Summary

Landslides followed by heavy rain and or earthquakes occur relative frequently in El Salvador. The consequences in terms of casualties and damage on properties and infrastructure are significant. Future risk may probably increase as there is a clear trend to build more and more in potential landslide exposed hilly areas, especially in urban areas.

The report presents the results of a study where existing codes and regulations are reviewed to see if the relative high landslide risk in the country is adequately accounted for. The study forms a part of an ongoing institutional cooperation between SNET (Servicio Nacional de Estodious Territorales) and NGI (Norwegian Geotechnical Institute) which is financially supported by the Ministry of Foreign Affairs.

The conclusion of the study indicates that there is a need to expand the codes and regulations with some clauses that better reflect the landslide hazard. The report contains a set of proposals for detailed formulations that could be added in the codes. In addition, the report contains some recommendations for how SNET could be proactive and initiate activities to enhance that codes and regulations become a vital instrument in reducing loss figures in the future.

Contents

1 INTRODUCTION

With financial support from the Norwegian Ministry of Foreign Affairs, NGI and SNET (Servicio Nacional de Estodious Territorales) have an institutional cooperation program where the objectives are as follows:

- To implement mitigation measures in a specific critical volcano slope area that will serve as a pilot project and provide a fundamental basis for further work in critical areas of the country where the newly established Government organisation SNET will be in charge.

- To assist SNET in building up of capacity in the fields of assessment, prevention, mitigation and management of natural hazards and to disseminate knowledge.

This report covers Task 4 in the program and deals with review of building codes and regulations to identify whether the relatively high landslide hazard and risk is adequately reflected in the codes and to which extent the regulations could be improved on this subject.

The following 4 documents have been reviewed:

- The Law of Urbanisation and Construction
- Regulations for Structural Safety of Constructions
- Technical Standards for Foundation and Slope Stability
- Technical Standards for Earthquake Design

Sections 2 to 5 contain NGI's comments and suggestions to these 4 documents and some more general recommendations are given in Section 6.

2 THE LAW OF URBANISATION AND CONSTRUCTION

In the national law of Urbanisation and Construction the following two paragraphs are proposed to be included:

1. **Danger zones:** Areas exposed to natural hazards, earthquakes, volcanic activity, flooding, landslides or other kinds of natural dangers, should be identified and established as *Danger zones* in the municipal development plans.

2. **The Building site:** Areas can only be built on when the safety against danger or major inconvenience caused by natural hazards is acceptable. The safety against dangers is further defined in the "Regulations for Structural Safety of Constructions".

3 REGULATIONS FOR STRUCTURAL SAFETY OF CONSTRUCTIONS

This Regulation seems to have a similar purpose as CEN EN1990 Basis of structural design (Eurocode 0), i.e. establish the common and basic rules for all design standards. Clauses in a regulation like this should deal with main principles and not details.

Title I General Requirements

Article 1. Add to the text:

The *acceptable safety against danger or major inconvenience* is defined as a maximum yearly probability for the occurrence of a natural hazard that will cause unacceptable deformations or collapse of the building construction.

Buildings are grouped into *safety classes* according to the consequences of a natural hazard. The buildings in the different classes should either be located outside the *danger areas* for the given probability of the natural hazard, or being constructed to withstand the potential hazard.

Table 1. Proposal for safety classes, yearly probability, and examples of building types:

Safety class	Maximum probability of hazard per year for hazards that may be lethal and cause collapse of structures, like earth quakes, slides and debris flows	Maximum probability of hazard per year for hazards that will cause material damage only, like flooding in low gradient terrain	Example of building types
1	10^{-2}	2×10^{-2}	Shacks, garages, minor storehouses
2	10^{-3}	10^{-2}	Dwelling houses, major storehouses, towers, silos
3	10^{-4}	5×10^{-3}	Major bridges, shopping centres, tribunes, schools, hospitals
4	10^{-6}	10^{-3}	Nuclear power plants, dams with risk of major flooding, major industry plants

The figures in table1 shall be regarded as notional, i.e. the numbers are usually not possible to calculate. Decision of the final figures should be based on a national discussion of the appropriate safety level in the country.

Title II Structural Design Criteria.

Chapter 1. General.
Articles dealing with safety and actions (paragraph typically 14 and 15) should be harmonized with the final figures used in table 1.

Chapter 6 Foundation design and Stability of Slopes
Article 51. Add to the text: Run-out distance of debris or mud-flows from potential slides shall be calculated or evaluated.

Article 52. Add to the text: Buildings vulnerable to debris flow or mudflow from potential slides shall be adequately protected.

4 TECHNICAL STANDARDS FOR FOUNDATION AND SLOPE STABILITY

Chapter 3 Design Considerations

Proposed new text: 3.3 Foundations in slopes

The provisions in Chapter 6 shall apply for foundations located at and close to slopes.

The effect of excavations and/or embankments nearby foundations and structures shall be considered.

Unless specific structural remedies are planned and implemented, excavation and construction of foundations shall not reduce the bearing capacity of foundations at a superior level and embankments or foundations shall not impose additional actions on structures at a lower level.

Chapter 4 Geotechnical Investigations

Proposed new text in a few clauses:

4.1 General

Geotechnical investigations shall be planned in order to ensure that relevant information and data are available at the various stages of the project.

4.2 Site Knowledge and Inspection

Knowledge about topography, geology, development history and existing structures shall be obtained including an inspection of the site.

4.3 Ground Investigation Methods

4.3.1 Standard Penetration Test (SPT)

The SPTs shall fulfil the following:
Previous 4.2.1 ....
1)..........
2).........

4.3.2 Location and Depth of Borings

The spacing of investigation points and the depth of the investigations shall be selected as a function of the geological conditions, the dimensions of the structure and the engineering problems involved. For structures near or at slopes and steps in the terrain (including excavations), investigations should also be arranged outside the project area so that the stability of both the overall slope and the step can be assessed.
+ Previous 4.2.2...........

4.3.3 Other Investigation Methods

Alternatively to, or in addition to the SPT, other field investigations may be considered like for example:

- Test pits for sampling and ground water measurements
- Cone Penetration Tests (CPT) (with pore pressure measurement , PCPT)
- Measurement of pore pressure in the ground (pietzometers)
- Geophysical investigation (e.g. seismic profiling, ground penetration radar, etc.)
- Monitoring, for example of existing structures.

4.4 Allowable stresses and bearing capacity

This section may be moved to Chapter 3?? More design issues than geotechnical investigation subjects?

4.5 Geotechnical Report

Add at the end of previous text:
For structures located near or at slopes, the Geotechnical Report shall address data for assessment of the overall stability, see Chapter 6.

Chapter 5 Retaining Walls

5.2 General considerations

New 5.2.1 Overall stability

The principles in Chapter 6 shall be used to demonstrate that an overall stability failure will not occur and that the corresponding deformations are sufficiently small.

5.2.2 Old 5.2.1 etc………

Chapter 6 Stability of slopes

Proposed new text in a few clauses:

6.1 Scope

This chapter defines basic criteria for assessment of natural slopes and for overall stability aspects in design of foundations, retaining structures, embankments and excavations in slopes.

6.2 General considerations

6.2.1 The groundwater levels/pore pressure conditions shall be selected to give the most unfavourable conditions that could occur in the design situations being considered.

6.2.2 The extent of the area of the slope to be considered shall be selected according to the consequences of failure of the slope and the impact on the adjacent areas.

6.2.3 In analysing the overall stability of the ground all relevant modes of failure shall be taken into account.

6.2.4 The overall stability and movements of natural or made ground shall be assessed by including comparable local experiences.

6.2.5 The design shall ensure that all construction activities in and on the site can be planned and executed such that failure is sufficiently improbable.

6.2.6 Slope surfaces exposed to potential erosion shall be protected if required to ensure that the safety level is retained. Measures to drain superficial water shall be taken. Aqueducts and/or sewer systems shall be able to be deformed without cracking.

6.3 Static and dynamic analyses

Use previous text.

6.4 Run-out distances

The run-out distance from potential sliding masses shall be calculated or evaluated.

6.5 Site rejection

If the stability of a site cannot readily be verified or the movements are found to be not acceptable for the site's intended use, the site shall be judged to be unsuitable without stabilizing measures. Areas exposed to debris flow from potential slides shall also be classified according to the table 1.

Commentary to the technical Standard for design and construction of foundations, earth structures and slope stability

These commentaries contain local experience and must be written by local experts.

In Chapter 9. Stability of slopes, safe stand-off zones are introduced (C9.2 Protection Zones). The term Protection Zones is not found in the Technical Standard. One may consider including this in Chapter 6 in the Technical Standard as one safety precaution?

5 TECHNICAL STANDARDS FOR EARTHQUAKE DESIGN

The code has only one place where soil come into picture and that is soil types for definition of acceleration, like other codes. As such, there is not much to comment.

One may, however, include some warning related to geohazards, for example by adding the following text in the first article in Chapter 3:

3.1 BASIS FOR DESIGN

Previous text.

Detailed site investigation is required in cases where liquefaction or slope stability are potential hazards.

6 RECOMMENDATIONS TO SNET

6.1 Land use regulations

Avoiding new developments, building and occupation in landslide prone areas is a challenge in El Salvador as well as in many of the other Central America countries. NGI recommend that SNET takes an active role in this challenge. The ongoing efforts trying to identify the most landslide prone areas are a fundamental task. By including this information in a GIS-based system one will obtain an effective tool to categorize the land with respect to areas at high risk.

A possible practical way of implementing a system for better land use control could be:

- SNET keeps responsibility for the landslide hazard and risk maps at a national level.

- Municipalities have access to this information and the information can be used for more detailed studies to identify development potential and hazard constraints in selected areas.

- It could be the city /municipality officials who implement the criteria for where it is possible to locate, for instance, vulnerable lifeline networks, hospitals, schools, health centers, fire stations and police stations. The same goes for the development of new areas for family housing.

- Guidelines for how to carry out detailed investigations at local/regional level need to be established, as well as defining safety criteria. This work could be done by SNET.

- We foresee that it should be the city authorities / municipalities who have the responsibility to have local hazard maps made in risk prone areas.
- There should preferably be a regulation stating that SNET should be informed and keep a register of the investigations.

Another challenge is the poor groups of people who occupy potentially dangerous areas, but have decided to live there because they have few alternatives within their economic reach. In the areas where they have settled, they are less likely to be forcefully evicted by the landowner. The worse the environmental conditions, the more likely the landowner will allow them to stay. Such settlements are usually highly vulnerable to landslide disasters. Regulations cannot solve this problem. The only instrument that will work is probably financial support for relocation to less risk-exposed areas.

6.2 Codes and regulations

Implementation of Building Regulations and Codes is one of the oldest and most commonly used methods of trying to secure that building and houses are adequately designed and constructed to fulfill the national safety requirements.

NGI's detailed comments to the reviewed national documents are given in the previous sections. Some more general comments are:

- If there is to be a full revision of the documents in the coming years, we would suggest that the structure of the documents is revised to make them more user friendly.
- The detailed comments that NGI has suggested are believed to reflect and take into account the rather high landslide risk for the country.
- One of the greatest challenges with codes is to have them implemented and used. May be SNET sees a possibility to be proactive in this respect by taking initiatives and offer courses and training seminars for city /municipalities setting the landslide risk in focus?

Another challenge equally difficult to deal with is the control function verifying that codes and regulations are followed in the execution of a building and construction project. Although the law is clear on this aspect, it is easy to see that in practice it is a matter of capacity and capability of the local authorities. NGI will therefore hesitate to give recommendations on this matter.

6.3 National plan for risk reduction strategies

With the strong position that SNET already enjoys as a national center for geo-related hazards, NGI suggests that an initiative is taken to prepare a national plan for landslide risk reduction.
A national plan for for landslide risk reduction should contain the following main items:

1. Detailed hazard mapping
2. Plans for defense structures
3. Preparedness and evacuation plans
4. Hazard forecasting
5. Improved education of professionals
6. Improved building codes and regulations
7. Improved public informations

We can see several advantages with having established such a plan:

- It helps in setting priorities for the use of resources.

- It can be useful to be proactive and have a beneficial effect when it comes to funding, either from Government or from external sources.

7 REFERENCES

European Committee for Standardisation (CEN) (2002)
EN 1990 Basis of structural design
Definitive text made available to CMC April 2002.

Appendix A - The Law for Building and Construction

LA ASAMBLEA LEGISLATIVA DE LA REPUBLICA DE EL SALVADOR

CONSIDERANDO:

I. Que la gran mayoría de las urbanizaciones que se han llevado a efecto en la ciudad capital y demás poblaciones de la República, lo han sido en forma desordenada, mirando por regla general sólo el beneficio de los urbanizadores y no el de las personas que habrían de llegar a poblar las nuevas zonas urbanizadas.

II. Que habiéndose dejado sin satisfacer las necesidades que toda urbanización de por sí acarrea, se ha creado con ello serios problemas y graves dificultades, no sólo al Gobierno que se ha visto competido a reparar y subsanar esos errores y omisiones, sino que también a los propios moradores de esas nuevas zonas, por lo que se hace necesario bajo todo punto de vista, dictar una ley que venga a poner coto a esa forma desordenada del ensanchamiento urbano, y fije las normas básicas y fundamentales a que realmente deberá sujetarse en el futuro toda urbanización.

POR TANTO:

En uso de sus facultades constitucionales y a iniciativa del poder ejecutivo,

DECRETA la siguiente:

LEY DE URBANISMO Y CONSTRUCCION

Art.1 El Viceministerio de Vivienda y Desarrollo Urbano, será el encargado de formular y dirigir la política Nacional de Vivienda y Desarrollo Urbano; así como de elaborar los Planes Nacionales y Regionales y las disposiciones de carácter general a que deben sujetarse las urbanizaciones, parcelaciones y construcciones en todo el territorio de la República

La elaboración, aprobación y ejecución de planes de Desarrollo Urbano y Rural de la localidad, corresponde al respectivo Municipio, los que deberán enmarcarse dentro de los planes de Desarrollo Regional o Nacional de Vivienda y Desarrollo; en defecto de los planes de Desarrollo Local, tendrán aplicación las disposicones de carácter general y los planes a que se refiere el inciso primero de este artículo.

Cuando los Municipios no cuenten con sus propios planes de Desarrollo Local y Ordenanzas Municipales respectivas, todo particular entidad oficial o autónoma, deberá solicitar la aprobación correspondiente al Viceministerio de Vivienda y Desarrollo Urbano, antes que a cualquier otra oficina, para ejecutar todo tipo de proyecto a que se refiere este artículo.

Art. 2 Para que el Viceministerio de Vivienda y Desarrollo Urbano, pueda otorgar la aprobación a que alude el artículo anterior, es indispensable que los interesados hayan llenado los requisitos siguientes:

a) Levantamiento topográfico del terreno, con curvas de nivel a un metro de equidistancia como máximo.

b) Clase de urbanización, con indicación del respectivo parcelamiento.

c) Proyecto de calles principales y secundarias.

d) Resolución del problema de vía de comunicación con el resto de la Ciudad y sus alrededores.

e) Destinar para jardines y parques públicos una fracción de terreno equivalente al 10 %, como mínimo, del área útil del inmueble a urbanizar, cuando se ubique en las Ciudades o centros poblados existentes; y 12.5 metros cuadrados, como mínimo por lote a parcelar, cuando se ubique fuera de los centros poblados existentes.

Su ubicación deberá ser adecuada a los fines mencionados.

El reglamento respectivo establecerá las excepciones, así como el equipamiento en cada caso.

f) Reservar espacios de terreno suficientes para la instalación de los sevicios públicos necesarios cuya especificación y ubicación quedará a juicio del Viceministerio de Vivienda y Desarrollo Urbano.

g) Destinar para escuela un terreno cuyo tamaño deberá ser el equivalente a 8 metros cuadrados por lote a parcelar o urbanizar.

El reglamento respectivo establecerá las excepciones del caso.

h) Resolución de factibilidad emitida por el organismo correspondiente del problema de agua potable, drenaje completo de aguas lluvias, aguas negras, alumbrado eléctrico, servicio telefónico indicando sus conexiones con los servicios públicos ya establecidos.

i) Especificar la clase de materiales que se piense usar para las obras de agua potable, aguas lluvias, aguas negras, cordones, cunetas y tratamiento de las superficies de las vías de tránsito.

j) Los Planos topográficos y planimétricos serán presentados a una escala no menor de 1:500 y los planos denominados "perfiles" serán presentados a escala no menores de 1:50 en la vertical y de 1:500 en la horizontal. Además, para grandes conjuntos se deberá presentar un plano adicional a una escala 1:1000.

En los espacios de terreno a que se refiere las letras e) y g), quedan obligados los urbanizadores a realizar las obras a que las mismas comprenden; pero pueden exonerarse de tales obligaciones donando irrevocablemente el dominio de los referidos terrenos a la Municipalidad respectiva, sí no se principian y concluyen estas obras en el tiempo que el reglamento de esta ley determine.

Lo ordenado en las letras e), f), g) y h) del presente artículo será exigible de conformidad con el reglamento respectivo cuando así lo amerite la extensión del área a urbanizarse o parcelarse y la población que en ella ha de residir.

Art. 3 Los materiales a usarse en las obras de urbanización tendrán que llevar el visto bueno del laboratorio de prueba de materiales del Ministerio de Obras Públicas.

Art. 4 No serán aprobadas aquellas urbanizaciones que consideren únicamente el estudio local y no incluyan la superficie a urbanizar como parte integrante de la zona metropolitana, lo mismo que aquellas urbanizaciones cuyo proyecto y construcción no sean ejecutadas por ingenieros civiles o arquitectos autorizados legalmente para el ejercicio de la profesión en la República.

Art. 5 Las personas o instituciones que hubieren obtenido la aprobación a que alude el Art. 1 de esta ley, estarán en la obligación de dar aviso por escrito dentro de los ocho días hábiles subsiguientes, al Viceministerio de Vivienda y Desarrollo Urbano o a la respectiva Municipalidad, según el caso, para fines de supervigilancia técnica, de las correspondientes fechas en que habrán de dar comienzo a la realización de las obras respectivas. El no cumplimiento de la obligación anterior, hará incurrir a los infractores en una multa del 25% del valor del terreno a parcelar o urbanizar incluyendo el valor de la constucción si fuere el caso; multa que será exigible por los Municipios de conformidad a leyes y reglamentos. Si las obras no se estuvieren realizando de conformidad a los planos y especificaciones aprobadas, se podrá ordenar su suspensión y corrección; y si ya se hubieren llevado a efecto, se podrá ordenar su demolición a costa del infractor.

