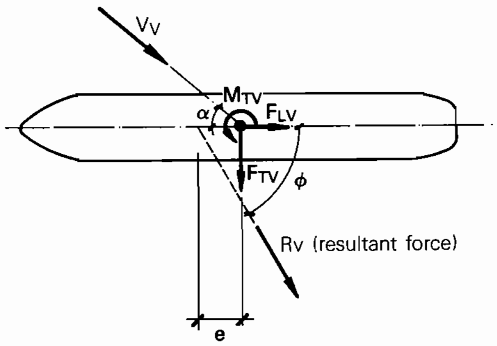
— *Currents*

The action of currents upon a moored ship can produce three types of forces: pressure forces, drag forces and forces included by dynamic instability phenomena that produce lateral self-excited oscillations (flutter effect).

The resultant forces from the pressure and friction forces produced by currents upon the ships shall be simplified into one horizontal force in the longitudinal direction of the ship, another in the transversal direction and a vertical axis moment, all of them applied at the ship's center of gravity.

These resultant forces can be estimated by means of the formulas in tables 3.4.2.3.5.10 and 3.4.2.3.5.11 or by applying other methods of recognized validity for specific ships.

TABLE 3.4.2.3.5.9 RESULTANT FORCES DUE TO WIND PRESSURES ON SHIPS




$$R_V = \frac{\rho}{2g} \cdot C_V \cdot V_V^2 \cdot (A_T \cos^2\alpha + A_L \sin^2\alpha) = \frac{C_V \cdot V_V^2}{16{,}000} \cdot (A_T \cos^2\alpha + A_L \sin^2\alpha)$$

$$\tan \phi = \frac{A_L}{A_T} \cdot \tan \alpha \qquad F_{LV} = R_V \cdot \cos \phi$$

$$F_{TV} = R_V \cdot \sin \phi \qquad M_{TV} = F_{TV} \cdot e = F_{TV} \cdot N \cdot K_e \cdot L$$

given that :

$R_V$ = Resultant wind force on a ship (t)

$\phi$ = Angle between the ship's longitudinal axis, considered from bow to stern, and the resultant wind force on the ship (degrees).

$F_{TV}$ = Transverse component of wind force on a ship (t)

$F_{LV}$ = Longitudinal component of wind force on a ship (t)

$M_{TV}$ = Resultant wind moment on a ship, relative to a vertical axis at the ship's center of gravity (t · m.).

$\rho$ = Specific weight of the air (1.225 . $10^{-3}$ t/m3)

g = Acceleration of gravity (9.81 m/s2).

$C_v$ = Shape factor (dimensionless).
Varies between 1.0 y 1.3
Lacking a more precise determination through model studies, a value of 1.3 shall be adopted for any kind of ship and wind action direction.

$\alpha$ = Angle between the longitudinal axis of a ship, considered from bow to stern, and the wind direction (degrees). Any wind direction can be considered.

$V_v$ = Design wind velocity at a height of 10 meters (m/sec), considered constant all along the height.

The mean wind velocity determined in the shortest interval (gust) capable of overcoming the ship's inertia shall be adopted as the basic velocity. A mean velocity corresponding to the following gust values shall be adopted.

- 1 minute for ships of length equal to or greater than 25 m
- 15 seconds for ships of length less than 25 m

For installations that consider that the ships remain moored at all times, the value corresponding to the extreme value associated with the maximum admissible risk ($V_{V15s}$ or $V_{V1min}$ according to the type of ship) shall be adopted as the basic value ($V_V$).

TABLE 3.4.2.3.5.9 (Continued)

The extreme values associated with maximum admissible risk shall be determined according to the criteria in section 3.4.2.4. Environmental Loads, and in ROM 0.4. Recommendations for the Consideration of Environmental Variables/II : Atmospheric and Seismic Conditions. If the gust velocity values cannot be differentiated directionally, it shall be considered that the indicated scalar values are valid for all directions.

For installations where the established use criteria does not permit the mooring of ships at all times or conditions, those velocities expressly defined as moored ship's permanence limit condition shall be adopted as the basic velocity, whether associated or not with a determined ship configuration (e.g. ballast of the ship to reduce its exposed surface).
Lacking defined operational criteria, without applying reduction factors due to ship configuration variation, a value of :

$$V_{V1min} = 22 \text{ m/s } (\approx 80 \text{ Km/h})$$

shall be adopted as the limit permanence velocity, as long as tugs are available with a total bollard pull of 150% of the maximum force resulting from the wind upon the ship.

For the mooring calculations in normal operations conditions (loading and unloading, operation of cargo handling installations, etc) lacking other specific operating criteria, a design velocity equal to :

$$V_{V1min} = 17 \text{ m/s } (\approx 60 \text{ Km/h})$$

shall be adopted.

$A_T$ = Transverse projected area of the ship exposed to wind ($m^2$)

$A_L$ = Longitudinal projected area of a ship exposed to wind ($m^2$)

Lacking other known values, these areas may be approximated by means of the following equations :

$$A_T = B \cdot (G + h_T)$$

$$A_L = L_{pp} \cdot (G + h_L)$$

given that :

B = Ship's beam

G = Ship's freeboard (Ship depth-Draught)

$L_{pp}$ = Length between perpendiculars

$h_T$ = Mean height of the transverse projected area of a ship's superstructure above deck

$h_L$ = Mean height of the longitudinal projected area of a ship's superstructure above deck.

Common values of B, G and $L_{pp}$ for the fully loaded design ship may be obtained from table 3.4.2.3.5.1.
For minimum light ships, the draught may be approximated by :

$$\text{Light draught} = \alpha \cdot (\text{maximum draught})^{\beta}$$

for the following values of $\alpha$ and $\beta$, and draughts in m

| TYPE OF SHIP | $\alpha$ | $\beta$ |
|---|---|---|
| Bulkcarrier up to 200,000 DWT | 0.551 | 0.993 |
| Oil Tanker up to 300,000 DWT | 0.548 | 0.966 |
| General cargo up to 50,000 DWT | 0.352 | 1.172 |

**TABLE 3.4.2.3.5.9 (Continued)**

The determination of freeboards and draughts of partially loaded ships, and those not found in the prior table, shall be done assuming that the ship block coefficient is constantly maintained in every load state.

Common values of $h_T$ and $h_L$ may be approximated by using the following table, according to the design ship type :

| Ship Type | Tonnage | Mean Heights (m) | |
|---|---|---|---|
| | | $h_T$ | $h_L$ |
| BULK CARRIERS AND MULTIPURPOSE | DWT | | |
| | 300,000 | 25.00 | 5.00 |
| | 250,000 | 23.00 | 5.00 |
| | 200,000 | 21.00 | 5.00 |
| | 150,000 | 19.00 | 5.00 |
| | 100,000 | 16.50 | 5.00 |
| | 90,000 | 16.00 | 5.00 |
| | 70,000 | 14.50 | 5.00 |
| | 50,000 | 13.00 | 5.00 |
| | 40,000 | 12.00 | 5.00 |
| | 30,000 | 11.00 | 5.00 |
| | 20,000 | 10.00 | 5.00 |
| | 15,000 | 9.50 | 5.00 |
| | 10,000 | 9.00 | 5.00 |
| OIL TANKERS | DWT | | |
| | 500,000 | 32.00 | 4.00 |
| | 400,000 | 29.00 | 3.80 |
| | 300,000 | 25.00 | 3.70 |
| | 250,000 | 23.00 | 3.60 |
| | 200,000 | 21.00 | 3.40 |
| | 150,000 | 19.00 | 3.20 |
| | 120,000 | 17.50 | 3.10 |
| | 100,000 | 16.50 | 3.00 |
| | 80,000 | 15.00 | 2.90 |
| | 70,000 | 14.50 | 2.80 |
| | 60,000 | 14.00 | 2.70 |
| | 50,000 | 13.00 | 2.60 |
| | 40,000 | 12.00 | 2.40 |
| | 30,000 | 11.00 | 2.20 |
| | 20,000 | 10.00 | 2.00 |
| | 15,000 | 9.50 | 2.00 |
| | 10,000 | 9.00 | 2.00 |
| | 5,000 | 8.50 | 2.00 |
| | 3,000 | 8.00 | 2.00 |
| | 2,000 | 7.50 | 2.00 |
| | 1,000 | 7.00 | 2.00 |
| | 700 | 7.00 | 2.00 |
| LNG CARRIERS | DWT | | |
| | 75,000 | 19.00 | 10.00 |
| | 50,000 | 13.80 | 8.00 |
| | 20,000 | 12.00 | 6.00 |
| | 4,000 | 9.00 | 6.00 |

| Ship Type | Tonnage | Mean Heights (m) | |
|---|---|---|---|
| | | $h_T$ | $h_L$ |
| LPG CARRIERS | DWT | | |
| | 60,000 | 14.50 | 4.00 |
| | 47,000 | 13.80 | 4.00 |
| | 40,000 | 13.00 | 4.00 |
| | 18,000 | 10.00 | 4.00 |
| | 16,000 | 9.50 | 4.00 |
| | 5,000 | 7.50 | 6.20 |
| | 3,000 | 7.00 | 5.00 |
| GENERAL CARGO SHIPS | DWT | | |
| | 50,000 | 18.00 | 5.00 |
| | 40,000 | 17.00 | 5.00 |
| | 30,000 | 16.00 | 5.00 |
| | 20,000 | 14.00 | 5.00 |
| | 15,000 | 13.00 | 5.00 |
| | 10,000 | 11.50 | 5.00 |
| | 9,000 | 11.00 | 5.00 |
| | 8,000 | 10.50 | 5.00 |
| | 7,000 | 9.50 | 5.00 |
| | 6,000 | 9.00 | 5.00 |
| | 5,000 | 8.50 | 5.00 |
| | 4,000 | 8.00 | 5.00 |
| | 3,000 | 7.50 | 5.00 |
| | 2,000 | 7.00 | 5.00 |
| | 1,000 | 6.50 | 5.00 |
| | 700 | 6.00 | 5.00 |
| CONTAINER AND RO-RO SHIPS | DWT | | |
| | 50,000 | 18.00 | 8.50 |
| | 42,000 | 17.00 | 8.50 |
| | 36,000 | 16.00 | 8.50 |
| | 30,000 | 13.50 | 8.50 |
| | 25,000 | 12.00 | 8.50 |
| | 20,000 | 10.50 | 8.00 |
| | 15,000 | 9.00 | 7.50 |
| | 10,000 | 8.00 | 7.50 |
| | 7,000 | 7.50 | 7.50 |

**TABLE 3.4.2.3.5.9 (Continued)**

| Shyp type | | Tonnage | Mean Heights (m) | |
|---|---|---|---|---|
| | | | $h_T$ | $h_L$ |
| PASSENGER SHIPS | LINERS | GRT | | |
| | | 50,000 | 17.00 | 14.00 |
| | | 40,000 | 16.50 | 13.00 |
| | | 30,000 | 15.00 | 12.50 |
| | | 20,000 | 14.50 | 12.00 |
| | | 15,000 | 14.00 | 11.60 |
| | | 10,000 | 13.50 | 11.20 |
| | | 9,000 | 13.00 | 11.00 |
| | | 8,000 | 12.80 | 10.70 |
| | | 7,000 | 12.40 | 10.30 |
| | | 6,000 | 12.00 | 10.00 |
| | | 5,000 | 11.80 | 9.80 |
| | | 4,000 | 11.50 | 9.60 |
| | | 3,000 | 11.00 | 9.40 |
| | | 2,000 | 10.00 | 8.50 |
| | | 1,000 | 9.00 | 7.80 |
| | | 500 | 8.00 | 7.00 |
| PASSENGER SHIPS | FERRIES | GRT | | |
| | | 13,000 | 17.00 | 14.00 |
| | | 10,000 | 15.50 | 13.00 |
| | | 8,000 | 14.00 | 12.00 |
| | | 6,000 | 12.00 | 10.50 |
| | | 4,000 | 10.00 | 9.00 |
| | | 3,000 | 9.00 | 8.00 |
| | | 2,000 | 8.00 | 7.00 |
| | | 1,000 | 7.00 | 6.00 |

| Ship type | | Tonnage | Mean Heights (m) | |
|---|---|---|---|---|
| | | | $h_T$ | $h_L$ |
| FISHING VESSELS | | GRT | | |
| | | 2,500 | 8.00 | 5.00 |
| | | 2,000 | 7.50 | 5.00 |
| | | 1,500 | 7.00 | 5.00 |
| | | 1,000 | 6.80 | 5.00 |
| | | 800 | 6.50 | 5.00 |
| | | 600 | 6.00 | 5.00 |
| | | 400 | 5.70 | 5.00 |
| | | 200 | 5.50 | 5.00 |
| PLEASURE CRAFT | MOTOR CRAFT | t | | |
| | | 50.0 | 5.50 | 4.00 |
| | | 35.0 | 5.00 | 3.50 |
| | | 27.0 | 4.40 | 3.00 |
| | | 16.5 | 4.00 | 2.80 |
| | | 6.5 | 3.40 | 2.40 |
| | | 4.0 | 2.70 | 2.00 |
| | | 1.3 | 2.10 | 1.50 |
| PLEASURE CRAFT | SAIL CRAFT | t | | |
| | | 60.0 | 4.60 | 5.50 |
| | | 40.0 | 4.30 | 5.00 |
| | | 20.0 | 4.00 | 4.80 |
| | | 13.0 | 3.70 | 4.50 |
| | | 10.0 | 3.40 | 4.20 |
| | | 3.5 | 3.00 | 4.00 |
| | | 1.5 | 2.70 | 3.00 |

$K_e$ = Eccentricity coefficient (non-dimensional)

The values of the eccentricity coefficient shall be approximated from the following table, lacking other specific data :

| SHIPS WITH SUPERSTRUCTURE AMIDSHIPS | | |
|---|---|---|
| $\alpha$ (degrees) | $K_e$ | |
| | Light ship | Fully loaded ship |
| 0 | 0 | 0 |
| 30 | 0.15 | 0.10 |
| 60 | 0.05 | 0.03 |
| 90 | –0.02 | –0.02 |
| 120 | –0.10 | –0.10 |
| 150 | –0.20 | –0.20 |
| 180 | 0 | 0 |

TABLE 3.4.2.3.5.9 (Continued)

| SHIPS WITH SUPERSTRUCTURES ALL AFT | | |
|---|---|---|
| $\alpha$ (degrees) | $K_e$ | |
| | Light ship | Fully loaded ship |
| 0 | 0 | 0 |
| 30 | 0.16 | 0.10 |
| 60 | 0.05 | 0.12 |
| 90 | –0.04 | –0.16 |
| 120 | –0.18 | –0.27 |
| 150 | –0.33 | –0.37 |
| 180 | 0 | 0 |

L = Ship length, in m.

NOTE :

When various ships are moored on both sides of a berth or range alongside one another, the resultant forces of the wind upon the protected ship shall be approximated as 50% of that obtained for the exposed ship.

Generally, the friction forces shall be neglected, except when currents act in a direction practically parallel to the ship's longitudinal axis.

The forces induced by the flutter effect are difficult to calculate with mathematical formulas, making model tests or prototype measurements necessary for their determination.
The flutter effect is only important for ships moored in zones with strong influence of currents of relatively constant intensity and in a direction practically parallel to the ship's longitudinal axis (e.g. mouth of a river or estuaries).

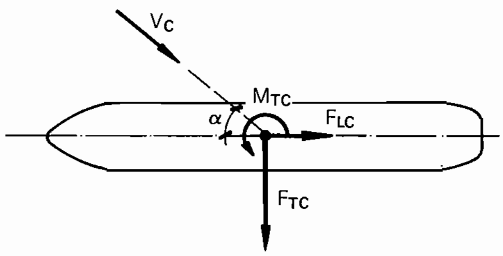
— *Wave action*

In order to calculate the mooring forces, wave action shall be considered as the action of short period waves (<20-30 s) on the moored ship. These waves are distinguished from those of periods between 20-30 seconds and five minutes, and small amplitude (10 to 50 cm), considered as long waves.

Short period waves are reduced by the breakwaters and other reflecting elements. They are generally not preponderant actions for the mooring calculations in those cases where the berths are located in zones with low agitation (protected zones) : Average agitation coefficient $k_a = H_s/H_b < 0.1$, given that $H_s$ is the significant wave height at the berth, and $H_b$ is the significant wave height at the mouth of the port or the protected zone).

Likewise, the wave periods are generally far lower than the natural oscillation periods of the ship/moorings/fender systems for medium and high displacement ships. Therefore, large amplifications of ship movements and forces due to resonance phenomena are not expected. Heave is the movement (see ship movements description in table 3.4.2.3.5.12) most prone to amplification due to short period wave action. The orders of magnitude of the oscillation periods of a moored ship are given in table 3.4.2.3.5.13.

TABLE 3.4.2.3.5.10 RESULTANT FORCES DUE TO CURRENT PRESSURES ON SHIPS




$$F_{TC} = \frac{\gamma_w}{2g} \cdot C_{TC} \cdot V_C^2 \cdot A_{LC} \cdot \sin \alpha$$

$$F_{LC} = \pm \frac{\gamma_w}{2g} \cdot C_{LC} \cdot V_C^2 \cdot A_{TC}$$

$$M_{TC} = F_{TC} \cdot e = F_{TC} \cdot K_{ec} \cdot L$$

given that :

$F_{TC}$ = Transverse component of current force on ship (t).

$F_{LC}$ = Longitudinal component of current force on ship (t).
As a simplification, it is assumed that the magnitude of the longitudinal force is independent of the performance angle of the current, so a + or - sign can be applied indistinctly. The sign that causes the most unfavorable effects upon the structure shall be applied.

$M_{TC}$ = Resultant current moment on a ship, relative to a vertical axis at the ship's center of gravity (t · m.)

$\gamma_w$ = Specific weight of the water (1.03 t/m$^3$ salt water)
(1.00 t/m$^3$ fresh water)

g = Acceleration of gravity (9.81 m/s$^2$)

α = Angle between the longitudinal axis of a ship, considered from stern to bow, and the direction of the current (degrees). Unless detailed studies are done on the directions of the currents, it shall be considered that they can act in any direction, except in those cases where the existence of physical or topographical features restrict the possible directions (e.g. mouths of rivers, estuaries, harbour mouths, straight bay entrances, etc).

