C.- CONSIDERATIONS ABOUT THE VULNERABILITY TO NATURAL HAZARDS

C.1.- USED INFORMATION

The required levels for the earthquake and extreme winds actions to be used in the safety verification, were given in Sections A.2.4 and A.3.3 respectively. In the same way, in Sections B.2 through B.7 the site gathered information of the hospital buildings was described.

No structural layout or drafts, neither dimensions, nor installation information for any of the buildings was found. . Therefore, the considerations that are presented here are based on previous experiences with buildings of similar materials, structuration configurations.

A sample of the type of nails used for the fixation of the corrugated metallic sheets that serve as roof cover was obtained (Photos 38, 39 and 40). The pull-out resistance of those type of straight nails, which pierce through wood, was determined experimentally as shown in Figure C.1. For b = 14 mm, it was reached F = 23 Kg and for b = 25 mm, F exceeded 40 Kg.

In both cases, the resistance under a static stress suffered reductions when vibrations were introduced to the system by means of banging on top of the wooden beam.

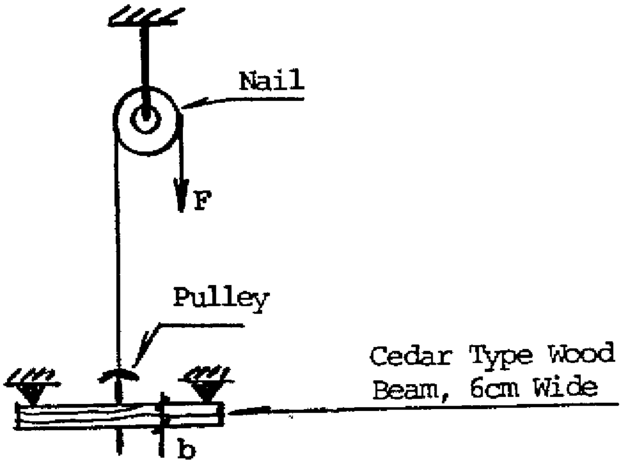

FIGURE C.1 PULL-OUT TEST OF A NAIL

Based on the obtained results and keeping in mind that the effects of gusts, specially at the edges of roofs and close to the ridge, introduce dynamic requirements, the following resistance for the tested nails is addopted: 16 Kg for 1/2" wood planks and 32 Kg for 1" wood planks, which are the widths observed at the site.

The nails were also tested under bending stresses: out of 4 tests, all exceeded bending of 45° without breackage and 3 exceeded 90° without breackage. There were no tests done on the extraction of bent nails; its resistance can be estimated at least as twice as those of the straight nails.

In several inspected rooftops the practice of leaving the projected nails straight was recorded; see as an example, Photo 27.

C.2.- COMPARISON OF TOTAL LATERAL FORCES DUE TO QUAKES AND EXTREME WINDS

In Part A of this report, Sections A.2.4 and A.3.3, the requirements for the verification of the actions due to extreme winds and expected earthquakes were established. Independently from the local strains and performance demands that each requirement imposes, which are of different nature, the total lateral forces due to each action represent an adequate comparison index.

This is the main goal of this section, which as will be seen, is a guide of the critical aspects that deserve attention.

C.2.1.- SELECTION OF THE CASE TO BE STUDIED

Having seen the variety of exposed buildings in the hospital area, it was decided to select a building with a large exposed area to the actions of winds of Eastern predominance, with a wind vulnerable roof, having an average slope of about 30 degrees, of somewhat above-average total height; base dimensions should be large enough in order to ensure enough stiffness for having a small fundamental period, generating inertia forces representative of the seismic action.

This is a three level building, similar to Building H (the tallest of the hospital) although less slender, having a cross section like the one described in Figure C.2. Of rectangular base, it has a total length of 38 m.