Art. 6 La autorización para realizar una parcelación o urbanización con base en los respectivos proyectos aprobados, tendrá vigencia por un año; contado a partir del día siguiente de la aprobación correspondiente.

Si transcurrido el plazo señalado en el inciso anterior no se hubieren iniciado las obras, será indispensable para ello, obtener del Viceministerio de Vivienda y Desarrollo Urbano o de la respectiva Municipalidad, según el caso, una nueva aprobación de los planos respectivos.

Art. 7 Se tendrán por caducados y sin ningún efecto ni valor, las aprobaciones que hayan sido otorgadas con anterioridad a la fecha de la vigencia de la presente Ley, sobre urbanizaciones que no se hayan iniciado en la fecha de referencia.

Art. 8 Todo proyecto de construcción de edificios que se desee llevar a efecto, ya sea por particulares, entidades oficiales, edilicias o autónomas, deberá ser elaborado por un Arquitecto o Ingeniero Civil autorizado legalmente para el ejercicio de la profesión en la República, e inscrito en el Registro Nacional de Arquitectos, Ingenieros, Proyectistas y Constructores; debiendo, además, figurar su firma y sello en los correspondientes planos que presente al Viceministerio de Vivienda y Desarrollo Urbano o a la respectiva Municipalidad, según el caso; y la realización de las respectivas obras de construcción deberán ser ejecutadas y supervisadas, también por un Arquitecto o Ingeniero Civil legalmente autorizado e inscrito en el Registro referido.

Exceptuándose de los dispuesto en el inciso anterior, las construcciones de bahareque, adobe y las de ladrillo y sistema mixto de un sólo piso y techo con estructura de madera, lo mismo que las construcciones de madera de un sólo piso.

Todas estas obras podrán ser proyectadas y construidas por Proyectistas y Constructores de reconocida capacidad, inscritos en el Registro a que alude el inciso anterior; sujetándose a las normas que para tal clase de construcciones establezca el Viceministerio de Vivienda y Desarrollo Urbano. En todo caso cuando se tratare de la construcción de edificios destinados a fábrica, talleres y otro género de instalaciones industriales o comerciales, no se otorgará la aprobación respectiva sin que la Dirección del Departamento Nacional de Previsión Social haya dictaminado antes, que el proyecto reúne las condiciones necesarias sobre seguridad e higiene del trabajo.

Art. 9 Las Alcaldías respectivas, al igual que las autoridades del Ministerio de Obras Públicas, estarán obligadas a velar por el debido cumplimiento de lo preceptuado por esta ley; debiendo proceder según el caso, a la suspensión o demolición de obras que se estuvieren realizando en contravención de las leyes y reglamentos de la materia, todo a costa de los infractores, sin perjuicio de que la respectiva Alcaldía Municipal les pueda imponer por las violaciones a la presente Ley y Reglamento, multas equivalentes al 10% del valor del terreno en el cual se realiza la obra, objeto de la infracción.

Cuando el Viceministerio de Vivienda y Desarrollo Urbano o las Alcaldías Municipales soliciten el auxilio de los distintos cuerpos de seguridad para el cumplimiento de sus resoluciones o para evitar infracciones a la presente Ley o cualquiera otras leyes reglamentos relativos a construcciones , urbanizaciones, parcelaciones o cualquier otro desarrollo físico; se les proporcionará de inmediato; también deberán colaborar con esa misma finalidad el resto de las instituciones gubernamentales, edilicias o autónomas involucradas en el desarrollo urbano.

Art. 10 En caso de denegarse la aprobación de proyectos de urbanización o de construcción, podrán los interesados apelar la respectiva resolución dentro de los tres días subsiguientes al de su notificación, para ante el Ministerio de Obras Públicas, el que resolverá únicamente con vista de autos y la sentencia que pronuncie causará ejecutoria y no admitirá más recurso que el responsabilidad..

Art. 10 bis El Presidente de la República emitirá los Reglamentos que fueren necesarios para facilitar la aplicación y ejecución de la presente Ley.

Art. 11 Quedan derogadas todas las disposiciones que de un modo u otro se opusieren a lo preceptuado por la presente Ley.

Art. 12 El presente Decreto entrará en vigencia, ocho días después de su publicación en el Diario Oficial.

Dado en el Salón de Sesiones de la Asamblea Legislativa; Palacio Nacional: San Salvador a los cuatro días del mes de junio de mil novecientos cincuenta y uno.

Reformas según Decreto Legislativo No. 708 del 13 de febrero de 1991. D.O. No. 36 Tomo 310 del 21 de febrero de 1991

REPUBLICA DE EL SALVADOR EN LA AMERICA CENTRAL

LEY DE URBANISMO Y CONSTRUCCION

REGLAMENTO A LA LEY DE URBANISMO Y CONSTRUCCION EN LO RELATIVO A PARCELACIONES Y URBANIZACIONES HABITACIONALES

JULIO 1996

SAN SALVADOR, EL SALVADOR, C.A.

VICEMINISTERIO DE VIVIENDA Y DESARROLLO URBANO

Appendix B - Regulations for Structural Safety of Constructions

CONTENTS

TITLE I

GENERAL REQUIREMENTS

1.1 SCOPE

ARTICLE 1. This technical standard sets forth the minimum requirements for structural design, execution, structural supervision, and the use of constructions with the following objectives:

1. Guarantee structural and service safety conditions in normal operation conditions and moderate seismic events.

2. Minimize the possibilities of collapse of structures and the loss of lives and injury to human beings in case of a severe seismic event.

3. Keep the best possible functioning of those buildings that offer services or that store essential facilities for recuperation after a catastrophe.

ARTICLE 2. The design procedures for earthquake and wind actions, and the specific requirements for design and construction for some materials and structural systems are defined in the technical standards that are part of this regulation.

ARTICLE 3. The requirements of this regulation are applicable to new constructions and the existing ones that may be subject to modification, reparation, or demolition and shall be followed in all the country.

ARTICLE 4. Those special structures such as bridges, tunnels, piers and others, with structural design requirements not set forth in this regulation, shall be designed according to internationally known standards using the parameters of this regulation and its technical standards, leaving them clearly established in the corresponding computing report.

ARTICLE 5. If the requirements contained in this regulation and its technical standards are not followed a penalization according to article 9 of the law shall apply.

ARTICLE 6. In order to bring up to date and inspect this regulation and its technical standards, the technical commission of structural safety shall be created as an advisory organism of the Ministerio de Obras Publicas. Its main functions shall be:

1. Study and propose reforms to these instruments in order to incorporate them to the corresponding technological and scientific advances.

2. Evaluate any innovative system or material that is attempted to introduce concerning structural design.

3. Verify fulfillment of this regulation and its standards in those projects that according to the committee, either work, accusation or warning, require it.

4. Collaborate in the evaluation of structural damages caused by a catastrophe.

ARTICLE 7. the technical committee of structural safety shall be formed by an agent of the Ministerio de Obras Publicas, an agent of the Salvadorian Association of Engineers and Architects, an agent of the School of Architects of El Salvador, an agent of the Camara Salvadoreña de la Industria de la Construccion, an agent of the Salvadorian Society of Seismic Engineering, An agent of the Universidad de El Salvador, An agent of the Universidad Centroamericana “Jose Simeon Cañas” and an agent of the Universidad Albert Einstein.

ARTICLE 8. The technical committee of structural safety shall meet at least once a month and the necessary times when it is convoked by the Ministerio de Obras Publicas.

TITLE II

STRUCTURAL DESIGN CRITERIA

CHAPTER 1

GENERAL

ARTICLE 9. Structures and each of its parts shall be designed to meet the following basic requirements:

1. Provide adequate safety against the occurrence of any possible failure limit state given the combinations of the most unfavorable actions that might come during its useful life.

2. Never exceed any service limit state given the combinations of the actions that correspond to normal operating conditions.

ARTICLE 10. Any situation that corresponds to the exhaustion of the bearing capacity of the structure or any of its components shall be considered as a failure limit state, including foundations or the fact that any irreversible damage occur that affect significantly the resistance, given other load applications.

The most important failure limit states for each material and for each type of structure are set forth in the technical standards.

ARTICLE 11. it shall be considered as a service limit state any occurrence of deformations, cracking, vibrations or damage that affect the correct functionality of the construction, but they shall not affect its capacity to resist loads.

The service limit state shall be considered as fulfilled if it is proven that the values of its effects do not exceed the ones specified in the corresponding technical standards.

ARTICLE 12. In the design of structures the effects of permanent, variable and accidental actions shall be considered. The intensities of these actions, the way they shall be integrated or combine and the way to analyze their effects in structures shall be in accordance with the general criteria established in this chapter.

ARTICLE 13. Three categories of actions shall be considered, according with the time they last with their maximum intensity over structures:

1. Permanent actions, Qp, are the ones that act in a continuous matter over the structure, varying their intensity only a little with time. The main actions that belong to this category are: dead load, pressure of liquids and the deformations and displacements superimposed to structures that vary only a little with time, such as the ones due to prestress or permanent differential movements of the supports.

2. Variable actions, Qv, are the ones that act over the structure with an intensity that varies significantly with time. The most important actions that belong to this category are: live load, temperature effects, earth pressures,

superimposed deformations and differential settlements with a variable intensity with time and the actions due to equipment functioning, including the dynamic effects that might occur by vibrations, impact or braking.

3. Accidental actions, Qa, are the ones that are not given by the normal functioning of construction and might reach significant intensities only during brief times. Actions belonging to this category are: seismic actions, wind effects, explosions actions, fires and other phenomenon that might occur in extraordinary cases. It shall be necessary to be cautious while structuring and when detailing for construction, in order to avoid a catastrophic behavior of the structure in the case that one of these actions occur.

ARTICLE 14. When the effects of actions with intensities not specified in this regulation or in its technical standards shall be considered in the design, these intensities shall be established based on the following general criteria:

1. For permanent actions, the variability of the dimensions of the elements, the volumetric weights, and other relevant properties of materials shall be considered, therefore a maximum probable value of intensity shall be considered. When the effect of the permanent action is favorable to the stability of the structure, a minimum probable value of intensity shall be determined.

2. For variable actions the following intensities that correspond to the combinations of the actions for which the structure shall be revised shall be determined:

a) The maximum intensity shall be determined as the maximum probable value during the expected life of the construction. It shall be used to be combined with the effects of the permanent actions.

b) The instant intensity shall be determined as the maximum probable value in the period of time an accidental action might occur, such as earthquakes or wind; it shall be employed for combinations that include accidental actions or more than one variable action.

c) The mean intensity shall be estimated as the average value an action might have in a period of time of many years, and shall be employed to estimate long term effects.

d) When the effect of the action is favorable to the stability of the structure, its intensity shall be taken as zero.

3. For accidental actions the value that corresponds to a return period of 50 years shall be taken as the design intensity.

The supposed intensities for the actions not specified shall be justified in the computing report and a record shall be kept in the structural drawings.

ARTICLE 15. The safety of a structure shall be verified for the combined effect of all the actions that have a non worthless probability to occur simultaneously, considering two combination categories:

1. Combinations that include permanent and variable actions. All permanent and variable actions that act over the structure shall be considered, concerning variable actions, the most unfavorable shall be taken with its maximum intensity and the rest with their instant intensity or all of them with the mean intensity if the long term effects are being evaluated.

For the combination of dead load plus live load, the maximum intensity of the live load shall be considered from article 26 in this regulation, considering it uniformly distributed over all the area. When other live load distributions more unfavorable than the uniformly distributed are considered, the values of instant intensity specified in article 26 shall be taken.

2. For combinations that include permanent, variable and accidental actions, all permanent actions, the variable actions with its instant values and only one accidental action in each combination shall be considered.

In both types of combinations, the effects of all the actions shall be multiplied by the appropriate load factors, according to article 21 in this chapter.

ARTICLE 16. Internal forces and deformations produced by actions shall be determined by a structural analysis performed by a recognized method that includes the properties of the materials given the load types that are being considered.

ARTICLE 17. Resistance shall be understood as the magnitude of an action, or a combination of actions, that causes a failure limit state of the structure to appear or any of its components.

In general, the resistance shall be expressed in terms of the internal force, or combination of the internal forces, that correspond to the maximum capacity of the critical sections of the structure. Internal forces shall be understood as the axial and shear forces and the bending and torsion moments that act in one section of a structure.

ARTICLE 18. The procedures to determine the design resistance and the resistance factors corresponding to materials and most common constructive systems shall be established in this regulation. In order to determine the design resistance before failure limit states of foundations, the procedures and resistance factors specified in chapter 6 of this title and the corresponding technical standards shall be employed.

In those cases that are not treated in the documents mentioned above, the design resistance shall be determined with analytic procedures based on theorical and experimental evidence, or with experimental procedures according to the following article of this regulation.

ARTICLE 19. The determination of the resistance may be carried out by tests designed to simulate, through physical models of the structure or portions of it, the effect of the combinations of the actions that shall be considered according to article 15 of this title.

When the structures or structural members that are produced in an industrialized matter shall be considered, the tests shall be done over samples of the production or prototypes. In other cases, the tests may be done over models of the structure.

The parts of the structure that shall be tested and the loading system that shall be applied shall be chosen in such a way that the most unfavorable conditions that may occur in reality are obtained, but including the interaction with other structural elements.

Based on the results of the tests, a design resistance shall be deduced, taking in count the possible differences between the mechanical properties and geometric measures of the tested specimens and the ones expected in real structures.

The type of test, the number of specimens and the criteria to determine the design resistance shall be set based on probabilistic criteria, which may be verified through load tests.

ARTICLE 20. it shall be revised that for the different combinations of actions specified in article 15 of this title and for any possible failure limit state, the design resistance shall be greater or equal to the effect of the actions that intervene in the load combinations studied, multiplied by the corresponding load factors, according to the specified in article 21 of this title.

It shall also be revised that under the effect of the possible combinations of actions without multiplying the load factors, any service limit state is exceeded.

ARTICLE 21. The required resistance U, that shall resist the dead load CM and the live load CV, shall be at least equal to:

$$U=1.4CM+1.7CV \quad (1)$$

If during design the resistance of the structural effects of a specified earthquake load, S, is included, the following combinations of CM, CV and S shall be investigated to determine the greater required resistance U:

$$U=0.75(1.4CM + 1.7CV+1.87S) \quad (2)$$

Where the load combinations shall include the total value and the zero value of CV to determine the most critical condition and

$$U=0.9CM + 1.43S \quad (3)$$

But in any combination CM, CV, and S, shall the required resistance U be less than the one required by equation (1).

If the resistance of wind loads, V, shall be included in the design, the load combinations of equations (2) and (3) shall be applied excepting that 0.91V shall substitute S.

If the resistance of lateral earth pressure, E, shall be included, the required resistance U shall be at least equal to:

$$U=1.4CM + 1.7CV +1.7E \quad (4)$$

Except when CM or CV reduce the effect of E, 0.9 CM shall substitute 1.4CM and the zero value of CV shall be used to determine the greatest required resistance U. in any combination of CM, CV, or E, shall the required resistance U be less than the one required by equation (1).

When the resistance of loads caused by weight or pressure of liquids with well known densities and well controlled maximum depths, Pl, are included in the design, such loads shall have a load factor equal to 1.4 that shall be added to all the load combinations that include live load.

If during design the resistance to the impact effects is taken in count, these shall be included in the live load CV.

When the structural effects T of the differential settlements, yield, contraction or temperature changes are significant in the design, the required resistance U shall be at least equal to:

$$U=0.75(1.4CM+1.4T+1.7CV) \quad (5)$$

But the required resistance U shall not be less than:

$$U=1.4(CM+T) \quad (6)$$

The differential settlement, yield, contraction or temperature changes estimation shall be based on a realistic determination of such effects during the service of the structure.

In order to design for work stresses or to revise service limit states, a load factor equal to unity shall be taken in all cases.

ARTICLE 22. Different design criteria from the ones specified in this chapter and the technical standards may be used as long as it is substantiated that the design procedures used result in safety levels equal or greater than the ones specified here.

CHAPTER 2

DEAD LOADS

ARTICLE 23. It shall be considered as dead loads the weights of all the constructive elements, finishes and all the elements that take a permanent position and that have a weight that will not change substantially with time.

In order to evaluate dead loads the specified dimensions of the constructive elements and the unit weights of the materials shall be used. For unit weights of materials the minimum probable weights shall be used when it is more unfavorable for the stability of the structure, as in the case of overturning, floating, pressure or suction produced by the wind. In other cases maximum probable values shall be used.

CHAPTER 3

LIVE LOADS

ARTICLE 24. The weights that are produced by the use and occupancy of the constructions and that do not have a permanent character shall be considered as live loads. Unless other values are rationally justified, these loads shall be taken equal to the specified in the table of minimum unit live loads.

The specified loads do not include the weight of equipment, furniture or objects of uncommon weight, such as strongboxes, heavy shelves or drapery in spectacle halls.

When such loads are foreseen, they shall be quantified and be included in the design independently from the specified live load, and the adopted values shall be justified in the calculus report and shall be indicated in the structural drawings.

ARTICLE 25. For the application of the unit live loads the following requirements shall be considered:

1. The maximum live load Wm shall be used for structural design as a gravity force and to calculate immediate settlements in soils, and during structural design of foundations given the gravity loads.

2. The instant live load Wi, shall be used for seismic and wind analysis.

3. The mean live load W shall be used when calculating deflections and differed settlements.

4. When the effect of the live load is favorable for the stability of the structure, as in the case of flotation, overturning and suction produced by wind, its intensity shall be considered as zero in the whole area.

5. Uniform live loads of the table of minimum unit live loads shall be considered distributed over the tributary area of each element.

ARTICLE 26. In order to guarantee the safety of the structures during the construction process, constructions shall be propped adequately to be able to resist the transitory live loads that might be produced, which shall include the weight of: materials that are temporally stored, the vehicles and equipment, pending concrete pouring in stories that are supported above the analyzed story and the necessary personnel to be able to fulfill these activities, which shall be at least 150 Kg/m2.

ARTICLE 27. The owner of a building shall be responsible of the damage that is caused by the change of use of the building.

CHAPTER 4

EARTHQUAKE DESIGN

ARTICLE 28. The basis and minimum general design requirements are established in this chapter so that structures have an adequate safety given the seismic effects. The methods of analysis and the requirements for specific structures are detailed in the technical standard for earthquake design.

