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13. ABSTRACT

The Second U.S.-Asia Conference on Engineering for Mitigating Natural Hazards Damage (EMNHD-2) was held at Yogyakarta, Indonesia, 22-26 June 1992. The primary purpose of this conference was to bring together American and Asian researchers, practitioners and public officials who are involved in seeking ways to mitigate damage caused by natural hazards. This conference, in support of the International Decade for Natural Disaster Reduction, was a sequel to the first EMNHD meeting which was held in Bangkok, Thailand, 14-18 December 1987. Participants were from the U.S.A., Indonesia, Singapore, Malaysia, Thailand, Hong Kong, China (Taipei), Japan, Republic of Korea, Philippines, Bangladesh, Nepal, and India. Papers were limited to: earthquakes; floods; ground failures; volcanoes; and extreme winds. The technical papers were bound as the *Proceedings of the Second U.S.-Asia Conference on Engineering for Mitigating Natural Hazards Damage*. A field trip to the Merapi Volcano Observatory Office and the Volcanic Sabo Technical Center (VSTC) was included in the conference program. A workshop followed the technical presentations, to delineate possible projects for mitigating damage from these five natural hazards. The *Final Report* contains the recommended projects and resolutions.

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EMNHD-2
Yogyakarta, 1992

Proceedings of the Second US-Asia Conference

on

ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE

Edited by

Arthur N.L. Chiu
Aspan S. Danusatmodjo

ia

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SECOND US-ASIA CONFERENCE ON ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
Yogyakarta, 1992

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PREFACE

The 1990's have been designated by the United Nations as the International Decade for Natural Disaster Reduction (IDNDR). In the support of the IDNDR, the 2nd US-Asia Conference on Engineering for Mitigating Natural Damage (EMNHD-2) was held 22-26 June 1992 in Yogyakarta, Indonesia. The first conference was in Bangkok, Thailand 14-18 December 1987. The main objectives of these conferences are to bring together researchers, practitioners and staff members of agencies and institutions from the U.S.A. and Asia to exchange information on methodologies for mitigating natural hazards damage as well as to propose potential collaborative research projects.

The EMNHD-2 conference technical program included a keynote paper, theme papers, regular papers, a field trip and group workshop sessions. Topic selected for this conference were limited to five natural hazards: volcanoes, earthquakes, extreme winds, floods and ground failures. This volume contains the papers received in time for publication prior to the conference. The efforts of the many authors in meeting the deadlines and their active participation in the conference are sincerely appreciated, and the valuable assistance of the EMNHD-2 Secretariat is acknowledged gratefully for the preparation of this publication. A final report to be published subsequent to the EMNHD-2 conference will contain the recommendations from the five workshop groups.

The EMNHD-2 conference was sponsored by the U.S. National Science Foundation, the USAID/Office of Foreign Disaster Assistance, the University of Hawaii at Manoa, and the Swa Bhatara Foundation.

We hope that this publication will be useful to the design professionals and researchers who are constantly seeking ways and means to mitigate natural hazards damage.

Arthur N.L. Chiu

Aspan S. Danuatmodjo

June 1992

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EARTHQUAKE HAZARD

SECOND US-ASIA CONFERENCE ON
ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
Yogyakarta, Indonesia
22-26 June 1992

ATTENUATIONS OF SEISMIC WAVES

Pisidhi Karasudhi
Professor, Asian Institute of Technology
G.P.O Box 2754, Bangkok 10501, Thailand

ABSTRACT: Far-field displacement components in vibration problems of multilayered isotropic elastic and viscoelastic half spaces, taken as the idealized models of the earth media, are presented in closed forms for three simple fundamental problems: a homogeneous half space, a homogeneous full space, and two different half spaces perfectly bonded together. Results for transient waves are also presented.

1. INTRODUCTION AND NOMENCLATURE

1.1 Fundamental Problems

In this paper, far-field displacement components in isotropic elastic and viscoelastic spaces are presented in closed forms. Three fundamental problems are considered: a homogeneous half space, a homogeneous full space, and two different half spaces perfectly bonded together. These fundamental problems are supposed to represent, respectively, the following parts of a multilayered half space: the surface, the interior of each and every layer, and each interface between every pair of adjacent layers of different properties. We shall use Cartesian coordinates (x, y, and z), and cylindrical coordinates (r, θ, and z). The Cartesian displacement components are denoted by u, v, and w, respectively; and the cylindrical displacement components are u_r, u_θ, and u_z ($\equiv w$), respectively. In the second fundamental problem, the homogeneous full space is treated as two half spaces bonded together.

1.2 Plane Problems

Plane problems are those of two-dimensional (2-D) spaces. The xy-plane is used as the plane of reference, the x-dimension is infinite, i.e. $-\infty < x < \infty$, while the y-dimension is constantly finite or infinite. For isotropic *in-plane* problems, w vanishes, while u and v are functions of x and y only, independent of z. In isotropic *antiplane* problems, u and v vanish, while $w(x,y)$ is the only non-vanishing displacement component. In the first fundamental problem, the x-axis is put on the surface of the homogeneous half plane; so, $0 \leq y < \infty$. In the second and third fundamental problems, the x-axis is at the interface where the two half planes are perfectly bonded together; so, $0 \leq y < \infty$ and $0 \geq y > -\infty$ for the underlying and overlying half planes, respectively. Unless specified otherwise, subscripts 2 and 1 are used to denote such respective half planes.

1.3 3-D Problems

In a three-dimensional (3-D) space, the x and y-dimensions are infinite, i.e. $0 \leq r < \infty$, while the z-dimension is constantly finite or infinite. For isotropic axisymmetric problems, u_θ vanishes, while u_r and u_z are functions of r and z only, independent of θ. In isotropic *pure torsion* problems, u_r and u_z vanish, while $u_\theta(r,z)$ is the only non-vanishing displacement component. In a general 3-D problem, there are three non-vanishing displacement components; $u_r(r,\theta,z)$, $u_\theta(r,\theta,z)$, and $u_z(r,\theta,z)$. In the first fundamental problem, the x and y-axes are put on the surface of the homogeneous half space; so $0 \leq z < \infty$. In the second and third fundamental problems, the x and y-axes are at the interface where the two half spaces are perfectly bonded together; so, $0 \leq z < \infty$ and $0 \geq z > -\infty$ for the underlying and overlying half spaces, respectively. Unless specified otherwise, subscripts 2 and 1 are used to denote such respective half spaces.

1.4 Loading and Problem Classification Numbers

In each problem, the load is of unit intensity and concentrated at the origin 0 of the reference coordinate systems. In Table 1, a loading classification number is assigned to each case. Later (in Tables 3 and 4), each problem will be referred to by three numerals separated by periods; the first numeral stands for the fundamental problem number, the second for the dimensions of the space, and the third for the loading classification number.

TABLE 1. LOADING CLASSIFICATION NUMBERS.

Fundamental	2-D: Loading			3-D: Loading		
Problems	Type	Direction	No.	Type	Direction	No.
	Force	y	1	Force	z	5
1	Force	x	2	Force	x	6
	Force	z	3	Torque	z	7
	Moment	y	4	-	-	-
2	Force	y	1	Force	z	5
and	Force	x	2	Force	x	6
3	Force	z	3	Torque	z	7
	Moment	y	4	-	-	-

2. HARMONIC VIBRATION

2.1 Wavenumbers and Types of Attenuation

Displacement components in harmonic vibration problems take the forms $F(x,y)\exp(i\omega t)$ and $F(r,\theta,z)\exp(i\omega t)$ for 2-D and 3-D spaces, respectively; where ω is the frequency, t = time, and $i = \sqrt{-1}$. The inverse integral transforms involved (Karasudhi, 1991a) are; for isotropic elastic planes,

$$\int_0^\infty f(\eta,\alpha_p,\alpha_s)\left(e^{-\alpha_p y}, e^{-\alpha_s y}\right)\cos\eta x\,d\eta, \quad \int_0^\infty f(\eta,\alpha_p,\alpha_s)\left(e^{-\alpha_p y}, e^{-\alpha_s y}\right)\sin\eta x\,d\eta \quad \text{(1a,b)}$$

and for isotropic elastic 3-D spaces,

$$\int_0^\infty f(\eta,\alpha_p,\alpha_s)\left(e^{-\alpha_p z}, e^{-\alpha_s z}\right)J_m(\eta r)\,d\eta \quad (2)$$

where J_m is a Bessel function of the first kind, and

$$\alpha_p(\eta) = +\sqrt{\eta^2-\eta_p^2}, \quad \alpha_s(\eta) = +\sqrt{\eta^2-\eta_s^2} \quad \text{(3a,b)}$$

in which $\eta_p = a\omega/c_p$, $\eta_s = a\omega/c_s$, $c_p = \sqrt{(\lambda+2\mu)/\rho}$, $c_s = \sqrt{\mu/\rho}$, a is a positive real constant of length dimension, η_p and η_s are dimensionless pressure and shear *wavenumbers* respectively, c_p and c_s are pressure and shear wave speeds respectively, λ and μ are Lamé's constants, and ρ is the mass density. Together, P-waves (P for pressure) and S-waves (S for shear) are called *body waves*. A *surface wave* exists, when f assumes the quotient form

$$f(\eta,\alpha_p,\alpha_s) = g(\eta)/F_R(\eta) \quad (4)$$

and if the root of F_R exists at $\eta = \eta_R$; $\eta_R > \eta_s > \eta_p \geq 0$. The existence of these wavenumbers for the far field, i.e. where x or r is large, in elastic fundamental problems

TABLE 2. WAVENUMBERS FOR FAR FIELD IN ELASTIC PROBLEMS.

Fundamental Problems			Wavenumbers Body	Wavenumbers Surface
1	In-plane	u, v	η_p, η_s	η_R
	Antiplane	w	η_s	None
	3-D	u_r, u_z	η_p, η_s	η_R
		u_θ	η_s	None
2	In-plane	u, v	η_p, η_s	None
	Antiplane	w	η_s	None
	3-D	u_r, u_z	η_p, η_s	None
		u_θ	η_s	None
3	In-plane	u, v	η_{p1}, η_{s1}, η_{p2}, η_{s2}	η_R or none
	Antiplane	w	η_{s1}, η_{s2}	None
	3-D	u_r, u_z	η_{p1}, η_{s1}, η_{p2}, η_{s2}	η_R or none
		u_θ	η_{s1}, η_{s2}	None

(Karasudhi, 1991a) is summarized in Table 2.

TABLE 3. BODY WAVE RADIATION FOR PLANE (2-D) PROBLEMS.

Problems	Displacements	P-waves	S-waves
1.2.1	u, v	$x^{-3/2}$	$x^{-3/2}$
1.2.2	u, v	$x^{-3/2}$	$x^{-3/2}$
1.2.3	w	-	$x^{-1/2}$
1.2.4	w	-	$x^{-1/2}$
2.2.1	u by $(1.2.1)^*$	$x^{-3/2}$	$x^{-3/2}$
	v	$x^{-3/2}$	$x^{-1/2}$
2.2.2	u	$x^{-1/2}$	$x^{-3/2}$
	v by $(1.2.2)^*$	$x^{-3/2}$	$x^{-3/2}$
2.2.3	w	-	$x^{-1/2}$
2.2.4	w	-	$x^{-1/2}$
3.2.1	u, v	$x^{-3/2}$	$x^{-3/2}$
3.2.2	u, v	$x^{-3/2}$	$x^{-3/2}$
3.2.3	w	-	$x^{-3/2}$
3.2.4	w	-	$x^{-3/2}$

$(\;)^*$ = substitute problem, since actual problem gives trivial component.

For isotropic viscoelastic solids, the Lamé's constants (in fact, functions of ω) are complex with both real and imaginary parts positive, leading to complex wavenumbers with positive real parts but negative imaginary parts, i.e.

$$\zeta_p = \eta_p - i\xi_p, \quad \zeta_s = \eta_s - i\xi_s, \quad \zeta_{sR} = \eta_R - i\xi_{sR} \tag{5a,b,c}$$

and equations (3a,b) become, respectively,

$$\alpha_p(\eta) = +\sqrt{\eta^2 - \zeta_p^2}, \quad \alpha_s(\eta) = +\sqrt{\eta^2 - \zeta_s^2} \tag{6a,b}$$

Karasudhi (1991b) obtained the Cauchy's principal value of each of the infinite integrals defined by equations (1) in outgoing and attenuating wave forms, for $y = 0$ and as x increases, i.e.

$$x^{-n}\exp(-i\zeta_p x), \quad x^{-n}\exp(-i\zeta_s x), \quad \exp(-i\zeta_{sR} x) \tag{7a,b,c}$$

and by equation (2), for $z = 0$ and as r increases, as

$$r^{-n}\exp(-i\zeta_p r), \quad r^{-n}\exp(-i\zeta_s r), \quad r^{-1/2}\exp(-i\zeta_R r) \tag{7d,e,f}$$

where n are positive constants and signify the *geometric attenuation* or *radiation*. While the surface waves of plane problems have no radiation, those of 3-D problems have in the order of $r^{-1/2}$. In elastic solids, there is no *material attenuation* since

TABLE 4. BODY WAVE RADIATION FOR 3-D PROBLEMS.

Problems	Displacements	P-waves	S-waves
1.3.5	u_r, u_z	r^{-2}	r^{-2}
1.3.6	u_r, u_z	r^{-2}	r^{-2}
	u_θ	-	r^{-2}
1.3.7	u_θ	-	r^{-1}
2.3.5	u_r by (1.3.5)*	r^{-2}	r^{-2}
	u_z	r^{-2}	r^{-1}
2.3.6	u_r	r^{-1}	r^{-2}
	u_z by (1.3.6)*	r^{-2}	r^{-2}
	u_θ	-	r^{-1}
2.3.7	u_θ	-	r^{-1}
3.3.5	u_r, u_z	r^{-2}	r^{-2}
3.3.6	u_r, u_z	r^{-2}	r^{-2}
	u_θ	-	r^{-2}
3.3.7	u_θ	-	r^{-2}

( )* = substitute problem, since actual problem gives trivial component.

$$\xi_p = \xi_s = \xi_R = 0 \qquad \text{(8a,b,c)}$$

Thus substituting the equations above into viscoelastic solutions yields readily the corresponding elastic solutions.

2.2 Radiation of Body Waves

Body wave radiation for 2-D and 3-D problems, classified in Section 1.4, are presented in Tables 3 and 4, respectively.

3. PROPAGATION OF TRANSIENT WAVES

To follow are speeds and material attenuation of *transient waves* in plane problems. Replacing x by r in the text below leads to the one for waves in 3-D spaces right away. The transient geometric attenuation (or radiation) can be assumed to be the same as in harmonic vibration (Tables 3 and 4).

3.1 P-Wave Propagation in x-Direction

The term $\exp[i(\omega t-\zeta_p x)] = \exp[i(\omega t-\eta_p x)-\xi_p x]$ in harmonic vibration indicates that the P-wave is outgoing and attenuates as x increases. The transient wave speed and the material attenuation factor are, respectively (Karasudhi, 1991a),

$$c_p(0^+) = \sqrt{[\lambda(0^+) + 2\mu(0^+)]/\rho}, \quad \Omega_p = [\lambda'(0^+) + 2\mu'(0^+)]/[\lambda(0^+) + 2\mu(0^+)] \qquad \text{(9a,b)}$$

where 0^+ stands for the initial time, i.e. $0 \leftarrow t$, and a superprime ($'$) denotes differentiation with respect to t. The material attenuation is in the form

$$\exp(\Omega_p t/2) = \exp\{\Omega_p x/[2c_p(0^+)]\} \qquad \text{(10)}$$

in which x is dimensional.

3.2 S-Wave Propagation in x-Direction

The term $\exp[i(\omega t-\zeta_s x)] = \exp[i(\omega t-\eta_s x)-\xi_s x]$ in harmonic vibration indicates that the S-wave is outgoing and attenuates as x increases. The transient wave speed and the material attenuation factor are, respectively,

$$c_s(0^+) = \sqrt{\mu(0^+)/\rho}, \quad \Omega_s = \mu'(0^+)/\mu(0^+) \qquad \text{(11a,b)}$$

3.3 Surface Wave Propagation in x-Direction

The term $\exp[i(\omega t-\zeta_R x)] = \exp[i(\omega t-\eta_R x)-\xi_R x]$ in harmonic vibration indicates that the surface wave is outgoing and attenuates as x increases. It should be considered logical to assume the analogy in which the transient wave speed and the attenuation factor are proportional to those in the S-wave propagation, with the proportionality factors η_s/η_R and ξ_R/ξ_s, respectively. More explicitly, the transient wave speed is

$$c_R(0^+) = c_s(0^+)\eta_s/\eta_R \tag{12a}$$

and the material attenuation is in the form

$$\exp(\Omega_R^{(s)}x) \tag{12b}$$

where

$$\Omega_R^{(s)} = \Omega_s\xi_R/[2\xi_s c_s(0^+)] \tag{13}$$

It is recommended that ζ_s which is closest to ζ_R should be employed in the present analogy.

