CHAPTER 3. CONSTRUCTION OF MODELS

3. 1 Construction Sequence

Location of the site for 4 no models is Barrio Las Minas, Baruta. The site has been provided by Baruta municipality. The site is a backfilled area that was filled during the construction of highway roads in 1960's. The slope has the inclination of 21.8 degrees (1.0: 0.4). The reinforced concrete work for models was done at first, and seismic reinforcement works such as brick walls and concrete block walls at lower floor were completed by the middle of July 2004 (photo 3.1~3.8).

The embedment of foundation footing from the ground surface is assumed to be 1.0m to 1.2m by the hearing before construction, and 1.2m is used considering the condition of filled slope. Detail construction works are shown in photos 3.9~3.44. These photos show characteristics of construction works for Barrio houses.

3. 2 Aspects of Non-Engineering during Construction

Following aspects of non-engineering works are observed during construction.

(1) Concrete mixing

Concrete mixing is 'homemade' and made by hand based on experience. General mix-proportion of concrete at the site is 24 carts for fine aggregate (sand), 12 carts for coarse aggregate (gravel), 4 bags (45kg per bag) of cement, and some water for $1m^3$ concrete. It is noted that mix proportion of sand and gravel is opposite compared to engineering mixing due to workability, and volume of water which decides strength of concrete is not measured. AE additive agent is not used. Concrete strength is unknown at the time of mixing accordingly. Test pieces of cylinder are taken for the test of 28 day strength of concrete. Sizes of coarse aggregate seem to be too big considering small sizes of members (photos 3.9, 3.10, 3.11, 3.12).

(2) Fabrication of Hoop Re-bars

Hook of hoop re-bar is 90 degree and is not 135 degree that is required for seismic performance (photo 3.13, 3.14).

(3) Concrete foundations

The concrete of foundations is cast without perimeter framework. When mixing the soil into the concrete, it reduces the quality.

(4) Longitude of overlap of the bars

Short lap length of column re-bars is observed. This is by the lack of engineering coordination of re-bar arrangement and position of construction joint (photo 3.18).

(5) Concrete Cover

It is observed that the main column re-bars are uncovered and there is no concrete recovering , which reduces column strength and durability. This is caused by the lack of engineering coordination regarding the size of the hoops, the formwork and the coarse aggregate of concrete (photo 3.5.27).

(6) Re-bar Anchorage

Shortage of beam re-bar anchor to column is observed. Beam main re-bars stop at the outer face of formwork. The main re-bars of the beams hit the external face of the framework. This is caused by no-understanding of importance of re-bar anchorage. Un-proper re-bar arrangement at joint of beam and column is also observed. Appearance of cast concrete shows this (photo 3.28).

(7) Construction Joints

Un-proper horizontal joint of beam is observed. Horizontal construction joint of beam reduces strength of beam (photo 3.29).

(8) Removal of Form work

Early removal of beam bottom formwork is observed. Bottom formwork of beam is removed in one or two days only after concreting. This may cause deflection and cracks of beams. Longer curing is required subject to confirmation of concrete strength at the removal (photo 3.30).

(9) Others

Twist of columns is observed. This is caused by the twisted installation of column re-bars by the lack of surveying before casting concrete of foundation (photo 3.23). Height difference of column joints is observed. This causes height adjustment of column by casting additional concrete or level difference of beams and floors later (photo 3.24).

Photo 3.1 Site grading Work

Photo 3.2 Excavation Work

Photo 3.3 Column Work

Photo 3.4 Beam Work

Photo 3.5 Floor Work

Photo 3.6 Column Work

Photo 3.7　Roof Work

Photo 3.8　Brick Wall work

Photo 3.9　Concrete Mixing

Photo 3.10　Coarse Aggregate

Photo 3.11　Portland Cement (45kg/bag)

Photo 3.12　Concrete Test Cylinder

Photo 3.13 Fabrication of Re-bars(1)

Photo 3.14 Fabrication of Re-bars(2)

Photo 3.15 Fabrication of Re-bars(3)

Photo 3.16 Excavation for Foundation

Photo 3.17 Concrete Casting for Foundation

Photo 3.18 Short Column Re-bar

Photo 3.19　Short Column Form Work(1)　　**Photo 3.20　Short Column Form Work(2)**

Photo 3.21　Short Column Concreting　　**Photo 3.22　Short Column Concreted(1)**

Photo 3.23　Short Column Concreted(2)　　**Photo 3.24　Short Column Concreted(3)**

Photo 3.25 Long Column Concreting

Photo 3.26 Long Column Concreted

Photo 3.27 Floor Beam

Photo 3.28 Beam Re-bar Installation

Photo 3.29 Construction Joint at beam

Photo 3.30 Removal of Form Work

Photo 3.31 Long Column and Floor

Photo 3.32 Tabelone Floor and Concreting

Photo 3.33 Column Form Work (1)

Photo 3.34 Column Form Work (2)

Photo 3.35 Roof Beam Form Work

Photo 3.36 Roof Floor Work

Photo 3.37 Clay Brick Wall Work (1)

Photo 3.38 Clay Brick Wall Work(2)

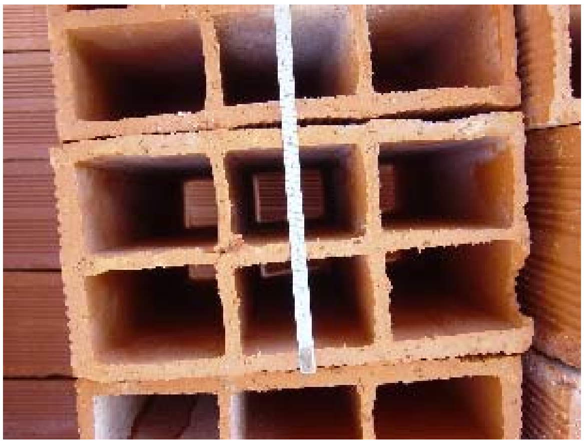


Photo 3.39 Hollow Clay Brick

Photo 3.40 Grade Beam

Photo 3.41 Concrete Block Wall Work(1)

Photo 3.42 Concrete Block Wall Work(2)

Photo 3.43　Retaining Wall　　**Photo 3.44　Completion of Models**

CHAPTER 4. MATERIAL TESTS

4. 1 General Information of Materials

Concrete: refer to chapter 3.2. (1) Concrete Mixing.

Reinforcing main steel bar: Grade A42 (fy (yield strength) = 4,200kg/cm^2), diameter 1/2"(Area=1.27cm^2).

Hoop and stirrup re-bars: no specific standard materials, and fy = 5,000kg/cm^2, diameter is 4mm.

Clay brick: no specific standard material, sizes are 10cmx20cmx30cm, ave.17pieces/m^2. Thickness of plate consisting hollow is 5~7mm (photo 2.3.31).

Concrete block: no specific standard material, sizes are 15cmx 20cmx40cm (photo 2.3.33).

Tabelone for floor: sizes are 6.5cmx20cmx80cm, and weight is 8kg/piece, thickness of floor concrete is ave.3.5cm, located on H-steel joist (weight 7kg/m).

Epoxy grout: used with drilling for the embedment of re-bar (3/8" Grade A36) to existing columns and beams for concrete block walls for Model 4.

4. 2 Material Test

Concrete cylinder test at 28 days is summarized in Figure S7-4.2.1. Average strength of concrete for beam/column is 58 kg/cm^2 only and is about 1/3 of normal engineering concrete. Water cement ratio is estimated approximately 110%, that is very high compared to not more than 65% of normal engineering concrete. Other test results including concrete are summarized in Table S7-4.2.1. Materials are tested by IMME of UCV.

Table S7-4.2.1 Material Tests (Concrete, Re-bar, Clay Brick, Concrete Block)

Concrete Test

Cylinder	max. stress (kg/cm^2, for full section)	
1	124	Foundations
2	113	
3	96	
4	97	
5	122	
6	121	
7	103	
8	101	
9	49	Columns over foundation to beam
10	53	
13	58	
14	68	
15	72	Beams
16	68	
17	37	Grade beam
18	39	
19	66	Grade beam model 1
20	57	
21	69	Floor
23	64	Columns model 1 -2
25	62	Beam roof model 1
26	66	Column model 3 - beam model 2
28	29	roof
29	133	roof
40	62	wall
41	40	wall

Reinforced bar

Diameter	yielding stress	max stress (Kg/cm^2)
3/8"	4729	6643
3/8	4761	6789
1/2	4532	6683
1/2	4532	6532

Diameter: 3.85 mm max load: 840 kgf max stress: 7216 kg/cm^2

Clay brick: max stress (kg/cm^2 for full section)

10 cms	23
10 cms	23
10cms	17
10 cms	21.8
10 cms	23

Clay brick sizes:

9.60 x 19.6 x 29.7cm weight 3.80 kg

9.60 x 19.9 x 29.7cm weight 3.80 kg

9.80 x 20.2 x 29.8cm --- 3.9 kg

Concrete block sizes:

14.3 x 19.8 x 39.0 weight 10.40 Kg

Concrete block strength (kg/cm^2, for full section)

15cms	19

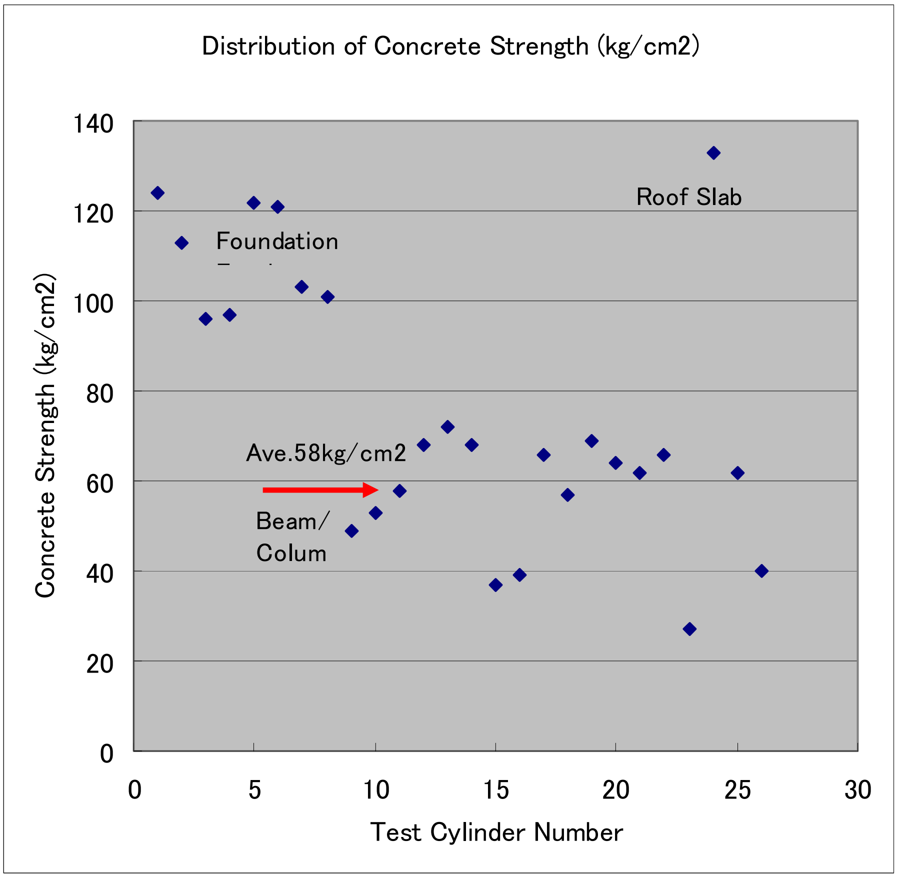


Figure S7-4.2.1 Distribution of Concrete Strength by Cylinder Test, Tested by IMME

CHAPTER 5. HORIZONTAL LOADING AND MEASUREMENT

5. 1 Horizontal Loading

Horizontal load is applied at the floor with slope direction. Horizontal load is applied statically by hydraulic jacks. 2 no synchronized hydraulic jacks with capacity of 50 ton each and with stroke of 50mm are used for loading of a model. Manual operation for pumping is used. Step of loading of $2kg/cm^2$ for hydraulic pump pressure is used for loading and this is converted to 500kg/step for hydraulic jacks according to the calibration test result. Re-setting of hydraulic jacks that has 50mm stroke only is planned when required.

Load cell for the measurement of loading is not used, and the loading after the maximum strength is not measured in this case. RC reaction wall is provided at the slope side to resist horizontal load by hydraulic jacks through steel frames. Steel frames have length of 2.85m, and are detailed for easy assembly and re-assembly. A steel loading beam is provided at the floor level, to transfer loads from hydraulic jacks to frames of a model. Sizes of reaction walls are 1.2mx3.0m for model 1 to 3, 1.2mx4.0m for model 4 (photos 3.43, 5.1~5.4, figure S7-2.2.5).

5. 2 Measurement

Horizontal deflection for models is measured by flex-meters (dial gauges) located at the floor level. Deflection at the roof level and ground level are also measured for reference. Total 8 locations are measured for horizontal deflection. Flex-meters have stroke length of 5cm or 2.5cm.
Loading and measurement is done by IMME of UCV (photo 5.5~5.6, Figure S7-2.2.10).

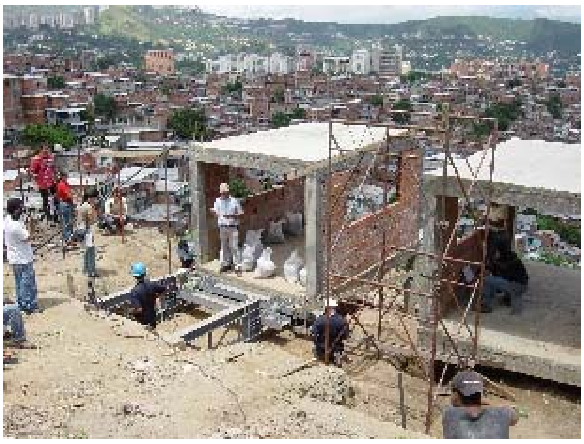


Photo 5.1 Overview

Photo 5.2 Steel Frame for Load Transfer

Photo 5.3 Hydraulic Jack

Photo 5.4 Hydraulic Pump

Photo 5.5 Measurement Equipment (1)

Photo 5.6 Measurement Equipment (2)

CHAPTER 6. RESULTS OF THE FIELD TEST

As stated in chapter 5, strength of model 1 and strength increase for reinforced models 2, 3 and 4 is evaluated mainly through the load deflection curve up to the maximum strength. Load deflection curve is not measured after the maximum strength by the reason of the limitation of measurement equipment, while general behavior is observed visually up to the horizontal deflection of 100mm~130mm. Photos are also taken for record at this final stage.

6. 1 Schedule of Test

Field test was done by following schedule;

26 August 2004	: Field test for Model-2
27 August	: Field test for Model-1
31 August	: Field test for Model-3
1 September	: Field test for Model-4

6. 2 Results

The load deflection curve up to the maximum strength for 4 models is shown in figure S7-6.2.5. The data of load and deflection of each model is shown in table S7-6.2.1 to S7-6.2.4. In this table, point 2 and 5 are the deflections at the floor, and average value is used in figure S7-6.2.1. Point 1 and 4 are the deflections at the roof, point 3 and 6 are the deflections at the ground at upper side, and point 7 and 8 are the deflections at the lower side of the slope.

Odd number point is the right side and even number point is the left side of the frame from the view of hydraulic jacks. The surface ground level at the time of testing is, 20cm to 30cm at short column position and 50cm to 60cm for long column position respectively, higher than those shown in figure S7-2.2.1 to S7-2.2.10, by the rainfall and other reason.

(1) Model-1

Failure mode of model 1 frame is column collapse mode and plastic hinges are provided at the top of columns. Floor beams are not damaged seriously. Elastic stiffness is 8.25t/cm, and yield strength is 8.75 ton. Maximum strength (max. load) is 10.25ton (photos 6.1~6.4). Deflection at yield strength is 10.6mm, and storey deflection is 1/170 (10.6/1,800) for short column and 1/226 (10.6/2,400) for long column respectively. Deflection at maximum strength is 16.4mm, and storey deflection is 1/110 (16.4/1,800) for short column and 1/207 (16.4/3,400) for long column respectively. Bending failure of columns is occurred at the beginning, and diagonal shear crack of short columns is also observed at mid-span at later stage (photo 6.2).

It is confirmed that the bottom of the short column is not damaged by the visual inspection after the excavation (photo 6.3).

Yield point is evaluated as the yield of short columns, and point of the maximum strength is evaluated as the yield of long columns. It is evaluated from the appearance of top of column at the final stage of the test of which horizontal deflection is approx.120mm, ductility with some extent is expected.

Axial stress of column by vertical load is 2,500kg/20.5cmx20.5cm=5.95kg/cm^2, and stress ratio is 5.95/58=0.10. Shear stress of short column at yield strength is estimated as 11.6kg/cm^2 (8,750x0.85/(2x0.8BD)), if 85% is supported by short columns. This stress level is high and is approx. 1/5 of compressive strength of concrete.

(2) Model-2

Failure mode of short columns is bending/shear mode at yield strength and shear failure occurs at final stage of test. Failure mode of long columns is bending failure mode, while shear diagonal crack is also observed (photos 6.5~6.10). Yield strength is 10.25 ton, which is 1.17 times of that of model 1. Maximum strength is 14.75 ton, which is 1.44 times of that of model 1. Initial stiffness is increased to 25.0ton/cm, which is 3.0 times of that of model 1. Deflection at yield strength is 4.1mm, and storey deflection is 1/439 (4.1/1,800) for short column and 1/829 (4.1/3,400) for long column respectively. Deflection at maximum strength is 17.6mm, and storey deflection is 1/102 (17.6/1,800) for short column and 1/193 (17.6/3,400) for long column respectively. Deflection at the ground surface (almost same to grade beam) at yield and maximum strength is 2.4mm (lower ground level) and 1.1mm (lower ground level) respectively.

Grade beams are provided so as to maintain ratio of column clear length/column depth is 3.0 to prevent shear failure which is brittle failure. It is assessed that shear failure of short columns occurre by the reason of unexpected low strength of concrete which is average 58 kg/cm^2. It is confirmed that the short column under grade beam is not damaged by the visual inspection after the excavation (photo 6.10). Cost impact of strengthening is 5 to 7% of the total cost of building.

(3) Model-3

Load deflection curve is similar to that of Model 2. Separation of clay hollow brick walls from columns and beams appears from the beginning of loading and combined effect with frames is not expected. Maximum strength is 16.75 ton, which is 1.13 times only of that of model 2, at the deflection of 17.6mm. It is found that clay brick walls have no contribution to stiffness and strength compared to those of model 2. Stiffness and strength of clay brick walls is very low for structural use and for structural reinforcement (photo 6.11~6.15). Cost impact is 10 % of the total cost of building.

(4) Model-4

Separation of hollow concrete block walls without re-bars from columns and beams starts at early stage of load 6~7ton. Yield strength appears at the load of 13.75 ton and deflection of 2.7mm, by the separation of hollow concrete blocks with re-bars from columns (photo 6.16~6.21). The maximum strength 15.25 ton is observed at deflection 12.8mm. Initial stiffness is increased by providing hollow concrete blocks, while strength is almost similar to those of Model 2 and 3. Horizontal deflection is increased after the max. strength and is provided more than 100mm as the final stage of loading. It is found that the strength of hollow concrete blocks is low for structural use and for seismic reinforcement. Concrete hollow block wall without re-bars is separated from column/beam at early stage, while wall with re-bars is not separated until lap joint of horizontal re-bars is broken. Strength of concrete block is low, and lower than that of mortar (photo 3.41, 6.21).

Cost impact is 15% of the total cost of a building.

6. 3 Summary

- Strength of frames without reinforcement is 9 to 10 ton for 4 columns.
- Providing grade beams is effective for seismic strengthening and increases the strength by approx.40%, and need to pay attention clear length of column, to prevent shear failure considering strength of concrete. Cost impact is 5%~7 %.
- Clay hollow brick wall is not effective for seismic strengthening. Cost impact is 10%.
- Concrete block wall will be effective, if concrete strength of block is increased, together with the use of re-bars for seismic reinforcement.
    - Drilling and epoxy grouting method is suggested for re-bar anchorage to existing column/beam.
    - Cost impact will be 15%.
- Video report is used to improve awareness to the public

- Other seismic reinforcement methods (practical and economical method) are also suggested to investigate in future.

- This kind of full scale field test is done for the first time in Caracas.

It is recommended strongly to continue and develop seismic assessment and reinforcement through model tests and analyses for Barrio houses in future.

Table S7-6.2.1 Model 1 Load and Deflection

Manometer	Pressure	Loading (t)			Reading #5 (mm)	Reading #2 (mm)	Deflection #5 (mm)	Deflection#2 (mm)	Average Deflection(mn
					19.74	19.36			
3	0	0	19.74	19.36	19.74	19.36	0	0	0
6	3	0.75	19.74	19.36	19.65	19.3	0.09	0.06	0.075
8	5	1.25	19.74	19.36	19.26	19.14	0.48	0.22	0.35
10	7	1.75	19.74	19.36	18.84	18.78	0.9	0.58	0.74
12	9	2.25	19.74	19.36	18.98	18.15	0.76	1.21	0.985
14	11	2.75	19.74	19.36	17.11	17.4	2.63	1.96	2.295
16	13	3.25	19.74	19.36	15.74	16.26	4	3.1	3.55
18	15	3.75	19.74	19.36	14.48	15.23	5.26	4.13	4.695
20	17	4.25	19.74	19.36	13.23	14.14	6.51	5.22	5.865
22	19	4.75	19.74	19.36	12.28	13.24	7.46	6.12	6.79
24	21	5.25	19.74	19.36	11.75	12.58	7.99	6.78	7.385
26	23	5.75	19.74	19.36	11.44	11.94	8.3	7.42	7.86
28	25	6.25	19.74	19.36	11.2	11.25	8.54	8.11	8.325
30	27	6.75	19.74	19.36	11.02	10.54	8.72	8.82	8.77
32	29	7.25	19.74	19.36	10.85	9.88	8.89	9.48	9.185
34	31	7.75	19.74	19.36	10.69	9.16	9.05	10.2	9.625
36	33	8.25	19.74	19.36	10.49	8.39	9.25	10.97	10.11
38	35	8.75	19.74	19.36	10.3	7.54	9.44	11.82	10.63
40	37	9.25	19.74	19.36	9.72	5.7	10.02	13.66	11.84
42	39	9.75	19.74	19.36	8.49	2.75	11.25	16.61	13.93
44	41	10.25	19.74	19.36	6.43	-0.1	13.31	19.46	16.385
46	43	10.75	20.39	20.78					

Reading #6 (mm)	Reading #3 (mm)	Deflection#6 (mm)	Deflection#3 (mm)
20.8	20.2		
20.8	20.2	0	0
20.72	20.19	0.08	0.01
20.52	20.05	0.28	0.15
20.21	19.81	0.59	0.39
19.6	19.35	1.2	0.85
18.96	18.78	1.84	1.42
17.98	17.84	2.82	2.36
17.05	16.88	3.75	3.32
16.14	16.07	4.66	4.13
15.44	15.28	5.36	4.92
15.05	14.69	5.75	5.51
14.81	14.14	5.99	6.06
14.62	13.45	6.18	6.75
14.52	12.78	6.28	7.42
14.37	12.09	6.43	8.11
14.28	11.41	6.52	8.79
14.17	10.66	6.63	9.54
14.04	9.95	6.76	10.25
13.55	8.58	7.25	11.62
12.61	6.7	8.19	13.5
11.05	4.09	9.75	16.11

Reading #7 (mm)	Reading #8 (mm)	Deflection#7 (mm)	Deflection#8 (mm)
0.39	0.95		
0.39	0.95	0	0
0.42	0.96	0.03	0.01
0.47	0.99	0.08	0.04
0.56	1.05	0.17	0.1
0.77	1.18	0.38	0.23
1.04	1.36	0.65	0.41
1.49	1.72	1.1	0.77
1.95	2.09	1.56	1.14
2.46	2.49	2.07	1.54
2.84	2.85	2.45	1.9
3.05	3.11	2.66	2.16
3.23	3.37	2.84	2.42
3.41	3.67	3.02	2.72
3.55	3.97	3.16	3.02
3.68	4.28	3.29	3.33
3.82	4.56	3.43	3.61
4.04	4.87	3.65	3.92
4.24	5.22	3.85	4.27
4.59	5.97	4.2	5.02
5.19	6.91	4.8	5.96
5.98	8.15	5.59	7.2

Reading #1 (mm)	Reading #4 (mm)	Deflection#1 (mm)	Deflection#4 (mm)
53.6	48.39		
53.6	48.39	0	0
53.6	48.29	0	0.1
53.6	47.95	0	0.44
53.12	47.48	0.48	0.91
52.35	46.58	1.25	1.81
52.35	45.6	1.25	2.79
50.12	44.22	3.48	4.17
49.35	42.92	4.25	5.47
47.82	41.65	5.78	6.74
46.78	40.67	6.82	7.72
46	40.15	7.6	8.24
45.4	39.82	8.2	8.57
44.5	39.6	9.1	8.79
43.7	39.4	9.9	8.99
42.86	39.27	10.74	9.12
41.85	39.12	11.75	9.27
41.1	38.94	12.5	9.45
40.12	38.73	13.48	9.66
38.27	37.92	15.33	10.47
35.73	36.71	17.87	11.68
31.42	34.72	22.18	13.67

Table S7-6.2.2 Model 2 Load and Deflection

Manometer	Pressure	Loading (t)	Reading #5 (mm)	Reading #2 (mm)	Deflection #5 (mm)	Deflection#2 (mm)	Average Deflection(mr	Reading #6 (mm)	Reading #3 (mm)	Deflection#6 (mm)	Deflection#3 (mm)
8	5	1.25	18.21	17.26				17.01	19.25		
10	7	1.75	18.06	17.25	0.15	0.01	0.08	17.01	19	0	0.25
12	9	2.25	18.06	17.19	0.15	0.07	0.11	16.98	19	0.03	0.25
14	11	2.75	17.87	16.98	0.34	0.28	0.31	16.85	18.93	0.16	0.32
16	13	3.25	17.7	16.84	0.51	0.42	0.465	16.77	18.83	0.24	0.42
18	15	3.75	17.44	16.55	0.77	0.71	0.74	16.61	18.69	0.4	0.56
20	17	4.25	17.16	16.3	1.05	0.96	1.005	16.45	18.51	0.56	0.74
22	19	4.75	16.81	15.93	1.4	1.33	1.365	16.27	18.31	0.74	0.94
24	21	5.25	16.5	15.65	1.71	1.61	1.66	16.08	18.17	0.93	1.08
26	23	5.75	16.18	15.24	2.03	2.02	2.025	15.88	17.89	1.13	1.36
28	25	6.25	15.8	14.89	2.41	2.37	2.39	15.62	17.66	1.39	1.59
30	27	6.75	15.41	14.4	2.8	2.86	2.83	15.37	17.33	1.64	1.92
32	29	7.25	15.24	14.08	2.97	3.18	3.075	15.29	17.15	1.72	2.1
34	31	7.75	15.08	13.83	3.13	3.43	3.28	15.23	17	1.78	2.25
36	33	8.25	15.02	13.65	3.19	3.61	3.4	15.21	16.89	1.8	2.36
38	35	8.75	14.91	13.35	3.3	3.91	3.605	15.17	16.66	1.84	2.59
40	37	9.25	14.81	13.11	3.4	4.15	3.775	15.14	16.54	1.87	2.71
42	39	9.75	14.71	12.83	3.5	4.43	3.965	15.11	16.38	1.9	2.87
44	41	10.25	14.59	12.6	3.62	4.66	4.14	15.07	16.36	1.94	2.89
46	43	10.75	14.39	12.04	3.82	5.22	4.52	15.02	15.91	1.99	3.34
48	45	11.25	13.83	10.28	4.38	6.98	5.68	14.72	14.7	2.29	4.55
50	47	11.75	13.18	8.22	5.03	9.04	7.035	14.45	13.69	2.56	5.56
52	49	12.25	12.99	5.92	5.22	11.34	8.28	14.29	13.04	2.72	6.21
54	51	12.75	12.57	5.5	5.64	11.76	8.7	14.08	12.43	2.93	6.82
56	53	13.25	12.34	4.15	5.87	13.11	9.49	13.88	11.65	3.13	7.6
58	55	13.75	11.92	2.74	6.29	14.52	10.405	13.59	10.95	3.42	8.3
60	57	14.25	11.46	0.5	6.75	16.76	11.755	13.06	9.33	3.95	9.92
62	59	14.75	11.46	15.44	6.75	17.81	12.28	10.99		6.02	
62	59	14.75	9.59	5.99	8.62	27.26	17.94				
64	61	15.25	7.8		10.41						
66	63	15.75									

Reading #7 (mm)	Reading #8 (mm)	Deflection#7 (mm)	Deflection#8 (mm)	Reading #1 (mm)	Reading #4 (mm)	Deflection#1 (mm)	Deflection#4 (mm)
3.2	1.9			47.78	49.49		
3.2	1.9	0	0	47.78	49.49	0	0
3.2	1.92	0	0.02	47.78	49.38	0	0.11
3.22	1.98	0.02	0.08	47.63	49.12	0.15	0.37
3.26	2.01	0.06	0.11	47.45	49	0.33	0.49
3.3	2.09	0.1	0.19	47.15	48.66	0.63	0.83
3.35	2.14	0.15	0.24	46.88	48.33	0.9	1.16
3.42	2.21	0.22	0.31	46.52	48.02	1.26	1.47
3.51	2.29	0.31	0.39	46.26	47.68	1.52	1.81
3.58	2.39	0.38	0.49	45.8	47.25	1.98	2.24
3.67	2.51	0.47	0.61	45.37	46.8	2.41	2.69
3.76	2.64	0.56	0.74	44.81	46.34	2.97	3.15
3.83	2.72	0.63	0.82	44.45	46.15	3.33	3.34
3.86	2.81	0.66	0.91	44.15	46.02	3.63	3.47
3.89	2.85	0.69	0.95	43.98	45.97	3.8	3.52
3.91	2.92	0.71	1.02	43.62	45.93	4.16	3.56
3.94	3.01	0.74	1.11	43.28	45.85	4.5	3.64
3.99	3.1	0.79	1.2	42.87	45.81	4.91	3.68
4.03	3.19	0.83	1.29	42.4	45.69	5.38	3.8
4.1	3.37	0.9	1.47	41.63	45.55	6.15	3.94
4.28	3.72	1.08	1.82	39.7	44.89	8.08	4.6
4.46	4.04	1.26	2.14	38.05	44.5	9.73	4.99
4.56	4.32	1.36	2.42	36.87	44.21	10.91	5.28
4.67	4.55	1.47	2.65	35.49	43.82	12.29	5.67
4.81	4.91	1.61	3.01	34.05	43.5	13.73	5.99
4.95	5.24	1.75	3.34	32.57	43.03	15.21	6.46
5.17	5.86	1.97	3.96	30.29	42.37	17.49	7.12
5.49	7.1	2.29	5.2				

Table S7-6.2.3 Model 3 Load Deflection

Manometer	Pressure	Loading (t)	Deflection #5 (mm)	Deflection#2 (mm)	Average Deflection(mr	Deflection#6 (mm)	Deflection#3 (mm)	Deflection#7 (mm)	Deflection#8 (mm)	Deflection#1 (mm)	Deflection#4 (mm)
3	0	0	0	0	0	0	0	0	0	0	0
6	3	0.75	0	0	0	0	0	0.01	0.02	0	0.17
8	5	1.25	0.05	0.06	0.055	0.01	0	0.01	0.02	0.07	0.17
10	7	1.75	0.13	0.15	0.14	0.07	0.08	0.03	0.03	0.16	0.17
12	9	2.25	0.22	0.28	0.25	0.1	0.11	0.04	0.06	0.29	0.16
14	11	2.75	0.29	0.34	0.315	0.11	0.13	0.06	0.06	0.39	0.27
16	13	3.25	0.45	0.51	0.48	0.18	0.19	0.09	0.1	0.58	0.41
18	15	3.75	0.61	0.66	0.635	0.25	0.24	0.12	0.13	0.73	0.59
20	17	4.25	0.79	0.84	0.815	0.28	0.29	0.15	0.18	0.95	1.02
22	19	4.75	0.99	1.08	1.035	0.3	0.37	0.19	0.22	1.25	1.38
24	21	5.25	1.24	1.34	1.29	0.37	0.44	0.22	0.28	1.53	1.52
26	23	5.75	1.61	1.71	1.66	0.43	0.53	0.28	0.36	1.91	2.17
28	25	6.25	2.22	2.36	2.29	0.5	0.64	0.38	0.49	2.65	2.65
30	27	6.75	2.82	2.99	2.905	0.58	0.77	0.48	0.61	3.32	3.27
32	29	7.25	3.24	3.48	3.36	0.65	0.82	0.57	0.72	3.79	3.77
34	31	7.75	3.62	3.97	3.795	0.71	0.95	0.64	0.84	4.35	4.09
36	33	8.25	3.94	4.52	4.23	0.75	1.07	0.69	0.96	4.9	4.47
38	35	8.75	4.08	4.82	4.45	0.77	1.15	0.74	1.04	5.2	4.72
40	37	9.25	4.19	5.17	4.68	0.79	1.23	0.78	1.11	5.59	4.73
42	39	9.75	4.28	5.47	4.875	0.8	1.31	0.81	1.18	5.87	4.87
44	41	10.25	4.36	5.94	5.15	0.81	1.41	0.83	1.32	6.4	5.27
46	43	10.75	4.645	6.43	5.5375	0.875	1.58	0.9	1.455	7.065	5.46
48	45	11.25	4.81	6.845	5.8275	0.91	1.73	0.945	1.57	7.535	5.705
50	47	11.75	5.28	7.43	6.355	1.07	2.04	1.07	1.75	8.2	6.3
52	49	12.25	5.465	7.875	6.67	1.115	2.265	1.125	1.855	8.38	6.64
54	51	12.75	5.635	8.505	7.07	1.16	2.585	1.17	1.995	9.36	6.765
56	53	13.25	5.95	9.31	7.63	1.22	3	1.24	2.15	10.24	7.36
58	55	13.75	6.07	10.09	8.08	1.23	3.35	1.28	2.27	10.99	7.37
60	57	14.25	6.3	12.17	9.235	1.24	4.33	1.34	2.61	12.91	7.94
62	59	14.75	6.38	14	10.19	1.24	4.95	1.4	2.87	14.47	7.94
64	61	15.25	6.62	16.07	11.345	1.24	5.475	1.485	3.115	16.665	8.365
66	63	15.75	7.81	19.52	13.665	1.24	6.08	1.65	3.21	20.27	9.76
68	65	16.25	8.82	22.42	15.62	1.17	6.95	1.81	3.43	23.39	10.34
70	67	16.75	9.53	25.595	17.5625	1.155	8.01	1.995	3.55	25.87	10.57
72	69	17.25	14.14	37.23	25.685	1.35	9.18	2.2	3.49	38.71	14.6
72.5	69.5	17.375		50	25					50	

Table S7-6.2.4 Model 4 Load Deflection

Manometer	Pressure	Loading (t)	Deflection #5 (mm)	Deflection#2 (mm)	Average Deflection(mr	Deflection#6 (mm)	Deflection#3 (mm)	Deflection#7 (mm)	Deflection#8 (mm)	Deflection#1 (mm)	Deflection#4 (mm)
3.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6.00	3.00	0.75	0.02	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.23
8.00	5.00	1.25	0.07	0.08	0.07	0.05	0.05	0.00	0.00	0.00	0.26
10.00	7.00	1.75	0.12	0.12	0.12	0.06	0.07	0.00	0.00	0.26	0.33
12.00	9.00	2.25	0.17	0.16	0.17	0.09	0.09	0.00	0.00	0.26	0.41
14.00	11.00	2.75	0.24	0.21	0.23	0.13	0.13	0.00	0.01	0.31	0.49
16.00	13.00	3.25	0.30	0.25	0.28	0.16	0.16	0.00	0.02	0.34	0.56
18.00	15.00	3.75	0.42	0.32	0.37	0.22	0.20	0.00	0.04	0.44	0.76
20.00	17.00	4.25	0.44	0.41	0.43	0.31	0.26	0.00	0.05	0.59	0.95
22.00	19.00	4.75	0.64	0.46	0.55	0.35	0.29	0.00	0.07	0.64	1.00
24.00	21.00	5.25	0.67	0.53	0.60	0.39	0.33	0.00	0.07	0.64	1.11
26.00	23.00	5.75	0.91	0.62	0.76	0.43	0.39	0.00	0.09	0.66	1.21
28.00	25.00	6.25	1.07	0.71	0.89	0.63	0.46	0.00	0.10	0.66	1.38
30.00	27.00	6.75	1.25	0.83	1.04	0.75	0.54	0.00	0.13	0.78	1.62
32.00	29.00	7.25	1.45	0.96	1.21	0.86	0.63	0.00	0.16	1.15	1.87
34.00	31.00	7.75	1.61	1.09	1.35	0.95	0.71	0.03	0.18	1.16	2.02
36.00	33.00	8.25	1.82	1.27	1.55	1.06	0.83	0.07	0.23	1.33	2.23
38.00	35.00	8.75	1.94	1.39	1.67	1.13	0.92	0.09	0.25	1.52	2.42
40.00	37.00	9.25	2.07	1.51	1.79	1.20	1.00	0.12	0.27	1.58	2.61
42.00	39.00	9.75	2.19	1.66	1.93	1.26	1.11	0.14	0.30	1.99	2.88
44.00	41.00	10.25	2.26	1.78	2.02	1.31	1.18	0.15	0.32	2.32	2.95
46.00	43.00	10.75	2.32	1.88	2.10	1.34	1.27	0.17	0.35	2.32	2.95
48.00	45.00	11.25	2.38	2.01	2.20	1.37	1.35	0.20	0.38	2.41	2.99
50.00	47.00	11.75	2.45	2.22	2.34	1.42	1.48	0.20	0.43	2.54	3.08
52.00	49.00	12.25	2.49	2.37	2.43	1.43	1.58	0.21	0.46	2.56	3.16
54.00	51.00	12.75	2.54	2.59	2.57	1.48	1.70	0.21	0.53	2.76	3.33
56.00	53.00	13.25	2.59	2.74	2.67	1.50	1.80	0.22	0.58	3.27	3.41
58.00	55.00	13.75	2.62	2.89	2.76	1.53	1.90	0.22	0.63	3.52	3.51
60.00	57.00	14.25	4.42	3.17	3.79	2.41	2.41	0.51	0.90	4.52	5.47
62.00	59.00	14.75	6.74	4.30	5.52	3.81	3.04	0.88	1.22	6.26	7.61
64.00	61.00	15.25	16.73	6.68	11.71	8.98	4.79	2.12	2.10	9.38	18.52
66.00	63.00	15.75	25.63		12.82	13.36					25.79
68.00	65.00	16.25									
70.00	67.00	16.75									
72.00	69.00	17.25									
72.50	69.50	17.38									

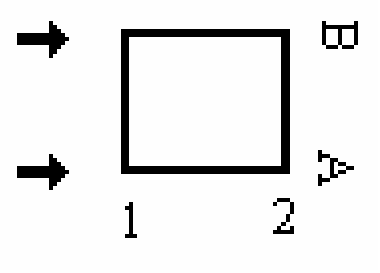


Figure S7-6.2.1　Plan of the Models

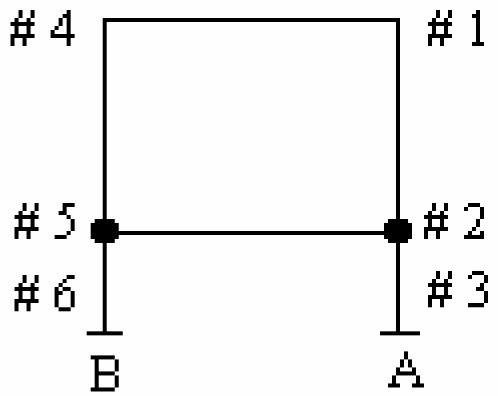


Figure S7-6.2.2　Façade of Models

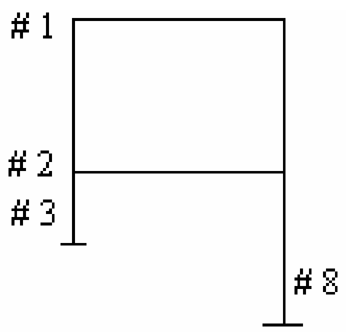


Figure S7-6.2.3　Side View A

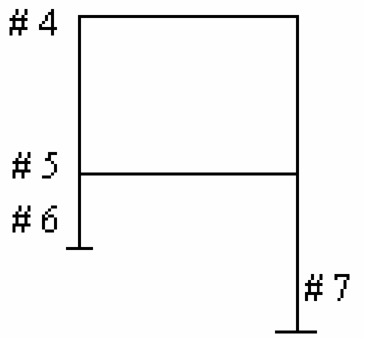


Figure S7-6.2.4　Side View B

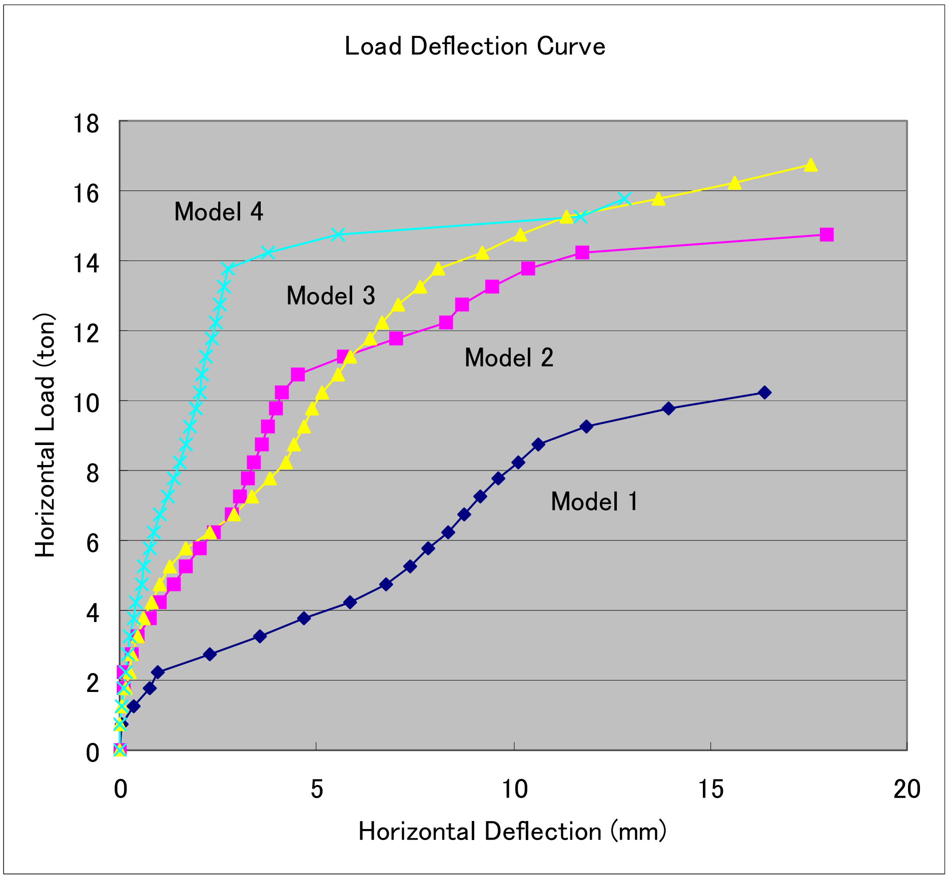


Figure S7-6.2.5　Load Deflection Curve

Photo 6.1 Model 1-Short Column Failure (1)

Photo 6.2 Model 1-Short Column Failure(2)

Photo 6.3 Model 1-Short Column Failure (3) **Photo 6.4 Model 1-Long Column Failure**

Photo 6.5 Model 2 **Photo 6.6 Model 2-Shear Failure of Short Column (1)**

Photo 6.7 Model 2-Shear Failure of Short Column (2)

Photo 6.8 Model 2-Shear Crack of Short Column (3)

Photo 6.9 Model 2-Long Column Failure

Photo 6.10 Model 2-Short Column under Grade Beam

Photo 6.11 Model 3

Photo 6.12 Model 3-Diagonal Shear Crack of Short Column

Photo 6.13 Shear Failure of Column and Clay Brick Wall (1)

Photo 6.14 Shear Failure of Column and Clay Brick Wall (2)

Photo 6.15 Separation of Wall and Frame

Photo 6.16 Model 4

Photo 6.17 Separation of Concrete Block Wall without Re-bars from Frame

Photo 6.18 Failure of Column and Concrete Block Wall

Photo 6.19　Shear Failure of Column and Concrete Block wall with Re-bars (1)

Photo 6.20　Shear Failure of Column and Concrete Block wall with Re-bars (2)

Photo 6.21　Failure of Concrete Block Wall with Re-bars

Photo 6.22　Demolition of Models

APPENDIXA1

Elastic and Strength Analysis for Model 1

Elastic analysis considering soil reaction at ground for columns and foundation, and strength evaluation by simple plastic hinge method was done for Model 1. This result of load deflection curve by the analysis was compared to the result of Model 1.