ARTICLE 29. Structures shall be analyzed under the action of two non simultaneous horizontal orthogonal components of ground motion, specified in the technical standard for earthquake design. Deformations and internal forces that result shall be combined between them as specified in the technical standards, and shall be combined with the effects of gravity loads and other actions that correspond according to the criteria established in Title II in this regulation.

Structures may be analyzed by seism effects by the static method or one of the dynamic procedures described in the technical standard for earthquake design, with the limitations established there.

During the analysis, the stiffness of all the resisting elements shall be included. Seismic forces, deformations and lateral displacements of the structure, including rotations caused by torsion and keeping in mind bending effects of its elements and, when they are significant, the effects of shear forces, axial forces and torsion of the elements shall be calculated, also, the second order effects, that are understood as the ones of the gravity loads acting on the deformed structure given the actions of those forces and the lateral ones shall be calculated.

It shall be verified that the structure and its foundations never reach any failure or service limit state referred in this regulation.

ARTICLE 30. All buildings shall be separated from its boundaries with the neighboring lands a distance at least equal to the one specified in the technical standard for earthquake design. The previous regulation shall be applied for the separation between the different structural bodies of a building. The separations shall be free from any rigid materials that might cause any obstruction. At the same time they shall be detailed in such a way that rain water infiltration or any other agent that affects the correct functioning of buildings shall be avoided.

The details of these separations shall be clearly indicated in the architectural and/or structural drawings.

ARTICLE 31. Special care shall be taken when designing the anchoring or supporting of decorative elements whose falling might cause damage or injure people or buildings.

ARTICLE 32. All resisting elements of buildings shall be designed in such a way that they behave as a unit given the seismic actions, unless some of them are adequately separated from the principal structure, according to the separations indicated in the technical standard for earthquake design.

ARTICLE 33. Furniture, electromechanical equipment and other elements whose overturning, break or detachment might injure or damage people or facilities, or whose damage might affect the correct functioning of the building, shall be adequately anchored in order to resist the loads prescribed in the technical standard for earthquake design.

ARTICLE 34. affixed, hanging and in roofs announcements, whose failure might cause damage or affect the correct functioning of the building, shall be object of structural design in the terms of this title, particularly wind effects. Its supports and anchoring to the principal structure shall be designed and the effect to the stability of the structure shall be checked.

ARTICLE 35. Any modification in a structural

element to store installations or other purposes shall be designed and detailed indicating the necessary reinforcements.

Pipes of installations in general that cross separations between different building bodies will not be allowed unless flexible tracts or connections are provided.

CHAPTER 5

WIND DESIGN

ARTICLE 36. In this chapter the basis for revising the safety and service conditions of structures given the wind effects are established. The design requirements are detailed in the technical standard for wind design.

ARTICLE 37. The global stability of the structure to wind effects shall be analyzed. It shall also be considered the effect of the interior pressures in constructions where significant openings exist.

ARTICLE 38. Local effects in structural and non structural elements shall be analyzed, these elements include front elements, affixed and hanging announcements and also the ones located in roofs that are directly exposed to that action.

ARTICLE 39. The direction that produces the most unfavorable effects in the structure shall be investigated. In buildings and in regular structures it is enough to revise independently the action of wind in the two orthogonal directions that match with the principal axes of the structural system.

ARTICLE 40. The wind action shall be combined with the permanent and variable actions according to article 21 in this regulation.

ARTICLE 41. For structures classified in the technical standard for wind design as type 1, the effect of wind shall be evaluated by equivalent static pressures.

ARTICLE 42. For structures classified in the technical standard for wind design as type 2 and 3, special procedures that include the dynamic characteristics of the wind action are required.

ARTICLE 43. The wind velocity and the pressures produced by this velocity shall be evaluated according to the procedures established in the technical standard for wind design.

CHAPTER 6

FOUNDATION DESIGN AND STABILITY OF SLOPES

ARTICLE 44. The basis and general minimum requirements for design and construction of foundations and stability of slopes are established in this chapter.

ARTICLE 45. Previous to the design of any project, a geotechnical study must be done as established in articles 46 and 47.

ARTICLE 46. The geotechnical study referred in article 45 shall contain as minimum, the following: permissible bearing capacity, identification and classification of soil, humidity conditions, consistency limits, presence of polluting agents and underground water flows; and also the definitions of the minimum foundation depth, at levels under which there are no harmful amounts of organic matter and where the soil has minimum acceptable characteristics.

Additionally, all those properties required for the analysis and design of the project shall be studied.

ARTICLE 47. For structures classified as occupancy category I, according to the established in the technical standard for earthquake design, besides the indicated in article 46, the identification and location of the geological faults shall be considered, and also any additional tests that by the geotechnist and the structural designer criteria are required.

ARTICLE 48. In the design and construction of foundations, the phenomenon of erosion, undercut, piping and liquefaction of soils shall be prevented.

ARTICLE 49. Fillings under foundations shall be done with appropriate and compacted material to the specified density.

ARTICLE 50. When buildings on natural or artificial slopes or on terraces are planned, a general and local stability analysis of the loaded slopes shall be performed, including the required protection works.

ARTICLE 51. Slopes shall be analyzed and/or be designed taking in count the characteristics of the materials of the soil layers, the probable failure mechanisms and also consider the gravity and seismic loads, forces caused by infiltration, pore pressure, overloads and other that influence in the stability of structures. Also the load combinations specified in title II of this regulation shall be considered.

ARTICLE 52. Buildings located near slopes shall be protected adequately with protection works and/or protection zones.

ARTICLE 53. Slopes shall be composed with an integral superficial drainage system and adequate protection works, with the purpose of preventing damage caused by saturation, erosion and undercut. Periodical maintenance shall be provided to such system in order to preserve its efficiency.

ARTICLE 54. slopes shall be constructed including the recommendations made in the geotechnical report written for this purpose.

ARTICLE 55. Protection works constructed to provide with stability land level differences, shall be designed to resist the gravity and dynamic forces, in such a way that the failure and service limit states are not exceeded.

ARTICLE 56. Protection works located near other buildings shall be separated adequately so that collisions between them caused by displacements by dynamic actions can be prevented.

ARTICLE 57. Exposure of the protection works to erosion and undercut shall be avoided, particularly on the surface where the passive pressure wedge is generated and on the foundation.

ARTICLE 58. Faces of excavations with the possibility of instability shall be propped appropriately.

ARTICLE 59. When excavations are made near slopes and buildings, the necessary cautions shall be taken to avoid damages that might affect the stability of the constructions.

TITLE II

REPAIRINGS, MODIFICATIONS, DEMOLITIONS AND MAINTENANCE

ARTICLE 60. Every construction that has been visibly affected by wind, earthquake, explosion, fire, ground settlement, excess of loads, constructive processes or deterioration of materials shall be performed a preliminary revision of the structure to carry out the appropriate safety measures for this case, in a time not greater than 72 hours, ruled by a qualified civil engineer or an architect.

ARTICLE 61. The project of structural reinforcement of a construction shall meet at least the following:

1. The safety requirements for new structures established in this regulation and its technical standards.

2. Be based on structural drawings of the original project and/or a detailed investigation of all the structural elements.

3. Detect, locate and evaluate any damage occurred in the structure, for which all coverings and finished that hide them shall be removed.

4. Perform a geotechnical study.

5. Investigate the status of potable water, sewerage and rainwater pipe lines.

6. Determine the resistance of the structural materials of the building.

7. Measure around to detect leanings and unlevelings.

8. Perform the corresponding analysis to determine the probable causes of the damage of the structure.

9. Perform the necessary analysis in order to guarantee that the original and the structuration of the reinforcement resist together the loads specified in this regulation and its technical standards.

10. Draw the corresponding structural drawings including the necessary details for the correct execution of the project of structural reinforcement.

11. Detail and locate the necessary provisional propping and reinforcement during the phase of execution of the reinforcement project.

ARTICLE 62. If in accordance with article 60 it is necessary to demolish a construction, totally or partially, the following shall be met:

1. Prohibit access to any unauthorized person.

2. Perform the demolition process according to technical studies in such a way that it will not affect the safety of the people and their goods.

3. Respect the requirements set forth in this regulation and in other laws and regulations.

ARTICLE 63. Any construction where its structural configuration is modified, in the dimensions of the structural elements and in its use, shall be subject to the verdict of a specialist in structures and meet with the requirements of article 61.

ARTICLE 64. It is the responsibility of all owners of a building to give preventive and corrective maintenance to all and each of the parts of the building.

TITLE IV

STRUCTURAL INSPECTION

ARTICLE 65. Structural inspection is the set of activities of control so that the constructive process of the structure of a project, complies with the drawings and design specifications and that these are done according to this regulation and the corresponding technical standards. The structural inspection shall be in charge of a civil engineer with notorious experience and capacity in the field; other construction professionals shall be subject to the established in article 8 of the “Ley de Urbanismo y Construction”.

ARTICLE 66. It is obligatory to supervise the structure of projects classified in the technical standard for earthquake design with an occupancy category I and II, and also any building dedicated to commerce, office and/or housing by floors and apartments. Any structure that will be reinforced or modified according to articles 61 and 63 shall also be supervised.

ARTICLE 67. Structural inspection shall verify as a minimum the following:

1. There is a geotechnical study, computing report, technical specifications and complete structural drawings for any project.

2. There shall be consistency between the previously mentioned documents and the project to be constructed.

3. Drawings and specifications shall contain the necessary information to execute the project.

4. The minimum requirements of this regulation shall be met.

ARTICLE 68. During the constructive process, structural inspection shall watch over the accomplishment of the following:

1. That the structure is constructed according to the structural drawings and specifications of the project.

2. That the structural materials used meet the requirements.

3. That the project is developed according to the specified in the corresponding technical standard.

4. That the changes that are proposed during the constructive process that affect the structural safety, shall be considered by the structural designer.

5. That any change shall be registered in drawings.

TITLE V

SEISMIC SAFETY OF THE VITAL SYSTEMS OF PUBLIC SERVICES

CHAPTER 1

SCOPE

ARTICLE 69. The minimum requirements that earthquake design of potable water systems, sewage, electrical power, and telecommunication shall meet are established in this title, with the purpose to maintain working the services that these services offer, given any severe seismic event.

ARTICLE 70. The requirements established in this title are applicable for new systems and for components of existing systems that are modified or repaired.

ARTICLE 71. The requirements of this title shall be applied to the design of the systems indicated in article 69, in all the country.

ARTICLE 72. The seismic design requirements indicated in this title shall be supplemented with the design requirements contained in the technical standards of the institutions in charge of potable water, sewage, electrical power and telecommunication supply.

CHAPTER 2

GENERAL REQUIREMENTS

ARTICLE 73. The supply systems mentioned in article 69, are classified as occupancy category I, according to the established in the technical standard for earthquake design.

ARTICLE 74. The systems referred to in article

73, shall be designed by specialized professionals, who will be responsible in the different disciplines that the formulation of the project requires. If during the execution of the project, any modification that varies the original design is made, these shall be performed by professionals equally qualified. Any modification shall be registered in the as made drawings.

ARTICLE 75. The necessary geotechnical studies shall be performed for the projects referred to in article 73, which shall include the geological characteristics of the zone where the project is located. These studies shall be performed according to the criteria established in the technical standard for foundation design and slope stability.

ARTICLE 76. The electrical and mechanical equipment, control boards, antennas and other elements, whose detachment or break might affect the correct functioning of the pumping stations, treatment plants transformation and transference substations telephone centrals and state radio and television stations, shall be adequately attached to resist the loads specified in title II of this regulation.

ARTICLE 77. The inspection of the projects and quality control of materials during the execution of the projects of public service systems shall be performed by responsible professionals in the disciplines that the project requires. For structural inspection the requirements established in title IV of this regulation shall be met.

CHAPTER 3

POTABLE WATER SUPPLY SYSTEMS

ARTICLE 78. For the uses of this regulation potable water supply systems shall be understood as the system that is composed by reception, conduction, treatment, storage and distribution.

ARTICLE 79. In the design of the conduction and principal distribution lines of potable water the following requirements shall be met:

1. Avoid crossing active geological faults and areas susceptible to important displacements, unless the indicated in article 80 of this title is complied.

2. Avoid locating pipelines in compressive lands, with low bearing capacity or susceptible to liquefaction. In the case where the location of the pipeline can not be avoided, corrective measures shall be taken to control the effect of these soils according to the indicated in the corresponding geotechnical study.

3. Provide flexible lines by choosing ductile pipes and flexible joints.

4. Joints shall be designed to resist the stresses and allow relative displacements that are induced by the ground motion in pipelines.

5. Install closing valves, considering the sites of geologic danger determined in the geotechnical study. These valves will allow locating and isolating damaged tracts.

6. Provide redundancy of the system, duplicating elements, designing alternate lines and/or connections with other systems.

ARTICLE 80. When the geotechnical study indicates the existence of active geological faults in the trace of the conducting lines and main distribution lines, and crossing those faults can not be avoided, the following recommendations shall be followed:

1. The pipeline shall cross the fault with an angle between 70° and 90° with respect to the alignment of the fault.

2. In pipelines that cross a fault, special flexible

joints shall be used in the tract determined by the corresponding geological study.

3. Install a closing valve in the pipeline, at the side of the fault and at the side with a greater pressure, at the distance determined by the corresponding geotechnical study.

4. In the case of a buried pipeline and in the length determined by the geotechnical study, the following shall be met:

4.1 The pipeline shall be placed at the lowest depth that the geometric project and the load conditions let it.

4.2 The material that covers the pipeline around shall be fine granular, non cohesive, with a low value of internal friction angle and with the minimum compaction that the load conditions require. The filling of the pit shall be performed according to the conditions specified in the project.

4.3 Alternatively the pipeline may be placed concentrically inside another one with an interior diameter at least 40 cm greater than the exterior diameter of the protected tract, supporting the internal pipeline on a layer of low dense material.

5. If the pipeline is not buried, it shall be supported on supports placed on the ground that permit the motion of the pipeline during an earthquake.

ARTICLE 81. In conduction lines, valves shall be installed in order to divide the pipeline and isolate tracts, in points defined by the characteristics of the project.

ARTICLE 82. Sanitary structures shall be designed including the effects of the permanent, variable and accidental actions, according to the specified in title II of this regulation and meet the following basic requirements:

1. Never pass over any service limit state given the combinations of the actions that correspond to the normal operating conditions.

2. Maintain its functioning given the combination of the most unfavorable seismic actions that might appear during its expected life.

3. Provide a structural design that decreases the possibilities of cracking in structures by an adequate quantity and distribution of reinforcement and joint spacing.

4. Minimize differential settlements, with the purpose of minimizing cracking in structures.

5. Achieve a complete justness under service conditions.

ARTICLE 83. In the location of sanitary structures, unstable, erosionable or susceptible to liquefaction soils shall be avoided. In the case that the location of these structures on these soils can not be avoided, corrective measures shall be taken to control their effect according to the indicated in the corresponding geotechnical study.

ARTICLE 84. In the seismic design of tanks the hydrodynamic effects of the stored liquid shall be included in addition to the inertia of mass of the whole, attending the established in the technical standard for earthquake design.

ARTICLE 85. In places where pipelines are connected to heavy equipment and to sanitary structures, flexible joints shall be used to avoid damage produced by differential damages in foundations and by the change of stiffness between the connected elements. These types of joints shall also be used in pipelines that cross control joints between structures.

ARTICLE 86. The diameter of the covering pipe of wells shall be at least 5 cm greater than the external diameter of the suctioning element to allow relative movement.

ARTICLE 87. Pumps and their engines shall be adequately mounted to prevent damage in their joints originated by differential settlements.

CHAPTER 4

SEWERAGE SYSTEMS AND RAINWATER SEWER SYSTEMS

ARTICLE 88. For the purposes of this regulation sewerage system shall be understood as the one composed by collector nets, main collectors emissaries, treatment plants and complementing works; and by rainwater sewer systems the one composed by collector nets, main collectors and complementary works.

ARTICLE 89. In the design of sewerage systems and rainwater sewer systems, the requirements set forth in numerals 1 and 2 of article 79 of this regulation shall be met.

ARTICLE 90. When the geotechnical study indicates the presence of active geological faults in the trace of collectors and it can not be avoided to cross those faults, the requirements established in numeral 1, 2 and 4 of article 80 of this regulation shall be met. It shall also comply with the following requirements:

1) At both sides of the fault, in the distance determined by the corresponding geotechnical study, concrete pipes without reinforcement or pipes made of non-ductile materials shall not be used, avoiding in all cases rigid joints.
2) At a distance of the fault, determined by the corresponding geotechnical study, emergency discharge points shall be established upstream of the sewer.

ARTICLE 91. Precast concrete pipelines used in sewerage systems and rainwater sewer systems shall meet the requirements specified in the technical standard for quality control of structural materials.

ARTICLE 92. Channel beams and their supports shall be designed to resist the combinations of the most unfavorable seismic actions that might appear during its expected life, according to the parameters established in this regulation. Channel beams shall also be untied adequately from the transition boxes, by means of flexible elements that allow differential movements between both structures, maintaining the conduct with justness.

ARTICLE 93. In connections of pipelines with register structures a consistent flexible joint shall be provided in an annular space between them, to avoid damage in pipelines by the effects of earthquakes. This space shall be filled with material that guarantees staunchness and flexibility in the connection.

CHAPTER 5

ELECTRICAL POWER SUPPLY SYSTEMS

ARTICLE 94. The seismic design criteria treated in this chapter is applicable to the following components of an electrical power supply system: buildings in power stations, transformation and transmitting substations and transmitting and distribution lines.

ARTICLE 95. When locating buildings of power stations, transformation and transmitting substations, towers and piles of transmitting and distribution and transmitting lines, unstable, erosionable or susceptible to liquefaction soils shall be avoided. In the case where it can not be avoided to locate these structures on these types of soils, measures to control their effect shall be taken according to the specified in the corresponding geotechnical study.

ARTICLE 96. Mechanical and electrical

equipment and control boards that are used with the electrical power supply systems shall be guaranteed to behave adequately given the most unfavorable seismic actions expected in the zone, according to the established in the technical standard for earthquake design.

ARTICLE 97. In the design of facilities for substations, the use of damping elements for the porcelain pieces shall be considered.

ARTICLE 98. In the connections of collector bars or ducts of cables with massive equipments, flexible joints shall be used, which shall let absorb differential movements.

ARTICLE 99. In the design of towers and piles of high tension and distribution lines, the parameters established in title II of this regulation and its technical standards shall be met.

CHAPTER 6

TELECOMMUNICATION SYSTEMS

ARTICLE 100. The seismic design criteria set forth in this chapter is applicable to the following components of a telecommunication system: telephone centrals, distribution nets, link stations; and radio and television stations.