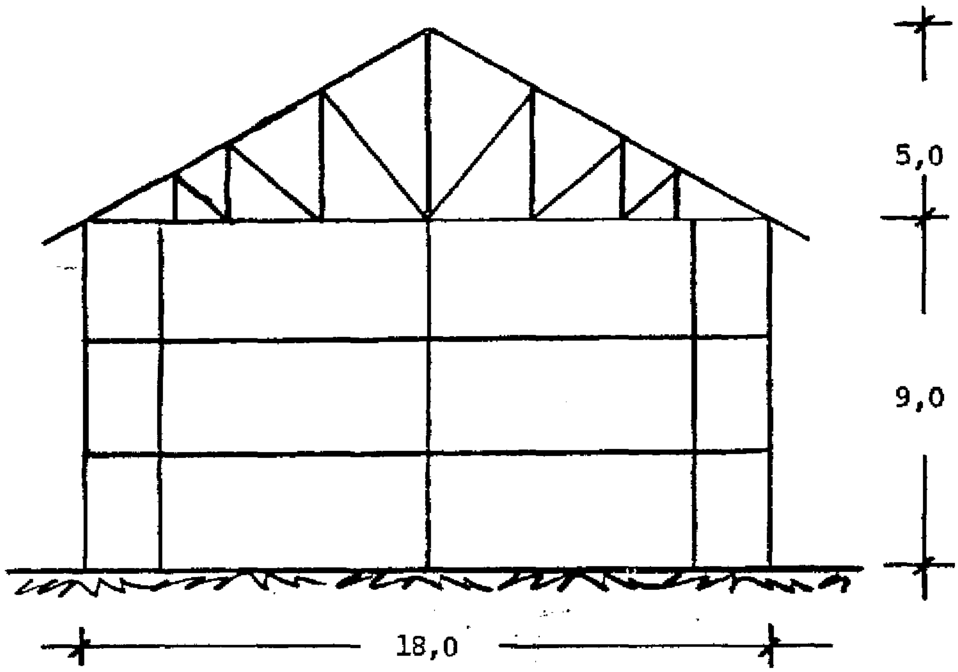

FIGURE C.2 CROSS SECTION OF THE SELECTED BUILDING; LENGTH = 38 m

For the determination of its own weight, the following unitary weights have been adopted based in the COVENIN code 2002-88:

- Roof metallic covering + asphalt sheet .... 15 Kg/m^2
- Structural woods .... 1000 Kg/m^3
- Dovetailing (1"), on top of spandrel beams .... 60 Kg/m^2
- Lateral wood pannels and windows .... 50 Kg/m^2
- 15 cm thick hollow clay block walls , rubbed on both sides .... 230 Kg/m^2
- Reinforced concrete slabs, one way reincorced spandrels, 20 cm thick, with ceramic or gres tyle floors .... 350 Kg/m^2
- Liveload for Hospitalization and Operating Rooms under service conditions : (250 Kg/m^2 x 0,25) .... 63 Kg/m^2

Weights:

- Slabs: 18 x 38 x 2 x (350)	479 Ton
- Partition walls 1 ml for each 10 m2 of base times 6 m of effective height = 410 x 230	94 Ton
- Roofs 2 x (9/cos 30) x 38 x (60 + 15)	59 Ton
- Lateral pannels and windows 2 x 38 x 8 x 50	30 Ton
- Service load: (18 - 4) x 38 x 2 x 63	67 Ton
TOTAL:	729 Ton

The total weight is typical of a light structure, like most of the hospital buildings with the exception of Building J.

C.2.2.- WIND ACTIONS

The analysis of the wind forces will be executed according to criteria established in Section A.3.3, for a maximal wind velocity in Kingstown of 35 m/sec = 126 Km/hour (CUBIC, Part 2, Section 2). The building is classified in Group A, with an eolic importance factor of α = 1,15 (Section A.1.2). Because of its configuration (slenderness 14/18 = 0,78 < 5), sheet roofs, with relatively closed faÇades (fixed shade), natural period estimated to be less than 1 second, this building classifies as Type I (Section 4.2). Finally, because of being located in urban areas (or suburban), with obstructions which have an average height of less than 10 m, the exposition classifies as Type B (Section 5 2).

According to Table 6.2.2 (a), the wind pressures to be considered are.

Windward:

$$p_z = q_z G_h C_p - q_h GC_{pi}$$

Leeward:

$$p_h = q_h G_h C_p - q_h GC_{pi}$$

$$GC_{pi} = \pm\ 0{,}25$$

For the Type B exposition and average roof height of h = 11,5, from Table 6.2.4 (α) we have:

$$G_h = 1{,}46$$

WIND TRANSVERSE TO THE RIDGE

For : L/b = 18/38 = 0,47, Table 6.2.5.1 gives:

For FaÇades:

Windward: $C_p = 0{,}8$

Leeward: $C_p = 0{,}5$

Lateral: $C_p = -\,0{,}7$

For roofs, with L/h = 18/11,5 = 1,56: α = arctg (5/9) = 29º:

Windward: C_p = - 0,20

Leeward: C_p = - 0,70

WIND PARALLELL TO THE RIDGE

For Façades:

Relationship: L/b = 38/18 = 2,11:

Windward: C_p = 0,8

Leeward: C_p = - 0,3

Lateral: C_p = - 0,7

For roofs:

Windward and Leeward: C_p = - 0,7

C.2.3.- DYNAMIC PRESSURE

According to the formula (6.7) of the Code:

$$q = 0{,}00485\ K\ \alpha\ V^2$$

Where K is read in Table 6.2.3.1.

The dynamic pressures to be considered in the surfaces of faÇades and roofs are synthesized in Tables C.1 and C.2. The last two columns of the tables represent: (i) external action + interior drive; and (ii) external action + interior suction, respectively.