$V_c$ = Design horizontal current velocity at a depth of 50% of the ship's draught, considered constant all along the depth, in m/s.
The mean velocity of the current determined in the interval of 1 minute ($V_{C1min}$) shall be adopted as the basic velocity.
For installations that consider the permanence of the ships at all times, the value corresponding to the extreme value associated with the maximum admissible risk shall be adopted as the basic velocity ($V_{C1min}$).
The determination of the extreme values associated with maximum admissible risk shall be done according to the criteria in section 3.4.2.4. Environmental Loads, and ROM 0.3. Recommendations for the Consideration of Environmental Variables/I : Waves, currents, tides and other variations of water level.
If the values of velocities cannot be differentiated by direction,it shall be considered that the scalar values are valid for all directions.

**TABLE 3.4.2.3.5.10 (Continued)**

For installations where the established use criteria does not permit the permanence of ships at all times or states, the velocity expressly defined as limit permanence condition of ships in berth, whether associated or not with a determined ship configuration (e.g. ballast reduction to reduce the exposed surface) shall be adopted as the basic velocity

Lacking other defined operating criteria, without applying reduction factors due to ship configuration variation, the following shall be adopted as the limit permanence velocity :

- Cross currents : $0° < \alpha < 180°$ — $V_{C1min} = 1$ m/s (2 knots)
- Bow or stern current : $\alpha = 0°$, $\alpha = 180°$ — $V_{C1min} = 2.5$ m/s (5 knots)

as long as the values are less than those corresponding to the extreme value associated with the maximum admissible risk, tugs are available with a total bollard pull of 150% of the maximum resultant force from the action of currents upon the ship, and the values are compatible with the particular characteristics of the project location.

For the mooring calculation in normal operating conditions, (loading and unloading, operation of cargo handling installations, etc) lacking other specific operating criteria, a design velocity equal to :

- Cross currents : $0° < \alpha < 180°$ — $V_{C1min} = 1$ m/s (2 knots)
- Bow or stern current : $\alpha = 0°$, $\alpha = 180°$ — $V_{C1min} = 1.5$ m/s (3 knots)

shall be adopted. Likewise, whenever these values are less than those corresponding to the associated extreme value, and are compatible with the particular characteristics of the project location.

$C_{TC}$ = Shape factor for the calculation of transverse current force on a ship (dimensionless). This factor depends on the ratio of Water depth/Design ship draught, increasing as the value of this ratio approaches 1.00. It can vary between 1.00 for deep waters and 6.00 for (Water depth/Draught) ratios ≈ 1.00 according to the following graph, for any kind of ship and current direction :

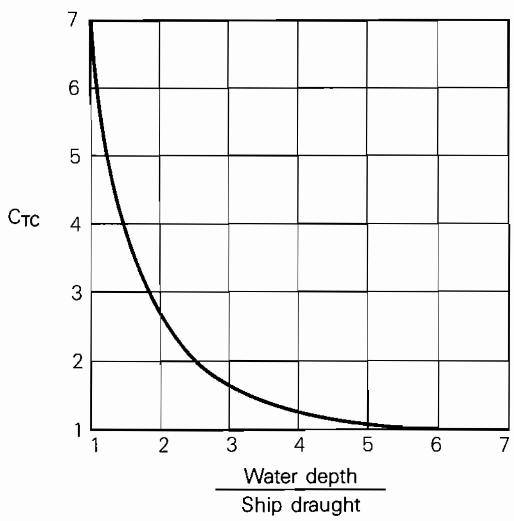


TABLE 3.4.2.3.5.10 (Continued)

$C_{LC}$ = Shape factor for the calculation of longitudinal current force on a ship (dimensionless). This factor depends on the geometry of the ship's bow. It can vary between 0.2 and 0.6. Lacking a more precise determination, a value of 0.6 is adopted for conventional bows.

$A_{LC}$ = Submerged longitudinal projected area of a ship exposed to current ($m^2$)

$A_{TC}$ = Submerged transverse projected area of a ship exposed to current ($m^2$)

Lacking known values, these areas shall be approximated by the following expressions :

$$A_{LC} = L_{pp} \cdot D$$

$$A_{TC} = B \cdot D$$

given that :

$L_{pp}$ = Length between perpendiculars.
$D$ = Ship draught.
$B$ = Ship beam

Common values of $L_{pp}$, D and B for the design ship can be obtained from tables 3.4.2.3.5.1 and 3.4.2.3.5.9.

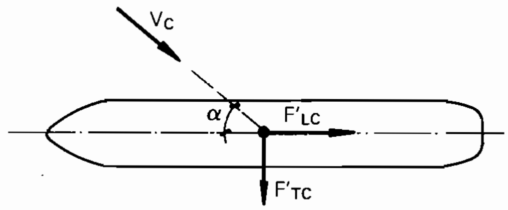
$K_{ec}$ = Eccentricity coefficient (dimensionless)

The values of the eccentricity coefficient shall be approximated based on the following table, lacking other specific data :

| $\alpha$ (in °) | $K_{ec}$ |
|---|---|
| 0 | 0 |
| 30 | 0.17 |
| 60 | 0.09 |
| 90 | 0 |
| 120 | −0.09 |
| 150 | −0.17 |
| 180 | 0 |

L = Ship length (m)

TABLE 3.4.2.3.5.11 RESULTANT FORCES DUE TO CURRENT DRAG ON SHIPS




$$F'_{TC} = \frac{\gamma_w}{2g} \cdot C_r \cdot V_C^2 \cdot A'_{TC} \cdot \sin^2 \alpha$$

$$F'_{LC} = \frac{\gamma_w}{2g} \cdot C_r \cdot V_C^2 \cdot A'_{LC} \cdot \cos^2 \alpha$$

given that :

$F'_{TC}$ = Transverse component of current drag force on a ship (t)

$F'_{LC}$ = Longitudinal component of current drag force on a ship (t)

$C_r$ = Friction coefficient (dimensionless) 0.004 shall be adopted for ships in service and 0.001 for new ships (e.g. for projects in shipyards).

$A'_{TC}$ = Transverse projected wetted surface of a ship ($m^2$)

$A'_{LC}$ = Longitudinal projected wetted surface of a ship ($m^2$)

Lacking other known values of these areas, they can be approximated by the following expressions :

$$A'_{TC} = (L_{pp} + 2D) \cdot B$$

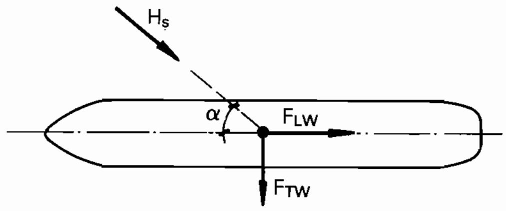
$$A'_{LC} = (B + 2D) \cdot L_{pp}$$

for values of $L_{pp}$, B and D defined according to the criteria in table 3.4.2.3.5.10.

$\gamma_w$, g, $\alpha$ and $V_c$ are defined and quantified in table 3.4.2.3.5.10.

The mooring system's response to this type of wave action, therefore, could be treated in terms of static analysis for the maximum value of the applied load, as long as moored ship oscillation periods similar to the incident wave action periods are not expected (usually this is not expected for ships larger than 20,000 DWT)

TABLE 3.4.2.3.5.12 RESULTANT FORCES DUE TO WAVES ACTING ON SHIPS




$$F_{TW} = C_{fw} \cdot C_{dw} \cdot \gamma_w \cdot H_s^2 \cdot D' \cdot \sin\alpha$$

$$F_{LW} = C_{fw} \cdot C_{dw} \cdot \gamma_w \cdot H_s^2 \cdot D' \cdot \cos\alpha$$

given that :

$F_{TW}$ = Transverse component of wave force on a ship (t)

$F_{LW}$ = Longitudinal component of wave force on a ship (t)

$\gamma_w$ = Specific water weight : (1.03 $t/m^3$ salt water)
(1.00 $t/m^3$ fresh water)

$\alpha$ = Angle between the longitudinal axis of a ship, considered from bow to stern, and the direction of the waves (degrees)
It shall be considered that the wave action can act in any direction, except in those cases where physical or topographical conditions exist, that restrict the possible performance direction (e.g. berths inside tidal basins, estuaries, etc)

$C_{fw}$ = Waterplane coefficient (dimensionless)

Values asigned in the following table shall be adopted in terms of the wave length at the location depth ($L_w$) and the ship draught (D).

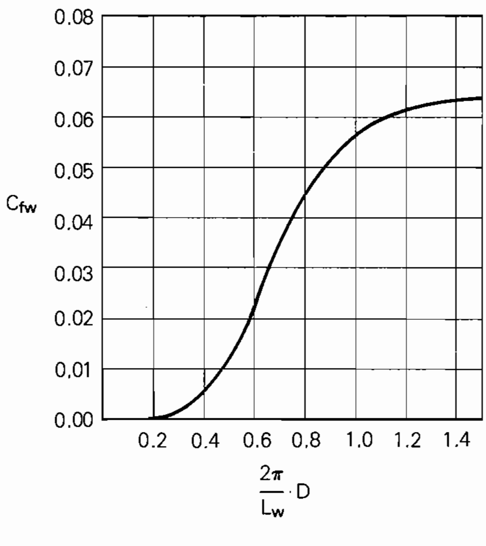
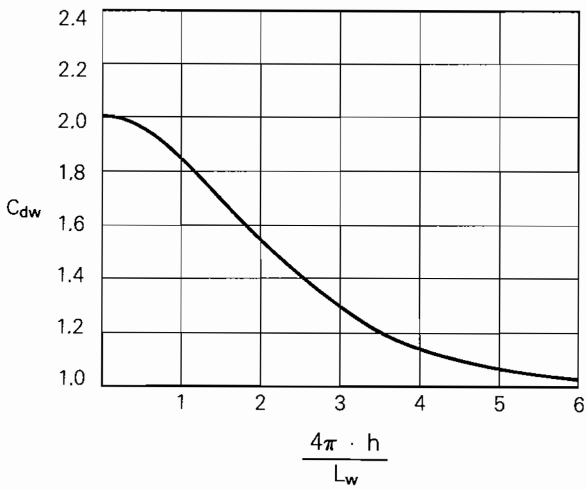


TABLE 3.4.2.3.5.12 (Continued)

$C_{dw}$ = Depth coefficient (dimensionless)

The values of this coefficient shall be obtained from the following table, in terms of the wave length at the location depth ($L_w$) and the depth of the water existing at this location (h).




D' = Projection of the ship's length in the direction of incident waves (m)
Lacking other known values, it shall be approximated by the following :

$$D' = L_{pp} \cdot \sin\alpha + B \cdot \cos\alpha$$

given that :

$L_{pp}$ = Length between perpendiculars (m)
B = Beam (m)
$\alpha$ = Angle of the incident waves (degrees)

$H_s$ = Design significant wave height for the specified direction at the site's depth (h) in meters.

For installations that consider the permanence of the ships at all times with the required safety, $H_{1/3}$ corresponding to the extreme value associated with maximum risk admissible for each wave action direction shall be adopted as the significant wave height.The determination of the extreme values associated with admissible maximum risk shall be carried out according to the criteria in section 3.4.2.4. Environmental Loads and from ROM 0.3. Recommendations for the consideration of environmental variables/I : Wave action, currents, tides and other water level variations.

For installations where the established use criteria does not permit ship permanence at all times or states, the values expressly defined as limit permanence conditions of berthed ships shall be adopted as significant wave heights. The use criteria shall be associated with excedence probabilities according to general economic criteria. The use criteria shall be specified in terms of the maximum safety limits, using as data the maximum allowable loads for moorings and fenders and the ship's integrity (normally the maximum pressure upon the hull) corresponding to the atmospheric conditions defined as operating limits.

Generally, the waves considered shall depend on the type of ship, the load state, the rigidity of the mooring system and the availability of tugs with sufficient bollard pull to be able to get the ship out of the installation when these

TABLE 3.4.2.3.5.12 (Continued)

wave action conditions are present (total bollard pull of 150% of the maximum resulting force).

Lacking defined operating criteria, the following significant wave heights shall be adopted as permanence limits of moored ships, except if $H_{1/3}$ of the extreme wave distribution is smaller :

— Ships with up to 3,000 t displacement :

- Pleasure craft $H_s = 0.40$ m
- Fishing vessels
  - $L < 30$ m $H_s = 0.6$ m
  - $L \geq 30$ m $H_s = 1.0$ m
- Other types of ships $H_s = 1.0$ m

— Ships with displacement greater than 3,000 t.

$H_s = 2.00$ m

Nevertheless, the wave height in these cases, is a poor indicator of the operating limits, given the complex interactions between the berthing configuration, the mooring system and the ship.

The ship's movements and the associated forces shall form the most satisfactory design criteria.

In the same way, for mooring calculations in normal operating conditions (loading and unloading, operation of cargo handling installations, etc) the use limit wave height shall be determined in terms of the maximum admissible ship movements in design mooring conditions, and under the combined action of all the compatible exterior forces in this work hypothesis, through physical or mathematical models.

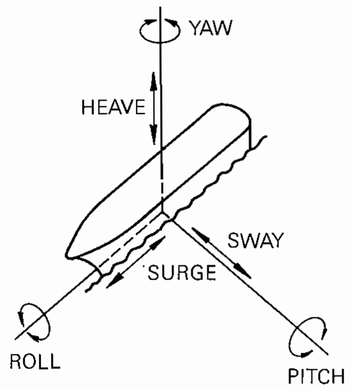
The maximum admissible amplitudes of these movements shall be subject to subjective criteria, basically depending on the type of ship and handling system used.
In the following table, the allowable movement ranges for the different types of ships during the loading and unloading processes are indicated :

| SHIP TYPE | Surge | Sway | Heave | Roll | Pitch | Yaw |
|---|---|---|---|---|---|---|
| THE CARGO HANDLING SYSTEM IMPOSES NO LIMITATIONS TO THE CONNECTION BETWEEN SHIP AND BERTH (1) | | | | | | |
| ≥ 30,000 DWT | ±1.00 m | — | — | ± 0.5 m | ±1.00 m | ±0.5 m |
| < 30,000 DWT | ±1.00 m | 0.5 m | ±1.00 m | ± 0.7 m | ±1.00 m | ±1.0 m |
| THE CARGO HANDLING SYSTEM IMPOSES LIMITATIONS TO THE CONNECTION BETWEEN SHIP AND BERTH (2) | | | | | | |
| Oil tank | ±4.00 m | ±4.00 m | ±1.00 m | ± 1° | — | ± 3° |
| LNG/LPG Carriers | ±0.10 m | 0.10 m | IN | IN | — | IN |
| Ore Carriers | ±1.50 m | 1.00 m | ±0.50 m | ± 2° | — | ± 2° |
| Bulk Carriers | ±0.50 m | 0.50 m | ±0.50 m | ± 1° | — | ± 1° |
| General Cargo Ships | ±1.00 m | 0.50 m | ±0.50 m | ± 2° | — | ± 2° |
| Container Ships | ±0.60 m | 0.40 m | ±0.60 m | ± 1° | — | IN |
| Ro-ro ships (side ramp) | ±0.20 m | 0.20 m | ±0.10 m | IN | — | IN |
| Ro-ro ships (bow or stern ramp) | ±0.10 m | IN | ±0.10 m | IN | — | IN |

TABLE 3.4.2.3.5.12 (Continued)

Generally, the significant wave height ($H_s$) operating limit in normal operating conditions (loading and unloading, etc) shall be within the 0.15 to 0.50 m range.

SYMBOLOGY AND NOTES :




IN : Insignificant.

(1) The movements are measured with respect to the rest position of the ship in both directions, at the height of the fender where the greatest relative movements are observed.

(2) ± signifies movements in both directions with respect to the rest position.

(3) The values reflected in the table are valid for short period movements (< 20 s). For longer period movements, larger values can be accepted.

The analytical quantification of the forces produced by wave action on moored ships is very complex due to its dependency on many variables, including the following :

— Incident wave action characteristics : wave type (progressive or stationary), height, period and direction.
— Configuration of the berth and type of structure : diaphanous, solid, semi-solid.
— Elastic characteristics of the fenders and mooring lines.
— Ship characteristics : typology, displacement, dimensions, underkeel clearance, etc.
— Type of ship movement.

Nevertheless, as a first approximation, and lacking other more specific studies such as model tests or prototype measurements, the formula in table 3.4.2.3.5.12. can be adopted, deduced from the consideration of the wave action forces as a result of fluid pressures upon the ship's hull produced by a regular incident wave action. In terms of static analysis, the resultant force can be simplified in one horizontal force in the longitudinal direction of the ship, and another in the transversal direction.

— *Resonance due to long wave phenomena*

Long waves, or waves of long period (> 20-30 s), and low amplitude in open seas, upon penetrating natural or artificial confined areas can give rise to changes in the oscillations conditions of these areas if the long wave period is similar to the natural oscillation period of the confined area. This could produce long period stationary or quasi stationary oscillations, with characteristic points of maximum horizontal (nodes) or vertical (antinodes) movements.

The oscillation periods of tidal basins due to long waves are close to those of surge, sway and yaw motions of moored ships. This, together with the low attenuation in this frequency range, gives rise to the possibility that dynamic amplification phenomena occur in the ship/mooring/fender system. In this situation, the moored ship absorbs a large fraction of the energy of the waves and sudden high amplitude movements occur, with a consequent increase in the mooring forces. These oscillations and the associated forces are very difficult to describe and quantify due to the non-linearity of the mooring and fender systems and the existing phase lag between the ship oscillation and tidal basin oscillation that creates forces with a long period subharmonic component that can be the source of even larger ship movements due to resonance.

The response of the mooring system to this type of wave action is not susceptible to being treated in terms of static analysis, so model studies should be performed to allow the ship movements and the mooring and fender forces to be measured.

The correction against this type of phenomenon is not so much the calculation of forces generated, but the anticipation and prevention by means of an appropriate mooring configuration in order to shift the natural oscillation periods of the most critical movements as far as possible from the periods corresponding to the forces acting upon the ship. Likewise, the possibility that sheltered zones enter in resonance should be avoided.
It shall be necessary, therefore, to have information regarding the following :

— Natural oscillation periods and patterns of the basin or bay.
— Horizontal and vertical displacement amplitudes.
— Location and orientation of the nodal lines.
— Existence of natural phenomena with periods close to the tidal basin's resonance period (underset by rapid wind or atmospheric pressure variations in open sea, long «set-down» waves, tsunamis, edge waves, and "surf beats"; including their frequencies of occurrance.
— Oscillation periods of the moored ships in terms of the type, displacement and load condition of the ship; mooring and fender layout; water depth, occupation percentage of the piers, etc.