ARTICLE 101. When locating telephone centrals, link stations, parabolic antennas and radio and television stations, unstable, erosionable, or susceptible to liquefaction soils shall be avoided, in the case that the location of these structures on these types of soils can not be avoided, measures to control their effects shall be taken according to the specified in the corresponding geotechnical study.

ARTICLE 102. Commuter equipment, control boards and other special equipment, that are used in telecommunication systems and radio and television stations, shall be guaranteed to behave adequately given the most unfavorable actions expected in their zone, according to the established in the technical standard for earthquake design.

ARTICLE 103. For the design of towers and attachment elements of antennas of telecommunication systems and radio and television stations, the parameters established in title II of this regulation and its technical standards shall be considered.

TABLE OF MINIMUM LIVE LOADS, Kg/m^2

PURPOSE OF THE FLOOR OF COVER	w	wi	wm	observa-tions
a) habitation (apartments, housing, dormitories, hotel rooms, boarding schools, prisons, reformatories, hospitals and similar)…………………………….........	70	120	170	(1)
b) offices, classrooms and laboratories……	100	180	180	(2)
c) communications for pedestrians (corridors, stairs, ramps, halls and free accsess passages)……………………………	40	150	350	(3)
d) stadiums and meeting places without individual seats………………………………..	40	350	500	
e) stadiums with individual seats……………	40	200	300	
f) other places for meetings (temples, movie theaters, theaters, gymnasiums, dancing halls, restaurants, libraries, game rooms and similar)……………………………	40	250	350	
g) commerce, factories and warehouses….	0.8wm	0.9wm	wm	(4)
h) covers and roofs with an inclination not greater than 5%......................................	15	50	100	(3), (5)
i) covers and roofs with an inclination greater than 5%.......................................	5	0	20	(3), (5)
j) cantilevers in public ways (balconies, marquees and similar)……………………..	15	70	300	
k) garages and parking lots (exclusively for cars)……………………………………….	40	150	250	(6)

OBSERVATIONS TO THE TABLE

1. For elements with a tributary area, A, greater than 36m^2, wm may be reduced considering it equal to 100 + 420/(A)^1/2. when it is more unfavorable instead of considering wm, a load of 500 kg shall be considered applied on an area of 50x50 cm in the most critical location.
2. For elements with a tributary area, A, greater than 36m^2, wm may be reduced, considering it equal to 180+420/(A)^1/2. when it is more unfavorable instead of considering wm, a load of 1000 kg shall be considered applied on an area of 50x50 cm in the most critical location.
3. in the design of railings for covers, roofs and stair handrails, ramps, corridors and balconies, a

horizontal live load not less than 100 Kg/m shall be supposed acting at the level and at the most unfavorable direction.
4. According to the purposes of the floor it shall be determined with the criteria of article 24 the unit load wm, which shall not be less than 350 Kg/m^2 and shall be specified in the structural drawings and in metal plates located in places easily visible of the building.
5. Live loads specified for covers and roofs do not include the loads produced by water recipients and announcements, nor loads produced by equipment or heavy objects that can be supported or hanged. These loads shall be foreseen separately and shall be specified in the structural drawings. Additionally, the elements of covers and roofs shall be revised with a concentrated load of 100 kg applied at the most critical location.
6. Additionally a force of 1,500 Kg shall be applied at the most unfavorable location of the structural member.

Appendix C - Technical Standards for Foundation and Slope Stability

CONTENTS

CHAPTER 1

GENERAL REQUIREMENTS.

1.1 SCOPE.

1.1.1 This Technical Standard provides minimum requirements for design and construction of foundations and slope stability and is part of the “Regulation for Structural Safety of Constructions”.

1.1.2 This Technical Standard shall govern in all matters pertaining to soil properties for foundations and slope stability, except wherever this standard is in conflict with other requirements referred in this technical standard.

1.1.3 The design and construction requirements presented in this technical standard are applicable in all the country.

1.2 CONSTRUCTION DRAWINGS.

Construction drawings shall indicate the necessary detailing of foundations. The specifications shall include permissible soil pressure and other relevant aspects about soil treatment.

1.3 INSPECTION.

The construction processes of foundations and soil treatment shall be inspected throughout the various work stages.

CHAPTER 2

NOTATION.

2.1 NOTATION.

α: Angle defined by expression (5-5).

φ: Angle of internal friction.

Θ: Angle of inclination of the internal part of a wall with respect to a vertical plane.

δ: Angle of wall friction.

ß: Angle of inclination of the surface of soil with respect to a horizontal plane.

τ: Volumetric weight of soil.

H: Height of wall.

Ka: Active lateral earth pressure coefficient

Ea: Total force of active lateral earth pressure.

Kp: Passive lateral earth pressure coefficient.

Ep: Total force of passive lateral earth pressure.

Kh: Horizontal seismic coefficient.

Kv: Vertical seismic coefficient.

Kae: Active combined seismic lateral earth pressure coefficient.

Eae: Total force of active combined seismic lateral earth pressure.

Kpe: Passive combined seismic Lateral earth pressure coefficient.

Epe: Total force of passive combined seismic lateral earth pressure.

FS: Safety Factor.

Fg: Gravitational forces.

Fs: Seismic Forces.

Ff: Filtration induced forces.

CHAPTER 3
DESIGN CONSIDERATIONS.

3.1 APPLIED LOADS.

3.1.1 For foundation dimensioning and confirmation of pressures transmitted by structures to the soil, an elastic distribution of pressures can be adopted in the base of foundations, according to the working efforts method. The different load combinations established in title II of this standard shall be revised, such as impact effects and vibration of moving loads.

3.1.2 The maximum pressure transmitted by foundations under the greatest loads possible can not be greater than the permissible soil pressure, established according to this technical standard

3.1.3 For foundations supported directly under intact rock, the permissible soil pressure shall be based in the resistance of the intact part of the rock, considering the influence that cracked and decomposed rocks might have.

3.1.4 Regarding piles, the following shall conform the following:

1) The maximum transmitted load under the most severe load combination, shall not exceed the permissible load of the pile.

2) They shall be able to resist, besides a required compression force, a tension force equivalent to the shear strength of the transversal soil layers.

3) When they are prefabricated, they shall be designed to resist the transportation and handling stresses.

4) Their transversal dimension shall not be less than 0.25 m. The center to center spacing between piles shall not be less than 3 times its transversal dimension or 0.90 m, the greatest.

3.2 SOILS UNDER FOOTINGS

3.2.1 Regarding pad foundations, combined or strip, mats, shells or any combination of the previous foundations, the following recommendations shall be satisfied:

1) Never construct on soft or organic soil or on disposed matter.

2) It is necessary to determine the expansion or contraction degree of clays when they are detected in the foundation soil, and therefore, take the corrective measures when designing and constructing.

3) The vertical settlements, total or differential, that occur during construction and during its useful life shall not affect stability or the correct functionality of house facilities or the adjacent constructions.

4) The land where foundations stand shall be protected against internal erosion or decrement of its compactness by effects of saturation. Noxious superficial drainage, puddles or any kind of filtration shall be avoided.

When there is water flowing in the foundation zone, appropriate drainage shall be constructed.
In case of the possibility of liquefaction or tunneling, foundations of structures shall be laid deeper than those soil layers susceptible to this phenomenon or soils shall be practiced corrective measures.

3.3 FOUNDATIONS UNDER SLOPES.

Foundations located in sites near slopes, it will be verified that the slope is stable for transmitted static and dynamic loads.

In adjacent foundations and constructed at different levels, an intersection of the influence zone of the foundation of the superior level and the foundation in the lower level shall be avoided. If this is not possible, dimensioning of the inferior foundation will take in count the influence of the pressures induced by the foundation in the superior level.

3.4 COMPENSATED FOUNDATIONS.

In compensated foundations the weight of the building can be substituted totally or partially by the total weight of the dug soil.

The above mentioned is subject to verification that there are no soft soils, galleries, cracks or other defects underground.

3.5 MACHINE FOUNDATIONS.

To design machine foundations, vibration and impact forces produced by the operation of the equipment shall be considered and it will be verified that:

1) The static and dynamic stresses induced in the foundation shall not exceed the limits of the permissible capacity of the soil where it is supported.

2) The soil shall be able to resist the static and dynamic forces that are transmitted by foundations, without suffering important settlements.

3) The vibration produced in the foundations, for any load combination possible and/or operating velocities, shall not be denied.

3.6 FOUNDATION STRENGTHENING

Every strengthening foundation work will require to be practiced a geotechnical investigation, such as a detailed study that reasonably explains the causes of failure, or an investigation that relates existing and proposed structuring.

3.7 LIQUEFACTION.

It shall be considered that there is a potential for soil liquefaction when a geotechnical investigation shows that there are fine sand layers with more than 50% of its grains with sizes lower or equal to 2 mm (No. 10 sieve) and a relative density lower than 70% and when the phreatic level is close to the considered soil layer.

3.8 PROTECTION ZONES.

3.8.1 Retaining walls or slopes shall be provided with a protection zone between the top of slope and the constructions above it and between the toe of slope and the constructions under it.

3.8.2 The dimension of the protection zone shall be determined by an appropriate soil mechanics analysis.

3.8.3 The protection zones that are not provided with a permanent coating, shall be provided with appropriate vegetation, according to the soil surface and soil type.

3.8.4 When the existing land shows level changes greater than one meter, it shall be protected with outworks such as lawning slopes, pegs, live barriers, etc. slopes higher than 10 m shall be provided with an appropriate drainage.

CHAPTER 4

GEOTECHNICAL INVESTIGATION

4.1 SITE INSPECTION.

General information such as soil type in places surrounding construction shall be obtained, based on a site inspection and all concerning the type of structure to construct, its purpose and estimated loads that the structure shall transmit.

4.2 UNDERGROUND EXPLORING METHODS.

4.2.1 Standard penetration test (SPT).

Regarding this technical standard, it shall be verified that the standard penetration tests fulfill the following:

1) Number and spacing of perforations.

The number and spacing of underground explorations shall be determined by the structural designer together with the geotechnical engineer, in accordance with the individual characteristics of the site and the loads transmitted by the structure to the soil.

2) Perforation depth.

The minimum perforation depth, considered from the planned terrace levels, shall be the indicated bellow:

a) One story buildings, 3.0 m.

b) Two story buildings, 5.0 m.

In all cases, the geotechnical specialist shall
4.3.3 the allowable bearing capacity of a point bearing pile or friction pile shall be determined define the exploration depth; this specialist will be able to decide if the exploration depth shall be increased depending on the real soil conditions detected during the exploration process.

4.2.2 When loads are required to be transmitted directly to rock, the minimum exploration depth into the intact rock shall be 3.0 m. If the rock is cracked and/or weathered, the exploration depth shall be increased to the geotechnical specialist criterion.

4.2.3 Other procedures.

Alternatively to the standard penetration test (SPT), to obtain altered or unaltered samples, open sky pit could be practiced.

4.3 ALLOWABLE STRESSES AND BEARING CAPACITY.

Superficial and deep soil foundation allowable stresses or bearing capacity shall be estimated based on information obtained from a triaxial compression test, standard penetration test or any other underground exploration method commonly accepted.

4.3.1 The allowable stress of granular soils could be based on correlations with the standard penetration test.

4.3.2 The allowable stress of soils, based on triaxial tests, can be computed dividing the ultimate bearing capacity by a reduction factor of 3.

dividing the ultimate bearing capacity by a reduction factor of 3.

4.4 DYNAMIC PARAMETERS OF SOILS.

The dynamic parameters of soils could be determined using empirical correlations based on obtained results of standard penetration tests or any other accepted procedure.

In case of requiring more detailed information of the characteristics and dynamic properties of soils, geophysical tests such as “down hole” or “cross hole” can be practiced, correlations with the shear rigidity modulus, determined by procedures such as the simple torsion pendulum or any other accepted procedure can also be practiced.

4.5 GEOTHECNICAL REPORT.

As established in articles 46 and 47 of the Regulation, the geotechnical report shall contain at least the following information: permissible bearing capacity, soil identification and classification, humidity conditions, consistency limits, presence of contaminant agents and underground water flows; also, the minimum foundation depth, at levels where there are no organic materials and the soil has minimum acceptable characteristics. For structures classified with an occupancy category of I, the report, in addition, shall include the identification and location of geological faults and all additional tests that are required by the geotechnical specialist and the structural designer criterion.

CHAPTER 5

RETAINING WALLS.

5.1 SCOPE.

This chapter sets forth requirements for designing retaining walls and the protection measures are indicated.

5.2 GENERAL CONSIDERATIONS.

5.2.1 The acting forces to consider when designing retaining walls are: selfweight of the wall, lateral earth pressure, soil-wall friction, hydrostatic lateral pressure, overloads on the surface of the retained soil and seismic forces.

5.2.2 The design shall consider failure caused by overturning, sliding, stresses applied to the ground and required resistance of wall transverse sections.

5.2.3 Walls shall include a drainage system that lowers the effects of lateral hydrostatic pressure greater than considered during design.

5.3 STATIC AND DYNAMIC EFFECT.

The design shall include static forces such as active and passive lateral earth pressure, overloads applied on the embankment, hydrostatic lateral pressure and the specified seismic forces.

5.3.1 Static lateral earth pressure can be determined by any accepted method or by the Coulomb expressions that are shown,

1) Active Pressure.

Feil! Objekter kan ikke lages ved å redigere feltkoder. (5.1)

In equation 5.1
Feil! Objekter kan ikke lages ved å redigere feltkoder.

This can be taken as
Feil! Objekter kan ikke lages ved å redigere feltkoder.

Feil! Objekter kan ikke lages ved å redigere feltkoder. (5.2)

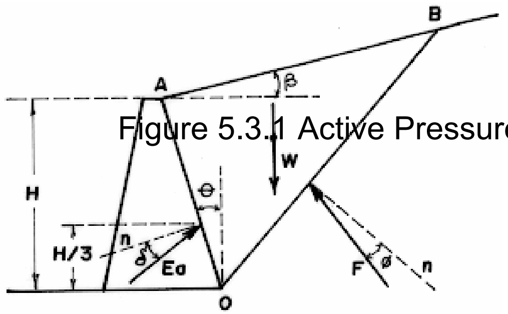


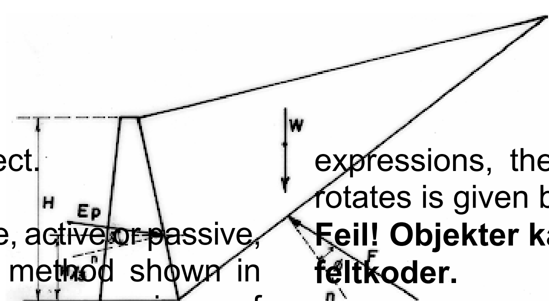
Figure 5.3.1 Active Pressure

2) Passive Pressure.

Feil! Objekter kan ikke lages ved å redigere feltkoder. (5.3)

Feil! Objekter kan ikke lages ved å redigere feltkoder. (5.4)

Figure 5.3.2 Passive Pressure



5.3.2 Combined seismic effect.

The combined earth pressure, active or passive, can be determined by any method shown in technical books, or by the expressions of Mononobe-Okabe shown here. In these expressions, the angle by which the ground rotates is given by

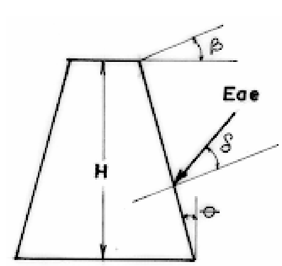
Feil! Objekter kan ikke lages ved å redigere feltkoder. (5.5)

1) Active Pressure.

Feil! Objekter kan ikke lages ved å redigere feltkoder. (5.6)

Feil! Objekter kan ikke lages ved å redigere feltkoder. (5.7)

Figure 5.3.3 Active and earthquake pressure combination



2) Passive Pressure.

Feil! Objekter kan ikke lages ved å redigere feltkoder.

(5.8)

Feil! Objekter kan ikke lages ved å redigere feltkoder.
5.9)

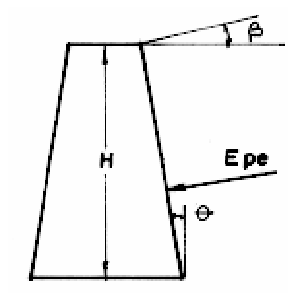


Figure 5.3.4 Passive and earthquake pressure combination

5.3.3 The seismic horizontal coefficient, kh, shall be taken equal to 0.16 for zone I and equal to 0.12 for zone II. The vertical seismic coefficient, kv, shall be taken equal to zero for both zones

5.3.4 Equations (5-7) and (5-9) express the sum of the active or passive pressure, respectively, combined with the seismic pressure, therefore, the seismic pressure, Δae or Δpe, shall be obtained subtracting both effects.

5.3.5 The point of application of gravitational pressure and seismic pressure in the internal face of the wall, shall be considered at H/3 and 2H/3 respectively, both measured from the base of the wall.

5.3.6 The seismic forces produced by the wall mass and the corresponding to the mass of earth supported behind the wall on its heel, shall be determined multiplying those weights by the corresponding seismic coefficient indicated in 5.3.3.

5.3.7 The seismic forces determined according to 5.3.6 shall be considered applied in the center of gravity of those masses.

5.3.8 When seismic load is considered, the permissible stresses of the soil shall be increased 33%.

5.3.9 For walls with less than 6.0 m high, it is acceptable to estimate the acting pressures with any semi-empiric method that produces similar results to the obtained with reliable theories.

5.3.10 A distributed uniform overload applied in the retained earth, can be considered as an equivalent earth weight.

5.3.11 The combination of the different forces and the Safety Factors SF are shown in Table 5.1.

5.3.12 The depth of foundation of gravity walls shall be the one that protects the erosion of the backfill of the base of the wall.

5.4 JOINTS.

Vertical joints shall be provided when:

1) Zones where abrupt changes of transversal vertical section of the wall occur.

2) At ah maximum horizontal distance of 12.00 m.

3) At foundation level changes and/or change of direction of alignment of the wall.

Table 5-1
Force combination and Safety Factor

Combination	Forces	Condition	SF
1	Fg	Overturning Sliding	1.5 1.5
2	Fg + Fs	Overturning Sliding	1.2 1.2

CHAPTER 6

STABILITY OF SLOPES.

6.1 SCOPE.

This chapter defines basic criteria for analysis and design of slopes and the protection measures that are required.