TABLE C.1

PRESSURES FOR WIND TRANSVERSE TO THE RIDGE

SURFACE		Z	h	k_Z	k_h	q	$q\,G_h\,C_p + q_h\,GC_{pi}$	$q\,G_h\,C_p - q_h\,GC_{pi}$
		(m)				(kg/m^2)		
Façades	Windward	4,5		0,363		32,1	25,3	49,7
		6,0		0,413		36,6	30,5	54,9
		9,0		0,494		43,7	38,8	63,2
	Leeward		11,5		0,551	48,8	-47,8	-23,4
	Lateral		11,5		0,551	48,8	-62,1	-37,7
Roof	Windward		11,5		0,551	48,8	-26,4	-2,0
	Leeward		11,5		0,551	48,8	-62,1	-37,7

TABLE C.2

PRESSURES FOR WIND PARALLEL TO THE RIDGE

SURFACE		Z	h	k_Z	k_h	q	$q\,G_h\,C_p + q_h\,GC_{pi}$	$q\,G_h\,C_p - q_h\,GC_{pi}$
		(m)				(kg/m^2)		
Façades	Windward	4,5		0,363		32,1	25,3	49,7
		6,0		0,413		36,6	30,5	54,9
		8,0		0,469		41,5	36,3	60,7
		10,0		0,518		45,9	41,4	65,8
		12,0		0,562		49,8	46,0	70,4
		14,0		0,601		53,2	49,9	74,3
	Leeward		11,5		0,551	48,8	-9,2	-33,6
	Lateral		11,5		0,551	48,8	-62,1	-37,7
Roof	Windward		11,5		0,551	48,8	-62,1	-37,7
	Leeward		11,5		0,551	48,8	-62,1	-37,7

C.2.4.- TOTAL LATERAL FORCES

WIND TRANSVERSE TO THE RIDGE

The two load cases of Table C.1 are:

V_1 = 38 (25,3 x 4,5 + 30,5 x 2 + 38,8 x 2,5 + 47,8 x 9 + 62,1 x cos 60 x 5 - 26,4 x x cos 60 x 5) = 30,1 Ton

V_2 = 38 (49,7 x 4,5 + 54,9 x 2 + 63,2 x 2,5 + 23,4 x 9 + (37,7 - 2) cos 60 x 5) = = 30,1 Ton

WIND PARALLELL TO THE RIDGE

The two load cases of Table C.2 are:

V_1 = 18 (25,3 x 4,5 + 30,5 x 1,5 + 36,3 x 2,0) + 17 x 41,4 x 2 + 10 x 46,0 x 2 + + 49,9 x 4 x 2 + (18 x 9 + 18 x 5/2) 9,2 = 7,9 Ton

V_2 = 18 (49,7 x 4,5 + 54,9 x 1,5 + 60,7 x 2,0) + 17 x 65,8 x 2 + 70,4 x 10 x 2 + + 74,3 x 4 x 2 + (18 x 9 + 18 x 5/2) 33,6 = 18,9 Ton

The maximum lateral load (30,1 Ton), represents 4,1% of the total estimated weight of the building (729 Ton).

The suction forces act over the roof cover and sheeting; these are particularly elevated at the edges of roofs and gable-ends which require a greater number of fixation nails in the majority of the inspected roofs (see Photos 35, 36, 37 and 38).

The wood stresses, as a consequence of the bending moments created by maximum wind pressures (62,1 Kg/m^2 according to Table C.1 and C.2), are generated on the flat beams (11 x 12 cm) that support the roofs of the corridors (see Figure B.7, Detail 2).

With 2,9 m spans and 1,5 m wide wind load, the acting bending moment is worth:

$$M = 62{,}1 \times 1{,}50 \times 2{,}90^2 \;/8 = 97{,}9 \;\text{Kg-m}$$

Therefore the maximum stress due to wood bending is equal to:

$$\sigma = (99{,}7 \times 100 \times 5{,}5 \times 12) \,/\, (11^3 \times 12) = 41 \;\text{Kg/cm}^2$$

This stress is less than 50% of the allowable stresses under bending for Type C wood (weaker wood group, with densities of less than 550 Kg/m^3 PADT-REFORT, 1980).

C.2.5.- SEISMIC ACTIONS

The eventual vibrations of seismic type coming from nearby earthquakes associated to volcanic activity, with shallow focus and moderate magnitudes, are covered by the reference acceleration levels implicit in Figure A.9.

For the expected vibration periods, less the 0,5 sec, and limited ductilities which are assigned to the majority of the hospital buildings, the maximum shear forces that are infered from Figure A.9 are equal to 12,5% of the total estimated weight of the building. The center of gravity of this lateral force, three times greater than that of the wind, is somewhat lower than this latter.

Hammering between the slabs of adjacent buildings is foreseeable due to its closeness; because of being at the same level, only limited local damages are expected.

C.2.6.- OTHER ACTIONS

Due to its distance to the eventual center of emission of ashes and predominant winds in the hospital area, the ash widths on top of the roofs is not expected to exceed the calculated wind pressures.

As far as eventual ground settlements is concerned, there has been no record of movements nor observed damages attributable to ground settlements. The only collected information about local soil conditions, is that from a construction worker from the area who qualified that soil as: "stony", reason why the classification of the ground as S2 Type has been kept.