A1.1. Conditions of Elastic Analysis

1. Member size: column 20.5x20.5cm, beam 20.5x(20+10)cm (stiffness is increased by Φ1.5 for floor), foundation 100x100x20cm

2. Young's modulus: column & beam 0.5x2.1x10-5kg/cm^2 (reduced by concrete strength 60kg/cm^2)

3. Moment of inertia of column/beam section is increased for 1.5 times for area of main re-bar

4. Pin support at base of foundation to support vertical load

5. Spring constant for ground soil: horizontal ground reaction coefficient kh=6.0kg/cm^3
   (N value=10 equivalent, assumed) for column and foundation under ground surface
   by following formula, and is converted to unit of kg/cm^3;
   kh=0.08E0(B/10)-3/4 (N/mm3), where E0 is ground deflection coefficient and estimated
   E0=0.7N,
   B is column size

6. Axial load for each column: 2.45 ton, foundation; 0.5ton

7. Horizontal load for floor of each column: 2.25ton (referenced from the test result)

A1.2. Strength Analysis

Horizontal strengths of short and long columns are calculated simply using bending strength at the top of columns and moment distribution of elastic analysis.

(**Note**) Plastic hinges at top of columns are observed at the test of model 1, while it is not clear at the bottom portion whether plastic conditions are occurred at the ground soil or foundation footing. Columns at the bottom are not damaged by visual inspection after excavation (photo 6.3).

A1.3. Load Deflection Curve by Analysis

Each elastic stiffness and strength of short and long columns are combined together, and load deflection curve of tri-linear for model 1 is given as shown in figure A1.4.

The strength by this analysis is 9.6 ton and is 9% lower than that of test result. Possible reason of this difference is that the actual ground level is 50cm~60cm higher than design level at lower side of slope.

The strength of long columns is estimated lower by the analysis accordingly.

Attachment: Figure A1.1 Analysis Model-Model 1
Figure A1.2 Bending Moment Diagram (tm)
Figure A1.3 Displacement Diagram (cm)
Figure A1.4 Load Deflection Curve

Appendix A2

Assessment of Seismic Capacity for Existing Barrio Houses on Slope, Caracas

Seismic assessment of Barrio houses for 1 to 5 stories on slope are shown in Appendix A2. Response spectrum and base shear coefficient are estimated using Venezuela seismic code 1982, and the estimation of heavily damaged Barrio houses is shown.

(Note) Above assessment is done with respect to main frame only as a part of Disaster Prevention Plan of Caracas, and for future planning only and is not applied directly for a individual house. Assessment of a individual house shall be studied and be investigated based on a characteristics of each house.

A2.1.Conditions and Assumptions;

(1) Frame and member sizes

- Span of columns; 3.8mx2.8m (center to center of Column) and a frame of 2 columns.

- Member sizes; column 20cmx20cm, beam 20cmx30cm(includes floor slab 10cm), and same sizes for every floor(this is general understanding for Barrio houses).

(2) Used Materials

- Floor; Tabelone floor, concrete 3.4cm with wire mesh, and steel joist total 10cm thickness

- Wall; clay hollow brick wall 10cm and internal finish mortar 1.5cm

- Roof; Metal sheet with steel joist

(3) Weight of unit area

- Unit weight of floor per area including live load 60kg/cm^2 for calculation; 600kg/m^2, for roof; 200kg/m^2

- 1 to 5 storey house on slope (5 types of storey) is assessed for seismic capacity.

(4) Frame Capacity (horizontal strength) and Material Strength

- Frame capacity of 2 columns on slope is evaluated within the range of 4 to 5 ton.

- Frame capacity of 2 columns on typical floor is assumed as approximately 3 ton (beam collapse mode) to 4 ton (column collapse mode). Concrete strength; 60kg/cm^2

- Main re-bar; total 4 no dia.1/2" (A=1.27cm^2), fy =4700kg/cm^2 for columns and beams.

(5) Maximum Ground Acceleration

- 1967 earthquake is estimated as m.g.a A=0.15g, that is half of 1812 earthquake estimated as A=0.30g.

(6) Response Spectrum

- Response spectrum of Venezuela Seismic Code 1982 is used, and maximum ground acceleration

- Ao=0.30g is used, and this is estimated to be the same size to that of 1812 earthquake.

(7) Ductility of Frames

- Ductility factor is assumed and is decreased based on ratio of axial load of column/axial yield strength. Ductility Factor of not more than 3 is assumed.

(8) Miscellaneous

- Building period is estimated as T=0.02h (total height), instead of T=0.061h3/4 of Code 1982.

- Distribution of seismic shear force at each floor is calculated using modified form of Code 1982.

Attachment: Figure A2.1 Response Spectrum and Base Shear Coefficient, Code 1982

Figure A2.2 Response Spectrum and Base Shear Coefficient, Code 2001(reference)

Table A2.1 Seismic Assessment of Barrio Houses (1)

Table A2.2 Seismic Assessment of Barrio Houses (2)

Table A2.3 Estimation of Heavily Damaged Barrio Houses

Attachment: Tables, Figures and Photos

Photo 1.1　Barrio houses on a hill (1)

Photo 1.2　Barrio Houses on a hill (2)

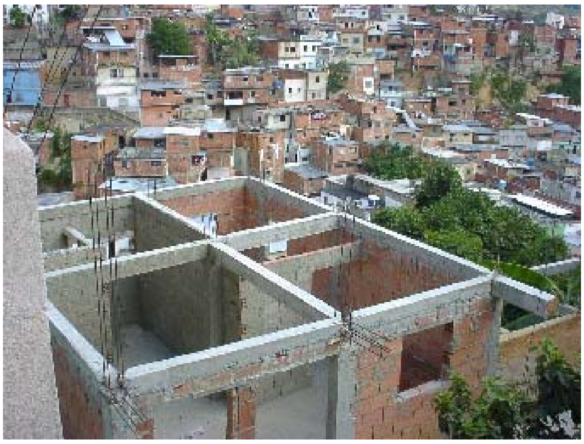


Photo 1.3　A Barrio house under construction

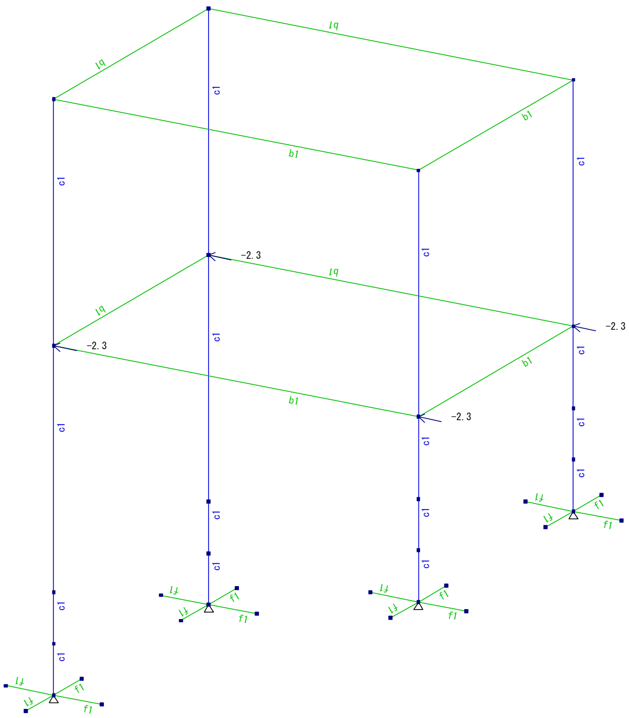
Photo 2.1　A Barrio House on a Slope(1)

Photo 2.2　A Barrio House on a Slope(2)

Appendix A1

Analysis of Stiffness and Strength for Model 1

1. Elastic Analysis and Results – Model 1



FigureA1.1　Analysis Model

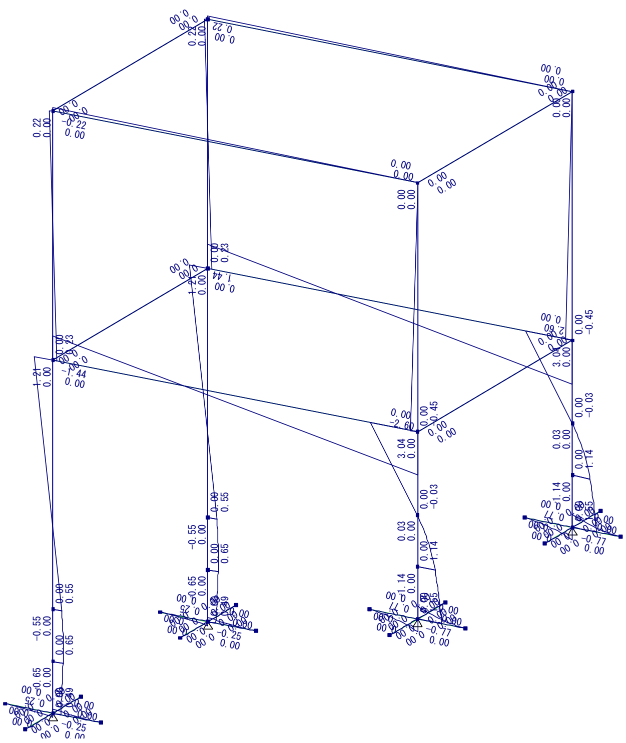


Figure A1.2
Bending Moment (tm)

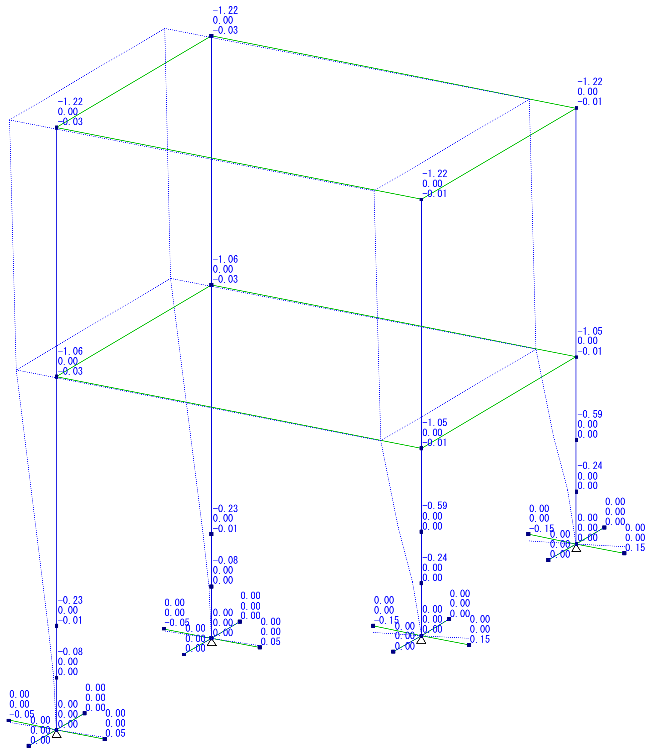


Figure A1.3
Displacement (cm)

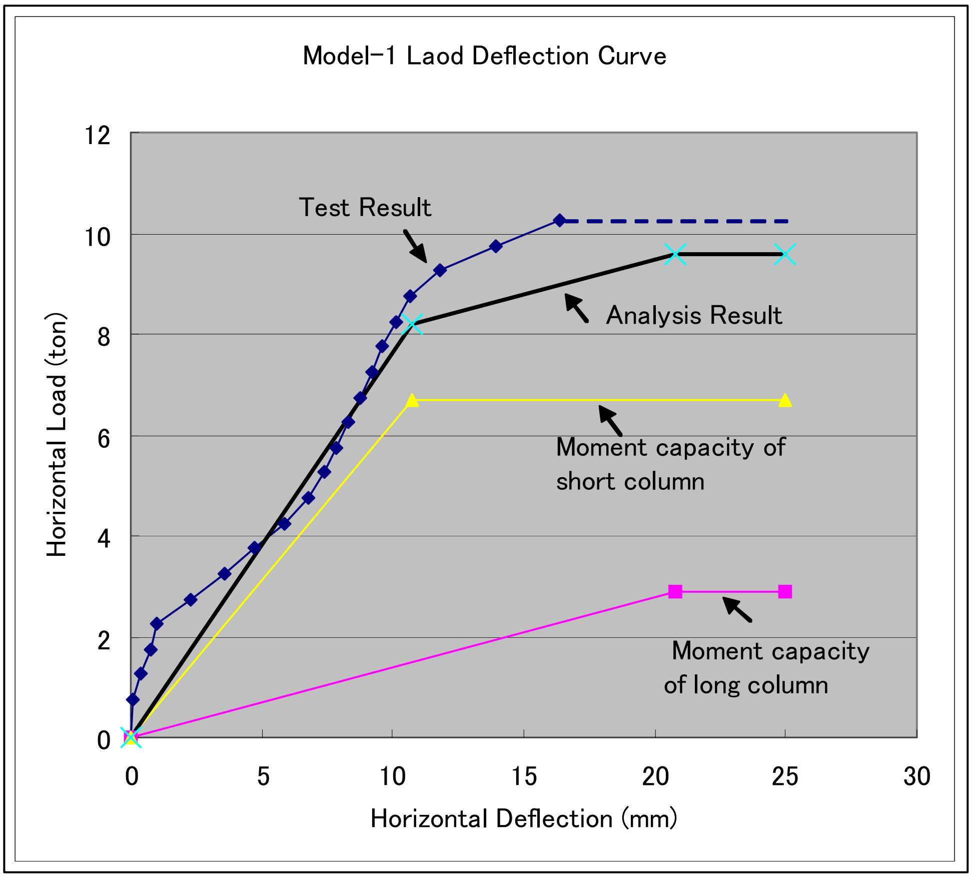


Figure A1.4　Analysis Result of Load Deflection Curve for Model 1

Appendix A2

Assessment of Seismic Capacity for Existing Barrio Houses on Slope, Caracas (PRELIMINARY)

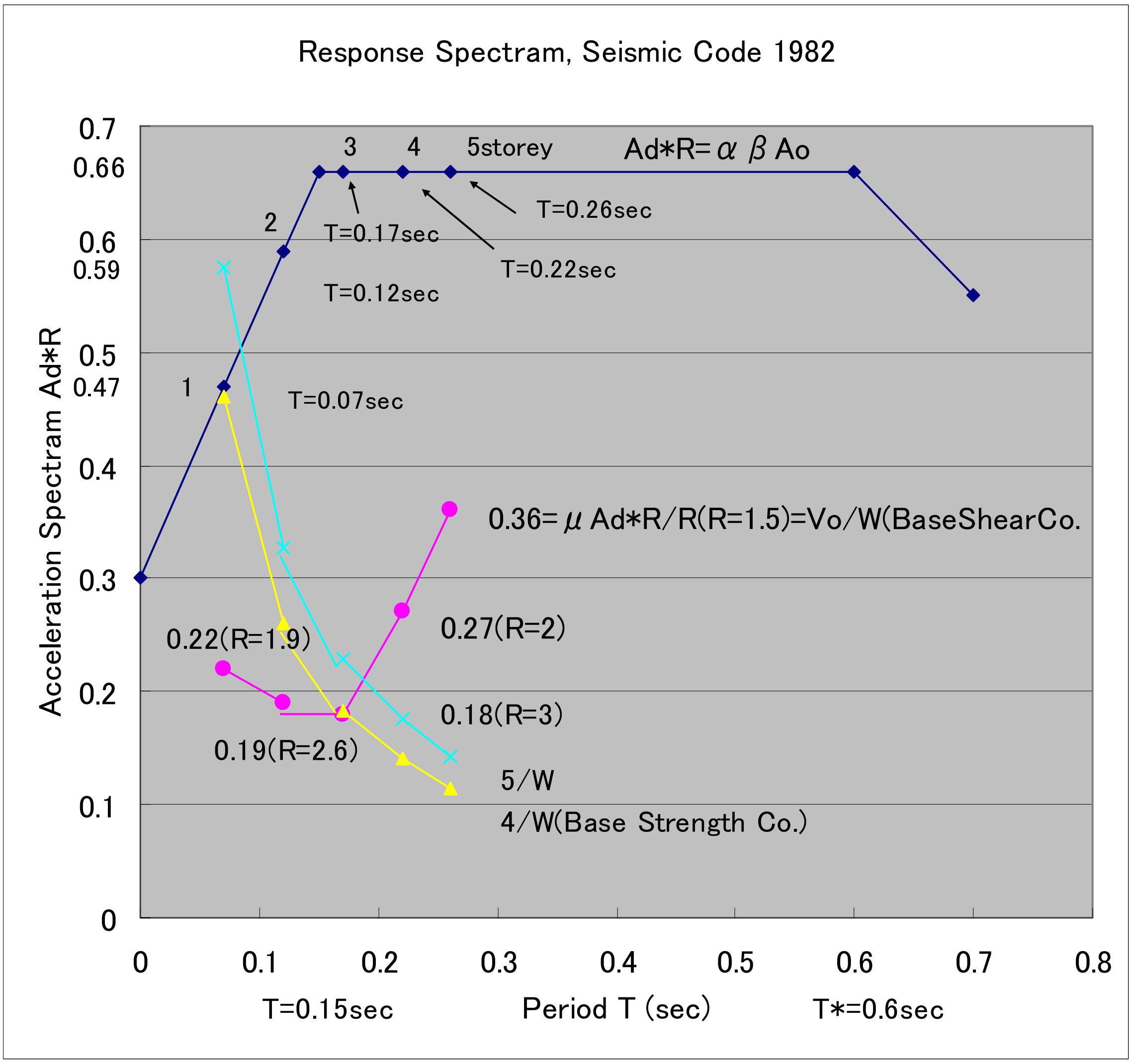


Figure A2.1 Response Spectrum and Base Shear Coefficient by Code 1982

Where: Ad (Ordinate of the design spectrum)

Ao = 0.30g (maximum horizontal ground acceleration), Zone 4

β = 2.2 (average magnification factor), T* =0.6 sec, Soil Profile S2

α = 1.0 (use coefficient)

R = 3 to 1.5 (response reduction factor)

D = 3 to 1.5 (ductility factor)

μ (factor related to no of storey)

W (total weight of the building)

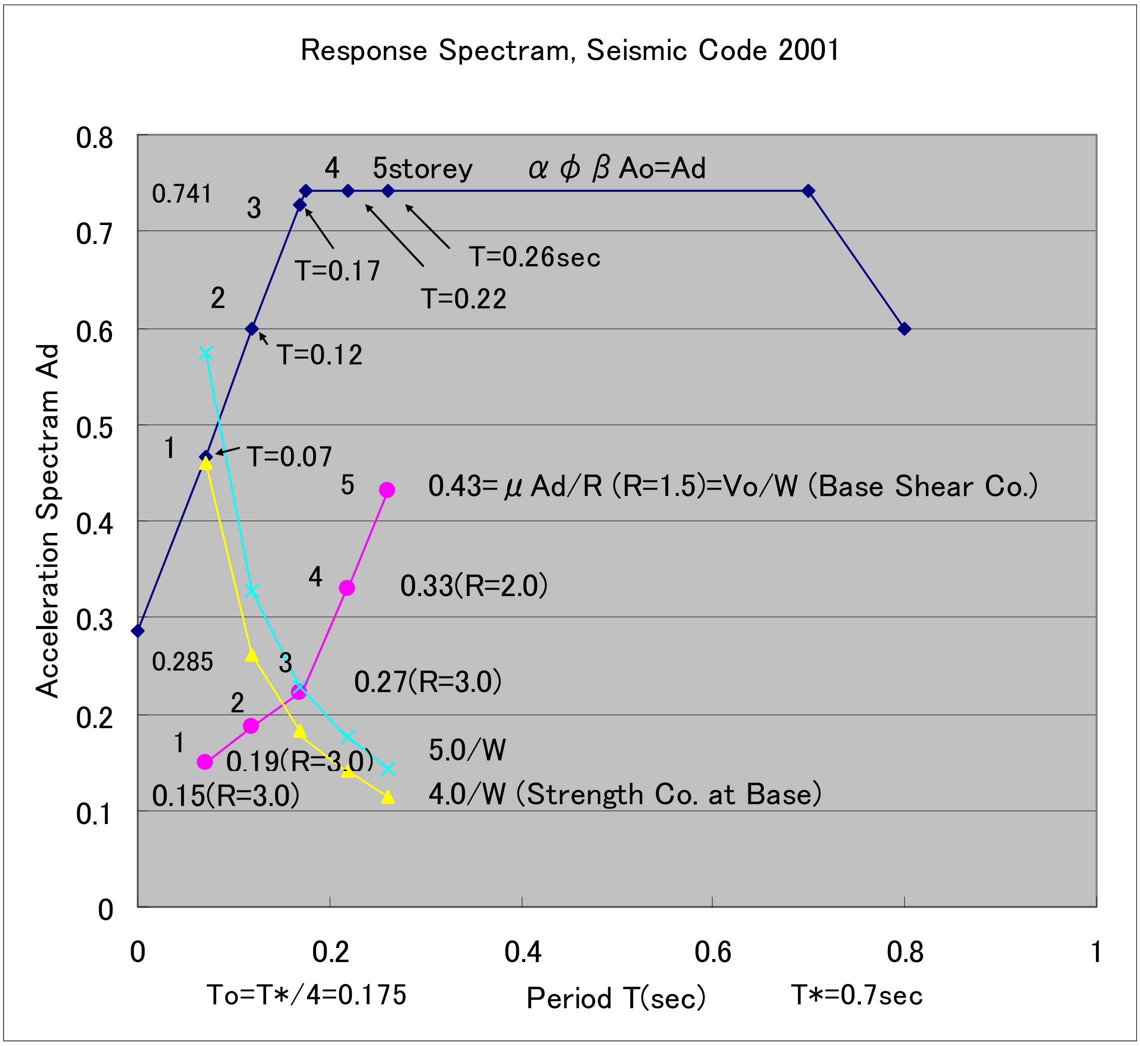


Figure A2.2 Response Spectrum and Base Shear Coefficient by Code 2001

(Reference only for comparison purpose)

Where: Ad (ordinate of the design spectrum)

Ao = 0.30 (coefficient of horizontal acceleration in zone 5)

ϕ = 0.95 (correction factor of the horizontal acceleration coefficient, S2 is used)

α = 1.0 (importance coefficient)

β = 2.6 (average response magnification factor, P = 1.0, T* = 0.7sec, spectral form S2

R = 3.0 to 1.5 is used (reduction factor)

μ (shear modification factor)

W (total weight of the building)

Table A2.1 Seismic Assessment of Barrio Houses (1)

Seismic Assessment of Barrio Houses on Slope rev.1

(strength for 1 frame (2 columns))		Weight(ton)	Wi	σ/Fc (60kg/cm2)	Ci(Co=1.0)	Qi(ton)	Assumed Ductility Factor	Assumed Response Red. Factor
5story	RFL(5F)	2.18	2.18	0.046	2.15	4.69	3	3
h=13.2m	5FL(4F)	6.55	8.73	0.182	1.51	13.18	3	3
T=0.26sec	4FL(3F)	6.55	15.28	0.318	1.32	20.17	3	3
	3FL(2F)	6.55	21.83	0.455	1.19	25.98	3	3
	2FL(1F)	6.55	28.38	0.591	1.09	29.39	2	2
	1FL(B1F)	6.55	34.93	0.728	1	34.93	1.5	1.5
4story	RFL(4F)	2.18	2.18	0.046	1.94	4.23	3	3
h=10.8m	4FL(3F)	6.55	8.73	0.182	1.4	12.22	3	3
T=0.22sec	3FL(2F)	6.55	15.28	0.318	1.22	18.64	3	3
	2FL(1F)	6.55	21.83	0.455	1.1	24.01	3	3
	1FL(B1F)	6.55	28.38	0.591	1	28.38	2	2
3story	RFL(3F)	2.18	2.18	0.046	1.69	3.68	3	3
h=8.4m	3FL(2F)	6.55	8.73	0.182	1.27	11.09	3	3
T=0.17sec	2FL(1F)	6.55	15.28	0.318	1.11	16.96	3	3
	1FL(B1F)	6.55	21.83	0.455	1	21.83	3	3
2story	RFL(2F)	2.18	2.18	0.046	1.47	3.2	3	2.6
h=6.0m	2FL(1F)	6.55	8.73	0.182	1.13	9.86	3	2.6
T=0.12sec	1FL(B1F)	6.55	15.28	0.318	1	15.28	3	2.6
1story	RFL(1F)	2.18	2.18	0.046	1.2	2.62	3	1.9
3.6m,.07se	1FL(B1F)	6.55	8.73	0.182	1	8.73	3	1.9

Table A2.2 Seismic Assessment of Barrio Houses (2)

Seismic Assessment of Barrio Houses on Slope (strength for 1 frame (2 columns)) rev.1

	Case 1(m.g..a.=0.30g) Venezuela Seismic Code 1982					Case 2(m.g.a.=0.15G)		
	Ad(0.66/R) (ordinate)	μ Ad= Vo/W	Vi(ton) (xQi)	Vui(ton) Assumed	Assess	μ Ad (0.33/R)	Vi(ton)	Assess
RFL(5F)	0.22	0.178	0.83	3.0 to 4.0	(μ=0.808)	0.089	0.42	
5FL(4F)	0.22	0.178	2.35	3.0 to 4.0		0.089	1.17	
4FL(3F)	0.22	0.178	3.59	3.0 to 4.0		0.089	1.8	
3FL(2F)	0.22	0.178	4.62	3.0 to 4.0		0.089	2.31	
2FL(1F)	0.33	0.267	7.84	3.0 to 4.0		0.134	3.92	
1FL(B1F)	0.44	0.356	12.44	4.0 to 5.0	collapse	0.178	6.22	collapse
RFL(4F)	0.22	0.18	0.76	3.0 to 4.0	(μ=0.818)	0.09	0.38	
4FL(3F)	0.22	0.18	2.2	3.0 to 4.0		0.09	1.1	
3FL(2F)	0.22	0.18	3.36	3.0 to 4.0		0.09	1.68	
2FL(1F)	0.22	0.18	4.32	3.0 to 4.0		0.09	2.16	
1FL(B1F)	0.33	0.27	7.66	4.0 to 5.0	collapse	0.135	3.83	seri.damage
RFL(3F)	0.22	0.183	0.67	3.0 to 4.0	(μ=0.833)	0.092	0.34	
3FL(2F)	0.22	0.183	2.03	3.0 to 4.0		0.092	1.02	
2FL(1F)	0.22	0.183	3.1	3.0 to 4.0		0.092	1.55	
1FL(B1F)	0.22	0.183	3.99	4.0 to 5.0	seri.damage	0.092	2	damage
RFL(2F)	0.23	0.194	0.62	3.0 to 4.0	(μ=0.857)	0.097	0.31	
2FL(1F)	0.23	0.194	1.91	3.0 to 4.0		0.109	0.96	
1FL(B1F)	0.23	0.218	2.96	4.0 to 5.0	damage	0.109	1.48	sli.damage
RFL(1F)	0.25	0.222	0.58	3.0 to 4.0	(μ=0.90)	0.111	0.29	
1FL(B1F)	0.25	0.312	1.94	4.0 to 5.0	damage	0.156	0.97	no damage

Table A2.3 Estimation of Heavily Damaged Barrio Houses

Estimation of Number of Damaged Barrio Houses rev.1

	Case 1(m.g.a.=0.30g)				Case 2(m.g.a.=0.15g)			
	Collapse	Serious Damage	Damage	No Damage	Collapse	Serious Damage	Damage	Slight/ No Damage
5story	1,214				1,214			
4story	8,390					*4,195	*4,195	
3story		*15,456	*15,456				30,912	
2story			47,582					47,582
1story			22,301					22,301
Slope48%	9,604	15,456	85,339		1,214	4,195	35,107	69,883
	(sub total	25,060)			(sub total	5,409)		
5story	1,316					*658	*658	
4story		*4545	*4545				9,090	
3story			33,488					33,488
2story			51,548					51,548
1story				24,159				24,159
Others52%	1,316	4,545	89,581	24,159		658	9,748	109,195
Total	10,920	20,001	174,920	24,159	1,214	4,853	44,855	179,078

Note * shows number allocated to 50% and 50%

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LIFELINE/INFRASTRUCTURE DATABASE

"One Threat well-known and handled by the Community,

is a calculated risk"

Marielba Guillen

STUDY ON
DISASTER PREVENTION BASIC PLAN
IN THE METROPOLITAN DISTRICT OF CARACAS

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LIFELINE/INFRASTRUCTURE DATABASE

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S-8 LIFELINE/INFRASTRUCTURE DATABASE

CHAPTER 1. OBJECTIVES OF DATABASE ESTABLISHMENT

In the earthquake prone country, densely populated major-cities with millions citizens face serious issues on catastrophic earthquake disaster damages including human casualties, building damages, lifeline damages/malfunctions, etc. Most of those major-cities do not have experience of catastrophic earthquake disaster damages after their rapid urban expansion. In order to solve those issues, central government and local governments of major-cities are trying to establish a database of socio-economic conditions, building, lifeline and infrastructure to formulate proper countermeasures to foreseeable earthquake disaster damages. This supporting report focuses on creating GIS database of lifeline and infrastructure, which are indispensable input data for the following works;

1 Estimation of earthquake disaster damage on lifeline and infrastructure for formulation of proper, operational and tangible plans

2 Formulation and implementation of mitigation measures for earthquake disaster damages of lifeline and infrastructure

3 Formulation and operation of emergency response plan for recovering damaged lifeline and infrastructure

4 Formulation and implementation of rehabilitation plan of damaged lifeline and infrastructure

5 Formulation of earthquake resistant urban development plan before earthquake occurrence

6 Establishment of simulation model of earthquake damage (to be able to contribute to quick and proper emergency response and rehabilitation works of the above items 3/4).

Lifeline and infrastructure are used for earthquake damage estimation in each earthquake scenario in order to:

1 identify collapse and damage of major road bridges

2 assess vulnerability of open cut and shield section of metro tunnel

3 estimate number of damage points on water supply pipeline networks in each micro zone

4 estimate number of damage points on sewage pipeline network in each micro zone

5 estimate number of damage points on natural gas supply pipeline in each micro zone (the

analytical results of pipe damage can be used to define malfunction area of each lifeline service)

6 estimate damage length of electric power supply cable in each micro zone

7 estimate damage length of telecommunication cable in each micro zone, (the analytical results of cable damage can be used to define malfunction area of each lifeline service)

8 estimate number of fire outbreaks from identified hazardous facilities in each micro zone.

CHAPTER 2. REQUIRED DATA TO ESTABLISH THE PROPER DATABASE

In this chapter, methodology of damage estimation are explained together with the required data format.

2. 1 General

Earthquake disaster damage on lifeline and infrastructure networks is estimated from three input factors as follows:

1 **GIS Network Database:** homogeneous section of pipeline/cable and facility with required attribute data of characteristics such as pipe size, material, gas pressure, etc, which are directly related to damage functions.

2 **Damage Functions in the Country:** damage ratio by earthquake motion and characteristics of each lifeline and infrastructure on the past earthquake disaster damage statistics in the country.

3 **Earthquake Motion in Micro Zone:** peak ground acceleration/velocity, seismic intensity, liquefaction potential, and ground type in each seismic micro zone or mesh cell for each earthquake scenario.

In order to estimate earthquake damages, all the collected GIS based network and facility data of lifeline and infrastructure in each micro zone or mesh cell (analytical zone for damage estimation) have to be categorized and divided into homogeneous pipe and cable sections, which correspond to the set of damage functions of each lifeline and infrastructure.

In Venezuela, earthquake damage estimation method and damage functions for infrastructure and lifelines have not been established yet based on the limited earthquake events and lack of major urban earthquake disaster damage experience and lack of statistical damage data. Hence, the Study Team proposes to apply Japanese earthquake damage estimation methods with Japanese damage functions for damage estimation study, which is shown on Section 9 in Supporting Report.

Earthquake scenarios to be used for damage estimation here are the earthquakes in 1812 and in 1967 in Venezuela show in Table S8-2.1.1.

2. 2 Road Bridge

Two kinds of bridge databases are required to apply the two different damage estimation methods in Japan, which are Katayama's Method and Tokyo Metropolitan Seismic Micro-zoning Study Method. Katayama's Method is used to assess the possibility of girder falling. Bridge database format with required data items for Katayama's Method is in Table S8-2.2.1.

Bridge database format for the other bridge damage estimation method on multi-span type elevated urban highway bridges. The method, which assesses the damage possibility on bridge piers, was established based on the bridge damage statistics of Hanshin Awaji Earthquake Disaster and was applied in the Seismic Micro-zoning Study of Tokyo Metropolitan Government. The format of the database is in Table S8-2.2.2.

2. 3 Metro Network

Metro Network Databases are required to assess vulnerability of metro tunnel section. Metro Network has to be classified according to the type of sections with attribute data as is showed in Table S8-2.3.1.

2. 4 Water Supply Network

The following database format (Table S8-2.4.1) is required for damage estimation of water supply pipe in Seismic Micro-zoning Study of Tokyo Metropolitan Government. Collected data of water supply pipeline network has to be sub-classified into homogenous pipe section of pipe size and material within each micro-zone or mesh cell.

The following categories of pipe size and materials on the method are applied for the database creation:

<table>
<tr><th>Category of Pipe Material</th><th>Category of Pipe Size</th></tr>
<tr><td rowspan="4">1: Ductile Cast Iron</td><td>1: less than 75mm</td></tr>
<tr><td>2: 100mm to less than 450mm</td></tr>
<tr><td>3: 500mmto less than 900mm</td></tr>
<tr><td>4: more than 1000mm</td></tr>
<tr><td rowspan="3">2: Cast Iron</td><td>1: less than 75mm</td></tr>
<tr><td>2: 100mm to less than 250mm</td></tr>
<tr><td>3: 300mm to less than 900mm</td></tr>
</table>

	4: more than 1000mm
3: Steel	1: less than 75mm
	2: 100mm to less than 250mm
	3:more than 300mm
4: Chloro-ethylene	1: less than 75mm
	2: more than 100mm
5: Asbestos Cement	1: less than 75mm
	2: 100mm to less than 250mm

2. 5 Sewage Disposal Network

The following database format (Table S8-2.5.1) is required for damage estimation of sewage pipe in Seismic Micro-zoning Study of Tokyo Metropolitan Government. Collected data of sewage pipeline network has to be sub-classified into homogenous pipe section of pipe size and material within each micro-zone or mesh cell.