6.2 GENERAL CONSIDERATIONS.

6.2.1 Prior to the design of a slope, a geotechnical study has to be made in order to consider all probable failure surfaces and the location of phreatic level.

6.2.2 The size of the zone to be considered in the study, shall be defined by the geotechnical specialist according to the importance of the slope an the site conditions.

6.2.3 Measures to drain superficial water efficiently shall be taken. The aqueduct and/or sewer system in the zone shall be able to be deformed without cracking.

6.3 STATIC AND DYNAMIC EFFECT.

The analysis shall include static and dynamic applied forces.

6.3.1 The seismic force, Fs, shall be determined multiplying the weight of each sliding wedge by the corresponding seismic coefficient indicated in 5.3.3.

6.3.2 The Safety Factors are indicated in Table 6.2.

TABLE 6.2
Safety Factors.

Condition	Force Combination	SF
1	Fg	1.4
2	Fg + Ff + Fs	1.1

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CHAPTER 3.

UNDERGROUND INVESTIGATION

El Salvador is a country geographically located in a seismically intense region, which makes it essential that during the planning, designing and execution phases of any Engineering and Arquitecture project, geological aspects, soil mechanics, rock mechanics and earthquakes are considered.

Therefore, planning and execution of perforation and laboratory tests, shall agree with the magnitude of the project and the results of these tests shall be understood by the geotechnical engineer, who, based in the results will conclude and recommend all related to foundations, protection works, etc. therefore obtaining a safe construction from the foundation point of view.

The local experience regarding underground investigation, reveals that the standard penetration test (SPT) is an adequate instrument for the evaluation of the resistance underground, there, its importance, however, in the future, with the development of new equipment and exploration methods and underground sampling other methods shall be used that substitute or complement the SPT as long as the obtained results reveal reliable results.

In case of structures classified in group 1 according to chapter 2 of the Technical Standard for Seismic Design, potable water supply systems, sanitary sewer system and pluvial sewer system shall be included in the geotechnical study. This study shall also include relevant aspects of the site.

The relevant indicative properties of altered samples shall determined following the generally accepted procedures for this kind of tests.

The mechanical properties of soils (shear strength, compressibility and permeability) shall be determined with accepted laboratory or site procedures.

It is acceptable to estimate mechanical properties based on the results of indicative tests such as the standard penetration tests, vane shear test, or other site and laboratory tests, if the results have already been previously and reliably compared to the conventional tests for the soils in study.

CHAPTER 4.

FOUNDATION TYPES

There is no exact depth limit that separates shallow from deep foundations; however: any foundation shall be considered as shallow when its depth does not exceed two or three times its shorter dimension; in some cases the geotechnical study shall determine the possibility to use deep and shallow foundations in the same building; this depends on the material detected in the underground investigation.

This alternative shall be considered by the structural engineer when designing foundations.

As an example of a shallow foundation, the use of pad and strip foundations prevails in the country given the predominance of pyroclastic materials in soils.

However, in some regions of the country, foundation walls are usually used to support strip foundations. This type of foundation has demonstrated an adequate behavior, particularly in clays and sands (close to coastal zones).

Regarding deep foundations experience has demonstrated that it is recommendable to construct previously perforated point bearing piles.

Friction and combined piles have a limited use, however their use depend on the importance of the project and on the results of the geotechnical study.

Regarding retrofit foundations, their purpose is to transfer loads of an existing foundation to a deeper soil layer, with a greater bearing capacity. When more loads are added to an existing structure, the bearing capacity shall be revised, and depending upon the results it shall be strengthened or not. This operation shall be necessary to prevent settlement to continue, to increase the soil bearing capacity or to allow an adjacent foundation next to an existing without inducing any structural damage. Foundation strengthening can be temporary or permanent.

Below some reasons why it is necessary a foundation strengthening are shown:

a) Overload soil increasing the number of stories, changing their original use or purpose.

b) Location of an eccentric load in the foundation.

c) Presence of fast growing trees, close to clays.

d) Total or differential settlements and loss of underground bearing capacity due to the effects of an earthquake.

CHAPTER 5.

BEARING CAPACITY OF SOIL.

The majority of the theories developed to evaluate the bearing capacity of soil are based in simplified hypothesis of the behavior of soils and in mathematical developments given those hypotheses; in other theories, especially in recent studies observation and experience play a much more important role.

Basically, the theories are based on hypotheses that consider soil as a continuous, semi infinite, homogeneous and isotropic mean under plane deformation conditions.

In general, the theories often applied in our country are commented below:

Terzaghi’s theory is recommendable for all shallow foundation in any soil; it shall be applied up to a depth equivalent to two or three times the shortest foundation dimension with a lot of reliability.

Skempton’s theory is adequate for shallow or deep foundations in highly plastic clays ($\varphi = 0$), including the computation of bearing capacities in cylinders and piles.

Meyerhof’s theory shall be used to determine the bearing capacity in deep foundations in sands and gravels, including cylinders and piles; however, it is convenient to watch carefully its results because sometimes it gives high values with respect to results obtained applying other theories.

From all the above mentioned it is deduced that it is not possible to follow an itinerary indiscriminately to determine the bearing capacity of soils, especially with deep foundations, those for which existing theories do not give a total reliability and perhaps not even a good one, which justifies the use of high values of safety factors to find the permissible bearing capacity of soils, as mentioned in chapter 7 of this technical standard.

CHAPTER 6.

SETTLEMENTS.

Given the geological characteristics of our country where pyroclastic soils domain, settlements are little and the consolidation phenomenon of soils shows low values of the consolidation coefficient (Cc).

For the particular case of the AMSS where the urban development is, in general terms, the typical soils are pyroclastic material ("Tierra Blanca joven") and this soil shows no settlement problems, as long as it has normal humidity conditions, since having saturated soils during an earthquake may result in significant settlements that might cause cracking or collapse in structures.

If soils with plastic characteristics are detected it shall be required to make a settlement evaluation which shall guarantee that will not cause cracking or settlements in walls, frames and pipes.

It shall be mentioned that the permissible settlements of the structure shall be determined depending on the importance of the structure.

The geotechnical engineer shall establish the depth where the layer is detected, the induced pressure to that layer shall be evaluated and its corresponding influence in the behavior of the future structure.

If there are no unidirectional consolidation tests, the following expressions shall be used to determine the compressibility coefficient from the consolidation plot.

$Cc = 0.009(LI-10)$

Where:

LI = Liquid Limit.
Cc = Compressibility coefficient.

The use of this equation shall not substitute the results of the unidirectional consolidation test.

SETTLEMENT POSSIBILITY

Local experience has shown that pyroclastic material shows instant (elastic) settlements at the beginning of the construction and not in a long term (consolidation).

The possibility of settlement occurring (total or differential), can be estimated from the natural water content (Wn) and the Attenberg limits. If the natural water content (Wn) is close to the liquid limit (LL) it is estimated that the soil is normally consolidated (NC) this condition shows the possibility to occur significant settlements, and if the natural water content (Wn) is close to the plastic limit (Lp) the soil is classified as preconsolidated (Sp) this condition shows the possibility to occur little settlements.

EXPANSIONS AND CONTRACTIONS

In our country there are some zones such as the coastal, oriental, occidental and the northern zone where clays with high liquid limits (L.L.), greater than 50% and high plasticity index (I.P.) greater than 9 can be found; which evidence strong expansions and contractions that turn to cracking in strip foundations, walls or floors particularly in one or two story constructions this is due to the volumetric changes when humidity varies.

These expansions shall be estimated in function of the consistency limits, based on natural humidity (Wn), Contraction limit (Lc), Liquid limit (L.L) and specific gravity of clay using volumetric and gravimetric relationships
Expansions shall be estimated computing the volumetric changes based on the Wn (natural

water content) until reaching the Liquid Limit (L.L). The contractions shall be determined based on the natural water content (Wn) until reaching the Contraction Limit (L.C.) where there are no volumetric changes.

The contraction limit shall be estimated by:

$$Lc = \left(\frac{Vf + \gamma w}{Ws} \quad \frac{1}{Ss} \right) \times 100$$

Where:

Vf = Final dry Volume of the sample
Ws = Dry weight of the sample
γw = Volumetric weight of water
Ss = Specific gravity

One way to minimize expansions or contractions is to spread under floors granular material or stabilize clays with sands, salt, lime or any other chemical composite that reduces the consistency limits. Gardens next to constructions shall be avoided and preferably construct perimetral sidewalks around constructions, which reduces drying close to foundation strips or walls avoiding this phenomenon to happen close to them.

A good practice when clays are detected is to recommend a narrow river sand layer around foundations so that it works as a shock absorber and this way avoid wall cracking.

There is a way to qualify the expansion degree of clays.

LL<35	PI<6	Low expansion
35<LL<50	PI<9	Medium expansion
LL<50	PI<9	High expansion

CHAPTER 7

SAFETY FACTORS

All bearing capacities that chapter 5 of this technical standard refers to correspond to failure values, that is, values such that if these stresses were transmitted by the foundation to the soil, this would turn to a equilibrium limit state. It is important to clarify that these values are not the ones assigned to real foundations. Here begins the concept of permissible bearing capacity or work bearing capacity, with which foundations shall be designed. This will always be lower than that of failure and shall be enough far from it to give the necessary safety margins, in order to cover all uncertainties related to soil properties and its determination, to the magnitudes of the acting loads, to the bearing capacity specific theory used and to the problems and uncertainties of construction.

In practice, it is common to express the permissible bearing capacity as a fraction of the failure, obtained dividing it by a number greater than 1, which is called safety factor (SF). However, at least for cohesive soils, the previous criterion is not practical from a conceptual point of view and to the numeric value of the bearing capacity that is obtained.

To determine the value of the safety factor, the following considerations shall be taken in count:

1. Damage magnitude, if a failure occurs (life loss, property damage and lawsuit).

2. relative cost of increasing or decreasing safety factors (SF).

3. relative change in the probability of failure by the change of the safety factor (Fs).

4. site data credibility.

5. Change in the properties of soil by construction operations and later by external causes.

6. Accuracy in the methods usually used in design (analysis method).

It is common to use safety factors (SF) in the order shown in the following table.

COMMON FACTORS OF SAFETY FACTORS (SF)

FAILURE TYPE	FOUNDATION TYPE	SECURITY FACTOR
SHEAR	Excavations for dams and land fillings, etc.	1.2-1.6
SHEAR	Earth retaining structures	1.5-2.0
SHEAR	Pit props, propped excavations (temporary)	1.2-1.5
SHEAR	Pad foundations, foundation slabs.	1.7-2.5

In general terms, the ultimate bearing capacity of foundations in purely cohesive soils is given by an expression of the following type:

$$qc = cNc + \gamma Df$$

The maximum safety condition is evidently:

$$qc = \gamma Df$$

in this expression, all the soil resistance is reserved. In the case of applying a safety factor, this shall act only upon the part of qc that exceeds γDf, that is cNc. Therefore the following expression for the permissible bearing capacity results:

$$qad = \frac{cNc}{Fs} + \gamma Df$$

In the case of pure frictional soils, the bearing capacity is much greater than the acting pressure at the foundation level, therefore when dividing the failure bearing capacity by a safety factor drives to an error, which is similar to the one made with cohesive soils, on the other hand, it is numerically, very little. This is why, the permissible bearing capacity of a frictional soil is usually expressed as:

$$qad = \frac{qc}{SF}$$

The values of SF to use in a given case shall vary according to the importance of the project and the order of uncertainties managed; formally, it shall be different for every single case and a product of a particular study of that case. However, there are typical values usually accepted. In the case of shallow foundations, if the analysis of the acting loads takes in count only permanent loads, it is recommendable to use a minimum SF of 3. If permanent and eventual live loads are taken in count, the previous value can be reduced to 2 or 2.5. If a detailed load analysis is performed, which includes seism effects in regions where that applies, the safety factor can be reduced to values as low as 1.5.

In the case of deep foundations, uncertainties that are managed are usually greater, by the contribution of the heterogeneity of soil and the construction methods. It is common to use a SF of at least 3.

CHAPTER 8
DETERMINATION OF THE DYNAMIC PROPERTIES OF SOIL.

Regarding shear wave velocities more investigation shall be carried out in different types of soils to be able to correlate reliably with the values obtained from commonly used tests in the country (e.g. SPT).

In general terms it is convenient to carry out the corresponding studies to determine values of shear wave velocities in a reliable way.

CHAPTER 9

STABILITY OF SLOPES

Geologically speaking, El Salvador has predominantly unconsolidated soils with a volcanic origin. In the San Salvador Area, around Ilopango lake and around Coatepeque lake, there are soils locally known as “white earth”, with characteristics such as: high angle of internal friction ($\varphi > 30$), apparent cohesion susceptible to humidity changes, low value of density, given the pumice presence in its matrix; high permeability and low compressibility in a natural state.

In some zones of the country there are low, medium and high plasticity clays and also different types of igneous and sedimentary rocks with a mechanical behavior in slopes different to volcanic ashes.

Volcanic ashes shape natural slopes that are mainly observed in the metropolitan San Salvador area and around it, and are characterized because the slopes formed by them have almost vertical angles with heights greater than 50 m but they are easily susceptible to erosion and the seismicity of the zone makes them prone to failure.

After the earthquake occurred in October 10, 1986, it could be observed the significant number of collapsed slopes in the metropolitan San Salvador area and around it, showing different failure mechanisms, evidencing that failures in volcanic ashes or white earth slopes are different from the cylindrical or rotation type defined in the Bishop or Fellenius methods. This takes to limitations in the applicability of the classical theories of failure mechanisms according to what was observed in the behavior of soils in El Salvador, thus, this shall be considered in the analysis when designing a structure.

Slides in volcanic ashes are generally revealed as a superficial failure with a short length with respect to the top and the depth of the slid surface is generally the one corresponding to the thickness of the saturated superficial layers.

The classical theories to determine the stability of slopes, shall be used conservatively as long as any other wide and deep investigation regarding volcanic ashes is not developed, this investigation shall include the seismic aspect, the observation of the behavior of the existing slopes, determination of the index and mechanical properties, as well as water flow.

A reasonably useful method, could be the pseudostatic method, which considers the gravitatory and lateral forces equivalent to the seismic forces.

C9.1 SLIDING MECHANISMS

The most usual types of slides are the following:

Rotational slides, which occur when the mass of soil slides along failure surfaces considered for effects of analysis, with a cylindrical shape.

Translational slides, which occur when the mass of soil moves through a plane failure surface, approximately horizontal in most cases. Slides by mud flow occur when the mass of soil with certain humidity content, turns to a viscous fluid. Slides by undercut of a slope occur when instability at the bottom of the slope is caused by the haulage of materials at the bottom of it, causing landslides, which are increased by seismic activity.

Volcanic ashes or white earth is an easy to disintegrate material during rainfalls and is able to be transported easily.

Historically, because of this type of problems many cities have changed location. In Central America examples such as the old city of Guatemala, destroyed the 10 of September, 1541 by mud flows that descended upon the Volcan de Agua when the lagoon that existed in the crater suddenly emptied. In Honduras the city of Ocotepeque was buried by a flow of mud in the beginning of the century. In El Salvador, the 19 of September, 1982, the west Montebello neighborhood located northwest in San Salvador, was affected by a flow of mud with large dimensions, destroying the neighborhood and causing hundreds of deaths and victims.

In the case of slides caused by undercut of a slope, it is necessary to construct protection to the slope and river flow canalization to be able to solve this type of failure. The foundation depth of these constructions, shall be given by the general and local undercut depth, which shall be determined taking in count the hydraulic parameters of the river bed such as: maximum water level for a return period according to the importance of the slope, velocity, inclination, ruggedness, etc. as well as the physical and mechanical characteristics of the layers conforming the slope.

When cities grow in lands where topography is irregular, the dangers caused by land and rock slides limit urban development.

Landslides have occurred in our country during strong rains and earthquakes; generally, the rock ones, are less frequent and occur in road cuts.

C9.2 PROTECTION ZONES

C9.2.1 CRITERION

The protection zone is the land stripe that is established adjacent to rivers and gorges, to protect urban parcels from soil instability caused by progressive erosion originated by the superficial water, allowing and assuring the normal course of water flow. This stripe can also be established to lands which, given their topographical configuration, present great level differences inside or with the adjacent lands.

It is important to notice that the protection zones shall be established and delimited by the responsible professionals of constructions, based in geotechnical studies and the following criteria:

A) Depth of the gorge.
B) Hydrological and hydraulic study of the river bed.
C) Natural inside and adjacent land level difference.

C9.2.2 PROPERTY, USE AND MAINTENANCE OF PROTECTION ZONES

Lands affected by protection zones shall be private or public property.

In case of being private property, this shall be incorporated to the neighboring lands and this shall be described and identified in the corresponding title deed, indicating the limitations and obligations detailed in the regulation.

The protection zones in public property shall be part of the ecological vegetation areas of the parcel. Public or private protection zones shall not be used to construct residence, commercial or industrial buildings.

In order to determine the protection zone in places close to rivers or gorges, the width of the protection zone shall be measured from the top of the slope along the land in the affected zone. The height of the slope shall be measured vertically, from the bottom to the superior border or top of it.

The width of the protection zone shall be reduced by constructing walls or combination of walls and slopes; but this measure shall be justified by a study made by a geotechnical specialist.

CHAPTER 10

PROTECTION WORKS

C10.1 CLASSIFICATION

Protection works are classified according to two criterions: the ones regarding their mechanical behavior and the applicability fields of the theories of lateral earth pressures, and the ones that are classified according to their duration.

According to the first criterion, structures can be rigid or flexible according to its deformability before pressures transmitted by the backfill. Concrete and masonry structures are rigid. Flexible retaining structures are those that by their constitutive materials and section have a high deformability; the most representative ones are gabions and sheet piles.

The ones classified according to their duration, are referred to the term of life indicated and can be permanent or provisional. The permanent retaining structure is the concrete or masonry wall or any other durable material. Provisional works are called pit props and are usually constructed of timber or steel.

C10.2 RETAINING WALLS

C10.2.1 TYPES OF WALLS

A) GRAVITY WALLS: In this type of walls the tension stress capacity is minimum, and its stability depends on the weight of masonry or concrete and the backfill. Only concrete walls where a minimum nominal reinforcement is placed close to the parameters to avoid cracking by temperature and concrete contraction shall be reinforced.

B) SEMIGRAVITY WALLS: These are slenderer and in order to be stable they require structural and temperature reinforcement additional to the weight of masonry or concrete.

C) BUTTRESSED WALLS: These walls are formed by a thin slab usually vertical, supported at intervals, by buttresses placed perpendicularly to the slab. The slab and the buttresses are connected to a foundation slab.

