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CURRENT SEISMIC DESIGN REGULATIONS FOR COSTA RICA

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ABSTRACT

Some comments on the earthquake engineering development in Costa Rica are given. A description of the newly started Strong Motion Instrumentation Program is given. The description includes remarks on both free field strong motion recording stations as well as instrumented buildings. Also, a description of the current norm for determination of lateral forces due to earthquakes is given. Some comments are also offered as to the appropriateness of using high ductility ratios for certain structural systems.

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I. INTRODUCTION

Safeguarding life and property from the destructive effects of earthquakes is a major worldwide problem. In spite of the increased awareness of this problem, earthquakes each year claim many lives and cause enormous damage to man-made structures and other facilities. In order to design safe, economical structures and facilities in earthquake prone regions of the world, it is necessary to understand the nature of the ground motions that these systems may be expected to experience during their lifetimes. It is equally important to understand the behavior of the materials from which these structural systems are made as well as the interaction between the different structural elements in the system.

The purpose of every seismic design regulation should then be that of preserving human life and reducing the socioeconomic impact of strong ground motions. However, it is necessary to bear in mind that design regulations only represent a lower bound approximation to the problem of safe construction. Furthermore, it should be realized that design codes are continuous processes that only represent the state of knowledge, or the lack of it, at a certain stage of the development of the profession. Many lessons have been learned from every earthquake and many more will continue to be learned.

II. SEISMICITY IN CENTRAL AMERICA

Central America is located in a region of high seismicity. Figure 1 shows a map of the seismicity of Mexico and Central America for the period 1962-1969. The entire isthmus is contained in the so-called Caribbean Plate. The western border of this plate coincides with the Pacific coast line of the isthmus. There, the Cocos Plate is sliding under the Caribbean Plate. The origin of many of the larger earthquakes in the region can be traced to this subduction zone. In the most defined areas of the subduction zone, earthquakes have been found to have hypocenters between 50 and 100 Km deep.

There are also earthquakes of lower magnitude that occur inside the volcanic arc. They may not be called intraplate ground motions but nevertheless, they are shallow and of low magnitude and have usually affected smaller areas.

The most destructive earthquakes in the region in the last two decades have had the second type of origin mentioned above. These are, San Salvador in 1964 and in 1987 and Managua in 1972. All these events have been associated with local faults in the intermountain valleys of volcanic and alluvial origin. In the case of San Salvador in 1964 and Managua in 1972, the cities sustained heavy damage due to poor construction methods, particularly in the low income housing sectors. Once again in

1987, San Salvador suffered heavy damage in the adobe and bahareque construction. However, it is important to note that there was considerable damage in a good number of modern buildings. Most regrettable of all was the damage to important facilities like hospitals and clinics. All except one of these facilities suffered enough damage to put them out of operation at the most critical time.

Guatemala suffered an earthquake in 1976 due to the strike slip fault between the Caribbean Plate and the North American Plate. Lifelines and important large buildings were damaged during this event.

III. EARTHQUAKES IN COSTA RICA

Costa Rica is the fifth republic of the former Central American Confederation. It occupies the southernmost portion of the isthmus. As with the rest of Central America, there are two main causes for strong ground motion. The first one is the subduction zone and the second one is the faulting associated with the volcanic arc.

The data about past earthquakes goes back to about 400 years. The reconstruction of the seismic history for the country has been through the newspaper accounts as well as through recollections from the local authorities. The most recent damaging earthquakes that have ocurred in this century were the 1910 and 1911 Cartago Earthquakes, the 1925 Orotina Earthquake and most recently the Tilaran Earthquake of 1973.

The Cartago Earthquakes had epicenters within 20 Km of the capital city, San Jose. They caused the collapse of a great number of masonry and adobe constructions. The causative fault was identified as a local fault and the rupture length was of about 10 to 15 Km. The magnitudes of the shocks were estimated at around 5.5 to 6 in the Richter Scale. The events are very significant because in spite of their low magnitude, they could prove to be a bigger threat than the much larger tectonic ground motions. The characteristics of the near field events of moderate magnitude have not fully been determined. An excellent opportunity of gaining some insight into the subject was presented by the large number of records obtained during the San Salvador Earthquake of 1987.

The Earthquake in Orotina in 1925 has an unclear origin. It is believed to have been caused by an undefined and transitional area of the subduction zone towards the central portion of the Pacific Coast of Costa Rica. There is little information on the effects of the motion because of the lack population in the area.

The Tilaran Earthquake in 1973, was due to a local fault in the Northwestern part of the country. It caused moderate damage to the church in the local town. It also cause damage in most of the one story concrete block housing.

The first attempt to regulate the construction to make it safe against earthquakes came after the Cartago Earthquakes. Those first recommendations included the utilization of a tying beam on the top of the masonry walls for one story housing. In essence, the observed damage in the earthquakes was the sole basis of the seismic design recommendations. Later on, the provision for the application of a lateral load equivalent to 10 percent of the total weight of the structure was added. The code was seldom enforced and as a result disappeared into oblivion.

The Managua Earthquake of 1972 was the cause of considerable concern among the Civil Engineering community. As it is often the case, in the first few month after the earthquake, several initiatives for the prevention of a disaster of similar magnitude in Costa Rica got under way. The most significant of them all was the establishment of a Permanent Seismic Code Commission. It was charged with the task of drafting a Seismic Design Code intended to regulate all civil engineering construction. The first edition of the Code was put in use in 1974.

Together with the effort of drafting a Seismic Code, a National Seismological Network was implemented with the cooperation of the University of Costa Rica and the National Institute for Electricity.

Another two major seismic motions occurred in the subsequent years. In 1978 a 7.0 magnitude earthquake was felt off the coast in the northern portion of the Pacific shore. In 1983, a 7.2 magnitude earthquake shook the southern portion of the Pacific coast. This time the epicenter was inland. However, the level of damage was small in the vicinity of the epicenter. The level of damage in the city of San Jose was higher than in the epicenter region. There was collapse of cladding in several buildings and cracking of some columns in corners of structures with soft stories.

IV. STRONG MOTION INSTRUMENTATION PROGRAM

Soon after the Golfito Earthquake of 1983, the Engineering Research Institute of the University of Costa Rica initiated installation of a strong motion instrumentation program. The main objective of the program is to obtain basic information for the design of earthquake resistant structures. The program had the initial support of the United States Agency for International Development, the University of California at Santa Cruz, as well as the Government of Costa Rica. At present the Program is being supported solely by the University of Costa Rica.

The Strong Motion Program operates 19 Recording Stations distributed throughout the country. The map in figure 2 shows the location of most of the stations. The central cluster of stations in the city of San Jose is not depicted in its entirety.

The siting of the different stations was based on the Seismic Risk Study for Costa Rica performed by Morgart et al. in 1977. It has been revised several times after the first lay out of the siting criteria. Figures 3 to 6 show the maximum acceleration maps for return periods of 50, 100, 500 and 1000 years proposed in the above mentioned study. The maps for the higher return periods show the assumption of fundamentally three seismic sources for the territory. All of these seismic sources go along the Pacific coast. Figure 2 shows the location of the seven recorded ground motion since the installation of the network. There appears to be an indication of agreement between the location of the epicenters and the proposed risk maps. Other important sources like the one mentioned above on the central valley of the country have shown higher seismic activity along local faults during the present year.