The following categories of pipe size and materials on the method are applied for the database creation:

Pipe Materials
1. Hume pipe/reinforced concrete pipe
2. non-reinforced concrete pipe
3. ceramic pipe
4. chloro-ethylene pipe
5. shield pipe
6. on-site reinforced concrete pipe
7. box culvert

Pipe Size
1. > 4000mm
2. 2000～4000mm
3. 1000～2000mm
4. 500～1000mm
5. 150～500mm
6. 50～150mm

2. 6 Natural Gas Supply Network

Collected network data of natural gas supply pipeline has to be sub-classified into homogenous pipe section of gas pressure, pipe material/joint within each micro-zone or mesh cell. The following database format (Table S8-2.6.1) with attribute data is required to apply the natural gas supply pipe damage estimation method in Seismic Micro-zoning Study of Tokyo Metropolitan Government:

The following categories of gas pressure and pipe materials/joint type on the method are applied:

Category of Gas Pressure
1: medium pressure
2: low pressure

Category of Pipe Material/Joint Type
1. Steel
2. Cast Iron
3. Steel(welded)
4. Steel (screw)
5. Steel (mechanical)
6. Old Type Ductile Cast Iron (joint type-1)
7. Old Type Ductile Cast Iron (joint type-2)
8. Ductile Cast Iron (Mechanical)
9. Ductile Cast Iron (other joint)
10. Polyethylene
11. Chloro-ethylene

2. 7 Electric Power Supply Network

Collected data of electric power supply cable network has to be subdivided and categorized into aerial section and buried section within each micro-zone or mesh cell. The following database format (Table S8-2.7.1) is required to apply the electric power supply cable damage estimation in Seismic Micro-zoning Study of Tokyo Metropolitan Government:

2. 8 Telecommunication Network

Database format with required attribute data and categories of telecommunication network are the same as the electric power cable damage estimation above.

2. 9 Hazardous Facility

During an earthquake event, damaged hazardous facilities in the metropolitan area may generate many secondary disasters by fire outbreaks/spreading, explosion and toxic gas spreading from damaged facilities. These incidents can not be easily managed under disordered conditions by the emergency taskforce teams. Before an earthquake event, weak facilities with hazardous and toxic materials and products have to be defined in order to mitigate such secondary disaster.

Damage functions of hazardous facilities in the Seismic Micro-zoning Study of Tokyo Metropolitan Government are set on the correlation coefficient obtained by statistical data of past earthquake in terms of ground motion (PGA) and identified damaged for each category of hazardous facility classified by Tokyo Metropolitan Fire Fighting Dept.

The category of hazardous facility, type of damage, and damage ratio by PGA in the method is shown in Table S8-2.9.1.

The following database format (Table S8-2.9.2) including 3 categories of hazardous facilities, number tank/storage and size of tank/storage that is applicable for the method:

Table S8-2.1.1 Scenario Earthquakes and their Parameters

Scenario	Mw	Seismogenic Depth	Fault Length	Mechanism	Fault system
1812	7.1	5 km	105 km	Strike slip	San Sebastian
1967	6.6	5 km	42 km	Strike slip	San Sebastian

Source: JICA Study Team 2004

Table S8-2.2.1 Bridges Database Format for Katayama's Method

Bridge Code No.	Name or Number of Bridge	Name or No of Road	number of crossing road/river/metr	Girder Type	Bearing	Max. Height of Abut./Pier	Number of span	Materials of Abutment and Pier	Min. Bridge Seat Width	Foundation type

Source: JICA Study Team 2003

Table S8-2.2.2 Road Bridge Database Format of Tokyo Metropolitan Method for Elevated Urban Highway Damage Estimation

Bridge Code No.	Name or Number of Bridge	Name or No of Road	Name or number of crossing road/river/metro	Width (m)	Length (m)	Rank/Function of Road

Source: JICA Study Team 2003

Table S8-2.3.1 Metro Network Database Format

Section Code No.	Number of Metro Line	Category of section	Type of construction	Length (m)	Completion year
		1. station	1. shield tunnel		
		2. carriage	2. mountain tunnel		
			3. open cut tunnel		
			4. others		

Source: JICA Study Team 2003

Table S8-2.4.1 Water Supply Pipe Database Format of Tokyo Metropolitan Method

Code of Pipe Section	Municipality Code	Microzone or Mesh Cell Code	Pipe Size (diameter)	Category of Pipe Material	Length of Pipe Section

Source: JICA Study Team 2003

Table S8-2.5.1 Sewage Pipe Database Format of Tokyo Metropolitan Method

Code No. of Pipe Section	Municipality Code	Microzone or Mesh Cell Code	Pipe Size (diameter)	Category of Pipe Material	Length of Pipe Section

Source: JICA Study Team 2003

Table S8-2.6.1 Natural Gas Supply Pipe Database Format of Tokyo Metropolitan Method

Code No. of Pipe Section	Municipality Code	Microzone or Mesh Cell Code	Gas Pressure	Category of Pipe Material/joint type	Length of Pipe Section

Source: JICA Study Team 2003

Table S8-2.7.1 Electric Power Supply Cable Database Format of Tokyo Metropolitan Method

Code No. of Cable Section	Municipality Code	Microzone or Mesh Cell Code	Aerial or Buried	Length of Cable Section
			1. Aerial	
			2. Buried	

Source: JICA Study Team 2003

Table S8-2.9.1 Category of Hazardous Facility, Type of Damage and Damage Ratio of Tokyo Metropolitan Method

Category of Hazardous Facility	Type of Damage	PGA										
		100	150	200	250	300	350	400	450	500	550	600
1. Large storage tank of flammable Liquid	1. small spill from tank and pipe joint	4.10E-05	1.50E-04	4.90E-04	1.40E-03	3.30E-03	6.90E-03	1.30E-02	2.00E-02	3.00E-02	3.80E-02	4.70E-02
	2. continuous certain volume of spill	1.00 E-05	3.80E-05	1.20E-04	3.40E-04	8.20E-04	1.70E-03	3.20E-03	4.90E-03	7.50E-03	9.40E-03	1.20E-02
	3. overflow from protection dike	2.40E-06	8.90E-06	2.90E-05	8.00E-05	1.90E-04	4.00E-04	7.40E-04	1.10E-03	1.70E-03	2.20E-03	2.80E-03
	4. fire outbreak of oil in protection dike	1.00E-06	3.80E-06	1.20E-05	3.40E-05	8.20E-05	1.70E-04	3.20E-04	4.90E-04	7.50E-04	9.40E-04	1.20E-04
	5. large fire spreading on tank-yard	2.40E-07	8.90E-07	2.90E-06	8.00E-06	1.90E-05	4.00E-05	7.40E-05	1.10E-04	1.70E-04	2.20E-04	2.80E-04
2. Tanks and gas-holder of flammable gas	6. spill from pipe joint to tank (emergency shut-down)	1.50E-05	4.20E-05	1.10E-04	2.50E-04	5.60E-04	1.10E-03	2.20E-03	3.70E-03	6.30E-03	9.50E-03	1.40E-02
	7. continuous spill of certain volume (hazard of explosion)	3.80E-06	1.00E-05	2.70E-05	6.30E-05	1.40E-04	2.80E-04	5.40E-04	9.20E-04	1.60E-03	2.40E-03	3.50E-03
	8. fire outbreak of spilled gas in protection dike	3.80E-07	1.00E-06	2.70E-06	6.30E-06	1.40E-05	2.80E-05	5.40E-05	9.20E-05	1.60E-04	2.40E-04	3.50E-04
	9. explosion of large spilled gas	3.80E-08	1.00E-07	2.70E-07	6.30E-07	1.40E-06	2.80E-06	5.40E-06	9.20E-06	1.60E-05	2.40E-05	3.50E-05
3. Tank of toxic gas/ liquid nitrogen	10. spill from pipe joint of tank	3.00E-06	8.40E-06	2.10E-05	5.10E-05	1.10E-04	2.30E-04	4.30E-04	7.40E-04	1.30E-03	1.90E-03	2.80E-03
	11. continuous spill of certain volume (hazard for citizen)	7.60E-08	2.10E-07	5.30E-07	1.30E-06	2.80E-06	5.70E-06	1.10E-05	1.80E-05	3.20E-05	4.70E-05	7.10E-05

Source: Damage ratio of hazardous facility on the Seismic Micro-zoning Study of Tokyo Metropolitan Government, 1997

Table S8-2.9.2 Hazardous Facility Database Format of Tokyo Metropolitan Method

Code No. of facility	Municipality Code	Microzone or Mesh Cell Code	Category of hazardous facility	Size of tank or storage (m^3)	Number of tank or storage
			1.Flammable liquid		
			2. Flammable gas		
			3. Toxic gas		

Source: JICA Study Team 2003

CHAPTER 3. COLLECTED DATA OF LIFELINE AND INFRASTRUCTURE

Data collection works to establish lifeline and infrastructure database were implemented during the first and second works in Venezuela in the first study phase.

In the collection works, limited data on the water supply network, telecommunication and hazardous facilities were obtained from responsible agencies. However, natural gas and electric power supply network data could not be obtained from responsible agencies for security reasons. Sewage disposal system has not been properly introduced and established in the study area at the present. The details of data availability and condition of collected data are described below.

3. 1 Available Data to Establish the Required Database in the Metropolitan District of Caracas

Status of digital /or GIS based database establishment of lifelines and infrastructure are directly related to needs of development, upgrading and improvement works for each sector of lifeline and infrastructure. Accordingly, advanced GIS Databases were established in natural gas supply network based on social needs of development/expansion and required proper management system for security. On the other hand, road and bridge databases are not established because of lack of need for new major roads and bridges. Most main roads and bridges were developed and established before 1990. Availability of the required data to establish GIS database for earthquake damages estimation of each lifeline and infrastructure are limited as described below.

3. 2 Road Network: MIINFRA and Local Government

Most of public road networks with bridges were developed and established by the central government (MIINFRA) by 1990. Major road network of regional highways, urban expressways and arterial roads are maintained by MIINFRA. The responsibility of maintenance of other local road networks is on the municipal government.

3. 2. 1. Available GIS Road Network Data

Road network database has not been established in digital and GIS bases in the metropolitan area by MIINFRA at present. Responsible local governments also did not establish the other local road network data. However, the Feasibility Study of Metro Development by METRO Company established GIS road network database in recent years as shown in Figure S8-3.2.1, which will be useful database for the study. But the study does not deal with local road networks in general.

3. 2. 2. Existing Condition of Road Network

Total length of the digitized road network from the above study is 77,727 km within the 5 municipalities of metropolitan area and 56,845 km in the study area as shown in Table S8-3.2.1. The digitized road network is composed of 184 km of expressways (15% of total length), 347 km of arterial roads (29%), 192 km of primary collector roads (16%), 201 km of secondary collector roads (17%), 194 km of local roads (16%), and 89 km of other roads (7%).

The existing expressway network consists of road network structure for the metropolitan area with the concept of Ring and Radial roads. However, part of the Ring, which is single ring system surrounding the major part of Caracas Metropolitan basin, is still missing and undeveloped yet. During a natural disaster, the missing Ring will not be able to provide a substitute route for primary road function. Radials are taking the function of regional link with centers in the surrounding regions.

In Barrio areas, which are illegally occupied and developed without proper road and lifeline development, limited narrow road and pedestrian access exist. Barrio areas can be categorized as high-risk area on emergency evacuation and response operations in disaster, which will be easily and completely isolated.

The estimated road densities are around 1.7 km/km^2 and 0.4 m/person in the study area and 1.6 km/m^2 in the metropolitan area. In Chacao municipality, road network is densely and well developed with 4.1 km/km^2 and 1.1 m/person, which are comparatively higher density than the other 4 municipalities.

3. 2. 3. Availability of Road Bridge Data: JICA Study Team

MIINFRA has not yet established digital database for bridges on major road network in the study area. Responsible municipal governments also did not establish the other bridge data for local roads either.

Hence, digital databases of road bridge for damage estimation methods are not available at the present.

JICA Study team established the following Major Road Bridge Database for damage estimation of Katayama's Method by field observation survey and interpretation of the existing bridge drawings in MIINFRA. The developed database covers 119 major bridges on expressways as listed in Table S8-3.2.2, and in Figure S8-3.2.2, with attribute data of ground type, liquefaction potential, and 7 items of bridge data as follows:

1 **Ground Type:** Categories are 0.5: stiff, 1.0: middle, 1.5: soft, and 1.8: very soft

2 **Liquefaction:** Categories of potential are 1.0: none, 1.5: possible, and 2.0: probable

3 **Girder Type:** Categories are 1.0: rigid, 2.0: continuous, and 3.0: simple

4 **Bearing Type:** categories are 0.60: with girder connection device, 1.00: fixed/move, and 1.15: move/move

5 **Maximum Height of Abutment and Pier:** categories are 1.00: less than 5m, 1.35: 5 to 9.9m, and 1.70: more than 10m height

6 **Number of spans:** categories are 1.00: single span and 1.75: tow spans and more

7 **Minimum Bridge Seat Length:** categories are 0.8: wide width and 1.2: narrow width

8 **Foundation work:** categories are 1.4: pile bent and 1.0: others

9 **Materials of Abutment and Pier:** categories are 1.4: brick and 1.0: others

3. 3 Metro Network: Metro Company

Mass and public transit system is indispensable to mitigate and avoid socio-economic loss in motorized mega-cities. In the metropolitan area, three lines of Metro have been developed and its total length is 44.3 km.

Outline of three Metro lines are shown in Table S8-3.3.1 and its location and open cut and box type tunnel sections are shown in Figure S8-3.3.1.

3. 4 Water Supply Network: Hydrocapital and IMAS

3. 4. 1. Availability of Water Supply Network Data

Most of the existing pipeline networks in the study area has been digitized on AutoCAD system by the responsible agencies, which are Hydrocapital covering most of the study area and IMAS covering the eastern part of Sucre Municipality. The obtained digital network data of Hydrocapital is still in the establishing stage with pipe size attribute data without required pipe material data, which is also required for damage estimation. IMAS digital network data also lacks the required pipe material data and is not covering 20% to 25% of IMAS Service Area in Eastern Sucre Municipality.

3. 4. 2 Existing Conditions of Water Supply System

Water resource development, transmission and distribution for major part of the metropolitan area are maintained and managed by Hydrocapital, which is planned to privatize in the near future. Water distribution in the eastern part of Sucre municipality is developed and managed by IMAS, which is organized under Sucre Municipality.

Total length of the obtained digitized pipeline networks from Hydrocapital and IMAS are around 1,984 km in the metropolitan area and 1,383 km in the study area as summarized in Table S8-3.4.1 and mapped in Figure S8-3.4.1. Transmission and trunk water supply networks with pipe diameter more than 1,000 mm area around 81 km in the metropolitan area and 73 km in the study area, which shares 4% to 5% of the total pipeline length. City main water supply networks with pipes diameter between 500 and 900 mm are around 185 km in the metropolitan area and 148 km in the study area, which shares 9% to 11% of total pipeline length. Distribution networks with pipe diameter less than 450 mm are around 1,718 km in the metropolitan area and 1,162 km in the study area, which shares 84% to 87% of total pipeline length.

Newly expanded and expanding water supply networks are utilizing ductile cast iron pipes, categorized as a pipe to resist seismic motion. The old water supply pipe networks in the city center and the surroundings had been replaced by ductile cast iron pipes. Hence, existing water supply pipeline networks in the metropolitan area are mostly composed of seismic resistant ductile pipelines.

The existing water supply pipeline network is densely located in Chacao Municipality, which has 2 m length of pipe per person and 74 m/ha. On the other hand, Libertador has 0.6 m per person and 30 m/ha and Sucre has only 0.2 m per person and 6 m/ha, which are quite low densities. In Barrio areas, public water supply system are limitedly developed, which are only installed by main water supply pipes on narrow access road in the area by Hydrocapital or IMAS. Local community and families are tapping from those main pipes without payment for water consumption. Those self-tapping private pipes are not digitized and not counted in the above data.

3. 5 Sewage Network Database: Hydrocapital

Hydrocapital is responsible for sewage in the metropolitan area. However, sewage disposal system is not introduced and established. At present, household and domestic wastewater is directly discharged to rivers through drainage pipes. Data for those drainage pipes is not digitized and compiled as database for damage estimation study in the study area. Hence, database development and damage estimation of sewage network is decided not to be taken into account in this study.

3. 6 Natural Gas Supply: PDVSA Gas

PDVSA Gas is responsible for transmission of high-pressure natural gas, medium pressure trunk network and distribution to subscriber in the area. Based on the interviews with PDVSA Gas, the requested database of natural gas supply pipeline networks has been established on GIS for maintenance and security purposes by themselves. However, the established GIS database is categorized as confidential, and was not given to the JICA Study Team for security reasons. Hence, high-pressure transmission pipe and medium-pressure city main pipe networks are only available on the schematic paper map format, which is not enough for damage estimation in the study. Thus, database development and damage estimation of natural gas supply network are not taken into account in this study.

3. 7 Electric Power Supply: Electricidad Caracas

Electricidad Caracas is responsible for electric power supply in the metropolitan area. Based on the interviews with Electricidad Caracas, the requested database of electric power supply cable networks have been established on GIS for maintenance and security purpose by themselves. However, the established GIS database and other all network maps with base map information are also categorized as confidential data and were not given to the JICA Study Team for security reasons. Thus, database development and damage estimation of electric power supply network are not taken into account in this study.

3. 8 Telecommunication: CANTV

3. 8. 1. Existing Condition of Telecommunication System

Telecommunication system in Venezuela has been privatized and rapidly developed. Telecommunication system in the metropolitan area is covered by fixed cable network of CANTV and mobile phone network of CANTV, TELCEL and DIGITAL. However, city main loops of fixed and mobile telecommunication systems are dependent on fiber optic cable networks of CANTV.

The existing telecommunication cable networks are composed of 4 types of aerial, underground, buried, and cable box sections. 26% to 32% of total networks are shared by aerial cable, which is categorized as weak cable network against earthquake, which will generate secondary disaster by short circuit of damaged cables and damaged poles.

Total length of fixed telecommunication networks is 7,128 km in the five municipalities of the metropolitan area and 5,423 km in the study area as summarized in Table S8-3.8.1. The existing cable networks are densely developed as 6.9 m per person and 260 m/ha in Chacao municipality. The cable length is of 2 m per person and 110 m/ha in Libertador municipality and 1.3 m per person and 46 m/ha in Sucre municipality. Those lower densities conditions are caused by lack of fixed phone service for Barrio areas in both municipalities.

3. 8. 2. Availability of Telecommunication Data

The existing fixed telecommunication network in the metropolitan has not been digitized yet, but cable network lengths by types were counted and tabulated in their service zone by CANTV. In order to establish GIS database, the obtained cable length data by their service zone has been redistributed and converted to micro zone or mesh cell based on the share of road length and share of housing unit in zone or mesh cell within their service zone. The redistributed cable length in each micro zone of mesh cell is established as minimum required input for damage estimation work.

3. 9 Hazardous Facility: Hazardous Material Division of Fire Fighting Dept.

3. 9. 1. Existing Condition of Hazardous Facilities

Based on the interviews with Hazardous Material Division of Fire Fighting Dept., most of major hazardous facilities in the Metropolitan area were relocated to the eastern and western industrial zones in Venezuela to avoid high-risk hazards in the metropolitan area under the government policy. On the other hand, registration and monitoring system of hazardous materials in the metropolitan are not properly established yet. Hazardous facilities in private sector is identified and checked by Fire Fighting Dept. as follows:

1 Small scale private storage and filling station of high pressure gas: most hazardous

2 Small scale private chemical industry: most hazardous

3 Petrol station: well standardized to resist seismic motion

4 Major public high pressure gas storage

5 Major public petrol storage: unclear

3. 9. 2. Availability of Hazardous Facility Data

Part of the requested database of hazardous facility data, which is gas stations, was supplied by the Hazardous Material Division of the fire fighting department. However, data of other hazardous facilities belonging to the Ministry of Energy and others are not available.

The obtained hazardous facility data of 62 gas stations are mapped in Figure S8-3.9.1.

Table S8-3.2.1 Digitized Road Length by Municipality and Category in the Metropolitan Caracas

Municipality	Libertador		Chacao		Sucre		Study Area		Baruta		El Hatillo		Total length	
Category of Road	(km)	(share)	(km)	(share)	(km)	(share)	(km)	(share)	(km)	(share)	(km)	(share)	(km)	(share)
Expressway	113	17%	0	0%	53	22%	166	17%	18	10%	0	0%	184	15%
Arterial Road	163	25%	30	39%	91	37%	285	30%	54	28%	8	16%	347	29%
Primary Road	99	15%	12	15%	16	6%	126	13%	47	25%	18	35%	192	16%
Secondary Road	113	18%	21	28%	25	10%	160	17%	37	19%	5	9%	201	17%
Local Road	84	13%	14	18%	41	17%	140	14%	33	17%	21	40%	194	16%
Others	71	11%	-		17	7%	88	9%	2	1%	-		89	7%
Total length	644	100%	78	100%	242	100%	963	100%	191	100%	52	100%	1,206	100%
Area (ha)	37,704		1,885		17,256		56,845		8,317		12,565		77,727	
Road Density (km/km^2)	1.7		4.1		1.4		1.7		2.3		0.4		1.6	
Population (1000 pop)	2,036		71		601		2,709							
Road Density (m/pop)	0.3		1.1		0.4		0.4							

Source: Feasibility Study of Metro Network Development

Table S8-3.2.2 Bridge Database for Damage Estimation of Katayama's Method

Code No.	1.Ground Type	2.Liquefaction	3.Girder Type	4.Bearing Type	5.Max. Height of Abut./Pier	6.Number of spans	7.Min. Bridge Seat	8.Foundation work	9.Materials of Abutment and Pier	Code No.	1.Ground Type	2.Liquefaction	3.Girder Type	4.Bearing Type	5.Max. Height of Abut./Pier	6.Number of spans	7.Min. Bridge Seat	8.Foundation work	9.Materials of Abutment and Pier
1	1.0	1.0	3.0	1.00	1.00	1.75	1.2	1.0	1.0	61	1.5	1.0	3.0	1.15	1.70	1.75	1.2	1.0	1.0
2	1.0	1.0	3.0	1.00	1.00	1.75	1.2	1.0	1.0	62	1.5	1.0	3.0	1.15	1.35	1.75	1.2	1.0	1.0
3	1.0	1.0	2.0	1.00	1.35	1.75	1.2	1.0	1.0	63	1.5	1.0	3.0	1.15	1.35	1.75	1.2	1.0	1.0
4	1.0	1.0	2.0	1.00	1.35	1.75	1.2	1.0	1.0	64	1.5	1.0	3.0	1.15	1.35	1.75	0.8	1.0	1.0
5	1.0	1.0	2.0	1.00	1.35	1.75	1.2	1.0	1.0	65	1.5	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0
6	1.0	1.0	2.0	1.00	1.00	1.75	1.2	1.0	1.0	66	1.5	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0
7										67	1.5	1.0	2.0	1.15	1.00	1.75	1.2	1.0	1.0
8										68	0.5	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0
9	0.5	1.0	2.0	1.00	1.00	1.75	1.2	1.0	1.0	69	1.8	1.0	2.0	1.00	1.70	1.75	0.8	1.0	1.0
10	0.5	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0	70	1.8	1.0	2.0	1.00	1.35	1.75	0.8	1.0	1.0
11	1.0	1.0	3.0	1.15	1.35	1.75	1.2	1.0	1.0	71	1.8	1.0	2.0	1.00	1.35	1.75	0.8	1.0	1.0
12										72	1.8	1.0	2.0	1.00	1.35	1.75	0.8	1.0	1.0

13										73	1.5	1.0	2.0	1.00	1.35	1.75	1.2	1.0	1.0
14	1.0	1.0	1.0	1.00	1.70	1.75	1.2	1.0	1.0	74	0.5	1.0	3.0	1.15	1.35	1.75	1.2	1.0	1.0
15	1.0	2.0	2.0	1.00	1.35	1.75	1.2	1.0	1.0	75	1.5	1.0	2.0	1.15	1.35	1.75	0.8	1.0	1.0
16	1.8	1.0	3.0	0.60	1.35	1.75	1.2	1.0	1.0	76	1.5	1.0	2.0	1.15	1.35	1.75	0.8	1.0	1.0
17	1.8	1.0	3.0	0.60	1.35	1.75	1.2	1.0	1.0	77	1.0	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0
18	1.0	1.0	3.0	0.60	1.35	1.75	1.2	1.0	1.0	78	1.8	1.0	2.0	1.15	1.00	1.75	1.2	1.0	1.0
19	1.0	1.0	3.0	1.00	1.35	1.75	1.2	1.0	1.0	79	1.8	1.0	2.0	1.00	1.00	1.75	0.8	1.0	1.0
20	0.5	1.0	2.0	1.00	1.70	1.75	1.2	1.0	1.0	80	1.5	1.0	2.0	1.15	1.35	1.75	0.8	1.0	1.0
21	0.5	1.0	3.0	0.60	1.00	1.75	0.8	1.0	1.0	81	1.5	1.0	2.0	1.15	1.35	1.75	0.8	1.0	1.0
22	0.5	1.0	3.0	1.00	1.35	1.75	0.8	1.0	1.0	82	1.5	2.0	2.0	1.15	1.35	1.75	0.8	1.0	1.0
23	0.5	1.0	3.0	1.15	1.35	1.75	0.8	1.0	1.0	83	1.5	2.0	2.0	1.15	1.35	1.75	0.8	1.0	1.0
24	1.0	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0	84	1.0	1.0	2.0	1.00	1.35	1.75	0.8	1.0	1.0
25	1.0	1.0	3.0	1.15	1.35	1.75	1.2	1.0	1.0	85	1.5	1.0	2.0	1.15	1.00	1.75	0.8	1.0	1.0
26	1.5	1.0	2.0	1.15	1.35	1.75	0.8	1.0	1.0	86	1.0	2.00	3.0	1.00	1.70	1.75	1.2	1.0	1.0
27	0.5	1.0	3.0	0.60	1.00	1.00	1.2	1.0	1.0	87	1.0	2.00	3.0	1.00	1.35	1.75	1.2	1.0	1.0
28	0.5	1.0	3.0	1.15	1.00	1.00	0.8	1.0	1.0	88	1.0	2.00	3.0	1.15	1.00	1.75	1.2	1.0	1.0
29	0.5	1.0	3.0	1.15	1.00	1.00	1.2	1.0	1.0	89	1.0	2.00	3.0	1.15	1.35	1.75	1.2	1.0	1.0
30	0.5	1.0	1.0	0.60	1.35	1.75	0.8	1.0	1.0	90	1.0	2.00	3.0	1.15	1.00	1.75	1.2	1.0	1.0
31	0.5	1.0	1.0	0.60	1.35	1.00	0.8	1.0	1.0	91	1.0	1.50	3.0	1.15	1.35	1.75	1.2	1.0	1.0
32	0.5	1.0	3.0	1.15	1.35	1.75	1.2	1.0	1.0	92	1.0	2.00	3.0	1.15	1.35	1.75	1.2	1.0	1.0
33	0.5	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0	93	1.0	2.00	3.0	1.15	1.70	1.75	1.2	1.0	1.0
34	1.0	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0	94	1.0	2.00	3.0	1.15	1.35	1.75	1.2	1.0	1.0
35	0.5	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0	95	1.0	2.00	3.0	1.15	1.35	1.75	1.2	1.0	1.0
36	0.5	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0	96	1.0	1.0	2.0	1.15	1.35	1.75	0.8	1.0	1.0
37	0.5	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0	97	1.0	1.0	2.0	1.15	1.35	1.75	0.8	1.0	1.0
38	0.5	1.0	3.0	1.00	1.00	1.00	1.2	1.0	1.0	98	1.5	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0
39	0.5	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0	99	1.5	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0
40	0.5	1.0	3.0	1.15	1.70	1.75	1.2	1.0	1.0	100	0.5	1.0	2.0	1.15	1.00	1.75	1.2	1.0	1.0
41	0.5	1.0	3.0	1.15	1.70	1.75	1.2	1.0	1.0	101	1.0	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0
42	1.0	1.0	3.0	1.15	1.70	1.75	1.2	1.0	1.0	102	0.5	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0
43	1.0	1.0	3.0	1.15	1.70	1.75	1.2	1.0	1.0	103	0.5	1.0	2.0	1.15	1.00	1.75	1.2	1.0	1.0
44	1.0	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0	104	0.5	1.0	3.0	1.15	1.00	1.00	0.8	1.0	1.0
45	1.0	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0	105	1.0	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0
46	1.0	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0	106	0.5	1.0	2.0	0.60	1.00	1.75	0.8	1.0	1.0
47	1.0	1.0	2.0	1.15	1.00	1.75	1.2	1.0	1.0	107	0.5	1.0	2.0	1.15	1.00	1.75	1.2	1.0	1.0
48	1.0	1.0	3.0	1.15	1.00	1.00	1.2	1.0	1.0	108	1.0	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0
49	1.0	1.0	3.0	1.15	1.00	1.00	1.2	1.0	1.0	109	1.0	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0
50	1.0	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0	110	0.5	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0
51	1.0	1.0	1.0	0.6	1.00	1.00	0.8	1.0	1.0	111	0.5	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0
52	1.0	1.0	2.0	1.15	1.00	1.75	1.2	1.0	1.0	112	1.0	1.0	2.0	1.15	1.00	1.75	0.8	1.0	1.0
53	1.0	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0	113	0.5	1.0	1.0	1.15	1.00	1.00	0.8	1.0	1.0
54	1.0	1.0	1.0	0.6	1.00	1.00	0.8	1.0	1.0	114	0.5	1.0	2.0	1.15	1.35	1.75	1.2	1.0	1.0
55	0.5	1.0	3.0	1.15	1.00	1.00	1.2	1.0	1.0	115	0.5	1.0	3.0	1.15	1.35	1.75	1.2	1.0	1.0
56	0.5	1.0	1.0	0.6	1.00	1.00	0.8	1.0	1.0	116	0.5	1.0	3.0	1.15	1.35	1.75	1.2	1.0	1.0
57	0.5	1.0	3.0	0.6	1.00	1.00	0.8	1.0	1.0	117	0.5	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0
58	0.5	1.0	3.0	0.6	1.00	1.00	0.8	1.0	1.0	118	0.5	1.0	3.0	1.15	1.00	1.75	1.2	1.0	1.0
59	0.5	1.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0	119	0.5	2.0	1.0	0.60	1.00	1.00	0.8	1.0	1.0
60	1.5	1.0	3.0	1.15	1.35	1.75	0.8	1.0	1.0										

Source: JICA Study Team 2003

Table S8-3.3.1 List of Metro Lines in Caracas Metropolitan

Line Name	Total Length (km)	Year of Completion	Number of Station	Length of Tunnel Section (km)				
				Shield Type	Mountain Tunnel Type	Open Cut Type	Others	Station
Line 1: Propatria-Palo Verde	20.6	1983	22	N/A.	N/A.	N/A.	N/A.	N/A.
Line 2:Silencio-Zoo Logico/Las Adjuntas	18.4	1987	13	1.6	1.7	4.5	8.4	2.2
Line 3:Plaza Venezuela-El Valle	5.3	1994	5	2.2	1.0	1.2	0	0.9
Total	44.3		40	3.8	2.7	5.7	8.40	3.1

Source: Metro Company

Table S8-3.4.1 List of Digitized Water Supply Pipeline in the Metropolitan Caracas

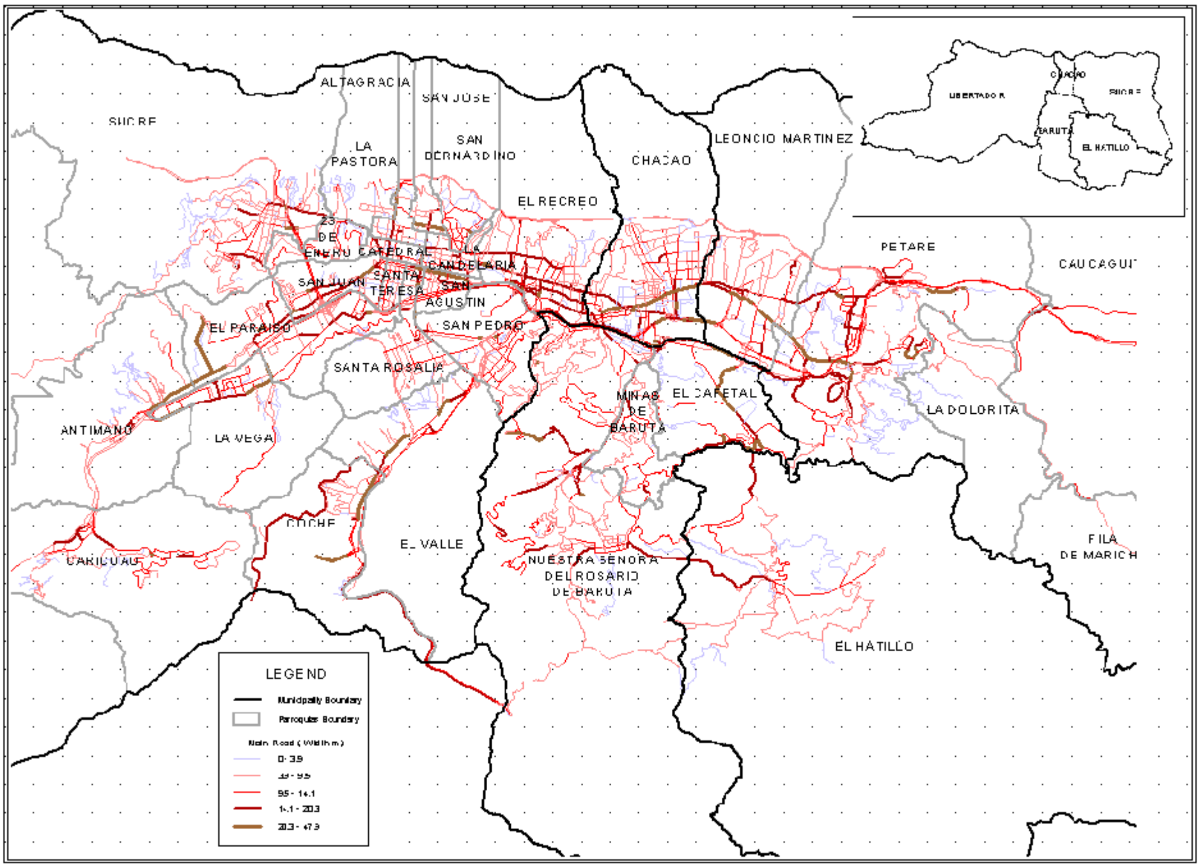
Municipality	Pipe Length by Size (km)						Population		Area		Pipe Density	
	<75mm	100-450mm	500-900mm	>1000mm	Total	Share (%)	"(1000)	Share (%)	(ha)	Share (%)	(m/p)	(m/ha)
Libertador	151	810	121	67	1,149	58	2,036	75	37,704	49	0.6	30
Chacao	24	93	16	6	139	7	71	3	1,885	2	2.0	74
Sucre	6	79	11	0	95	5	601	22	17,256	22	0.2	6
Baruta	31	381	32	8	452	23			8,317	11		54
El Hatillo	3	140	6	0	149	8			12,565	16		12
Total of Study Area	180	982	148	73	1,383	70	2,709	100	56,845	73	0.7	24
(share: %)	13.0	71.0	10.7	5.2	100.0							
G. Total	215	1,503	185	81	1,984	100			77,727	100		35
(share: %)	10.8	75.8	9.3	4.1	100.0							

Source: the obtained AutoCAD data from Hydrocapital and IMAS 2003

Table S8-3.8.1 List of Telecommunication Cable Network in the Metropolitan Caracas

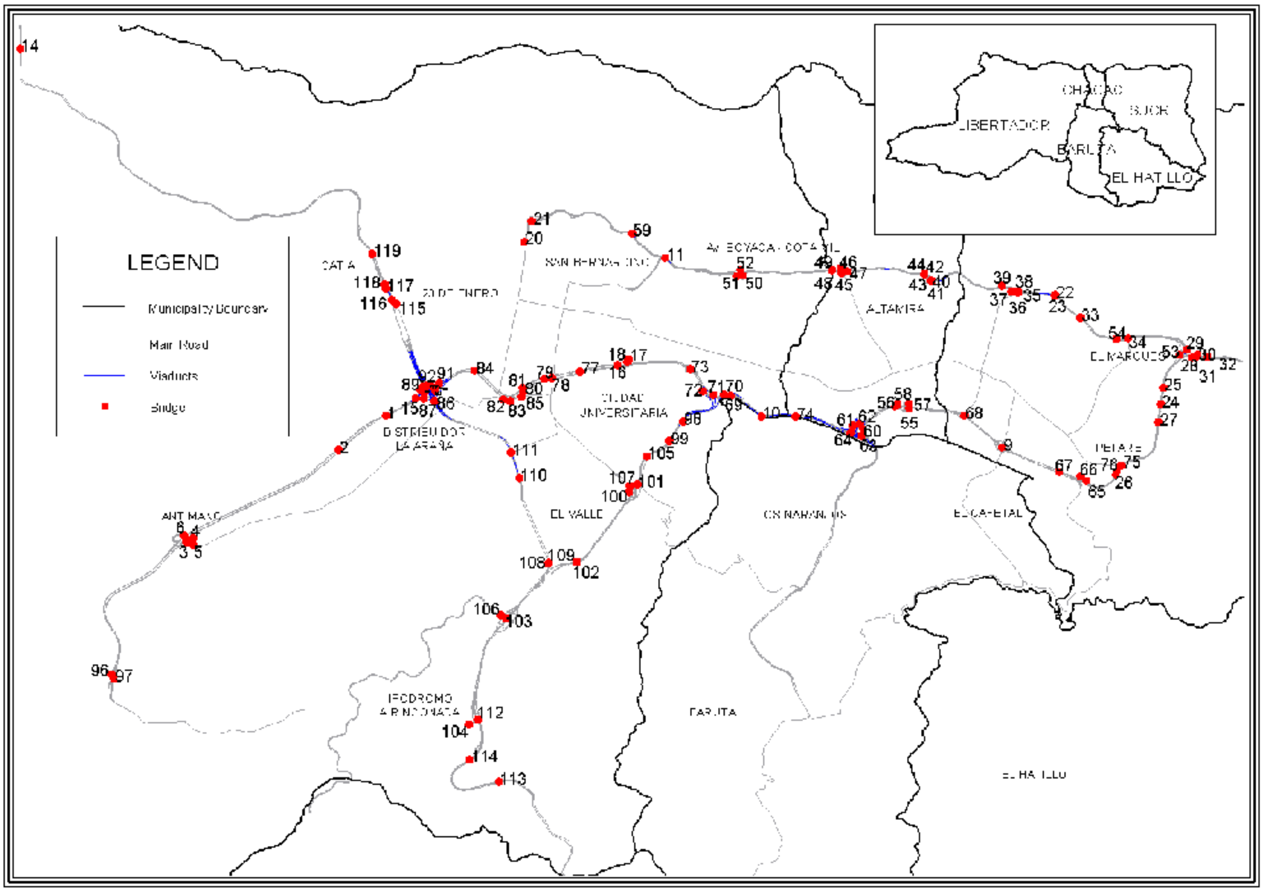
Municipality	Length of Network (km)						Population		Area		Network Density	
	Under-ground	Buried	Cable Box	Aerial	Total	Share (%)	"(1000)	Share (%)	(ha)	Share (%)	(m/p)	(m/ha)
Libertador	1,844	0	676	1,614	4,134	58	2,036	75	37,704	49	2.0	110
Chacao	320	0	160	10	490	7	71	3	1,885	2	6.9	260
Sucre	537	0	186	75	799	11	601	22	17,256	22	1.3	46
Baruta	854	1	297	73	1,224	17			8,317	11		147
El Hatillo	268	0	115	98	480	7			12,565	16		38
Total of Study Area	2,701	0	1,022	1,699	5,423	76	2,709	100	56,845	73	2.6	95
(share: %)	49.8	0.0	18.9	31.3	100.0							
G. Total	3,823	1	1,434	1,871	7,128	100			77,727	100		125
(share: %)	53.6	0.0	20.1	26.2	100.0							

Source: the obtained existing cable network table by service zone and type, CANTV 2003