C10.2.2 WALLS CONSTRUCTIVE PROCESS

Any significant change that shall be done to design, specifications and construction procedure shall be analyzed based on the information contained in the geotechnical study and shall be considered by the designer before executing those changes.

The design memory shall include a justification of the foundation type planned and the specified construction procedures. An explicit description of the methods of analysis used and the foreseen behavior for each of the limit states indicated in numeral 11.3 of this standard shall also be considered. The results of underground investigation, laboratory tests and other analysis and determinations shall be attached, magnitudes of actions considered in the design considered interaction with the foundations of adjacent properties foundations and the distance between these foundations shall also be considered.

C10.5 PARTITION JOINTS.

The values of partitions indicated in table 10-3, are giver for granular soils classified from medium dense to dense. For cohesive soils, it shall be for semicompact to compact consistency.

In order to classify a material as granular according to determined in table 11-3 of this technical standard, this shall not have a fines proportion greater than 5% and its plasticity index shall be lower than 3 (NP).

For intermediate soils the plasticity index shall not be greater than 15 ($I_p < 15$). When $I_p > 15$ it shall be considered as cohesive.

Organic soils shall not be used for filling nor used as foundation soil; mainly if the soils are classified as OL and OH, which shall be restored if detected.

Soils classified as ML or CL with a plasticity index lower or equal to 15 ($I_p < 15$), shall be considered as intermediate.

When the geotechnical study indicates the presence of cohesive soils in the foundation zone, this soil shall be treated according to the recommendations given by the corresponding study.

CHAPTER 11

LIQUEFACTION

The liquefaction phenomenon is the temporary loss of shear stress resistance. This loss drives to collapse of any structure built on a liquefacted material, and it is attributed to the pore pressure increment produced by the seismic movement, explotions or any other vibrating phenomenon of soil.

Nullification of the effective pressure induced by an earthquake or any other phenomenon as the ones mentioned above, makes the soil to lose its resistance to shear strength.

Liquefaction failure in a phenomenon observed in saturated sands and with no drainage, when they try to compact when loaded by a dynamic force. Because sands are not able to reduce their volume given the lack of drainage, The pore pressure increases and eventually equals the confinement pressure, eliminating the effective pressure in such a way that large deformations are produced in the surface.

When the null effective pressure is reached, the deformation process starts and depending on the density of the soil, dilation can be produced and therefore pore pressure can decrease. In soils with low densities, dilation can not be produced, which makes this type of soils flow; on the other hand, when the soil has certain density, it will tend to expand in the deformation process, and this way, the pore pressure that canceled the effective pressure decreases.

Almost instant liquefaction has occurred in silts and saturated fine soft sands

For the characteristics mentioned above, this phenomenon is most likely to occur in coastal zones close to the sea or around rivers and lagoons.

CHAPTER 12

EROSION, UNDERCUT AND PIPING

C12.1 EROSION

It can be said that erosion is the mechanical process that transforms the shape of earth surface through exogenous agents acting on the surface, the most important one is water; this process contains three important stages:

- Particle detachment (rock, soil or combined).
- Transportation
- Deposition.

Pluvial erosion initiates with atmospheric precipitation when water drops hit directly on the surface of soils and rocks removing particles that are later carried by surface water.

If the amount of rainfall is of considerable volume, the action produced by the superficial water can drag the soil layer that works as a support to the vegetal cover, leaving the main rock exposed and endangering adjacent buildings.

C12.1.1 EROSIVE ACTION.

The erosion process shall be of great or low intensity, according to the water content that flows on the surface and its dragging capability, and the type of material that finds on its way.

A factor that increases erosion of slopes next to rivers and gorges in the country is the inadequate discharge of storm water and sewage disposal poured directly in them with no protection work or energy dissipaters turning them vulnerable to landslides.

C12.2 UNDERCUT

Undercut is a very important problem that shall be considered in the analysis and design of slopes, because when this effect is given, there is haulage of material in the base of slopes and under foundation of existing structures near that site.

Particle detachment during undercut, provokes instability causing settlements and collapse of the layers of soil and adjacent structures.

Depending on the place where water flows, undercut can occur more frequently in the following places:

C12.2.1 NARROWINGS

This is produced by the increment of the hauling capacity of solids, when the flow increases its velocity caused by the reduction of the hydraulic area in the river bed. This effect shall be considered in the design of bridges, which are common to be constructed in narrowings.

C12.2.2 CURVES

Occur when flow lines located outside the flow curvature, reach such velocity that increases the capacity of hauling those particles.

C12.2.3 PILES

Construction of piles require more attention since obstruction produced by bridge piles, modifies locally the morphology of the river bed and increases undercut, producing haulage of the constituting soil of the river bed, and foundation imbalance can produce collapse of the structure.

There, the importance of determining the minimum foundation depth of a pile or wall, taking in count the parameters of the maximum water level of rivers.

12.3 PIPING

If the soil of a foundation is not uniform, water can drag fine material, leaving only coarse material, with a tendency to create an inverted filter that avoids piping formation. Since it is difficult to determine when this phenomenon shall occur, in any specific case, it is recommendable to plan a structure in such a way that this phenomenon can not be produced.

Magnitude of filtration forces through foundations where piping can occur, depend on the variation of the loss of load of the water that filters. Experience has demonstrated that the size of grains and gradation of the soil that will support foundations have an important influence in piping failures. Therefore, it seems that many of the failures produced by piping are of the underground erosion type, as a result of filtrations that small geologically weak zones follow.

CHAPTER 13

PIT EXCAVATION AND PIT PROPS

The majority of structures constructed are founded under the surface of ground, and the depth of excavations depends on the foundation type chosen according to the previously treated topics.

There are various factors that shall be taken in count to carry out an excavation; some of the most important aspects are described below:

C13.1 EXCAVATIONS WITH SLOPES

The most important condition to carry out this type of excavation is that there shall be enough space around the place in order to develop the slopes with the inclination determined in the analysis; this inclination depends on the type and properties of soil or rock, the depth of excavation and the time that the excavation shall remain open.

The most frequent failure types that in slopes of an excavation, are the following:

- Rotational failures.
- Sliding or translational failures.

In the first case the movement of the slope occurs along a curved failure surface; this surface forms a trace with the paper plane that can be supposed, easily and with a low error, as a circumference.

In the second case, the failure occurs along weak surfaces in the body of the slope or in its foundation, which are usually horizontal or slightly with respect to a horizontal plane.

C13.2 PROPPED EXCAVATIONS

A project with many buildings, mainly in crowded urban areas, is made in such a way that it includes the available property totally or adjacent to existing structures which means that when performing excavation, this shall be carried out vertically, requiring pit props. Generally these props are made of timber, steel, a combination of wood and steel or reinforced concrete (concrete struts and cast in place walls).

The procedure that is followed in the case of timber, concrete (precast pieces) and steel struts is, in general, the following: first, the strut shall be driven following the contour of the excavation and until a depth greater than the bottom of excavation and as it continues, steel or timber props are put traversely against the strut, these struts shall be supported on longitudinal stringers called “madrinas” (godmothers).

The procedure stops until excavation reaches the foundation level.

The procedure of “cast in place wall” consists in, first, cast perimetral foundation walls inside pits using a ladle provided with a guide bar, stabilizing the pit with bentonite mud and casting concrete inside the pit with a cast pipe after accommodating reinforcing steel. High slump concrete evicts bentonite mud and the foundation walls of the constructing structure are formed. The length of the stringers is generally from 5 to 6 m and the depth shall be the one that leaves approximately 1.50 or 2.50 m under the excavation depth. Once walls have set, the earth prism between them shall be excavated, propping walls as the excavation goes on.

When the width of the excavation is too large to use props between walls, the procedure usually followed is to excavate the center of the area until the bottom and cast the corresponding foundation part so that the constructed part acts like a supporting element for props.

C13.2.1 LOADS IN PROPS

Generally the most important information that the planner engineer shall be concerned about is the load of props; it shall be necessary to know the magnitude and distribution of the earth pressure on the strut.

The magnitude and distribution of lateral pressures depend not only on the soil properties but on the restrictions that the supporting element applies to earth deformation and the flexibility of the retaining structure in general.

As the excavation goes on, the rigidity given by the already placed props, stops earth displacement in zones close to the prop supports. On the other hand, under the effect of lateral pressure, the pit prop rotates to the interior of the excavation in the bottom zones, therefore, locating props in those regions assumes a larger displacement of soil that will be as greater as the depth of excavation. This type of deformation is equivalent, from the pressure distribution point of view, to a rotation around the superior end of the supporting element. In these deformation conditions, the classical theories of lateral earth pressures are not applicable and therefore, in order to calculate the lateral earth pressure of this type of structure, it is necessary to measure scaled models or real structures.

Terzaghi and Peck, based on site measurements, proposed a simple envelope for designing, with a trapezoidal shape, useful to be applied in any pit propped excavation.

To calculate the load that the props shall resist, a simplified procedure has been developed, which ignores the effects of continuity of the strut turning the problem in a statically determined structure. The loads of the props are obtained calculating the reactions of many independent beams.

So that the strut resists the load imposed by the earth pressure, the maximum allowed distance between supporting stringers shall be calculated.

In any case the greater dimension of the stringer section, shall be placed parallel to the direction of the load.

Props to support struts shall be supported in wedges and shall be calculated as columns.

C13.2.2 FAILURES AT THE BOTTOM OF EXCAVATIONS

One of the important aspects that shall be considered in the stability of pit propped excavations in clays, is the possible failure at the bottom of it.

It has been observed in many deep excavations made in soft clays with no appropriate cautions, that when a given depth is exceeded, the bottom becomes unstable, deformations increase considerably and clay begins flowing to the excavation, tending to close it. This causes the bottom of the excavation to lift and brings deformations in the excavation zone and considerable settlements in the adjacent zones in a short period of time.

Upcoming consequences could be disastrous if there is any structure constructed at a distance lower or equal to the width of the excavation.

In general, all criteria about the analysis of the failure of the bottom of excavations, consider the problem as a problem equivalent to the bearing capacity, where the material at the top of the excavation shall have enough shear resistance to resist stresses produced at the bottom by the unbalanced vertical pressure at the excavation level produced by the weight of the earth blocks that limit it at both sides.

Just as the bearing capacity problem, the minimum values of the S.F., correspond to an infinitely long excavation with respect to its width and the greater to a square excavation.

C13.2.3 EXPANTIONS

Earth removal during an excavation produces unloading of the soil layers that are under it; such unloading, produces an expansion of the affected layers if the excavation is carried out in clays, the magnitude of the expansion depends on the dimensions of the excavated areas, the depth, the expansibility coefficient of soil and the time that the excavation remains open.

The expansion phenomenon during excavations shows two stages: the first, a relatively fast expansion that can be verified at the same velocity of the excavation advance which seems to be an elastic phenomenon type and, the second, slower, that goes with an increment of the water content of the clay and is a process that extends with time, that is why, it is important to keep the excavation open the minimum time possible.

Some measures that have proved their practical value to decrease expansions, movements that later turn to structure settlements are mentioned below:

A) Excavation by Stages

The realization of an excavation by stages decreases importantly the magnitude of expansions because, as seen previously, the dimensions of the excavated area influence a lot in the magnitude of the expansion.

B) Decreasing of the phreatic level

Another factor that contributes importantly to control expansions during excavations when this is carried out under phreatic level is decreasing it, because water pumping induces an overload underground, when it changes from an immersed state to a saturated state. This overload balances the unloading suffered by the excavation given by the removal of soil.

When a foundation construction requires an excavation under phreatic level, it is necessary to decrease that level under the foundation depth.

The phreatic level can be lowered using methods that depend on the size and the depth of the excavation, geological conditions and soil characteristics. To be able to accomplish an effective lowering it is fundamental that the system is well designed, installed and operated.

The phreatic level lowering is necessary by the following reasons:

A) Intercepts water flow that comes to slopes and excavation bottom maintaining the excavation dry.

B) In the case of excavations with slopes, it increases the stability of them.

C) In the case of propped excavations, it helps the bottom failure safety factor by the reasons exposed in the corresponding chapter.

D) In the case of excavations in clays with high compressibility under loading and high expansiveness when unloaded, the lowering of the phreatic level helps control expansions that are produced during excavation according to the previous chapter. When expansions decrease at their minimum possible value, it is guaranteed that the shear resistance of the soil under excavation shall not decrease too much, keeping SF against the excavation stability.

C) Lowering of the time that the excavation remains open.

It is important to emphasize, that the time that the excavation remains open is an important factor that influences the magnitude of expansions. Therefore it is important to cast foundation decks immediately after reaching the foundation depth in the minimum time possible. This shall decrease importantly the magnitude of total expansions

In small excavations and in other soil types (dense or cemented) it is sometimes possible to let water flow in slopes to collect in pits that discharge in wells where water can be extracted through pumps.

Sometimes, it is necessary to place filters in pits and in wells so that material haulage can be prevented, mainly when soil contains fine sand lenses or sandy limes.

The phreatic level in granular materials can be lowered through well points at depths of approximately 5 m. a well point is a perforated pipe of approximately 1 m long and 1 ½" diameter covered by a cylindrical mesh so that fine particles can not enter. In the bottom end of the pipe there is a head that lets the placing of the well through pressured water, with no need to realize driven actions.

In order to lower water level in wells they are placed in a line spaced 1.0 or 2.0 m between them and connected by a main pipe on the surface of land, which is connected to the suction pump.

People who are not directly related to this work, shall not be let inside the places where excavations are made.

CHAPTER 14

QUALITY CONTROL OF EARTH WORKS

A commonly practiced test in the country, given its facilities is the proctor test (modified and standard). An easy way to measure the quality of landfill is through site density tests which are correlated with the corresponding density-humidity test, these values are compared to the specified in the corresponding reference terms.

As a local practice, it is considered that the compaction degrees shall never be less than 90% of the index proctor (modified or standard), which depends on the importance of the project.

Using the sand cone and the densimeter according to ASTM D and ASTM D is a common practice, these have demonstrated satisfactory results with the granular soils in the country.

Nowadays, nuclear equipment is being used in a limited way to determine the degree of compaction and its most commonly used in construction works of highways.

The thicknesses of the layers to compact depend on the equipment; however, these shall not be greater than 0.15 m so that the layer can be compacted adequately.

Appendix D - Technical Standards for Earthquake Design

CONTENTS

CHAPTER 1

GENERAL

1.1 SCOPE

This technical standard sets forth the minimum requirements for earthquake design of structures and is part of the “Regulation for Structural Safety of Constructions”.

1.2 GENERAL REQUIREMENTS

1.2.1 Every structure and each part of it shall be designed and constructed to resist the effects of seismic ground motions as provided in this technical standard.

1.2.2 When the wind design produces greater effects, the wind design shall govern, but detailing requirements and limitations prescribed in this technical standard shall be followed.

1.2.3 The report and structural drawings shall contain the adopted criteria for earthquake design and shall include the following information:

1) Identification and location of the construction.

2) Seismic zone where it is located. (Fig. 1).

3) Site Parameters. (Table 2).

4) Occupancy Category. (Table 3).

5) Description and identification of the lateral force resisting system.

6) Seismic Coefficient(s) used for design.

1.2.4 When computational programs are used, the report shall include the following extra information:

1) Scheme of the complete mathematical model used to represent the structure in the analysis.

2) Description of the program containing the necessary information in order to determine the nature and extension of the analysis.

3) Entries and results clearly differenced.

CHAPTER 2

NOTATION

2.1 NOTATION

A = Seismic zone factor as set forth in table 1.

Ac = the combined effective area, in square meters, of the shear walls in the first story of the structure, as determined in expression (4.4).

Ae = the horizontal cross sectional effective area, in square meters, of each shear wall in the first story of the structure used in expression (4.4).

Ax = the torsional amplification factor at level x, as determined in expression (4.9).

Bx = the design shear force amplification factor at level x, as determined in equation (4.10).

Co = site coefficient as set forth in table 2.

Cd = Displacements amplification factor as set forth in table 7.

Cs = seismic coefficient given in expression (4.1).

Csm= Modal seismic coefficient.

Cp = numerical coefficient given in chapter 6 and set forth in table 9.

Ct = numerical coefficient given in expression (4.3).

De = the length, in meters, of each element of a shear wall in the first story in the direction parallel to the applied forces, used in expression (4.4).

δi = horizontal displacement at level I relative to the base due to applied lateral forces, fi.

δx = Total horizontal displacement of the center of mass at level x, computed by equation (4.11).

δmax = maximum horizontal displacement of level x, including accidental torsion, at the ends of the structure.

δprom = the average of horizontal displacements of level x, including accidental torsion at the ends of the structure.

δxe = horizontal displacement of the center of mass at level x, determined by an elastic analysis.

Θ= stability coefficient.

fi = Lateral force at level i to be used in formula (4-5).

Fi, Fn, Fx= Lateral forces applied at level I, n or x respectively, used in expressions (4.6), (4.8) and (7.1).

Fp = lateral forces in a part of the structure, as determined in expression (6.1).

Ft = that portion of the base shear, V, considered concentrated at the top of the structure in addition to Fn, to be used in expressions (4.6), (4.8) and (7.1).

g = Acceleration due to gravity, to be used in formula (4-5).

hsx = height of the story under level x.

hi, hn, hx = height, in meters above the base to level i, n or x, respectively, to be used in expression (4.8).

I = Importance factor set forth in table 4.

Level i = level of the structure referred to by the subscript i, “i=1” designates the first level above the base.

Level n = that level that is uppermost in the main portion of the structure.

Level x = that level that is under design consideration, x=1 designates the first level above the base.

$Px = \sum_{i=1}^{n} Wi$, the total gravitational load acting on level x.

Qa = Accidental actions.
Qp = Permanent actions.
Qv = Variable actions.

R = response modification factor set forth in table 7.

Co and To = Site Coefficient due to soil characteristics, set forth in table 2.

T = fundamental period of vibration, in seconds, of the structure in the direction under consideration, determined according to 4.2.2.

Tm = modal period of vibration.

V = base shear force, determined according to (4.1).

Vx = the story shear in story x.

W = the total seismic load defined in 4.1.3.

wi, wx = that portion of W located at or assigned to level i or x, respectively, used in expressions (4.5) and (4.8).

Wp = the weight of an element or component, to be used in expression (6.1).

Δ = story drift, defined in section 4.6.1 and that occurs simultaneously with Vx in equation (4.12).

Δmax = Maximum drift at story x, including accidental torsion, at one of the ends of the structure, in the direction of analysis.