The typical cross section of the subduction fault is shown in figure 7. The suggested disposition of strong motion instrumentation along this type of fault is one of parallel lines of instruments as it is indicated in the figure. The chosen disposition of the strong motion stations tries to follow the same pattern. However, the density of the array falls somewhat short of the suggested level. The average suggested distance between the stations is 20 Km on each of three lines following the fault. The current density of the array is about twice that amount and in some portions it gets closer to triple the amount.

The instrumentation in the Central Valley is more random in nature. There, the principal objective has been more to instrument buildings. The objective here is to facilitate response studies that could lead to improved understanding of the dynamic behavior and the potential for damage to structures under seismic loading. There are currently four multi-story buildings instrumented in the city of San Jose. A typical diagram of the instrumentation performed in those buildings is given in figure 15. The chosen arrangement has been to locate one instrument in the basement or ground floor and then one in the top floor. They are kept in the same vertical line as much as possible. The problems of torsional motion cannot be measured with this instrumentation. Hence, the instruments are kept as close as possible to the center of rigidity of the structure, usually the elevator core. The buildings have been chosen to have the most diverse qualities in view of the budget constraints. The structures are made out of steel, light weight concrete, reinforced concrete and they have the two most used structural arrangements: moment resisting frame and combined frame-wall resistant structure.

Table 1 indicates the location of every strong motion recording station. These locations are given by instruments.

So, the first two accelerographs show the same coordinates since they belong to the same station. On Table 2 the orientation of the instruments is given so as to permit the identification of the maximum acceleration components in a ground motion.

The largest event recorded so far by the network was registered on 15 July, 1987. There were three seismic events on the same day and within a radius of about 35 Km. The range of magnitudes in the Richter scale was of 4.0 to 4.4, the body wave magnitude reached 5.0 on the largest event. Figure 9 shows the Modified Mercalli intensity map for the first recorded event, denominated Quepos1, the figure also shows the locations of the triggered instruments in the network. The pattern seems to suggest directivity of the signal in the north direction. The closest station to the epicenter did not triggered. After thoroughly checking the instrument it was concluded that it was in proper working condition during both events. It is unfortunate that the network did not have more stations between the origin point and the cluster of stations in the Central Valley.

During this events an apparent amplification of the signal was recorded in two nearby stations in San Jose. Figure 11 shows the recorded ground motions in the ICE and Hatillo Stations respectively. It is interesting to notice that even though the stations are only 2 Km apart the peak accelerations for the Hatillo Station more than double the ones recorded at ICE. Truly, for a low level excitation like the one at hand it would be adventurous to make any definite statements about the phenomena. However, it does call for a closer observation in future events. Table 3 shows the values of all the recorded peak accelerations for both Quepos1 and Quepos2 events.

The next stage in the development of the Strong Motion Instrumentation Program contemplates the installation of five new stations. The projected growth includes the construction of two more stations along the Pacific shore at intermediate points form the existing ones, a station on a rock site near the city of San Jose, a station in the Caribbean coast and finally a station at an intermediate distance between the Pacific shore and the city of San Jose. With the new stations it is hoped that a better understanding of the source mechanism, attenuation conditions as well as the wave propagation characteristics from the subduction zone will be attained.

Another important projected development of the Program contemplates the installation of a local array for measuring the effects of surface geology in the seismic motion in the Central Valley. Figure 8 shows the typical configuration of such an array for a wide valley such as the one under consideration. The configuration proposed will only include three instruments; one downhole set on rock at about 45 m depth, another one located directly above the first one at surface level and a third one approximately 600 m apart on a rock outcrop. Adequate instrumentation to achieve the objectives of the experiment should be

a. Triaxial instruments

b. Accurate relative timing.

c. A sample rate of at least 100 sps.

d. A band width of at least .1- 25 Hz.

Investigation of site geometry, velocity of bedrock, material properties for soft soil deposits including data from several bore holes is currently under way.

Also currently under way is a study of active faults in the Central Valley of the country. This is a much needed work that will help improve the location of the strong motion recording stations.

V. RECOMMENDED PROVISIONS FOR BUILDING CONSTRUCTION

The present revision of the Costa Rican Seismic Code (CSCR-86) is based on the designation of a Seismic Force Coefficient SFC for each projected building. Such coefficient is used to determine the total base shear applied to the structure as a result of the earthquake action. The total base shear force is then obtained as a fraction of the total weight of the structure, i.e.

$$\text{Total Base Shear} = \text{SFC} \times \text{Total Weight} \qquad (1)$$

As in the case of the U.S. code standards (SEAOC, ATC, UBC, etc.), there is not a clear agreement on the number of parameters on which the seismic force coefficient should depend. Neither is there a clear idea of what parameter or combination of parameters is most appropriate to measure the level of damage that can be attributed to the seismic excitation.

The traditional idea has been to utilize the maximum ground acceleration as a measure of the level of damage expected in an earthquake. The rationale of this assumption being that the destructive potential is all due to the inertial forces excited during the ground motion. However, observation of earthquake response of buildings in different parts of the world seem to indicate that the correlation between level of damage and maximum ground acceleration is not very good. (Ang, 1988, DerKiureghian, 1988; McCabe, 1987).

The SFC used in the provisions depends ultimately on the following parameters

- Linear Dynamic Characteristics of Structure; i.e., natural periods of vibration, structural damping.

- Type of Supporting Soil; i.e., rock, stiff soil, soft soil.

- Expected Maximum Level of Ground Acceleration.

- Structural Ductility.

The maximum level of ground acceleration is obtained from the maps of isoacceleration showed in figures 3 through 6. Thus, the maps amount to a seismic zonation for the country on the basis of return periods of 50, 100, 500 and 1000 years. The determination of the return period for a certain building is based on the expected life span for the structure as well as on the probability of exceedance of a certain acceleration level. The graphs have a very strong dependency on the reliability of the attenuation curves used for the country as well as the wave propagation characteristics for the different types of paths encountered. For this particular topic the code continues to be based in the study of Morgart et al.

The weight of the structure is calculated as the dead weight plus 15% of the live load as a minimum.

The rest of the parameters are summarized in figures 12, 13 and 14 in the form of curves for Dynamic Amplification vs. Period for different types of soil foundation and different types of structural arrangements.

For structures in the short period range, i.e., below .4 seconds, there is a 20% increment in the dynamic amplification factor for soft soil foundation with respect to rock of stiff soil foundation. For periods longer than .4 seconds the increment on the coefficient could be as high as 100% when comparing soft soil to rock location for a period of 1.0 seconds.

The structural ductility is defined as a function of the resisting structural system. Table 2.4.1 of the code shows this requirement as well as the proposed structural damping associated with each structural type.

For buildings that are classified as regular in plan and elevation and seven or less stories of 30 m or less above ground level, the calculation of the fundamental period is facilitated by the following empirical formula:

$T = 0.12N$ Steel Frame Building

$T = 0.10N$ Reinforced Concrete Frame Building

$T = 0.08N$ Reinforced Concrete Frame Wall Building, Steel Braced Frame

Building

T = 0.05N Reinforced Concrete Structural Wall Building.

where:

T = Fundamental Natural Period in seconds.

N = Total number of stories.