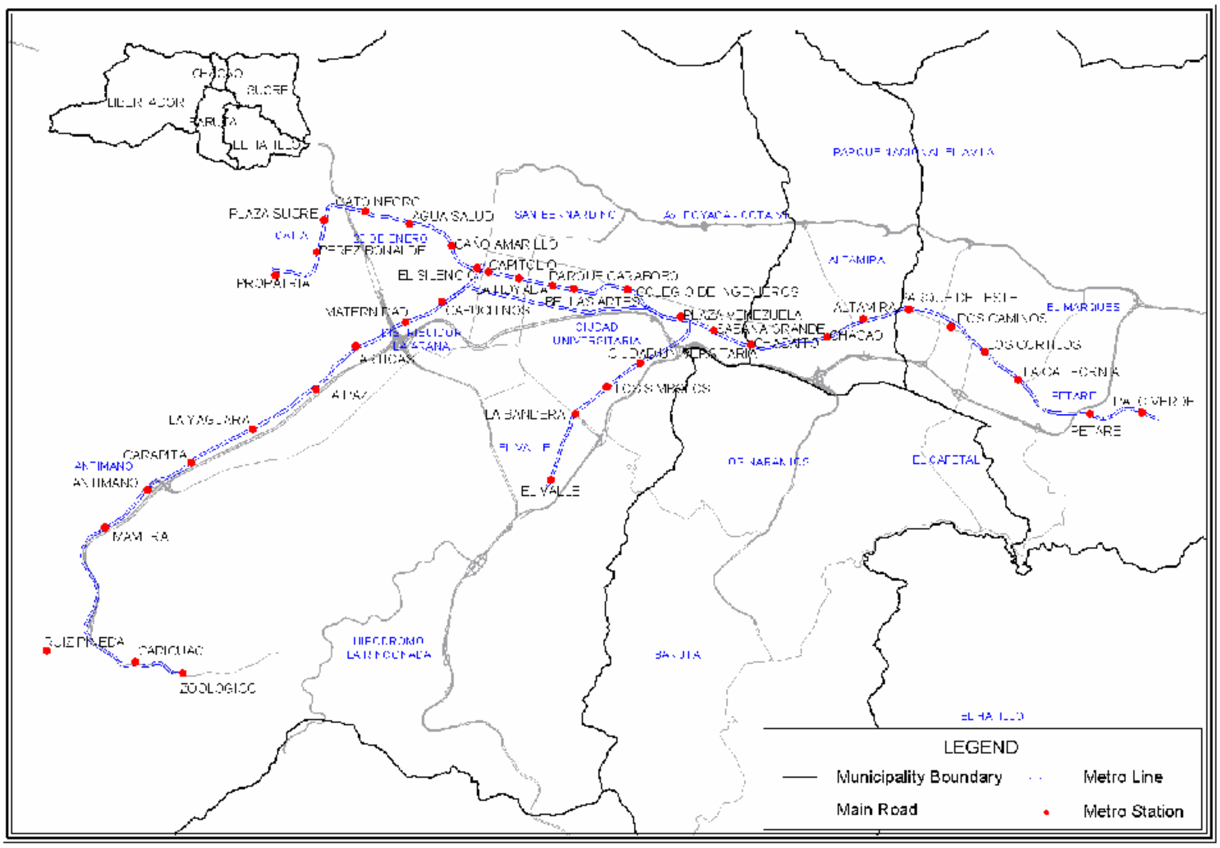
Source: The Feasibility Study of Metro Network Development

Figure S8-3.2.1 Digitized Road Network in the Metropolitan Caracas



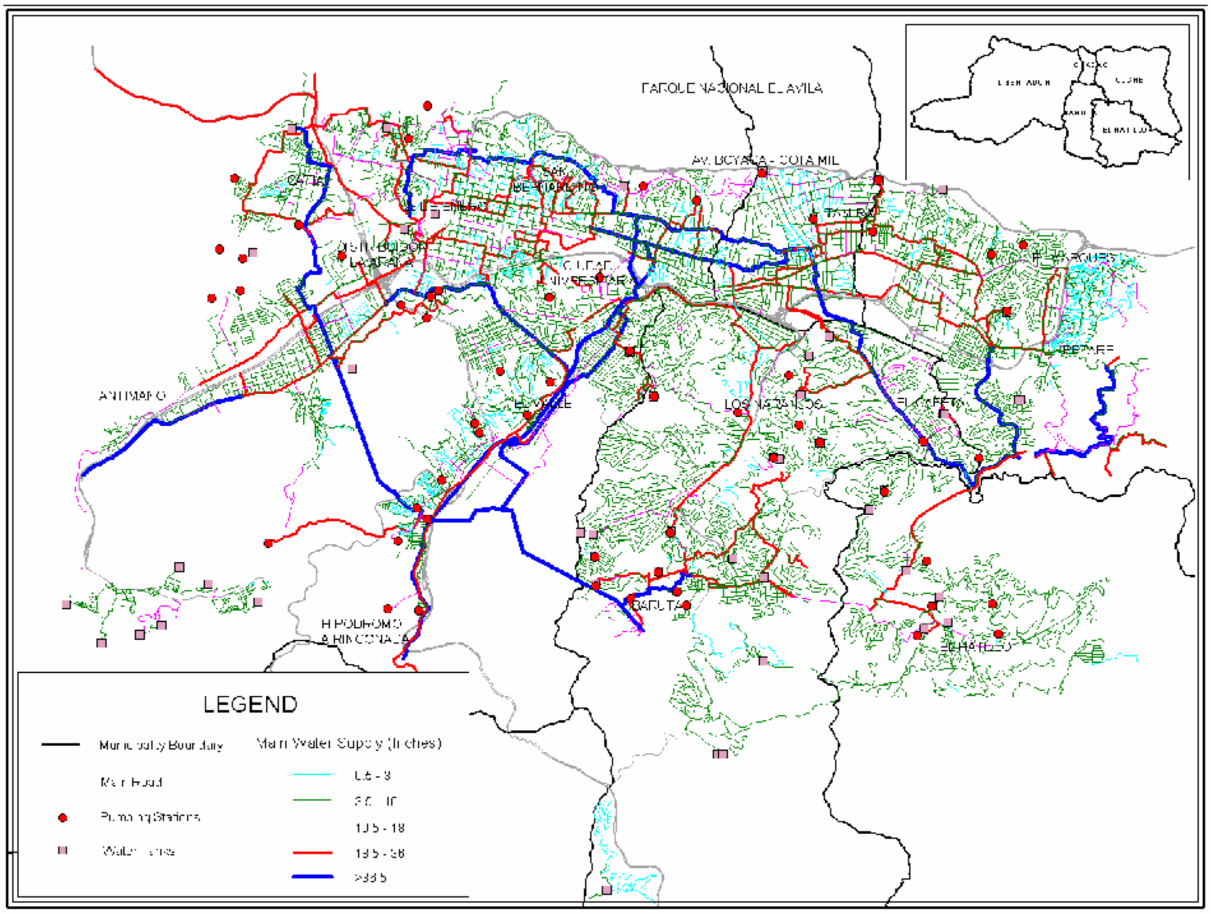
Source: Field Survey, JICA Study Team

Figure S8-3.2.2 Location Map of Listed Major Bridges and Viaduct



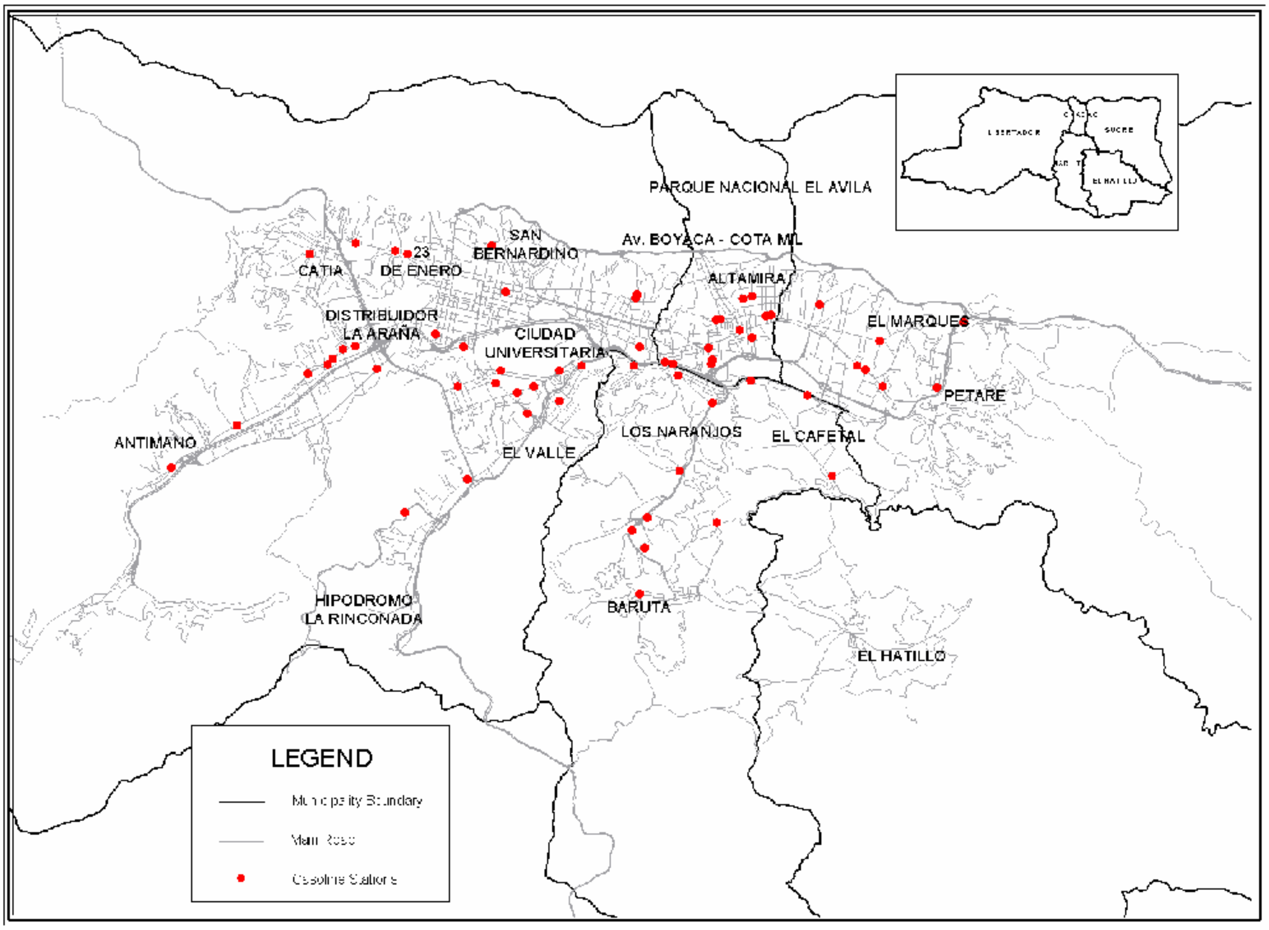
Source: Metro Company

Figure S8-3.3.1 Existing Metro Network in Caracas Metropolitan



Source: AutoCAD Water Supply Network Files of Hydrocapital and IMAS

Figure S8-3.4.1 Water Supply Pipeline Network in the Metropolitan Caracas



Source: Hazardous Material Division of Fire Fighting Department

Figure S8-3.9.1 Location Map of Gas Stations in the Metropolitan Area

CHAPTER 4. RECOMMENTATION FOR GIS DATABASE ESTABLISHMENT

4. 1 Road Network with Bridge

Establishment GIS road network database is proposed not only for assessment of urban disaster vulnerability, formulation and management of emergency road network, but also for daily traffic management.

4. 2 Water Supply Network

Establishment of water supply network data with material data is proposed not only to establish proper GIS Database for damage estimation, but also for formulation of emergency response plan and ordinal maintenance works.

4. 3 Natural Gas Supply Network

The risks of natural gas pipeline during earthquake are not limited to pipeline damages and malfunction of their service, but also include secondary disasters of explosion by spilled gas from damaged points. GIS based pipeline network database is proposed to utilize for estimation earthquake damages and formulation/operation of emergency response operation.

4. 4 Electric Power Supply Network

The risks of electric power supply networks is not limited to cable damages and malfunction of their service, but also include secondary disasters of fire out-breaks by damaged cable points. The establishment of GIS based cable network database is proposed to be utilized for estimation of earthquake damages and formulation/operation of emergency response operation.

4. 5 Telecommunication Network

Establishment of GIS Telecommunication Network Database is proposed not only for estimation of earthquake damages, formulation and management of emergency response operation, but also for daily data transmission management and maintenance works.

4. 6 Hazardous and Toxic Materials and Substance

Weak and inappropriate structure facilities of hazardous materials and products in urbanized area will be damaged and it will generate fire outbreaks, explosions, spreading toxic gas after an earthquake event. Those secondary disasters will also generate serious human casualties, but it will not be responded to and managed by ordinal emergency response forces under disordered disaster conditions.

In order to mitigate and avoid those secondary disaster, proper monitoring and control system for hazardous facilities is proposed to be established in the metropolitan area.

S9

LIFELINE/INFRASTRUCTURE DAMAGE PREVENTION

"Because the prevention of disasters is part of your life"

Antonio Aguilar M.

STUDY ON
DISASTER PREVENTION BASIC PLAN
IN THE METROPOLITAN DISTRICT OF CARACAS

FINAL REPORT

SUPPORTING REPORT

S9

LIFELINE/INFRASTRUCTURE DAMAGE PREVENTION

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S-9 LIFELINE/INFRASTRUCTURE DAMAGE PREVENTION

CHAPTER 1. INTRODUCTION

1. 1 General

The study area, Libertador, Chacao and Sucre in Caracas Metropolitan District, is located at the isolated valley where social and economic activity are supported by a wide road network and lifelines such as express highway, viaduct (elevated highway), water supply, gas supply, electric supply, telecommunication system, etc. The population of study area was about 2.7 million in 2001.

When a disastrous earthquake occurs near the study area, the road network and lifelines may experience serious damage that may cause physical malfunction of the city life. In order to secure and maintain the city functions of the Caracas Metropolitan District, it is indispensable to strengthen vulnerable infrastructures and lifelines against earthquakes.

Seismic damage estimations for infrastructure and lifelines in the study area were carried out and necessary countermeasures are recommended for strengthening the structures against earthquakes.

1. 2 Collected Data of Infrastructure and Lifeline

Data of infrastructure and lifelines of the study area were obtained from the related agencies or authorities; however, the collected data is quite limited mainly because data from private sector was not obtained. Therefore the seismic damage estimations could be made only for the collected data and the information available from the investigation at the site and commercial maps.

1. 3 Scenario Earthquake

Scenario earthquakes 1967 and 1812 are adopted for the seismic damage estimations. The details of each scenario are shown in Table S9-1.3.1.

Table S9-1.3.1 Scenario Earthquakes and their Parameters

Scenario	Mw	Seismogenic Depth	Fault Length	Mechanism	Fault system
1967	6.6	5 km	42 km	Strike slip	San Sebastian
1812	7.1	5 km	115 km	Strike slip	San Sebastian

CHAPTER 2. METHOD OF DAMAGE ESTIMATIONS

2. 1 Bridge

2. 1. 1. Assumptions

A statistical method based on Japanese experiences is adopted, since information on collapse of bridges in Venezuela is not recorded. The "point evaluation procedure" (i.e., multi-dimensional theory) was adopted. The result obtained from the "point evaluation procedure" describes what amount of damage to bridges may be expected at the time of an earthquake. If some bridges are estimated to collapse, a detailed seismic analysis should be undertaken as precise as the original design, and countermeasures should be taken to avoid serious damage by earthquake.

2. 1. 2. Procedures

The Express Highways in the Caracas Metropolitan Area connect the east-west and north-south areas. JICA study team surveyed bridges in the field which are located along the Express Highways.

The bridges were evaluated in terms of seismic damages according to an earthquake scenario. The study flow is shown in Figure S9-2.1.1.

2. 1. 3. Method of Damage Estimation

The criteria for seismic damage of the bridge is based on the method proposed by Tsuneo Katayama, which has been adopted in the Disaster Prevention Council of Tokyo Metropolitan Area (1978), and is widely used in Japan for practical purposes. This method only evaluates bridge collapse due to the superstructure falling down, but not damages (widespread damages and slight damages, etc.) regarding all structural members.

The following items are taken into account for evaluation:

- Ground type, Liquefaction, Girder type, Number of spans
- Bearing type (shoe type), Minimum bridge seat length
- Maximum height of abutment and pier
- Foundation type, Material of abutment and pier
- Peak Ground Acceleration (Earthquake intensity scale)

Estimated seismic damage is expressed as a total score. Stability judgment of bridges is defined as shown in Table S9-2.1.1.

The score regarding each item is shown in Table S9-2.1.2.

2. 2 Viaduct (Elevated Highway)

According to the Hanshin/Awaji Disaster (M7.2, 1995), only a few general bridges crossing over river/road collapsed, but many viaducts in express highway such as multi-span type collapsed.

The rate of collapses and damages in the Hanshin/Awaji Disaster are shown in Tables S9-2.2.1 and S9-2.2.2.

The Disaster Prevention Council in the Tokyo Metropolitan Area analyzed the Hanshin/Awaji Disaster data in the Table S9-2.2.1 and adopted the damage ratio per km in the table regarding the multi-span viaduct for estimation of seismic damage (1997).

There are some multi-span viaducts, which are in the Express Highway, in Caracas Metropolitan Area. JICA study team will apply the same damage ratio per km as proposed by the Tokyo Metropolitan Government in this project.

2. 3 Metro

The underground structure is rather stable against earthquakes compared with the structure on the ground due to the less seismic force under the ground. But those structures constructed by the cut and cover tunnel (Figure S9-2.3.1) will be affected due to the embankment on the structure..

Damage of the subway tunnel in the Hanshin/ Awaji Disaster is shown in Table S9-2.3.1.

In the case of the Hanshin/Awaji Disaster, some 2-cell reinforced concrete type boxes were collapsed by vertical motion of the overlying soil on the box.

2. 4 Water Supply Pipeline

The facilities of the water supply network are shown in Figure S9-2.4.1.

2. 4. 1. Assumptions

The basic assumptions applied for damage estimation of water supply pipelines are as follows:

A statistical approach for damage estimation for city main pipes, distribution pipes and service pipes are applicable only when information on their materials, diameter, lengths is available in any given area.

In the study, assumptions are as follows:

- Node facilities such as inlet facility, water purification plant, and transmission pipe are not included for damage estimation. In this study, the subject facilities are water pipe, distribution pipe (main and small), and service pipe. The individual diagnosis should be made on such node facilities to evaluate the safety against earthquakes.

- Damage due to the direct result of ground motion is estimated, such as breakage or disjoint of pipelines. Such damages caused by landslides or building collapses, so called secondary damages, are not included.

- The damage estimation method is based on the past damage experiences in Japan.

- In case proper data is not available, input data is set based on reasonable assumptions.

2. 4. 2. Method of Damage Estimation

The characteristics of water supply networks and pipeline structures are considered similar to those of Japan. Therefore, an analysis method for the damages estimation of water pipelines proposed by Disaster Prevention Council of the Tokyo Metropolitan Government was applied to the study, taking account of the experience in the Hanshin/Awaji Earthquake Disaster (Figure S9-2.4.2).

The standard damage ratio R_1 for water pipeline proposed by the Tokyo Disaster Prevention Council (1997) has been commonly used to evaluate seismic damages of water pipelines in Japan. The damage ratio for pipeline N_d is defined as follows:

$$N_d = C_1 \cdot C_2 \cdot C_3 \cdot R_1 \cdot L$$

Where: N_d: damage ratio (damage point/km)

C1 : correction factor for liquefaction (Table S9-2.4.2).

C2 : correction factor for pipe material (Table S9-2.4.1).

C3 : correction factor for pipe diameter (Table S9-2.4.1).

R_1: standard damage ratio (damage point/km).

$R_1=2.24\times 10^{-3}\,(PGV-20)^{1.51}$

PGV: peak ground velocity (cm/sec).

The curve of standard damage ratio is shown in Figure S9-2.4.3.

2. 5 Natural Gas Pipe Line

The facilities of the natural gas network are shown in Figure S9-2.5.1.

2. 5. 1. Assumptions

Assumptions are basically same as the case of the Water Supply Pipe Line.

2. 5. 2. Methods of Damage Estimation

Damage estimation regarding gas pipe lines is based on the data of the Hanshin/Awaji Disaster in Japan. The Standard Damage Ratio is set for the relation between peak ground velocity and standardized steel pipe, and then the modification of the damage ratio is made according to pipe materials, diameter and liquefaction. This method was applied by Disaster Prevention Council in the Tokyo Metropolitan Area (1997). The damage ratio for pipe line, Nd is defined as follows:

$$Nd = C_1 \cdot C_2 \cdot R \cdot L$$

Where: C_1 : correction factor for liquefaction.
C_2 : correction factor for pipe material.
R : standard damage ratio (damage point/km).
L: Pipe-line extension in Total (km)

The standard damage ratio is:

$$R = 3.89 \times 10^{-3} \times (PGV-20)^{1.51}$$

Where: PGV: peak ground velocity (cm/sec).

The correction factors are shown in Tables S9-2.5.1 and S9-2.5.2.

The curve of standard damage ratio is shown in Figure S9-2.5.2.

2. 6 Electric Power Supply

Electric Power Supply Network is shown in Figure S9-2.6.1.

The subject facilities for seismic damage estimation are electric poles and underground electric cables as shown Figure S9-2.6.1.

2. 6. 1. Assumptions

- Assumptions are basically the same as the case of Water Supply Pipe Line.

- Damage of an electric pole means collapse such as falling down or damaged severely.

2. 6. 2. Method of Damage Estimation

1) The seismic damage of an electric power pole is evaluated based on the Hanshin/Awaji Disaster in Japan. The number of collapsed poles Ndp is defined as follows:

$$Ndp = C_1 \times R/100 \times N$$

Where: C_1: correction factor by liquefaction (Table S9-2.6.3).
R: damage ratio (Table S9-2.6.1).
N: number of poles in total

Damage ratio is assumed to be the same as Hanshin/Awaji Disaster.

2) Seismic damage of underground structure such as buried electric power line is calculated as follows:

$$Nd = C_1 \times R/100 \times L$$

Where: Nd: extension of damage (km)
C_1: correction factor by liquefaction (Table S9-2.6.3).
R : damage ratio (Table S9-2.6.2).
L : extension in total (km)

2. 7 Telecommunication Cables

The method of seismic damage estimation is the same as Electric Power Lines.

2. 8 Hazardous Facility

Damage functions of hazardous facilities on the Seismic Micro-zoning Study of Tokyo Metropolitan Government are used in the statistical analysis of past earthquake ground motion (PGA) with identified damaged of certain categories of hazardous facility by the Tokyo Metropolitan Fire Fighting Department.

The category of hazardous facility, type of damage, and damage ratio by PGA are shown in the Table S9-2.8.1.

Table S9-2.1.1 Stability Judgment of Bridges

Stability	Total Score
High Seismic Risk	30 ≦ S
Medium Seismic Risk	26≦ S ＜30
Low Seismic Risk	S＜26

Table S9-2.1.2 Seismic Damage Evaluation Factor

Item	Category		Score	Note
(1) Ground type	Stiff		0.5	The ground classification depends on the division of “Road Bridge Design for Earthquake-proof Indicator”
	Middle		1.0	
	Soft		1.5	
	Very Soft		1.8	
(2) Liquefaction	None		1.0	Depends on the Formula for “Road Bridge Design ”
	Possible		1.5	
	Probable		2.0	
(3) Girder type	Arch/ Rigid Frame		1.0	
	Continuous		2.0	
	Simple/ Gelber		3.0	
(4) Bearing	Connection Device		0.6	F : Fix support M : Movable Support
	F・M		1.0	
	M・M		1.15	
(5) Max Height of Abutment/Pier	≤5m		1.0	Height is the maximum value from ground level
	5～１0		Interpolated	
	≥10m		1.7	
(6) Number of Spans	＝１		1.0	
	≥２		1.75	
(7) Bridge Seat Length	Wide（A/S≥１）		0.8	A=(Seat Length)cm S=(70+0.5L) cm L=Span Length (m) Ground type (very soft) D=A/70 Ground type (others) D=A/60
	Narrow（A/S<１）		1.2	
	Gelber bearing on pier cap	D≥１	0.8	
		D<1	1.2	
(8) Earthquake Intensity Scale	5*	120~209 gal	1.0	Mark * means earthquake intensity scale in Japan.
	5.5*	210~349 gal	1.7	
	6*	350~699gal	2.4	
	6.5*	700~1299 gal	3.0	
	7*	1300~3299 gal	3.5	
(9) Foundation Type	Excluding Pile Bent		1.0	1.4 for obviously weak foundation such as friction piles
	Pile Bent		1.4	
(10) Material of Abutment/Pier	Brick/ Plain concrete		1.4	
	Not listed above		1.0	
Total score	= (1)×(2)×(3)×(4)×(5)×(6)×(7)×(8)×(9)×(10)			

Table S9-2.2.1 Seismic Damage of Viaduct in Express Highway

Earthquake Intensity*	Collapsed	Damage of Bearing Shoe	Damage of Pier	Viaduct Extension (km)	Rate of Collapse (place/km)	Rate of Damage (place/km)
7	19	-	1	18.8	1.010	0.053
6+	5	5	7	58.2	0.086	0.206
6-	1	1	4	347.3	0.003	0.014
Total	25	6	12	424.3	-	-

Note : 6+ means 6.0≦6+<6.5 and 6- means 5.5≦6-<6.0 (Japanese Intensity Scale)

Table S9-2.2.2 Seismic Damage of Bridges (Not Express Highway)

Earthquake Intensity*	Collapse	Displacement of Girder and Pier	Damage of Abutment and Bearing shoe	Damage of Pier	Cracks on the Pier Stem
7	1	-	-	1	-
6+	1	3	5	1	-
6-		-	6	4	2
Total	2	3	11	6	2

Note : 6+ means 6.0≦6+<6.5 and 6- means 5.5≦6-<6.0 (Japanese Intensity Scale)

Table S9-2.3.1 Seismic Damage of Subway Structure in Hanshin/Awaji Disaster

Inten-sity*	Open Cut Type Tunnel			Mountain Tunnel	Shield Type
	Middle Column Collapse	Side Wall Damage	Other	Damage at Lining	
7	Hanshin Railway: 344 piece Kobe City Trans.: 457 piece Kobe express: 362 piece Kobe Railway: 59 piece Sanyo Railway: 36 piece	HanshinRailway: 3365 m Kobe express: 595 m Kobe Railway: 14 m	-	Rokkou T. Higashiyama T. Kaishimoyama T	No Damage
6+	Sanyo Railway : 1 piece	-	-		
6-	-	Kobe Railway 84m	-	Kikusuiyama T Arima T. Gosha T. Kitakami T	

Note: * Japanese Seismic Intensity Scale

Table S9-2.4.1 Correction Factor for (C_2) and (C_3)

Pipe material	Correction factor C_2	Correction factor C_3	
Ductile cast iron	0.3	$C_3 \leqq 75$mm	2.0
		100mm $\leqq C_3 \leqq$ 450mm	1.0
		500mm $\leqq C_3 \leqq$ 900mm	0.3
		1000mm $\leqq C_3$	0.15
Cast iron	1.0	$C_3 \leqq 75$mm	1.7
		100mm $\leqq C_3 \leqq$ 250mm	1.2
		300mm $\leqq C_3 \leqq$ 900mm	0.4
		1000mm $\leqq C_3$	0.15
Welded steel pipe	0.3	$C_3 \leqq 75$mm	2.8
		100mm $\leqq C_3 \leqq$ 250mm	1.4
		300mm $\leqq C_3$	0.8
Chloroethlene	1.5	$C_3 \leqq 75$mm	1.0
		100mm $\leqq C_3$	0.8
Asbestos	3.0	$C_3 \leqq 75$mm	2.3
		100mm $\leqq C_3 \leqq$ 250mm	0.9
		300mm $\leqq C_3$	0.4

Table S9-2.4.2 Correction Factor for Liquefaction (C_1)

Liquefaction potential	Correction factor C_l
PL=0	1.0
$0 < PL \leqq 5$	1.2
$5 < PL \leqq 15$	1.5
$15 < PL$	3.0

Table S9-2.5.1 Correction Factor for Liquefaction (C_1)

PL value	C_1
PL=0	1.0
$0 < PL \leqq 5$	1.2
$5 < PL \leqq 15$	1.5
$15 < PL$	0.068

Table S9-2.5.2 Correction Factor for Pipe Materials (C_2)

Pipe material		Correction factor C_2
Middle Pressure	Steel	0.01
	Cast iron	0.02
Low Pressure	Steel (welded)	0.02
	Steel (bolt)	1.00
	Steel (mechanical)	0.02
	Ductile cast iron (joint 1)	0.46
	Ductile cast iron (joint 2)	0.23
	Ductile cast iron (Gas type)	0.05
	Ductile cast iron (mechanical type)	0.02
	Polyethylene	0.00
	Polyvinyl chloride pipe	0.70

Table S9-2.6.1 Damage Ratio for Electric Poles

Intensity*	R (%)
Less 5	0.00
6	0.55

*Earthquake Intensity Scale in Japan

Table S9-2.6.2 Damage Ratio for Electric Lines

Intensity*	R (%)
Less 5	0.00
6	0.30

*Earthquake Intensity Scale in Japan

Table S9-2.6.3 Correction Factor for Liquefaction

PL value	C1
PL=0	1.0
0 <PL ≦5	1.1
5 <PL ≦15	1.3
15 <PL	2.1

Table S9-2.8.1 Category of Hazardous Facility, Type of Damage and Damage Ratio of Tokyo Metropolitan Area

Category of Hazardous Facility	Type of Damage	PGA										
		100	150	200	250	300	350	400	450	500	550	600
1. Large storage tank of flammable Liquid	1. small spill from tank and pipe joint	4.10E-05	1.50E-04	4.90E-04	1.40E-03	3.30E-03	6.90E-03	1.30E-02	2.00E-02	3.00E-02	3.80E-02	4.70E-02
	2. continuous certain volume of spill	1.00E-05	3.80E-05	1.20E-04	3.40E-04	8.20E-04	1.70E-03	3.20E-03	4.90E-03	7.50E-03	9.40E-03	1.20E-02
	3. overflow from protection dike	2.40E-06	8.90E-06	2.90E-05	8.00E-05	1.90E-04	4.00E-04	7.40E-04	1.10E-03	1.70E-03	2.20E-03	2.80E-03
	4. fire outbreak of oil in protection dike	1.00E-06	3.80E-06	1.20E-05	3.40E-05	8.20E-05	1.70E-04	3.20E-04	4.90E-04	7.50E-04	9.40E-04	1.20E-04
	5. large fire spreading on tank-yard	2.40E-07	8.90E-07	2.90E-06	8.00E-06	1.90E-05	4.00E-05	7.40E-05	1.10E-04	1.70E-04	2.20E-04	2.80E-04
2. Tanks and gas-holder of flammable gas	6. spill from pipe joint to tank (emergency shut-down)	1.50E-05	4.20E-05	1.10E-04	2.50E-04	5.60E-04	1.10E-03	2.20E-03	3.70E-03	6.30E-03	9.50E-03	1.40E-02
	7. continuous spill of certain volume (hazard of explosion)	3.80E-06	1.00E-05	2.70E-05	6.30E-05	1.40E-04	2.80E-04	5.40E-04	9.20E-04	1.60E-03	2.40E-03	3.50E-03
	8. fire outbreak of spilled gas in protection dike	3.80E-07	1.00E-06	2.70E-06	6.30E-06	1.40E-05	2.80E-05	5.40E-05	9.20E-05	1.60E-04	2.40E-04	3.50E-04
	9. explosion of large spilled gas	3.80E-08	1.00E-07	2.70E-07	6.30E-07	1.40E-06	2.80E-06	5.40E-06	9.20E-06	1.60E-05	2.40E-05	3.50E-05
3. Tank of toxic gas/ liquid nitrogen	10. spill from pipe joint of tank	3.00E-06	8.40E-06	2.10E-05	5.10E-05	1.10E-04	2.30E-04	4.30E-04	7.40E-04	1.30E-03	1.90E-03	2.80E-03
	11. continuous spill of certain volume (hazard for citizen)	7.60E-08	2.10E-07	5.30E-07	1.30E-06	2.80E-06	5.70E-06	1.10E-05	1.80E-05	3.20E-05	4.70E-05	7.10E-05

Source: Damage ratio of hazardous facility on the Seismic Micro-zoning Study of Tokyo Metropolitan Government, 1997

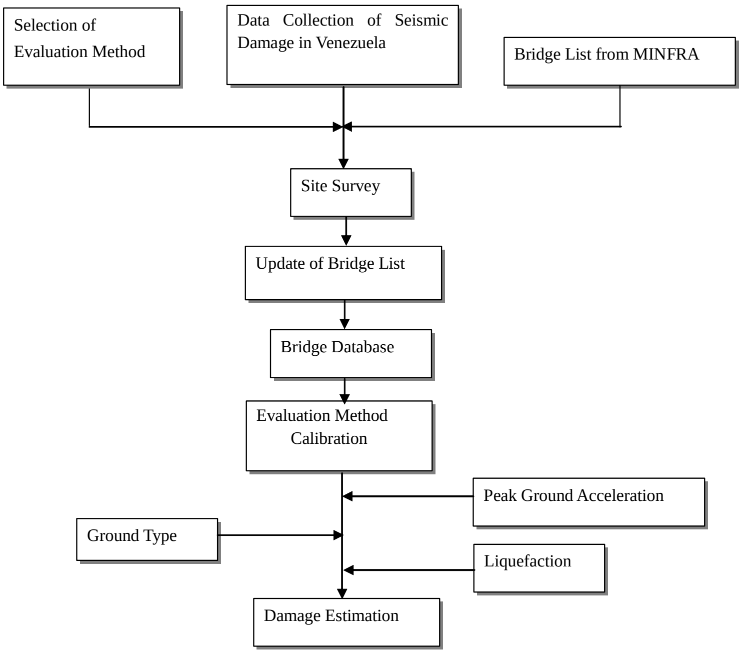


Figure S9-2.1.1 Procedure of Seismic Damage Estimation for Bridges

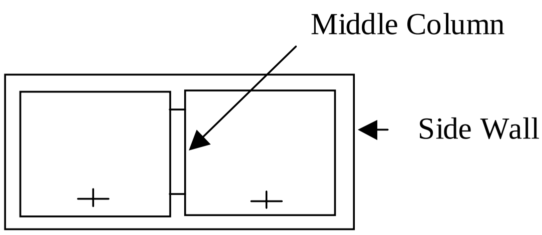


Figure S9-2.3.1 Cut and Cover Type Tunnel

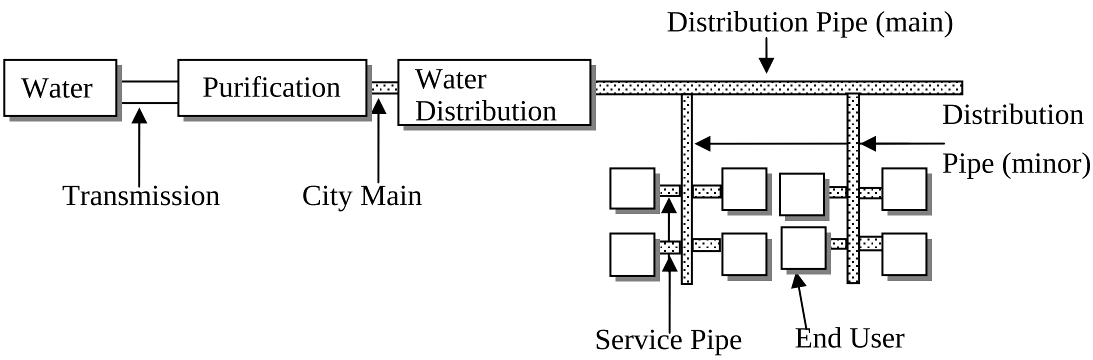


Figure S9-2.4.1 Water Supply System

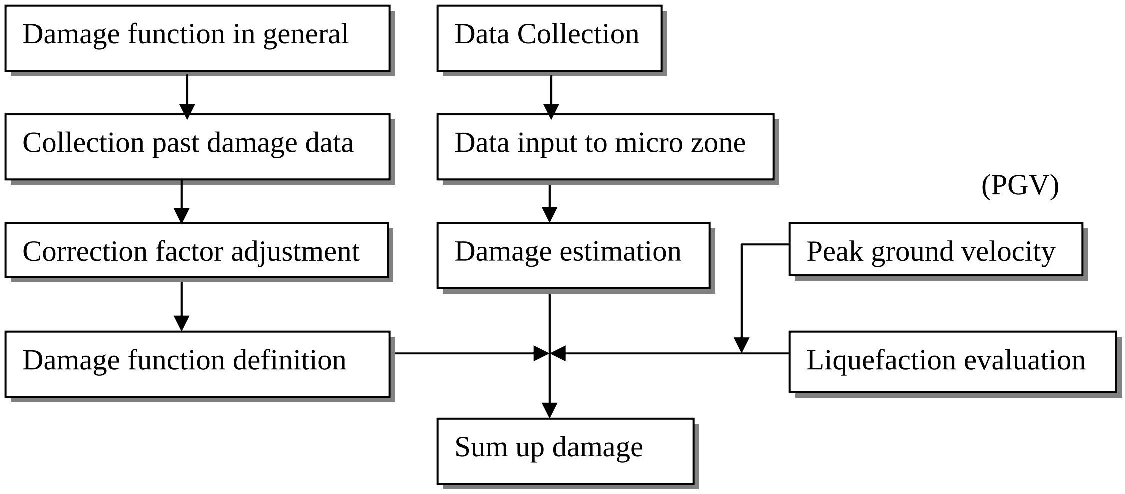


Figure S9-2.4.2 Flow Chart of Damage Estimation for Water Supply

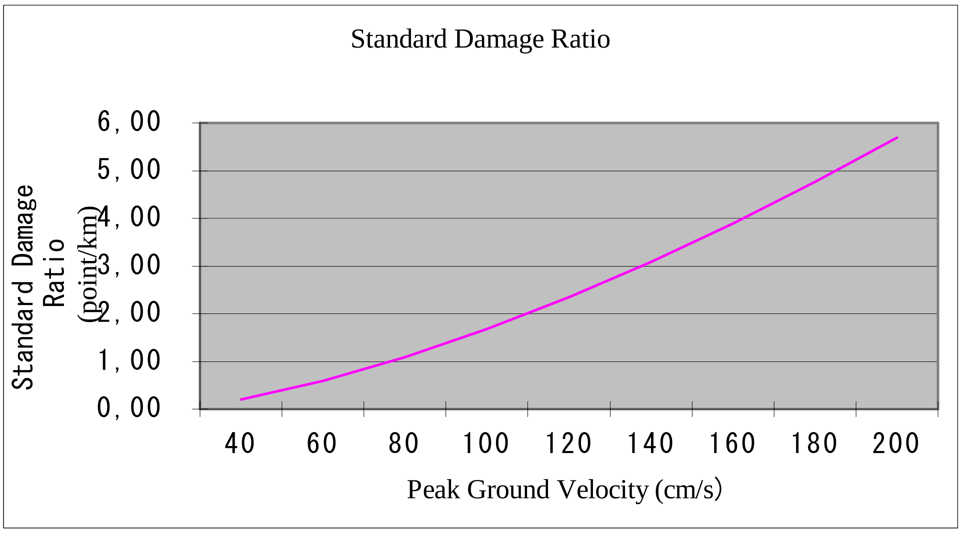


Figure S9-2.4.3 Standard Damage Ratio

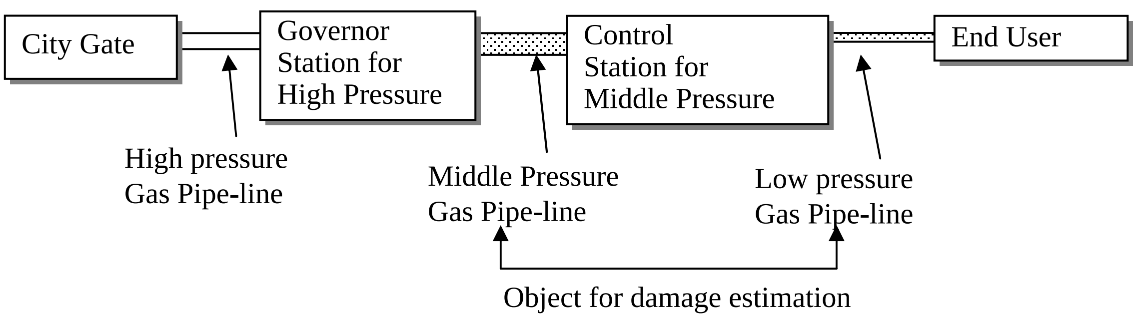


Figure S9-2.5.1 Natural Gas Pipe Line Network

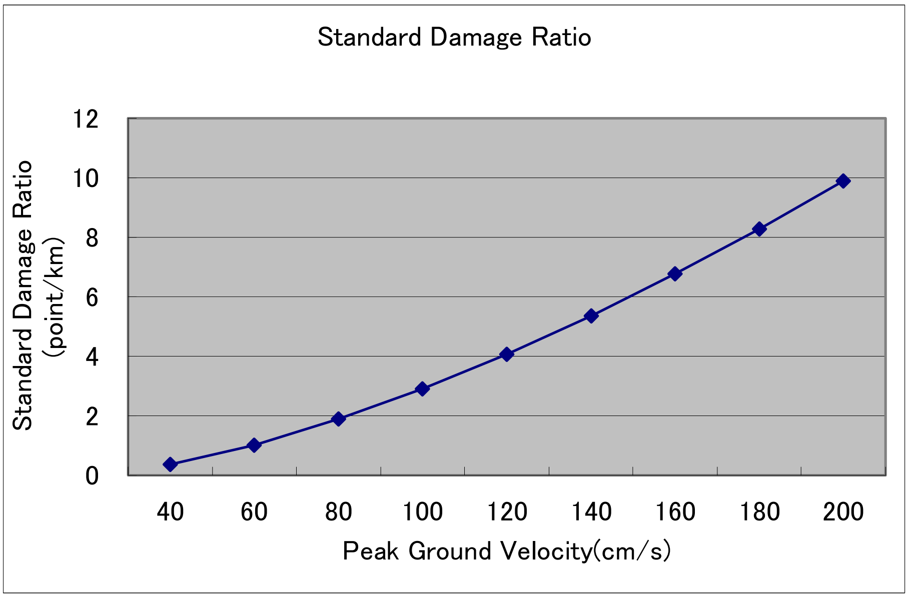


Figure S9-2.5.2 Standard Damage Ratio for Gas Pipe Line

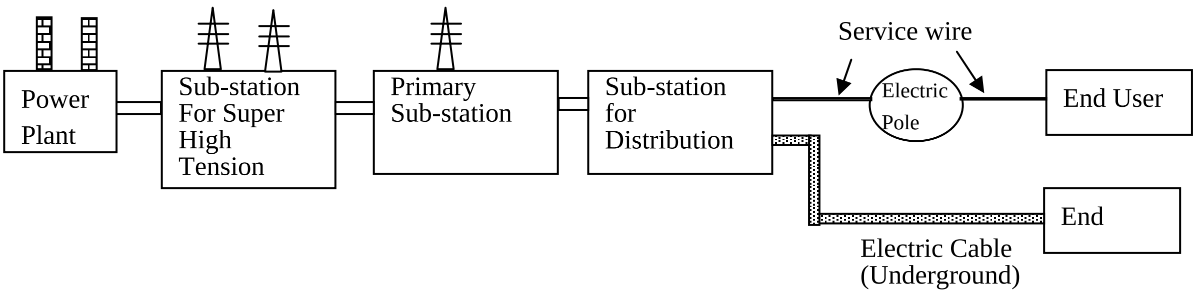


Figure S9-2.6.1 Electric Power Supply Network

CHAPTER 3. RESULT OF DAMAGE ESTIMATIONS

3. 1 General

Seismic damage estimations were made for the infrastructures and lifelines based on data obtained, but quite limited from relevant agencies/authorities and site investigation.