In the case of tidal basins with defined geometric forms, there are empirical formulas that give the oscillation periods and the corresponding modes (Iribarren, Bruun, et al.). These formulas should only be used if the basins are very similar to those in the theoretical formula. There are also mathematical and physical models that give information regarding the basin's oscillation patterns.

Others of magnitude of the natural oscillation periods of a moored ship are given in table 3.4.2.3.5.13.

— *Tides and other water level variations*

These do not exert forces upon the moored ships in themselves. However, modifications in the relative height between the ship and the mooring points induce variations in the anticipated mooring loads.

The effects of the water level variations shall be studied for each particular situation in terms of whether the use criteria and conditions compel the correction of the mooring system, the use of constant tension winches, or on the other hand, adjustments are not foreseen, thus giving rise to continuous alterations in the mooring system configuration. In the absence of specific criteria, the Project Engineer shall not consider the possibility of adjustments in the mooring system.

As a minimum, the following limited situations shall be checked :

— Maximum descent of the ship (Fully loaded ship with ebb tide)
— Maximum ascension of the ship (Light ship with flow tide)

For installations that consider the permanence of ships at all times or states the values corresponding to the extreme value associated with the maximum admissible risk shall be adopted as maximum and minimum water levels (see characteristic free outer water levels in Spanish coastal zones in table 3.4.2.1.1).

TABLE 3.4.2.3.5.13 ESTIMATED NATURAL OSCILLATION PERIODS OF A MOORED SHIP (seconds)

| SHIP TYPE | PITCH | ROLL | HEAVE | SWAY | YAW | SURGE |
|---|---|---|---|---|---|---|
| Pleasure and Fishing Craft | 2-4 | 3-5 | 6-9 | — | — | — |
| Small freighters (< 7,000 DWT) | 5-8 | 7-10 | 7-10 | — | — | — |
| Freighters (7,000-11,000 DWT) | 7-11 | 9-14 | 10-11 | — | — | — |
| Passenger liners | 12-14 | 16-18 | 9-12 | 14-32 | 15-20 | 45-85 |
| Oil tankers (30,000 DWT) | 7-9 | 9-12 | 12 | 14-32 | 15-20 | 45-65 |
| Oil tankers (100,000 DWT) | 8-11 | 10-14 | 15 | 40-55 | 25-50 | 70-85 |
| Oil tankers (300,000 DWT) | 9-12 | 12-16 | 17 | 50-100 | 50-100 | 100-115 |

NOTES :

1. The influence of the ship's load state is very strong, resulting in the highest periods for fully loaded vessels and the lowest periods for light ships.
2. The values reflected in the table are applicable for elastic moorings and normal fenders.The use of rigid moorings and fenders reduce the natural oscillation periods, but increase the forces in the moorings and fenders.
3. The lesser the ship's mass, the shorter its natural oscillation period will be.

For installations where the established use criteria does not permit the permanence of a ship at all times or states, the levels specifically defined as operating condition limits shall be adopted for the calculation. The limit levels for normal operating conditions (loading and unloading, etc) shall be defined in the same way.

In these work hypothesis, lacking other defined operating limits,the outer water levels established in table 3.4.2.1.1 for normal operating conditions shall be used (e.g. HWSL and LWSL in seas with astronomical tides).

Tides and other water level variations shall also be taken into account to determine the ship position relative to the fenders in different water levels. In the calculation, the friction of the fenders with the ship's hull due to vertical displacement, must be taken into account, whenever it is relevant to the design of the considered element.Likewise, tides can modify the approximation conditions and incident wave characteristics and the reflecting capacity of the berthing structures, modifying the wave action in the calculation and the free oscillation periods of the basins. These effects shall be taken into account in the determination of mooring loads.

— *Mooring located in zones with significant water flows and backflows*

The existence of significant water flows and backflows, caused by fluvial or tidal currents, in the proximities of a diaphanous berthing structure can give rise to hydrodynamic forces upon the moored ship due to its interposition in the water flow. These forces are difficult to quantify and can be taken into consideration by means of model tests or prototype measurements.

— *Navigation of other ships*

The navigation of other ships near the berthings can provoke an increase in the agitation due to their wake. This effect is generally not considered in the calculation, however it shall be taken into account when excessive passage velocities are foreseen or when it takes place in very narrow basins.
The analytical quantification of the forces produced can be done according to the natural wave action formula.

Moreover, if the second ship passes very close to the moored ship, it can produce a suction phenomenon, giving rise to hydrodynamic forces approximately proportional to the square of the relative speed.
This effect can be disregarded in the calculations, except in special cases.

— *Loading and unloading of the ship*

The loading and unloading processes can give rise to significant variations in the ship's draught, depending on the type of ship and cargo. In any case, as is the case with tides and other water level variations, the problem consists of the modification of the relative height between ship and berth.

The effects of these processes in the determination of mooring loads shall be studied for each particular case in terms of the foreseen use criteria. In the absence of specific information or criteria, the Project Engineer shall not consider the possibility of adjustments in the mooring system during these processes.

As a minimum, the limit situations indicated for tides shall be checked, given that in that section the added effect of the loading and unloading processes to the water level have already been taken into account.
Also, attention shall be paid to the asymmetrical loading conditions that produce significant trim or heel angles. In these cases, the symmetry of the mooring system is altered and the loads are distributed nonuniformly

— *Ice*

The effects of ice upon the ship shall rarely be considered in the evaluation of mooring loads, except in extremely cold geographic zones with tidal or fluvial currents.
In these case, ice can create a longitudinal force upon a ship when it is trapped between the ship and the berth, and currents are produced by variations in the water level.

In Spanish coastal zones, the possibility of significant freezing shall not be considered in the determination of mooring loads.

— *DISTRIBUTION OF FENDERS, MOORING LINES AND MOORING POINTS*

The Project Engineer, together with the Client or Government, shall set in the project a basic configuration of the berth and mooring system in order to minimize the movements of the moored ships and the loads transmitted to the resistant structure.
The configuration of the berthing shall be defined for each one of the ships whose use in the installation is foreseen, based on these factors : location and orientation of the berthing, and distribution, type and number of the mooring lines and mooring points and fender systems, for the environmental conditions adopted as operating limits of the installation.
Reinforcements in the basic berth configuration shall be considered when the environmental actions surpass preset values.

These configurations of the berth shall be used for the distribution of the effects resulting from the action of the external forces upon the ship to the mooring lines, fenders and mooring points.

The following points shall be taken into account for the design of the configuration of the berth and mooring :

— The least possible number of mooring lines shall be adopted. In this way, the mooring maneuvers and the tensioning shall be simplified.
— The mooring lines and the fenders shall be symmetrically distributed with respect to the center of the ship. The objective is to distribute the loads homogenously among all the moorings or fenders and reduce, where possible, the coupling between different movements. For the calculation, it shall be considered that the moorings remain tensioned at all times.
— The mooring lines shall be laid as horizontally as possible, in order to increase their efficiency.
The maximum vertical angle with the horizontal shall be 25 to 30° in the worst loading condition and water level.
— The moorings shall be laid out as aligned as possible with the movement they seek to restrict.

- For longitudinal mooring, and continuous berthing lines, the optimum disposition of the mooring lines shall consist of head, stern, and breast lines drawn from the ship as far bow and stern as possible, together with springs connected to the ship at bow and stern at distances equivalent to 1/4 of the length. The head and stern lines shall be laid out forming 45 ± 15° with the longitudinal axis of the ship. The breast lines shall be distributed practically perpendicular to this axis (90°), but layouts with angles of 90° ± 30° shall be admitted. The spring lines shall form angles of 5 to 10°.
Other configurations, especially those that omit the breast lines are also customary.
Likewise, for large ships, spring line duplication can occur, located symmetrically with respect to the bow and stern.

- For longitudinal mooring, and discontinuous berth Fronts (e.g. "T" piers, mooring dolphins, etc) the optimum disposition of the mooring lines shall consider the location of mooring points behind the berth front alignment in order to improve the mooring conditions by increasing their length. Head and stern lines and breast lines almost perpendicular to the ship and short springs almost parallel to it shall be laid out. If the longitudinal forces are preponderant, it is recommended to open the head and stern lines (up to 45°), in order to contribute to the resistance of these forces together with the springs.

— The mooring lines of the same function (breast lines, head or stern lines or spring lines) shall be of the same material and of equal length in order to maintain load symmetry.
Mooring lines shall preferably be long and few and made of a very elastic material (natural or synthetic fibers), and preferably, since they have a higher deformation capacity, and therefore transmit lower loads for the same ship motion amplitudes with the exception of trying to restrict very high movements or resultant loads, in which case, galvanized steel lines shall be necessary.
The optimal length shall vary from 35 to 50 m, according to the type of ship. Lengths of less than 30 m shall not be accepted.

— The number and the optimum separation between fender axes shall be :

- In continuous berth fronts, fender separations shall be less than or equal to 0.15 · L, given that L is the shortest length among the design ships. The fenders will be positioned along the total length of the berth front.

- In discontinuous berthing configurations, two fenders shall be sufficient, with a separation of between 0.25 · L (for ships of up to 10,000 DWT) and 0.50 L (for ships of more than 10,000 DWT) for the foreseen range of ships.

— The integral mooring and fendering systems shall behave best when both elements have comparable stiffness. A very elastic mooring/fendering system with damping capacity shall be of interest in order to best reduce the forces and movements.

The optimum distributions of mooring lines and fenders are found in table 3.4.2.3.5.14.

— *CALCULATION OF MAXIMUM LOADS ON FENDERS, MOORING LINES AND MOORING POINTS*

Once the forces resulting from the actions of external forces upon the moored ship are determined as indicated in this section, the loads upon each mooring line and mooring point and fender shall be calculated by resolution of the elastic ship/moorings/fender system foreseen in the project, by means of manual procedures or by computer.

Due to the difficulty of the analysis, it shall be admitted, lacking more detailed calculations, that the transversal and longitudinal horizontal forces and the vertical axis moment resulting from the action of exterior forces upon the ship are statically transmitted to the mooring lines, mooring points and fenders, according to any of the following methods :

- METHOD 1

  The longitudinal resultant load shall be resisted only by the springs lines. The transversal resultant load and the vertical axis moment shall be resisted only by the breast lines, or in their absence, by the head and stern lines, as long as the direction of the action produces tension in the moorings. In the opposite case, they shall be resisted by the fenders located in the central half of the design ship (0.50 · L) if the ship is larger than 10,000 DWT or in the center quarter part (0.25 · L) in ships up to 10,000 DWT, adopting a uniform distribution of the transversal resultant load and a lineal distribution of the moment between all of them. If fenders are not available, loads shall be resisted directly by the resistant structure, adopting the identical ship/structure contact length as that adopted for ship/fenders (0.25 · L/0.50 · L).

  All the mooring lines and the fenders shall be considered to have the same characteristics, taking into account the geometric configuration of the berth foreseen in the project (mooring lengths and their vertical and horizontal angles and the fender layout).

- METHOD 2

  It shall be assumed that all the moorings work with the same tension, considering that all of them have identical material characteristics and cross-section. The resultant of the external forces upon the ship shall be resisted by the horizontal components of the forces in the moorings according to the following formulas, as long as the direction of the action produces tension in all the moorings :

$$R_L = \sum_i S \cdot \cos \phi_i \,.\, \cos \theta_i$$

$$R_T = \sum_i S \cdot \cos \phi_i \,.\, \sin \theta_i$$

  given that :

  $R_L$ = Component longitudinal to the ship of the resultant of the external forces upon the ship.
  $R_T$ = Component transversal to the ship of the resultant of the external forces upon the ship.
  $S$ = Load in each mooring, assumed constant and identical for each of them.
  $\phi_i$ = Vertical angle of mooring i.
  $\theta_i$ = Angle between the horizontal projection of mooring i with the longitudinal axis of the ship, considered stern to bow.

  In case the resultant of the external forces does not produce tension in all the moorings, the transversal component shall be resisted by the fenders or directly by the structure according to Method 1. In any case, the longitudinal component shall be resisted by the springs.

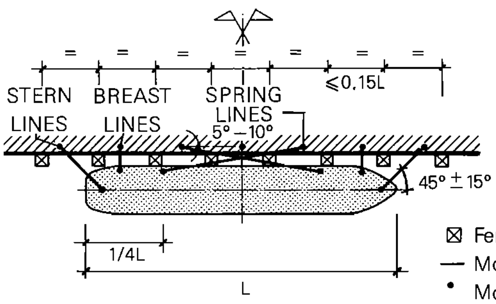
- METHOD 3

  If the project consists of a geometric configuration of the mooring with six mooring points, it may be assumed that each one of them absorbs 1/3 of the transversal resultant. The longitudinal resultant shall again be absorbed by the spring's mooring points.

## TABLE 3.4.2.3.5.14 SUGGESTED MOORING AND FENDER LAYOUTS

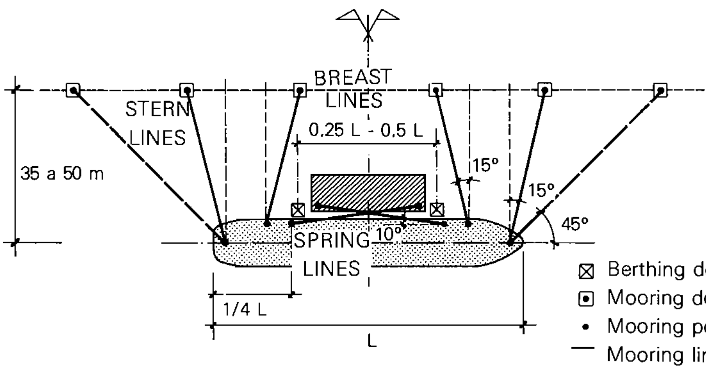
### PLAN LAYOUT

1. CONTINUOUS BERTH FRONT



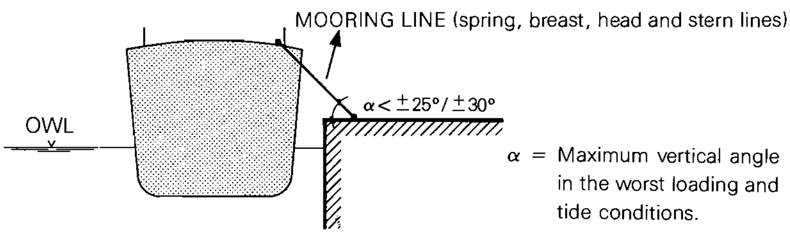

⊠ Fender.
— Mooring line.
• Mooring point.
The breast lines can be omitted and/or substituted by head and stern lines.

2. DISCONTINUOUS BERTH FRONT




⊠ Berthing dolphin.
⊡ Mooring dolphin.
• Mooring point.
— Mooring line.
The head and stern lines can also be projected alternatively with and inclination of 45°

### FRONT VIEW




$\alpha$ = Maximum vertical angle in the worst loading and tide conditions.

If the project consists of only four mooring points, it may be assumed that each one of them receives 1/2 of the transversal resultant.

The horizontal mooring loads obtained according to these methods shall be complemented by the vertical component obtained from the vertical angles foreseen in the berth's geometrical configuration.

Likewise, the real application level of the horizontal loads, with respect to the resistant structure, shall be taken into consideration in the calculation.

— *MINIMUM MOORING LOADS*

When foreseeing possible changes during the design life of the structure concerning the use conditions, the mooring's geometrical configuration or the use criteria considered in the project, the resistant structure shall be calculated alternatively with the minimal horizontal mooring loads given in table 3.4.2.3.5.15 in terms of the maximum displacement of the design ship.
If the data regarding the characteristics and order of magnitude of the external forces that can act upon the ship, or the specific mooring system configuration, is insufficient or unreliable, it shall be sufficient to adopt only the minimum horizontal mooring loads.

When the minimum mooring loads are applied according to table 3.4.2.3.5.15, the simultaneous action of a vertical pull with a value of 1/2 the horizontal shall be considered.

The conditions of application of these loads (separation of mooring points and compatible fenders, cases of application of equivalent uniform loads, etc) shall be considered identical to those set for minimum mooring loads for design ships up to 20,000 t. displacement (see paragraph $b_1$), as long as this does not contradict the notes in table 3.4.2.3.5.15.

In the same way, the minimum pressures of the moored ship upon the fenders or the structure may be estimated, for the cases indicated, according to the criteria in paragraph $b_1$ of this section.

— *DYNAMIC EFFECTS*

In order to take into account in the calculation the dynamic effects neglected by the simplified methods for the determination of mooring loads, it shall be adopted that the mooring load acting upon the resistant structure, mooring equipment and fenders is equal 1.5 times the calculated theoretical value. This consideration shall not be valid when minimum mooring loads are applied.

— *MOORING LOAD DISTRIBUTION CRITERIA*

The loads upon mooring points shall be considered concentrated in their point of application, unless simplified uniform distributions are adopted.

The mooring loads due to ship pressure upon the fenders shall be considered applied in the contact area between the ship's hull and fender or structure, as a function of the hull's geometry and the characteristics of the fender system.

The contact pressures upon the ship's hull shall be maintained within the admissible limits (see Berthing Loads). For continuous fender system, and lacking more precise information, it shall be adopted that the pressure is distributed in a rectangular area whose largest dimension coincides with the length of the contact between the moored ship and the fender system. This length shall be estimated as :

- $0.25 \cdot L$ for ships up to 10,000 DWT
- $0.50 \cdot L$ for ships larger than 10,000 DWT

in order to consider local effects in the berthing structure, the pressures shall be distributed over the fender/structure contact area.

Lacking other data, in indefinite lineal structures, it shall be adopted that the mooring loads are resisted by a structural length equal to :

TABLE 3.4.2.3.5.15 MINIMUM HORIZONTAL MOORING LOADS FOR SHIPS WITH DISPLACEMENT GREATER THAN 20,000 t

| DISPLACEMENT (t) | MOORING LOADS (t) |
|---|---|
| 20,000 ~ 50,000 | 80 |
| 50,000 ~ 100,000 | 100 |
| 100,000 ~ 200,000 | 150 |
| > 200,000 | 200 |

NOTES :

- The values shown in the table shall be applicable for mooring points formed by simple or multiple bollards, bitts, quick release mooring hooks, pulley wheel, etc.
- For sites exposed to strong winds or currents, the values given in the table shall increase by 25%. (very unfavorable conditions in table 3.4.2.3.5.2.)
- The principle mooring points situated at the extremes of isolated berthing structures shall be designed with mooring loads of :

  250 t for ships of up 100,000 t
  300 t for ships of 100,000 t ~ 200,000 t
  400 t for ships of less than > 200,000 t
- The tension pull shall be produced towards the water at any angle with the berth front line.
  Tension pull towards land shall not be considered unless the mooring point serves a berth in that direction or it is designed especially as a corner mooring point.
- The mooring point separation associated with the indicated loads, shall be considered 30 m.