This formula is used as a first approximation to the period of the structure. In a second stage, the period must be recalculated using the elastic displacements resulting from the response of the structure when subjected to the seismic loads acting statically at each floor level. The periods should be calculated using the following equation

$$T = 2\pi \sqrt{\frac{1}{g} \frac{\sum_{i=1}^{N} W_i \cdot (\delta_i^c)^2}{\sum_{i=1}^{N} F_i \quad \delta_i^c}} \qquad (2)$$

where

δ_i = Elastic displacement at level i due to seismic forces.

$F_i = C\eta \, h_i \, W_i$

F_i = Seismic force at level i.

C = Seismic coefficient

$$\eta = \frac{\sum_{k=1}^{N} W_k \quad h_k}{\sum^{N} W_k \quad h_k^2}$$

For more general buildings, a dynamic modal analysis is required. The number of modes to be used in such procedure is taken as one fourth of the number of degrees of freedom. The modal responses are then combined using the square root of the sum of the square method. An indication is given to provide for better modal combination when the system has coupling modes.

The code also provides an upper bound for the expected level of drift in a building. For each type of resisting structural system, the total horizontal displacements as well as the relative story displacements are estimated for the inelastic range as follows

$$\delta_i = K\delta_i^e$$
$$\Delta_i = K\Delta^e_i \qquad (3)$$

where

δ_i = Total horizontal inelastic displacement at level i.

Δ_i = Relative inelastic displacement for level i.

K = Inelastic displacement factor given in Table 2.8.1

δ_i^e, Δ_i^e = Elastic displacements.

The non-structural components on the building must be separated from the resisting system using the previous calculations.

The drift limitation in the code is given in terms of the relative interstory displacement. The corresponding values for the different types of resisting structural systems are given in Table 2.8.2 of the norm.

CONCLUSIONS

The efforts on designing structures against earthquakes has been a continuing process. It has been mentioned that the efforts of providing the designers with adequate provisions has been a task undertaken since 1972. However, the understanding of the effects of earthquakes is not by any means completed. The current norm is likely to be modified by the measurement of motion characteristics for the different regions of the country. A new seismic risk study that could include a larger data based is needed. It is also necessary to provide some form of verification to the assumptions made on the behavior of the materials currently used in common construction. The current version of the code does not make a clear distinction between member ductility and structural ductility. The ductility requirements are rather high for type 5 resisting structural system. However, it must be said that the drift provision takes care of this deficiency in an indirect manner.

May it be added that Costa Rica, as any underdeveloped country is facing a harsh economic situation. As the population increases the resources become more scarce. The problem of providing housing for every citizen is therefore becoming more acute. Hence, investigation on new building materials, new resisting structural systems, and new construction techniques is a high priority. Bearing this in mind, it should be stated that the proposed solutions to the housing problem must be durable.

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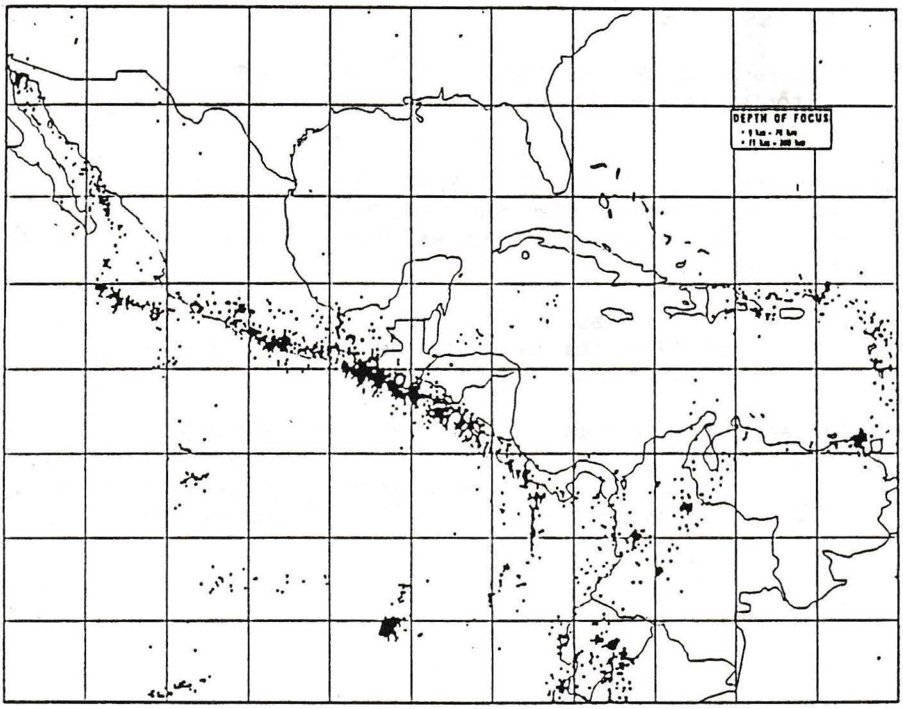


FIGURE 1 Seismicity of Mexico and Central America 1962 - 1969.

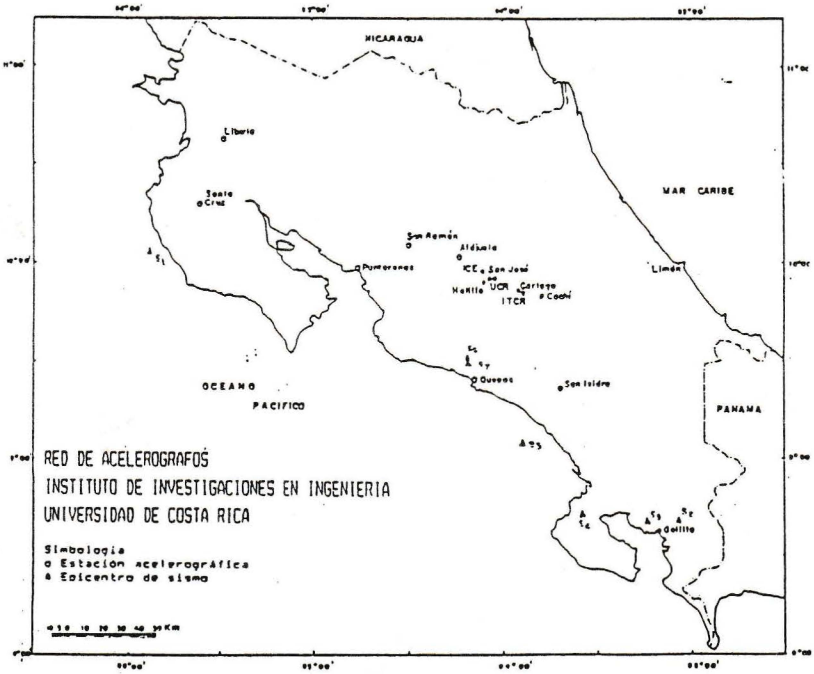


FIGURE 2 Location of Strong Motion Recording Stations

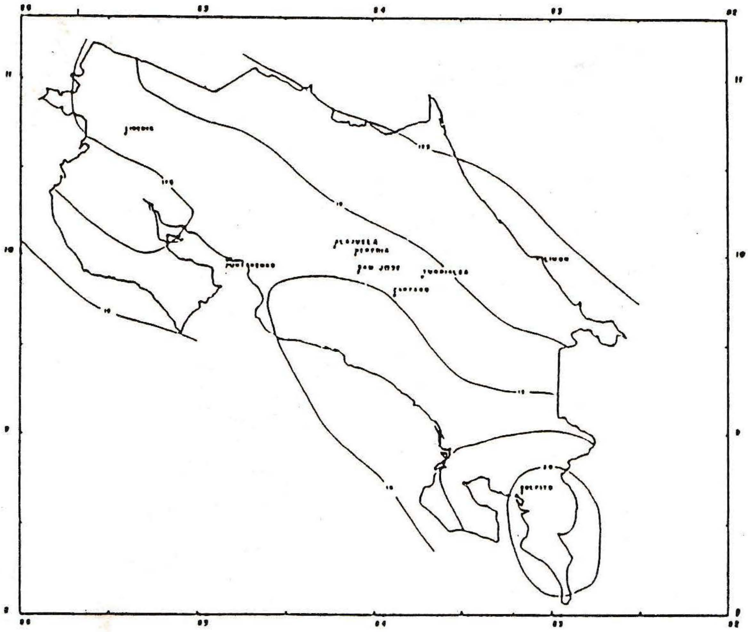


FIGURE 3 Maximum Acceleration Map for 50 year Return Period, as % of g.