The collected data is as follows:

1) Bridge
2) Viaduct (Elevated Highway)
3) Metro
4) Water Supply Pipeline
5) Telecommunication Line
6) Hazardous Facility (Gasoline Station)

3. 2 Bridge

115 bridges on the express highway were selected for the seismic damage estimation in consideration of the significance of emergency activity for rescue and transportation at the time of earthquake occurrence.

Most of bridges were constructed before 1967 and no serious damage was reported when an earthquake occurred in 1967 except one minor damage of the pier at the interchange Pulpo.

The results of damage estimation of bridges indicate the existing bridges are strong enough against the scenario earthquake 1967 and the damage estimation also shows the same result.

In case of the scenario earthquake 1812, 15 bridges are estimated as a high seismic risk and 2 bridges estimated as a medium seismic risk to collapse when such scale of earthquake occurs. The details of those bridges and location are shown in Tables S9-3.2.1 to S9-3.2.4 and Figure S9-3.2.1.

Among 15 bridges estimated as a high seismic risk, 10 bridges are located at the interchange Arana which is on sedimentary deposit and susceptible for liquefaction. The interchange Arana is the biggest interchange in Caracas which was opened to the traffic in 1966 and the height of bridge is more than 10 m at the center for crossing. This interchange plays an important role for the transportation for east-west and south-north direction. The securing of this interchange is vital for the social and economic activities in Caracas city.

3. 3 Viaduct (Elevated Highway)

Seismic damage estimation was made for the viaduct (elevated highway) referring to the experience of Hanshin/Awaji Disaster data 1995 in Japan. Due to the estimation, two locations may collapse and three locations may get damage at interchange Arana.

Damage Estimation, earthquake intensity and its length of Viaduct are shown in Table S9-3.3.1. and each viaduct location is shown in Figure S9-3.3.1.

At the interchange Arana, the flyovers were constructed in 1966 and old seismic code was applied to the design. There are three flyovers constructed at the center of interchange Arana and the height of structure is more than 10 m and the structure may be easily affected by the earthquakes.

It is recommended to investigate the design code applied to the bridges and on the basis of the design code, it is required to take a countermeasures to strengthen the structures against the earthquake.

3. 4 Metro

There are three Metro lines in Caracas Metropolitan District and their total length is 44.3 km. An outline of Metro is shown in Table S9-3.4.1 and its location and open cut and box type tunnel location is shown in Figure S9-3.4.1.

Line 1 : Peak Ground Acceleration (PGA) is estimated Max. 581 gal at the station between Capitolio and Chacaito (about 5.8 km) in case of scenario earthquake 1812. This PGA is equivalent to Japan Meteorological Intensity 6+.

In case of Hanshin/Awaji Disaster, middle columns were collapsed due to the extra vertical force by the earthquake. Especially the weight of embankment is considered to apply to the tunnel structure vertically. It is recommended to check the design and the type of tunnel structure and strengthen the middle column in consideration of extra vertical force on the tunnel.

Line 2 : PGA is estimated Max. 721 gal at the station of Antimano. The open and cut box type tunnel between Artigas and Mamera is recommended to reinforce at the middle column in consideration of scenario earthquake 1812.

Line 3 : PGA is estimated Max. 409 gal at the Box Type tunnel in scenario earthquake 1812. This PGA is equivalent to JMI 6- and no damage of middle column collapse was recorded in Hanshin/Awaji disaster. But the damage of Metro in Caracas may be different in accordance with the embankment thickness on the box tunnel. It is recommended to check the design and strengthen the middle column if the middle column is not strong enough against the vertical force to the tunnel.

Damage against shield tunnel of Metro in Hanshin/Awaji disaster was not reported and it showed shield tunnel is a very strong structure against earthquakes.

3. 5 Water Supply

No information or material is available, therefore seismic damage estimation was carried out on the assumption that the material would be ductile cast iron. Recently the water supply authority is promoting the policy to gradually use ductile cast iron for water supply piping.

The damage estimation is shown in Figure S9-3.5.1 in scenario earthquake 1812.

According to the damage estimation, no damage is expected in scenario earthquake 1967. In case of scenario earthquake 1812, the maximum estimated damage number of point per mesh (500 x 500 m) is only 0.56 points.

The most affected area are Neveri and Sanpedro which locations are shown in Figure S9-3.5.1., but the estimated damage points are quite small.

But this estimation is based on the assumption that all material of pipe are made of ductile cast iron. Ductile cast iron is strong against the earthquakes. It is recommended to continue to promote the policy to use the ductile cast iron piping.

3. 6 Telecommunications

In case of scenario earthquake 1967, most of the earthquake intensity is equal to or less than 5 of Japan Meteorological Intensity (JMI) and the possible damage is only 0.07% against the total length. In case of scenario earthquake 1812, 0.25% of total telecommunication cable may be damaged.

The length of damage estimation of telecommunication in each area is shown in Table S9-3.6.1.

3. 7 Hazardous Facility (Gasoline Station)

Total 54 gasoline stations are located in the study area and their locations are shown in Figure S9-3.7.1.

Scenario earthquake 1967 : Estimated Max. PGA is less than 250 gal and the probability of small spill from tank and pipe joint is only 0.14% in accordance with the study of Tokyo Metropolitan Government,1977 and no damage is anticipated.

Scenario Earthquake 1812 : Estimated Max. PGA is 400~450 gal and there are 13 gasoline stations in that area. The probability of small spill from tank and pipe joint is only 2.00% in accordance with the study of Tokyo Metropolitan Government,1977 and also the damage is quite small.

Even considering all area, the number of affected gasoline stations is less than one location.

The Max. PGA area and rather density built-up area of gasoline station are shown in Table S9-3.7.1.

Gasoline stations located at the high acceleration area should be improved as seismic resistant structures.

The number of gasoline stations in accordance with the PGA are shown in Figures S9-3.7.2. and S9-3.7.3.

In case of scenario earthquake 1967, the PGA of location of gasoline station is under 200 gal. But in case of scenario earthquake 1812, the PGA is higher and the figure is showing that many gasoline stations are located at the high PGA area.

Table S9-3.2.1 List of Bridges Estimated Risk A and B

Level of Risk	Code No.	Name or Number of Bridge	Name or No of Road	Name or number of crossing road/river/metro	Year of Built before'87 : 1 unknown : 2 after '87 : 3
A	61	Dist. Ciempies, Pte. S/Autopista enlace Norte-Sur	Rampa de entrada Autopista del Este desde Chacao	Autopista Fco. Fajardo (2 vías)	1
	62	Dist. Ciempies, Pte. S/Autopista enlace Sudeste-Oeste	Salida a Autopista Fco. Fajardo sentido Oeste desde Autopista del Este	Salida a las Mercedes desde Chacao ida y vuelta (2 vías)	1
	63	Dist. Ciempies, Pte. S/Autopista enlace Sudeste-Este	Salida desde Autopista del Este hacia Chacao	Salida a las Mercedes desde Chacao ida y vuelta (2 vías)	1
	82	Dist. Baralt, Pte.Oeste	Entrada desde Av. Baralt hacia el Paraiso (1 vía)	Autopista Fco. Fajardo ambos sentidos (2 vías) y Río Guaire	1
	83	Dist. Baralt, Pte.Este	Entrada desde la Av. Baralt hacia Autopista sentido Este (1 vía)	Autopista Fco. Fajardo ambos sentidos (2 vías) y Río Guaire	1
	86	Dist. La Araña, Pte. Paraiso-Planicie	Salida desde Planicie dirección El Paraiso (1 vía)	Autopista Fco. Fajardo ambos sentidos (2 vías)	1
	87	Dist. La Araña, Pte. Caricuao-Paraiso	Vía Caricuao-Paraiso (1 vía)	una (1 vía)	1
	88	Dist. La ArañA, Pte. Paraiso-Qta. Crespo	Vía Qta. Crespo-Paraíso (1 vía)	Autopista Fco. Fajardo ambos sentidos (2 vías)	1
	89	Dist. La Araña, Pte. Caricuao-Planicie	Vía Caricuao-Planicie (1 vía)	Autopista Fco. Fajardo ambos sentidos (2 vías) y entrada Barrio (1 vía)	1
	90	Dist. La Araña, Pte. Qta. Crespo-Planicie 1	Vía Qta Crespo-Planicie (1 vía)	Paralela una vía del Dist. La Araña	1
	91	Dist. La Araña, Pte. Qta. Crespo-Planicie 2	Vía Planicie-Qta Crespo (1 vía)	Autopista Fco. Fajardo ambos sentidos (2 vías)	1
	92	Dist. La Araña, Pte. Planicie-Caricuao	Vía desde Planicie 1 hacia Caricuao (1 vía)	Paralela una vía del Dist. La Araña	2
	93	Dist. La Araña, Pte. Planicie 2-Qta.Crespo	Vía Planicie-Qta. Crespo (1 vía)	Autopista Fco. Fajardo ambos sentidos (2 vías)	1
	94	Dist. La Araña, Pte. Qta. Crespo-Paraíso	Vía Qta. Crespo-El Paraíso (1 vía)	Río Guaire	1
	95	Dist. La Araña, Pte. Planicie 2-Caricuao	Vía Planicie-Caricuao (1 vía)	Paralela una vía del Dist. La Araña	1
B	15	Puente Santander (Puente Lara)	Avenida Santander	Autopista Francisco Fajardo, Rio Guaire	1
	98	Pte. Ricardo Zuluaga	Vía Sta. Mónica-Los Chaguaramos ambos sentidos (2 vías)	Autopista Valle-Coche ambos sentidos (2 vías) y Río Guaire	1

Risk A: High Seismic Risk Risk B: Medium Seismic Risk

Table S9-3.2.2 Bridge Damage Estimation in Case of Earthquake Scenario 1967

Code No.	(1) Ground Type	(2) Liquefaction	(3) Girder Type	(4) Bearing Type	(5) Max. Height of Abut./Pier	(6) Number of Spans	(7) Min. Bridge Seat Width	(8) PGA *Japanese Earthquake Intensity	(9) Foundation Work	(10) Materials of Abutment and Pier	Total Score	Code No.
	Stiff : 0.5	None : 1.0	Rigid : 1.0	girder connection device :0.6	Less 5m : 1.0	one span : 1	wide : 0.8	5* 120-209 : 1.0	Pile bent:1.4	Brick : 1.4		
	Middle : 1.0	Possible : 1.5	Continuous : 2.0		5-9.9m : 1.35	more 2 span :1.75	narrow : 1.2	5.5* 210-349 : 1.7	others :1.0	others : 1.0		
	Soft : 1.5	Probable : 2.0	Simple : 3.0		more 10m : 1.7			6* 350-699 : 2.4				
	Very Soft : 1.8			Fix & Mov : 1.0				6.5* 700-1299 : 3.0				
				Mov & Mov : 1.15				7* 1300-3299: 3.5				
15	1.0	1.5	2.0	1.00	1.35	1.75	1.20	1.0	1.0	1.0	8.5	15
61	1.5	1.0	3.0	1.15	1.70	1.75	1.20	1.0	1.0	1.0	18.5	61
62	1.5	1.0	3.0	1.15	1.35	1.75	1.20	1.0	1.0	1.0	14.7	62
63	1.5	1.0	3.0	1.15	1.35	1.75	1.20	1.0	1.0	1.0	14.7	63
82	1.5	1.5	2.0	1.15	1.35	1.75	0.80	1.0	1.0	1.0	9.8	82
83	1.5	1.5	2.0	1.15	1.35	1.75	0.80	1.0	1.0	1.0	9.8	83
86	1.0	1.5	3.0	1.00	1.70	1.75	1.20	1.0	1.0	1.0	16.1	86
87	1.0	1.5	3.0	1.00	1.35	1.75	1.20	1.0	1.0	1.0	12.8	87
88	1.0	1.5	3.0	1.15	1.00	1.75	1.20	1.0	1.0	1.0	10.9	88
89	1.0	1.5	3.0	1.15	1.35	1.75	1.20	1.0	1.0	1.0	14.7	89
90	1.0	1.5	3.0	1.15	1.00	1.75	1.20	1.0	1.0	1.0	10.9	90
91	1.0	1.5	3.0	1.15	1.35	1.75	1.20	1.0	1.0	1.0	14.7	91
92	1.0	1.5	3.0	1.15	1.35	1.75	1.20	1.0	1.0	1.0	14.7	92
93	1.0	1.5	3.0	1.15	1.70	1.75	1.20	1.0	1.0	1.0	18.5	93
94	1.0	1.5	3.0	1.15	1.35	1.75	1.20	1.0	1.0	1.0	14.7	94
95	1.0	1.5	3.0	1.15	1.35	1.75	1.20	1.0	1.0	1.0	14.7	95
98	1.5	1.0	3.0	1.15	1.00	1.75	1.20	1.0	1.0	1.0	10.9	98

Stability	Total Score
High Seismic Risk	30≦S
Medium Seismic Risk	26≦S<30
Low Seismic Risk	S<26

Table S9-3.2.3 Bridge Damage Estimation in Case of Earthquake Scenario

Code No.	(1) Ground Type	(2) Liquefaction	(3) Girder Type	(4) Bearing Type	(5) Max. Height of Abut./Pier	(6) Number of Spans	(7) Min. Bridge Seat Width	(8) PGA *Japanese Earthquake Intensity	(9) Foundation Work	(10) Materials of Abutment and Pier	Total Score	Code No.
	Stiff : 0.5	None : 1.0	Rigid : 1.0	girder connection device :0.6	Less 5m : 1.0	one span : 1	wide : 0.8	5* 120-209 : 1.0	Pile bent:1.4	Brick : 1.4		
	Middle : 1.0	Possible : 1.5	Continuous : 2.0		5-9.9m : 1.35	more 2 span :1.75	narrow : 1.2	5.5* 210-349 : 1.7	others :1.0	others : 1.0		
	Soft : 1.5	Probable : 2.0	Simple : 3.0		more 10m : 1.7			6* 350-699 : 2.4				
	Very Soft : 1.8			Fix & Mov : 1.0				6.5* 700-1299 : 3.0				
				Mov & Mov : 1.15				7* 1300-3299: 3.5				
15	1.0	2.0	2.0	1.00	1.35	1.75	1.20	2.4	1.0	1.0	27.2	15
61	1.5	1.0	3.0	1.15	1.70	1.75	1.20	2.4	1.0	1.0	44.3	61
62	1.5	1.0	3.0	1.15	1.35	1.75	1.20	2.4	1.0	1.0	35.2	62
63	1.5	1.0	3.0	1.15	1.35	1.75	1.20	2.4	1.0	1.0	35.2	63
82	1.5	2.0	2.0	1.15	1.35	1.75	0.80	2.4	1.0	1.0	31.3	82
83	1.5	2.0	2.0	1.15	1.35	1.75	0.80	2.4	1.0	1.0	31.3	83
86	1.0	2.00	3.0	1.00	1.70	1.75	1.20	2.4	1.0	1.0	51.4	86
87	1.0	2.00	3.0	1.00	1.35	1.75	1.20	2.4	1.0	1.0	40.8	87
88	1.0	2.00	3.0	1.15	1.00	1.75	1.20	2.4	1.0	1.0	34.8	88
89	1.0	2.00	3.0	1.15	1.35	1.75	1.20	2.4	1.0	1.0	46.9	89
90	1.0	2.00	3.0	1.15	1.00	1.75	1.20	2.4	1.0	1.0	34.8	90
91	1.0	2.00	3.0	1.15	1.35	1.75	1.20	2.4	1.0	1.0	46.9	91
92	1.0	2.00	3.0	1.15	1.35	1.75	1.20	2.4	1.0	1.0	46.9	92
93	1.0	2.00	3.0	1.15	1.70	1.75	1.20	3.0	1.0	1.0	73.9	93
94	1.0	2.00	3.0	1.15	1.35	1.75	1.20	2.4	1.0	1.0	46.9	94
95	1.0	2.00	3.0	1.15	1.35	1.75	1.20	2.4	1.0	1.0	46.9	95
98	1.5	1.0	3.0	1.15	1.00	1.75	1.20	2.4	1.0	1.0	26.1	98

Stability	Total Score
High Seismic Risk	30≦S
Medium Seismic Risk	26≦S<30
Low Seismic Risk	S<26

Table S9-3.2.4 Result of Damage Estimation of Bridges

Earthquake Scenario	Earthquake 1967	Earthquake 1812
High Seismic Risk	0	15
Medium Seismic Risk	0	2
Low Seismic Risk	115	98
Total of Bridge	115	115

Table S9-3.4.1 Outline of Metro

Line Name	Total Length (km)	Year Com-pleted	No. of Station	Length (km)				
				Shield Type	Mountain Tunnel Type	Open Cut Type	Others	Station
Line 1 Propatria-Palo Verde	20.6	1983	22	n.a.	n.a.	n.a.	n.a.	n.a.
Line 2:Silencio-Zoo Logico/Las Adjuntas	18.4	1987	13	1.6	1.7	4.5	8.4	2.2
Line 3:Plaza Venezuela-El Valle	5.3	1994	5	2.2	1.0	1.2	0	0.9

n.a. : not available

Table S9-3.3.1 MMI of Viaduct and Damage Estimation based on Hanshin/Awaji Disaster Data

Location	Total Length (km)	Earthquake 1967						Earthquake 1812					
		MMI	JMI	Rate of Collapse (place/km)	No. of Collapse Place	Rate of Damage (place/km)	No. of Damage Place	MMI	JMI	Rate of Collapse (place/km)	No. of Collapse Place	Rate of Damage (place/km)	No. of Damage Place
Distribuidor La Arana	13.4	VIII+	5.5～6	0.003	0.04	0.014	0.19	IX−	6	0.086	1.15	0.206	2.76
Distribuidor Ciempies	4.0	VII＋～VIII−	5～5.5	0.003	0.01	0.014	0.06	VIII+	5.5～6	0.003	0.01	0.014	0.06
Distribuidor Pulpo	3.2	VII+	5	−	−	−	−	VIII+	5.5～6	0.003	0.01	0.014	0.04
Francisco Fajardo	3.3	VII−～VII+	4.5～5	−	−	−	−	VIII−～VIII+	5～6	0.003	0.01	0.014	0.05
Planicie	1.1	VII−～VII+	4.5～5	−	−	−	−	VIII+	5.5～6	0.003	0.00	0.014	0.02
Cotal Mil	2.2	VI+～VII−	4.5～5	−	−	−	−	VII+～VIII−	5～5.5	0.003	0.01	0.014	0.03
Cementerio	0.8	VI+	4.5	−	−	−	−	VIII+	5.5～6	0.003	0.00	0.014	0.01
Total	28.0				0.05		0.25				1.19		2.97

JMI	PGA
5	120～ 209
5.5	210～ 349
6−	350～ 524
6+	525～ 699
6.5	700～ 1299
7	1300～ 3299

1) MMI:Modified Mercalli Intensity
2) JMI:Japan Meteorological Intensity

*Relationship between MMI and PGA
MMI=(Log(PGA)−0.014)/0.3

Table S9-3.6.1 Damage Estimation of Telecommunication Lines in Each Central

Central	Length (km)	Earthquake Scenario 1967					Earthquake Scenario 1812				
		Av.MMI	Av.JMI	R (%)	C1	Nd	Av.MMI	Av.JMI	R (%)	C1	Nd
23 De Enero	38.1	7.42	5.0	0.0	1.0	0.00	8.26	5.5	0.3	1.0	0.11
Alta Florida	75.3	7.77	5.5	0.3	1.0	0.23	8.41	6.0	0.3	1.0	0.23
Alto Prado	5.5	6.77	4.5	0.0	1.0	0.00	7.35	5.0	0.0	1.0	0.00
Bello Monte	113.9	7.37	5.0	0.0	1.0	0.00	8.12	5.5	0.3	1.0	0.34
Boleita	184.3	7.37	5.0	0.0	1.0	0.00	7.92	5.5	0.3	1.1	0.61
Caobos	98.7	7.94	5.5	0.3	1.0	0.30	8.52	6.0	0.3	1.0	0.30
Caracas	75.2	8.33	5.0	0.0	1.0	0.00	8.70	6.0	0.3	1.0	0.23
Caricuao	238.0	6.92	5.5	0.3	1.0	0.71	7.71	5.5	0.3	1.0	0.71
Chacao	226.8	7.77	5.0	0.0	1.0	0.00	8.21	5.5	0.3	1.0	0.68
Chaguaramos	73.8	7.44	5.0	0.0	1.0	0.00	8.30	5.5	0.3	1.0	0.22
Chuao	5.0	7.59	5.0	0.0	1.0	0.00	8.35	5.5	0.3	1.0	0.02
Coche	69.0	7.02	4.5	0.0	1.0	0.00	7.81	5.5	0.3	1.1	0.23
El Cafetal	168.6	6.84	5.5	0.3	1.0	0.51	7.35	5.0	0.0	1.0	0.00
El Rosal	46.2	7.85	5.0	0.0	1.0	0.00	8.46	6.0	0.3	1.0	0.14
Fajardo	202.1	7.29	5.0	0.0	1.0	0.00	8.14	5.5	0.3	1.1	0.67
Fco. Salias	278.0	6.91	5.0	0.0	1.0	0.00	7.50	5.0	0.0	1.0	0.00
Jardines	78.0	7.25	5.5	0.3	1.0	0.23	8.09	5.5	0.3	1.1	0.26
La florida	130.3	7.91	5.5	0.0	1.0	0.00	8.65	6.0	0.3	1.0	0.39
La Salle	54.9	7.73	5.0	0.0	1.0	0.00	8.46	6.0	0.3	1.0	0.16
La Urbina	33.7	6.90	5.5	0.3	1.0	0.10	7.31	5.0	0.0	1.0	0.00
Las Mercedes	160.7	7.85	4.5	0.0	1.0	0.00	8.52	6.0	0.3	1.0	0.48
Los guayabitos	13.0	6.70	5.0	0.0	1.0	0.00	7.28	5.0	0.0	1.0	0.00
Los Palos Grande	156.7	7.60	5.0	0.0	1.0	0.00	8.04	5.5	0.3	1.0	0.47
Macaracuay	57.4	7.18	5.5	0.3	1.0	0.17	7.66	5.0	0.0	1.0	0.00
Maderero	134.6	7.76	5.0	0.0	1.1	0.00	8.52	6.0	0.3	1.3	0.52
Miranda	1.9	6.68	4.5	0.0	1.0	0.00	6.91	5.0	0.0	1.0	0.00
Palo Verde	63.0	6.91	5.0	0.0	1.0	0.00	7.39	5.0	0.0	1.0	0.00
Pastora	282.1	7.65	5.0	0.0	1.0	0.00	8.50	6.0	0.3	1.0	0.85
Petare	11.2	6.91	5.0	0.0	1.0	0.00	7.35	5.0	0.0	1.0	0.00
Prado De Maria	36.8	7.39	5.0	0.0	1.0	0.00	8.33	5.5	0.3	1.0	0.11
Rdo. Zuoloaga	106.1	7.46	5.0	0.0	1.0	0.00	8.31	5.5	0.3	1.1	0.35
San Agustin	30.3	8.16	5.5	0.3	1.0	0.09	8.72	6.0	0.3	1.0	0.09
San Martin	69.2	7.62	5.0	0.0	1.1	0.00	8.34	5.5	0.3	1.3	0.27
Url Valle Arriba	98.8	6.59	5.0	0.0	1.0	0.00	7.54	5.0	0.0	1.0	0.00
Total (km)	3417.20					2.34					8.43
Total (%)	100%					0.07%					0.25%

Table S9-3.7.1 Max. PGA and G.S. Massed Area

Item	Location Code No.	No. of G.S.	PGA	Area Name
PGA Max. Area	No.37, No.26	2	714, 723	Antimano, Catedral La Candelaria
G.S. Massed Area At High PGA (I)	No.17, No.19 No.21~No.24	6	356~559	Neveri (near interchange Arana)
G.S. Massed Area At High PGA (II)	No.10~No.13 No.15 No.28~No.30	8	359~590	Las Acascias、Valle Abajo Collinas Las Acalias Lios Chaquaramamos