- b for loads in mooring points
- l + 2b for loads upon fenders or the structure.

given that l is the fender/structure or ship/structure length of contact, and b is the width of the resistant structure.

If the lineal structure is narrow, other distribution conditions may be defined, in terms of the structural typology and characteristics.

c) DRY DOCK LOADS

Dry Dock loads are static loads generated by ships during their operation in dry dock for repair, construction or maintenance, when the ship is supported on the floor and walls of the dry dock, using rows of blocks and/or struts.

The dry dock loads shall be determined in magnitude, application and distribution, taking the following factors into account :

— The operational or use criteria of the dry dock.

- *Design ships*

  The maximum value of the dry dock loads shall be limited in terms of the highest displacement ships foreseen in the project in the load condition preset in the use criteria. Lacking defined use criteria, the following shall be considered :

  - In normal operating conditions :
    Ballast ships for ship construction dry docks.
    Minimally lighted ships for ship repair or maintenance dry docks.

  - In exceptional conditions :
    Fully loaded ships for repair or maintenance docks.

- *Possibility of dry docking one or more ships simultaneously*

  The individual dry docking of the greatest length and displacement ship foreseen in the project shall always be considered, centered with the axis of symmetry of the dry dock. At the same time, if the dock is sufficiently wide or long, other multiple lesser displacement ship dispositions shall be considered, placed laterally or longitudinally parallel to the dock's symmetry axis with symmetrical and asymmetrical location.

— The geometric characteristic of the design ships

— The weight distribution curves of the design ships in the loading conditions foreseen in the use conditions of the installation.

— The dry docking process.

The ship begins to dry dock normally, resting the stern on its end, producing a point load at the stern during the time needed to rotate the ship to cancel its trim. Lacking other data, the force in the stern may be approximated as :

$$0.004 \cdot (\Delta/L) \cdot (R\text{-}a)$$

given that :

$\Delta$ : The ship's displacement, in t.

L : The ship's length, in m.

R-a : The longitudinal metacentric radius, in m.

Next, the bow touches the bottom until the ship is fully supported along the length of the keel. The reactions in the bow and stern during this process shall be deduced considering that the ship works as a beam supported at both ends.

The length of the supporting zones shall be in terms of the deformation capacity of the bed (see dry dock bed typology).

— Dry dock bed typology

- *Disposition and distribution of blocks and struts for each type of design ship*

  Blocks shall be distributed in longitudinal alignments or combinations of longitudinal and transversal alignments. Given that the transversal sections of the majority of the large ships are rectangular, the blocks shall generally be distributed in an odd number of alignments (1,3,5,7,...) symmetrically distributed relative to the longitudinal axis of the ship (keel) and with a minimum separation of 3 m. between axes, so that the maximum admissible tension upon the block, supposedly distributed in the effective contact area with the ship's hull is not exceeded.

  In the absence of specific criteria, a unique keel alignment shall be adopted. The struts shall be placed arbitrarily to support eaves or large cantilevers of the ships, and supported on the floor or the walls of the dry dock.

- *Block strength and deformation characteristics*

  The elasticity of the blocks (together with the floor of the dry dock and the soil), in relation to that of the ship considered as a flexible beam (fictitious double T beam equivalent to the ship concerning resistance and deformation) is of great importance to avoid large overloads during the dry docking process. Blocks with very low elasticity in relation to the ship or stiff blocks (e.g. hard wood in the upper or lower face of a concrete block) shall produce high load concentrations in the bow and stern zones during the dry docking. On the other hand, once the ship is set the dry dock floor, the distribution of loads upon it shall be very uniform, regardless of the weight distribution of design ships.
  Blocks with very high elasticity (e.g. butylic rubber sections combined with laminated steel plates) shall eliminate the point loads during the dry docking process, even though they will reproduce more closely the design ship weight distribution in the loads upon the floor.

  The load transmitted to the floor by the ship supported on the blocks shall always be estimated based on the assumption that the fictitious beam equivalent to the ship is supported by a series of springs (blocks) of known and not always constant

elasticity, for a vertical load distribution coinciding with that of the ship.
In order to simplify the calculations, and to be able to equate the ship to a fictitious beam, lacking a more precise analysis, the following load distributions in the dry dock's floor shall be considered :

- Uniform or trapezoidal uniform loads for rigid blocks.
- Distributions coincident with the weight curves of the ship for elastic blocks.

For rigid blocks made of hard wood in the upper or lower face of a concrete block, in the absence of more precise information, 180 t/m$^2$ shall be adopted as a maximum admissible tension. For the usual dimensions of the block (1.20 x 1.20 m$^2$ in plan) it shall be adopted, therefore, that the maximum load transmitted to the dry dock's floor shall be 260 t. The usual separation of the blocks is 1.80 m between centers in the longitudinal direction.

— Environmental load action

In spite of the fact that the action of the environmental loads, especially wind, upon dry docked ships gives rise to unforeseen loads in blocks and struts, their effects shall be neglected in the determination of the dry dock loads. They shall be considered to be absorbed by the mooring lines and mooring points. Consequently, mooring loads shall be introduced for the design of dry docks according to paragraph b) of this section.

In summary, dry dock loads shall basically be considered static vertical loads. The horizontal components induced by eccentric transmissions of the weights to the blocks, environmental actions or accidental impacts shall be neglected in the calculations.

They can be described as :

— Linear loads distributed along the length of one or various alignments according to the use criteria of the installation, equivalent to actions due to dry docked ships. They shall be simplified in uniform or trapezoidal sections in accordance with the design ship's weight distribution tables and with the stiffness relationship between the dry docking bed and ship.

— Concentrated load trains in a zone, each one of them distributed in the block/structure contact area. These loads shall be equivalent to the actions caused during the dry docking process, or mounting and dismounting of the blocks with dry docked ship. The maximum load admissible per block shall not be exceeded.

— Surface loads equivalent to the actions transmitted to the floor by the struts, storage of the blocks, materials and other structural elements for ship repair; mobile cranes, etc.

— *MINIMUM DRY DOCK LOADS*

Given the mobility of blocks and the variability in their exact location, disposition, and strength characteristics, and foreseeing other possible modifications in the use conditions of the installation throughout its design life, the minimum dry dock loads given in table 3.4.2.3.5.16 shall be considered alternatively.

The linear uniform loads given in this table shall act simultaneously with a uniform superficial overload of 2.0 t/m$^2$, completely or partially distributed over the dry dock floor so that it produces the most unfavorable effects.

Likewise, the local effects shall be analyzed considering the actions of a minimum concentrated load of 400 t., distributed over a surface of 1.20 x 1.20 m$^2$, applied where the most unfavorable effects are foreseen. This load shall not act together with the lineal or superficial uniform loads specified.

Lacking precise data regarding the use criteria of the installation, ship weight distribution curves and typology of the dry docking bed, it may be sufficient to use the minimum dry docking load.

For high tonnage ships (≥ 100,000 DWT) with rectangular transversal sections, the minimum dry dock loads given in table 3.4.2.3.5.16 may not be considered in the application conditions given in this table. In these cases, the minimum dry dock load shall be uniformly distributed in three or more odd alignments, as long as the keel alignment receives at least half of the total load. The lateral alignment of blocks shall be located at a distance between 1/12 and 2/5 of the design ship's beam from the keel alignment, extended over the length of the keel (0.85 · L).

For ships of low tonnage (<100,000 DWT) the minimum dry dock load distributed in three or more odd alignments shall be considered, as long as the keel alignment receives 70% of the total load. In this case the lateral alignment of blocks shall be extended over a length of between 0.5 · L and 0.7 · L.

d) SLIPWAY AND SHIPBUILDING BERTH LOADS

Slipway and shipbuilding berth loads are loads generated by the ships during their dry docking for repair, construction or maintenance, by means of hauling or pulling along sloped planes, whether or not they are provided with complementary means to facilitate sliding (rails, travelling carriages, rolling systems, blocks, etc).

The loads on ramps, rails, travelling carriages, blocks and hauling chains, shall be determined in magnitude, application and distribution, by taking the following factors into consideration :

— Type of slipway

  Two types of slipways shall be distinguished in terms of the hauling direction in relation to the longitudinal ship axis :

  - Longitudinal haul slipway
  - Side haul slipway

— Operational or use criteria of slipway

  The maximum value of the slipway loads shall be limited in terms of the type and displacement of the design ships in the loading condition established in the installation use criteria. Lacking defined use criteria; the following shall be considered :

  - In normal operation conditions :
    Ballast ships in slipways for ship construction
    Lighted ships in slipway for ship repair and maintenance

  - In exceptional conditions :
    Fully loaded ships in slipway for repair or maintenance

  The greater displacement design ship does not always transmit the largest unit loads. However, the displacement of the largest ship that can be put in dry dock, in the load conditions established in the use criteria shall be adopted as a slipway's capacity index.

  Except for very special cases, the dry docking of ships larger than 10,000 t. displacement by means of hauling is not common.

— Typology and geometric characteristics of the different design ships in the use conditions.

  As a minimum, the following shall be set :

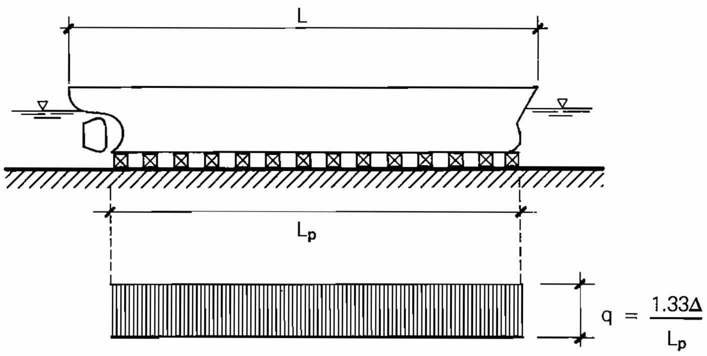
  - Maximum displacement
  - Weight distribution
  - Maximum length
  - Maximum keel length
  - Maximum beam
  - Maximum stern draught

— Typology and geometric characteristics of the slipway bed : sloped plane and complementary elements for hauling.

  As a minimum, the following shall be set :

  - Slope and length of ramp
  - Type, dimensions, weight, stiffness and wheel type of the travelling carriage
  - Length and number of rails
  - Disposition and inclination of the blocks

**TABLE 3.4.2.3.5.16 MINIMUM DRY DOCK LOADS IN TERMS OF THE SHIP DISPLACEMENT. UNIQUE ALIGNMENT. KEEL BLOCKS.**




**LEGEND :**

q : Vertical lineal uniform load.
Δ : Displacement of the largest size and displacement design ship in the following conditions :

- In normal operating conditions :
  - Ballast ships for shipbuilding dry docks
  - Minimally lighted ships for ship repair or maintenance dry docks
- In exceptional conditions :
  - Fully loaded ships for repair or maintenance docks

$L_p$ : Keel length
Lacking other data, it shall be approximated as 0.85 L.

**APPLICATION CONDITIONS :**

The minimum dry dock loads shall be applied in the symmetry axis of the dry dock. If the dry dock has sufficient width or length, multiple dispositions of lower displacement ships positioned laterally and/or longitudinally parallel to the symmetry axis of the dock, with symmetrical and asymmetrical location shall be considered in the design.

For longitudinal shipbuilding berths, the following shall be adopted as minimum characteristic :

- Slope and length of the ramp : It is preferable to design the ramp with a unique slope, generally between 1/6 and 1/12. However, ramps with greater slopes in the submerged zones may be designed, due to the fact that less pulling force of the travelling carriage is needed in these zones.
  The ramp length shall not be less than 2 times the length of the largest design ship. Generally, the minimum rail length ($L_c$) increased by 30 m shall be adopted.
- Travelling carriage dimensions : These shall be set in terms of the maximum length, beam and draught of all the design ships, and also in terms of the maximum height foreseen for the stern keel blocks. Margins of 4.5 m on each longitu-

dinal end, with respect to the greatest length between perpendiculars among the design ships ($L_{pp}$), and of 1.5 m on each transversal side, with respect to the largest beam (B), shall be maintained.

The travelling carriage shall be designed so that its highest point remains dry in the mean high water level (0.6 . HWSL), with a minimum freeboard of 0.30 m.

- Rail length ($L_c$) : The minimum rail length shall be equal to :

$$L_c = L_{pp} + (1/s) \cdot h$$

given that :

$L_{pp}$ : Length between perpendiculars of the largest design ship
s : Slope of ramp
h : Travelling carriage height = maximum draught + maximum height of the keel blocks upon rails.

If the wheel system is upon rollers, 2 times the diameter shall be added to the minimum length.

- Disposition of the blocks : The keel blocks usually correspond with the central launchway and the central part of the travelling carriage, while the bilge blocks correspond with the lateral launchways and lower part of the travelling carriage.

— Tides and other water level variations.

The slipways are generally designed allowing for the dry docking of the largest design ship in high water level, with the travelling carriage centered with the ship's center of gravity in this water level condition.

Lacking other use conditions, the mean high water level (0.6 · HWSL) shall be adopted as the water level for the determination of slipway loads.

— Environmental actions : Wind and currents.

When the ship is in the dry docking process or in dry dock, it shall be subject to additional loads due to the action of winds and currents. The action of wind upon a ship that is completely on dry land, as well as the action of wind and currents when the ship is still afloat in the beginning of the dry docking maneuver at the time when the mooring lines (breast lines) have been made fast to the travelling carriage shall be considered.

The evaluation of the external forces upon the ship and their transmission to different elements may be carried out according to paragraph b) Mooring Loads, of this section. In the work hypothesis for slipway maneuvering, increases of the mooring pulls of 100%, with respect to those obtained in the calculations must be taken into account, in order to evaluate the dynamic impact loads originated by the ship movements during this phase.

Lacking other criteria, the following value shall be adopted as the operating condition limit for the start of the slipway maneuver :

$$V_{1min} = 11 \text{ m/s} \qquad (40 \text{ km/h})$$

given that $V_{1min}$ is the mean wind velocity that corresponds to one minute gusts.

Operating condition limits shall not be considered when the ship is put on dry land.

The action of environmental loads shall generally be critical for the stability of all the additional elements that intervene in the slipway (travelling carriages, blocks, etc).

Given the complexity of the analysis, and lacking more detailed studies, the following simplifications shall be admitted for the evaluation and distribution of the static loads upon ramps, rails, travelling carriages and hauling chains :

$d_1$) LOADS UPON LONGITUDINAL HAUL SLIPWAYS

For the determination of vertical load, three characteristic ship positions during the sliding upon a sloped plane shall be considered :

A. First contact of the ship with the travelling carriage at the position of the bow blocks.

B. Keel completely supported by the blocks with reactions of different values, almost zero in the lower blocks and maximum in the upper blocks.

C. Ship completely on dry land with theoretically uniform reactions along the length of its support.

In accordance with the described process, the maximum slipway loads shall take place near the water line at a distance from the first contact point, toward land, which varies according to the design ship conditions.

Although the reaction line is not straight, the simplified longitudinal envelope of vertical linear loads in tables 3.4.2.3.5.17, may be adopted for rigid travelling carriages (single frame) that allow uniform load distributions on the rails. This load system shall be applicable for the blocks, travelling carriages, rails and ramp calculations; with the vertical and horizontal load's transversal distribution criteria also given in this table.

To calculate the maximum pull necessary for the design ship's hauling, the following formula may be used:

$$T = W \cdot \sin \phi + W \cdot C$$

given that :

T : Maximum pull
W : Total weight of the largest design ship in the use conditions + travelling carriage weight + tractor and hook up weight.
$\phi$ : Ramp slope angle
C : Friction coefficient. Lacking other data, the following can be used :

0.03 for travelling carriages upon rollers
0.04 for travelling carriages on wheels.

$d_2$) LOADS UPON SIDE HAUL SLIPWAYS

To determine the loads upon side haul slipways, the longitudinal weight distributions of the design ships and the weight of the travelling carriage and other additional elements shall be taken into account, assuming uniform transversal distributions along the length of the ship's beam.

When the project allows for the installation of several travelling carriages with simultaneous movement, each one of them shall absorb part of the corresponding ship's weight. A uniform distribution of the loads on the rails may be assumed.

As an approximation, in the absence of the design ship's weight distribution tables, the load corresponding to the uniform longitudinal distribution of 1.33 x (Largest design ship's displacement), distributed over the whole surface of the travelling carriage, may be assumed.

The effect of the action of the external environmental forces upon the ship may be taken into account, assuming trapezoidal increments in the uniform distribution of the loads on the travelling carriages and rails.

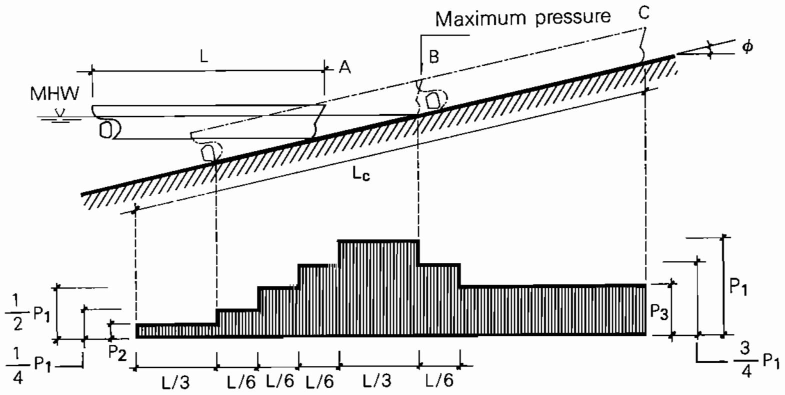
### 3.4.2.4 ENVIRONMENTAL LOADS ($Q_{Mk}$)

■ DEFINITION

Environmental loads are defined as those due to the direct action of natural, climatic or environmental phenomena upon a resistant structure or upon non-structural elements that affect the structure, independently of the fact that they can also affect or influence the determination of other variable or accidental loads (e.g. in mooring or berthing loads, cargo handling equipment or installation loads, thermal loads, etc).