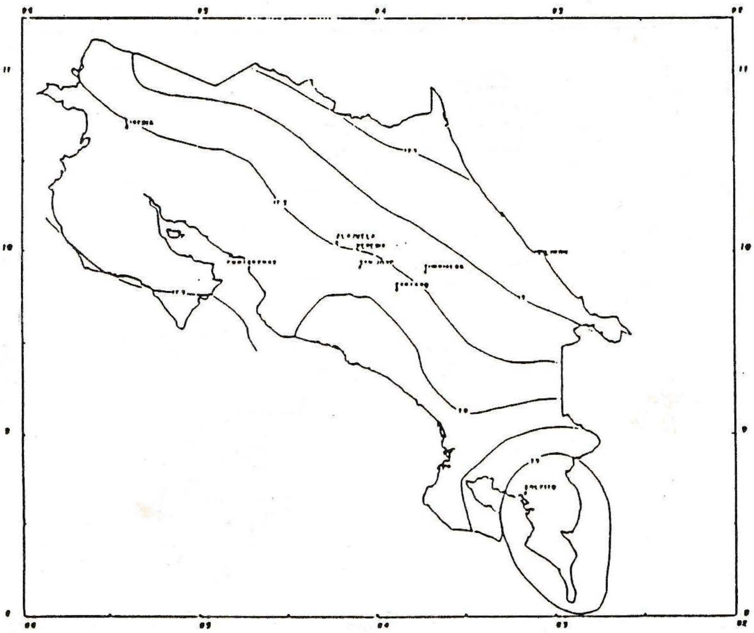


FIGURE 4 Maximum Acceleration Map for 100 year Return Period, as % of g.

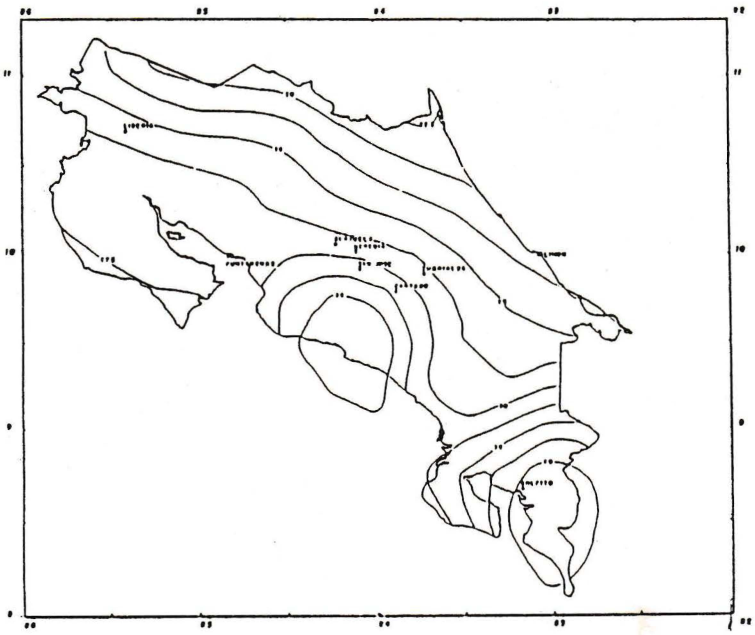


FIGURE 5 Maximum Acceleration Map for 500 year Return Period, as % of g.

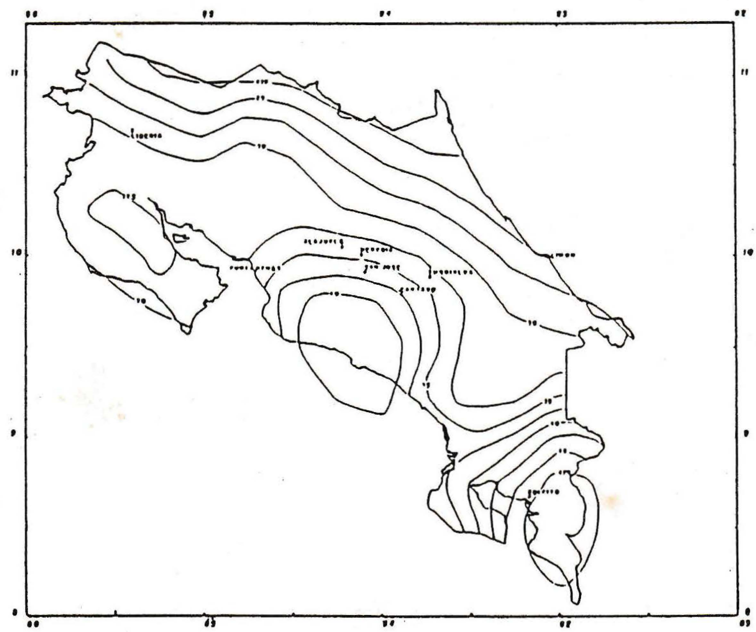


FIGURE 6 Maximum Acceleration Map for 1000 year Return Period, as % of g.

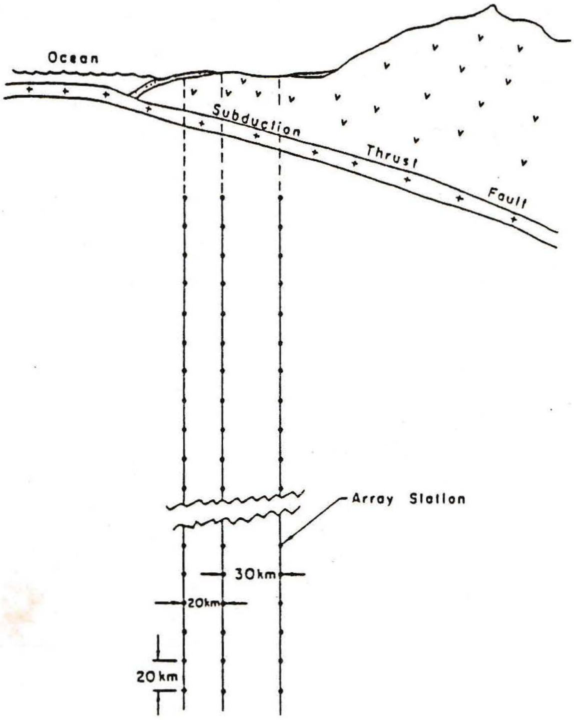


FIGURE 7 Typical Source Mechanism and Wave Propagation Array for Subduction Thrust Fault.