4. UNDERLYING HALF PLANE OR SPACE

4.1 Underlying Half Plane

Results for large r inside the underlying half plane of a multilayered plane can be obtained easily by setting, in the second fundamental problems, the x'-axis of another coordinate system (x', y') to coincide with a certain r-direction. Since such results happen to be at $y' = 0$, they can be readily taken from Sections 2.1 and 2.2.

4.2 Underlying Half Space

Results for a large spherical radial coordinate R inside the underlying half space of a multilayered 3-D space can be obtained easily by setting, in the second fundamental problems, the x'-axis of another coordinate system (x', y', z') to coincide with a certain R-direction. Since such results happen to be at $z' = 0$, they can be readily taken from Sections 2.1 and 2.2.

5. CONCLUSIONS AND SUMMARY

This paper presents closed form far-field solutions to three fundamental problems of isotropic elastic and viscoelastic spaces undergoing harmonic vibration: a homogeneous half space, a homogeneous full space, and two different half spaces perfectly bonded together. Both two and three dimensional spaces are considered. Each solution is obtained as a linear combination of discrete surface and body waves. The speed, material attenuation, and geometric attenuation of each wave are given explicitly. The transient propagation of waves is derived from the harmonic vibration. Far-field displacement components inside the underlying half space of a multilayered space can be obtained by simple coordinate transformations.

The far-field displacement functions obtained in this study can serve as rational and efficient shape functions of infinite elements, into which the far field of a multilayered half space is discretized (Rajapakse and Karasudhi, 1986). In addition, such far-field functions can be utilized to improve the existing (empirical) models of propagation of seismic waves from epicenters, e.g. those by Brune (1970, 1971), and McGuire, Becker and Donovan (1984).

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DISASTER ESTIMATION CAUSED BY FAILURE OF CRITICAL FACILITIES DUE TO NATURAL HAZARD

Heki Shibata, Professor-Dr.
Department of Mechanical Eng'g. and Material Sci.
Faculty of Eng'g. Yokohama National University
156 Tokiwadai, Hodogaya Yokohama 240, JAPAN

1. Introduction

It is necessary for establish the counter measure against the seismic induced disaster to estimate how failures of facilities would be going on. Most of losses of human lives will come from

i) collapsing of buildings,

ii) spreaded fire after the event and

iii)failure of critical facilities.

This paper deals with the simulation method to estimate the disasters of type ii) and iii), because we don't afraid that the first type of the disaster will become dominant in Japan. The author has been working for the earthquake-resistant design of nuclear power plants and petro-chemical industries for long years. If they will fail, the radio active material or poisonous gas would defuse to the outside of plants, and it might kill thousands and thousands. The techniques for estimating such disasters are almost the same as that for spreaded extended fire. The author will discuss the problems to apply simulation techniques for estimating both types of in the following sections. One of the points is how to estimate it in probabilistic way or deterministic way. Behind this subject, there is the essential point for estimating the disaster induced by natural hazard, by not only an earthquake. In relation to this subject, establishing the scenario of the consequence to the disaster is also the key to obtain the reliable result of the estimation. The author will discuss these subjects in the following sections.

2. Critical Facilities and Natural Hazard

One of serious disasters induced by natural hazards such as an earthquake, tornadoes may be a nuclear accident. Fortunately we have never experienced this. However, the engineers and scientists, like the author, have been working for this subject since 1960. We obtained very fruitful results in engineering seismology, civil engineering, mechanical engineering, system engineering as well as in other fields as a research, but it is more significant that we have never had any accident of nuclear power plants induced by a natural hazard. Of course, no nuclear power plant has experienced any serious natural hazard ever, but still it is significant.

On the other hand, Tokyo Metropolitan area experienced that 140,000 lives were lost by spreaded, extended fires after the Kwanto earthquake in 1923. It was almost 3% of the total population of the area. The Government of Tokyo Metropolitan Area had been working for the estimation of such loses after the similar shock which is expected to occur in the future. Most of works, which had been done by a committee for natural disaster prevention in Tokyo Metropolitan Government last twenty five years, are shown in Fig.1.

At first, the distribution of peak ground acceleration (PGA) is estimated based on an assumed epi-center and a magnitude of the event. Then distribution of fire-starting points is estimated by the distributions of wooden buildings and PGA, and the simulation of extended fires is made under a certain meteorological condition, such as winter, strong north-west wind, and dry weather. Finally, the loss of lives caused by such extended fires can be estimated. The effect of dangerous materials was evaluated, and the methodology will be discussed in Section 5 briefly.

The approach of the evaluation was described in one [1] the previous papers, and it was made in a stochastic way. Even though Seismic Probabilistic Safety Assessment was developed and was made recently, the most of seismic design and assessment of equipment and piping systems of nuclear power plants are done in the deterministic way.

For the evaluation on seismic resistant capacity of an individual facility like a spherical tank of LPG can be made in both ways. But the details of structure is not available in general, therefore, we employed the probabilistic approach. There is a big discussion which is better a deterministic approach or a probabilistic approach for the disaster or hazard evaluation. The policy of Tokyo Metropolitan Government was that the assumption had to be made as the severest environmental and boundary condition, that is, a deterministic one. During the work, we need various probabilistiC assumptions; such as where the fires start, how flying sparks are distributed, and so on. It is difficult to make a simulation completely deterministic.

Contribution of critical facilities to the disaster is listed as follows:

i) supplying poisonous gas or radio active material,

ii) starting fire to its vicinity,

iii)giving blasting pressure wave to its surrounding structures,

iv)giving heat-radiation to its surrounding structures and inhabitants by fire ball or flame,

v) supporting spreaded fire in surrounding area by its flammable material.

The evaluation on a sequence of occurring such disasters, a probability and its consequence can be made in various way, however it is important to overview the total scope of such disasters, and to establish the scenario. These works can be done only based on the experience or at least on the careful study of reports on previous disasters. Sometimes, the people experienced a particular type of disasters, it is a tendency too much emphasizing to it. To avoid this, we should have the scientist's eyes to understand the phenomena themselves.

The author described various examples of the mode of failures in his previous papers [2) 3)] since 4 World Conf. on Earthq. Eng'g., therefore, only he wants to emphasize that the knowledge on the previous disasters is very significant for this work.

3. Role of Hazard Evaluation

It is clear that the necessity of hazard evaluation work to prevent the disaster induced by a natural hazard. It is not clear for the author which term "hazard evaluation" or "deserter evaluation" is adequate, however, the purpose is clear. To prevent such disasters or to establish the counter measures, this work is extremely important, and the governmental organization of a particular area must work under the support of scientists and engineers, and also city-planners, economists, and other social scientists. Insurance engineers have been working on this subject, but their policy is not so clear for our engineers, because it is a business and is governed by their experience or thumbs low.

Most of reports referred to the total death or the total property loss by a particular event. These approaches are a deterministic method as mentioned in the previous section. As the result of future event, they give only approximate figure, and it is an index to think about the size of the disaster, that is, we don't know whether it might be twice or a half as a result.

City of Kawasaki recently developed a computer and network system to avoid the disaster in their city. Information Network System for Actions of Counter Measure to Seismic Disaster is a system, which was designed by Ohta, as follows. Its function is to collect the data of peak ground acceleration of several observation points in the City area, and figuring out the distribution of seismic intensity immediately after the event. And the main computer calculates and displays distribution of occurring some type of damages, such as failure rate of wooden houses, life-line systems and so on. Then, it indicates the estimated degree of the disaster of the whole city, and what degree of the emergency action for fire fighting and rescue. The population of City of Kawasaki is approximately one million, and it locates in the south of Tokyo, and a part of Metropolitan area on the Tokyo Bay. The shape is the longer in East and West, and the eastern part of the City is heavily industrized on the Tokyo Bay, and the western part is hilly residential area, and there is a commercial center in between. Therefore, it is rather difficult to grasp the total figure how the disaster is developing at the time. For the prompt action of the City, such informations are necessary, then before collecting the data which are existing, the estimated damages obtained from the distribution of seismic intensity are useful of the decision of their action.

To develop such estimations, we need several simulation program packages in the system.

4. Event Flow and Scenario

To establish the scenario of disasters occurring at the event, we need a wide knowledge from engineering seismology to system engineering. As shown in Section 2, we need the flow of events after the shock. The author tried to discuss on the "Event Flow Chart" in a previous paper.[4] At that time, the author intended to use the concept only for the seismic design of equipment in critical facilities such as those in a petro- chemical industry. But it is clear that the concept can be expanded to the wider figure. If we experienced a new event, and observed new types of failure modes, we can add the new flows to the previous event flow. Even it is difficult to give a qualitative flow, such as a transit probability, the topological relation may be useful to establish the scenario. However, it is a problem, if the subjective judgement is too strong to design the scenario as mentioned before.

In late 1960's, Tokyo Metropolitan Government started from only the estimation of total loss of lives by spreaded fire, but soon after the San Fernando Earthquake-1971, the scenarios was expanded to other types of deaths such as by flood, by poisonous gas, by land slide and so on. As The second phase, the reevaluation was started in 1985, and the items were decided as Table 1, and completed in the last summer, 1991. The conclusion was complicated and it is not so understandable to the public as well as the office of the Metropolitan Government. A single figure of the total loss of lives is easy to be understood, but the more precise work always needs various conditions, and it becomes difficult to understand it and also to establish the counter measure.

5. Flow of Analysis

The author tries to discuss two typical examples of simulation for disaster evaluations in this Section.

5.1Spreaded Fire simulation

This work [5] was done by Fujita, as his doctors thesis in 1974. His idea is very unique, and it became to be possible to simulate the way of spreading the fire after a seismic event. He started from the famous relation of fire spreading to various parameters including wind velocity and direction, so called Hamada's equations. This relation was established by Late-Professor Hamada in 1951 from the results obtained through his surveys on several big fires including the fire of Tokyo at Kwanto earthquake-1923. From these deterministic equations on fire spreading velocity, Fujita established the concept of "element fire " and simulate by the algorithm to solve the differential equation. However, it is opposite way to solve the differential equation. There is no differential equation and only exists the concept of "element fire" (in Fig.2) The second innovative point done by Fujita was to compute only the boundary of spreaded fire and not to compute the amount of material for burning as previous researches done by others. This idea could reduced the great amount of memory space in the computer. He used HITAC 8800/8700, the Computer Center, University of Tokyo, the largest computer in Japan at that time. He could simulate the fire of the area 3km x 2km less than for the lOOth of the real time. By his innovative works, we can simulate the fire of a certain area like lOkm x 5km by a work station, and display on CRT in the office now. Such a system may be built in the system like the Kawasaki's system (see Section 3) with some modification, because the sizes of meshes of both systems are totally different, that is, 12.5m x 12.5m against 500m x 500m.

5.2Seismic Risk Assessment of Storage Vessels

This work [1] was done for the second phase of the hazard evaluation in Tokyo by the authors' group. The flow of the assessment is shown in Fig.3, and it starts based on three data bases and one fragility curve on each item. In this case, for the spherical tank, the fragility curve, which they called "Damage Curve" in their paper, was available on real systems as shown in Fig.4. For the flat bottom cylindrical tank, that is, an ordinary cylindrical tank for an oil storage, their failure modes were studied based on the experience of damages at the Niigata earthquake-1964[2] and they considered that the strength of bottom corner was significant to spill the content. Then we obtained the relation of the tank capacity to critical peak ground acceleration based on an average current design scheme in Japan. It should be noted that the material of cylindrical wall and bottom plate is different according to their capacity. Therefore, the curve in Fig.5 is discontinuous.

The design scheme might change in the year of construction, then such curves for actual system become double or wider band, and for the simulation the probabilistic concept should be introduced. Anyway, in the paper, the author considers, the curve is the simple function of the capacity, but it is discontinuous at approximately 40,OOOkl as shown in Fig.5. This means the material is switched from mild steel to high-tension steel, because of the limitation of thickness of the wall coming from the technical reason of welding.

The cylindrical tank has three types of the mode of failure. The mode above-mentioned is the failure caused by the uplifting due to the horizontal acceleration. The other two modes are sloshing and the liquefaction of the foundation soil. The sloshing phenomenon may cause spill of the oil from the top of the storage either by large amplitude of response of floating roof or by breakage of the vessel at the top corner edge. Even though some examples of tank fire caused by splashing the oil such as the Niigata earthquake-1964 and the Great Alaska earthquake-1964, the amount of oil spill is limited compare to the breakage of the corner ring at the bottom, and we can neglect its effect for this evaluation

Failure of the foundation soil may cause also the breakage of the corner ring at the bottom, and it must be evaluated. The possibility of the failure of the foundation soil is the function of the seismic intensity as well as the soil condition, compaction method of soil and detailed design of its foundation ring. The relation of failure probability to the seismic intensity has more uncertainty compare to the failure by its uplifting. The evaluation method is almost the same, except the degree of the uncertainty mentioned above plus soil data which have been not known for assessment usually.

6. Results of Assessment and Countermeasure

Results of such assessment should be reflected to the future counter measure. Those results are usually probabilistic ones as frequently mentioned, however, the counter measure is deterministic. Therefore, the operation of measure must be flexible, and planned against some extreme cases. Again the scenario becomes important. In the case of Metropolitan

Tokyo, the severest case for spreading fire in winter has been employed for the assessment, because almost of all details of counter measures in Tokyo are how against fire and how residents would escape from fire. However, for the evaluation of loss of human lives by poisonous gases, weather often observed in the rainy season, June and early July would bring the severest results by inversion of atomospher.

It should be noticed that the evaluation of the potential maximum loses is different from the assessment to establish the well-balanced counter measure. We must study the scenarios which we should expect in future conditions.

7. Overview of Papers Presented in This Symposium from Asian District

The author has been requested to overview the thirteen abstracts from Asian District in relation to his presentation. It is difficult to find the common.subject between these papers and the author's in general. According to the author's methodology described in Section,3, he tries to look through these papers. "Development of an Assessment Model for Earthquake Fatalities and Discussion for its Practical Application" by Shiono [042/E10] is one of the papers near to the author's one. So the some problem may be considered. The model depends on the scenario how to cause the loss of human lives.

To define the seismic intensity of the local area which the evaluator is interested in, seismic zoning and other engineering seismology type studies are important. Those papers; "Seismic Zoning" by Srivastava and Basu [018/E061, "Attenuations of Seismic Waves" by Karasudhi [012/E041, "An Intsive Digital Strong Motion Accelerograph Program in Taiwan" by Yeh and Shin [008/E021, "Earthquake Hazard Assessment in the Himalayan Front Arc of India" by Gupta [028/E08] and "Probabilistic Seismic Hazard Estimates in North Sulawesi Province, Indonesia" by Thenhaus and thers [064/E15] are mentioned. As the author often mentioned the study on attenuation factor in very important, but it is very often observed that the researcher for the hazard, disaster evaluation becomes the specialist of the attenuation factor. The systematic program like Taiwan is very significant. The author has been working for it for more than twenty years, and also the colleagues in the Institute of Industrial Science, University of Tokyo has been operating the facilities including the engineering array in the China Field Station since 1981 "Seismicity, and its Relation to the Volcanic's Activity in Indonesia" by Ibrahim and Ahmad [068/E18] may be necessary to be studied, however, the direct damage by a volcano is usually more serious to its vicinity area than the volcanic induced earthquakes in Japan. And the relation mentioned by Ibrahim and Ahmad have been discussed, but doesn't reach to any conclusion. "Mitigating Seismic

Hazard in Bangladesh" by Choudhury [050/E13] is difficult for the author to understand, because of too short abstract, but it should be mentioned that it is important to define the zoning in relation to the earthquake resistant designs of a particular area. "Earthquake Hazard Assessment of Earthquake Resistant Design" by Agrawal [009/E03] treats also this subject. The estimation work of possible damages is significant to establish the counter measure, the author discussed on such a subject [6]. "Possible Building Damage in a Hypothelical 8.4 Magnitude Earthquake in North Bihar Region of India" by Arya [027/E07] treats more serious condition caused by a very huge shock which would occur in 1992, because of the building construction condition of the region and high population. "Seismic Response of a Base-Isolated Building in China" by Pan and Kelly [044/E17] indicates that the development of a base-isolation system for simpler and low-cost buildings is very urgent, besides the facts they mentioned in their papers. "Systematic parameter Estimation for Hysteretic Systems" by Loh [007/E01] is necessary to estimate the fragility curve of a structure, and many studies have been done by many researchers and organizations. The author expects a new idea to estimate more practical method in this paper.