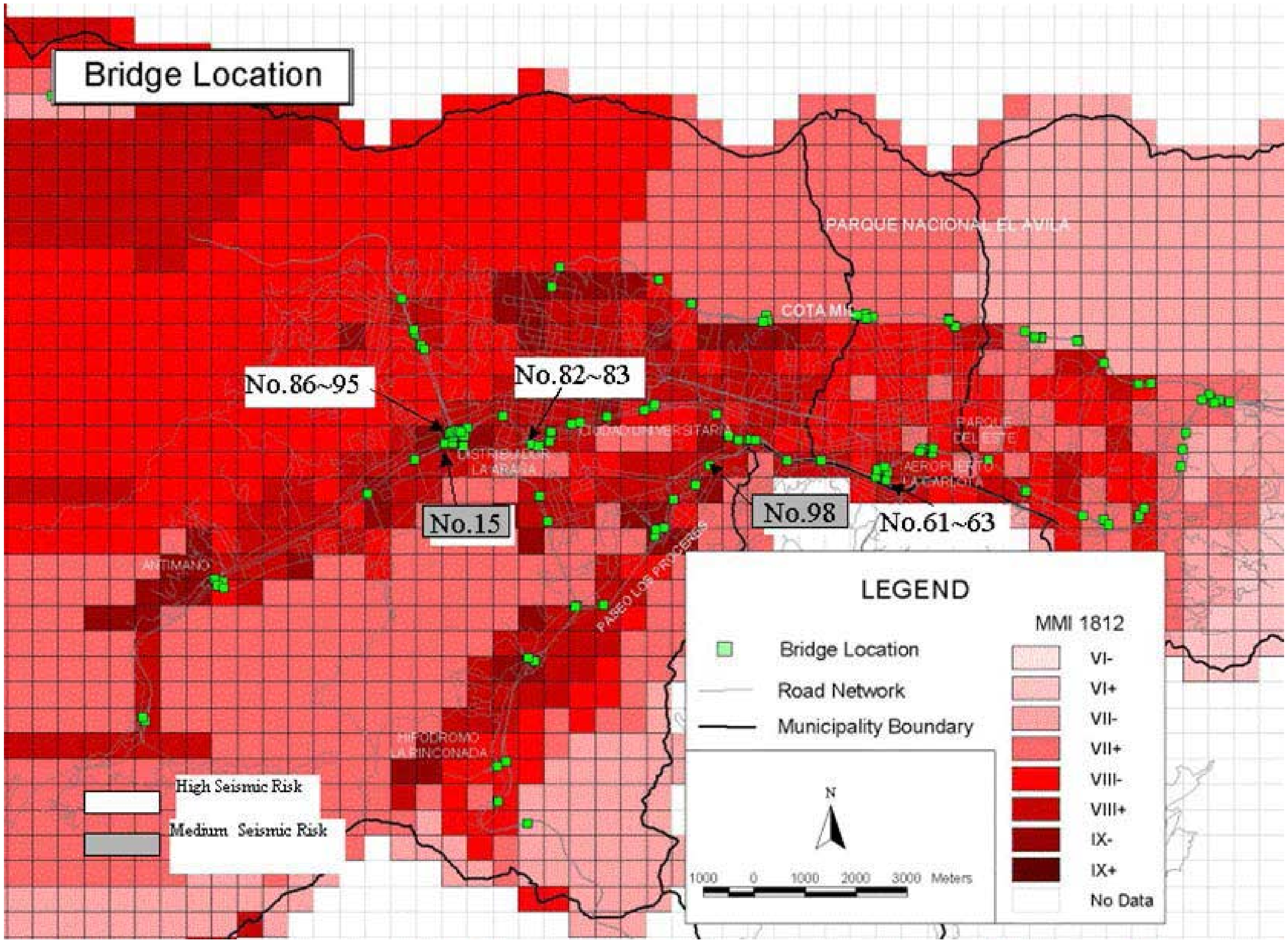


Figure S9-3.2.1 Bridge Locations

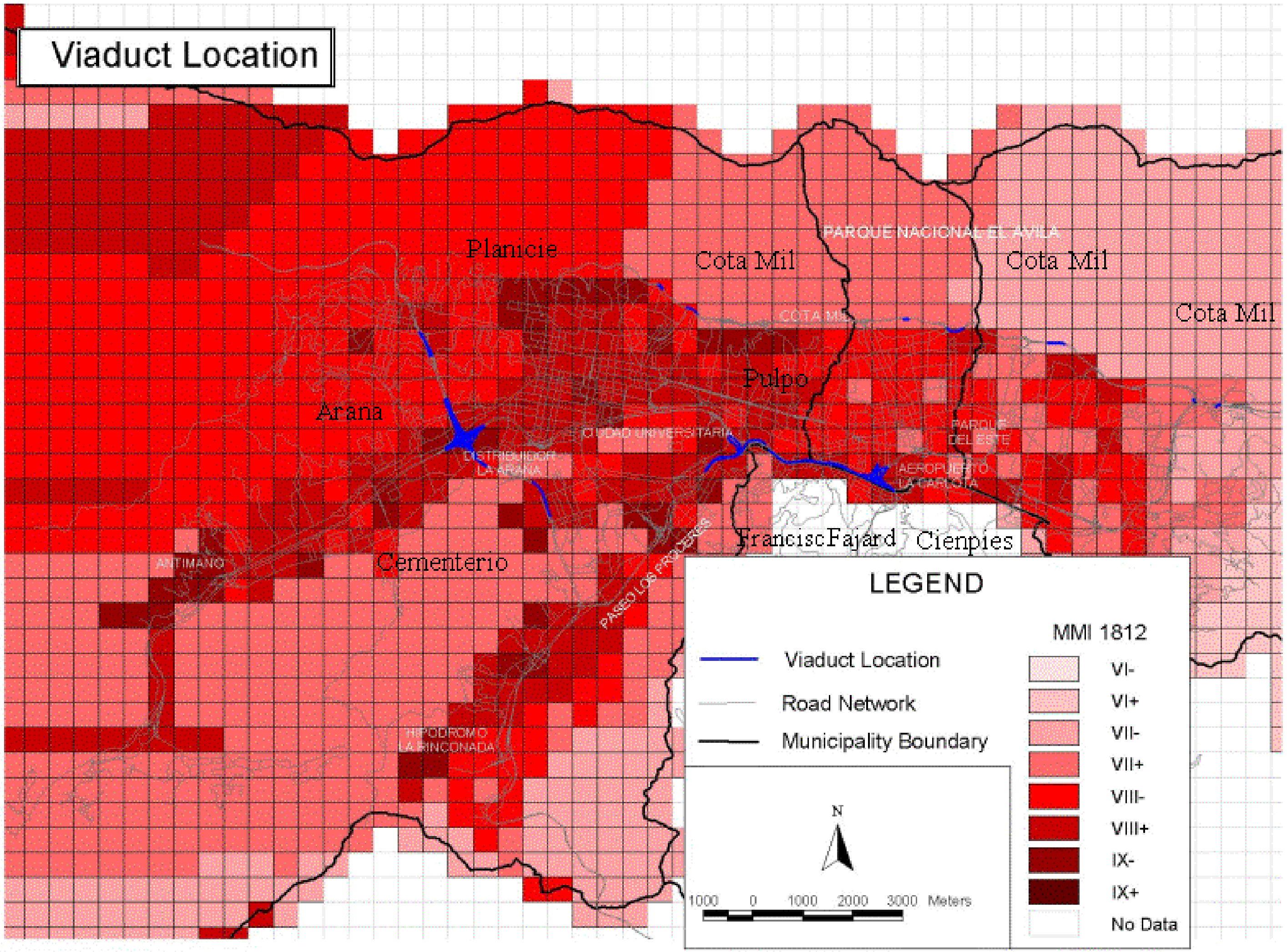


Figure S9-3.3.1 Viaduct Locations

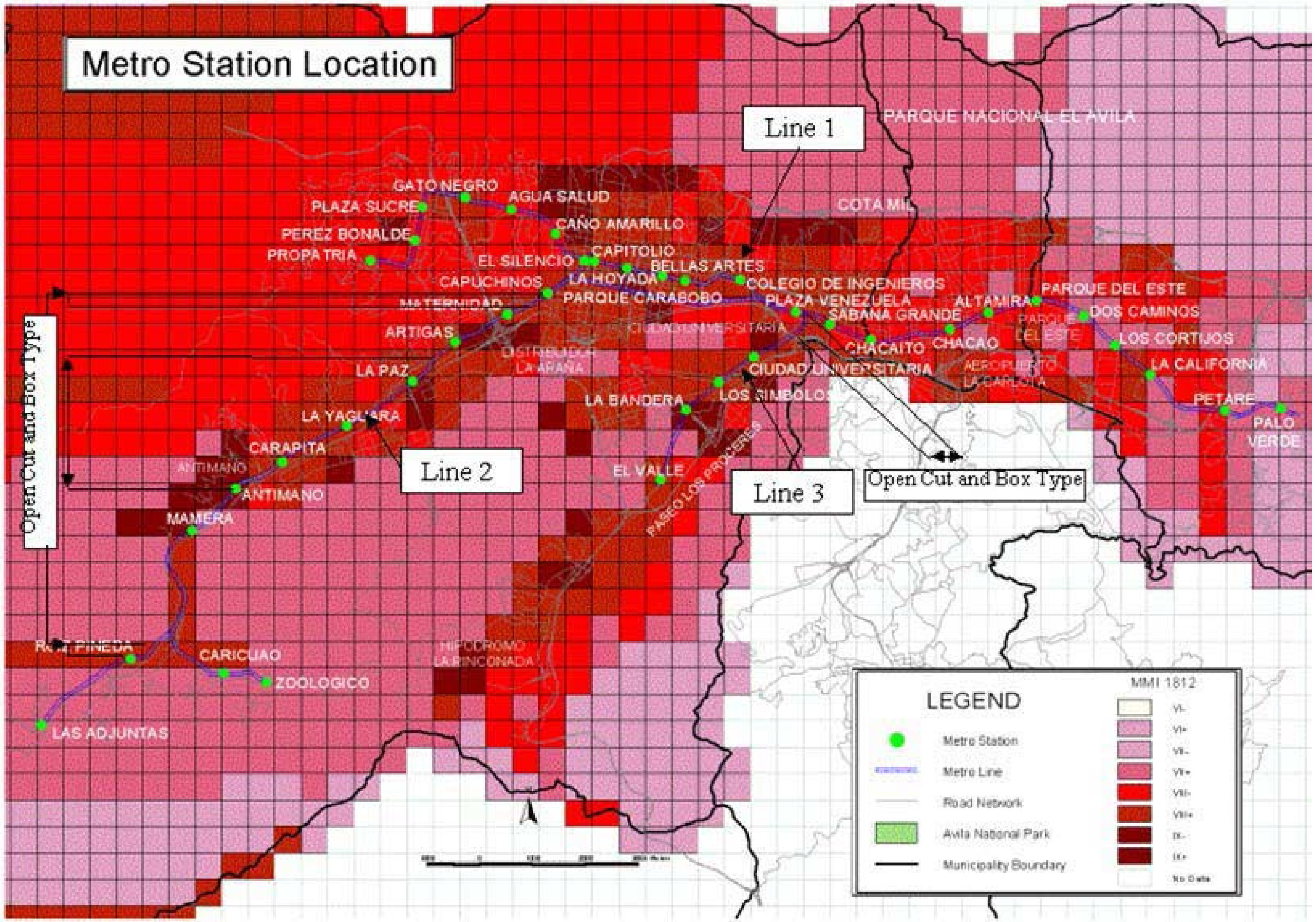


Figure S9-3.4.1 Metro Locations

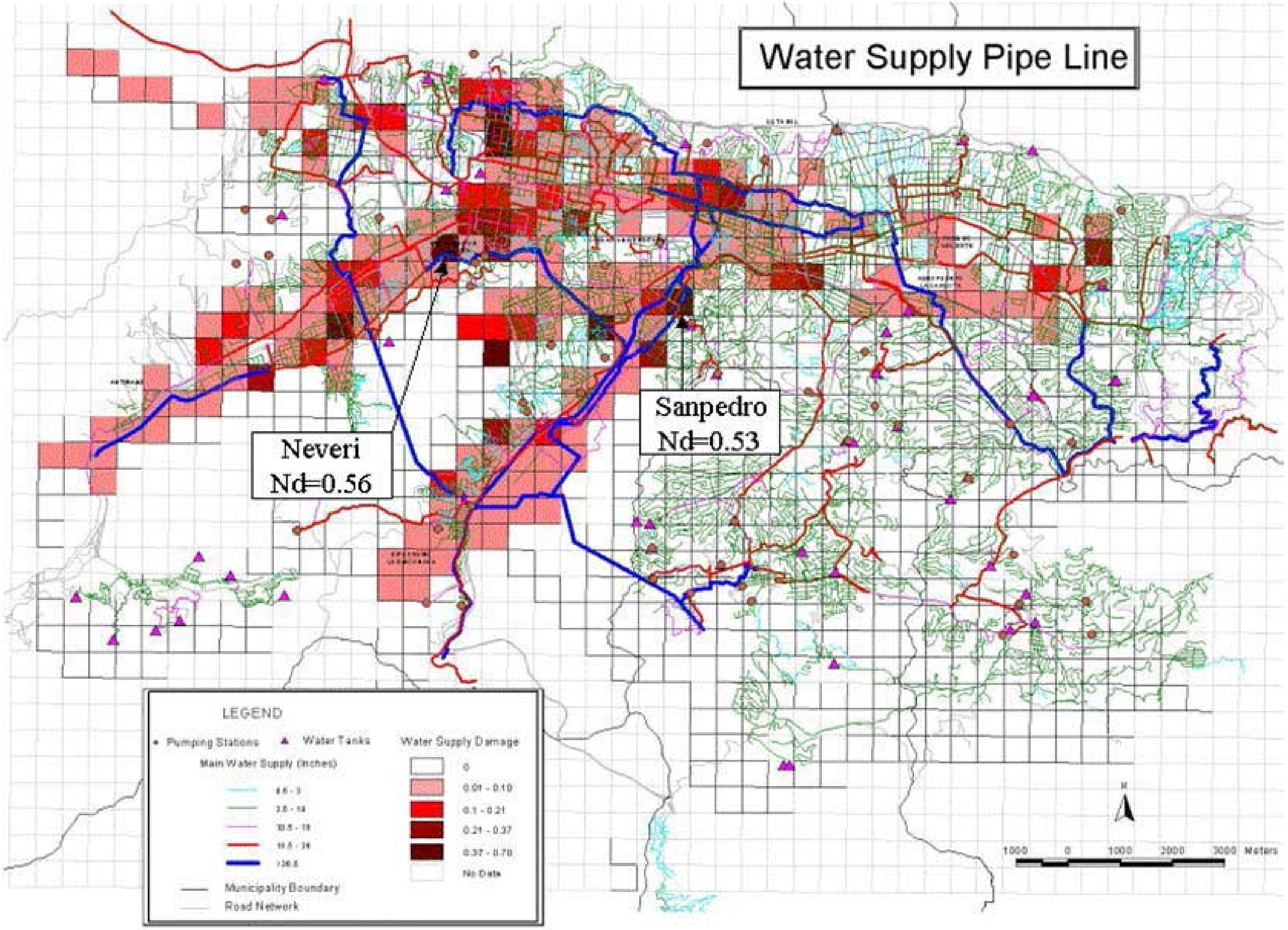


Figure S9-3.5.1 Water Supply Pipelines

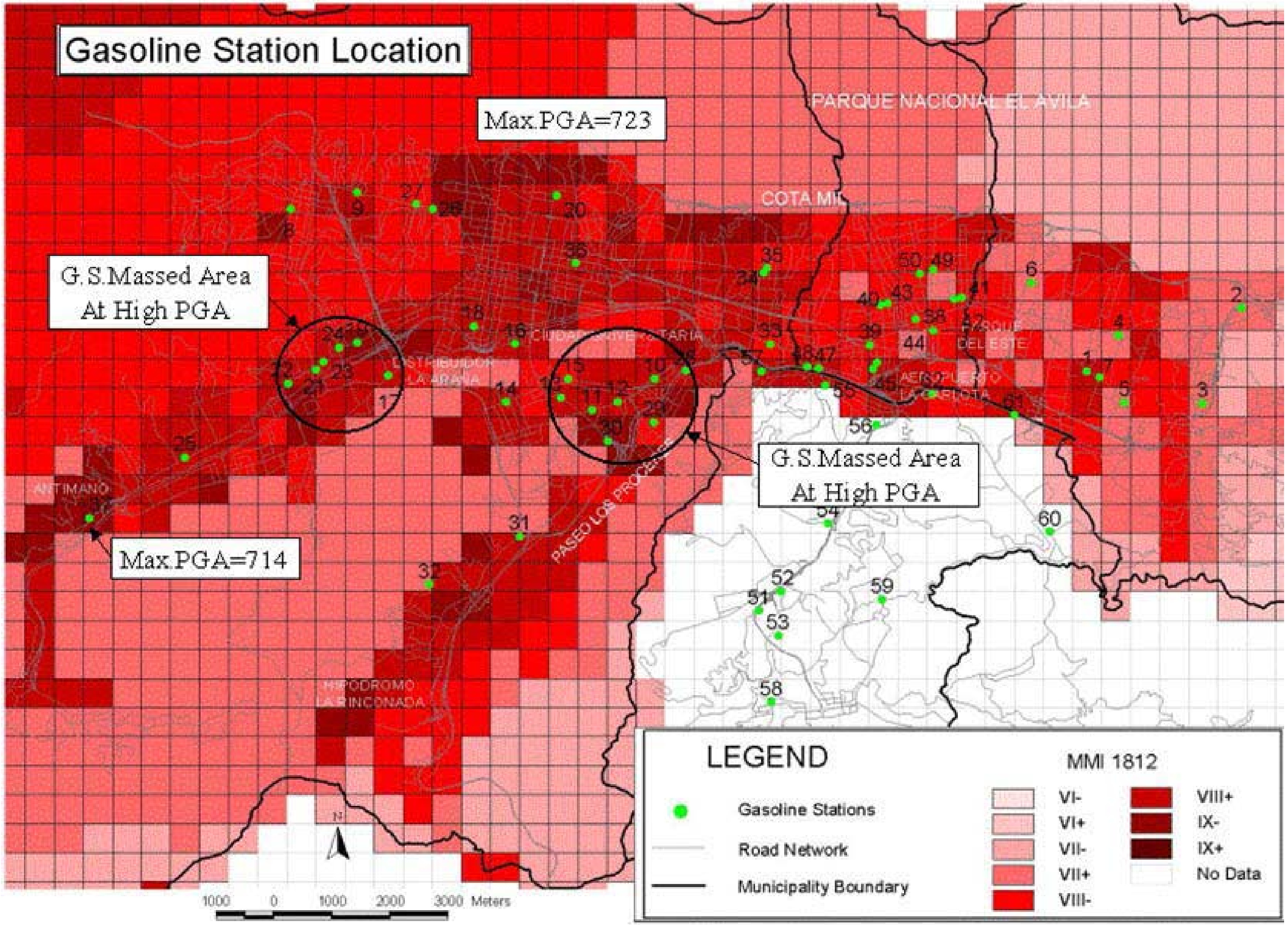


Figure S9-3.7.1 Gasoline Station Locations

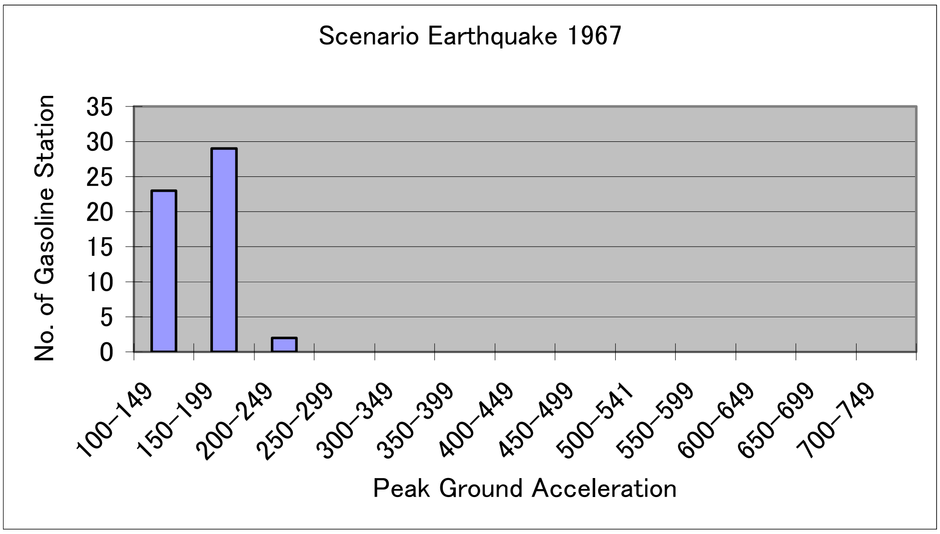


Figure S9-3.7.2 PGA and No. of Gasoline Station

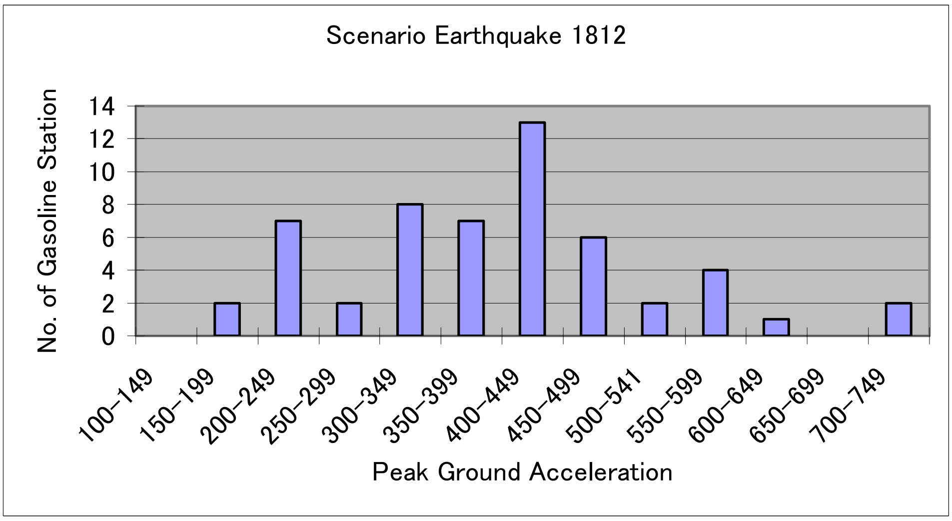


Figure S9-3.7.3 PGA and No. of Gasoline Station

CHAPTER 4. COUNTERMEASURES FOR BRIDGE REINFORCEMENT

4. 1 General

The collected data for seismic damage estimation are quite limited. Under limited data, seismic damage estimation was carried out for Bridges, Viaducts (elevated highway), Metro, Water Supply Pipe, Telecommunication Line and Hazardous Facilities (Gasoline Stations).

Among these damage estimations, it is revealed that bridges would be most affected and damaged by the scenario earthquake 1812 and thus adequate countermeasures are required for bridge reinforcement.

No serious damage is expected in case of scenario earthquake 1967, but in case of scenario earthquake 1812, 15 bridges are evaluated as high seismic risk and 2 bridges are evaluated as medium seismic risk. Seismic risk means the possibility of bridge collapsing.

These bridges are significant for the rescue activity, transportation of emergency good and quick recovery of lifelines. Therefore it is recommended to take necessary countermeasures for the bridge reinforcement against the earthquakes.

4. 2 Bridges

4. 2. 1. Prevention Measure Against Bridge Collapse

If the displacement of girders induced by earthquake exceeds the bridge seat length, the deck slab will collapse and the bridge could not maintain its function, even substructure and foundations are not damaged.

Depending on the type of bridge and the purpose, the prevention measure against the bridge collapse is different. There are two major countermeasures. One is to allow the displacement, but prevent the deck slab collapsing by lengthening the seat, and the other is to control the movement of girders within the length of the seat.

Typical samples of unseating system is shown in Table S9-4.2.1.

It is recommended that the countermeasure of lengthening of seat length is most effective for prevention of bridge collapse, because no force shall act on the substructure due to the displacement of girders and this could protect the substructure.

Sample of unseating concrete bracket is shown in Figure S9-4.2.1.

Countermeasures for unseating system shall be decided after detail investigation of design, the allowance for bracket installation and working conditions such as space for working, traffic control and height of working conditions are studied.

4. 2. 2. Strengthening of Pier

The strengthening of the pier is recommended based on the experience of the Hanshin/Awaji Disaster. The vertical seismic force in that disaster exceeded the design force and the piers collapsed due to the extra sharing force, and especially the single column pier was seriously damaged. After the experience of hazardous earthquake, bridges located on the trunk road and express highway were strengthened at the pier.

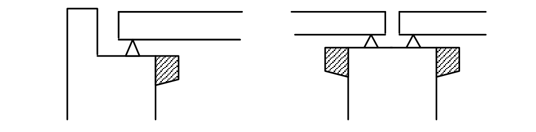
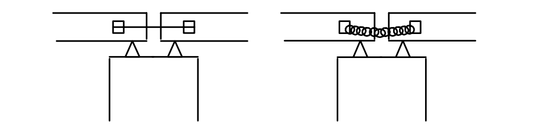
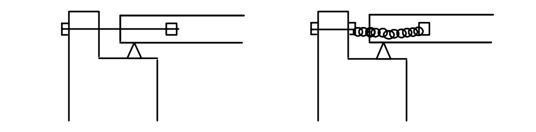
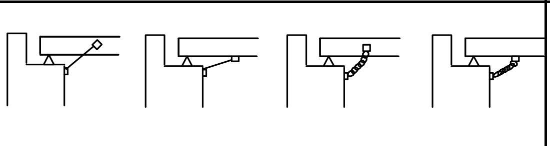
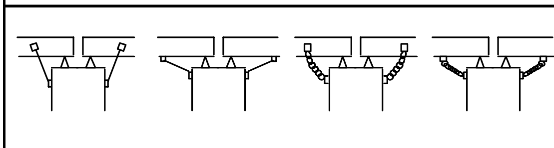
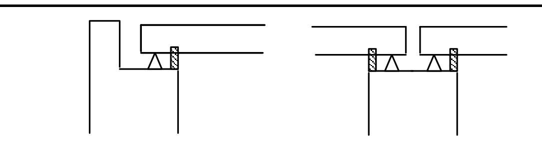
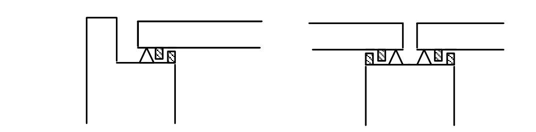
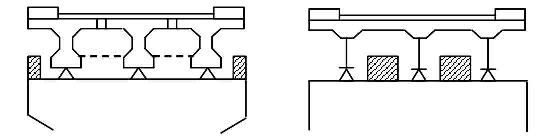
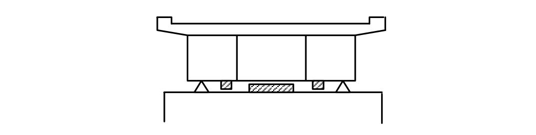
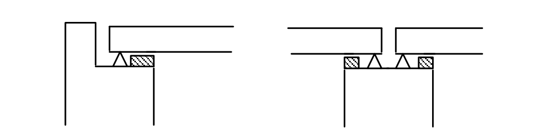
The bridges located at the most vulnerable area, interchange Arana and Pulpo were constructed before 1967 and superstructures are supported by small piers of rigid frame and single piers.

As for the damage of bridge foundation, no damage was reported in the Hanshin/Awaji Disaster. It is considered that the foundation in the ground is not easily affected by the earthquake and the ultimate strength of foundation is fairly large and so it is not easily damaged like the structure above the ground. From these points of view, the prevention measures against bridges collapsing and strengthening of pier are recommended.

A sample method of strengthening the pier is shown in Figure S9-4.2.2.

The size, shape and type of each bridge is different and detailed survey is required for the selection of adequate countermeasures.

Table S9-4.2.1 Typical Samples of Unseating System

Direction	Device	Type	Location	Item	Material	Schematic Configuration	Remarks
Bridge Longitudinal Direction	Widening of Seat Width				RC or Steel Plate		Bracket Type
Bridge Longitudinal Direction	Unseating prevention device	Connecting device between girder and adjoining girder			PC Cable or Steel Chain		
Bridge Longitudinal Direction	Unseating prevention device and restriction of displacement	Connection Type	Abutment	Connecting girder and parapet	PC Cable or Steel Chain		
Bridge Longitudinal Direction	Unseating prevention device and restriction of displacement	Connection Type	Abutment	Connecting girder and substructure	PC Cable or Steel Chain		Attach to girder side or under surface
Bridge Longitudinal Direction	Unseating prevention device and restriction of displacement	Connection Type	Pier	Connecting girder and substructure	PC Cable or Steel Chain		Attach to girder side or under surface
Bridge Longitudinal Direction	Unseating prevention device and restriction of displacement	Projection Type		Projection on Substructure	RC or Steel Bracket		
Bridge Longitudinal Direction	Unseating prevention device and restriction of displacement	Projection Type		Projection under Girder and on Substructure	RC or Steel Bracket		
Bridge Lateral Direction	Unseating prevention device and restriction of displacement	Projection Type		Projection on Substructure	RC or Steel Bracket		Adding outsid or between the Girders
Bridge Lateral Direction	Unseating prevention device and restriction of displacement	Projection Type		Projection under Girder and on Substructure	RC or Steel Bracket		
Adding of landing space				Projection on Substructure	RC or Steel Bracket		

Source : Japan Road Association, 2002

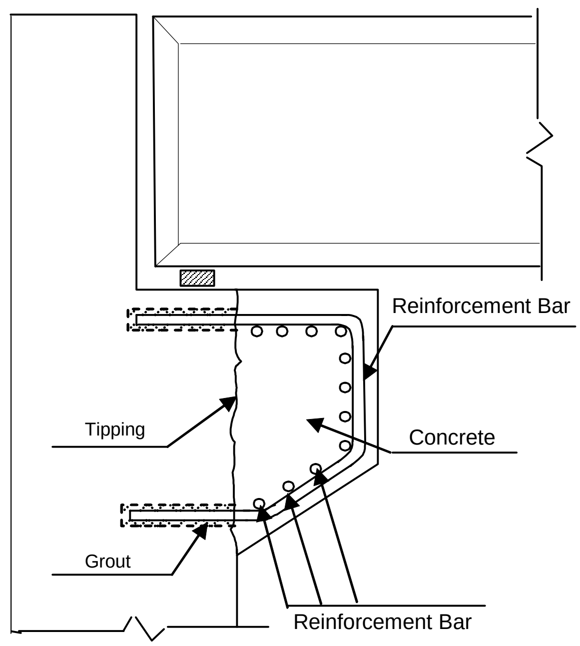


Figure S9-4.2.1 Sample of Unseating Concrete Bracket

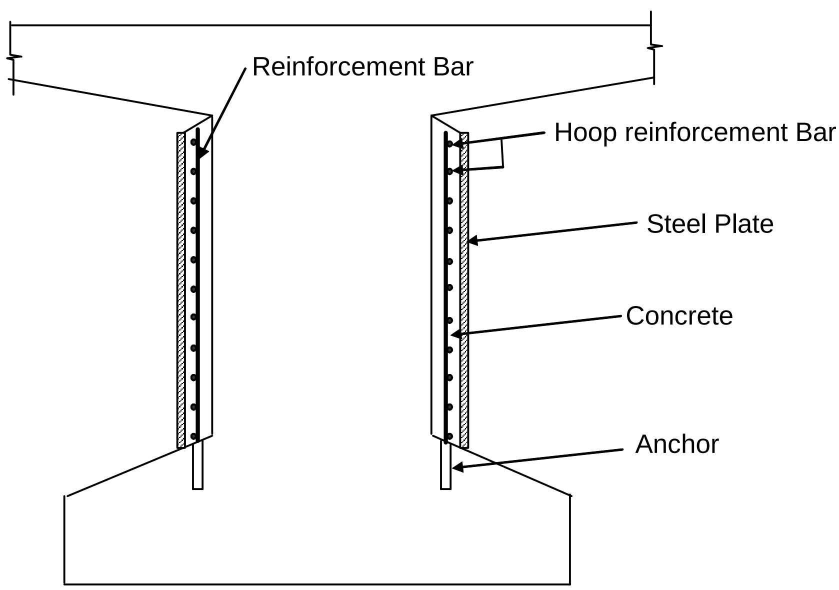


Figure S9-4.2.2 Sample for Strengthening the Pier

CHAPTER 5. RECOMMENDATIONS

5. 1 General

In order to estimate the seismic damage on infrastructures and lifelines, their detail data is indispensable. It is fundamentally recommended that the data of infrastructures and lifelines shall be managed and always updated by related organizations not only for operation and maintenance, but also for planning of seismic damage prevention measures.

The inventory lists are definitely required for the recovery activity after disastrous earthquake occurrence.

5. 2 Bridge and Viaduct (Elevated Highway)

Seismic damage evaluations revealed that bridges and viaducts (elevated highway) will be seriously damaged in case of the scenario earthquake 1812 and it is needed to strengthen those bridges and viaducts.

The first priority is to install the unseating device to the bridges for reasons as follows:

1) The bridges at the interchange Arana and Pulpo were constructed before 1967 and adopt the old seismic code. The bridge seat length is not enough for the displacement of superstructure at the time of earthquake occurrence.

2) The bridges and viaduct at the interchange Arana are more than 10 m and the displacement of superstructure caused by the earthquake is considered very large.

3) The interchange was constructed at the sedimentary deposits and the area is susceptible to liquefaction and much displacement is expected.

The interchange Pulpo was constructed before 1967. The pier size of interchange Pulpo is comparatively small and seismic resistance needs to be checked to determine whether the additional reinforcement of pier is required in accordance with the present seismic code.

There is a big arch type bridge, Viaduct No.1 near the north side of La Planicie Tunnel where the bridge is now under slope protection works at the Caracas side abutment. After countermeasures are completed, it is recommended to carry out a periodic inspection for the bridge about the conditions of anchors and sprayed concrete/mortal, and also special inspection is required at the time of earthquake occurrence.

5. 3 Metro

Underground structures are comparatively safer than the structures on the ground due to the less seismic force under the ground. No damage of shield tunnel section was reported in the Hanshin/Awaji Disaaster in Japan.

But some damages were reported at the cut and cover box tunnel due to the extra vertical force on the tunnel and middle column were broken as well as side wall.

It is recommended that the middle column of cut and cover box tunnel should be reviewed in the design, and if necessary, to strengthen them by steel, reinforcement bar and concrete like bridge pier strengthening method.

Countermeasures shall be decided after detail investigation of design method, earth cover depth and applied seismic force.

5. 4 Road Tunnel

There are 6 tunnels in the study area and one tunnel is located at the boundary of south municipality. Six tunnels location and lengths are shown in Table S9-5.4.1.

In accordance with the experience of Hanshin/Awaji Disaster, inside of the tunnel was not seriously damaged. The inside of the tunnel is more stable compared with the open area due to the less seismic force under the ground.

Most of the cutting slope of 5 tunnel entrances are protected by the protection measures such as anchor and sprayed concrete/mortal. Therefore, serious damage is not expected at the time of earthquake.

It is noted that some houses are located on the top of the cut slope at the south side of Tunnel La Planicie II and for the safety of those houses, it is required that the houses should be shifted to a place of safety. This is not only for their safety but also for the safety of the highway to the tunnel.

Water seepage was observed in La Planicie I and La Valle tunnels and adequate water treatment is needed so that the deterioration of concrete lining will be prevented, and consequently the strength of concrete will be kept in good condition.

Some damage to the ceiling was observed in Boqueron Tunnel. It is recommended to install an adequate guard bar in front of the tunnel to protect the tunnel facility and structure. It is important to check the vehicle height of emergency vehicles, to ensure that such vehicles may pass the tunnel for emergency activity and transportation at the time of earthquake.

Low level of lighting was observed in El Valle tunnel. Adequate tunnel lighting is required to prevent accidents due to low visibility in the tunnel and prevent against the black hole phenomena when entering the tunnel.

Road Tunnel was constructed at the mountain area and comparatively surrounded by rock; hence, it ia safer like the Metro shield section.

It is recommended that periodic inspection (such as for any new crack in the concrete/mortal on the slope protection) shall be made so that early countermeasures will be taken. Especially after earthquake occurrence, special inspection of the slopes are required.

5. 5 Countermeasures for Lifelines

Lifelines are indispensable for the functioning of city life in Caracas. Not only operation and maintenance are required, but also emergency action is required. Quick recovery of lifelines are the responsibility of each lifeline company.

Data of lifelines are required for the estimation of seismic damage and to plan for their strengthening against earthquakes as well as maintenance.

In order to make a plan and take effective measures against the earthquake disaster, the following items are recommended:

1) Management and updating of inventory list of lifelines such as water supply, electric supply, gas supply, telecommunication line, etc. These are necessary for daily and periodic maintenance as well as emergency recovery activity.

2) Manual for emergency action in earthquake occurrence. For the rescue activity, various lifelines are indispensable such as water, electricity and communication system. A manual should be prepared in consideration of establishing cooperation at the time of earthquake occurrence.

3) Improvement the lifelines for seismic resistant material and joint system through maintenance works. It is not realistic to change the material at once, but effort is required to gradually improve the lifelines.

4) Network system among the lifeline companies for exchanging policies of disaster prevention and recovery. Each lifelines should be inter-connected to each other. In consideration of quick recovery of lifelines, exchange of information is necessary.

5. 6 Hazardous Facility

There may be hazardous facilities such as flammable liquid, flammable gas, toxic gas and liquid nitrogen. But so far the collected data for hazardous facilities only covers the location of gasoline stations.

It is recommended to make a list of hazardous facilities, to prepare the emergency manual in case of earthquake occurrence and reinforce structures to be durable enough against earthquakes.

5. 7 Alternative Road

The west side of express highway Cota Mil is not connecting to the express highway Caracas-La Guaira. When the interchange Arana is damaged by the earthquake in any way, there is no alternative road to connect the south area. Connecting Cota Mil and Caracas La Guaira will provide an alternative route (approx.5.2 km) for the emergency activity and transportation.

Table S9-5.4.1 List of Road Tunnel in Caracas

Tunnel Name	Location	Tunnel Length (m)	Year Completed
Boqueron I	Autopista Caracas – La Guaira	1800	1953
La Planicie I	Autopista Caracas – La Guaira	600	1965
La Planicie II	Autopista Caracas – La Guaira	625	1986
El Paraiso	Autopista La Arana - Coche	(750)*	n.a.
El Valle	Autopista La Arana - Coche	(1050)*	n.a.
Turumo	Autopista Petare - Guarenas	600	1978

*Tunnel length is measured from a map that is available in the market.
n.a. : not available

CHAPTER 6. COST ESTIMATION

6. 1 General

In consideration of the priority of the earthquake disaster prevention measure, cost estimation is made on the countermeasures for bridge reinforcement.

Metro lines are comparatively safe structures because they are constructed under the surface of ground, but the reinforcement of middle column of cut and cover tunnel shall be reviewed in design in consideration of vertical force at the time of earthquake occurrence and countermeasures taken such as jacketing reinforcement if necessary.

Other project costs shall be estimated only after collection of detailed information and data. As for the lifelines, each lifeline companies should take their own seismic disaster prevention measures.

6. 2 Cost Estimation

The cost of countermeasures for bridge reinforcement is shown in Table S9-6.2.1.

The cost estimation indicates the approximate cost, and is estimated on the basis of standard countermeasures. This cost estimation indicates the reference cost for the evaluation of Master Plan and policy decision. This cost estimation is not a decision regarding any design of countermeasures. The detailed cost shall be estimated after more detailed investigations.

Table S9-6.2.1 Project Name and Cost Estimation

Project Name	Aim	Action	Cost (Million US$)
Bridge Investigation and Reinforcement Plan	To investigate the design of bridges and the bridge conditions at the site and make a reinforcement plan	Investigation of design method and design code, drawings and actual conditions of bridges	0.04
Bridge Reinforcement (I)	Prevention of bridge collapse	Lengthening of bridge seat or restriction of displacement of superstructure	5.6
Bridge Reinforcement (II)	Strengthening of piers	Jacketing of pier by steel, reinforcement bar and concrete	5.4

CHAPTER 7. IMPLEMENTAION SCHEDULE

7. 1 General

The target year is planned for 2020. In consideration of the project size, the schedule is planned in two stages: short term and long term.

Short term includes the prevention measures against bridge collapse and long term includes the strengthening of the bridge pier and strengthening of middle column of cut and cover tunnel sections.

7. 2 Schedule

List of plans relevant to the earthquake disaster prevention for infrastructure and lifelines and their implementation schedule are shown in Table S9-7.2.1.

Table S9-7.2.1 Implementation Schedule

Implementaion Plan	Short Term								Long Term							
	05	06	07	08	09	10	11	12	13	14	15	16	17	18	19	20
Bridge Investigation and Reinforcement Plan	■	■														
Bridge Reinforcement (I) – Prevention of Bridge Falling Down		■	■	■	■	■	■	■								
Bridge Reinforcement (II) – Strengthening of Pier									■	■	■	■	■	■	■	■
Metro Reinforcement – Strengthening of Middle Column									■	■	■	■	■	■	■	■
Lifelines (I) – Inventory List	■	■														
Lifelines (II) – Emergency Manual		■	■	■	■											
Lifelines (III) – Improvement of Material and Joint System		■	■	■	■	■	■	■	■	■	■	■	■	■	■	■
Lifelines (IV) – Disaster Prevention Network System	■	■														
Hazadous Facility (I) – List of Hazadous Facilities	■	■														
Hazadous Facility (II) – Emergency Manual		■	■	■	■											
Hazadous Facility (III) – Reinforcement of Structure				■	■	■	■	■	■	■	■	■	■	■	■	■
Alternative Road Cota Mil (West Side) ～ Caracas La Guaira									■	■	■	■	■	■	■	■

CHAPTER 8. EVALUATION OF THE PROJECT

8. 1 General

Infrastructures and lifelines are indispensable for the city life in Caracas where 2.7 million people reside. If these infrastructures and lifelines get damaged by earthquake, the function of the city will be seriously impaired.

Road network is one of the most important infrastructures not only for the social and economic activities, but also for the emergency activities in the disaster stricken area.

Among infrastructures and lifelines, priority project shall be selected from the viewpoint of the most effective project for disaster prevention and most effective project for contributing for emergency activities after earthquake occurrence as well as cost benefit evaluation.

8. 2 Priority Project

When severe earthquake occurs, rescue activity and transportation are critical for the quick recovery of the life of people.

Bridge collapse will create the most serious malfunction of road network and will take a significant amount of time to recover. It will interrupt the flow of vehicles for emergency activities.

In order to secure the trunk roads for emergency transportation, minimum bridge collapse prevention measures should be taken as the first priority.

There are two countermeasures for bridge reinforcement: one is to install the unseating device and the other is to strengthen the pier by jacketing with steel, reinforcement bar and concrete.

The interchange Arana is located at the center of east-west and north-south express highways. The interchange Arana consists of many bridges and viaducts but most of them were constructed before 1967 and the bridge seat length is not sufficient to cope with the displacement of superstructure which will be induced by earthquake. Minimum unseating prevention measures for bridges are required.

According to the seismic damage estimation, 15 bridges are estimated as high seismic risk and 2 bridges as medium seismic risk. Especially most bridges located at the interchange Arana are evaluated as high seismic risk and need to be reinforced as the first priority.

8. 3 Cost Benefit Evaluation

Cost benefit evaluation will be made to compare the cost with the project of unseating prevention measures and cost without the project.

The cost comparison will be made on the following assumptions:

1) The new bridge construction cost is estimated as approximately 7 million Bs (1USD=1920Bs) per m^2 of superstructure, subject to the bridge type, size, height and foundation type etc. In order to simplify the cost comparison, it is assumed that standard bridges are 25 m span and bridge width is 12 m; in accordance with this assumption, the construction cost will be 2.1 billion Bs and this cost includes the substructure and superstructure.

2) The required devices for the prevention of bridge unseating are 10 pieces for the above assumed bridges, and the cost of unseating device per piece is estimated 2.8 million Bs; thus, the total cost will be 28 million Bs.

3) If the damage would only be to the superstructure, the reconstruction cost would be reduced and its cost would be assumed half of item 1 above (new construction cost); thus, the unit reconstruction cost would be 1.05 billion Bs.

Hence, it is estimated that the installation of unseating devices for bridges will produce much benefit: approx. 1.022 billion Bs (= 2.1 - 0.028 - 1.05). This estimation is based on assumptions that the unseating device would work to prevent bridge collapse and that other substructures would not be damaged by the earthquake.

The bridge collapse will cause serious traffic jams and as a result induce large economic loss for the vehicles passing the expressway. The estimated number of vehicles passing the interchange Arana is more than 40,000 per day. If it takes six months to reconstruct the bridge, subject to the extent of damage and bridge size, more than 40,000 people will be affected by that reconstruction and travel time; hence, the total number of affected people will become 7 million people over six months. Furthermore, the poor road transportation caused by the traffic jam will affect the 2.7 million people in Metropolitan area.

Prevention measures for the unseating devices involve an adequate project with the minimum cost and maximum benefit. The total estimated cost for the installation of unseating device for 17 bridges is 10.7 billion Bs.

APPENDIX COST ESTIMAE DETAIL

Introduction

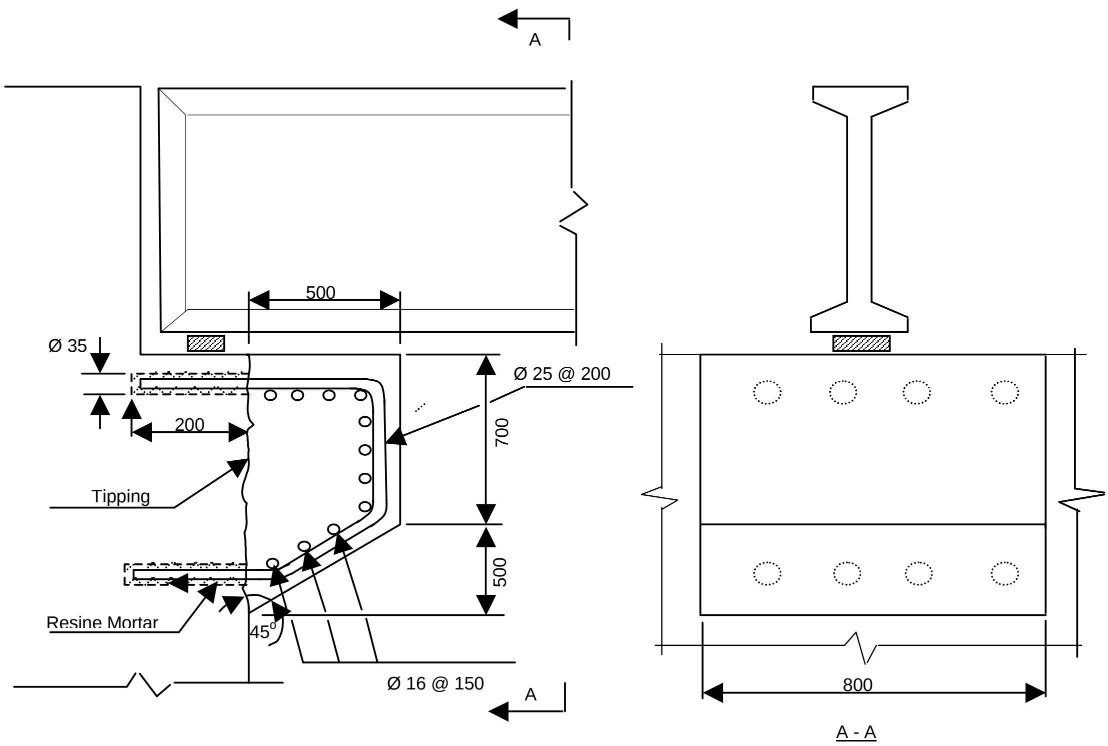
The purpose of Cost Estimation shown in Chapter 6 is to assess how the prevention measure of bridge falling down is effective. The cost estimation shown here is therefore rough estimation in order to evaluate the Cost Benefit.

Unit Cost of Bridge

In order to assess the construction cost of bridge, Approximately 400,000 Japanese Yen per square meter of super-structure for bridge construction is adopted in accordance with the experience is Japan. If 1 USD=110JPY and 1 USD=1,900 Bs, the cost of bridge per meter will be 400,000 / 110 x 1,900 = 6.9 Million, say 7 Million Bs. Then the cost of one bridge with 25 m span and 12 m wide is 25 x 12 x 7 Million Bs. = 2.1 Billion Bs.

Details of Cost Estimation

The cost estimation of 2.8 million Bs. for prevention of bridge falling down is shown in Attachment-1 and the total cost 10.7 Billion Bs. is shown in Attachment-2.



Attachment-1 Unit Cost Estimation of Prevention of Bridge Falling Down

Item	Unit	Quantity	Unit price (Bs)	Cost (Bs)
Tipping	m^2	0,96	3.000,00	2.880,00
Anchor Hole	piece	8,00	1.800,00	14.400,00
Grout	ml	1507,20	263,00	396.393,60
Reinforcement Bar	kg	72,03	2.071,56	149.223,25
Formwork	m^2	2,08	44.415,56	92.192,49
Concrete	m^3	0,38	588.206,69	223.518,54
Increment for High Elevated Working (50% of Cost)	L.S.	1,00		439.303,94
Temporary Staging (30% of Cost)	L.S.	1,00		263.582,36
Traffic Control (10% of Cost)	L.S.	1,00		87.860,79
Indirect Cost (25% of Total Cost)	L.S.	1,00		417.338,74
Tax (17% of Total Cost)	L.S.	1,00		354.737,93
Project Administration Cost (15% of Cost)	L.S.	1,00		366.214,75
			Total	2.807.646,38

Attachment-2 Total Cost of Brackets and Strengthening of Pier

Number of Brackets			
Bridge No.	Concrete Bracket	Steel Bracket	Steel Chain or PC Cable
61		80	
62		320	
63		230	
82			48
83			80
86	224		
87	32		
88	320		
89		320	
90	320		
91	80		
92	400		
93	320		
94	320		
95	320		
15	98		
98	12		24
Total	2446	950	152

Total cost

*Brackets Total Cost Bs. 10,679,200,000

Note:The cost of Steel Bracket, Steel Chain or PC Cable will be increased by 20% and 50% of Concrete Bracket cost respectively.

*Strengtheing of Pier Total Cost Bs. 10,400,000,000

Note:The total number of piers for strengthening 400

Assumptions: Total Length 28km, Span Length 25m, Location of Pier 28000/25=1120

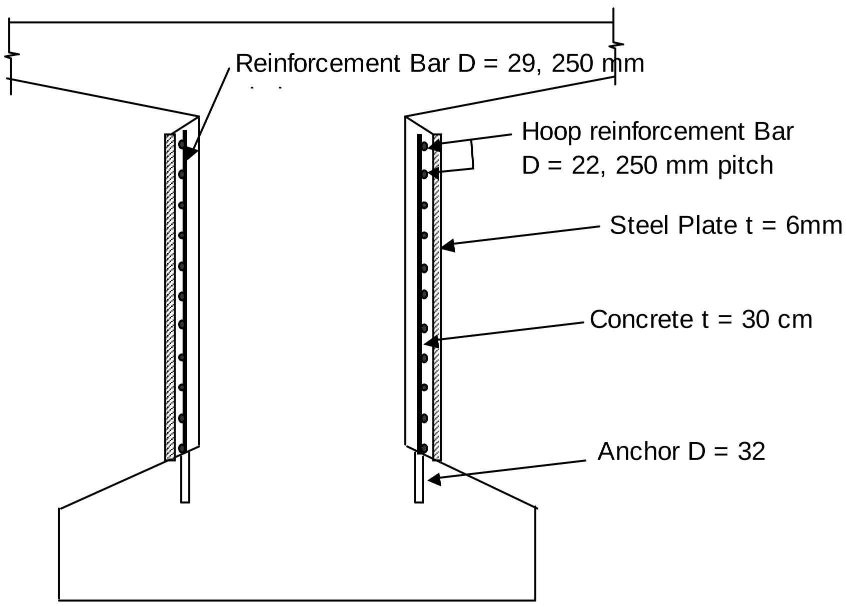
: Required Pier for Strengthening 30%, 1120/3=373

Unit cost for the strengthening of pier is estimated as shown in Attachment-3.