Δprom = average drift at level x, of both ends of the structure, in the direction of the analysis.

CHAPTER 3

DESIGN CRITERIA

3.1 BASIS FOR DESIGN.

The seismic design of structures shall be determined considering seismic zoning, site characteristics, occupancy, configuration structural system and height, according to this chapter. The minimum design seismic forces shall be determined according to chapters 4 and 5 of this technical standard.

3.2 SEISMIC ZONES.

According to the location of the structure set forth in figure 1, the zone factor A shall be used given in table 1.

3.3 LOCAL GEOLOGY AND SOIL CHARACTERISTICS.

For each soil profile, the site coefficients Co and To, shall be established according to table 2.

3.4 OCCUPANCY CATEGORIES.

Each structure shall be classified in one of the occupancy categories of table 3. Table 4 establishes the corresponding importance factors I.

3.5 STRUCTURAL CONFIGURATION.

Each structure shall be designated as being structurally regular or irregular in accordance with the following:

3.5.1 Regular Structures. Are those that have no significant physical discontinuities in plan, vertical configurations, or in their lateral force resisting systems.

3.5.2 Irregular structures. Are those that have significant physical discontinuities in configuration or in their lateral force resisting systems. Irregular features include but are not limited to, those described in Tables 5 and 6.

Structures having any of the features described in tables 5 or 6 shall be designed considered irregular, excepting when the story drift ratio of one story is less than 1.3 times the story drift ratio of the story above, the structure may be deemed to not have the structural irregularities of type A or B set forth in table 5. The story drifts may be calculated neglecting torsional effects and the story drift ratios of the two top levels do not need to be considered for this purpose.

3.6 STRUCTURAL SYSTEMS.

3.6.1 The structural systems are defined in this section. Their corresponding response modifying factors, R, and displacement amplification, Cd, such as height limits, H, are set forth in table 7.

1) System A. Structure formed by non bracing frames, which mainly resist, by flexural action of its members, the total gravitational and lateral loads, excepting to the indicated in 3.6.2(1).

2) System B. Structure formed by non bracing frames that support essentially gravitational loads and by bearing walls or braced frames that totally resist lateral loads. See 3.6.2(1).

3) System C. Structure formed by non braced frames and by bearing walls or braced frames. All the components of the structure resist totally vertical and horizontal loads, excepting the

indicated in 3.6.2(1). The components shall be designed to resist lateral forces, proportional to their relative rigidities in accordance with an interaction analysis. Anyways, unbraced frames shall be designed to resist at least 25 % of the lateral loads calculated for the structure.

4) System D. Structures in which the resistance to gravitational loads is proportioned essentially by walls or braced frames that also resist totally lateral loads. See 3.6.2(1).

5) System E. Structure with isolated (or shall be considered as that) lateral load resisting elements in the direction of the analysis. See 3.6.2(2)

6) Other systems. In these cases it shall be demonstrated through technical data and tests establishing the dynamic characteristics, that their resistance to lateral forces and capacity of energy absorption are equivalent to one of the systems defined here.

3.6.2 The following additional requirements are applicable to structural systems:

1) All frame elements not required by the design to be part of the lateral force resistance system, shall be able to resist gravitational loads when they move Cd times the elastic displacement, δxe, calculated for the structure. In these elements, the P-Delta effect shall be taken in count. When the design of these elements is based on the allowable stress design procedure, their resistance shall be determined based on a stress 1.7 times greater than the allowable.

2) In inverted pendulum structures (see system E in table 7), the effect of rotational inertia may be considered satisfied if the flexural moment in the mass supporting element is varied 0.50 m from the superior end to 1.5 M in the inferior end, being M the product of the shear force in the mass multiplied by the height of the supporting element.

3) Moment resisting frames may contain or be adjacent to elements that are more rigid that tend to brace the frame, if it is shown that the action or failure of the elements that are more rigid may not affect the capacity of the frame to resist vertical and lateral loads.

3.7 WEAK STORY STRUCTURES.

Structures with a discontinuity of their resistant capacity (vertical irregularity type E defined in table 5) shall not be allowed, where the weak story has a calculated resistance lower than 70% of the superior story, unless the structure has less than 2 stories or is less than 10 meters high and also the weak structure is able to resist a total lateral seismic force of 3R/8 times the design force prescribed in chapter 4.

3.8 COMBINATIONS OF STRUCTURAL SYSTEMS.

When combinations of structural systems are incorporated in the same structure, the following requirements shall be satisfied:

3.8.1 Vertical Combinations.

The value of R used in the design of any story shall be less or equal to the value of R used for the superior story. This requirement is not necessary in the story where the seismic load above it is lower than 10% of the total seismic load of the structure.

3.8.2 Combinations in different directions.

1) When the structure has a wall bearing system only in one direction, the value of R to be used in the orthogonal direction shall not be greater than that used for the wall bearing system.

2) Any combination of systems A, B or C, may

be used to resist the design seismic forces in structures lower than 50 m high. When structures exceed 50 m, only combinations of systems A and C with a special detailing shall be used.

3.9 SELECTION OF LATERAL FORCE ANALISYS PROCEDURE.

Any structure may be designed using the procedures of chapter 5.

The static lateral force procedure set forth in chapter 4 may be used in the following cases:

1) Regular structures under 70 m in height, except those that are located in a soil type S4 and have a fundamental period greater than 0.7 seconds.

2) Irregular Structures not more than 5 stories or 20 m in height. Structures with irregularities type A, B or C defined in table 5, or any other irregularity not described in Tables 5 or 6, shall, furthermore, conform with 3.8.1 so that this procedure is applicable.

CHAPTER 4

DESIGN STATIC LATERAL FORCES AND RELATED EFFECTS.

4.1 GENERAL.

4.1.1 Seismic forces act in any horizontal direction.

4.1.2 It may be supposed that the seismic design forces do not act simultaneously in the direction of each main axis of the structure, except as permitted in 6.1.4.

4.1.3 the seismic load W, is the dead load and the instant live load, defined in title II of the regulation.

4.2 DESIGN BASE SHEAR AND SEISMIC COEFFICIENT.

4.2.1 The design base shear in a given direction shall be determined from the following formula:

$$V = C_sW \qquad (4.1)$$

The value of the seismic coefficient Cs shall be determined by equation (4.2), where T shall not be less than To or greater than 6To.

$$C_s = \frac{AIC_o}{R}\left(\frac{T_o}{T}\right)^{2/3} \qquad (4.2)$$

4.2.2 Period of the Structure.

The value of T shall be determined from one of the following methods:

1) Method A. For all buildings, the value of T may be approximated from the following formula:

$$T = C_t\, h_n^{\;3/4} \qquad (4.3)$$

Where Ct is equal to 0.085 for systems A with steel frames; 0.073 for systems A with reinforced concrete frames and 0.049 for the rest of the systems.

Alternatively, for systems with concrete or masonry shear walls, the value of Ct may be taken as $0.074/\sqrt{A_c}$. The value of Ac is determined by the following expression:

$$A_c = \sum A_e\left[0.2 + (D_e / h_n)^2\right] \qquad (4.4)$$

The value of De/hn shall not exceed 0.9.

2) Method B. the fundamental period of the building may be calculated using the structural properties and the deformation characteristics of the resisting elements using an appropriate analysis. This requirement may be satisfied by using the following expression:

$$T = 2\pi\sqrt{\frac{\sum_{i=1}^{n} W_i\,\delta_i^2}{g\sum_{i=1}^{n} F_i\,\delta_i}} \qquad (4.5)$$

The values of Fi represent any approximate distribution of the lateral forces according to expressions (4.6), (4.7) and (4.8) or any other rational distribution.

The value of Cs determined shall not be taken less than 80% of the obtained value using the value of T given by expression (4.3).

4.3 VERTICAL DISTRIBUTION OF LATERAL FORCE.

With a lack of a more rigorous procedure, the total force at each level of the structure shall be calculated according to the expressions (4.6), (4.7) and (4.8).

$$V = F_t + \sum_{i=1}^{n} F_i \qquad (4.6)$$

The concentrated force, Ft, at the top story, which is additional to Fn, shall be determined by the following expression:

$$F_t = 0.07TV \qquad (4.7)$$

The value of T used to calculate Ft shall be the period that corresponds with the design base shear as computed using expression (4.1). Ft need not exceed 0.25V and may be considered as zero where T is 0.7 seconds or less. The remaining portion of the base shear shall be distributed over the height of the structure, including level n, according to the following expression:

$$F_x = \frac{(V - F_t) W_x h_x}{\sum_{i=1}^{n} W_i h_i} \qquad (4.8)$$

At each level designated as x, the force Fx shall be applied over the area of the building in accordance with the mass distribution at that level. Design seismic forces shall be calculated as the effect of forces Fx and Ft applied at the appropriate levels above the base.

4.4 HORIZONTAL DISTRIBUTION OF SHEAR.

4.4.1 the design shear, Vx, in any story, is the sum of the forces Fx and Ft above that story. Vx shall be distributed to the various elements of the vertical system resisting lateral forces in proportion to their stiffness, considering the rigidity of the diaphragm. See 3.6.2(1) and 7.2.6 for elements that are not intended to be part of the lateral force resisting systems.

4.4.2 When diaphragms are not flexible, the necessary considerations of the effects of torsion to increase shear forces shall be taken. Diaphragms shall be considered flexible when the maximum lateral displacement of the diaphragm is more than two times the average story drift.

4.4.3 The torsional design moment at a given story shall be the moment resulting from eccentricities between applied design lateral forces at levels above that story and the vertical resisting elements in that story plus an accidental torsion.

4.4.4 the accidental torsional moment shall be determined assuming that at each level the mass is displaced at both sides of the calculated mass center, a distance equal to 5% of the dimension of the building in that story in the direction perpendicular to the forces under consideration.

4.4.5 Where torsional irregularity exists, as described in table 6, the effects shall be accounted for by increasing:

a) The accidental torsion at each level by an amplification factor, Ax determined from the following expression:

$$A_x = \left(\frac{\delta_m x}{1.2\, \delta_{prom}} \right)^2 \leq 3.0 \qquad (4.9)$$

b) The design shear force in the direction of analysis by an amplification factor Bx determined from the following expression:

$$B_x = 3.0\,\Delta_m \frac{x}{\Delta_{prom}} - 2.6 \leq 1.4 \qquad (4.10)$$

4.4.6 For designing each element the most severe load condition shall be considered.

4.5 OVERTURNING

4.5.1 Every structure shall be designed to resist the overturning effects caused by earthquake forces specified in 4.3. At any level, the overturning moments to be resisted shall be determined using those seismic forces, Ft y Fx, that act on levels above the level under consideration. At any level, the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed in 4.4. Overturning effects on every element shall be carried down to the foundation.

4.5.2 Where any lateral load resisting element is discontinuous, such as for vertical irregularity Type D (table 5) or plan irregularity type D (Table 6), the columns that support such elements shall have the adequate strength to absorb the axial force resulting from the following load combinations, in addition to the other applicable load combinations:

1.0 Qp + 0.8 Qv + (3 R/8)Qa

0.90 Qp ± (3 R/8)Qa

4.6 DETERMINATION AND STORY DRIFT LIMITS

4.6.1 The story drift, Δ, shall be computed as the difference of the total displacements δx of the stories above and under the considered story. The total displacement δx of the mass center at level x shall be evaluated using the following expression:

$$\delta x = Cd\delta xe \qquad (4.11)$$

Where it is applicable, the story drift, Δ, shall be increased by the factor related to P-Delta effects as determined in 4.7.

4.6.2 The story drift computed as indicated in 4.6.1 shall not exceed the permissible values, Δa, given in table 8. For this purpose it is only allowed to calculate seismic forces using the fundamental period of the building in accordance with method B, omitting the limitation of the 80% indicated in 4.2.2 (2).

4.6.3. Every part of the building shall be designed and constructed to act as an integral unit when resisting seismic design forces, unless they are structurally separated by a distance far enough to avoid harmful contact when the total displacements δx appear, determined according 4.6.1.

4.7 P-DELTA EFFECTS

the P-Delta effect in shear, moments and drifts of the story x is not necessary to be considered when the stability coefficient, Θ, calculated with the following expression is less or equal to 0.10.

$$\Theta = \frac{Px\Delta}{VxhsxCd} \qquad (4.12)$$

The stability coefficient, Θ, shall not exceed, Θmax, determined from:

$$\Theta max. = \frac{0.7}{\beta\ Cd} \leq 0.25$$

where ß is the ratio of the shear force demanded and the shear force proportioned of the story that is between story x and x-1 and shall be taken conservatively as 1.0.

where the stability coefficient Θ is greater than 0.10, but lower or equal to Θmax, the story drift increment, the shear forces and moments, may be calculated adequately, multiplying these values by the factor 1/(1-Θ).

When Θ is greater than Θmax, the structure is potentially unstable and shall be redesigned.

4.8 VERTICAL COMPONENT OF SEISMIC FORCES

4.8.1 Horizontal cantilever members shall be designed for a net upward force of 0.5 A times the dead load, in addition to other applicable load combinations.

4.8.2 Horizontal prestressed members shall be designed, in addition to all other applicable load combinations, using not more than 50% of the dead load for the gravity load, alone or in combination with the lateral force effects.

CHAPTER 5

DYNAMIC ANALYSIS

5.1 SCOPE

The dynamic analyses procedures, when used, shall conform to the criteria established in this chapter. The analysis shall be based on the ground motions defined by the procedures established in 5.2. The structures designed according to this chapter, shall comply with all other applicable requirements of this technical standard.

5.2 GROUND MOTION

The ground motion may be represented by the following design spectra:

$$Si\ T_m < \frac{T_o}{3}$$

$$C_{sm} = \frac{IA}{R}\left[1 + \frac{3(C_o - 1)T_m}{T_o}\right]$$

$$Si\ \frac{T_o}{3} \leq T_m \leq T_o$$

$$C_{sm} = \frac{IAC_o}{R}$$

$$Si\ T_o < T_m \leq 4.0\ seg.$$

$$C_{sm} = \frac{IAC_o}{R}\left(\frac{T_o}{T_m}\right)^{2/3}$$

$$Si\ T_m > 4.0\ seg.$$

$$C_{sm} = \frac{2.5\,IAC_o\,T_o^{\ 2/3}}{R\,T_m^{\ 4/3}}$$

Any other representation of ground motion may be used as long as it has at least a 10 percent probability of being exceeded in 50 years, and it may be in any of the following ways:

a) A design spectra specific for the site, which shall be based on the geologic, tectonic, seismologic and soil characteristics of the site. Such spectra shall be developed considering a critical damping of 5%, unless a different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site.

b) Ground motion time histories for the site, which shall be representative of actual earthquake motions and developed in accordance with the geologic, tectonic, seismologic and soil characteristics of the site.

The vertical component of ground motion may be defined by scaling horizontal accelerations of the design spectra by a factor of 2/3. Different factors may be used when substantiated by site specific data.

5.3 MATHEMATICAL MODEL

The mathematical model of the structure shall represent the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its dynamic response. A three dimensional model shall be used for the dynamic analysis of structures with highly irregular plan configurations such as those defined in table 6 and having a rigid or semirigid diaphragm.

5.4 ANALYSIS PROCEDURES

5.4.1 Response spectrum analysis. An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

1) Number of modes. The requirements that all significant modes be included may be satisfied by demonstrating that for the modes considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

2) Combining modes. The peak member forces, displacements, story forces, story shears and base reactions for each mode shall be combined using methods established to estimate the maximum resulting values of these response parameters. When three dimensional models are used for analysis, modal interaction effects shall be considered.

3) Evaluation of results. When the base shear in one given direction, determined according 1) and 2), is less than the base shear determined by chapter 4, the first shall be modified as follows:

a) For irregular structures, it shall be used 100% of the static base shear.

b) For regular structures, it shall be used 90% of the static base shear, but not less than 80% of the base shear determined according to 4.2 using the value of T given by equation (4.3).

All the corresponding response parameters, including forces, moments and displacements, shall be adjusted in the same proportion as the base shear, except that for the adjustment of displacements the limitation of 80 % of method A may be omitted.

It is not necessary to use a base shear greater than the obtained according to the requirements prescribed in chapter 4.

4) Directional Effects. Directional effects for horizontal ground motion shall conform to the requirements of 4.1.1 y 4.1.2. the effects of vertical ground motion on horizontal cantilevers and prestressed elements shall be considered in accordance with 4.8. Vertical seismic response may be determined be dynamic response methods, in no case shall the response used for design be less than that obtained by the static method.

5) Torsion. The analysis shall account for torsional effects, including accidental torsional effects as prescribed in 4.4.

5.4.2 Time history analysis. This is an elastic or inelastic dynamic analysis, in which a mathematical model of the structure is submitted to a specific ground motion history (accelerogram). The dynamic response of the structure in function of time is obtained through a numeric integral of the movement equations. The base shear obtained by this procedure shall comply with the limitations prescribed in 5.4.1(3), literals (a) y (b).

CHAPTER 6

DESIGN REQUIREMENTS FOR STRUCTURAL FRAMING ELEMENTS

6.1 GENERAL.

6.1.1 All structural framing systems shall comply with the requirements of chapter 3. the components shall also comply with the specific requirements for the material contained in the corresponding Construction and Design Standards.

6.1.2 All building components shall be designed to resist the effects of seismic and gravity loads.

6.1.3 consideration shall be given to design for uplift effects caused by vertical loads. When the procedure of working stresses is used for designing, the dead loads considered to reduce uplift effects, shall be multiplied by 0.90.

6.1.4 Orthogonal Effects.

1) The effects of seismic motions that act in a direction non parallel to the resisting direction under consideration shall be considered when:

a) The structure has plan irregularity type E, accordance with table 6.

b) The structure has plan irregularity Type A, according to Table 6, for both major axes.

c) A column forms part of two or more intersecting lateral force resisting systems.

2) The requirement that orthogonal effects be considered may be satisfied by designing such elements for 100 percent of the prescribed design seismic forces in one direction plus 30 percent of the prescribed design seismic forces in the perpendicular direction. The combination requiring the greater component strength shall be used for design. Alternatively, the effects of the two orthogonal directions may be combined on a square root of the sum of the squares (SRSS) basis. When the SRSS method of combining directional effects is used, each term computed shall be assigned the sign that will result in the most conservative result.

6.2 STRUCTURAL FRAMING SYSTEMS.

6.2.1 General. Five types of structural systems defined in 3.6 are recognized in these provitions and are indicated in Table 7. Systems A, B, C and D are sub divided according to the vertical elements used to resist the lateral seismic forces. Special framing requirements are given in this chapter.