8. Acknowledgment

Through the committee works with the Government of Metropolitan Tokyo, City of Kawasaki and the Agency of Science and Technology, the author experienced many difficulties and had doubts for the effectiveness on simulation works of disaster development induced by a seismic event. He developed his doubt to a series of lectures at the University of Tokyo and Yokohama National University. This paper is a key portion of these lectures. He greatly appreciate the guidances of the committee members, especially Professors Wadachi, Kanai and Fujii related to the committee works above mentioned.

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Table 1

E05-10

List of Sub-committees for Disaster Evaluation in Tokyo Metropolitan Government

1) Engineering Seismology S.C.
2) Fire Estimation S.C.
3) Critical Facility S.C.
4) Life-line Facility S.C.
5) Social Impact Evaluation S.C.
6) Loss Evaluation on Human Lives S.C.

The titles of each sub-committees are not exactly translated from Japanese.

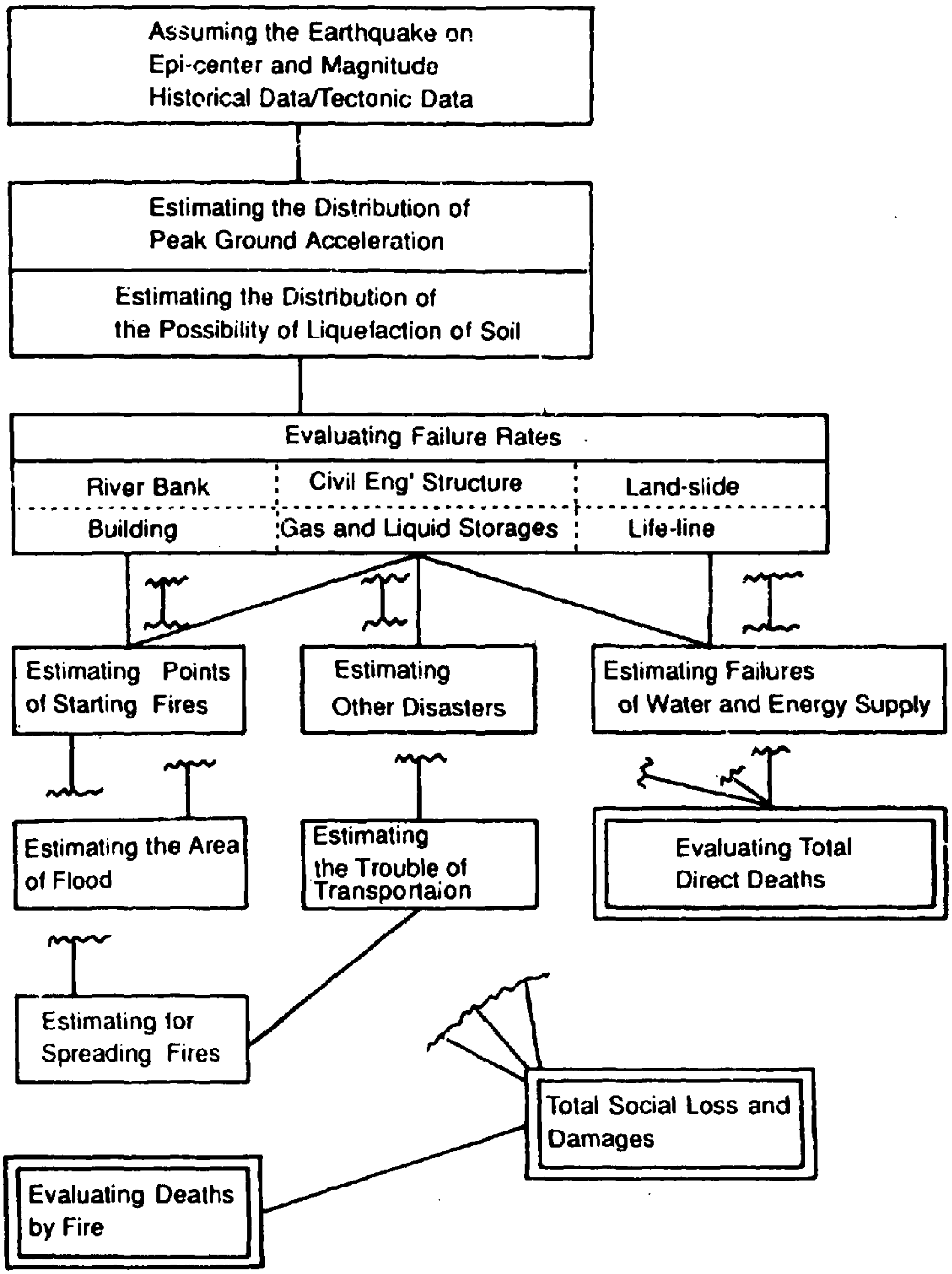


Fig. 1 Flow of Evaluation on Seismic Induced Disaster in Tokyo

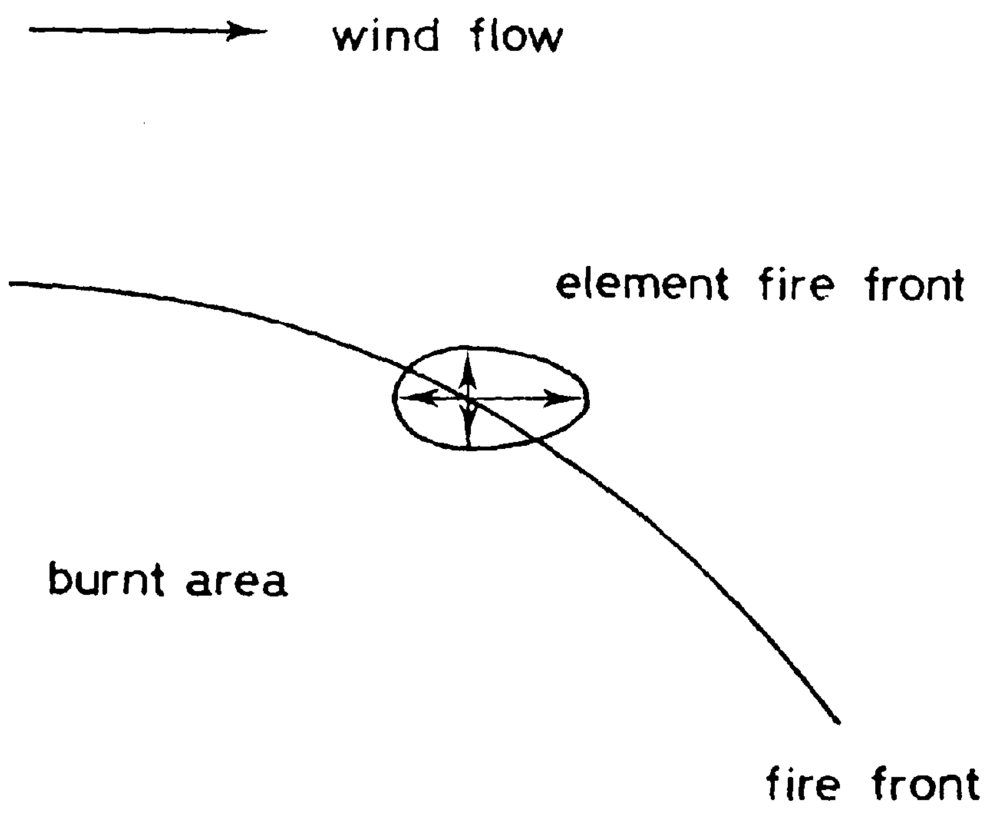


Fig. 2 Concept of Element Fire and Fire Front

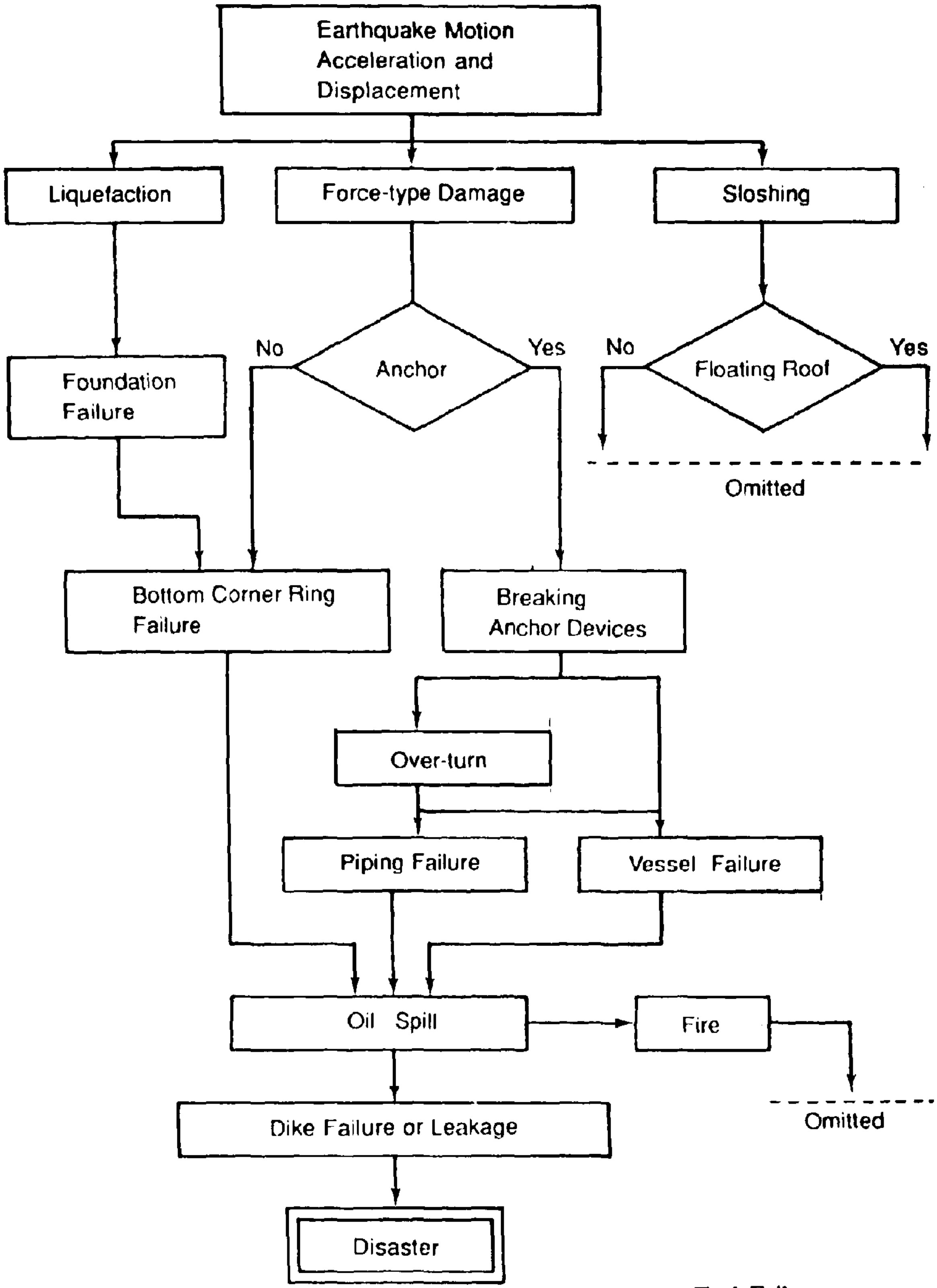


Fig. 3 Flow of Evaluation on Cylindrical Oil Storage Tank Failure

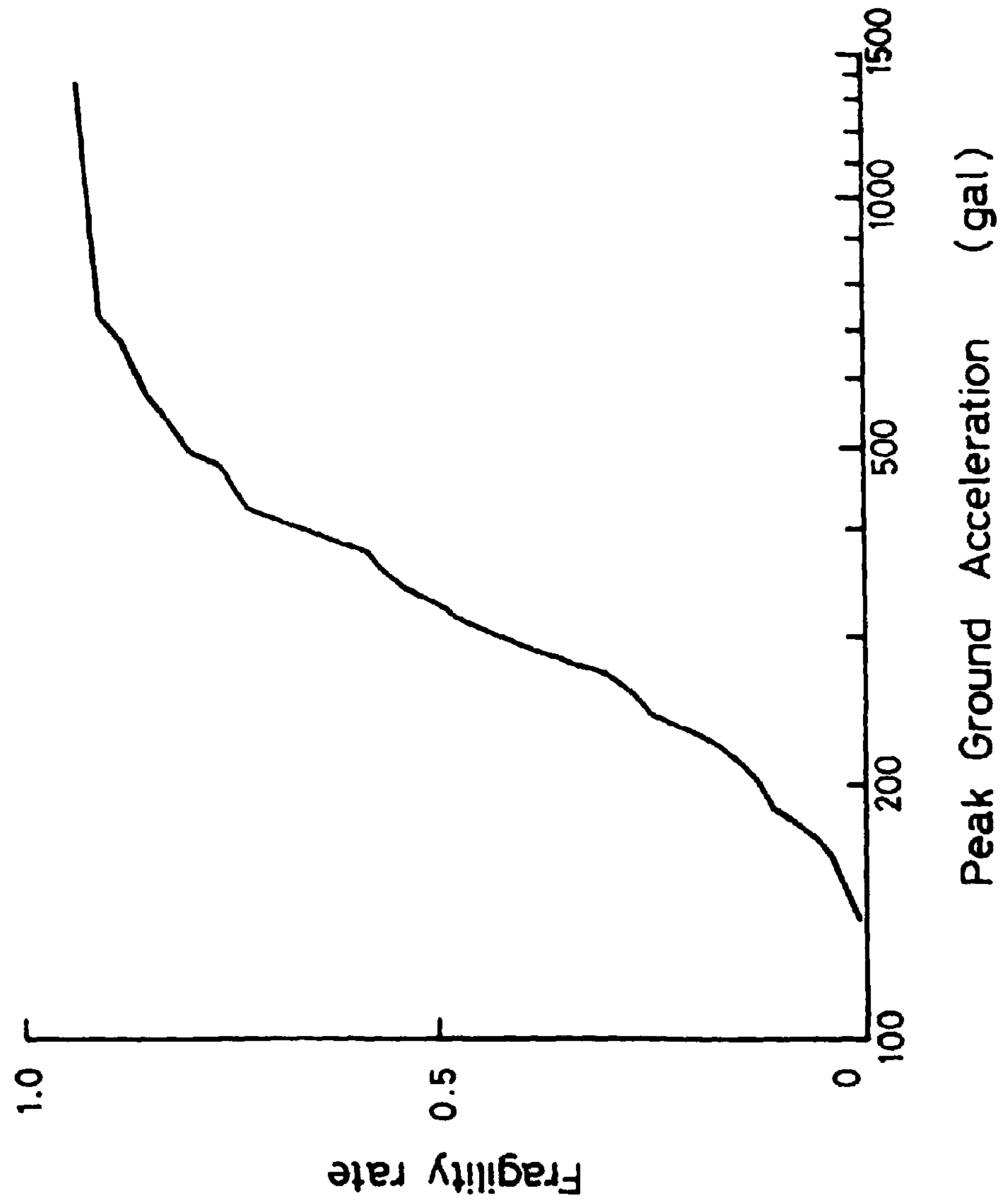
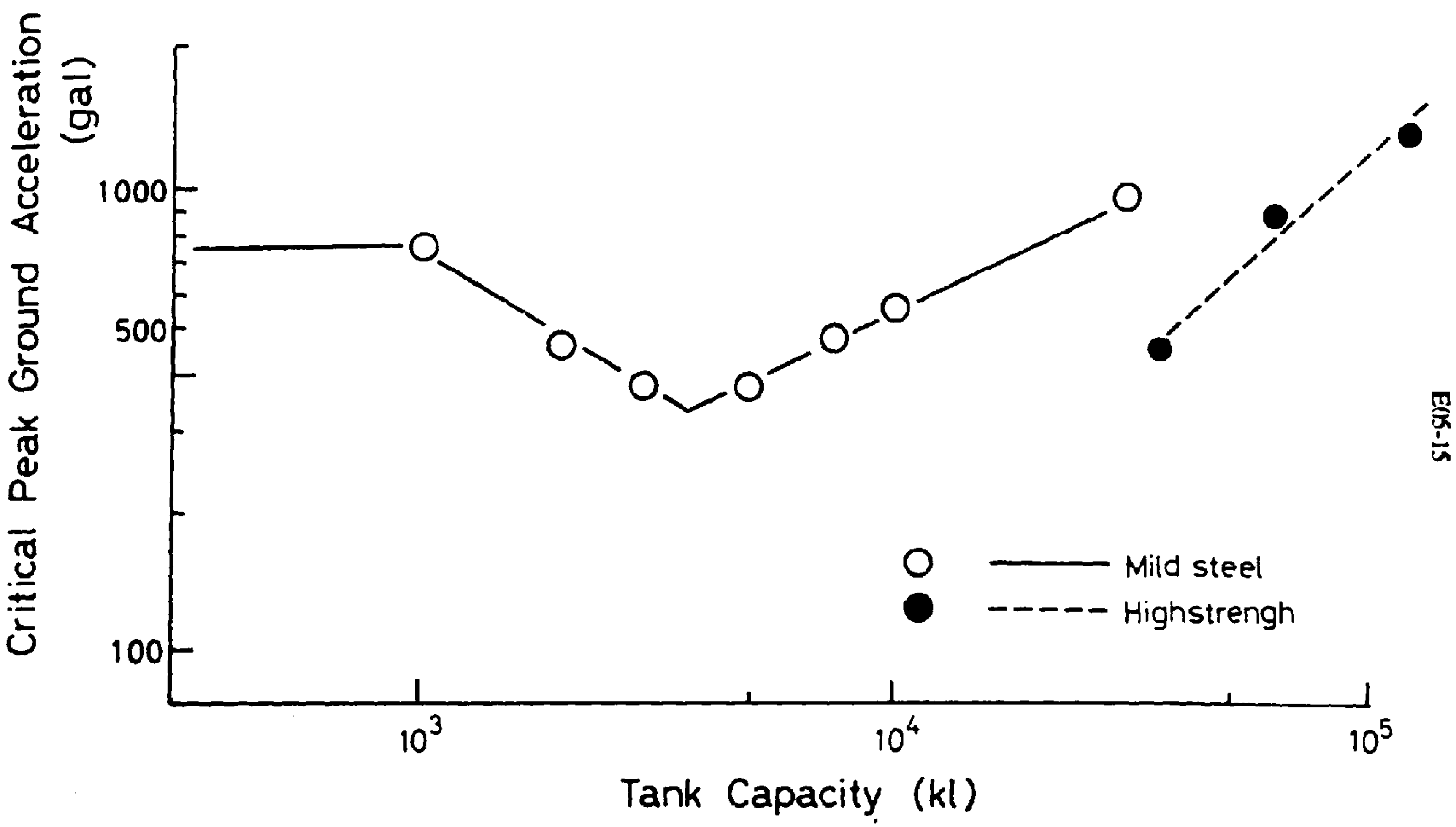