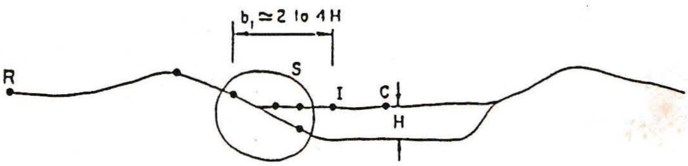


FIGURE 8 Typical Simple Extended Array Configuration for Wide Valley.

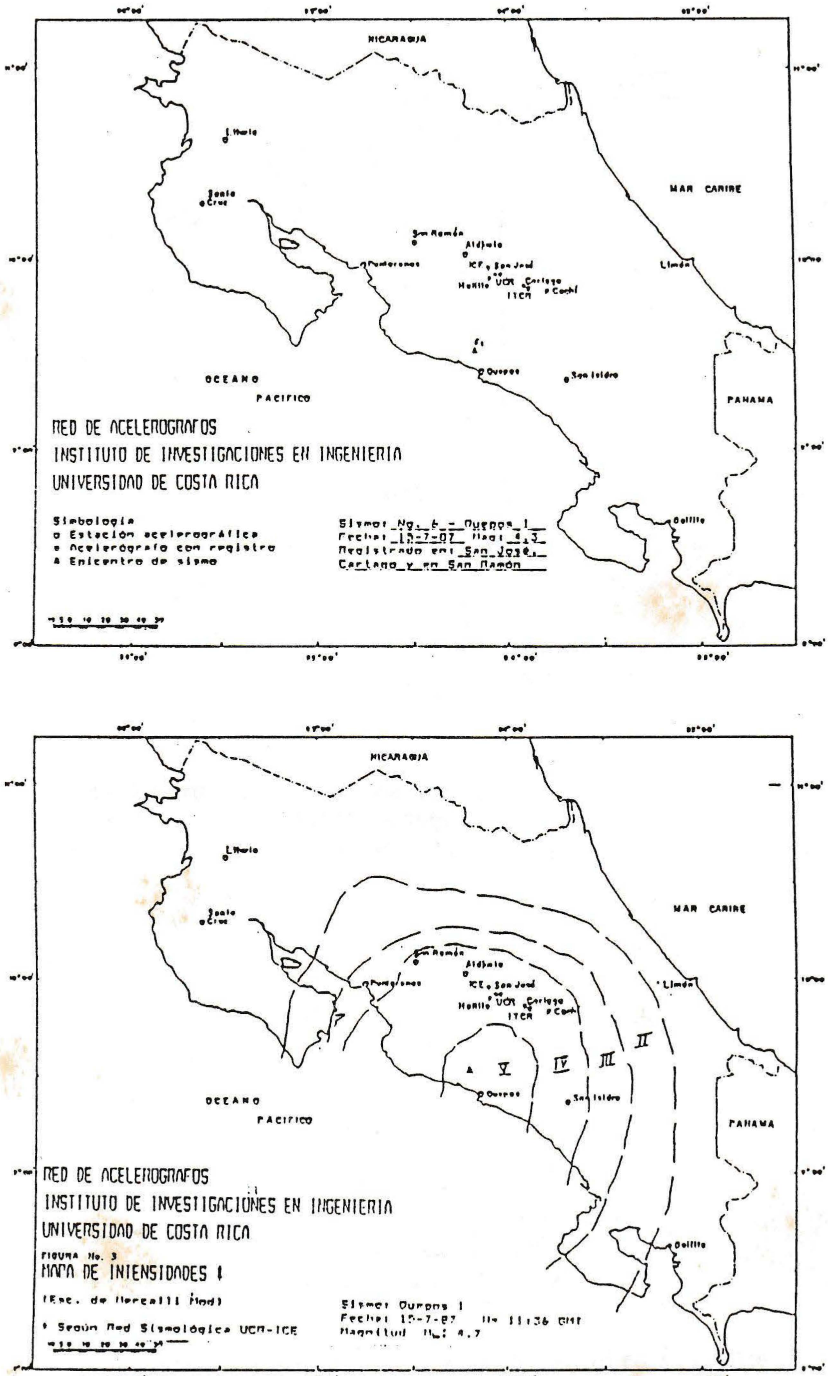
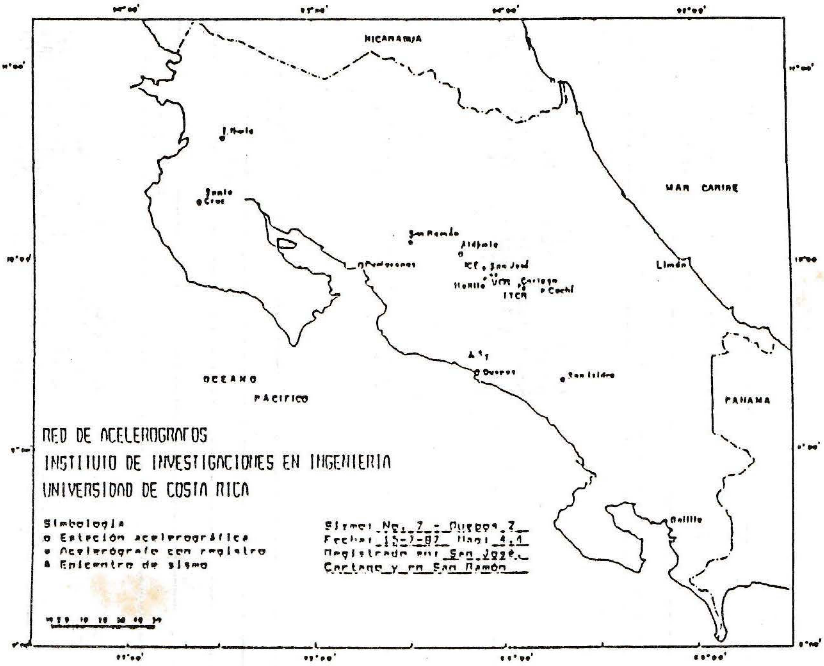


FIGURE 9 Modified Mercalli Intensity Map and Epicenter Location for the Quepos-1 Event of 15 July 1987



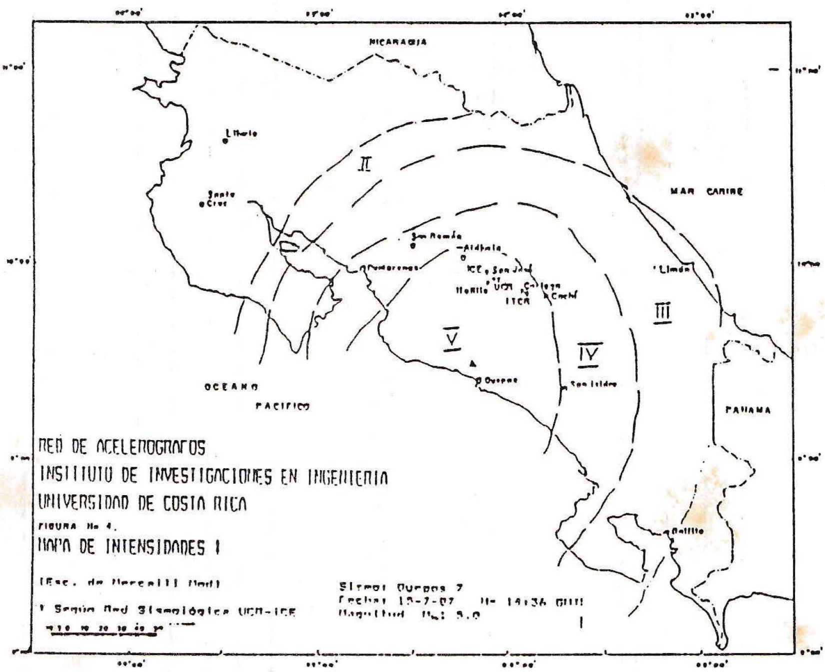


FIGURE 10 Modified Mercalli Intensity Map and Epicenter Location for the Quepos-2 Event of 15 July 1987