6.2.2 Ties and Continuity.

1) All parts of a structure shall be interconnected. The joints shall be capable of transmitting seismic forces to the lateral force resisting system. As a minimum, any smaller portion of the building shall be tied to the remainder of the building with elements having a transferring resistance of at least A/3 times the weight of the smaller portion. (See Table 1).

2) For each major or secondary beam or truss, an adequate adjoining shall be proportioned to resist a horizontal force acting parallel to the member. This strength shall be greater than A/5 times the sum of the dead load plus tributary live load acting on the member.

6.2.3 Collector Elements. Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element, providing the resistance to those forces.

6.2.4 Diaphragms.

1) Roof and floor diaphragms shall be designed to resist the forces determined in accordance with the following expression:

$$F_{px} = \frac{F_t + \sum_{i=1}^{n} F_i}{\sum_{i=1}^{n} W_i} W_{px} \qquad (6.1)$$

2) The Force Fpx determined from expression (6.1) need not exceed 0.75 A I Wpx, but shall not be less than 0.35 A I Wpx.

3) When the diaphragm is required to transfer seismic forces from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm due to offset in the placement of the elements or to changes in stiffness in the vertical elements, these forces shall be added to those determined from (6.1).

6.2.5 Framing below the base.

1) The strength and stiffness of the framing between the base and the foundation shall not be less than that of the superstructure. The detailing requirements of the corresponding Design and Construction Technical Standards shall be applied to columns supporting discontinuous lateral force resisting elements and the frame elements located below the base.

2) Foundations shall be capable of transmitting the base shear force and the design overturning forces, defined in chapter 4, from the structure to the supporting soil, but taking in count the short term dynamic nature of loads when the properties of soil are established.

6.2.6 All structures shall be separated from adjoining constructions and boundaries an appropriate distance in order to avoid contact by displacements caused by seismic motions. The separation shall permit a displacement equal to Cd times the one produced by design seismic forces.

CHAPTER 7

LATERAL FORCES IN NONSTRUCTURAL ELEMENTS

7.1 GENERAL

Non structural elements such as attachment elements for permanent equipment supported by a structure shall be designed to resist the total seismic forces described in 7.2. Friction resulting from gravity loads shall not be considered in the resistance of seismic loads.

7.2 TOTAL DESIGN SEISMIC FORCE.

7.2.1 The total design seismic force, Fp, shall be determined from the following expression:

$$F_p = A I C_p W_p \qquad (7.1)$$

7.2.2 The values of A and I of equation (7.1) shall be the values used for the building, defined in Tables 1 and 4 respectively.

7.2.3 In order to evaluate the coefficient Cp, non structural elements shall be classified as:

Rigid elements rigidly supported are those that have a vibration fundamental period less or equal to 0.06 seconds.

Non rigid elements flexibly supported are the ones that have a vibration fundamental period greater than 0.06 seconds.

7.2.3(1) The value of Cp for rigid elements rigidly supported is given in table 9.

7.2.3(2) Lateral forces for non rigid elements or flexibly supported shall be determined considering dynamic properties of the elements and its supports, but the value shall not be less than that given in Table 9. If no analysis is performed, the value of Cp shall be taken as two times the one specified in Table 9, but need not exceed 2.0. Piping systems, ducts and canalizations constructed with ductile materials and connections, where the values of Cp of table 9 can be used.

7.2.3(3) The value of Cp, for isolated members of the structure located at ground level or below, shall be taken as 2/3 of the value specified in table 9. However, design lateral force obtained shall not be less than that calculated using the requirements of chapter 8.

7.2.4 Total design Lateral seismic forces determined using expression (7.1) shall be distributed in proportion to the mass of each component of the element.

7.2.5 Forces determined in accordance with expression (7.1) shall be used to design the elements or its components and the joints and anchorages of the structure.

7.2.6 Deformation Compatibility. Non structural elements or the components that are attached to or are part of these elements, shall be designed to resist the forces determined by expression (7.1) and shall accommodate movements of the structure resulting from lateral forces or temperature changes. Such elements shall be supported or joined by structural members, according to the following considerations:

1) Connections and joints of non structural elements shall allow for a relative movement between stories of not less than two times story drift produced by wind, Cd times the elastic story drift calculated caused by seismic design forces or 2.5 cm; the greater.

2) Connections to permit movement in the plane of the element for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel or other connections providing equivalent sliding and ductility capacity.

3) Connections shall have sufficient ductility and rotation capacity to preclude fracture of the anchor elements or brittle failures at welds

4) Connections shall be designed for 1.33 times the force determined by expression (7.1).

5) All fasteners in connections, such as bolts, inserts, welds, dowels, etc. shall be designed to resist 4 times the forces determined by expression (7.1).

CHAPTER 8

OTHER STRUCTURES

8.1. GENERAL.

8.1.1 This chapter includes all isolated structures different from gravity load transmitting buildings and lateral load resisting systems. Any structure of this type shall be designed to resist minimum lateral forces specified in this chapter. The design shall comply with the applicable specifications of this technical standard.

8.1.2 The design of these structures shall provide enough and appropriate resistance and ductility according to the established requirements for buildings in this technical standard.

8.1.3 the seismic load, W, shall include all dead loads defined in Title II of the Regulation and the weight of materials contained in order to operate normally.

8.1.4 The fundamental period may be determined using method B of this technical standard or any similar method.

8.1.5 This type of structures need not comply the story drift limits indicated for buildings in this technical standard.

Drift limitations shall be established indirectly for structures whose failures may endanger human life, checking them in order to resist specified design forces and the calculated displacements and enlarged Cd times; P-Delta effects shall be evaluated considering calculated displacements multiplied by a factor of Cd. In structures where human life is not endangered, P-Delta effects shall be considered only for those structures with calculated drifts exceed the values of table 8.

8.1.6 The structures that support flexible non structural elements, with a combined weight that exceeds 25 percent of the weight of the structure, shall be designed considering interaction effects between the structure and the supported elements.

8.1.7 The procedure to determine the lateral force in structural systems similar to buildings (Systems A, B, C and D in Table 7), shall be selected in accordance with the requirements of chapter 3.

8.1.8 Structures with a period T less than 0.06 seconds, including connections, shall be designed for the lateral force obtained by means of the following expression:

$$V = 0.5\ A\ I\ W \quad (8.1)$$

The force V shall be distributed in accordance with the masses and shall be assumed to act in any horizontal direction.

8.1.9 Tanks with supported bottoms founded at or below grade, shall be designed to resist the seismic forces calculated using the procedures indicated in 8.1.8 for rigid structures, considering the entire weight of the tank and its contents. Alternatively, such tanks may be designed using a response spectrum analysis that includes consideration of the actual ground motion anticipated at the site and the inertial effects of the contained fluid.

8.1.10 Structures not covered in the previous sections, shall be designed to resist seismic lateral forces not less than the ones determined by the provitions of chapter, except that factors R and Cd shall be the ones specified in Table

10 and the seismic coefficient Cs used in the design shall not be less than 0.5 AI.

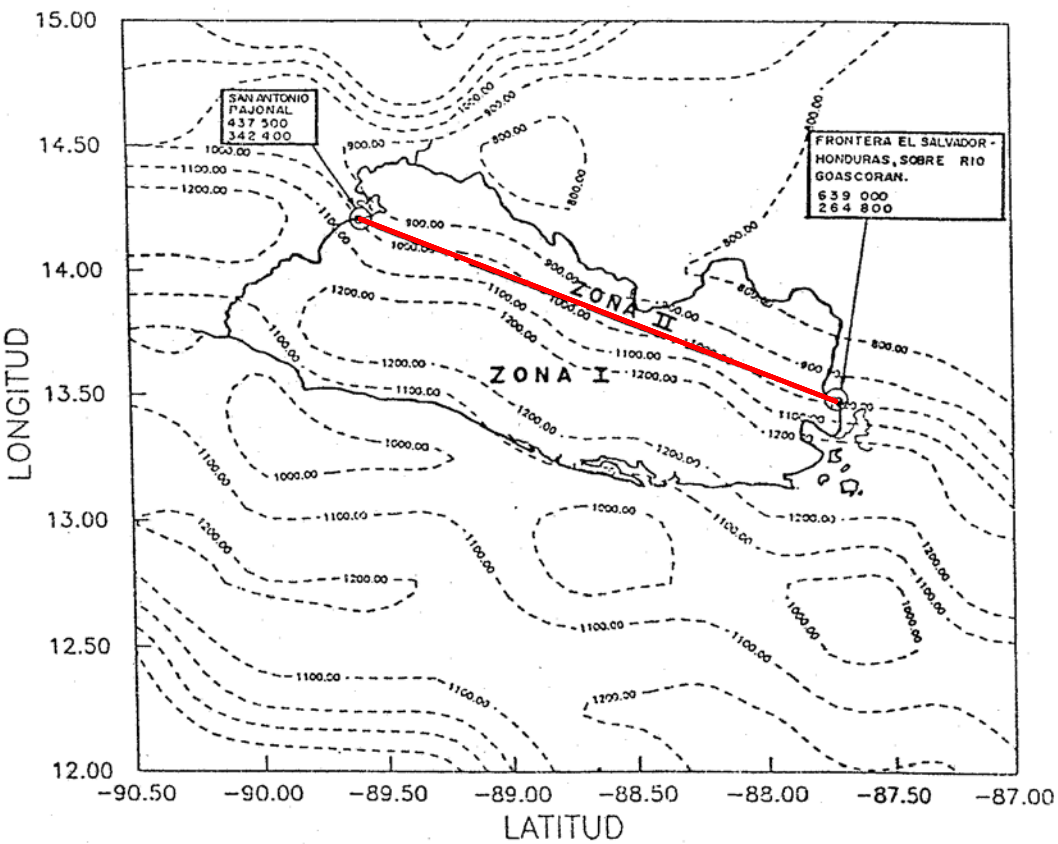
The vertical distribution of lateral seismic forces in these structures, may be determined using the provitions indicated in 4.3 or with the procedures specified in chapter 5.

TABLE 1
SEISMIC ZONE FACTOR A

ZONE *	FACTOR A
1	40
2	30

* See figure 1.

FIGURE 1
SEISMIC ZONE MAP OF EL SALVADOR



SYMBOLS

___.___.___.___. DIVIDING LINE OF SEISMIC ZONES. D= 215.93 Km. DIRECTION= S 66º 56' 06" E

NOTE: ZONE I INCLUDES ALL ISLANDS OF EL SALVADOR

SEISMIC ZONING FOR THE REPUBLIC OF EL SALVADOR, SEPT. 1993

TABLE 2

SITE COEFFICIENTS Co AND To(1)

Type	Description	Co	To
S1	The following Soil profiles: (a) Rock appearance materials characterized by shear wave velocities greater than 500 m/sec. (b) Rigid or dense soil condition with a thickness not greater than 30 m above the rock layer.	2.5	0.3
S2	The following soil profiles: (a) Rigid or very dense soil condition with a thickness equal to 30 m or more above the rock layer. (b) compact or very compact soils or medium dense with a thickness under 30 m.	2.75	0.5
S3	soil profile with an accumulated thickness from 4 to 12 m of cohesive soils from soft to medium compact or loose non cohesive soils.	3.0	0 6
S4	Soil profile that contains more than 12 m of soft cohesive soil or loose non cohesive soil and characterized by a shear wave velocity of less than 150 m/sec.	3.0	0.9

(1) The site soil profile shall be established by geotechnical data appropriately substained. In sites where the soil properties are not known with detail as to be able to establish the soil profile type, a soil profile type S3 shall be used.

NOTE: It shall be understood that at a greater depth than the one established for each soil profile, there is only rock as defined for S1 (a).

TABLE 3
OCCUPANCY CATEGORY

Occupancy Category	Occupancy or functions of structure
I	Includes those buildings that are essential after an earthquake in order to treat the emergency and preserve health and safety of people. Includes hospitals, health centers, fire stations, communication centers, schools and military structures and any other public service building or facility supporting toxic substances that are required in order to overcome the emergency.
II	Any building having high occupancy levels or facilities that are required to be operated immediately after an earthquake. Includes: government buildings, universities, nurseries, markets, commercial centers with an area greater than 3,000 m², warehouses with a plan area of 500 m² or more than 10 m high, halls with a capacity of more than 200 people, stadiums with outdoor bleaches with a capacity of 2,000 people or more, buildings with more than 4 stories or more than 1000 m² by level, museums, monuments, transportation terminals, hospitals different from the occupancy category I, occupancies that store especially expensive equipment, etc.
III	Constructions that have a low occupancy level, includes those common constructions assigned to housing, offices, commercial centers, hotels, industrial buildings and all other constructions not included in occupancy categories I y II.

TABLE 4
IMPORTANCE FACTORS

Occupancy Category (1)	Importance Factor I
I Essential or dangerous Facilities	1.5
II Special Occupancy Buildings	1.2
III Normal Occupancy Buildings	1.0

(1) Occupancy types or functions of structures in each of the categories are listed in table 3.

TABLE 5
VERTICAL STRUCTURAL IRREGULARITIES

	Irregularity type and definition
A.	Stiffness Irregularity – Soft Story A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.
B.	Mass irregularity Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.
C.	Vertical geometric irregularity Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force resisting system in any story is more than 130 percent of that in an adjacent story. One story penthouses need not be considered
D.	In plane discontinuity in vertical lateral force resisting element This discontinuity is considered when the lateral force resisting elements have an in plane offset of the lateral load resisting elements greater than the length of those elements.
E.	Discontinuity in capacity – weak story A weak story is one in which the story length is less than 80 percent of that in the story above. The story strength is the total strength of all seismic resisting elements sharing the story shear for the direction under consideration.

TABLE 6
PLAN STRUCTURAL IRREGULARITIES

	Irregularity type and definition
A.	Torsional Irregularity. If diaphragms are not flexible torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.
B.	Re-entrant corners Plan configurations of a structure and its lateral force resisting system contain re-entrant corners, when both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.
C.	Diaphragm discontinuity Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the building plan.
D.	Out of plane offsets Discontinuities in a lateral force path, such as out of plane offsets of the vertical elements.
E.	Nonparallel systems The vertical lateral load resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force resisting system.

TABLE 7
STRUCTURAL SYSTEMS

Basic Structural system (1)	Lateral force resisting system Description	Cd(2)	R(3)	H(4)
SYSTEM A	1. Special detailing concrete or steel frames	8	12	S.L. (5)
	2. Concrete frames with intermediate detailing	5	5	15
	3. Steel frames with ordinary detailing	6	7	30
SYSTEM B	1. Walls:			
	a. Concrete	7	8	50
	b. Masonry	6	7	35
	2. Braced steel frames:			
	a. Eccentrically	6	10	50
	b. Concentrically	7	8	50
SYSTEM C	1. Concrete wall combined with:			
	a. Concrete or steel frames with special detailing	9	12	S.L.
	b. Concrete frames with intermediate detailing or steel frames with ordinary detailing	7	8	S.L.
	2. Masonry walls combined with:			
	a. Concrete or steel frames with special detailing	6	7	50
	b. Concrete frames with intermediate detailing or Steel frames with ordinary detailing	5	6	30
	3. Braced steel frames combined with concrete or steel frames with special detailing:			
	a. Eccentric Bracing	6	12	S.L.
	b. Concentric Bracing	7	10	S.L.
SYSTEM D	1. Walls:			
	a. Concrete	6	7	35
	b. Masonry	5	6	25
	2. Braced steel frames	5	6	50
SYSTEM E	1. Systems with mass essentially concentrated in the superior end (Inverted Pendulum)	3	3	--
	2. Systems with mass essentially distributed in height	4	4	--

NOTES:
(1) Basic structural systems are defined in 3.6.
(2) Displacement Amplification Factor.
(3) For combinations of structural systems see 3.8.
(4) H = limit Height, in m.
(5) S.L.= No height limit.

TABLE 8
PERMISSIBLE VALUES OF STORY DRIFT Δa (*)

Type of Building	Occupancy Category		
	I	II	III
Structural steel, one story building without any equipment tied to the structure and without fragile finishes.	0.015 hsx	0.020 hsx	S.L. (**)
4-story buildings or less without fragile finishes.	0.010 hsx	0.015 hsx	0.020 hsx
All other buildings	0.010 hsx	0.015 hsx	0.015 hsx

(*) hsx is the height of the story under level x
(**) S.L. = No limit.

TABLA 9
HORIZONTAL FORCE FACTORS Cp
FOR NONBUILDING STRUCTURES

	Cp values	Notes
I. Parts of the structure		
1. Walls, including the following:		
a. Cantilevered column type structures	2.00	
b. Any wall type	0.75	
2. Appendixes (penthouses), except when they are inside the building frame	0.75	
II. Non structural components		
1. Boundary walls	0.75	
2. Ornaments and external and internal appendixes	2.00	
3. Chimneys, antennas, trussed towers and tanks on members:		
a. Supported or projected as unbraced cantilevers above the roof in a length greater than the half of its total height	2.00	
b. Others, including those supported under the roof with an unbraced projection above the roof less than the half of its height, or braced or guyed to the structural frame at or above its mass center	0.75	
3. Signs and Billboards	2.00	
4. Storage bins or Hoppers (including the content)	0.75	
5. Anchorages for permanent closets supported on the floor and for shelves with a height greater than 1.50 m (including its content)	0.75	
6. Anchorages for ceilings and lamps	0.75	*
7. Floor access Systems	0.75	
III. Equipments		
1. Tanks and silos (including the content) Together with the supporting and anchorage and supporting systems	0.75	
2. Electrical, mechanical and plumbing equipment, including pipes and equipment and related works	0.75	

* For purposes of determining seismic forces a weight of the ceiling of at least 20 kg/m² shall be used.

TABLE 10
FACTORS R AND Cd

	Type of structure	R	Cd
1	Tanks, Vessels or pressurized spheres, on braced or unbraced legs	3	3
2	Cast in place concrete Silos and chimneys having walls continuous to the foundations	4	4
3	All other distributed mass cantilever structures, not covered previously, including antennas, chimneys, silos and skirt supported vertical vessels	4	4
4	Trussed towers (freestanding or guyed) and chimneys	4	4
5	Inverted pendulum type structures	3	3
6	Cooling towers	4	4
7	Bins and hoppers on braced or unbraced legs	4	4
8	Storage racks	4	4
9	Signs and Billboards	4	4
10	Amusement structures and monuments	3	3
11	All other self supported structures not otherwise covered	4	4

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