Fig. 4 Fragility Curve of Spherical LPG Tank



Critical Peak Ground Acceleration (gal)
1000
500
100
10^3
10^4
10^5
Tank Capacity (kl)
Mild steel
Highstrengh
E05-15

SECOND US-ASIA CONFERENCE ON
ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
Yogyakarta, Indonesia
22-26 June 1992

GIS-A CONVENIENT TOOL FOR NATURAL HAZARD STUDIES AND ANALYSIS

Dr. Michael P. Gaus and Seong H. Kim
Dept. of Civil Engineering
State University of New York at Buffalo
Buffalo, NY 14260

ABSTRACT GIS systems and data formatted for use in such systems are becoming readily available at a relatively modest cost due to the wide applicability of these systems. This provides a new tool and opportunity for application of this tool as a part of a macro engineering approach for natural hazard problems. A brief description of some of the features of GIS systems and data available are provided. Two examples, one in a planning stage and the other almost developed to an operational level are described. The first example concerns the opportunities for display and manipulation of wind data and the other example concerns a GIS-based regional risk approach for bridges subjected to earthquakes.

INTRODUCTION

Many new scientific tools are becoming available as a result of high-tech investments in many fields. These tools provide new capabilities for sensing, data collection, computation, graphical display and manipulation,analysis and decision making and monitoring for natural hazard problems. Many of these tools are computer based and are becoming more powerful and practical as the costs of computer systems are reduced and as software is developed to take advantage of new capabilities. One of the rapidly developing areas is that of Geographic Information Systems (GIS). GIS systems have been evolving since around 1974 and these systems are finding wide use in numerous areas ranging from land use analysis, urban planning, hazard analysis, parcel, streets and utility documentation and vehicle routing to political boundary determination of location of fast food restaurants. In some cases the graphical and database manipulation capabilities of a GIS system can provide a complete analysis for a problem. For other cases the GIS system can serve as an integrating platform which forms one module in a macro system in which additional modules are designed to carry out analysis and modeling steps which are needed for the problem at hand.

This paper is concerned with ways in which readily available GIS systems and data could greatly improve our ability to study, plan for and better understand natural hazard problems. After briefly reviewing some of the features offered by GIS systems and related activities which could be of value in mitigating the impacts of natural disasters a couple of specific examples will be described.

FEATURES OF GIS SYSTEMS

GIS systems and the companion CAD systems, which are now a common feature in engineering, are a form of computer graphics utilizing many common concepts. Both types of system provide a means of representing spatial data and utilize concepts such as placing different types of features on layers which can be displayed separately or combined in whatever combination is desired. Topological information can be stored and displayed in raster or vector formats and data or attribute information can be stored in separate files which are linked to layers and points.

A significant difference between the two types of systems is in the scales with which they work. CAD applications generally involve buildings or other applications in which the dimensions are such that the representation on a flat surface does not generate any special problems. GIS systems on the other hand may consider relatively large distances or areas on a curved surface, the surface of the earth, and the mapping of the related information to a flat surface cannot be done without introducing some type of distortion of the information. This type of problem is given special consideration in most GIS systems. The trend seems to be to utilize vector-based systems in which geographic features are represented as a series of nodes connected by straight or curved lines which in turn enclose a series of polygonal areas. Each polygonal area is identified and can have various attributes such as soil type, land vs. water and so forth linked with the polygonal area. This description is encoded in data tables in separate or linked form. Due to this complex and possibly detailed data description the amount of information which is stored for graphically representing an area can become quite large. The greater the resolution and detail which it is decided to include, the larger the data base becomes. Most prominent GIS programs, such as the ARC/INFO program can now also process raster-based data and can display such information in conjunction with vector information as well as carry out a certain amount of conversion from raster to vector form.

The detailed node-line-polygon description of a geographic area also generates some special problems if the area is originally presented as a series of smaller areas which are to be joined to form a larger area. In this case special programming is included and special care must be taken to be certain that the information at the edges of each area will match the corresponding information on adjacent areas. One consequence of including a high-level of resolution or detail is the amount of time taken to "look-up" and plot this information on a computer screen.

GIS DATA

Due to the numerous applications in which GIS systems have been found to be of value, there has been a rapidly expanding effort to place information in a form which is suitable for GIS analysis. This is a very fortuitous situation as the collection of data and the processing of the data in a form suitable for use in natural hazard problems can be a very expensive and time consuming proposition.

A number of decisions must be made in relation to GIS-based wind data. One major problem is the size of the data files which are developed. The size is related to the level of resolution and amount of detail which it is desired to include in the data base. The finer the level of resolution and the more detailed information which is included, the larger the data base becomes. Of course it is always possible to break the data base into segments so that instead of displaying an entire country at one time, the display might consist of a region, a state, a county, a city or other suitable area. Aside from the amount of file storage required for a data base is the problem of retrieving this information and displaying it in some specified manner. An excessive amount of data can result in either a very long write time to display and manipulate graphical screen information or the need for high-level equipment to quickly process the information. Another consideration is the capability to develop stand-alone versions of GIS applications programs for the average user who would not be interested in investing in a full-blown GIS software system.

Some examples of GIS databases which are readily available in the U.S.A. are the TIGER files, USGS digitized map files and state, county and city base maps which are being developed in many parts of the country. The TIGER files were developed by the U.S. Census Bureau and the U.S. Geological Survey to provide an operational framework for conducting the 1990 census. These files cover the entire United States and are constructed on the pattern of the 1:24,000 U.S.G.S. sectional maps which have been totally converted into a digital format. Additional detail was added in the form of street & place names and other information which was needed to organize the census operation. TIGER digital files are available for purchase directly from the Census Bureau at modest cost but recently a number of firms have taken the TIGER files and added various enhancements and improvements. These files are for sale at a very reasonable cost and in several digital formats which are compatible with both leading GIS software systems and CAD systems such as AutoCad. The files can be purchased with the basic layers developed by the Census Bureau and with additional layers which have been generated and added by other groups and can provide topography at various resolutions, zip codes, political boundaries, population densities and many other features. In addition various satellite imagery, such as SPOT data, can be digitized for direct use or included with TIGER and other data bases. The horizontal resolution provided by TIGER data is on the order of 75 feet in urban areas and 200 feet in rural areas. Figures 2 and 3 show some typical TIGER data plotted at original and a magnified scale. Other types of information are becoming available from a variety of sources covering basic geography, topography, soil characteristics, hydrology, coastal characteristics and other features required for natural hazard analysis, planning, design and response.

Thus a new tool has become available for natural hazard studies and because of the wide applicability of the GIS systems, the cost of the most expensive item, the data bases, have been spread over a number of application areas. This provides a large amount of basic information which can be used as a starting point for natural hazard applications.

SOME EXAMPLES OF GIS APPLICATIONS IN NATURAL HAZARDS

In order to illustrate some of the possible applications of GIS systems in natural hazard planning and mitigation, a couple of examples will be described. These include an example which is in the planning stage and which has not yet been operationally implemented and an example of regional risk analysis which has been under development for some time and is approaching an operational status.

GIS as a Platform for Design Wind Data

Design wind data, at the present time, is generally presented in map form and usually consists of a series of contours which have been drawn with the aid of known data points along with a large input of judgement. A user locates the position he is interested in and interpolates between contours to select a design value. In the U.S. there are many problems with this procedure. The present ANSI-ASCE wind design map is constructed on the basis of relatively few points (On the order of an average of 2 points per state). No specific account is made for either regional or local topographic effects, this is left up to the user, or for wind directionality effects. It does not take much study of a topographic map of the U.S. to conclude that there are significant variations of topography over even relatively "flat" states which are not well accounted for. Clearly the present procedure has some real deficiencies and these deficiencies are magnified due to the squaring effect of wind velocities when used for design. The lack of additional data points and the level of effort needed to consider statewide or local topographic effects has discouraged a more detailed approach toward documenting even maximum wind design velocities much less other wind parameters which are needed or could be useful for studying other than extreme wind effects.

GIS systems provide a new tool which may provide a mechanism to improve the collection, characterization and manipulation of wind data. Much of the data needed to carry out this process is being generated through other GIS applications. Digital-based maps are being generated at a large variety of scales for different purposes and these data will be available for application to wind problems. The challenge is how to effectively utilize this new tool and source of information to improve our ability to plan and design for wind effects.

At the most elementary level it would be possible to simply transfer the present extreme wind map, or maps, to a GIS format by superimposing the present contours to an outline map of the U.S. Depending upon the data base utilized for the outline map it would then be possible for a user to enter a query in one of the common formats which are coming into wide use in GIS applications. This could be latitude-longitude, city, county or other geographic name, postal zip code, street address and many other formats. The GIS system would be programmed to locate and display the point selected and with some relatively simple programming could interpolate between contours of extreme wind velocity. This would be an automated version of the manual system now used.

A more useful application of a GIS system would be to include topographic information which might be important in wind effects. This could be at the level of helping a user to decide on roughness categories to be used in design or could start to include modifications of the rather crude contour information to take into account some of the larger influences of topography.

The capability of a GIS system with its' built-in data processing capabilities also presents a mechanism to improve the characterization of wind data and wind impacts. The amount of long-term data collected by established sources is fixed and limited. However, there is a large amount of data which has been and is collected from a variety of sources. These include data collected by local communities, by the military, by governmental organizations, by industry and by many other sources. The collection and processing of this type of data was previously a formidable problem but the GIS framework may make the use of this type of data much more feasible. A specific effort is needed to determine how this data could be utilized to improve wind characterization. Some may argue that not all of these data are reliable, but one feature of a GIS approach is that the GIS system can be utilized as an object oriented module in a macro system which includes other modules for processing of non-standard wind data and other desired analysis operations. The GIS system then can be used as an integrating platform which utilizes the internal and external modules to achieve the desired results and provides an interactive interface and display capability. With regard to extreme wind data it is perfectly feasible to display information based on "reliable" data and then to display in a different color or format similar information which is built on a wider base of data. If the data density becomes high enough, anomalies in locally collected data may show up. Research is needed to better develop the basis of this approach, to develop improved predictive relations to be used with shorter term and data of a fuzzier nature, to recommend data formats for efficient use of these data and to establish how to use the GIS tools which have become available to us with newer databases which are being developed and collected.

GIS-Based Regional Risk Analysis of Bridges Subjected to Earthquakes

In order to illustrate the procedures involved in utilizing GIS as an integrating tool in regional risk assessment of lifelines subjected to natural hazards, an example will be used involving one county, Erie County, New York and one type of lifeline element, bridges. The analysis is carried out using information currently available in suitable form on a wide geographic basis. A modular approach is utilized with an open system approach so that as additional data becomes available and as improved damage assessment models are developed, source data refined, soil characteristics better identified, attenuation models refined and other information made available, this new information can be added to or substituted for existing portions of the assessment program. In this way the system can be readily improved in the future.

General Analytical Approach The overall approach developed for this study involves three major interactive components:

(a) The use of GIS to provide the interface to display geographic data, to manipulate information stored in relational data bases, to calculate topological parameters such as distance to a selected source, to sequentially call subprograms which evaluate vulnerability conditions for each bridge in the selected analysis region and to display the result of a query for the effect of a selected source and magnitude event.

(b) A risk model for bridges which can statistically predict the expected level of damage due to a particular intensity of ground motion at the bridge site.

(c) A ground motion attenuation model to predict the intensity of ground motion at a particular bridge site due to energy release at a selected potential source.

These interactive components are supported by data files which encode characteristics such as potential earthquake sources and magnitudes, and bridge characteristics which are important for failure analysis. As is usual in a GIS system these files can be generated externally but in a format compatible with the GIS system.

GIS Data Base A commercially enhanced TIGER data file for Erie County, New York was utilized which was assembled from thirty one 7.5 Minute Quad maps which are available for most urban areas of the U.S. Although spatial resolution of the Tiger files only provides the location of a bridge within 75 feet, it was felt that this was quite adequate for this study, particularly in view of the uncertainties inherent in the earthquake source data and the attenuation models available. Figure 2 shows a portion of Erie County at the 1:24,000 scale of a 7.5 Minute Quad map. This provides an overall view of an area and it is possible to identify major features of the transportation road network and other major features. A great deal of detail is embedded in this map, however, the detail cannot be distinguished at this scale. The GIS system provides a capability to zoom-in or window on a section of map. Figure 3 shows a windowed section of the map in Figure 2. As can be seen in this Figure, details such as street names have now become visible on the enlarged section of the map.

Damage and Failure Assessment Model A general screening model for GIS-Based risk analysis of bridges in regional environments was developed for this program. The procedure followed was to first collect as much data as could be readily found in the published literature regarding bridge failures and damage from earthquakes. These data were then reviewed to determine if sufficient detail was included in descriptions to permit an identification of the particular elements of the bridge system which resulted in failure and if sufficient information was provided to arrive at a fairly reliable estimate of ground intensity at the bridge site. Relatively complete data were found for 74 bridges which were damaged or failed during an earthquake. This data was examined to establish a suitable classification scheme which could encompass the major variables related to bridge performance, damage and failure. Examples of some of the features considered are bridge type, foundation conditions, type of component which were involved in damage or failure and characteristics of loading. The latter item is related to potential earthquake source mechanisms and earthquake ground motion attenuation which is discussed in a later section of this paper.