Attachment-3 Unit Cost Estimation of Strengthening of Piers

Sample in case of D=1.0m, H=7.5m

Item	Unit	Quantity	Unit price (Bs)	Cost (Bs)
Anchor	kg	170,37	2.071,56	352.939,73
Reinforcement Steel	kg	1.106,08	2.071,56	2.291.307,31
Bore Hole	piece	18,00	1.800,00	32.400,00
Grout	ml	6.273,72	263,00	1.649.988,36
Concrete	m³	9,18	588.206,69	5.402.384,34
Steel Plate	kg	1.774,73	2.233,56	3.963.961,47
Staging (10% of Cost)	L.S.	1,00		1.369.298,12
Indirect Cost (25% of Total Cost)	L.S.	1,00		3.765.569,83
Tax (17% of Total Cost)	L.S.	1,00		3.200.734,36
Project Administration Cost (15% of Cost)	L.S.	1,00		3.304.287,53
			Total	25.332.871,06



MONOPILE COLUMN REINFORCEMENT

S10

TOPOGRAPY AND GEOLOGY

“Disasters happen - we are prepared”

Michael Schmitz

STUDY ON
DISASTER PREVENTION BASIC PLAN
IN THE METROPOLITAN DISTRICT OF CARACAS

FINAL REPORT

SUPPORTING REPORT

S10

TOPOGRAPY AND GEOLOGY

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S-10 TOPOGRAPY AND GEOLOGY

CHAPTER 1. OUTLINE OF THE AVILA MOUNTAINS

1. 1 Geomorphology

The area of Caracas could be subdivided into three topographic units, which conform part of La Costa Range (Figure S10-1.1.1). These topographic units, from north to south are:

- *Topographic unit 1*, represented by Avila Massif, with 2,765 m altitude as maximum height (in Naiguatá Peak).

- *Topographic unit 2*, integrated by Caracas Valley, with heights that do not surpass the 900 m.

- *Topographic unit 3*, composed by hills at the east, west and south of Caracas, which heights are between 1,200 and 1,500 m.

<u>Topographic Unit 1. Ávila Massif</u>:

The Ávila Massive (Avila Mountains) forms only a small part of La Costa Range, which extends from Cabo Codera (at the east) to Barquisimeto depression (at the west) with orientation E-W, it runs parallel to the Venezuela's center coast. It covers an area approximated of 30 km^2, which extends from Humbolt peak to Naiguata peak with about 5 km long, conforming the highest altitudes of the Costa Range.

This work focuses only on the area closer to Caracas city; the limits of this area are: Catuche river (at the west), Caurimare river (at the east), Cota Mil or Boyaca highway (at the south) and the top of Avila Mountain (at the north).

The higher altitudes of the area are at the east of the mountains, and these are: Naiguata Peak (the highest point, with 2,765 m), Oriental Peak (2,637 m), Occidental Peak (2,478 m) and Humboldt Peak (2,153 m) also called Avila Peak (Figure S10-1.1.2). The lower altitudes are at the west of Humboldt Peak, for example Topo Infiernito, with 1,945 m. At this point the mountain seems to be wider.

The southern part of the mountain has steeper slopes than the northern part. In addition, the crest of the mountain presents an almost straight pattern at the eastern side of Naiguatá Peak; this pattern turns into a little curved shape at the western side of this point.

The most important hydrographic basins on the study area are (from east to west): Tocome stream, Chacaito stream, Cotiza stream and Catuche stream. All of them flow into Guaire river, located at the south of Caracas city. They also are N-S oriented and some of them follow fault lines; for example, Chacaito stream course follows the line of Chacaito fault.

Topographic Unit 2. Caracas Valley:

Caracas Valley is located at the southern part of the Ávila Massif, and is mainly composed of quaternary sediments which come from the adjacent mountains, mainly the Avila Massif. Its altitudes are between 600 and 900 m. It limits at the north with the Ávila Massif, at the south with El Hatillo hills, at the East with Mariches Crest and at the West with Los Teques Mountain. This unit presents an estimated area of 144 Km^2. The main rivers are the Guaire river and Valle river, the first one goes through Caracas Valley from west to east along 21 km in Caracas, then it turns south to the Tuy Valleys, where it flows into Tuy river. The Valle river comes from southwest of Caracas and then flows into the Guaire river at the center of the city.

Figure S10-1.1.3 shows the geomorphology of the alluvial fans in the eastern part of Caracas according to Dr. Singer. This morphologic unit is a tectonic trench or symmetrical graben, limited towards the north by the Avila fault, normal and with East-West direction; the faults that limit this graben towards the south are less pronounced, which is the reason why, in many cases, it would be named as semi-graben.

In the limit between the mountainous area of the Avila Massif and the Valley of Caracas, the streams, affluents of the Guaire River that have their origin in the mountainous zone, abruptly change of slope, giving rise to the deposition of alluvial fans.

Topographic Unit 3. East, West and South Hills of Caracas:

This is the most southern topographic unit and it limits at the north with Caracas Valley and at the South with the Tuy Valleys. This unit is composed by Los Teques Mountain (about 1,500 m) at the West of Caracas, the southern hills (El Hatillo and Volcán Hill: 1,491 m), and by Mariches Crest at the East (approximately about 1,200 m). Los Teques Mountain has an estimated orientation of SW-NE, the southern hills an E-W orientation (almost parallel to the Ávila Massif), and Mariches Crest has an N-S estimated orientation.

1. 2 Geology

The Caracas area is lithologically composed by rocks that belong to Ávila Metamorphic Asociation and Caracas Metasedimentary Asociation (RODRÍGUEZ et. al, 2002). These lithological distributions around the Avila Mountains are shown in Figure S10-1.2.1.

The Ávila Metamorphic Asociation extends from Carabobo state to Cabo Codera, Miranda state (from west to east, respectively) and covers the southern part of the Ávila Massif, in the area between the Ávila's crest until the contact with the quaternary sediments that fill Caracas Valley, at about 900 – 1,000 m. It is composed by the metamorphic rocks of San Julián Complex and Peña de Mora Augen Gneiss.

The *San Julián Complex* has its official location in San Julián River (Caraballeda, Vargas State), which was born on the Silla de Caracas. It is mainly composed by quartz-plagioclase-micaceous schists, of grey color on fresh surface and greenish or brownish colors on weathered surface. It also presents quartz-plagioclase-micaceous gneisses, with a quick gradation in its foliation, being more foliated at the schist contact. In addition, there are also minor lithologies such as marble, quartzite and mafic metaigneous (amphibolite, gabbro, diorite, tonalite and granodiorite).

The official location of the *Peña de Mora Augen Gneiss* is located in *Peña de Mora* area, in the old road Caracas-La Guaira. The characteristic lithologies of this unit are the coarse grain-banded and quartz-plagioclasic-microclinic gneisses, and fine to medium grain–quartz-plagioclasic-epidotic-biotitic gneisses, associated to amphibolic rocks.

The Ávila Metamorphic Association rocks are from Pre-Cambrian to Paleozoic ages, and they are representatives of a continental crust passive margin, representing an exhumed basement, where the foliation shows a big scale antiform structure. The Ávila Massif is a horst structure, mainly controlled by Macuto, San Sebastián and Ávila faults (URBANI 2002).

The Caracas Metasedimentary Association is a continue belt oriented E-W, which extends from Yaracuy state to Barlovento basin, Miranda state; covers the 2 and 3 topographic units, with a fault contact with the Ávila Metamorphic Association at the North (Ávila fault). This Association is composed by Las Mercedes and Las Brisas Schist.

Las Brisas Schist is composed by light colored rocks, which are mainly schist with a combination of muscovite, chlorite, quartz and albite. There are also metasandstone and metaconglomerates (URBANI 2002).

The same author says that *Las Mercedes Schist* is mainly represented by phyllite and graphite schist. These one have dark gray to black colors, and also have important quantities of quartz, muscovite, albite and calcite. Metasandstones are eventually found.

Both the two units (Las Mercedes and Las Brisas Schists) have marble bodies, mainly dolomitic in Las Brisas (Zenda Marble) and calcitic in Las Mercedes (Los Colorados Marble).

Las Brisas Schist rocks correspond with sediments from shelf environments of shallow waters, while Las Mercedes Schist represents deeper marine environments and anoxic conditions, with some sand bodies transported by turbiditic fluxes.

The sedimentation happened in a passive continental margin, in a poorly known basement (Sebastopol Gneiss), that could probably correspond with the limit of South American plate over the Guayana Massif extension.

In addition, AUDEMARD et al. (2002) points out the presence of serpentinites and lithologies from *Antimano Marble* on the southern flank of the Avila. Antimano Marble is composed by quartz-micaceous-graphitic schists and epidotic schists intercalated with marbles. These lithologies outcrop in the area between Blandin and San Bernardino. However, recent studies indicate that these lithologies are not so common in the southern part of the Avila Mountains.

Sedimentary Units

Kantak et. al.(2002) divide the geological units within the Caracas Valley in three groups: Alluvial fan deposits, which can be subdivided into a proximal and a distal facies, floodplain, and terrace deposits. The grain size of these fan sediments diminishes towards the south, to mix and fuse with fluvial sediments of the Guaire River. According to Singer (1977), in the apical and proximal parts of these deposits, near the Cota Mil, there can be observed materials of different grain size, with blocks of several cubic meters swept away by torrential avalanches and that would have their origin due to the occurrence of exceptional climatic phenomena (torrential rains). According to this author, the volume of material carried by holocenic torrential flows in the Valley of Caracas would be of 30 million cubic meters with base on an average thickness of 3 m. Until approximately 25 years ago, rock blocks of great magnitude could have been observed in El Pedregal, Altamira, San Michele, Sebucan and Los Palos Grandes. In the case of El Pedregal, those blocks were reduced by using dynamite in the process of urbanization.

Faulting:

The Costa Range inclusive of Avila massive is suffered orogenesis and many faults were formed in the rock masses as shown in Figure S10-1.2.2.

The study area is dominated by 2 main faults (Figure S10-1.2.1):

1) The Avila Fault, oriented E-W, normal and right lateral, located almost on the same course of Cota Mil Highway. It puts in contact the lithologies from Avila Metamorphic Association and Caracas Metamorphic Association. It starts on Tacagua Fault (at the west) and ends in the east coast of Carenero, near Cabo Codera, for an estimate extension of 110 km.

2) The Chacaito Fault oriented N-S and left lateral. Coincide with Chacaito stream course. It extends almost 4 km from Avila Fault to the top of the mountain, and it also extends to the shore, coinciding with San Julian river course.

Chacaito Fault marks the limit between several characteristics observed along Avila Mountain southern part. For example, AUDEMARD et al. (2002) said that the lithological distribution varies from one side to another.

1. 3 Flora

HUBER & ALARCON (1988, in STEPHAN 1991) have defined eight (8) different kinds of vegetation on the Avila Mountains: Littoral xerophytic bushes, deciduous lower montane tropophytic forests, sub-montane ambrophytic forests, seasonal semi-deciduous, sub-always green montane ombrophytic forest (or transition forest), sub-montane ombrophytic forest and montane always-green (or cloud forest), coastal sub – high barren plain (paramo), savannas and other herbal plants and gallery forests.

However, STEYERMARK & HUBER (1978, also in STEPHAN 1991) proposed a basic classification of the vegetation and its distribution in the southern part of the Avila Mountains. Figure S10-1.3.1 shows two transitions, the first Figure along the cable car's way and the other one from Altamira to Oriental Peak. As Figure S10-1.3.1 shows, the higher plants are located around the middle altitudes (*transition forest and cloudy forest*), where there is more humidity caused by the cloudy conditions (about 1,600 to 2,200 m altitude). In this forest, palms and orchids are common. At the higher part of the mountains (2,200 m and more), the vegetation is adapted to the poor hydric conditions and the strong winds (*Mid barren plain*). At this level the conditions are dry and the temperature is lower, and the plants are about 1 to 3 meters high (moss, little bamboos, Avila rose and some herbaceous plants are common). In the lower part of the mountains (from 900 to 1,600 m), the temperature increases and the soil is drier. At this level the vegetation does not grow so much, it also lost about 25 to 75% of their leaves during the dry season, and the man sometimes has made harvests or reforestations on these areas (*savanna* or Sub-montane Ambrophytic Forests).

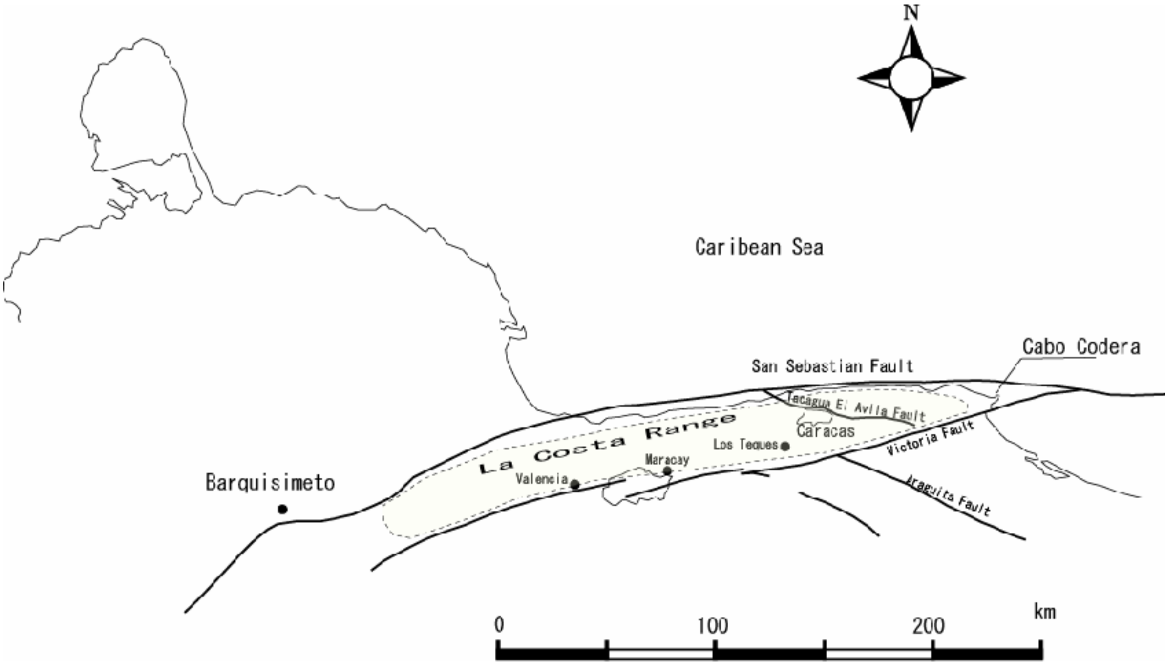


Figure S10-1.1.1　Location of La Costa Range

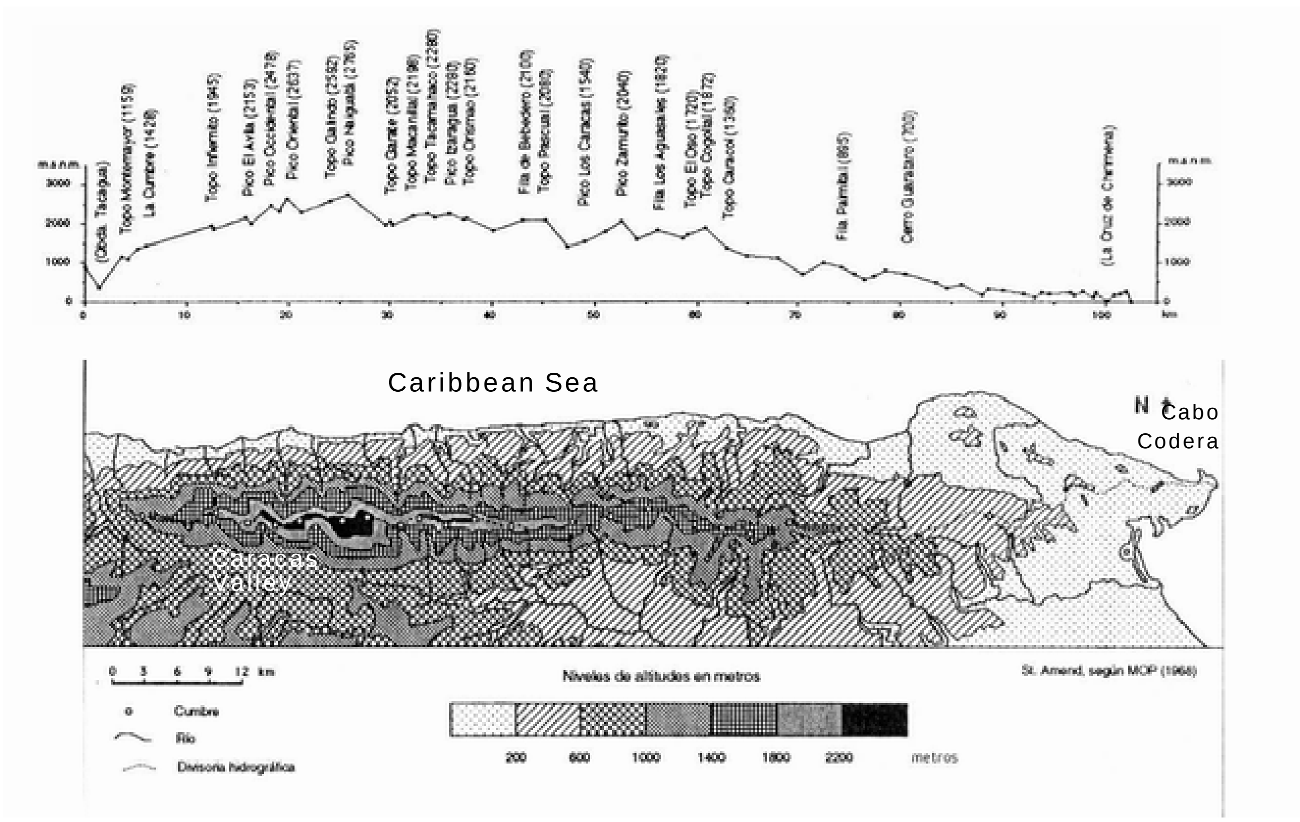


Figure S10-1.1.2　Topography of Caracas Valley [S10-6)]

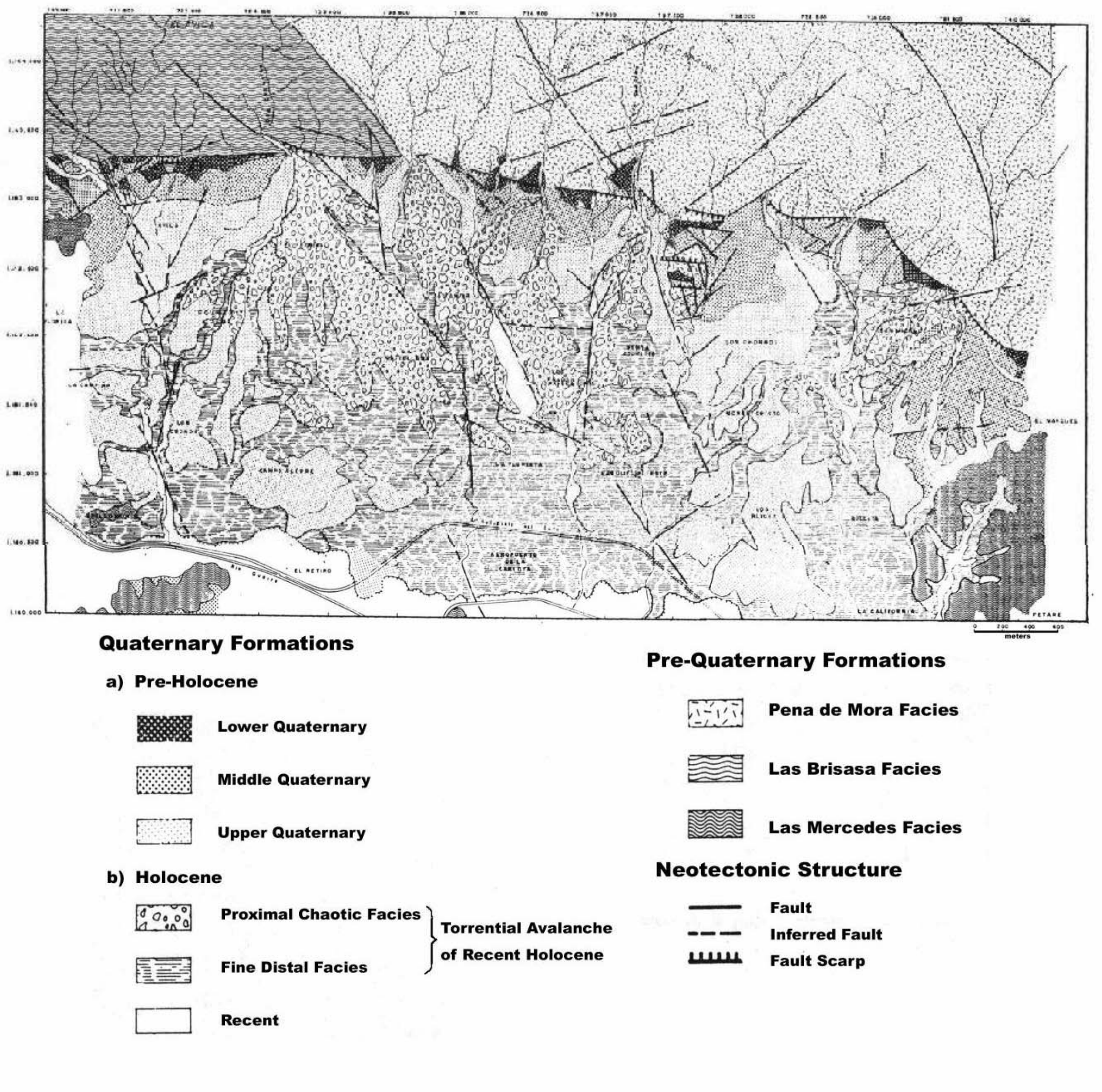


Figure S10-1.1.3 Geomorphology of the Alluvial Fans in Caracas Valley [S10-5)]

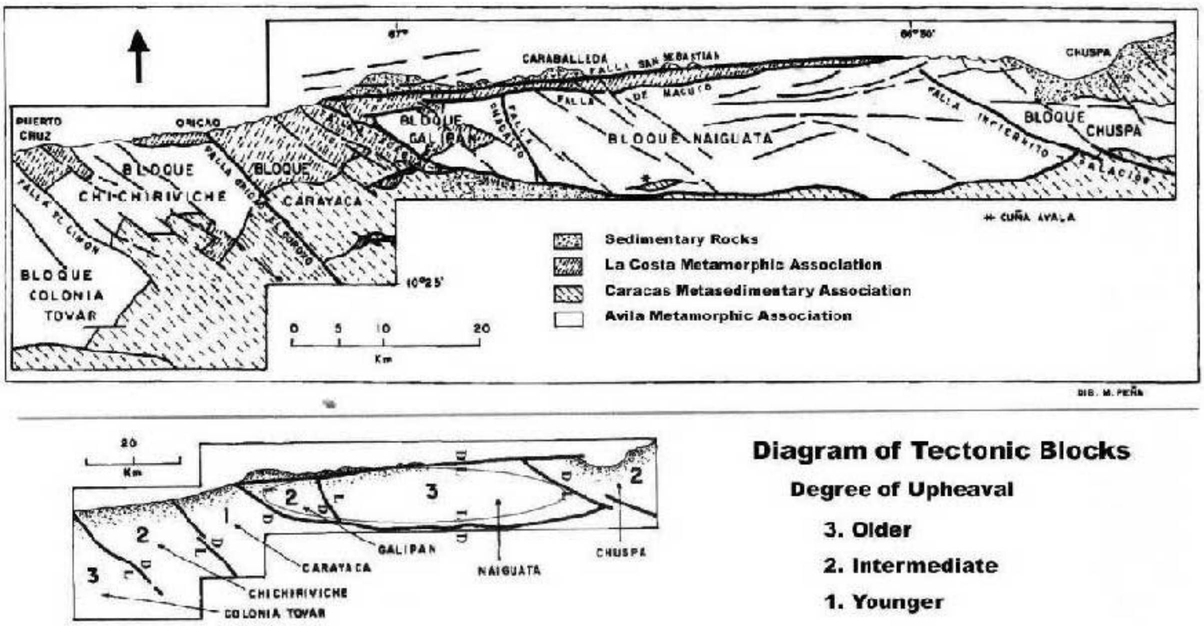


Figure S10-1.2.1 Geological Map of Avila Mountains [S10-2)]

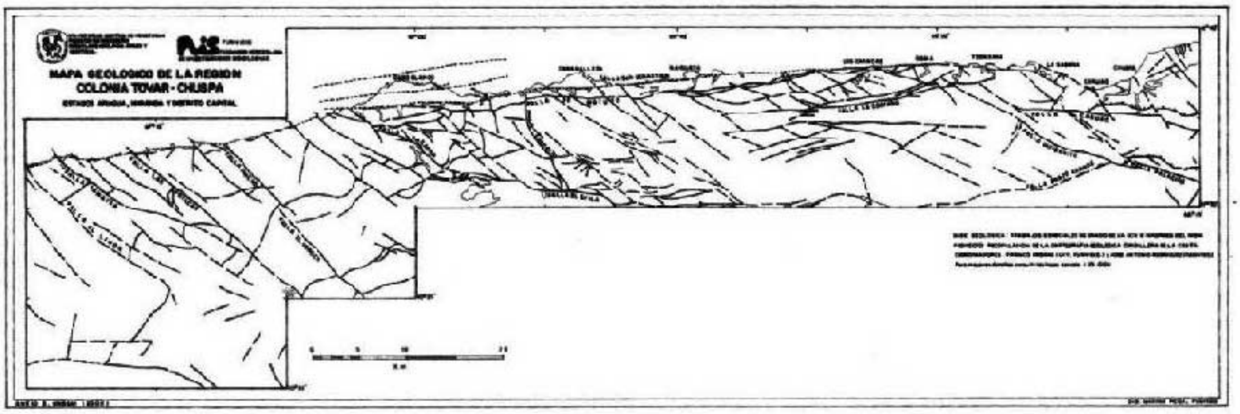


Figure S10-1.2.2 Faults in Avila Mountains [S10-2)]

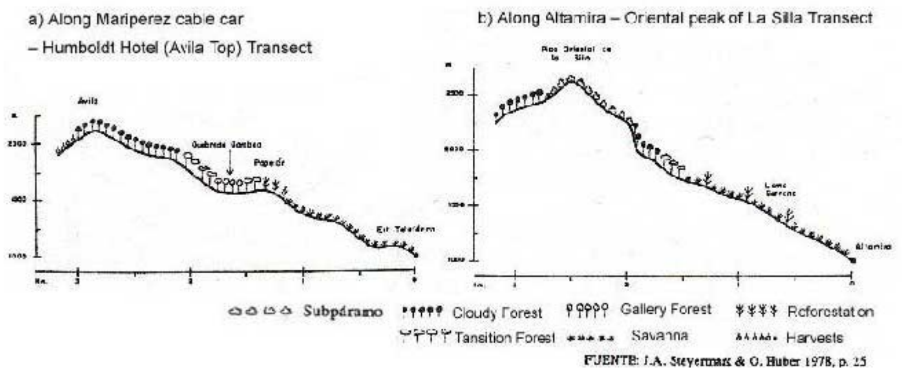


Figure S10-1.3.1 Vegetation Transition in South Slope of Avila Mountains S10-6)

CHAPTER 2. GEOMORPHOLOGIC AND GEOLOGICAL SURVEY

2. 1 Outline of the Study

2. 1. 1. Purpose of the Study

The purpose of the geomorphologic and geological survey is to find how much debris could be flow out in the next disaster. To do that, zoning of hazardous areas and estimation of unstable soil in basins were executed.

a. Zoning of Hazardous Area

From the geomorphologic, geological and biological point of view, the study area should be zoned based on the hazardous condition using the following information;

Geomorphology – aerial photos, satellite images, topographic map

Geology – existing information, site reconnaissance

Flora – existing information, satellite images

b. Estimation of volume of unstable soil in the basins

Investigating thickness of covered soil and thickness of highly weathered rocks in the collapse area, the volume of soil in the basins could be estimated.

Collapse area – aerial photos

Volume of unstable soil – site reconnaissance

2. 1. 2. Method of the Study

Topographic Map Analysis

Drainage systems, gradient of streams and geomorphic anomalies were analyzed using the topographic maps. The maps used for analysis were 1:5000 scale topographic maps issued in 1954 and 1984

Aerial Photo Analysis

The following topographic analysis has been done using stereoscopes on aerial stereo photos.

a. Geomorphologic signs of debris flows
b. Covered material in the basins such as vegetations
c. Existence of reservoirs, deforestations in the basins
d. Existence of civil works in the basins
e. State of collapses in the basins
f. State of bed material
g. Lineaments

The following aerial photos employed.

1:25,000 scale, March 2002 photographed

1:25,000 scale and 1:5,000 scale, December 1999 photographed

1:25,000 scale, February 1994 photographed

Satellite Image Analysis

Satellite Images are employed for the following purposes.

a. To watch wide area with homogeneous precision – perspective view
b. To identify lineaments
c. To analyse the states of vegetation with infrared band
d. To analyse the relative water content of the ground in non-vegetation land
e. As supplementary images to aerial photos

The satellite images employed are ASTER (Advanced Spaceborne Thermal Emission and Reflection radiometer) developed by Ministry of Economy, Trade and Industry, Japan. ASTER is the earth resource observation sensor with 14 bands and 15 m resolution loaded on Terra which was launched in December 1999.

The satellite Image employed in this project was taken on 20th May 2003.

Field Study

The following items were studied mainly in the 1st field study in the Avila Mountains in 2003;

a. Cross section sketches of streams (some sketches each stream)
b. Estimate the depth and volume of slope collapses
c. Estimate the thickness of sediment at streambeds and classify the material into gravel, sand and clay
d. Measure the size of major rolling stones
e. Existence of unstable ground
f. Details of debris flow prevention structure
g. Geology
h. State of water flow and spring
i. Details of slope collapse on upstream

The following items were studied mainly in the 4th field study in the Avila Mountains in 2004;

j. Rock weathering on slopes
k. Grain size of weathering rocks

2. 2 Result of the Study

2. 2. 1. Topography

Drainage System

Generally, a river and its tributaries constitute a network whose pattern can be influenced by the position and shape of boundaries separating the various rocks within a catchment. The drainage pattern can be classified into following 6 typical drainage patterns (Figure S10-2.2.2).

a. Dendritic pattern : uniform condition of drainage
b. Feather like pattern
c. Parallel pattern
d. Radial-centrifugal pattern
e. Radial-centripetal pattern
f. Trellis / Angular pattern

The drainage systems in the Avila Mountains are shown in Figure S10-2.2.1.

The dendritic pattern is significant in the south slopes of the Avila Mountains and the trellis / angular pattern is distinguished in many places. Figure S10-2.2.3 shows typical picture of trellis / angular type drainage pattern in the Avila Mountains. It is related to the development of faults in the area. Taking accounts of debris flows in the area, it is anticipated that the energy of debris flow could be weaken in the trellis / angular type drainage pattern. The drainage systems in Catuche basin and Cotiza basin, however, seem to be the dendritic pattern and to be more complicated than the drainage systems in other basins.

Grade of Streams

Figure S10-2.2.4 shows that the profiles taken along the course of the main streams. The profiles which show irregular curves are steeper where the river crosses more resistant rocks, and are flatter where it flows over more easily erode rocks, since the streams in the Avila Mountains are geologically young and actively eroding.

The profiles of streams in west side from the Chacaito stream tend to be gentler than the streams in east side from the Chacaito stream. This may show that the streams in the western side are in more mature stages and are lower capability to convey debris.

Making a comparison Chacaito stream and Tocome stream, the profile of Tocome stream has a steeper curve from lower stream and has some steps. The profile of Chacaito steam is gentle in lower stream and becomes steeper in the upper stream. There is no step in Chacaito stream, and is deeper from surrounding crests than other basins.　This may be because of the existence of Chacaito Fault along the bottom of the Chacaito Stream.　Catuche stream has no flatter portions in its profile as well as Chacaito stream.

According to Prof. Andre Singer of UCV, most of debris in Tocome basin was deposited at the flatter portions of the stream in 1999 Disaster.　This shows that the flatter portions has buffer against debris flow.

Lineament

Many lineament can be seen on the aerophoto and the satellite image in the study area.　Figure S10-2.2.5 shows the lineaments in the study area.　The lineaments from north east to south west are most distinguished in the area, and from north west to south east are next.

The Tocome and Gamboa basins have relatively less lineament.

Generally, most lineaments are topographical manifestation of faults.　According to the faults map shown in Figure S10-1.2.2, the faults from north west to south east are distinguished and the faults from north east to south west are less.　It is not clear why there are many faults from north east to south west which have not been regarded as lineaments, or that the lineaments in this area dose not show faults.　However, the lineaments suggest existence of more north east - south west direction faults in the Avila Mountains.

The major lineament which is consistent with fault is the lineament along Chacaito stream – Chacaito Fault.　The lineament is not clear on the major fault runs in Tocome basin from north west to south east.

The lineament shows that there may be another big fault along Quintero stream where no fault is shown in the geological map.

Geomorphic Anomalies

Upper end of Caurimare, Garindo, Quintero and Chacaito basins has flatter areas with small mounds (Pico Naiguata, Topo La Danta to Topo Galind、Pico Oriental to Asiento de La Silla、Pico Occidental to Lagunazo).　According to the geological map (Figure S10-2.2.6), these flatter areas are not the geological anomaly.

2. 2. 2. Geology (Lithology)

Figure S10-2.2.6 shows the geological map of the south slopes of the Avila Mountains which was prepared by field survey in this study being based on the geological map which was issued by UCV, FUNVISIS (2001).

The list of lithology is shown bellow. The general description of the lithology of the Avila Mountains is in Chapter 1.2.

By the filed survey in the Avila Mountains, any big differences in the lithology in the Avila Mountains were not found. The most important thing in the Lithology is whether there are geological differences between east side of Chacaito basin and west side of Chacaito basin. On the east side of Chacaito Fault, there are schist of San Julian Complex and gneisses (from Pena de Mora Augen gneiss) outcropping. These rocks are competent and break forming big blocks. On the west side of the fault, there is schist from San Julian Complex mainly. Some marbles, graphitic schist (from Antimano Marble) and also serpentinite can be seen in the west side. These three (3) lithologies are more susceptible to chemical and mechanical weathering, which is equal to unstable material. If relatively soft rocks such as marble or serpentinite are more in west side, the debris flows are prone to occur in the west side. By this survey, it seems that there are more marble and serpentinite in the west side. However, marble and serpentinite is not the major rocks even in the west side and it is not clear whether there are less marble or serpentinite in the east side. Most of the rocks in the Avila Mountains even in the west side are member of schist or gneiss as shown in bellow and they are not much different in engineering characteristics such as strength, weathering-proof.

List of the Lithology Found in the Avila Mountains

Metamorphic Association La Cost (Mesozoic)	
CN	Nirgua Amphibolite
Meta-Sedimentary Association Caracas (Mesozoic)	
CaM	Las Mercedes Schist
Metamorphic Association Avila (Pre-Mesozoic)	
A	Metamorphic Association Avila
AN	Naiguata Metagranite
ATc	Tocome Metaigneous
ASJ	San Julian Complex
ASJe	Quarts – Muscovitic Schist
ASJa	Amphibolitic Schist and Plagioclase – Epidotic Schist

ASJap	Amphibole – Plagioclase – Epidotic Schist
ASJp	Plagioclase – Epidotic Schist
ASJt	Metatonalites
APM	Augengneiss Pena de Mora
APMp	Plagioclase – Micaceous – Epidotic Augengneiss
APMc	Plagioclase – Quartz – Micaceous Augengneiss

2. 2. 3. Slope Collapses

Generally, collapse in mountain slope occurs in heavy rain and can trigger off debris flow. To study the collapses in mountain, debris flow's history and potential of debris flow in the mountain could be estimated.

Traces of collapse were collected on the aerial photos and the field survey and marked on map as shown in Figure S10-2.2.8. The traces of collapse in the basins of the Avila Mountains can be classified into following 5 types based on freshness and vegetation. Figure S10-2.2.7 also shows the types of collapses.

Type 1: Very Active Collapse :	active collapse with exposure of soil /rock, no vegetation covers
Type 2: Active Collapse 1 :	active collapse covered with bush or grass, collapse occurred in recent year
Type 3: Active Collapse 2 :	active collapse covered with sparse trees, a collapse might occur under trees in recent year
Type 4: Old Collapse 1 :	old collapse covered with bush or grass
Type 5: Old Collapse 2 :	old collapse covered with trees

Many of the Type 1 to Type 3 collapses can be seen in the Cotiza and Catuche basins. Type 1 is hard to be seen in other basins, and Type 2 and Type 3 are relatively less in other basins. Type 1 to Type 3 may be scars of 1999 Disasters and others are older scars before 1999 Disaster. Figure S10-2.2.8 shows that there were many collapses occurred in the Cotiza and Catuche basins where debris flows occurred in 1999 Disaster. The existence of many Type 1 collapses in the basins shows that the basins have not been recovered. Type 4 and Type 5 are scattered in whole area of the site, however more congested in lower altitude of the Avila Mountains than higher. It shows that collapse and debris flow triggered by the collapse can occur in any other places of the Avila Mountains.

It seems that many of Type 2 and Type 3 collapses are on the west or north portion of each basin, especially in Tocome basin.

Ratio of volume of remaining soil on collapses to the total volume of collapse soil is about 0.3, according to the field survey (Figure S10-2.2.9).

2. 2. 4. Weathering of the Avila Mountains

Rock weathering in the Avila Mountains was studied in accordance with BS 5930 which is shown on Figure S10-2.2.10. Sketches of weathering on slopes in whole Avila Mountains were made in the field study as examples are shown in Figure S10-2.2.11. Photographs in Figure S10-2.2.12 show the actual image of each weathering grade. The material belongs to Weathering Grades VI to IV on slopes tend to fall down or be flushed out in heavy rain, and Grade III which is rocky and quite hard could remain on slopes. We call Grade VI to IV “Weathered Zone” in this report.

Figure S10-2.2.13 shows thickness of weathered zone in each basin. The thicknesses of weathered zone in the east basins are thinner and in the west are thicker. The thicknesses of weathered zone of eastern side from Chacaito are less than 5 m and average about 2.5 m, and of western side from Chapellin are mostly over 10 m and average about 7.5 m. This may be because of gradient of mountain slopes. The slopes in the east side of the Avila Mountains are steeper and weathered material hardly stays on slopes.

Figure S10-2.2.14 shows relation between the thickness of weathered zone and elevation. In the east side from Chacaito, the relation is not clear. In the west side from Cotiza, the thickness of weathered zone seems to become thicker as elevation increases. In between Chacaito and Cotiza, the thickness of weathered zone seems to become thinner as elevation increases. The Avila Mountains can be zoned into 3 levels of thickness of weathered zone as shown in Figure S10-2.2.15.

Figure S10-2.2.16 and Table S10-2.2.1 show that grain size of soil collected from weathered zone. Most of the samples are classified into gravel, and silt, clay contents are less than 10 %.