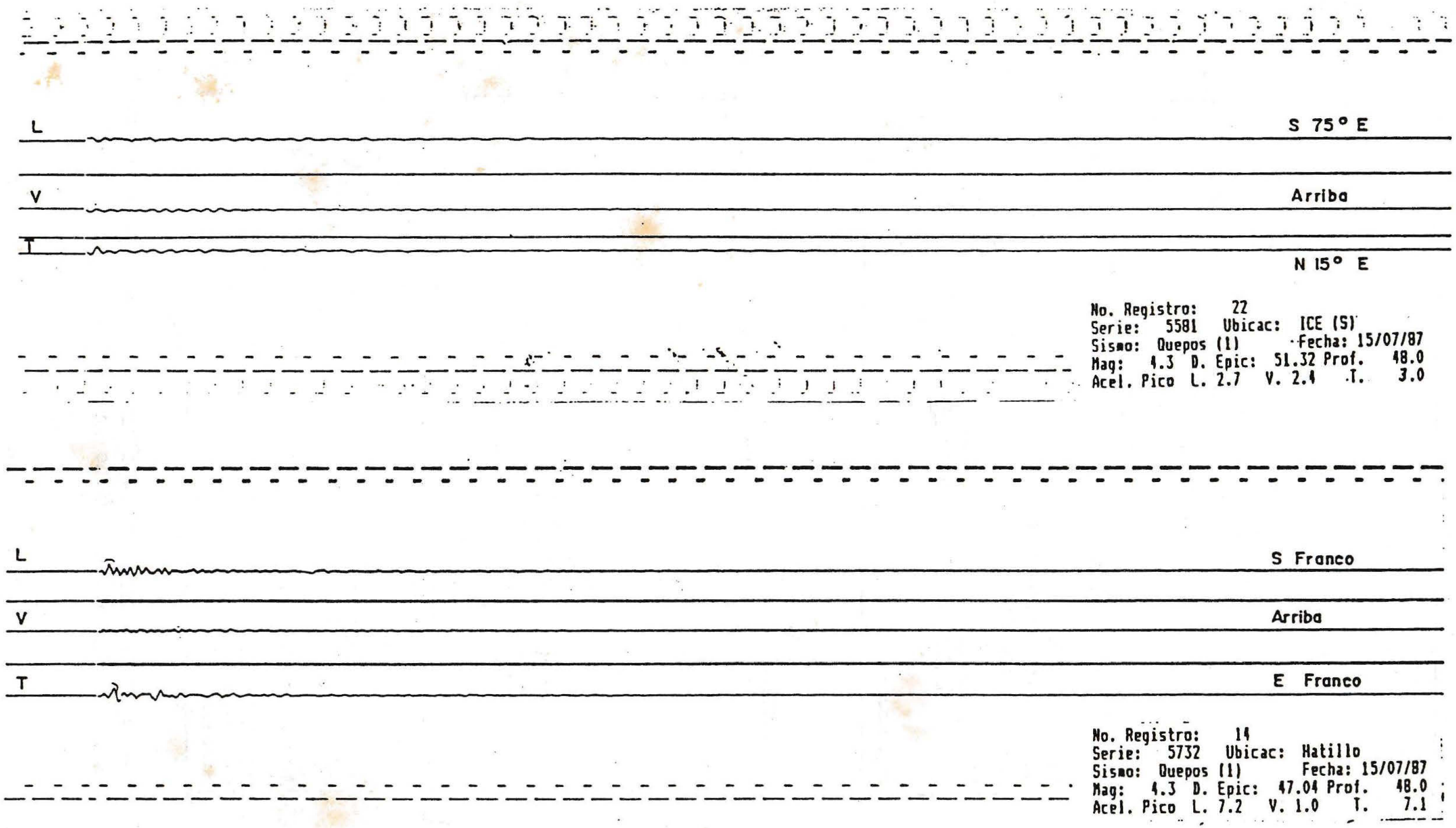


FIGURE 11 Accelerogram Records Obtained at ICE and Hatillo Stations in the City of San Jose

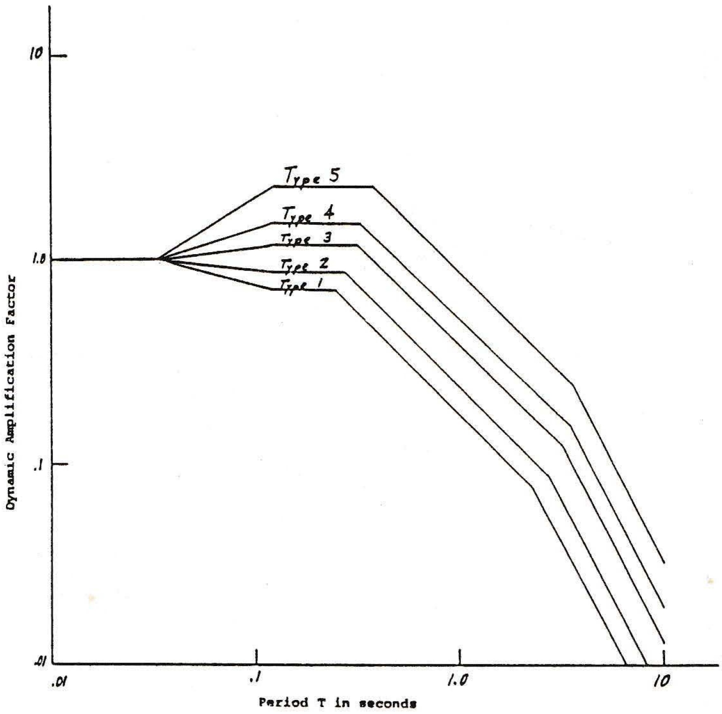


FIGURE 12 Dynamic Amplification Factor for Rock Site.