TABLE 3.4.2.3.5.17 **LOADS ON LONGITUDINAL HAULING SLIPWAYS IN FUNCTION OF MAXIMUM DESIGN SHIP DISPLACEMENT (FOR RIGID TRAVELLING CARRIAGES)**

1. LONGITUDINAL DISTRIBUTION OF VERTICAL LOADS




L : Total Length
$L_c$ : Rail length

$P_1$ : $\begin{cases} \dfrac{\text{Largest design ship displacement}}{8}\text{, in t/m. For displacements > 2,000 t.} \\ \dfrac{\text{Largest design ship displacement}}{3}\text{, in t/m. For displacements} \leq \text{2,000 t.} \end{cases}$

$P_2$ : Submerged unit weight of the travelling carriage (t/m), assumed linearly distributed

$P_3$ : 1.33 x (Ship's displacement + travelling carriage weight) dry, assumed linearly distributed

II. TRANSVERSE DISTRIBUTION OF VERTICAL AND HORIZONTAL LOADS.

- ON RAILS

  - 2 rails :

    (1) Half of the vertical load on each rail
    (2) Half of the vertical load on each rail + horizontal wind load on one of the rails.

  - 3 or more rails :

    (1) Total vertical load on the keel rails.
    (2) Total vertical load on the keel rails + horizontal wind load on the lateral rails of one side.
    (3) Half of the vertical load on the keel rail + 1/4 of the vertical load on the lateral rails of each of the sides.
    (4) Same as (3) + horizontal wind loads on the lateral rails of one side.

- ON BLOCKS

  Same as that for 3 or more rails.

The effect of the environmental actions in the evaluation of other variable or accidental loads and the applicable action conditions are analyzed in the corresponding sections of these loads, taking into account the load combination hypotheses given in Part 4. Calculation Bases.

They can be differentiated as :

— Wave actions ($Q_{M1}$).
— Current actions ($Q_{M2}$).
— Actions due to tides and other water level variations ($Q_{M3}$).
— Wind actions ($Q_{M4}$).
— Actions due to atmospheric pressure ($Q_{M5}$).
— Actions due to air and water temperature ($Q_{M6}$).
— Actions due to precipitation ($Q_{M7}$).
— Snow and ice actions ($Q_{M8}$).
— Seismic actions ($Q_{M9}$).

■ DETERMINATION

The characteristic values of the environmental actions shall preferably be determined based on statistical data that refers to the parameters which constitute the thysical origin of the action (e.g. mean wind velocity determined in a time interval for wind actions, or the wave height for the action of waves) or the action itself, for each one of the project phases and work hypotheses considered.
Only in those cases where a sufficient and reliable data base is not available, may empirical methods or models developed for the forcast and quantification of these actions be adopted. For normal operating conditions, the characteristic values of the environmental actions shall be defined by the operating or use limit conditions of the analyzed installation.

The general criteria for the determination and combination of these actions (mainly determination models of the representative values of the actions and risk criteria), are assigned in section 3.2.3. Representative values of variable loads.

Due to the importance of the environmental loads in the design of the maritime and port works and the necessary extent of the description and definition of parameters of the natural phenomena causing these actions and the actions themselves, specific Recommendations are developed for these types of actions. These Recommendations are:

— ROM 0.3 Recommendations for the consideration of environmental variables/I : Wave action, currents, tides and other water level variations.

— ROM 0.4 Recommendations for the consideration of environmental variables/II : Atmospheric and seismic variables.

In these Recommendations, all the necessary data for the complete definition and estimation of the phenomena causing environmental actions are included as well as the development of the criteria for their determination, the factors to take into account, and the application conditions.
Furthermore, in those cases where there is sufficient and reliable data, extreme values or exceedance distributions and charts of the action parameters corresponding to preset return periods are included.

### 3.4.2.5 DEFORMATION LOADS ($Q_{Dk}$)

■ DEFINITION

Deformation loads are direct or indirect actions originated by phenomena capable of generating internal forces in the cross sections of structural elements as well as longitudinal deformations upon imposing deformations in the resistant structure. They shall be, therefore, mainly a function of the strength characteristics, typology, and deformation characteristics of the structure, and the nature of the phenomenon that causes the deformation.

They shall be differentiated as :

— Prestressed load ($Q_{D1}$)
— Thermal loads ($Q_{D2}$)
— Rheologic loads ($Q_{D3}$)
— Loads from imposed movements ($Q_{D4}$)

■ DETERMINATION

The actions produced by imposed deformations shall be determined for each project phase and work hypothesis based on the guidelines given in the specific Codes for the calculation for structures according to their construction material. The following factors shall be taken into consideration :

— Strength and deformation characteristics of the construction materials of the resistant structure in the project conditions.
— Dimensions of the resistant structure
— Typology characteristics of the structure : isostatic or hyperstatic conditions.
— Possibilities of free deformation of the structure (e.g. precautions taken in the construction phase, existence of joints, etc).
— Nature of the phenomena causing the imposed deformations : causing agent and form of introduction (e.g. prestressing by means of cables, wires or tendons; thermal gradient, etc).
— Local physical and environmental conditions : humidity, sun exposure, water and air temperature, etc).
— Time elapsed since the construction of the structure and duration of the phenomenon capable of generating loads.

The effects resulting from the deformations imposed on the isostatic structures (those where the deformations in the structure are not impeded by internal or external coactions), or hyperstatic structures where the compatibility of the deformations is not taken into account, shall be considered primary effects. The loads associated with these effects can be direct actions (e.g. prestressing forces by means of cables or tendons) equivalent to the system of external forces, which causes the imposed deformation, or indirect actions (e.g. thermal gradient or rheologic loads).
In hyperstatic structures, additional loads due to the compatibility of the total deformation and the internal and external coactions of the structural system shall be combined with the primary effects. These effects shall be classified as secondary effects. The loads associated with them, mainly reactions in the external supports, shall be considered indirect.

The indirect deformation loads shall be taken into account only in the following cases :

— In the calculation of the structures in serviceability limit states.
— In the calculation of hyperstatic structures in the ultimate limit states in those cases where attenuating elements for the phenomena capable of generating these loads (e.g. dilation/contraction joints, construction methods, etc) have not been included.
— In the calculation of isostatic and hyperstatic structures in those ultimate limit states, where the second order effects (e.g. buckling ULS) are important; when the imposed deformations are very important in relation to the deformability of the resistant structure; or when the imposed deformations alter the magnitude of the acting loads (e.g. in concrete structures, reductions in the prestressing forces due to rheologic deformations of the materials).

a) PRESTRESSING LOADS ($Q_{D1k}$)

Prestressing loads shall be considered distinct actions artificially created through the intentional, direct or indirect introduction of deformations on the resistant structure. They are, among others :

— Prestressing of concrete, by wire, cable or tendons
— In composite concrete-steel structures, preflexing of the metallic element before its interconnection with the concrete elements
— Application of jacks
— Application of provisional loads
— Modifications in the supports
— Resistance of the supporting devices against the movement of the supported parts
— Use of expansive cements

The prestressing loads shall be direct or indirect actions or a combination of both, depending on the typology of the resistant structure and the prestressing procedure used. Direct loads shall be assimilated to a system of external forces, related to the way the deformations were introduced, acting upon the resistant structure considered free from the element causing the deformation.
In the particular case of concrete prestressing through active reinforcement, the external force system that this reinforcement transmits to the structure that supports it shall be composed of :

— Forces concentrated in the anchoring of the tendons
— Forces perpendicular to the tendons, resulting from their curvature and changes of direction
— Tangential forces due to friction and adherence

In hyperstatic structures, the forces and reactions in the supports caused by the compatibility of the imposed deformations with the internal or external coactions of the structural system shall also be taken into account.

The characteristic values of the prestressing loads in each section of the analyzed structure and project phase considered shall be determined taking into account the way they were introduced and the possibilities of deformation of the structure. It shall also be taken into account that the initial prestressing states shall change immediately and through time, due to the occurence of elastic or rheologic deformations in addition to the intentionally imposed deformations. The prestressing loads in each section shall vary in an even way over time in order to make the total deformation at each point compatible with the internal or external coactions of the structural system operating as a geometrically continuous unit.

Therefore, it shall be necessary to consider, in the majority of cases, two characteristic values of the prestressing loads, one maximum and the other minimum, linked to the evolution of the action over the course of time. It is usually sufficient to consider the maximum characteristic value as the value of the prestressing load in the instance $t = 0$ of the analyzed phase, and the minimum characteristic value as that of the prestressing load in the final moment of the studied design phase (in practice $t = \infty$ for the service phase).

All these effects shall be calculated based on the principles of analytical mechanics, together with the theories of strength of materials and elasticity, following the criteria included in the specific instructions and Codes in effect (Code for the design and construction of prestressing concrete works, EP-80; Eurocode nº 2, concerning concrete structures; Eurocode nº 4, concerning composite concrete-steel structures; Codigo Modelo CEB-FIP; etc) or in lieu of these, in handbooks, catalogs or specialized bibliography.

In the case of concrete prestressing through wires, cables or tendons, it shall generally be sufficient to distinguish for each section the following maximum and minimum characteristic values of the direct prestressing loads :

— Maximum characteristc value in the section x in the instance $t = 0$, or instantaneous characteristic value.

$$P_{k0}(x) = P_0 - \Delta P_0(x)$$

— Minimum characteristc value in the section x for $t = \infty$

$$P_{k\infty}(x) = P_0 - \left[\Delta P_0(x) + \Delta P_{t=\infty}(x)\right]$$

given that :

$P_0$ : Initial prestressing forces ($t = 0$) in the origin ($x = 0$)
$\Delta P_0(x)$ : Instantaneous losses of the prestressing forces in section x (friction, anchor wedge penetration, and instantaneous concrete shortening)
$\Delta P_{t=\infty}(x)$ : Deferred losses of the prestressing forces for $t = \infty$ (rheologic effects: shrinkage and creep of the concrete, and steel stress relaxation).

b) THERMAL LOADS ($Q_{D2k}$)

These are indirect loads due to the deformation of the construction materials of structures, mostly expansion and contractions, caused by :

— Variations in the temperature of the structure during the analyzed phase, referring to the temperature of the joint closure, or during the construction phase.

— Existence of different temperatures in diverse zones of the structural element (thermal gradients).

The characteristic values of the thermal actions shall be determined based on the thermal properties of the construction material of the resistant structure and the characteristic variations in the structure's temperature, following the specific criteria in the Codes corresponding to the construction materials of the resistant structure.

The characteristic variations of the temperature to adopt in the calculation of thermal loads shall depend on the following factors :

— Thermal inertia of the structure
— Climatic conditions in the location of the structure
— Location of the structure (emerged, submerged, or buried)
— The structure's dimensions
— Variation in outer water temperature
— Exposure or not to solar rays
— Resistant structure's protection against inclement weather (coverings)
— Possibility of artificial temperature variations in different structure zones (e.g. temperature of the handled or stored cargo).

Lacking other data, the other representative values of the thermal loads (combination, frequent and quasi-permanent values) shall be obtained through the coefficients employed for environmental actions (see section 3.2.3.2).

As for load combinations, in most cases, it shall not be necessary to consider thermal loads as variable actions with predominant effect.

In general, thermal loads can be disregarded in the calculation whenever expansion joints of sufficient width are placed at the appropriate distances for the structure type projected.

— *THERMAL EXPANSION COEFFICIENTS*

The characteristic property of the materials, for the determination of the deformations imposed by thermal variations is the thermal expansion coefficient ($\alpha$). The following values shall be adopted :

- Normal concrete elements (unreinforced, reinforced and prestressed) : $10^{-5}$ m/m°C.
- Light-weight concrete elements : $0.8 \cdot 10^{-5}$ m/m°C
- Steel elements : $1.2 \cdot 10^{-5}$ m/m°C
- Steel elements in composite concrete-steel structures : $10^{-5}$ m/m°C

— *CONSTRUCTION TEMPERATURE VARIATION*

The temperature of the structure shall depend upon the external air and water temperatures at the site, its exposure to sunlight, the thermal inertia of the construction material with respect to the atmospheric temperature, and the structure's protection against weather.

The amplitude of the thermal variations shall be obtained based on the maximum and minimum average monthly temperatures, for the air as well as the water, according to the evaluation criteria of variable actions through statistical data (see section 3.2.3). In normal operating conditions, an external temperature variation range, corresponding to mean values of the mean monthly temperatures of the coldest and hottest months of the year, respectively, for observation periods of several years may be adopted. For the service phase in extreme conditions and for the construction phase, the maximum and minimum average monthly temperatures corresponding to preset return periods shall be considered, according to the risk criteria and action combination criteria established in this Recommendation.

The determination of average and extreme values can be carried out using the data in ROM 0.4. Recommendations for the consideration of environmental variables/II : Atmospheric and seismic conditions.

In most cases, the simplification of calculating the thermal loads considering the following virtual temperature increments in the structure expressed below, shall be admissible. These temperature increments refer to the air or water temperature foreseen at the moment of joint closure, or the average temperature during the construction phase:

— *In structures exposed to weather*

Except when specially justified, a global characteristic thermal variation in the construction ($\Delta\theta$) shall be considered, not less than :

- Metallic structures

  Temporarily or permanently emerged elements, exposed to direct solar radiation on any of its faces.

  $\Delta\theta_{min} = \Delta T_a - 5,$ : in °C
  $\Delta\theta_{max} = \Delta T_a + 20,$ : in °C

  Submerged elements

  $\Delta\theta_{min\ or\ max} = \Delta T_w,$ : in °C

- Concrete structures

  Temporarily or permanently emerged elements, exposed to direct solar radiation on any of its faces.

  $$\left.\begin{array}{ll}\Delta\theta_{min} = \Delta T_a - (10 - 0.75 \cdot \sqrt{e}), & \text{in °C} \\ \Delta\theta_{max} = \Delta T_a + (10 - 0.75 \cdot \sqrt{e}), & \text{in °C}\end{array}\right\} \text{for } (10 - 0.75\sqrt{e}) \nless 0$$

  Submerged elements

  $\Delta\theta_{min\ or\ max} = \Delta T_w,$ in °C

- Composite concrete-steel structures

  In composite concrete-steel structures, different characteristic thermal increments for concrete and steel shall be considered, according to the aforementioned criteria for concrete and metallic structures, respectively.

  given that :

$\Delta\theta_{min}$ : Maximum virtual temperature decrease of the structure, applicable for the calculation of thermal loads in reference to the atmospheric temperature at the moment of joint closure or during the construction phase (mean monthly temperature).

$\Delta\theta_{max}$ : Maximum virtual temperature increase of the structure, applicable for the calculation of thermal loads, in reference to the atmospheric temperature at the moment of joint closure or during the construction phase.

Lacking other data, the average temperature that corresponds to the hottest or coldest month shall be adopted as temperature at the moment of joint closure, according to the time of year it is occures.

$\Delta T_a$ : Maximum decrease or increase of the water temperature (monthly mean in the aforementioned conditions in this paragraph) in reference to the atmospheric temperature at the moment of the joint closure or during the construction phase, in °C.

$\Delta T_w$ : Maximum decrease or increase of the water temperature (monthly mean in the aforementioned conditions in this paragraph) in reference to the water temperature at the moment of the joint closure or during the construction phase, in °C.
Lacking more precise information, the external temperature variation range in Spanish costal zones given in table 3.4.2.5.1 may be admitted; applicable for normal operating conditions as well as for extreme conditions.

e : Fictitious thickness of the considered element, in cm. The fictitious thickness is defined as : e = (2A/u); given that A is the area of the section of the element, and u is the perimeter of the same section. In surface elements, the fictitious thickness shall approximately coincide with its real thickness.

If the structure is made up of elements with different thicknesses, to simplify the calculations a tolerance of ± 5°C in the resulting values of the virtual temperature increments shall be assumed.

— *In structures protected from weather*

Except where there is special justification, in elements protected from weather, global characteristic thermal variations with values of 50% of those defined for exposed structures shall be considered.

In order to determine thermal loads in buried concrete elements, the thickness of the soil layer that isolates them can be included as the structure thickness; in which case the thermal effects shall be evaluated according to the definition for structures exposed to the weather.

For structures with protection or covering that restrict temperature variations in the structure to no more than ± 10°C, thermal loads, in general, can be neglected.

In those structures formed by elements in different conditions of exposure to solar action, it shall be necessary to consider differential thermal variations of 5°C between those elements.

— *THERMAL GRADIENTS*

The effects caused by the existence of thermal gradients in those cases where they may occur shall be superimposed upon the resulting thermal actions, considering the mean variations in temperature of the analyzed structure. For the study of the effects of the differential variations in temperature, a constant gradient can generally be assumed.

For the determination of thermal gradients, the mean daily atmospheric temperatures on the external surfaces of the structure, obtained by following the evaluation criteria of variable actions through statistical data (see section 3.2.3) shall be considered. Moreover, in those elements where certain parts can be subject to direct sun action, it shall be necessary to consider, except where specially justified, a temperature difference between the hottest and coldest part of 15°C for metallic structures and of 10°C in concrete ones without modifications in the average temperature on the surfaces. The same differential temperature variation shall be considered in those cases in which, due to different conditions of protection, elements that heat or cool in a different manner can exist.

**TABLE 3.4.2.5.1 ATMOSPHERIC TEMPERATURE VARIATION RANGES IN SPANISH COASTAL ZONES, APPLIED TO DETERMINE THERMAL LOADS**

| ZONE | TEMPERATURES (in °C) | |
|---|---|---|
| | WATER | AIR |
| North and Galicia | 7-20 | 13-20 |
| South Atlantic | 10-27 | 15-20 |
| Canary Islands | 15-25 | 17-23 |
| South Mediterranean | 12-25 | 15-22 |
| Levante | 10-25 | 13-25 |
| Catalonia | 6-25 | 13-25 |
| Baleares | 10-25 | 13-25 |

In composite concrete-steel structures, if a detailed study of the real thermal gradient is not performed, it may be assumed that in the concrete element a gradient is established between the global characteristic temperature variation in the metallic element ($\Delta\theta_{steel}$) and the global characteristic temperature variation in the concrete element ($\Delta\theta_{concrete}$). With the same criteria, a thermal gradient in the metallic element shall be established.