A major problem in extracting damage and failure information from published sources was the lack of any standard for reporting such data. In order to attempt to establish a reasonably consistent procedure for data evaluation, a series of levels of classification were established. On an overall or macro level the parameters can be grouped into loading environment or site intensity due to a given earthquake, degree of damage, soil conditions, foundation type and structural parameters such as pier details, materials used and details such as bearing type. Clearly bridge failures or damage could be initiated by liquefaction or surface faulting, however, in order to reduce the number of parameters to be considered in this study, these types of failure conditions were deferred for future study and primary concentration was placed on damage or failure due to ground shaking rather than ground failure. The 74 bridges used in this study excluded these types of ground failure.

A series of general (or perhaps fuzzy) categories were selected for the initial work on this problem. These categories are rather broad as the data available are also rather fuzzy and a high degree of computational precision does not seem appropriate. A more detailed description is given in a paper on this analysis (Gaus,92)

One important consideration in selecting the parameters is the information which is available for bridges in the area to be studied and displayed using the GIS system.

For the identification and characterization of the bridges in Erie County, New York which was used as the demonstration area, a tape of the New York State bridge inventory and inspection data was obtained through the courtesy of the New York State Department of Transportation. The bridges in Erie County were then extracted from the larger state data base along with the desired standard data categories for these bridges. The data selected for damage or failure evaluation from bridges actually subjected to damaging earthquakes were matched to the data available from the NYS inventory.

After data identification and classification was completed, a statistical analysis was performed utilizing a standard multiple regression technique. Four intensities of ground motion were selected using peak acceleration value as the defining criteria. The regression analysis simply evaluated the potential contribution of each parameter to the level of damage for each bridge in the data base used. The damage equations y_j which result have the following form:

$$y_j = \sum_{i=1}^{K} \beta_i \cdot X_i + C$$

where y_j is the damage or failure level. This approximate analysis provides a rough indication of how each parameter contributes to the statistical level of damage or failure for the bridges for which data was available.

The model developed above represents the expect level of damage for the entire ensemble of bridges in the data set available. It is also necessary to determine the level of reliability which will result when this model is applied to one individual bridge in the set.

Using a 3 group classification the actual and predicted rank of seismic vulnerability of bridges were compared. The results are shown in Table 1. It was found that the rank of seismic vulnerability of 57 of the bridges among the 74 bridge data set are predicted correctly. In other words for this set there is a 77% probability of correct prediction, which is satisfactory for an initial screening of vulnerability. Due to the procedure used, the probability of correct prediction is almost even for bridges in different ranks of seismic damage.

Many other damage or failure models could be formulated. For example another approach would be to evaluate the level of damage to each class of bridge for the particular code which was in effect in the year when the bridge was designed or modified. Ultimately it would be desirable to develop a damage or failure model which could be directly evaluated for each actual bridge design, or from field data collected specifically for this purpose and from an assessment of the current "state-of-health" of the particular bridge.

Earthquake Source Data and Attenuation model For a given study area the locations of possible earthquake sources which release energy and the variation of surface intensity of ground motion with distance from the source are needed. As the focus of this study in not on earthquake sources or attenuation laws, information available attenuation relationships from published literature was used for the study. The risk analysis procedure is a general methodology and when once developed can easily accommodate different source data or attenuation relationships. Because the data on probable magnitudes are extremely sparse, the GIS-based procedure is set-up to allow a user to carry out "what-if" analysis and to assign various magnitudes to source events to evaluate the potential consequences.

For this study an attenuation equation reported by Atkinson (Atkinson,84) was used which develops surface peak acceleration relationships in terms of earthquake magnitude. It is apparent that problems exist with most of the attenuation relationships currently available and a defensive position is to use a relationship which seems to fit data as close as possible for the study region being considered. As further information becomes available, improved relationships can be substituted. It is desirable to specify sources in terms of magnitude as this is the type of parameter most planners are accustomed to.

Interactive GIS-Based Analysis With the basic procedures for risk analysis defined, the integration with the GIS system can be formulated. As the inventory of bridges ranges from Interstate bridges to local bridges, a system of icons to identify the bridges and their importance in the regional transportation network is required. For each bridge a table or file is prepared containing the attributes which are required for the evaluation (i.e. soil conditions, foundations, piers, etc.)

An attribute table is also prepared for possible sources and their data encoded or digitized. Each source can include potential ranges of magnitudes and their probabilities. As an option the user can specify an arbitrary source and magnitude and display impacts on the bridge network.

Menu bars are defined to display choices such as source locations, bridge types and similar information. For each of these classes a submenu offers choices such as sources likely to have magnitudes greater than a certain level or between certain bounds.

The overall flow diagram for the program is shown in Figure 1.

Graphic data are stored in layered databases and tabular data are interactive with the graphic data. New information generated is contained in mapoverlays.

Standard features of the ARC/INFO program are used for map file input, zooming, print-out and report generation. The use of the GIS system provides a new dimension for engineering analysis and planning.

CONCLUSIONS

GIS systems provide new a new tool and opportunities for study of natural hazard problems including interactive analysis of regional or spatial risk analysis. The GIS system by itself is only an added tool and does not remove the necessity to formulate appropriate analysis techniques for evaluation of problems such as the example on risk assessment for bridges. A valuable feature of the GIS-based approach is that it can provide a general "open-system" methodology in which components such as a damage model or even type of hazard could be easily modified or substituted without having to remanufacture the entire system. The role of GIS in integrating these modules into a harmonic interactive system will find increasing use in the future for studying large-scale natural hazard problems.

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Table 1. Probability of Correct Prediction

A	78%
B	76%
C	77%
Total	77%

Figure 1. GIS - Model Interaction Flowdiagram

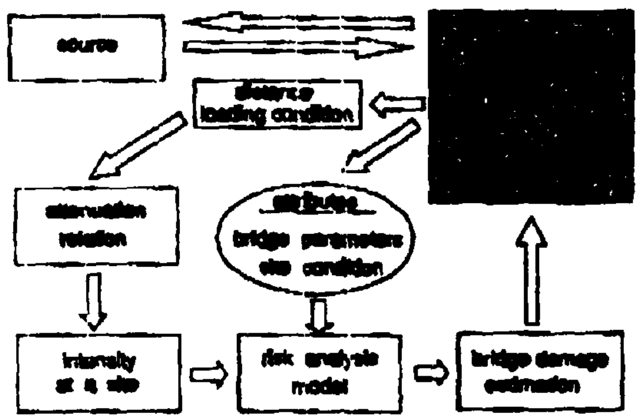
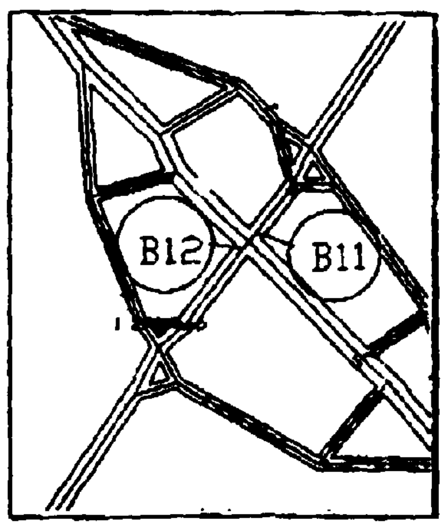


Figure 2. 1:24,000 scale map

Figure 3. Enlarged Area



SECOND US-ASIA CONFERENCE ON
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AN ASSESSMENT MODEL FOR EARTHQUAKE FATALITIES

Keishi Shiono
Department of Civil Engineering
Tokyo Metropolitan University, Japan

ABSTRACT By integrating existing knowledge in seismology, earthquake engineering, and injury epidemiology, we developed a computer model for the estimation of expected fatalities in any given earthquake. Our design purpose for this model was to provide adequate information applicable to the development of safety plans for the reduction of earthquake casualty. We used very limited range of input variables in order to enhance the model's applicability; Required in the estimation were earthquake magnitude, epicentral position, and three regional data of population, building type, and ground condition. We tested the performance accuracy of the model using the data collected from 16 significant disasters between 1962 and 1986.

INTRODUCTION

Since the reduction of earthquake fatalities, particularly, in developing countries has been one of the most important issues toward the enhancement of global disaster safety, a method for casualty estimation applicable to disasters throughout the world has practical significance. Despite the general understanding that damage estimates are helpful in the systematic approach of disaster prevention, any universal model for the calculation of expected casualties has not up to this point been established.

In this study, integrating existing knowledge in the fields of seismology, earthquake engineering, and disaster injury epidemiology, we developed a computer model that provides expected fatalities in a given earthquake. We comprehended that a considerable amount of knowledge applicable to the model has been established and, therefore, decided to lay the principal objective of this development on the earliest accomplishment of a prototype model. We also decided to use only a limited range of input data to enhance the applicability of the model to the real world.

METHOD

Figure 1 shows the general structure of the model developed in this study. Input data for the model are composed of two parts: 1) Seismic information and 2) Regional information. Seismic information includes: 1) Surface wave magnitude (Ms) and 2) Epicentral position. Regional information includes: 1) Population density, 2) Dominant construction type, and 3) Site effect in terms of increment in seismic intensity.

A nine-category classification representing construction types worldwide consists of: 1) Rubble, 2) Adobe, 3) Cut stone masonry, 4) Fired brick masonry, 5) Wood frame with poor infill walls, 6) Wood frame with good infill walls, 7) Wood frame with wood panel walls, 8) Poor quality reinforced concrete, and 9) Good quality reinforced concrete. The classification of building type was done based on wall materials. Other factors of a building such as size and roof material, which may affect the occurrence of human casualties, are expected to have a primary relationship with wall material.

Knowledge, or relationships, used in the model are: 1) Maximum seismic intensity versus magnitude, 2) Attenuation of seismic intensity with distance (Attenuation function), 3) Collapse rate of each construction type versus seismic intensity (Collapse rate function), and 4) Fatality rate in each construction type versus collapse rate (Fatality rate function).

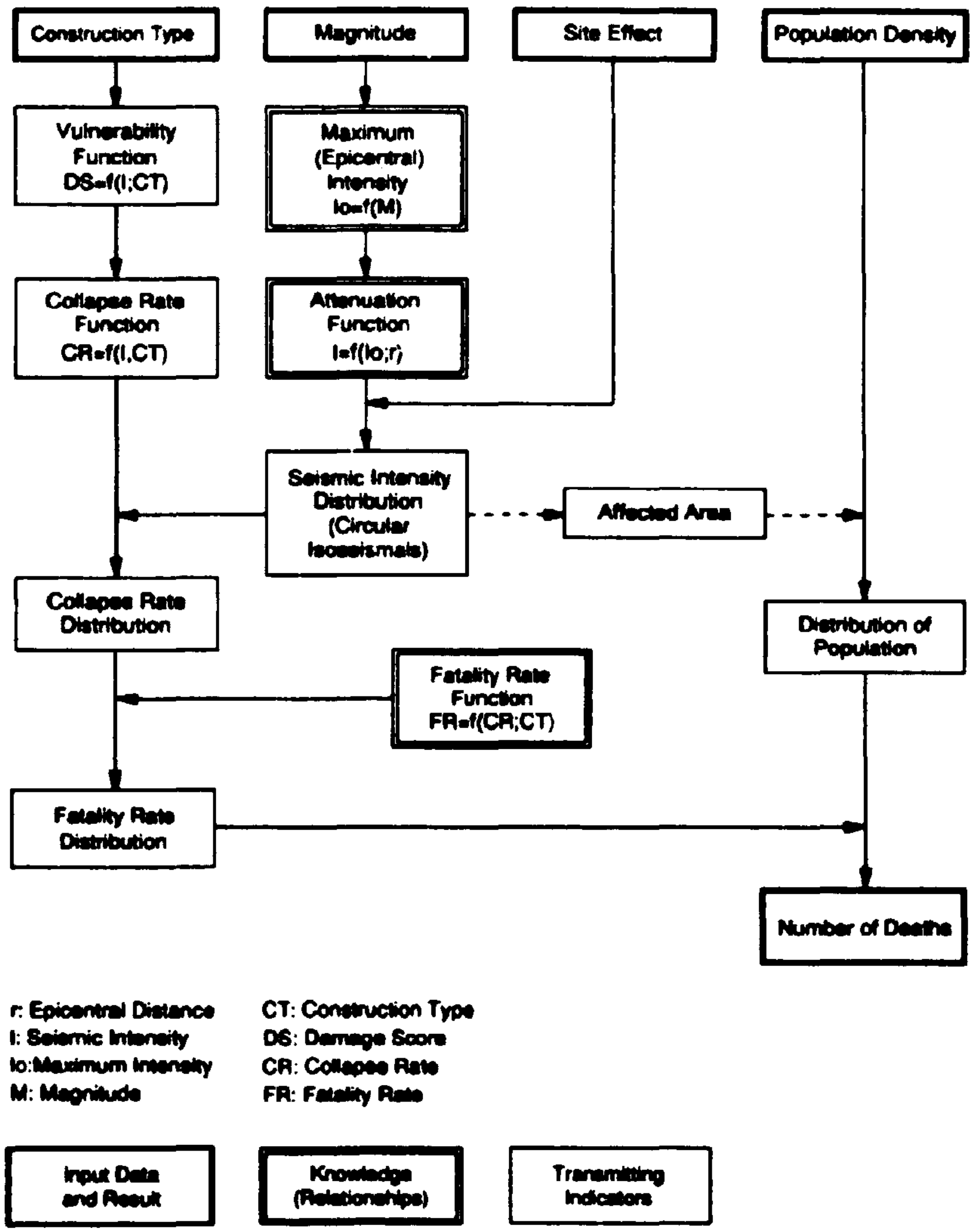

Figure 1. General structure of the model.

The distribution of seismic intensity was determined from: 1) Relationship between magnitude and maximum, or epicentral, intensity and 2) Attenuation function. They must be chosen so as to represent the observation for a particular country or region. Several currently available studies demonstrate these characteristics; for example: Karnik (1965) on maximum intensity and Chandra (1979) on attenuation. The distribution of seismic intensity (i.e., the shape of isoseismal curves) is represented by a set of circles which shear a center at the epicenter.

Structural vulnerability was given in terms of collapse rate functions (Figure 2), which were defined as a relationship between seismic intensity and the rate of buildings "beyond repair" among entire building stock. Seismic intensity on the Medvedev-Sponheur-Karnik (MSK) scale was used in this study.

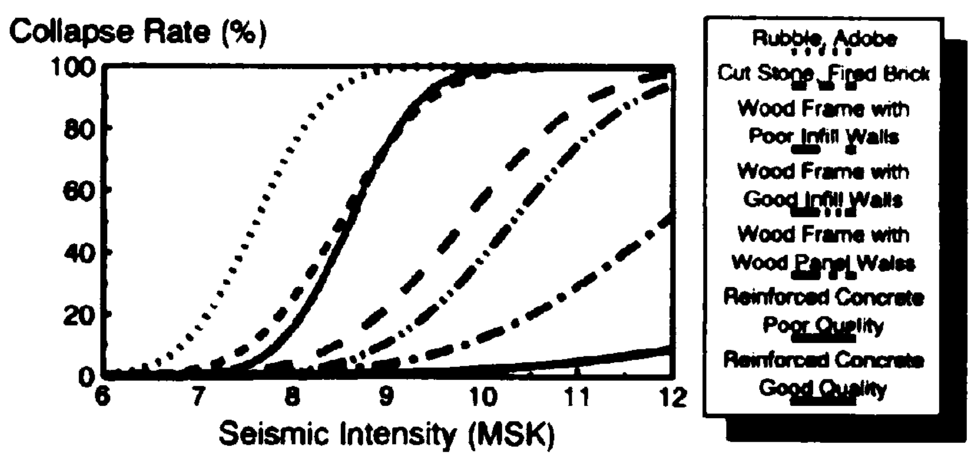

Figure 2. Collapse rate functions.

Fatality rate functions were given with a mathematical expression as follows:

$$FR(CT; CR)=FR_{100}(CT)*(CR/100)^n$$

where, FR: Fatality rate (%), CT: Construction type, FR_{100}: Fatality rate at a collapse rate of 100 percent, CR: Collapse rate (%), and n: Coefficient which account for the non-proportional character between collapse rate and fatality rate.