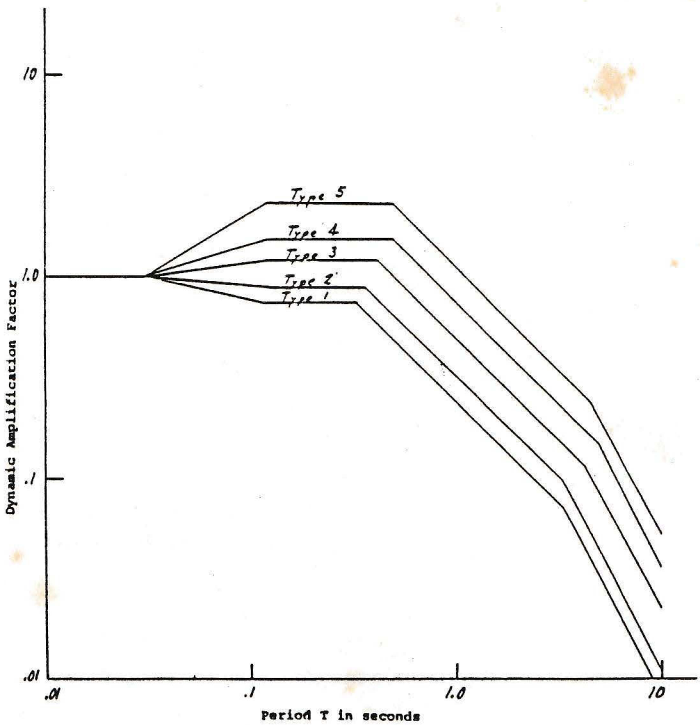


FIGURE 13 Dynamic Amplification Factor for Hard Soil Site.

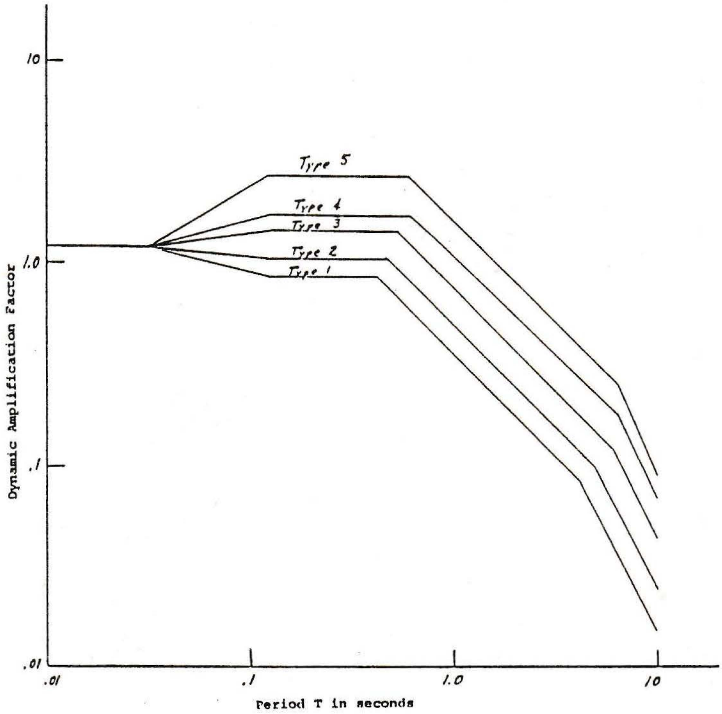


FIGURE 14 Dynamic Amplification Factor for Soft Soil Site.

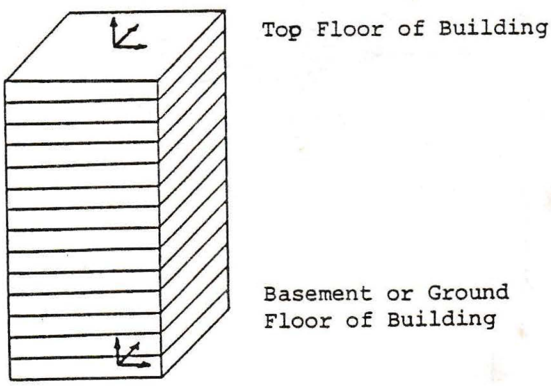


FIGURE 15 Instrumentation for a Multi-Story Building.

TABLE 1

Location of Strong Motion Recording Stations.

No.	Serie	Localización	Fecha Instalac.	Coordenadas Latitud N	Coordenadas Longitud W	Altitud (msnm)
1	5733	Bco Nal (S)	11-Oct-85	09:56'14"	84:04'55"	1145
2	5577	Bco Nal (17)	10-Oct-85	09:56'14"	84:04'55"	1200
3	5585	Aurola (S)	15-Nov-85	09:56'17"	84:04'38"	1145
4	5735	Aurola (16)	15-Nov-85	09:56'17"	84:04'38"	1170
5	5728	INS (S)	28-Aug-85	09:56'18"	84:04'31"	1140
6	1728	INS (P 12)	15-May-85	09:56'18"	84:04'31"	1185
7	5581	ICE (S)	09-Oct-85	09:57'25"	84:06'15"	1125
8	5729	San Pedro	17-Apr-84	09:56'18"	84:03'02"	1200
9	5732	Matillo	29-May-85	09:54'59"	84:05'53"	1130
10	5576	Cartago	10-Apr-84	09:52'02"	83:55'31"	1445
11	5730	Tecnológico	17-Apr-84	09:51'32"	83:54'46"	1400
12	5578	Cachí	10-Apr-86	09:50'32"	83:48'17"	1000
13	5582	San Isidro	06-Mar-85	09:22'25"	83:42'27"	705
14	5584	Golfito	07-Mar-85	08:38'41"	83:10'19"	10
15	5727	Quepos	21-May-85	09:25'54"	84:09'57"	5
16	5726	San Ramón	17-Nov-84	10:05'13"	84:29'00"	1120
17	5731	Puntarenas	08-Aug-85	09:58'36"	84:45'02"	10
18	5580	Santa Cruz	06-Aug-84	10:17'16"	85:35'35"	45
19	5583	Liberia	22-Jan-86	10:37'10"	85:27'37"	120
20	5734	Alajuela	12-Mar-86	10:01'07"	84:12'59"	750
21	5577	Recope	28-Sep-87	09:53'42"	83:56'26"	1560
22	2050	Geologia UCR	30-Sep-87	09:56'22"	84:03'16"	1200

TABLE 2

Orientation of Strong Motion Instruments.

No.	Serie	Localización	Orientación Long.	Orientación Vert.	Orientación Transv.
1	5733	Bco Mal (S)	S 82:00 E	Arriba	N 08:00 E
2	5579	Bco Mal (17)	S 08:00 E	Arriba	N 82:00 E
3	5585	Aurola (S)	S 85:00 W	Arriba	S 05:00 E
4	5735	Aurola (16)	S 85:00 W	Arriba	S 05:00 E
5	5728	INS (S)	N 10:00 E	Arriba	N 80:00 W
6	1728	INS (P 12)	N 07:00 W	Arriba	S 83:00 W
7	5581	ICE (S)	S 75:00 E	Arriba	S 15:00 E
8	5729	San Pedro	S Franco	Arriba	E Franco
9	5732	Hatillo	S Franco	Arriba	E Franco
10	5576	Cartago	S Franco	Arriba	E Franco
11	5730	Tecnológico	S Franco	Arriba	E Franco
12	5578	Cachi	S Franco	Arriba	E Franco
13	5582	San Isidro	S Franco	Arriba	E Franco
14	5584	Golfito	S Franco	Arriba	E Franco
15	5727	Quepos	S Franco	Arriba	E Franco
16	5726	San Ramón	S Franco	Arriba	E Franco
17	5731	Puntarenas	N 85:00 E	Arriba	N 05:00 W
18	5580	Santa Cruz	S Franco	Arriba	E Franco
19	5583	Liberia	S Franco	Arriba	E Franco
20	5734	Alajuela	S Franco	Arriba	E Franco
21	5577	Recope	S Franco	Arriba	E Franco
22	2050	Geologia UCR	S Franco	Arriba	E Franco

TABLE 3

Peak Acceleration Values Recorded During the Quepos Events of 15 July, 1987.