— *ARTIFICIAL TEMPERATURE VARIATIONS IN CONSTRUCTIONS*

The thermal effects produced by artificial temperature variations in the structural elements (e.g. due to the temperature of handled or stored cargo) shall be analyzed associated respectively to the maximum and minimum mean monthly atmospheric temperatures for the evaluation of global temperature variations, and to the mean daily temperatures for the evaluation of thermal gradients.

c) RHEOLOGIC LOADS ($Q_{D3k}$)

Rheologic loads are indirect loads originated in those structures whose construction materials are deformed or vary dimensionally over the course of time due to phenomena such as shrinkage or creep due to constantly applied loads or other causes.

The characteristic values of rheologic loads in each analyzed project phase shall be determined taking into account the criteria and guidelines of the specifications and Codes applicable to the calculation of structures made of these materials.
As rheologic actions vary continuously over time, it shall be necessary to consider, in the majority of cases, two characteristic values of the rheologic loads : one maximum and the other minimum, taken from the action evolution over the course of time. It shall be sufficient to consider the value of the action in the moment $t = 0$ in the analyzed phase as a minimum value, and the maximum as the value in the final moment of the studied project phase (in practice $t = \infty$ for service phase).

Rheologic actions shall be, in general, negligible in metallic material, but they must be considered in reinforced, prestressed and unreinforced concrete. The effects due to rheologic deformations shall be especially relevant in prestressed concrete structures, for the determination of the characteristic values of prestressing loads, and in concrete structures in general for the consideration of the influence of possible readaptations of the resistant structure due to these effects in the determination of other deformation loads.

— *LOADS DUE TO CONCRETE SHRINKAGE DEFORMATIONS*

Concrete shrinkage is defined as the shrinkage of the concrete ($\varepsilon_t$) during the time it takes to harden. It shall be a function of : the degree of atmospheric humidity, the thickness or least dimension of the structural element, the composition of the concrete, the quantity of reinforcement, and the time passed since concrete placement.

Lacking more detailed studies, the following values of the final concrete shrinkage deformation may be assumed :

— *In Mediterranean Zones :*

0.35 mm/m for normal concrete
0.50 mm/m for lightweight concrete

— *In Atlantic Zones :*

0.20 mm/m for normal concrete
0.30 mm/m for lightweight concrete

For submerged or buried structures, actions due to shrinkage may be neglected.

— *LOADS DUE TO CONCRETE CREEPING DEFORMATIONS*

Concrete creep is defined as the increase, over time, of the relative deformations due to constantly applied loads (permanent and quasi-permanent loads). They shall be a function of : the atmospheric humidity, the thickness or least dimension of the structural element, the age of the concrete when it is loaded, the magnitude and duration of the applied load, and the time passed since it was put under load.

Lacking more detailed studies, in the Spanish coastal zones, it shall be assumed that the total deformation corresponding to the constantly applied loads (instantaneous deformation plus deferred deformation) is equal to two times the instantaneous elastic deformation.

d) IMPOSED MOVEMENT LOADS ($Q_{D4k}$)

These are indirect loads originated by differential settlements or displacements of the resistant structure's supports, or by other movements intentionally imposed upon it.

The characteristic values of the loads from settlements shall be determined based on the differential displacements that are foreseen according to the geotechnical study, in accordance with Soil Mechanics theories. The type of foundation, the loads transmitted to the soil, and the typology of the structure during the construction process (particularly if the structure goes through an initial isostatic phase) shall especially be taken into account. Likewise, the evolution of the movements imposed by readaptations of the resistant structure for rheologic reasons, for the soil as well as for the structure, shall be taken into account in those cases where the imposed movements create a favorable effect.

### 3.4.2.6 CONSTRUCTION LOADS ($Q_{Ck}$)

■ DEFINITION

Construction loads are transitory or residual actions due specifically to the different manufacturing, assembly or mounting processes of the resistant structure and its elements during the construction phase. These actions can impose temporary load states in the resistant structure or permanent loads, if they are created by modifications in the structural schemes or in the supports during this phase. In this last case, the construction loads might be maintained or occur repeatedly during the design life of the structures (e.g. load eccentricities, misalignment in the assembly, off plumb, etc).

In any case (determination of representative values of the loads based on the characteristic value, action combination criteria, etc), actions that act upon the structure during the construction phase, not directly caused by the structure construction method shall not be considered construction loads, in spite of their differences in quantity and typology with the equivalent actions applicable to the service phase (e.g. use overloads or environmental loads in the construction phase).

They shall be differentiated as :

— External loads during manufacture ($Q_{C1}$) (e.g. in the dry construction of a floating caisson, the reactions in the support blocks)

— Exterior loads during transport ($Q_{C2}$) (e.g. pulling forces and pressures upon a floating caisson during the towing process to its final position, or to the waiting zone)

— External loads during installation ($Q_{C3}$) (e.g. differential hydrostatic pressures between floating caisson cells during its placement)

— Other external loads ($Q_{C4}$) (e.g. loads induced upon the structure in the construction phase by simultaneous soil treatments : earth pressure modification, vibration, etc).

■ DETERMINATION

The characteristic values of the construction loads shall be determined in terms of the construction and placement methods of the analyzed structure foreseen in the project.
Prefabricated construction cases shall be the object of special consideration, for which the maximum admissible placement tolerances that have been considered in the calculations shall be included in the project.
If, at the moment of execution of the work, the construction procedure is modified, its effect on the evaluation of the acting loads and therefore, on the design of the resistant structure shall be analyzed.

The project Engineer shall take into account all the actions that can act during the different phases of construction, whenever these actions give rise to loads which are higher than or of opposite sign from the in service actions; adopting the resistant scheme corresponding to the analyzed construction phase.

Given the great diversity of maritime and port works, and of construction phases and methods, the quantification and application conditions of the construction loads shall be lacking more precise information, according to the Project Engineer's criteria. In subsequent Recommendations regarding specific structural typologies, a more detailed analysis of these loads shall be performed.

### 3.4.3 ACCIDENTAL LOADS ($A_k$)

■ DEFINITION

Accidental loads are defined as loads of fortuitous or abnormal character that can occur as a result of an accident, human error, misuse, or exceptional work or environmental conditions. Accidental actions shall be considered, therefore, as variable actions with small occurance probability or that are applied for short durations during the design life of the structure.

The particular project phase, where the occurance of an accidental action upon the resistant structure is considered, shall be called the Service Phase in Exceptional Conditions (S3). The complete structure, and each one of its elements, shall be analyzed in this work condition whenever accidental actions compatible with the analyzed structure can occur, and its performance is relevant applying the action combination criteria given in Part 4. Calculation Bases.

In the absence of other criteria, the performance of accidental actions in non-service phases shall not be considered.

■ DETERMINATION

Accidental loads to consider in the design, and their characteristic values ($A_k$), shall be chosen by the Project Engineer, Client or Government Authority as those above which the survival probability of the structure is not assured, without disregarding the minimum loads set by these Recommendations or other general Codes that are applicable (e.g. Seismic Code PDS-1, Eurocode nº 8, corresponding to structures in seismic zones, etc).
In practice, loads due to extraordinary assumptions corresponding to actions that are difficult to foresee shall be disregarded. Generally, those actions corresponding to extreme values whose probability of exceedance in a year is less than $10^{-4}$ (average return period T = 1000 years), shall be disregarded, unless the return period selected for the determination of the characteristic value of the equivalent variable load in extreme conditions is greater.

Lacking specific criteria, accidental actions quantified in these Recommendations shall be included.

Generally, accidental loads shall be presented as variable loads, with differentiated values for service phases and exceptional work conditions. In the sections of this Recommendation that refer to each one of the variable actions and in the Recommendations 0.3 and 0.4, the differentiation of the variable loads, according to project phase is presented; defining, therefore, the characteristic values of the variable actions in exceptional conditions. These actions shall be considered in all respects as accidental actions.

The occurance of localized damage in the analyzed structure can be admitted as a consequence of the performance of accidental loads, as long as the ultimate limit state of a progressive collapse, and the serviceability limit state of permanent damage (see section 4.1.3) are checked.

The main accidental loads to consider in the design of maritime works shall be due to the following causes :

— *LOAD TESTS*

Overloads caused by tests and trials required by the legislation in effect that is applicable to each installation, or by those specifically included in the project shall be considered accidental actions.

In particular :

- In installations assigned for liquid bulk cargo with a specific weight less than 1 t/m$^3$, the overloads produced by filling of water during its hydraulic test shall be considered in the calculations.

- Overloads produced by cargo handling equipment during load tests shall be considered in the calculations. For heavy lifting equipment with restricted movement, the load tests indicated in the UNE Code 580-107-72 shall be adopted. Principally the test that consists of suspending a load equal to 150% of the nominal load, with maximum reach in calm wind conditions shall be adopted. The load test shall be carried out, one crane after another, with the rest of them in non-operative positions.

— *INUNDATIONS DUE TO THE RUPTURE OF CANALIZATION OR DEPOSIT*

In earth retaining structures, the possibility of exceptional phreatic levels in the backfill because of inundations of the shore, liquid pipe or conduit rupture in the structure's back, or other similar occurances shall be considered.

When liquid pipes or conduits are located in the back of earth retaining structures, the additional hydrostatic and earth pressures produced by the continuous spilling of the channelled liquid over 24 hours shall be adopted as accidental loads.

When liquid storage deposits exist, the additional pressures produced by possible inundations caused by deposit failure shall be considered as accidental loads, in accordance with the established evacuation conditions, for this hypothesis, in the specific storage regulations for the considered product.

— *DRAINAGE SYSTEM OR SUBPRESSURE CONTROL FAILURES*

When the possible reduction of piezometric levels is taken into consideration in the evaluation of hydraulic and earth loads due to the establishment of drainage systems or subpressure control systems, the additional pressures due to the condition in which the drainage system or subpressure control system fail to function for 48 hours shall be considered as accidental loads.

— *ELEVATION OF THE PHREATIC LEVEL OF BALLASTED PROJECTS*

Independently of the tolerances admitted in the project for to the theoretical phreatic levels inside ballast retaining structures, the additional hydraulic and soil pressures due to the rising of this level up to the highest part of the compartment, or to the lowest level at which the fluid can freely overflow, shall be considered accidental loads.

— *SOIL INSTABILITY*

For cases in which buried structures retain or pass through potentially unstable soil masses (e.g. progressivly sliding slope, expanding soil, etc), the lateral earth pressures upon the structure caused by the soil at the moment of instability shall be taken into consideration as accidental loads.

Soil masses that have a sliding safety factor (F) < 1.30 in the service phase shall be considered potentially sliding, except in exceptional work conditions (accidental hypothesis). In this hypothesis or construction phase, the soil mass shall be considered as potentially sliding, if (F) < 1.10. The sliding safety factor (F) shall be obtained through the Bishop method, without considering the possible stabilizing effect of the structure.

For the particular case of narrow isolated structure alignments (e.g. piling) that go through potentially sliding soil masses, the lateral pressures created by the soil at the moment of instability shall be quantified according to $b_4$), section 3.4.2.2. Earth Loads.

— *DEPOSITS AND OVERDREDGING*

Unless the project considers maximum material deposit levels, or sets maintenance dredging thicknesses and admissible tolerances for them, the additional actions or the reductions of the minimum favorable loads due to possible deposits or overdredging in front of retaining structures or at the foot of slopes, with respect to the theoretical depth of the project shall be accepted as accidental loads.

This thickness of the deposits to consider for the determination of the corresponding accidental loads shall depend on, among other factors : the coastal evolution

at the site, the losses of handled bulk, the existence of spills, and the period of time between maintenance dredgings.
The magnitude of the overdredging shall depend upon the following factors : Type of soil and volume to dredge; depth; thickness of the deposits between maintenance dredgings : type and size of the dredgers; cost of the dredging work in relation to the thickness of the zone to be dredged; environmental conditions in the zone, primarily wave action, tides and currents; control instrumentation aboard the dredger; stability of the submerged slopes, etc.
Due to the difficulty of evaluating these factors, and in the absence of more detailed studies, the following be taken as minimum deposit, or overdredging thicknesses, with respect to the theoretical project depth :

| Theoretical project depth with respect to LWSL (in m) | Thickness (in m) |
|---|---|
| 6.00 | 0.50 |
| 10.00 | 0.80 |
| 15.00 | 1.10 |

— *SCOURING OR EROSION OF THE SOIL DUE TO SHIP PROPELLERS OR EXTRAORDINARY CURRENTS*

For those cases in which a real danger of scouring or erosion in the front of earth retaining structures or at the foot of slopes exists and elements of sufficient protection against these phenomena are not included in the project, the effects originated in the resistant structure by this scouring or erosion shall be considered as accidental actions. These effects can be produced by the ship's propellers during exceptional maneuvers, by strong currents or by the extraordinary flow and ebb of the water (e.g. in fine granular soils, ro-ro or ferry berthing in which the vessels berth and depart in exactly the same position; berthing projects situated at the mouth of fluvial currents, etc).
Unless detailed studies are carried out, one meter deep scouring shall be adopted in these cases.

This accidental load shall be especially significant for the checking of the ultimate limit state of progressive collapse.

— *COLLISIONS AND EXCEPTIONAL LOCAL OVERLOADS*

Actions due to direct collisions of cargo handling equipment or conventional traffic vehicles against structural elements shall be considered as accidental loads. Loads transmitted by handling equipment under the effect of a collision (e.g. of the load handled against the ship, of one equipment against another, etc) shall also be considered as accidental loads.
In those areas where there is cargo or material handling by cranes or other lifting equipment, collisions due to falling or placement, upon the surface, of cargo from this handling equipment shall be considered accidental loads. Likewise, loads due to the collision of drifting objects against flood protection structures, or against structures situated in the area of the mouth of fluvial currents shall be considered accidental loads.

Accidental loads due to direct collision of vehicles and handling equipment shall be applied to structural elements whenever the following assumptions are met :

— Unprotected structural elements are located less than 10 m away from the vehicle or unrestricted mobility equipment circulation zones, or from the end of the traffic bands of the restricted mobility equipment.

— Structural elements are protected with elastic barriers located less than 1 m away from the element, and are less than 10 m away from the circulation zones.

The effects of these collision can be assimilated to the action of a static horizontal load, whose resultant is 1.20 m above the surface of the pavement or the traffic route, applied upon a surface or shock zone not greater than 4 $m^2$, and with the following values :

— *For conventional and unrestricted mobility cargo handling equipment traffic*

- In unlimited circulation or unchanneled areas (operation and storage areas): 50 t. in any direction
- In channeled circulation areas (maneuvering and access tracks) : 100 t. in the direction of traffic and 50 t. in the direction perpendicular to the traffic,not acting simultaneously.

The possible lateral collision of the wheel of a conventional vehicle or handling equipment against a barrier or border shall be assimilated to a static horizontal load of 10 t, applied to the highest part and perpendicular to the considered element, distributed in a width of 0.6 m.

— *For restricted mobility cargo handling equipment*

Given the usual low velocities of restricted mobility cargo handling equipment, (approximately 0.4 m/s), accidental actions due to direct collisions of this equipment shall not usually be considered, nor shall local overloads transmitted by equipment to the resistant structure at the moment of collision.

The accidental loads due to objects falling from the handling equipment shall be assimilated to a static vertical point load equal to the maximum weight that might possibly be handled. This load shall never surpass 10 t.

The accidental loads due to drifting objects shall be assimilated to a static horizontal point load with a minimum value of 3 t. This load shall be applied taking into consideration the maximum water levels corresponding to the extreme value associated with the maximum admissible risk.

— *IMPACTS AND OVERLOADS DUE TO EXCEPTIONAL MANEUVERS OR OPERATING SITUATIONS OF THE DIFFERENT CONVENTIONAL CARGO TRANSPORT EQUIPMENT*

In order to take into account the extraordinary possibility of locomotives with greater tonnage than the usual cargo trains, circulating in port areas with rail traffic exclusive to these zones, the concentrated load trains included as mobile use overloads in the Railway Bridge Code (train types A and C) shall be considered as accidental actions. These accidental loads shall be applied according to the conditions adopted in these Recommendations for railway traffic overloads (see section 3.4.2.3.3.).

Likewise, in areas considered in the use plans, and consequently in the project, such as heliports or helisurfaces, a load equal to 2.5 times that corresponding to helicopters in normal operating conditions (without increasing due to dynamic effects) (see section 3.4.2.3.3. Traffic Overloads), equivalent to the impact produced in an emergency landing situation due to failure of the landing gear, shall be considered as an accidental action.

— *IMPACTS AND OVERLOADS DUE TO EXCEPTIONAL MANEUVERS OR OPERATING SITUATIONS OF THE DESIGN SHIPS*

Accidental situations such as mechanical ship or tug failures during berthing maneuvers, mooring line breakage, sudden changes in the environmental conditions or exceptional environmental conditions, human error, etc, can give rise to abnormal impacts of the ship upon the structures and fender systems during berthing, and overloads on the mooring points and equipment, and on the fenders, not previously considered in the calculation of the mooring loads.

The accidental berthing load shall be assimilated to an impact load (R), obtained by considering that the energy transferred to the berthing system shall be double that which is calculated for normal operating conditions (see section 3.4.2.3.5.a) Berthing Loads). Likewise, an action upon the resistant structure and the mooring equipment equal to 1.5 times that calculated for the mooring points according to the criteria in section 3.4.2.3.5.b) Mooring Loads, including the increase for dynamic effects shall be considered an accidental action. This consideration shall also be valid when minimum mooring loads are applied.

In order to guarantee the security of the resistant structure, and avoid damage to it from unforeseen events, the fender systems, the mooring equipment (mooring, bollards, bitts, etc) and their anchors, shall be designed in a way that their ultimate energy absorption capacity before breakage or failure coincides with that foreseen for the performance of the aforementioned accidental loads.

— *OVERLOADS DUE TO SHIP OPERATIONS IN EXCEPTIONAL LOAD CONDITIONS*

Foreseeing possible changes in the use conditions of a port installations, or exceptional ship operation conditions, the operation overloads originated by the design ships when in load conditions not considered in the installation's use criteria shall be considered accidental loads.

For example, in the case of a berthing site in which only loading operations with the ships arriving lighted are foreseen, accidental berthing actions would be those caused by the fully loaded design ship, in order to take into consideration the exceptional possibility that the loaded ship must return to the berth.
Likewise, in the case of slipways or dry docks in maintenance and repair installations, those loads due to fully loaded design ships shall be considered accidental dry docking or slipway loads.