A value of 1.6 for coefficient n and values of 15%, 20% and 25%, depending on construction type, for coefficient FR_{100} were determined so as to obtain a consistency with the relationships compiled by Coburn and others (1989). FR_{100}'s by construction type are: 15% for wood frame buildings with wood panel walls, 20% for wood frame buildings with poor, or light, infill walls, and 25% for wood frame buildings with good, or heavy, infill walls, and masonry and reinforced concrete buildings.

Fatality rate is affected by, in addition to building damage, numerous factors such as the time of occurrence of an event (time of a day, day of the week, season), victims' characteristics (age, sex, health status), and the effectiveness of rescue activities. However, we used a simplification, in this development, that the most significant affecting factor is the extent of building damage observed with due consideration of construction type.

TEST

To test the performance accuracy of the estimation model, we collected data from 16 earthquakes as listed in Table 1. Those are events having shallow focuses in the interior, to which the majority of earthquake fatalities were attributed. We did not include several disasters in which damage was limited to sites distant from the epicenter region, such as the 1985 Mexico City earthquake, and human casualty was largely due to land failure, such as the 1970 northern Peru earthquake.

Death tolls estimated in the model were plotted in Figure 3 in comparison with reported fatalities. Correspondence between estimates and data was found satisfactory despite the model's simple structure. The average of the ratio of estimated deaths to reported deaths was 1.6, and the corresponding confidence interval at 70 percent was 0.54 to 4.8, or one-third to three times of the average. For the 13 earthquakes responsible for more than one thousand deaths, the average was 1.2, and the corresponding interval was 0.59 to 2.5, or a half to twice of the average.

TABLE 1
EARTHQUAKE DATA USED IN THE TEST

Disaster Area	Year	M	Deaths
1 Buyin-Zara, Iran	1962	7+1/4	12,000-15,000
2 Varto, Turkey	1966	6.5	2,394
3 Adapazari, Turkey	1967	7.5	89
4 Dasht-Bayaz, Iran	1968	7.3	7,000-10,000
5 Gediz, Turkey	1970	7.1	1,086
6 Burder, Turkey	1971	6.0	57
7 Bingol, Turkey	1971	6.7	878
8 Ghir, Iran	1972	7.1	5,000
9 Managua, Nicaragua	1972	6.6	5,000-11,000+
10 Lice, Turkey	1975	6.7	2,385
11 Caldiran, Turkey	1976	7.4	3,840
12 Guatemala	1976	7.5	22,778
13 Tangshan, China	1976	7.8	242,469
14 El-Asnam, Algeria	1980	7.3	2,263
15 Southern Italy	1980	6.8	2,735-4,689
16 San Salvador, El Salvador	1986	5.4	1,500

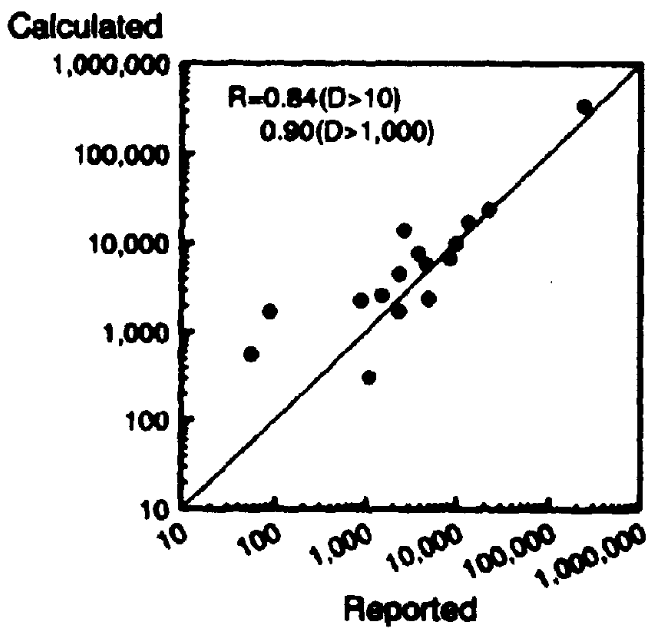

Figure 3. Comparison between calculated and reported death tolls.

CONCLUSIONS

We realized a computer model for the calculation of expected fatalities in a given earthquake and tested its performance accuracy using data collected from significant disasters in the period from 1962 to 1986. The model has a remarkable advantage that it requires only a very limited range of input data that may be collected in any part throughout the world.

In the test, the model performed fairly well in spite of its simple structure and crude nature of the elements that compose the model. Estimates were obtained between one-third and three times of actual data, that corresponds to the confidence interval at 70 percent.

Errors in the estimates were partly attributed to the affecting factors that we did not include in our current model. Some of those factors are: 1) building size, 2) building elements other than walls, 3) occupancy and its time dependency, and 4) effectiveness of rescue activity. In addition, errors were partly resulted from the assumptions used in the estimation such as circular approximation of isoseismals and coarse classification of building types worldwide into nine categories. These problems must be included in the future improvement of the estimation model.

For the model's applicability to practical use, preparation of input data is a major concern. Database development is presently a significant issue in addition to the improvement of the model itself.

ACKNOWLEDGEMENTS

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ANALYSIS AND DESIGN OF A BASE-ISOLATED BUILDING

Tso-Chien Pan[1], Alan H. Muhr[2] and James M. Kelly[3]
1. School of Civil and Structural Engineering
Nanyang Technological University, Singapore
2. Tun Abdul Razak Laboratory, Brickendonbury,
Hertford SG13 8NL, England
3. Dept. of Civil Engineering, UC Berkeley, CA 94720 USA

ABSTRACT: This paper presents the preliminary analysis and design of the first base-isolated building located in Shantou City, China. Based on the column load distribution, circular isolation bearings of 600-mm diameter and 190-mm thickness are proposed. The bearings are designed for a 120-mm design displacement and a 240-mm maximum displacement. Supported on 23 bearings, the isolated building has a 0.5 Hz natural frequency. Subjected to the site-specific spectrum of 0.2 g peak ground acceleration, the maximum acceleration response is 0.16 g for the isolated building compared with 0.43 g for the fixed-base building. The maximum displacement response at the base level of the isolated building is 105 mm.

INTRODUCTION

The first use of rubber for earthquake protection was in an elementary school in Skopje, Yugoslavia (Siegenthaler 1970). It rests on large blocks of unreinforced natural rubber which are unlikely to be used again. The first base-isolated building in the U.S. was the Foothill Communities Law and Justice Center in San Bernardino, California. The building is four-story high and sits on 98 natural rubber bearings reinforced with steel plates. In recent years, several Japanese construction companies have built demonstration buildings on isolation systems. A review of the status of base isolation in Japan is given in (Kelly 1988).

This paper describes the preliminary analysis and design of the first base-isolated building in China. The building is part of a base isolation demonstration project for Shantou City on the southeast coast of China. The design of the seismic isolation bearings is discussed in detail. Subjected to a site-specific response spectrum, the response of the isolated building is compared with that of the fixed-base building.

SHANTOU BUILDING

The Shantou building is an 8-story reinforced concrete beam-column frame structure. The front elevation and a typical section of the building are shown in Figs. 1(a) and 1(b), respectively. The total building height is 24.6 m. The ground floor with a story height of 3.6 m is for commercial use, while the upper floors with a typical story height of 3.0 m accommodate three residential units per floor. The plan dimensions of the building are 10.5 m × 24.3 m. Fig. 2(a) shows the plan of foundation that supports the isolation bearings located along the column grid lines. A typical section at the foundation level, Fig. 2(b), reveals the proposed base isolation scheme.

The design strength of concrete are 25 MPa for the columns and 20 MPa for the beams and slabs, while the yield strength of reinforcement is specified as 340 MPa. The floor system consists of an 80-mm concrete slab supported by beams with a 200 mm × 500 mm section. The column section varies along the building height. It starts with 350 mm × 550 mm for the first two floors, reduces to 350 mm × 450 mm for the next two floors, and finally becomes 350 mm × 350 mm for the upper floors. Besides the structural dead load, the building is designed for a live load of 3.0 kN/m^2 and 1.5 kN/m^2 for the ground floor and the other floors, respectively.

Shantou City is one of the many major cities in China that is located in an earthquake prone area. The peak ground acceleration (PGA) at Shantou City is assumed to be of 0.2 g for the design-basis earthquake that has a 10% probability of exceedance in 50 years. The maximum credible earthquake for the site is assumed to be twice the magnitude of the design-basis earthquake. The soil at the building site consists primarily of silty sands and clayey sands. The average shear wave velocity is determined to be 117 m/s for a 15-m depth and 140 m/s for a 30-m depth. The soil at the building site can thus be considered soft, i.e. Soil Profile Type S4 (UBC 1991).

Fig. 3(a) shows a site-specific ground acceleration response spectrum developed for a damping value of 5%. The spectrum represents the ground motion of the design-basis earthquake and is therefore anchored at the PGA value of 0.2 g. While the response spectrum is utilized in the preliminary analysis, synthetic time histories compatible with the spectrum will be utilized in the final analysis. For a base-isolated buiiding of 0.5 Hz natural frequency, Fig. 3(a) gives a spectral displacement of 140 mm. The design displacement for the bearings is however taken as 120 mm, reduced from the spectral displacement to account for the 10% damping provided by the high damping rubber bearings. The maximum displacement associated with the maximum credible earthquake is then taken as 240 mm for the bearing design.

DESIGN OF ISOLATION SYSTEM

The design requirements for the isolation system are (a) provision of a natural frequency of 0.5 Hz and 10% critical damping at the 120-mm design displacement, (b) the displacement arising from the maximum likely wind force to be limited to an acceptable level, and (c) to perform without failure up to the 240-mm maximum displacement. In order to maximize the likelihood of uptake of base isolation in China, the isolation system should also be inexpensive.

As a rule of thumb, a bearing may support columns of loads varying up to ±25% of its design load. According to the column load distribution, one type of bearing is designed for column loads ranging from 1,000 to 1,700 kN, and the other for loads from 1,700 to 2,800 kN. Thus, there will be 12 bearings supporting a mean load of

V = 1,424 kN per bearing (*Type I*) and 11 bearings supporting a mean load of V = 2,225 kN per bearing (*Type II*). The shear stiffness K_s of a bearing is given by V and the required natural frequency f as $K_s = (2\pi f)^2 V/g$. For $f = 0.5$ Hz, $K_s = 1.43$ kN/mm for Type I bearings, and $K_s = 2.24$ kN/mm for Type II bearings.

If the rubber will experience a 100% shear strain at the 120-mm design displacement, the radius r for a circular bearing can be obtained from $r^2 = 120 K_s/\pi G$, where G is the shear modulus of the rubber. If $G = 1$ MPa, substituting G and K_s = 2.24 kN/mm into the formula yields r = 293 mm for Type II bearings. For Type I bearings, a softer rubber compound is used in order to avoid a second set of tooling.

According to one specification (AASHTO 1989), the bearing stress must be limited by $V/A < 1.0GS$, where $A = \pi r^2$ is the area and $S = r/2h$ is the *shape factor* for a circular bearing. With r = 290 mm, the formula leads to a rubber layer thickness h < 17 mm, i.e. the minimum number of rubber layers should be seven. Thus, the division of rubber into eight layers would give sufficient vertical stiffness and safety factor for stability (AASHTO 1989, Thomas 1983). Choosing appropriate dimensions for the steel reinforcing plates and the rubber cover layer results in a circular bearing of 600-mm diameter and 190-mm thickness. The properties of final bearing design are listed in Table 1. In Table 1, the G value has been adjusted slightly, so as to maintain the K_s value when a more sophisticated formula is used.

SEISMIC RESPONSE

For seismic analysis of a base-isolated building, it is usual to represent the system by a linear viscously damped model. The first mode of the isolated building is mainly a rigid body mode with nearly all of the deformation occurring in the rubber bearings. The seismic input to the structure can thus be treated as an equivalent lateral load proportional to the rigid body mode. Since it is a characteristics of a linear vibrating system that all modes are mutually orthogonal, all higher modes will be orthogonal to the input motion. Therefore, the isolation system works not by absorbing the ground motion energies but by deflecting them.

The eigenvalue analysis of the Shantou building is carried out for comparing the dynamic characteristics of the fixed-base (FB) condition with those of the base-isolated (BI) condition. Fig. 4(a) shows the first mode of 1.8 Hz for the FB case, which exhibits the typical behavior of a frame structure. Fig. 4(b) shows the first mode of 0.5 Hz for the BI case, which demonstrates the nearly straight line rigid body mode shape above the isolation level.

The seismic response of the building with the FB and BI conditions to the site-specific design spectrum, Fig. 3(a), are determined. The acceleration response profiles along the building height are shown in Figs. 5(a) and 5(b) for the transverse and longitudinal directions, respectively. For the FB condition, the acceleration response is amplified along the building height and reaches a maximum of 0.43 g at the roof level. The maximum FB response is therefore amplified 215% from the PGA value of 0.2 g. For the BI condition, the acceleration response appears to be linear and gradually increases from 0.12 g at the base level to 0.16 g at the roof level. Therefore, the accelerations in the BI building are below the ground accelerations, and the maximum value is only 80% of the PGA value.

The displacement response at the base level of the BI building is 100 mm in the transverse direction and 105 mm in the longitudinal direction. The amplitudes are below the 120-mm design displacement for the bearings. Note that the displacement response would have been lower if a design spectrum of 10% damping is used.

CONCLUSIONS

Based on the column load distribution, two types of circular bearings are proposed. The bearings are of 600-mm diameter and 190-mm thickness. Providing a 0.5 Hz natural frequency to the BI building, the bearings are designed for a 120-mm design displacement and a 240-mm maximum displacement. Subjected to the site-specific spectrum of 0.2 g PGA, the maximum acceleration is 0.16 g for the BI building compared with 0.43 g for the FB building. The maximum displacement response at the base level of the BI building is 105 mm, below the design displacement.