Nombre Sismo	Mag. (ML)	Acelerógrafo Serie SMA-1	Ubicación	Acel. Pico L (%g)	V (%g)	T (%g)
Quepos 1	4.3	5732	Hatillo	7.20	1.00	7.10
		5733	Bco Nal (S)	1.10	1.60	1.70
		5579	Bco Nal (P 17)	1.1	2.1	2.2
		5576	Cartago	5.8	3.6	4.6
		5730	Tecnológico	5.9	2.8	7.6
		5735	Aurola (P 16)	1.0	2.1	2.0
		1728	INS (P 12)	3.0	3.8	2.2
		5728	INS (S)	2.8	1.0	2.8
		5581	ICE (S)	2.7	2.4	3.0
		5726	San Ramón	1.6	1.1	2.1
Quepos 2	4.4	5732	Hatillo	5.0	1.0	6.0
		5576	Cartago	4.2	2.1	4.1
		5733	Bco Nal (S)	1.4	1.1	1.7
		5579	Bco Nal (P 17)	0.9	1.6	1.4
		5735	Aurola (P 16)	2.1	2.6	1.5
		1728	INS (P 12)	2.0	3.3	1.7
		5728	INS (S)	2.3	0.5	1.7
		5581	ICE (S)	2.7	1.2	3.2
		5726	San Ramón	1.1	1.1	2.1

TABLA 1.2.1. Probabilidades de excedencia recomendadas.

Capacidad para resistir deformaciones inelásticas	Importancia de las estructuras Muy grande	Grande	Usual	Poca
NINGUNA (Ductilidad igual que 1)	.05-.10	.10-.15	.15-.25	.25-.40
POCA (Ductilidad mayor que 1 pero menor o igual que 3.5)	.10-.15	.15-.25	.25-.40	.40-.60
ADECUADA (Ductilidad mayor que 3.5)	.15-.25	.25-.40	.40-.60	.60-.75

NOTA: Para efectos de ilustración y guía, se presentan algunos ejemplos de importancia en estructuras:

Muy grande: Represa y tanque de oscilación de un gran proyecto hidroeléctrico, contenedor de un reactor nuclear.

Grande: Puente de una carretera muy importante, puerto principal, hospital, tanque de gran capacidad para almacenamiento de combustibles.

Usual: La mayoría de los edificios y puentes.

Poca: Construcciones rurales, puentes de caminos de penetración, estructuras provisionales.

Grupo	Vida Económica Util (años)	Probabilidad de excedencia	Período de Retorno (años)
A	100	0.20	500
B	50	0.40	100
C	30	0.45	50

TABLA 2.4.1. Valores de ductilidad y amortiguamiento para los tipos estructurales.

TIPO	DUCTILIDAD	AMORTIGUAMIENTO
1	6	.05
2	4	.05
3	2	.07
4	1.2	.10
5	1	.05

TABLA 2.5.1. Sobrecargas mínimas

DESTINO DEL PISO	SOBRECARGA (kg/m²)
Habitación (casas de habitación, apartamentos, viviendas, dormitorios, cuartos de hotel, edificios para internados en escuelas, cuarteles, cárceles, correccionales, hospitales y similares)	250
Oficinas, despachos y laboratorios	300
Comunicación de uso público para peatones (pasillos, escaleras, rampas, vestíbulos y pasajes de acceso libre al público)	400
Estadios, salones de baile y lugares de espectáculos desprovistos de asientos fijos.	500
Lugares de reunión con asientos fijos (templos, cines, teatros, gimnasios, salones de bailes, restaurantes, salones de lectura, aulas, salas de juego y similares).	400
Comercios, bodegas y fábricas de mercancía ligera.	500
Comercios, bodegas y fábricas de mercancía con peso intermedio.	650
Comercios, bodegas y fábricas de mercancía pesada.	800
Techos de fibrocemento, láminas de acero galvanizado y otros.	40
Azoteas con pendiente superior a 5 por ciento.	100
Azoteas con pendiente inferior a 5 por ciento.	200
Voladizos en vía pública (marquesinas, balcones y similares).	200
Garajes y aparcamientos (para automóviles exclusivamente).	400
Andamios y cimbras para concreto	80

Nota: Las cargas dadas en esta tabla son mínimas por lo que deberán tenerse en cuenta las condiciones reales. .

TABLA 2.8.1. Factor de desplazamiento inelástico

TIPO DE ESTRUCTURA	FACTOR K
1	6
2	4
3	2
4	1.2
5	1

TABLA 2.8.2. Límite superior de los desplazamientos relativos $\Delta i/Hi$

TIPO DE ESTRUCTURA	GRUPO A	GRUPO B Y C
1	0.010	0.016
2	0.010	0.014
3	0.010	0.011
4	0.008	0.008
5a	0.010	0.016
5b, c	0.008	0.008

donde:

$H_i = h_{i+1} - h_{i-1}$ = Altura entre el nivel inferior y superior del piso i.

TABLA 2.91. Factores de seguridad para la capacidad soportante de los suelos

Cargas Estático	Método Ultimo	Esfuerzos de Trabajo
$\frac{Pmín.}{Pmáx.} \geqslant .25$	2.0	3.0
$\frac{Pmín.}{Pmáx.} < .25$	1.7	2.5
Cargas Dinámicas		
$\frac{Pmín.}{Pmáx.} \geqslant .25$	1.5	2.0
$\frac{Pmín.}{Pmáx} < .25$	1.2	1.6

donde:

Pmáx. y Pmín. son las presiones máxima y mínima en el suelo, que se calcularán suponiendo una distribución lineal de ellas.

El caso Pmín./Pmax. < 0.25 incluye el caso de una distribución triangulas de presiones.

TABLA 2.10.1. Coeficiente sísmico para componentes arquitectónicos Xp

COMPONENTE	FACTOR Xp
Apéndices	
Muros exteriores no cargados	1.2
Elementos anclados a muros o techos	2.0
Enchapes	1.5
Elementos de cubierta y techos	1.2
Recipientes y componentes misceláneos	1.0
Divisiones y Muros	
De escalera y ascensores	1.3
De conductos verticales	1.2
De corredores de salida incluyendo el cielo raso	1.2
De corredores privados	1.0
Separaciones de áreas de altura completa	1.0
Otros componentes arquitectónicos anclados al cielo raso, las paredes o el piso	1.2

TABLA 2.10.2. Coeficiente sísmico para componentes mecánicos o eléctricos Xc

COMPONENTES	FACTOR Xc
Sistemas eléctricos de emergencia	2.0
Sistemas de detección de fuego y humo	2.0
Sistemas de extinción de fuego	2.0
Componentes de sistemas de seguridad humana	2.0
Calderas, hornos, incineradores, calentadores de agua y otros equipos que usen fuentes combustibles de energía o fuentes de alta temperatura.	2.0
Sistemas de comunicación	1.5
Sistemas primarios de cables eléctricos	2.0
Centros de control de motores eléctricos, dispositivos de control de motores, dispositivos de distribución, transformadores y subestaciones.	1.5
Equipos rotantes o reciprocantes	1.5
Equipos presionizados	2.0
Maquinaria de manufactura y proceso	1.2
Conductos y tuberías de sistemas de distribución	1.2
Pantallas y tableros eléctricos	1.5
Fajas transportadoras de material	1.2
Lámparas	1.5