Rocks become stable through the process of weathering from base rock to clay. The standard process of the weathering is; base rock, rock mass, boulder, gravel, sand and clay. In actual conditions, however, there are discontinuities of weathering process, and the natural rocks do not follow all of standard process of weathering. Granite presents as only rock mass or sand mostly. There is discontinuity in weathering of granite between rock mass and sand, and it hardly presents as gravel. The discontinuity of weathering is because of existence of stable and unstable stages of weathering process on rocks. The weathering is a complex mechanical (physical) and chemical processes. Mechanical weathering breaks down rocks into small particles by the action of temperature, by impact from rain drops and by abrasion from mineral particles carried in the wind.

In very hot and very cold climates changes of temperature produce flaking of exposed rock surfaces. Chemical weathering is the break-down of minerals into new compounds by the action of chemical agents; acids in the air, in rain and in river water, although they act slowly, produce noticeable effects especially in soluble rocks. The rocks broken down into small particles by mechanical process make its contact area against water wider, and the chemical disintegration of the rock becomes more active with wider contact area. Then, the rock broken down into small particles by mechanical process becomes smaller and clayey by chemical processes. The water is important role in chemical process. Therefore the chemical process is not active under dry climate.

In Caracas and the Avila Mountains where are not wet area, chemical process is not active compared with wet Japan. The weathered soil in Japan contains a lot of silt and clay, whereas the weathered soil in the Avila Mountains contains less silt and clay.

The event which triggers a debris flow is movement of moisturized soil at upper stream. It is neither rock fall nor rock avalanche but soil collapse. On the other hand, there are many big boulders on stream bed as shown in Figure S10-2.2.19. Most of the boulders were not brought by stream, but may have fallen from slope or cliff beside the stream. The fact that most of the boulders are under cliffs or rock slopes suggests it. These boulders might fall down by earthquake, heavy rain or waving trees in strong wind. Some boulders did not fall down to streams and stopped on slopes. When debris flow occurs, therefore, many boulders on stream bed will wash away with water and debris, and less boulders will fall down from slopes.

2. 2. 5. Debris

Figure S10-2.2.17 shows the deposited debris on stream bed. It was drawn by in-house study using aerophotos, topographic map and in the field survey. Figure S10-2.2.18 shows the examples of cross section of stream in the Avila Mountains drawn in the field survey. The thickness of debris is also estimated by the cross sections.

Main material of debris seems same as the material of weathered zone as shown in Figure S10-2.2.19. The material of debris founded at the east side is composed mainly of big blocks (schist and gneiss), while in the western side, the material is more trees, boulders and not so bigger blocks.

According to Figure S10-2.2.17, basins in eastern side from Chacaito have more debris on stream bed than the western side. The slope gradients in east side are steeper than west side and soil on slopes (highly weathered part of rock) flow down to the river bed as mentioned in the previous section. The streams in east side have gentle gradient steps and the soil flowed down can be staying on the steps.

2. 2. 6. Vegetation

Figure S10-2.2.20 shows the infrared band satellite image. Upper part of the Avila Mountains covered with vivid red colour. The vivid colour indicates thick vegetation and it is above about 1,700m altitude. The top of the Avila Mountains, above 2,400 – 2,500m altitude is brownish colour and shows changing of vegetation.

Satellite Image matches with the vegetation distribution shown in Figure S10-1.3.1 or Figure S10-2.2.23. It is easy to discern different types of vegetation arranged in horizontal stripes in the Avila Mountains.

Catuche basin and Cotiza basin are in relatively lower altitude. The vegetation in the basins of Catuche and Cotiza could be thinner, because the most of catchments areas are below 1,700m altitude.

Many traces of collapses in the north slopes of the Avila Mountains in Vargas can be seen in satellite image in Figure S10-2.2.20. They are the collapses of 1999 Disaster, which has not been recovered. There are not many collapses in the upper part of the Avila Mountains. It may shows that less collapses occur in thick vegetation above 1,700m altitude (the detail is in next chapter).

Gray patches in lower and western part from Tocome basin are trace of forest fires and herbage covers there.

Different types of vegetation arranged in horizontal stripes in the Avila Mountains shall be caused by horizontal stripes of average temperature and annual rain fall which are changing by altitude. Figure S10-2.2.22 is the distribution of annual average temperature and annual precipitation around the Avila Mountains. They show clear stripes of temperature and precipitation, which are parallel to the topographical contour.

AUDEMARD et al. (2002) said that the presence of organic matter (wood, plants and trees) on the west side is caused to the presence of saprolitic soils, on which the roots of the plants and trees grow in a shallow level. This causes that the vegetation in western side of Chacaito Fault, would not be so anchored to the soil.

Table S10-2.2.1 Summary of Grain Size Analysis

	mm	E-4-1	E-5-2	E-11-1	E-12-5	E-14-1	E-15-2	E-17-1	E-19-3	E-19-3M 31v	E-19-3 III	E-22-2	E-23-1	E-25-1
gravel	>200mm	58.6	56.1	49.6	69.5	68.8	32.0	57.1	54.5	60.5	63.5	51.3	58.0	69.4
sand	0.075-200mm	36.6	38.4	46.6	29.0	26.8	58.1	40.6	37.9	35.3	33.1	41.0	40.2	28.1
cohesive	<0.075mm	4.8	5.5	3.8	1.5	4.5	9.9	2.3	7.6	4.2	3.4	7.7	1.8	2.5
		G	G	G	G	G	S-F	G	G-F	G	G	G-F	G	G

	mm	E-28-2	E-30-2	E-33-3	E-37-1	E-41-6	E-41-9	E-42-2	E-44-2	E-44-4	E-44-6
gravel	>200mm	42.4	47.9	93.9	24.3	86.4	50.7	74.2	79.0	48.2	52.2
sand	0.075-200mm	49.1	48.4	5.7	69.3	11.9	45.3	22.7	18.2	44.7	42.3
cohesive	<0.075mm	8.5	3.7	0.4	6.4	1.7	3.9	3.1	2.7	7.1	5.5
		S-F	S	G	S	G	G	G	G	G-F	G

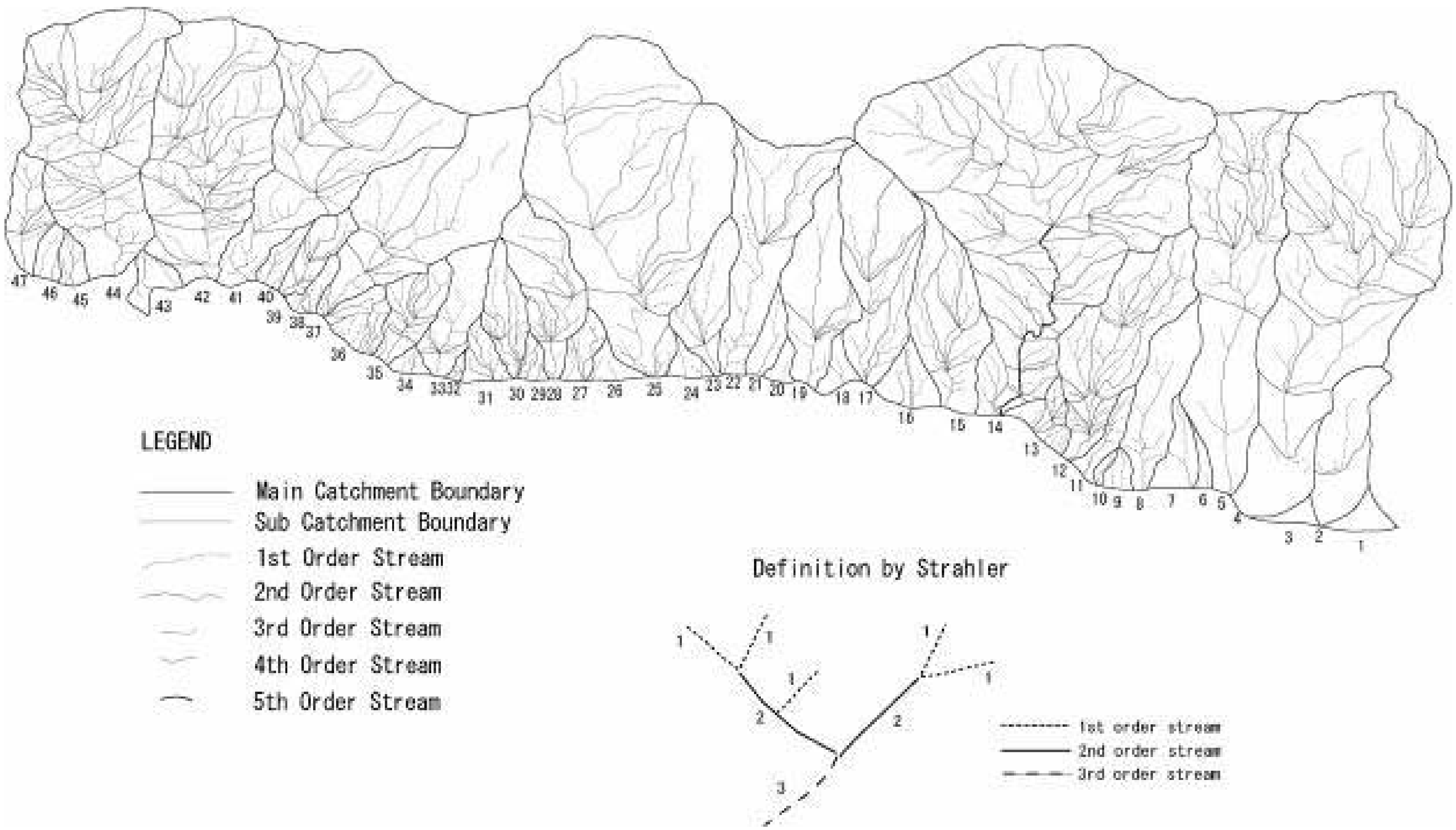


Figure S10-2.2.1 Stream Order

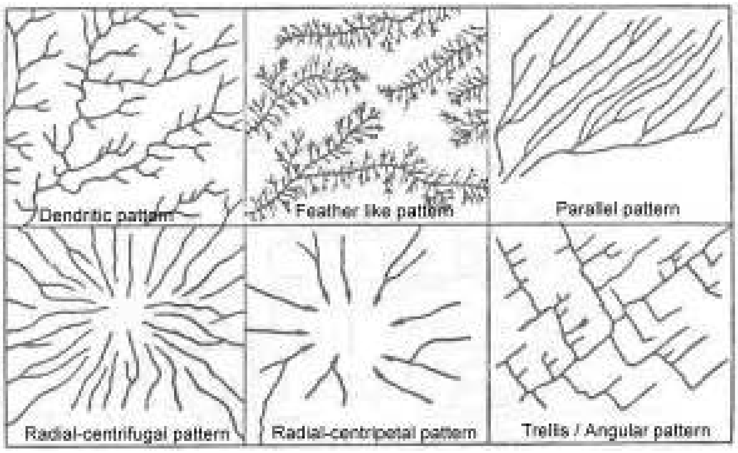
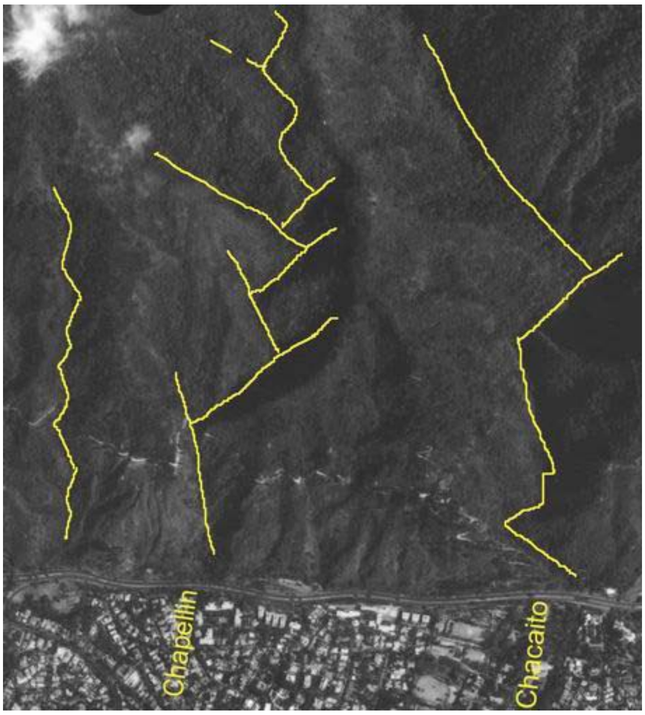


Figure S10-2.2.2　Drainage System (Imamura et al.)



Drainage Pattern

Original Photo

Figure S10-2.2.3　Typical Angular Pattern Drainage in Avilla Mountains (Aerophoto)

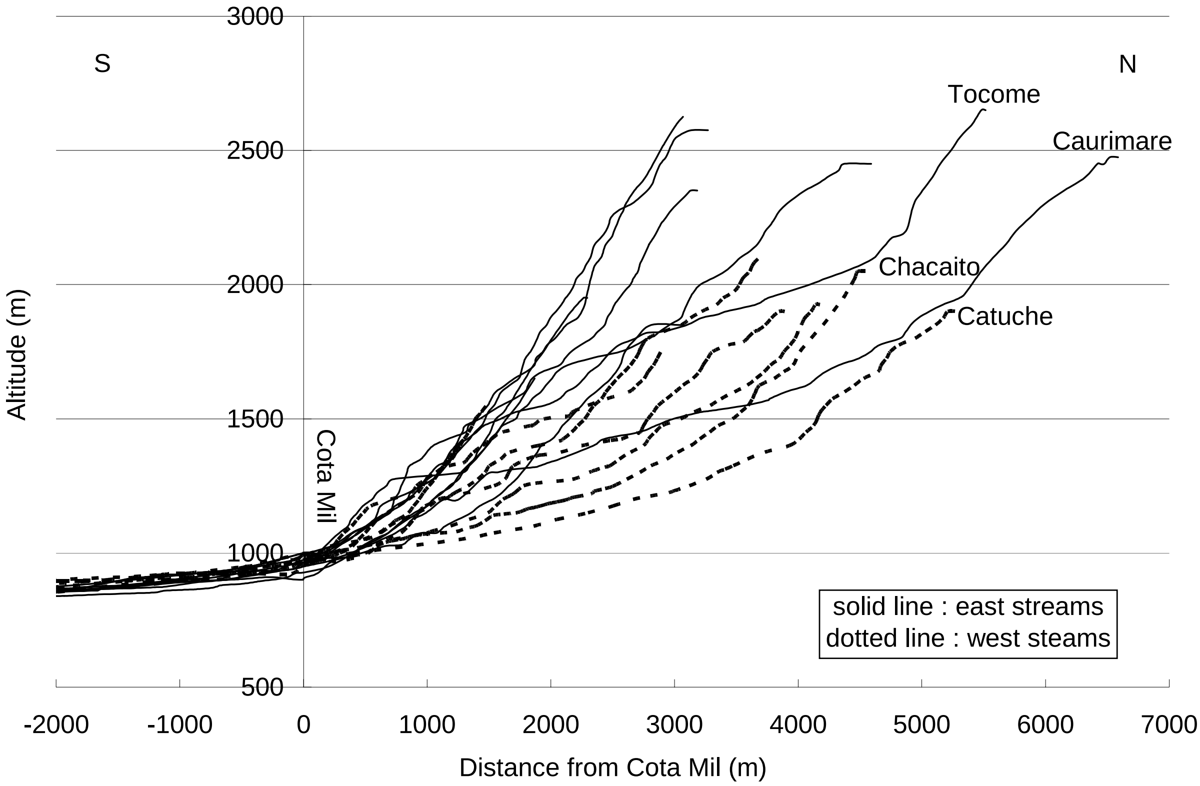


Figure S10-2.2.4 Gradient of Streams

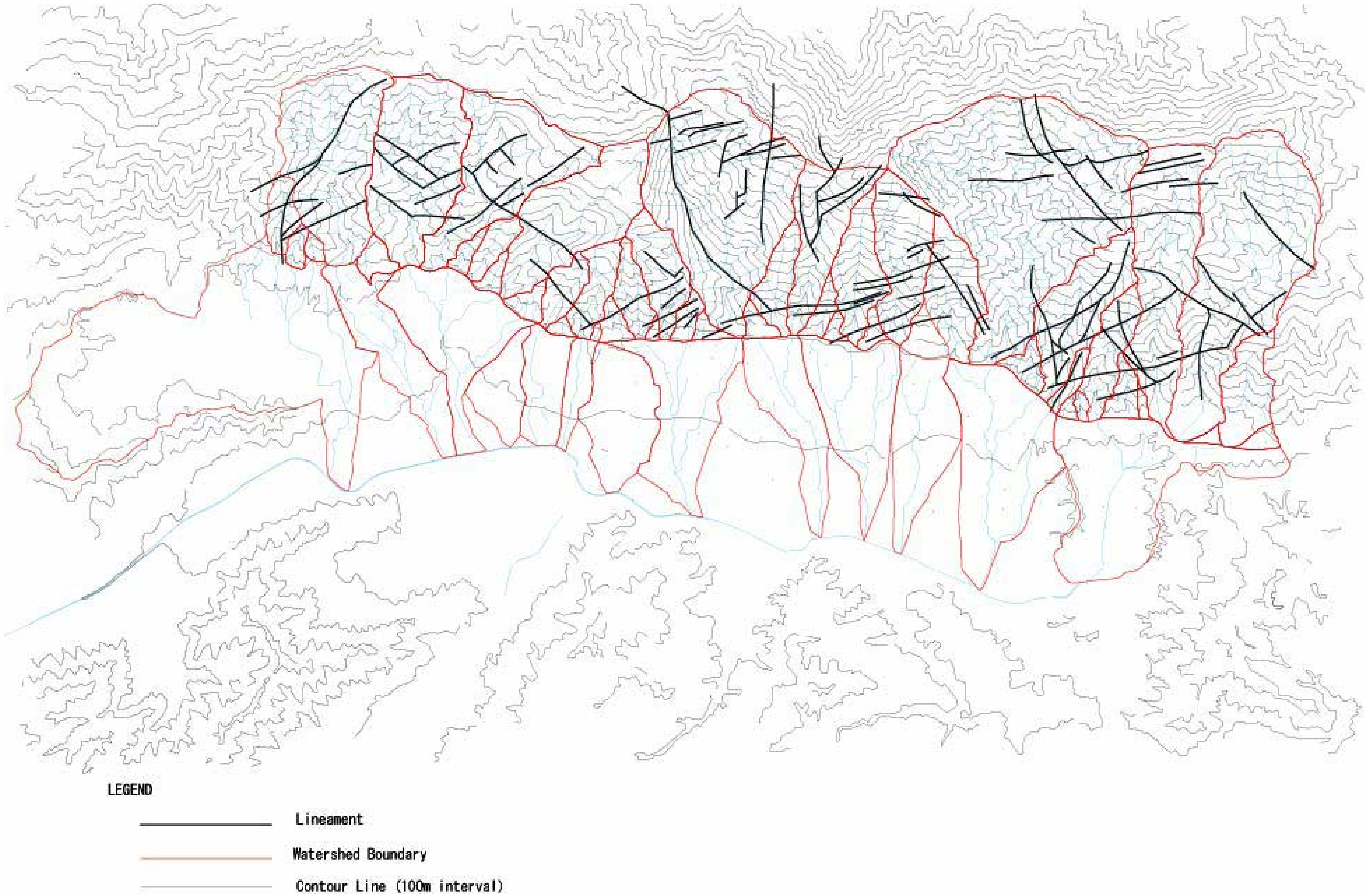


Figure S10-2.2.5 Lineament

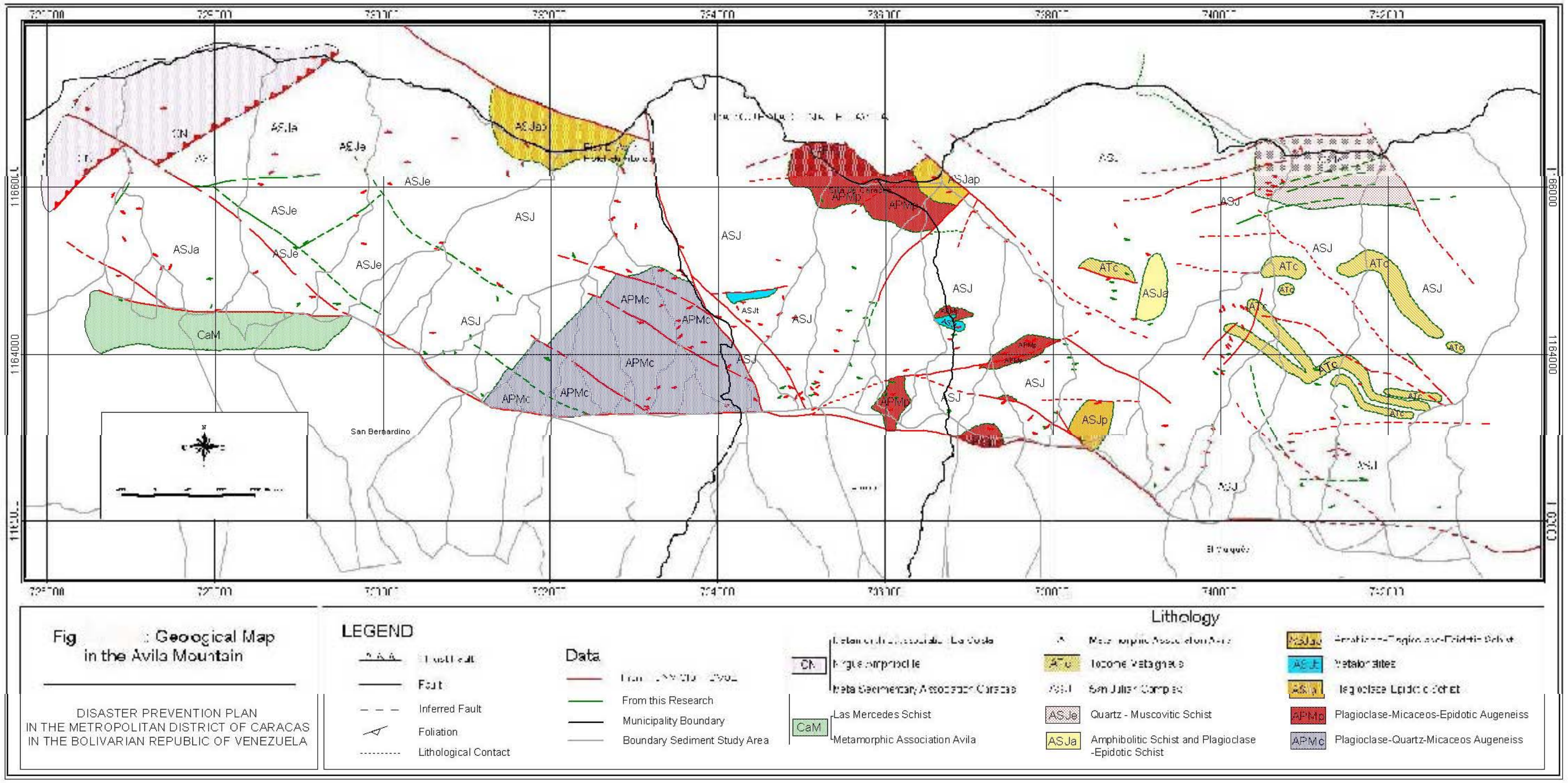


Figure S10-2.2.6 Geological Map of the Avila Mountain (Prepared by Study Team)

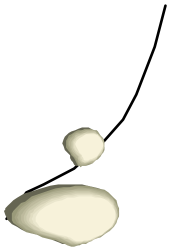
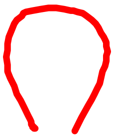
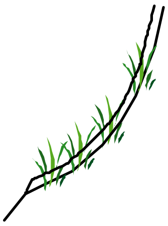
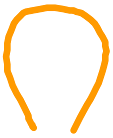
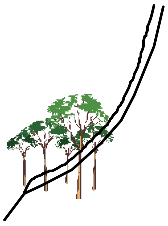
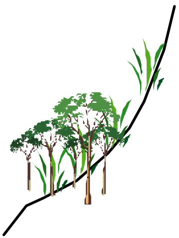
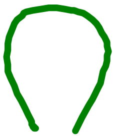
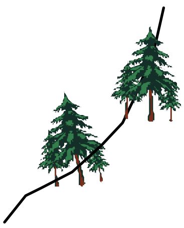
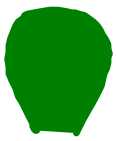
Type	Slope Condition	Legend	Symbol	Description
1		Active Collapse		an active collapse with exposure of soil /rock, no vegetation covers
2		New Collapse covered with Grass		an active collapse covered with bush or grass, collapse occurred in recent years
3		New Collapse under Trees		an active collapse covered with sparse trees, a collapse might occur under trees in recent years
5		Old Collapse without tree		an old collapse covered with bush or grass
4		Old Collapse covered with trees		an old collapse covered with trees

Figure S10-2.2.7 Interpreted Slope

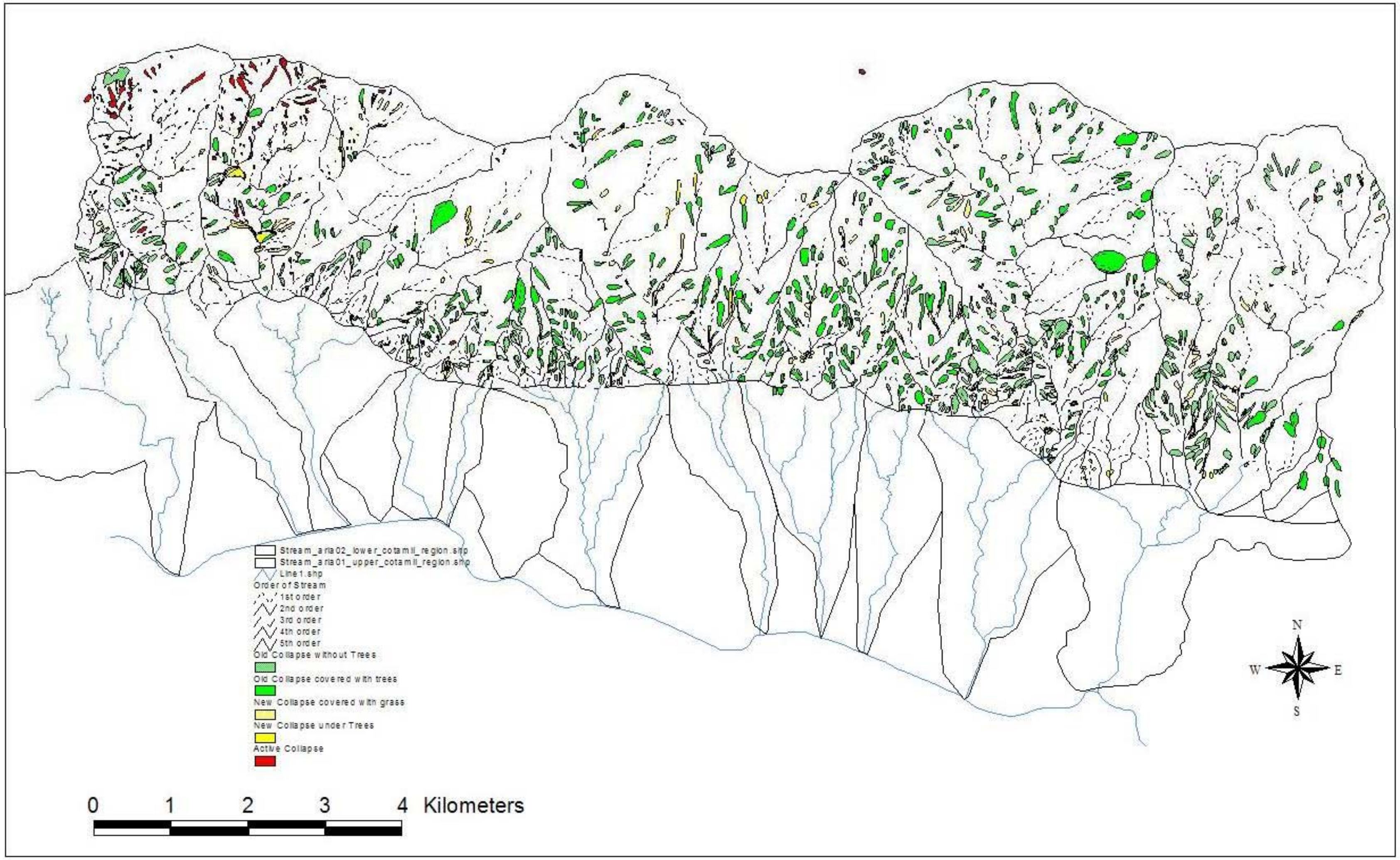


Figure S10-2.2.8 Potential Slope Failure

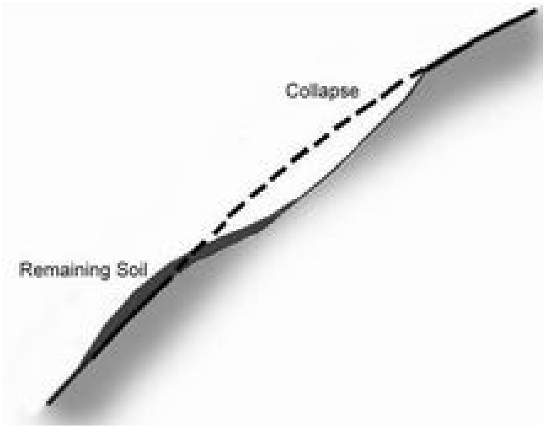
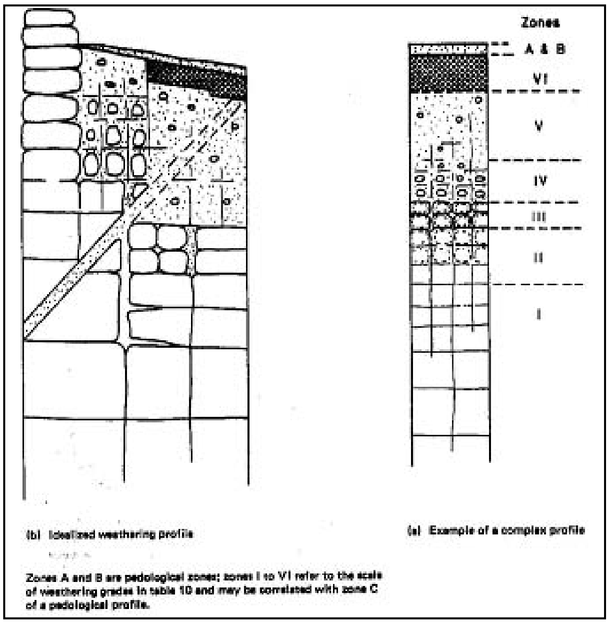
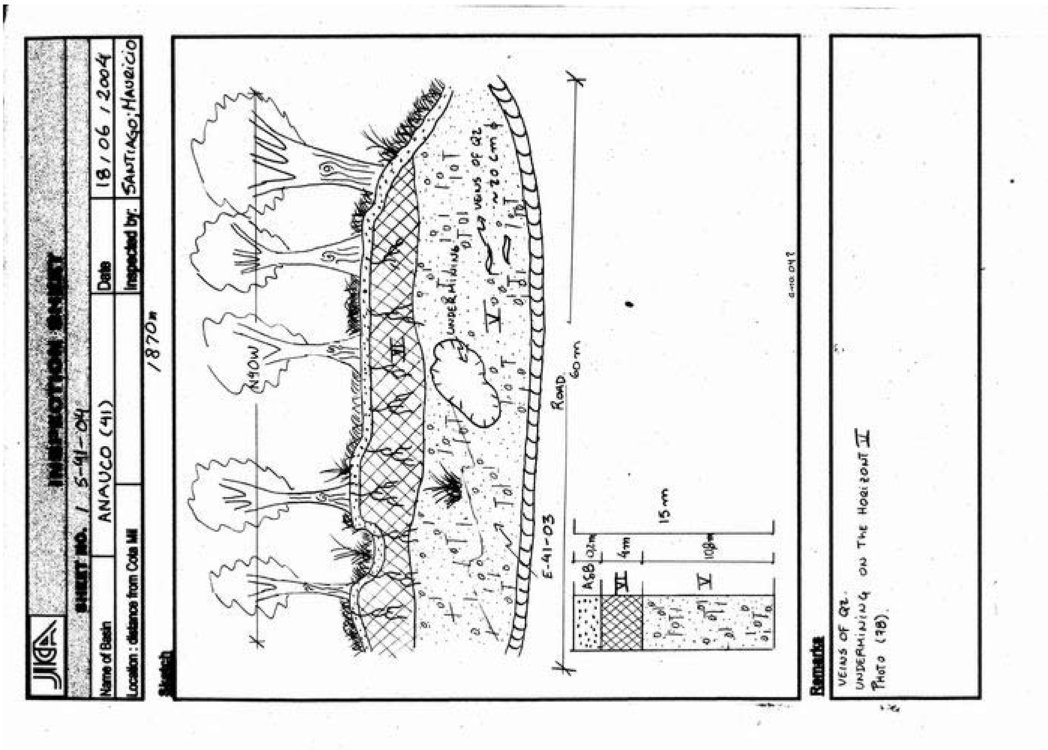


Figure S10-2.2.9 Remaining Soil of Collapse



Term	Description	Grade
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Highly weathered	More than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	IV
Moderately weathered	Less than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.	III
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering.	II
Fresh	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.	I

Figure S10-2.2.10 Diagrammatic Representation of a Simplified Weathered Profile in Massive Rock (British Standard ; BS5930)



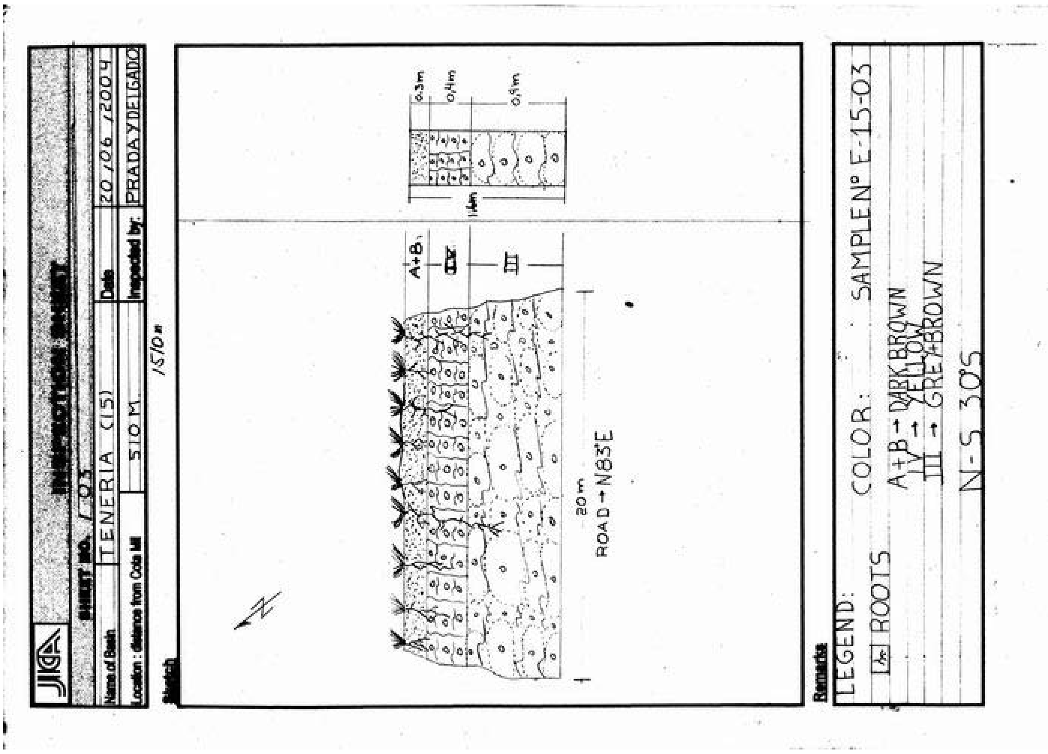


Figure S10-2.2.11 Sketches for Weathering Study

Weathering Grade VI – V

Upper part of slope (whitish color) is Grade VI

Weathering Grade V

This slope consists of only Grade V

Weathering Grade V – IV

Upper one third of this slope is Grade V

Weathering Grade III

Weathering Grade II

Figure S10-2.2.12 Rock Weathering Grade in Avila Mountains

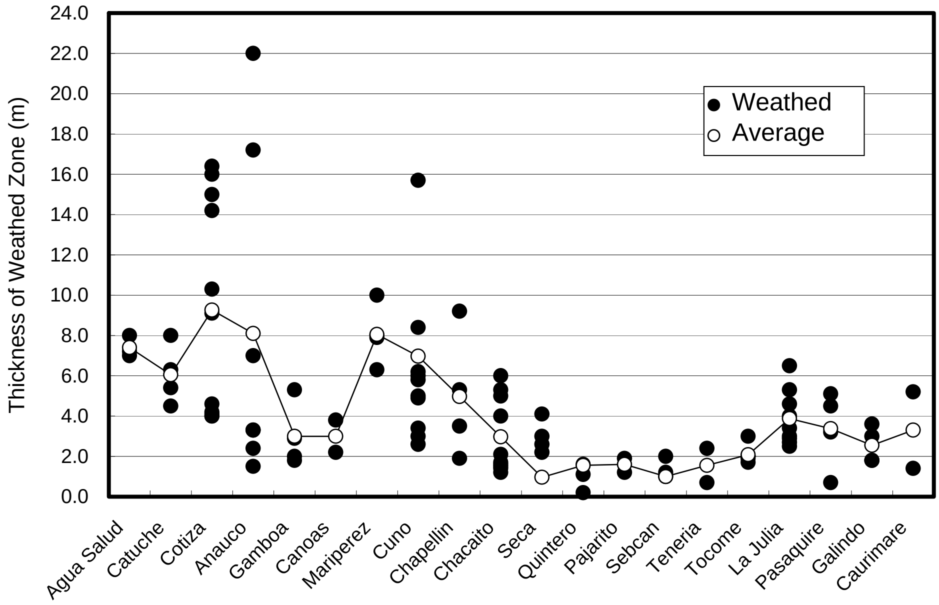


Figure S10-2.2.13 Thickness of Weathered Zone along Each Basin

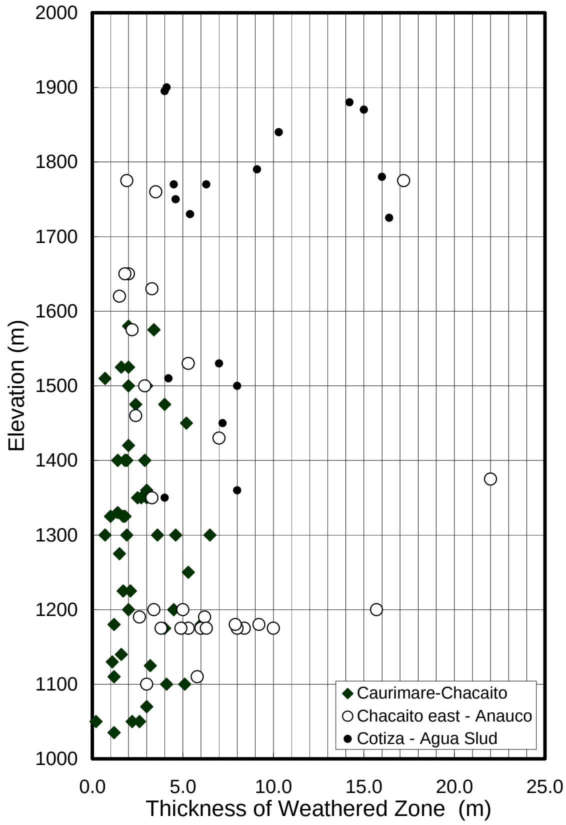


Figure S10-2.2.14 Thickness of Weathered Zone - Elevation