— *WAVE OVERTOPPING*

Only in checking the ultimate limit state of progressive collapse, and the limit use state of permanent damage, in structures that can be designed with a characteristic wave height that does not correspond to the mean maximum wave height, shall the action of this maximum wave be considered as an accidental action.

— *ACTION AND OVERLOADS PRODUCED BY EXCEPTIONAL ENVIRONMENTAL CONDITIONS*

The variable environmental loads whose risk or occurance probability during the design life of the structure is less than that adopted for the determination of the characteristic values of these variable loads in extreme conditions shall be considered accidental loads. That is, environmental actions with values corresponding to return periods higher than those preset for variable actions (see section 3.2.3. Representative Values of Variable Actions). As an exceptional case, seismic loads shall be considered accidental loads, independently of the evaluation criteria used, due to their small durations of applications throughout the design life. Therefore, minimum seismic loads shall be considered accidental loads, associated with extreme values corresponding to the maximum admissible risk set in these Recommendations for variable loads (see ROM 0.4. Consideration of Environmental Variables/II : Atmospheric and Seismic Conditions).

Lacking other criteria, the extreme value whose probability of exceedance in a year is equal to $10^{-4}$ (mean return period T = 1,000 years) shall be adopted as the characteristic value of the exceptional environmental action, unless the return period set for the determination of the characteristic value of the equivalent variable load in extreme conditions is greater. In this case, the performance of exceptional environmental actions shall not be considered.

Overloads transmitted by restricted mobility cargo handling equipment or installations when work operations are stopped and the equipment is in a stowed position, and also by railway traffic under the effect of exceptional environmental conditions, shall be considered as accidental actions.

— *EXPLOSION*

Assuming that the stage, storage or handling of cargo with a risk of explosion is carried out in the work project area, and the collapse of the work could give rise to subsequent explosions or additional damages that could be even more serious than those caused by the explosion itself, additional overloads produced by the explosion wave impacts, by the fall or collapse of structures, or by materials thrown as a consequence of the explosion, shall be considered as accidental loads on the resistant structure. The magnitude and application conditions of these overloads shall be a function of the characteristics of the protection elements defined by the applicable Technical Code for the storage and handling of each potentially explosive material and of their clearance from the analyzed site.

— *FIRE*

Independently of the minimum resistance times of the structure to the thermal conditions or to standardized heating equivalent to fire (structural stability and integrity of the materials themselves or by the applied insulation) required in order to assure the integrity and capacity of the structure during the period of time that allows for the evacuation of occupants and the actuation of fire fighting systems, the following shall be considered accidental loads due to fire :

— Additional overloads upon nearby structures, or upon the part of the structure not affected by fire, as a consequence of the structural failures that are caused by the fire (e.g. falling impact).

— Overloads produced by the water used to extinguish the fire, which can represent a considerable action in the case of ensilaged cargo handling, due to the increase in hydrostatic actions, as well as in the case of cargo that retains high percentages of humidity.

The minimum resistance times of structures during fires shall be established by specific Codes and Instructions in function of the dimensions and use of each structure. Lacking that information, the Project Engineer, Client or the Government Authority shall establish them.

# SECTION 4 CALCULATION BASES

# PART 4 TABLES

## Index

# PART 4 CALCULATION BASES

## 4.1 GENERAL CALCULATION PROCESS

### 4.1.1 GENERAL

The objectives of all structural calculation shall be to guarantee in each one of the project phases that :

— The analyzed structure or structural element is capable of resisting all the normal actions that act upon it, with a determined level of safety; being appropriately durable in relation to the design life, and the projected maintenance programs.
— The behaviour of the structure fulfills the functional and operational criteria that determine the appropriate use and maintenance conditions.
— The structure is capable of resisting, with an acceptable level of safety, the performance of fortuitous or abnormal actions that can occur as a result of accidents, misuse, or extraordinary environmental conditions (Accidental Actions).

### 4.1.2 CALCULATION PRINCIPLES

The general calculation process that is proposed in this Recommendation to verify that the objectives stated in 4.1.1. are met, corresponds to the Limit States Method, whose practical application shall be adjusted to that which is indicated in the Instructions and Codes in effect for the constituent materials of the structures, in accordance with the hypotheses and criteria of load combinations unique to Maritime and Port Works indicated in section 4.2.
Only this method shall be considered compatible with the action evaluation criteria, with the representative values of the actions, and with the load hypotheses and combination conditions included in this Recommendation.

Limit states are defined as those states or situations of the structure, or parts thereof, that upon being reached and exceeded, put the structure out of use by nonfulfillment of the preestablished stress or functional limit conditions.

The object of this method is to effectively limit the probability that in reality, the Limit States can be exceeded in any situation or design hypothesis. With this aim it shall be verified that, in general, the effects produced by actions acting upon a structure ($S_d$) do not exceed its response capacity ($R_d$), with a safety margin ($S_d \leq R_d$).
These checks shall be carried out by means of a calculation model or an experimental model, with the following variables, called basic variables, intervening :

— The actions applied (F)
— The properties of the structure's constituent materials (f)
— The geometric parameters used for the calculation of the effects produced by the actions and the structure's response capacities (a).

The effects produced by the actions (loads) shall be obtained based upon their Design Values ($F_d$). An action's Design Value or Weighted Value is that which results from applying the appropriate safety factors ($\gamma_f$) to its Representative Values ($F_k$ or $\psi_i \cdot F_k$) (see sections 3.2 and 4.2).

$$F_d = \gamma_f \cdot F_k \text{ or } \gamma_f \cdot \psi_i \cdot F_k$$

Diverse hypotheses and design load combinations that are mutually compatible $\Sigma_\theta (\gamma_f \cdot \psi_i \cdot F_k)_\theta$, with different positions and configurations of each one of them shall be considered to determine that which produces the most unfavorable effect upon the element or section considered (see section 4.2.2).

$$S_d = S\,[(\gamma_f \cdot \psi_i \cdot F_k)_{1,\dots,} (\gamma_f \cdot \psi_i \cdot F_k)_{\theta,\dots}]$$

To determine the response capacity of the analyzed structure or structural element, the characteristic values of the material's properties ($f_k$) shall be used, divided by a safety factor ($\gamma_m$).

$$f_d = f_k/\gamma_m$$

$$R_d = R\,(f_{d1, \ldots,}\ f_{di, \ldots})$$

The characteristic values of the material's properties (e.g. characteristic strength) are defined as those values associated with an exceedence probabilities of 95% for the statistical distribution obtained from tests carried out under conditions established in the corresponding Codes (lower characteristic value).

In some cases, increases in the values of the material's properties can cause reductions in the safety of the structure (e.g. increases in the concrete's strength gives rise to reductions in its deformability, causing harmful effects, such as in the case of cracking when imposed deformations exist, plasticity in seismic situations, etc). In these cases, higher or lower characteristic values associated with percentiles of 95% and 5%, respectively, shall be differentiated.

The safety factors ($\gamma_m$) are introduced in the calculation to take into account, among others, the following factors : Possible deviations from the values of the material's properties in the real structure with respect to the characteristic values ; possibility of inadequate evaluation or calculation of the properties of the materials caused by defects in their modelling ; effects of the material's geometry or imperfections upon their properties; variations in the material's properties in relation to the acting loads (e.g. in long duration loads); and evaluation of the considered limit state.

The values of the reduction safety factor of the characteristic values of the material's properties shall primarily be determined according to their specific Codes, in function of the considered property, the constituent material, the limit state being verified, the analyzed phase and hypothesis of work, the control level foreseen for the construction, the damage foreseeable in case of accidents and also the adopted calculation model. For cases not covered in the cited Codes, the information given in section 4.3 shall be used.

The geometric parameters shall usually be introduced in the calculations by means of their nominal values.

$$a_d = a_{nom}$$

In some cases, an additional safety factor ($\Delta a$ positive or negative) shall be introduced, producing a design value.

$$a_d = a_{nom} + \Delta a$$

### 4.1.3 LIMIT STATES

The Limit States to considerate in the calculation are :

— Ultimate Limit States (USL)
— Serviceability Limit States (SLS)

a) ULTIMATE LIMIT STATES

Ultimate limit states cover all those states corresponding to the structure being put out of service by collapse, breakage, loss of stability, or other forms of the structure's, or part of the structure's failure. These states are related to the maximum load capacity of the structural system.

The following Ultimate Limit States shall be taken into consideration, whenever they are significant :

— *Static Equilibrium Limit States,* defined by the loss of static stability in a part or in the entire structure, considered as a rigid body (e.g. overturning, sliding, support heaving, flotation, etc).
  This state shall be studied for the structure or complete structural element, taking into account the real support conditions, and in particular, those derived from soil behaviour, deduced according to Soil Mechanics methods.

Usually, the following condition shall be met : $E_{d,dst} < E_{d,est}$ ; given that $E_{d,dst}$ and $E_{d,est}$ are the effects produced by the destabilizing and stabilizing actions, respectively.
Unless a specific Regulation or common criteria of recognized validity exists for the univocal definition of which actions are stabilizing and which are destabilizing, for the type of structure and equilibrium limit condition analyzed, the Project Engineer shall be especially careful in addressing the checking conditions, including the applied criteria for the differentiation between stabilizing actions and destabilizing actions.
This checking condition shall be applicable whenever the structural equilibrium can be idealized in a sufficiently precise way to that of a rigid solid upon another rigid solid.

For more general cases, the checking condition can be established, based on the comparison between the calculation hypothesis and the hypothesis that produces the loss of static equilibrium. This can be done in various ways according to the structural type and the equilibrium state analyzed.

— *Breakage or Yield Limit States,* defined by the yielding or the excessive plastic deformation of one or various sections of the structure's elements.
Included in these limit states are those specific to tangential loads such as adherence or anchoring.
This state shall be studied in sections of the structure.
The checking condition shall be : $S_d$ £ $R_d$; given that, in general, $S_d$ is the effect produced by the design actions and $R_d$ is the yield condition associated with the design values of the construction material's properties.

The definition of $S_d$ and $R_d$ shall vary according to the specific analyzed problem, whether they are scalars, vectorials, or more complicated relationships.
In some cases, the design values of the actions can be directly compared with the design value of the strength capacity of the structure or structural element.

— *Limit State of Second Order Instability* or buckling of all or part of the structural element.
This state shall be studied for the structure as a whole or for complete structural elements.
It shall be verified that the instability mechanism is not produced unless the action design values are surpassed, for the design value of the structural properties.

— *Limit Fatigue State,* corresponding to the failure of one or several materials of the structure by the effects of fatigue under the action of repeated or variable actions over time (e.g. wave actions, currents, wind, ice, etc).
The checking of the limit fatigue state shall not be necessary if the accumulated number of cycles or fluctuations of the loads is less than 1,000 cycles.
This state shall be studied in sections of the structure.
It shall be verified that the deterioration caused by the reiterated application of the design loads upon the structure is less than the critical deterioration, in terms of the constituent material's properties.
Fatigue problems are expected in maritime and port structures, especially in steel structures subject to wave action, wind currents or cargo handling equipment overloads. They are also to be expected in pavements subject to conventional and cargo handling equipment traffic.

— *Progressive Collapse Limit State,* characterized by the breakage or loss of static equilibrium of the structure due to the progressive failure of its elements once initial failure occurs in one or a few of its individual elements (transformation of the structure into a mechanism).
The limit state shall have special significance in the extraordinary work hypothesis, since in these load states it is customary to admit localized failures or losses of equilibrium whenever they do not result in the total collapse of the structure. After the performance of the accidental loads, the structure shall be capable of resisting the extreme actions during the time period necessary for its complete repair.
The checking of this limit state shall usually be more a question of structural conception, or of constituent materials, than of calculations.
This state shall be studied for the structure as a whole.
It shall be verified that the structure does not transform in a mechanism unless the action's design values are surpassed.

— *Cumulative deformation limit state,* or unacceptable changes in the system's geometry.

— *Particular Limit States* associated with certain accidental situations such as fire resistance, thermal insulation during the course of a fire, etc.

b) SERVICEABILITY (USE) LIMIT STATES (SLS)

Serviceability limit states cover all states or structure situations when the structure is out of service because of functional, durability or aesthetic reasons.
It shall verified that : $E_d \leq C_d$; given that $E_d$ is the effects produced by the design actions, and $C_d$ is a nominal value set a prior, or as a function of the material's properties. The relationships can be scalars, vectorials, or more complex.

The following Serviceability Limit States shall be taken into consideration, whenever they are significant :

— *Limit States of Durability,* characterized by the fact that the resistance of the element against the aggressiveness of the environment, or of the actions, reach a determined limit value. Includes limit states such as fissuring, resistance against fissuring, corrosion, etc. The deterioration, provoked by the aggressive influences of the environment upon the structure's constituent materials can also lead to an ultimate limit state.
This state shall be studied in sections of the structure.

— *Limit States of Deformation,* characterized by determined movements or velocities and accelerations being reached in the structure that affect its functionality or aesthetics. Lacking criteria in the corresponding Codes, the Project Engineer shall set the deformation limits admitted in agreement with the preestablished use conditions and with the Client's and Government Authority's criteria.
This state shall be studied for the structure as a whole or for individual structural elements.

— *Limit State of Vibration,* in order to prevent vibrations with an amplitude and frequency capable of producing damage in the non-structural elements, interfering with its normal functioning, or causing discomfort. This state shall be studied for the structure as a whole or for complete structural elements.

— *Limit State of Permanent Damage,* characterized by the fact that the structure has a sufficient margin of safety to remain in service during its entire design life, assuming that irreparable damage occurs. One example would be the action of an accidental load (e.g. if the material is concrete, it shall be necessary to verify, in terms of the limited cracking state, that after the extraordinary action considered the resulting cracking does not cause difficulties of a functional type or loss of durability.
This state shall be studied for the structure as a whole or for complete structural elements.

— *Limit State of Impermeability,* for structures whose primary function is to create an impermeable compartment.
This state shall be studied in sections of the structure.

### 4.1.4 ANALYSIS CRITERIA

■ CALCULATION MODELS

The general calculation process corresponding to the Limit States Method and other established prescriptions in these Recommendations shall be applied, taking into account the following conditions :

— The response of the structure associated with the acting actions shall be determined by carrying out :

- A global structural analysis for the determination of the forces and loads.
- An analysis of the cross sections of the different structural elements and of the unions to determine their strength capacity.

based on a modelling or idealization of the structure, in terms of its constituent materials as well as its geometric characteristics.
This modelling shall be sufficiently precise to correctly predict the behaviour of the structure, and at the same time allow a valid definition of the conditions inherent in the considered limit state. Likewise, the idealization shall be compatible with the applications for which the means of analysis are valid.

— The models for the structural materials shall be :

- Lineal (or elastic) : The response of the structure is proportional to the loads, with the eventual possibility of redistributions.
- Non-lineal (or elastic-plastic) : With different approximations for the determinations of the forces (e.g. in concrete, elastic behaviour with plasticised zones ; plastic hinge theory), and of the strength capacities (e.g. simplified stress/strain diagrams in the concrete such as rectangular or parabolic diagrams).

— Geometric models shall be :

- First Order Models, based on the initial structure's geometry
- Second Order Models, based on the deformed structure's geometry

In the majority of cases, the loads shall be determined based on the first order models. It may be necessary to use second order models for very flexible structures, for certain complex forces, and for the study of certain types of structural instability.

■ EXPERIMENTAL MODELS

The calculation models may be partially or totally substituted or complemented by physical or mathematical experimental models, or by prototype tests, in order to :

— Contribute to the structural analysis by determining the structure's response experimentally.
— Define certain structural behaviour aspects in the conditions specified in the tests (e.g. berthing capacity, deformations, etc).

The experimental model tests shall generally by used when the calculation models have a high degree of uncertainly, giving rise to uneconomic or unreliable designs.

■ CHECKS

The calculations carried out, based on calculation or experimental models, shall guarantee that the Ultimate Limit States and Serviceability Limit States are not surpassed in all load hypotheses, considered in each one of the significant Project Phases and Work Hypotheses and established according to the combination load criteria of these Recommendations (see section 4.2).

## 4.2 ACTION AND LOAD HYPOTHESIS COMBINATION CRITERIA

### 4.2.1 ACTION DESIGN VALUES

The design value of an action to be considered in each one of the load combinations shall be obtained, weighting one of its Representative Values (Characteristic Value $F_k$; Combination Value $\psi_0 \cdot F_k$; Frequent Value $\psi_1 \cdot F_k$; Quasi-permanent Value $\psi_2 \cdot F_k$) by means of Safety Factors ($\gamma_f$).

The representative value of an action, adopted for the determination of its design value, shall depend on :

— The nature of the action
— Its favorable or unfavorable effect.
— The importance, or predominant character of the action in the design of the analyzed structure
— The analyzed project phase and work hypothesis
— The considered limit state.

Likewise, the safety factor for the consideration of the representative value shall depend on :

— The nature of the action
— Its favorable or unfavorable effect
— The analyzed project phase and work hypothesis
— The consider limit state
— The quality control level foreseen in the construction
— The foreseen damage in case of structure failure

— The constituent material of the structure

To determine the design value of the actions to use in the maritime or port works project, the general criteria established in table 4.2.1.1 for Ultimate Limit States (ULS) and 4.2.1.2 for Serviceability Limit State (SLS) shall be followed.

This specific regulation is established to substitute other simplified determination criteria of the action design values prescribed by the General Codes in effect (e.g. EH-88, EP-80, NBE-AE-88, NBE-MV-103; Seismic Code PDS-1, etc); especially applicable in bridge and building projects. In maritime and port works, the nature and characteristics of the predominant effect loads, the different possibilities of combined performance of various loads with different origins, the existence of variable loads with a maximum or minimum non-zero value, and the determination of the variable environmental actions by means of risk criteria, give rise to relevant differences with respect to the usual criteria in building and bridge works.

In determining the characteristic values and other representative values of the actions, the specifications in sections 3.2. Action Evaluation Criteria and 3.4. Characteristic Value of Actions, of these Recommendations shall be taken into account.
The coefficients $\psi_0$, $\psi_1$, and $\psi_2$, necessary for the determination of the representative values of the actions based on their characteristic values, are quantified in subsection 3.2.2.

The safety factors shall conform to the indications in section 4.2.2 for the calculation models established in the Codes in effect for the design of works with specific materials. The correction of coefficients in terms of the control level in the construction and the foreseen damage in case of accident, shall be carried out according to these Codes.