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Table 1: BEARING PROPERTIES

Type	G (MPa)	V (tonne)	K_s (kN/mm)	f (Hz)
I	0.60	145.2	1.43	0.49
II	0.96	226.8	2.24	0.50

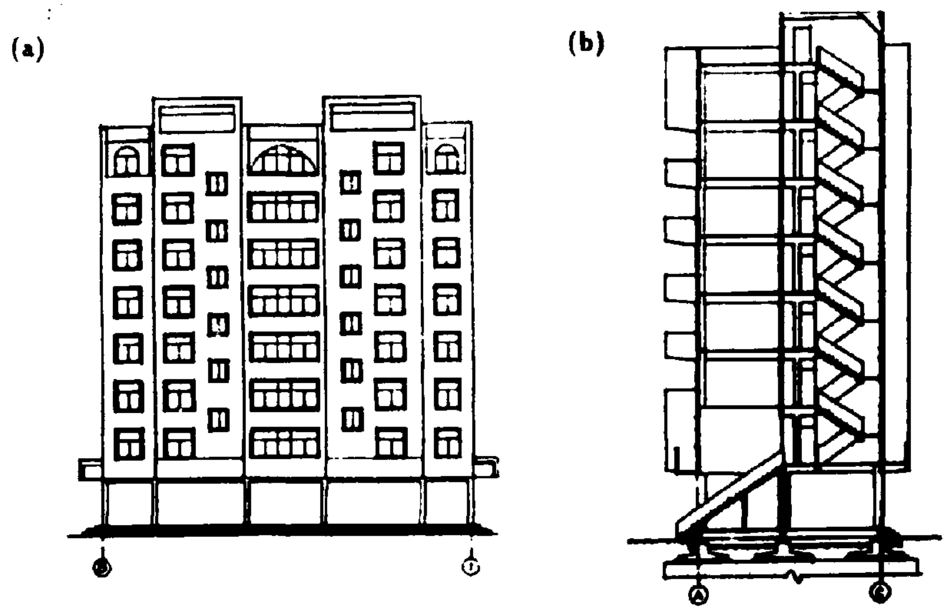

Fig. 1. Shantou building: (a) Elevation; (b) Section

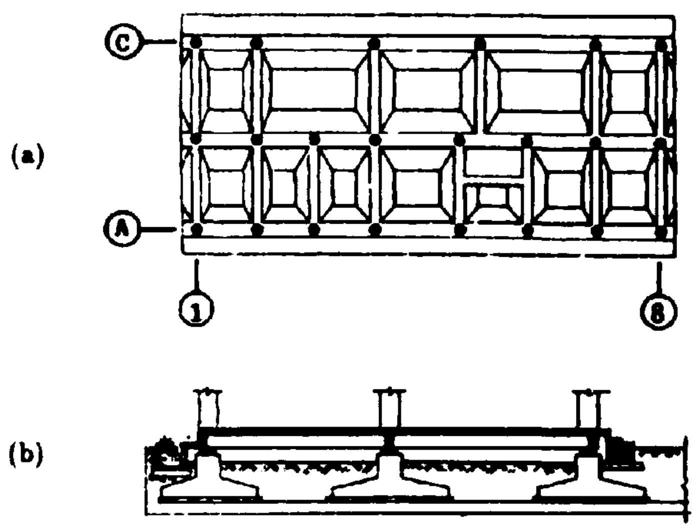

Fig. 2. (a) Foundation plan; (b) Section

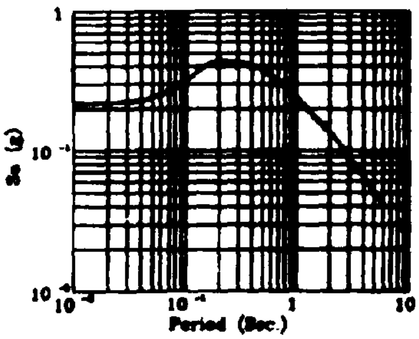

Fig. 3. Site-specific spectrum

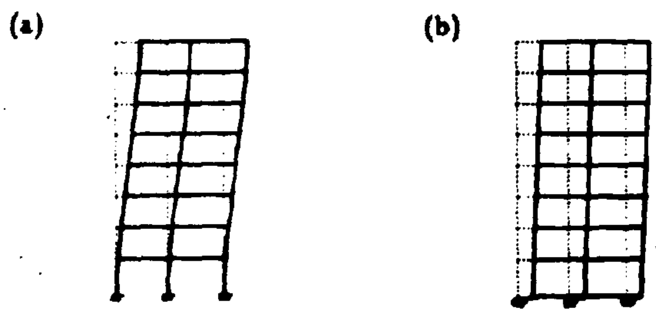

Fig. 4. Fundamental mode shape: (a) Fixed-base; (b) Base-isolated

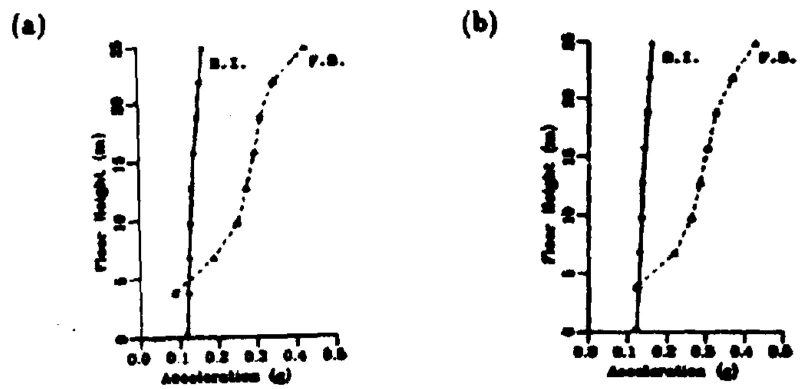

Fig. 5. Acceleration response: (a) Transverse direction; (b) Longitudinal direction

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RISK MANAGEMENT FOR NATURAL DISASTERS
A GLOBAL PERSPECTIVE

Haresh C. Shah
Obayashi Professor of Engineering
Chairman, Department of Civil Engineering
Stanford University

ABSTRACT Almost eighty-five percent of all deaths due to natural disasters occur in Asia and the Pacific Rim. As the urbanization of major cities and communities increase at a spectacular rate, the risk to lives and property continues to increase. It is no wonder that in spite of major technological and scientific innovations, the level of risk in many parts of the world is not decreasing.

Planners, scientists, and engineers have studied various natural hazards and their mitigation strategies. However, there is relatively very little work done on an integrated approach to risk management. Earthquakes, which kill more people than all other natural disasters, has been intensely studied over the past three decades. However, even for this hazard, very limited effort is spent in developing a balanced resource allocation strategy to maximize benefits and minimize losses.

Two important segments of the risk management groups that have not been involved in research or implementation are the financial and the insurance industries. Even though these two sectors deal with the enormous consequences of natural disasters, their expertise, their needs and their input have not been integrated with other professional and scientific sectors. In short, the scientific and technological communities (knowledge generators) do not communicate with the financial--banking and insurance--communities (knowledge users). The result is that less than optimum strategies are used to manage risk.

This paper will review the current state-of-the-art in earthquake risk management. It will then provide elements of integrated risk management strategies to mitigate the effects of earthquake hazard. The suggested strategies and how they should be implemented in developing and developed nations will be discussed. The goals of IDNDR and means for achieving these goals will be presented.

INTRODUCTION

During the past five decades, considerable research and developmental work has been conducted to mitigate the effects (life loss and injury, property loss, economic disruption, social and political impact, etc.) of natural disasters on mankind. Most of the fundamental research has been conducted to understand the natural phenomenon itself and the response of man made facilities to these disastrous events. Our knowledge about earthquakes, hurricanes, tornadoes, etc. is a lot better today than it was just a decade or two ago. Our understanding of how structures respond to these extreme events has also improved substantially.

In spite of such impressive scientific and technical gains, many lives are lost and considerable economic disruption occurs due to natural disasters. As a matter of fact, and great concern is the realization that the global risk due to natural hazards is not decreasing. It is natural to ask as to why this lack of risk reduction even though our technical and scientific knowledge is so much better? In this paper, we will explore some of the issues that are relevant to this question. The specific hazard that we will focus on will be earthquakes. We will explore as to why earthquakes cause so many casualties. Life loss and property loss reduction strategies will also be investigated.

The main theme of this paper is to introduce the concept of integrated risk management. There is very little effort currently underway to develop a strong, integrated and balanced risk management strategy which could maximize benefits of allocated resources and minimize the following category of losses life and injury; property damage; business interruption; lost opportunities; building contents and damage; long-term social, economic and political implications; other losses.

This paper does not provide all the needed answers. However, it does raise some questions about resource allocation, urbanization, disjointed risk management and fragmentation of applicable strategies.

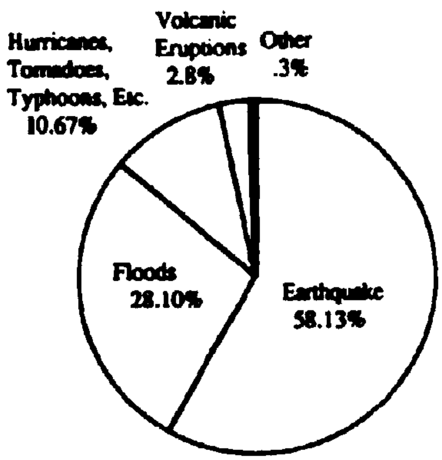

Figure 1a. Lives lost due to natural disasters
Data base 1900-1976. Total lives lost - 4.6 million

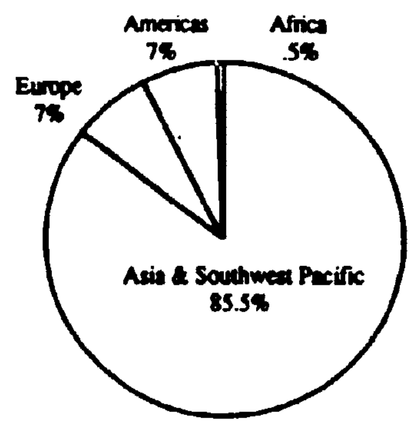

Figure 1b. Natural disasters death tolls
Territorial deaths, 1900-1987

Table 1. NATURAL DISASTERS BETWEEN 1900 AND 1976

Type	*Killed*	*Injured or Homeless*
Earthquakes	2,662,165	28,894,657
Volcanic Eruptions	128,058	337,931
Floods	1,287,645	175,220,220
Landslides	3,006	44,673
Avalanches	3,059	150
Cyclones	434,894	17,648,463
Hurricanes	18,513	1,197,535
Typhoons	34,103	5,437,054
Storms	7,110	3,432,641
Tornadoes	1,195	342,459
TOTAL	4,579,748	332,555,783

"Science Today," July 1980 issue, pp. 53-59
The table does not include the figures for drought

RISK OF NATURAL DISASTERS

During the past nine decades, approximately four and half million people have died world wide due to all natural disasters. The rate at which these casualties have occurred have remained relatively constant for each of the last nine decades. Figure 1a shows the causes due to which these deaths have occurred from 1990-1976. Table 1 provides the same data for numbers killed and numbers made homeless. Figure 1b shows that most lives lost due to natural disasters are in Asia and in Southwest Pacific ocean.

There are many reasons as to why such a major proportion of all lives lost occur in Asia and Southwest Pacific ocean. Some of these reasons may be large populations; large urban population living in substandard housing; the oceans of this region generate many storms and flooding; existence of the circumpacific "ring of fire", plate boundaries and volcanic chains; general economic conditions in countries of this region; and poor construction practices. Over the past centuries, we have seen that earthquakes have killed hundreds of thousands of people in major urban centers. Table 2 shows major historical earthquakes.

Table 2. MAJOR HISTORICAL EARTHQUAKES

Date	Location	Deaths
893 A.D.	Northeast India	180,000
Sept 8, 1138	Aleppo, Syria	100,000
Jan 23, 1156	Xian, China	830,000
Dec 30, 1727	Hokkaido, Japan	137,000
Oct 11, 1737	Calcutta, India	300,000
Jan 3, 1920	Ningxia, China	200,000
Sept 1, 1923	Tokyo, Japan	100,000
May 31, 1970	Chimbote, Peru	67,000
Feb 4, 1976	Guatemala	23,000
July 26, 1976	Tangshan, China	250,000

It can be seen from Table 2 that most casualties are in urban centers with very large populations. One of the reasons why we are not reducing global risk is that the rate at which we are urbanizing is much faster than the rate at which we are implementing our integrated knowledge about the science, engineering and socioeconomic aspects of earthquakes. Table 3 shows major urban centers that are in seismic zones. it can be seen that massive increase in population will take place between 1975 and 2000. It is estimated that an earthquake similar to the Great Kanto earthquake of 1923 can kill, even today, more than 100,000, injure more than 200,000 and can destroy more than 750,000 buildings in the Tokyo region. The total economic impact could be as much as one million dollars. Such an event not only would impact Japan and its economy, but could badly disrupt world economy.

In light of such earthquake induced hazards and the enormity of the potential losses, there is an increasing concern on how to manage earthquake risk Walter Hayes of the United States Geological Survey reinforces such concerns:

> *. . . An urgent need exists for earthquake risk management on all scales . . . it is clear that the economic value of the dwellings, buildings, public and private facilities, and lifeline systems that are at risk from earthquakes is not only very large (billions of dollars on the global scale), but also that it is growing with time. This situation calls for action now!*

Table 3. CITIES OF POPULATION 2 MILLION OR MORE THAT ARE IN SEISMIC ZONES (*POTENTIAL MAG* ≥ 7)

	1975	2000
Mexico City	11.6	26.0
Tokyo	16.4	20.0
Jakarta	5.5	13.0
Los Angeles	9.0	11.0
Beijing	8.9	11.0
Lima	3.7	9.1
Algiers	1.6	5.1
Baghdad	2.7	7.5
Naples	3.8	4.3
San Francisco	3.0	5.0

Many individuals, organizations, and governments have failed to recognize the full extent of their potential earthquake exposure. This has occurred due to several factors: seeming remoteness of a major seismic event; lack of appreciation of the loss potential (life, property, etc.); lack of tools to cost-effectively quantify risk; fragmented approach to risk management

It all seems so overwhelming. Earthquakes appear to occur at random intervals in random locations in random sizes and cause seemingly random amounts of damage. Fortunately, earthquake risk can be managed just like other economic uncertainties by knowing as much as possible about the risk, narrowing down the uncertainty, and planning a strategy or trade-off accordingly. In the next section, we will describe various elements of risk management strategies. It is appropriate to repeat here that the key to risk management is to develop an integrated approach which is consistent with the level of risk and the level of available resources.

AN INTEGRATED EARTHQUAKE RISK MANAGEMENT PROCESS

Impact assessment and loss reduction planning are complex issues that require a synergistic and integrated approach. Global management of earthquake risk demands proper understanding of all the risk reduction options and selection of those options in conformance with socioeconomic, political, technical, and scientific environment of the region.

Just like many risk management strategies in other sectors such as financial, medical, environmental, etc., earthquake risk can be managed. However, the necessary expertise and know how is not readily available to the end user. Earthquake risk assessment expertise is largely confined to a few research institutions, universities and specialized consulting firms and is not generally available to decision makers and planners in government, financial institutions, insurance industry and other non-technical users. It is difficult for non-engineers to define and obtain the relevant input for earthquake risk evaluation and use the analytical tools necessary to apply this data for decision making. Further confusing the situation is the fragmented nature of the earthquake research and consulting community. The requisite expertise is scattered over several disciplines such as seismology, geology, geotechnology, and structural and earthquake engineering; all play a role in research and implementation process. Advances made in one field are often overlooked by the other disciplines. Furthermore, research on the impact of earthquakes is largely based on empirical data and hence conclusions are modified with every new earthquake. As a result, it is difficult to keep up with a result, it is difficult to keep up with the dynamics of the profession.

Edwin Simner of Lloyds of London (one of the users of risk management strategies) sums of the insurance industry's frustration with the current state of earthquake knowledge dissemination in general:

. . . [w]e are asked to write earthquake insurance with only the most trivial of information. For example, I have too frequently refused to provide reinsurance because the broker presenting the request could not give me fundamental information . . . the scientists have it, and we should have it, too.

The above state of affairs exist because we do not look at earthquake risk management problem in an integrated manner manner. Let us enumerate life loss and property loss reduction strategies (see Table 4). The seven strategies outlined provide a comprehensive manner of managing earthquake risk. Let us investigate separately the seven strategies and then see as to how they should be viewed in an integrated manner.

Table 4. LIFE LOSS AND PROPERTY LOSS REDUCTION STRATEGIES

1.	Prediction	Life protection
2.	Collapse protection through engineering design	Life and economic loss potential
3.	Insurance	Economic loss potential
4.	Disaster preparedness	Mitigation of post-earthquake trauma
5.	Education and training	Better able to cope with disasters
6.	Transfer of knowledge from knowledge generators to knowledge users	Life loss and economic loss reduction due to easier implementation of developed strategies
7.	Land use planning and zoning	Reduce economic and life loss

Considerable research has been conducted by geologists, geophysicists, and other earth scientists to study anomalies and precursors. This strategy does provide life loss protection if the prediction for the event can be made with fairly high reliability in time and space. Unfortunately, at the current time, technology and science has not been able to provide such accurate prediction. However, even long term prediction (almost statistical forecasting) has considerable merit. For example, the seismic gap theory does provide excellent information to planners, engineers, and other public officials about the relative likelihood of earthquakes on different segments of the same fault or on different faults. Research and development in this direction needs to be continued, at least in countries where resources are available for research on this topic. However it should be noted that even with very accurate prediction, the economic losses cannot be minimized. As a matter of some concern is the negative impact of failed prediction.

Collapse protection through engineering design provides the most cost effective strategy to minimize life and economic losses. After every earthquake, it has been observed that "good" engineering minimizes life and property losses. Our state of knowledge about performance of structures has increased substantially over the past three decades. Unfortunately, there are many existing structures, built over the past 100 years, which are still in use and which do not conform to our concept of "good design" as viewed from our current state of knowledge. This is especially true for developing countries where rural housing, and to some extent urban housing does not have the needed resistance against collapse due to even a moderate size earthquakes. Even a small amount of strengthening can save many lives. An important risk management strategy should involve various strengthening strategies for existing structures.

Insurance is one of the best, and in fact, minimally utilized risk reduction strategy. Use of this option can minimize property losses, business interruption losses and other regional economic losses. Reasons for under utilization of this option are complex and numerous. They include lack of state of the art knowledge on the past of insurance carriers, resulting in reluctance on their part to market earthquake insurance, unreasonable deductibles and premiums, and lack of knowledge on the part of general public about the costs and benefits of this option. A better implementation of strategy 6 (Transfer of Knowledge. . .) will make this option both attractive and effective for major urban center of the world.