### 4.2.2 COMBINATION OF ACTIONS

a) GENERAL CRITERIA

To find the most unfavorable work hypothesis corresponding to each project phase and work condition, the following procedure shall be used :

— Of the actions classified in these Recommendations, those that do not have to be considered due to being negligible or nonexistent in the studied case shall be eliminated.

— The design values of the remaining actions shall be assigned according to the criteria in section 4.2.1.

— For each Limit State to verify, the work hypotheses included in paragraph b) of this section shall be considered, choosing the hypothesis which, in each case, proves most unfavorable.

— In each hypothesis, only those actions which have compatible simultaneous application shall be taken into account.

— Lacking a specific Regulation for the structural type analyzed, or sufficient experience of the Project Engineer, each significant variable load with different origins shall be taken successively as the variable action with predominant effects, unless it is evident that the resulting combination cannot be the most unfavorable.

— Although the number of variable actions of simultaneous compatible application can be high, it shall rarely be necessary to take combination of all these loads into consideration. Generally, more than two variable use loads and two environmental loads shall not be applied together.

— In each accidental combination that is considered, only one accidental action shall be included, in order to take into account the small probability of simultaneous action of more than one accidental load.

— When, in one load hypothesis, various load cases are possible, the most unfavorable cases for the analyzed limit state and section shall be considered.

— Regarding action combinations, the seismic loads shall be considered as accidental actions, independently of the applied evaluation criteria.

**TABLE 4.2.1.1 GENERAL CRITERIA FOR THE DETERMINATION OF THE DESIGN VALUES OF ACTIONS, FOR THEIR USE IN ULTIMATE LIMIT STATES**

| ANALYZED PROJECT PHASE | PERMANENT LOADS ($G_d$) | | VARIABLE LOADS ($Q_d$) | | | | ACCIDENTAL LOADS ($A_d$) |
|---|---|---|---|---|---|---|---|
| | | | Predominant effect variable load | | Other variable loads of simultaneous compatible action | | |
| | With unfavorable effects | With favorable effects | With unfavorable effects | With favorable effects | With unfavorable effects | With favorable effects | With unfavorable effects |
| Construction and Service phases except in exceptional conditions | $\gamma_{fg\,max} \times G_{ksup,i}$ | $\gamma_{fg\,min} \times G_{kinf,i}$ | $\gamma_{fq\,max,\,1} \times G_{ksup,l}$ | — | $\gamma_{fq\,max,\,j} \times \psi_{0,j} \cdot Q_{ksup,j}$ | $\gamma_{fq\,min,\,j} \times \psi_{0,j}\,Q_{kinf,j}$ | — |
| Service phase in exceptional conditions | $\gamma_{fga\,max} \times G_{ksup,i}$ | $\gamma_{fga\,min} \times G_{kinf,i}$ | $\psi_{1,1} \cdot Q_{ksup,1}$ | — | $\psi_{2,j} \cdot Q_{ksup,j}$ | $\psi_{2,j} \cdot Q_{kinf,\,j}$ | $\gamma_{fa} \times A_k$ |

**LEGEND**

$G_d$ : Design value of the permanent loads

$G_{ksup,i}$ : Maximum characteristic value of the permanent load i

$G_{kinf,i}$ : Minimum characteristic value of the permanent load i. Normally, for dead loads, this value is zero

$Q_d$ : Design value of variable loads

$Q_{ksup,1}$ : Maximum characteristic value of the variable load considered to have the predominant effect in the combination

$Q_{ksup,i}$ : Maximum characteristic value of the variable load j, distinguished from the predominant load in the combination

$Q_{kinf,j}$ : Minimum characteristic value of the variable load j, distinguished from the predominant load in the combination. Usually this value is zero, except for hydraulic loads, earth loads and deformation loads.

**TABLE 4.2.1.1 (Continued)**

| | |
|---|---|
| $A_d$ | : Design values of the accidental loads |
| $A_k$ | : Characteristic value of the accidental load |
| $\psi_{0,j}$ | : Factor to obtain the combination value of the variable action, j |
| $\psi_{1,1}$ | : Factor to obtain the frequent value of the variable action considered to have the predominant effect |
| $\psi_{2,j}$ | : Factor to obtain the quasi-permanent value of the variable action j |
| $\gamma_{fg\ max}$ | : Safety factor for the maximum characteristic values of the permanent loads. |
| $\gamma_{fg\ min}$ | : Safety factor for the minimum characteristic values of the permanent loads. |
| $\gamma_{fga\ max}$ | : The same as $\gamma_{fg\ max}$ except for exceptional conditions. |
| $\gamma_{fga\ min}$ | : The same as $\gamma_{fg\ min}$ except for exceptional conditions. |
| $\gamma_{fq\ max,1}$ | : Safety factor for the maximum characteristic values of the variable load considered to have the predominant effect. |
| $\gamma_{fq\ max,j}$ | : Safety factor for the maximum representative values of the variable load j. |
| $\gamma_{fq\ min,j}$ | : Safety factor for the minimum representative values of the variable load j. |
| $\gamma_{fa}$ | : Safety factor for the characteristic values of the accidental actions. |

b) LOAD HYPOTHESES

■ FOR ULTIMATE LIMIT STATES (ULS)

— Equilibrium Limit States

The checking of the static equilibrium conditions shall generally be considered in verifying the following inequality for each analyzed equilibrium state:

$$E[\Sigma\gamma_{fg\ min} \cdot G_{k\ inf,i} + \gamma_{fq\ min} \cdot (\sum_{j>1}\psi_{0,j} \cdot Q_{k\ inf,j})] \geq E[\Sigma\gamma_{fg\ max} \cdot G_{k\ sup,i} + \gamma_{fq\ max} \cdot Q_{k\ sup,1} + \sum_{j>1}\gamma_{fq\ max,j} \cdot \psi_{0,j} \cdot Q_{k\ sup,j}]$$

with the left hand side representing the effects produced by the stabilizing actions, and the right hand side those produced by destabilizing actions.

The parameters are defined in table 4.2.1.1.

The checking of Equilibrium Limit States is not usually carried out in extraordinary conditions, that is, when accidental actions are acting, since in these loads states, localized damages can be admitted as long as they do not result in the total collapse of the structure. For these cases, the equilibrium limit states shall be considered verified with the checking of the Progressive Collapse Limit State.

As a reference, the safety factor values ($\gamma_f$) to utilize in checking the equilibrium limit states shall be :

| TYPE OF ACTION | With unfavorable effects ($\gamma_{fmax}$) | With favorable effects ($\gamma_{fmin}$) |
|---|---|---|
| Permanent Load ($\gamma_{fg}$) | 1.10 | 0.90 |
| Variable Environmental Load obtained based on statistical data and risk criteria ($\gamma_{fq}$) | 1.00 | — |
| Deformation Load ($\gamma_{fq}$) | 1.10 | 0.90 |
| Other Variable Loads ($\gamma_{fq}$) | 1.50 | 1.00 |

These values shall be applicable when there is no specific Regulation for the analyzed structural type.

— *Breakage or Yield : Second Order Instability, and Progressive Collapse*

The checking of the breakage or yield, second order instability, and progressive collapse limit states shall be done based on the following combinations :

- Fundamental Combinations

  (For construction and service phases in normal operating conditions, in extreme conditions, and repair).

$$\Sigma\gamma_{fg\,max} \cdot G_{ksup,i} + \Sigma\gamma_{fg\,min} \cdot G_{kinf,i} + \gamma_{fq\,max,1} \cdot Q_{ksup,1} + \sum_{j>1}\gamma_{fq\,max,j} \cdot \psi_{0,j} \cdot Q_{ksup,j} + \sum_{r>1}\gamma_{fq\,min,r} \cdot \psi_{0,r} \cdot Q_{kinf,r}$$

  The parameters are defined in table 4.2.1.1.

- Accidental Combinations

  (For service phases in exceptional conditions, and service phase after an exceptional situation ($A_k = 0$)).

$$\Sigma\gamma_{fga\,max} \cdot G_{ksup,i} + \Sigma\gamma_{fga\,min} \cdot G_{kinf,i} + \psi_{1,1} \cdot Q_{ksup,1} + \sum_{j>1}\psi_{2,j} \cdot Q_{ksup,j} + \sum_{r>1}\psi_{2,r} \cdot Q_{kinf,r} + \gamma_{fa} \cdot A_k$$

  The parameters are defined in table 4.2.1.1.

Lacking specific Regulations for the analyzed structural type, the values of the safety factors ($\gamma_f$) to use in the verification of breakage or yield, second order instability, and progressive collapse limit states, shall be those indicated in A for fundamental combinations, and in B for accidental combinations.

— *Fatigue Limit State*

To check the fatigue limit state, all the fluctuating loads (e.g. wave action, wind, currents, etc.) that can act upon a structure during the analyzed project phase shall be taken into consideration with each one of their usual values in this phase.

The weighted factor ($\gamma_f$), applied to the fatigue loads is generally 1.00. Nevertheless, if the statistical data available for the determination of the fatigue loads is neither reliable nor sufficient, (minimum data recording periods of 1/20 of the duration of the analyzed phase), higher factors shall be considered.

**TABLE 4.2.1.2 GENERAL CRITERIA FOR THE DETERMINATION OF THE DESIGN VALUES OF ACTIONS, FOR THEIR USE IN SERVICEABILITY LIMIT STATES**

| COMBINATION TYPE | PERMANENT LOADS ($G_d$) | | VARIABLE LOADS ($Q_d$) | | | | ACCIDENTAL LOADS ($A_d$) |
|---|---|---|---|---|---|---|---|
| | | | Predominant effect variable load | | Other variable loads of simultaneous compatible action | | |
| | With unfavorable effects | With favorable effects | With unfavorable effects | With favorable effects | With unfavorable effects | With favorable effects | |
| Rare or infrequent | $G_{ksup,i}$ | $G_{kinf,i}$ | $G_{ksup,I}$ | — | $\psi_{0,j} \cdot Q_{ksup,j}$ | $\psi_{0,j}\ Q_{kinf,j}$ | — |
| Frequent | $G_{ksup,i}$ | $G_{kinf,i}$ | $\psi_{1,1} \cdot Q_{ksup,1}$ | — | $\psi_{2,j} \cdot Q_{ksup,j}$ | $\psi_{2,j} \cdot Q_{kinf,j}$ | — |
| Quasi-permanent | $G_{ksup,i}$ | $G_{kinf,i}$ | — | — | $\psi_{2,j} \cdot Q_{ksup,j}$ | $\psi_{2,j} \cdot Q_{kinf,j}$ | — |

**LEGEND**

See Table 4.2.1.1.

**NOTES**

As an exception to the criteria indicated in this table, to obtain the design values of the deformation loads with permanent character (e.g. prestressed), the following safety factor shall also be applied $\gamma_f$ :

$\gamma_{fq\ max}$ = 1.10 if the effect of the load in the action combination is unfavorable

$\gamma_{fq\ min}$ = 0.90 if the effect of the load in the combination of actions is favorable

| A. SAFETY FACTORS FOR FUNDAMENTAL COMBINATIONS | | |
|---|---|---|
| TYPE OF ACTION | With unfavorable effects ($\gamma_{max}$) | With favorable effects ($\gamma_{min}$) |
| Permanent Load ($\gamma_{fg}$) | 1.35 | 1.00 |
| Variable Environmental Load obtained based on statistical data and risk criteria ($\gamma_{fq}$) | 1.00 | — |
| Deformation Load ($\gamma_{fq}$) | 1.20 | 0.90 |
| Other Variable Loads ($\gamma_{fq}$) | 1.50 | 1.00 |

| B. SAFETY FACTORS FOR ACCIDENTAL COMBINATIONS | | |
|---|---|---|
| TYPE OF ACTION | With unfavorable effects ($\gamma_{max}$) | With favorable effects ($\gamma_{min}$) |
| Permanent Load ($\gamma_{fga}$) | 1.10 | 0.90 |
| Variable Loads ($\gamma_{fqa}$) | 1.00 | 1.00 |
| Accidental Loads ($\gamma_{fa}$) | 1.00 | — |

To check the fatigue limit state, the loads shall be condensed in uniform groups in which their components produce equivalent states of stress (e.g. wave action inside a height/period interval), or transformed in fictitious loads equivalent to the deterioration effects (e.g. loads transmitted by cargo handling equipment equivalent to an accumulated number of Design Loads. See section 3.4.2.3.4).

Thus a fatigue safety factor shall be defined in the following way :

Safety Factor : $\sum_{i=1}^{n_T} (n_i/N_i)<1$

given that :

$n_i$ : Number of actions of a determined group that act during the analyzed phase, or number of cumulative equivalent fictitious actions during this phase. (with a 50% exceedence probability).

$N_i$ : Number of actions of the determined group, or number of cumulative Standard loads necessary to produce the critical deterioration of the structure.

$n_T$ : Number of acting load groups considered.

■ FOR SERVICEABILITY (USE) LIMIT STATES (SLS)

The checking of the different Serviceability Limit States shall be done based on the following load combinations :

— Infrequent Combination

$$\Sigma G_{k,i} + Q_{k,1} + \sum_{j>1} \psi_{0,j} \cdot Q_{k,j}$$

— Frequent Combination

$$\Sigma G_{k,i} + \psi_{1,1} \cdot Q_{k,1} + \sum_{j>1} \psi_{2,j} \cdot Q_{k,j}$$

— Quasi-permanent Combination

$$\Sigma G_{k,i} + \sum_{j\geq 1} \psi_{2,j} \cdot Q_{k,j}$$

The parameters are defined in table 4.2.1.2.
For each action, its maximum or minimum value shall be adopted in function of its favorable or unfavorable effect in the combination of actions for the considered serviceability limit state.

As is seen in the indicated action combinations, the safety factors ($\gamma_f$) to use in checking the serviceability limit states shall be 1.00. Nevertheless, for permanent character deformation loads, (e.g. Prestressed Loads), the following shall be applied :

- $\gamma_{q\,max}$ = 1.10, if the effect of the load in the action combination is unfavorable.
- $\gamma_{q\,min}$ = 0.90, if the effect of the load in the action combination is favorable

The Project Engineer, Client or the Government Authority shall, with adequate justification, set the checking conditions in each serviceability limit state in each project phase and hypothesis, indicating the type of load combinations applicable. Nevertheless, the minimum conditions prescribed in the specific Codes, Instructions and other Regulations for the analyzed structural type shall always be taken into consideration.

Some examples of minimum compliance conditions of serviceability limit states prescribed in existing Regulations are :

Example 1 : Code EP-80 distinguishes three classes of verifications in a prestressed concrete structure relative to the cracking limit states in the service phase, applicable in terms of the analyzed structural type : Class I : corresponding to compliance with the decompression limit state for infrequent combinations; Class II : corresponding to compliance with the crack formation limit state for infrequent combinations, and of the decompression for frequent combinations; Class III: corresponding to compliance with the decompression limit state for quasi-permanent combinations.

Example 2 : Code NBE MV-103 sets deflexion limits in steel beams of straight axes subject to bending for infrequent combinations.

In some cases, the checking of a limit state can require the simultaneous consideration of two or more load combinations. This is particularly applicable when significant deferred effects are foreseen due to rheologic loads. In this case, the action combinations to take into consideration shall be :

- For instantaneous effects : infrequent combinations.
- For deferred effects : quasi-permanent combinations.

The checking of Serviceability Limit States shall not usually be done for extraordinary conditions, that is, when accidental actions are considered. It shall be sufficient to verify that, once the accidental action withdraws, the possible remaining effects derived from its actions do not impede the structure from functioning as it did before the extraordinary action occurred, in particular, in relation to functional and durability conditions (e.g. remaining cracks in concrete structures, remaining deformations in steel structures, etc). That is to say, the Permanent Damage Limit State shall be verified.

## 4.3 MATERIAL PROPERTY REDUCTION SAFETY FACTORS ($\gamma_m$) FOR THE VERIFICATION OF LIMIT STATES

According to the general calculation process presented in section 4.1 of these Recommendations, the determination of the response capacity of the analyzed structure or structural element shall be done based on the characteristic values of the materials's properties ($f_k$), divided by a safety factor ($\gamma_m$).

The safety reduction factors for the characteristic values of the material's properties shall primarily be determined according to their specific Codes.

The average safety factor values for the study of limit states is indicated in table 4.3.1. These values shall be considered applicable whenever normal quality control levels in the construction and intermediate damage in case of failure are foreseen, evaluated in accordance with the specific Norms.

For other control and damage evaluation levels, the corrections put forth in the cited Codes shall be applied.

A safety factor, higher or lower than what is indicated here shall be applied, as long as their application by means of more severe control measures is adequately justified.

## TABLE 4.3.1 AVERAGE VALUES OF MATERIAL PROPERTY REDUCTION SAFETY FACTORS ($\gamma_m$) FOR THE VERIFICATION OF LIMIT STATES

A. ULTIMATE LIMIT STATES

| MATERIAL | ACTIONS COMBINATIONS | |
|---|---|---|
| | FUNDAMENTAL | ACCIDENTAL |
| Compressed concrete ($\gamma_c$) | 1.50 | 1.30 |
| Passive reinforcing steel ($\gamma_s$) | 1.15 | 1.00 |
| Active reinforcing steel ($\gamma_p$) | 1.15 | 1.00 |
| Construction steel ($\gamma_a$) | 1.00 | 0.90 |
| Connecters in mixed structures ($\gamma_v$) | 1.25 | 1.10 |
| Wood ($\gamma_t$) | 1.40 | 1.10 |

B. SERVICEABILITY LIMIT STATES

| MATERIAL | ANY COMBINATION |
|---|---|
| All materials ($\gamma_m$) | 1.00 |

NOTES

— The prior values are not applicable to the following material's properties, to which a coefficient of reduction of 1.00 shall be applied :

- Modulus of elasticity (E)
- Shear modulus of elasticity (G)
- Poisson ratio ($\upsilon$)
- Thermal expansion coefficient ($\alpha$)

— The values in the table shall not be applicable to verify the fatigue and progressive collapse limit states.
In this case, the following differentiated factors shall be used :

For fatigue limit state :

$\gamma_c$ = 1.25
$\gamma_s$ = 1.00

For progressive collapse limit state :

$\gamma_c$ = 1.10
$\gamma_s$ = 1.00

Likewise, for the calculation of concrete structure fatigue, a modulus of elasticity equal to 0.80 times the instantaneous longitudinal secant deformation modulus shall be adopted.