Disaster preparedness is most central in managing any disaster. Proper level of preparedness on the part of governments can minimize post earthquake trauma and increase the pace of recovery. This option must be used in all risk management strategies, and at all levels -- from individuals to regional, state and federal governmental levels.

It is true that a better informed and trained citizenry can cope well with natural disasters. It is extremely important that simple, easy to understand information must be made available to the general public. There must be regular (and once in a while surprise) disaster drills. Thus, strategy 4 and 5 must go hand in hand.

One of the biggest problems in risk management is that knowledge generators and knowledge users do not communicate. Interdisciplinary communication, even for experts, is very poor. This has to be improved. Knowledge necessary for effective earthquake risk management exists. The missing link is a cost-effective mechanism to transfer the state of the art knowledge in a useable form. An example of such a system is called the Insurance and Investment Risk Assessment System (IRAS), developed at Stanford University. It is a link between science, engineering and financial community. More effort needs to be put in such knowledge transfer strategies.

Land use planning and zoning is one of the most effective risk reduction strategies. Properly implemented, it can mitigate losses (life and property) due to geological and geotechnical problems at a specific site. Even though this strategy is effective, its implementation involves considerable economic and political concerns. Hence, very often, political issues govern land use planning and zoning rather than technical or scientific issues. Whenever possible, this strategy should be a part of any global risk management strategy.

Having described the above seven strategies, it is obvious that a prudent (and practical) mix of these strategies can result in an effective risk management program. It seems that strategy 2, in conjunction with strategies 3, 4, 5 and 6 will buy maximum benefit. Strategies 4, 5 and 6 are extremely cost effective. Strategy 3 is the most under utilized and cost effective strategy if insurance companies, through strategy 6 can improve their insurance premium and deductible estimates. We have found that current premium and deductibles are high because there is considerable uncertainty (ignorance!!) amongst insurance companies about their financial exposure. Strategy 6 can certainly help reduce this ignorance. In any case, it is sufficient to understand that a combination of the seven strategies (based on cost-benefit analysis of social, political and economic considerations) will provide a strong integrated risk management strategy.

CONCLUSION

The greatest challenge in developing seismic risk management procedures is to make them less "mysterious" and more usable. Whether we are tying to identify high risk structures or developing strengthening strategies for such high risk structures to minimize risk, we must find vehicles by which knowledge generators and knowledge users are communicating. Earthquakes can affect large urban areas in developed or developing countries. Some of the largest urban regions of the world are in such countries with relatively high seismic risk. Government planners, economic decision makers do not have appropriate tools to help them understand the extent and relevance of seismic risk. There are ways in which they could manage that risk if they were properly informed. The knowledge base on which they currently make their decisions is not consistent with the state of knowledge currently existing in research (and to a lesser degree in practicing engineering) community. This lack of integration must change.

Another problem that exists is the problem of technology transfer between the developed and the developing regions of the world. Massive reliable data bases about earthquake occurrences, source modeling, source mechanism, instrumental recordings, and attenuation, exists in few central locations around the world. A global and regional network of such centers could help many developing countries who cannot afford to maintain their own data bases. The challenge is not only to develop new models for source mechanism or new attenuation or new occurrence model; the most pressing challenge is to assist the world in utilizing the current know how. The challenge is to develop IRAS type systems that any informed user can utilize for improving the quality of seismic know how. Such an effort would provide opportunities to assess earthquake risks in regions of the world where historically many have died in past earthquakes. Such an awareness and technology would help in identifying factors that increase or decrease seismic vulnerability of communities or regions.

The International Decade for Natural Disaster Reduction (IDNDR) could be an excellent vehicle to achieve the above stated objectives. Amongst the ten most important programs that the EERI Committee on IDNDR has identified as candidate projects for the decade, at least four projects address the issues of international cooperation, technology transfer and reduction of seismic risk through intelligent development of mitigation strategies, and through integrated risk management procedures.

SECOND US-ASIA CONFERENCE ON
ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
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PROBABILISTIC SEISMIC HAZARD ESTIMATES IN NORTH SULAWESI PROVINCE, INDONESIA

Paul C. Thenhaus[1], Iamet Effendi[2], Engkon Kertapati[2]
1. U.S. Geological Survey
Golden, Colorado USA
2. Geological Research and Development Center
Bandung, Indonesia

ABSTRACT Earthquake ground-motions in North Sulawesi on soft soil that have a 90 percent probability of not being exceeded in 50 years are estimated to be 0.46 g (46 percent of the acceleration of gravity) at Palu, 0.31 g at Gorontalo, and 0.27 g at Manado. Estimated ground motions for rock conditions for the same probability level and exposure time are 56 percent of those for soft soil. The hazard estimates are obtained from seismic sources that model the earthquake potential to a depth of 100 km beneath northern and central Sulawesi and include the Palu fault zone of western Sulawesi, the North Sulawesi subduction zone and the southern most segment of the Sangihe subduction zone beneath the Molucca Sea. An attenuation relation derived from Japanese strong-motion data and considered appropriate for subduction environments of the western Pacific was used in determination of expected ground motions.

INTRODUCTION

Following the 18 April 1990 North Sulawesi earthquake (M_s 7.3), the Indonesian Geological Research and Development Center and the U.S. Geological Survey (USGS) initiated cooperative pilot studies under the auspices of the International Decade for Natural Disaster Reduction. The purposes of the studies, performed through the Worldwide Earthquake Risk Management program (WWERM) of the USGS, were to 1) map the earthquake ground-motion hazard throughout North Sulawesi in terms useful to engineering design using a probabilistic methodology, 2) determine what sources of information were available for the estimation of economic loss to dwellings and demonstrate techniques of estimating loss at a chosen site in North Sulawesi, 3) exchange technical expertise in the seismotectonics of the Indonesian region and in probabilistic seismic hazard and risk assessment, and 4) provide the Geological Research and Development Center with the necessary computer hardware and software to carry out seismic hazard and risk assessments. This paper summarizes aspects of the seismic hazard investigation carried out under the project.

PROBABILISTIC GROUND-MOTION HAZARD MODEL

Probabilistic ground-motion hazard maps of the United States (Algermissen and others, 1982) have become the standard basis for earthquake-resistant design requirements, but also have found widespread application in regional land-use planning, emergency preparedness, and insurance analysis. Development of probabilistic ground-motion hazard maps involves three principal steps: 1) delineation of seismic source zones or faults, 2) analysis of the magnitude-frequency distribution of historical earthquakes in each seismic source zone, and 3) calculation and mapping of the extreme cumulative probability $F_{max,t}$ (a) of ground motion, a, for some time, t. In this study, the maximum amplitude of ground motion in a given number of years corresponding to any level of probability is determined using the Poisson extreme probability function:

$$F_{max,t}(a) = e^{\Phi t[1-F(a)]} \qquad (1)$$

where Φ is the mean annual rate occurrence of earthquakes above some minimum magnitude, t is a particular exposure time, and $F(a)$ is the probability that an observed acceleration is less than or equal to the value a, given that an earthquake above some minimum magnitude of interest has occurred.

The earthquake catalog for this study was obtained from a search of the world data base of earthquakes maintained by the U.S. Geological Survey, National Earthquake bins 0.6 units wide. All zones were considered capable of producing great (M_s 8.0+) earthquakes.

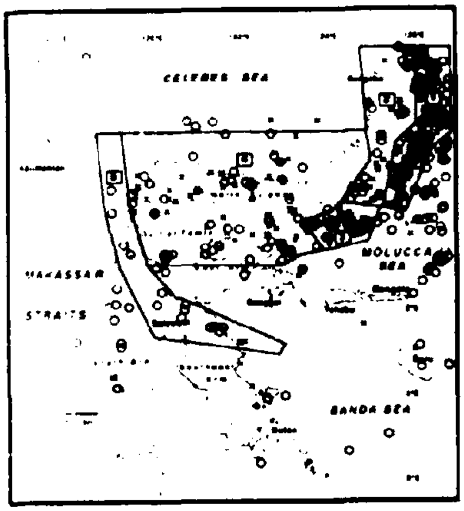

FIGURE 1a. Shallow seismicity and seismic source zones of North Sulawesi. Circles indicate hypocenters ≤25 km deep. X's indicate hypocenters 25<h≤50 km deep.

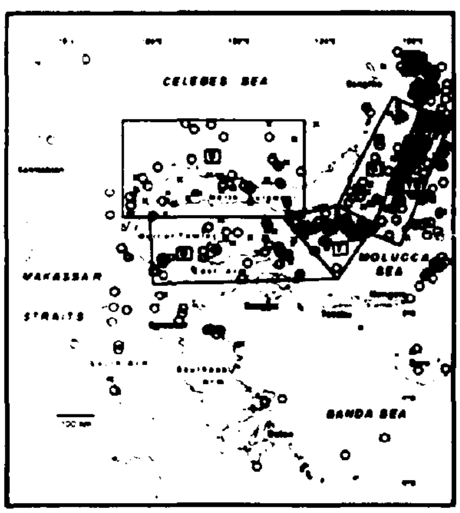

FIGURE 1b. Deep seismicity and seismic source zones of North Sulawesi. Circles indicate hypocenters 50<h≤75 km deep. X's indicate hypocenters 75<h≤100 km deep.

Figures 1a and b illustrate the seismicity of the Sulawesi region and the seismic source zones in which the zonal activity rates of Table 1 were modeled. The activity rate for Subcatalog 1 (Table 1) was determined on seismicity of the Molucca collision zone (Fig. 2) to 100 km depth. The activity rate for Subcatalog 2 was determined from seismicity to a depth of 100 km associated with the leading edge of the Sangihe plate and the subjacent Benioff zone associated with the subducted Molucca plate. Zone 1 models shallow seismicity of this area at a depth of 25 km. Zone 6 models the Benioff zone seismicity and dips 28° northwestward from 25 km at the Molucca collision zone to 75 km at its northwestern boundary. Subcatalog 3 is an east-west seismicity trend beneath the Gorontalo Basin that extends through the 100 km depth of the catalog (Fig. 2, 1a, b). The seismic zone lacks an apparent dip, even for well recorded earthquakes (Cardwell and others, 1980), which indicates that the earthquakes are not associated with the west-dipping Benioff zone of the subducted Molucca plate to the north. Subcatalog 4 (Table 1) contains seismicity occurring in the vicinity of

Information Service (Fig. 1a, b). Hypocentral depths in teleseismic earthquake catalogs of the region are of poor quality, having uncertainties of perhaps tens of kilometers for earthquakes that are well-recorded and significantly more for poorly recorded earthquakes (Cardwell and others, 1980). Hence, high uncertainty would exist in earthquake rate determinations derived from small samples of earthquakes that might be judged to correlate with the many tectonic elements in the region. To minimize such uncertainty in the rate estimates, seismicity parameters were determined for five subcatalogs of earthquakes that included seismicity to a depth of 100 km (Table 1). Each subcatalog corresponded regionally to related tectonic elements. Estimates of the percentage of the overall rate for

TABLE 1. SEISMIC ACTIVITY RATES FOR SEISMIC SOURCES OF NORTH SULAWESI AND THE SURROUNDING REGION.

Cat/Zone *(1)*	**log $N = a + bM$** *(2)*	**fraction** *(3)*	**area (km^2)** *(4)*	**a (per km^2)** *(5)*
Subcatalog 1	6.168 - 0.99 M_s (σ=0.06)	1.00		
Zone 1	5.953 - 0.99 M_s	0.60	37,211	1.382
Zone10	5.759 - 0.99 M_s	0.40	29,315	1.292
Subcatalog 2	8.703 - 1.54 M_s (σ=0.26)	1.00		
Zone 2	8.443 - 1.54 M_s	0.55	43,215	3.807
Zone 6	8.356 - 1.54 M_s	0.45	33,748	3.828
Subcatalog 3	5.789 - 1.01 M_s (σ=0.15)	1.00		
Zone 3	5.308 - 1.01 M_s	0.33	15,908	1.106
Zone 7	5.615 - 1.01 M_s	0.67	27,085	1.182
Subcatalog 4	6.354 - 1.07 M_s (σ=0.09)	1.00		
Zone 4	6.160 - 1.07 M_s	0.64	186,150	0.890
Zone 8	5.468 - 1.07 M_s	0.13	88,457	0.521
Zone 9	5.716 - 1.07 M_s	0.23	120,613	0.635
Subcatalog 5	3.897 - 0.79 M_s (σ=0.16)	1.00	57,903	-0.866

(1) Subcatalogs and source zones used for seismicity parameter determinations. Individual zone numbers are keyed to Figures 1a and b for identification. Subcatalog 5 corresponds directly to the rate determination for Zone 5. *(2)* Gutenberg-Richter parameters a and b for the subcatalog fits on incremental magnitude data giving the standard deviation of the regressions and parameters for the zonal activity rates. *(3)* the fractional percentage of the subcatalog fit distributed into each zone, *(4)* the area of each seismic source zone, *(5)* the area-normalized a value for each seismic source zone.

each subcatalog that belonged to each tectonic element (*i.e.*, each seismic source zone) were based on judgements of the number of earthquakes apparently related spatially to the tectonic element and on tectonic interpretations that are outlined subsequently. Activity rates for each subcatalog were determined according to the maximum liklihood method described by Weichert (1980) and Bender (1983) for magnitude data grouped in magnitude

North Sulawesi and the northern part of the East Arm of Sulawesi (Fig. 1a). In the shallow tectonic regime, this region of northeast Sulawesi appears to be rotating clockwise as a microplate that is decoupled from the peninsular arms of southern Sulawesi (Hamilton, 1979). Left-lateral differential movement is accommodated across the Palu and Matano fault zones through central and western Sulawesi (Fig. 2) A consequence of this rotation is that the North Sulawesi arm overrides the Celebes seafloor of the Sangihe plate along the North Sulawesi trench. Zone 9 (Fig. 2b) dips 12° southward at 25 km and models seismicity of the North Sulawesi subduction zone to a mean depth of 75 km. Subcatalog 5 (Table 1) corresponds to Zone 5 (Fig. 1a) and models seismicity alignments that follow the Palu and Matano fault zones. Both faults show geomorphic evidence of young movement with the Palu fault having ruptured historically (Hamilton, 1979). Earthquakes greater than M_s 6.4 are modeled as linear ruptures along the traces of the faults. Earthquakes M_s 5.0-6.4 are modeled as point sources distributed uniformly in Zone 5.

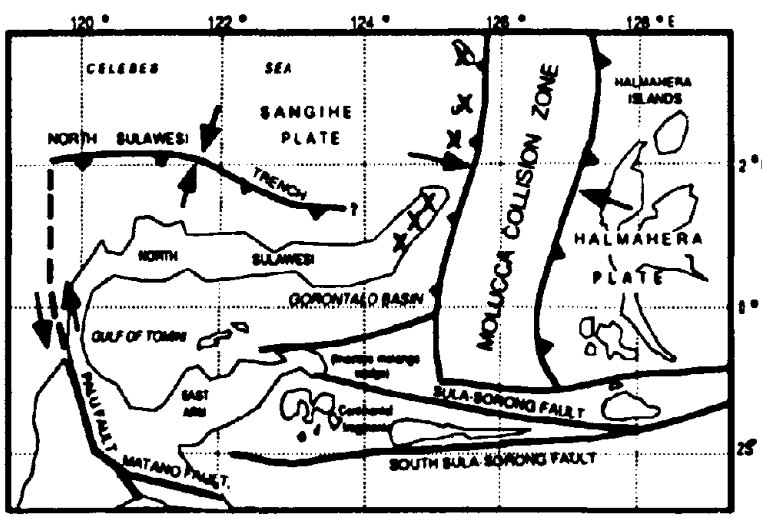

Figure 2. Index map of selected geographic and tectonic elements of North Sulawesi (adapted from Hamilton, 1979). Bold arrows indicate relative plate movements across tectonic boundaries. X's indicate Sangihe volcanic arc. Teeth indicate dip direction of Benioff zones.

The few strong ground-motion recordings available for Indonesia are of poor quality and are insufficient for generalizing attenuation characteristics in the region. Therefore, a recently developed attenuation relation for Japan (Fukushima and Tanaka, location above the south-dipping North Sulawesi subduction zone and the potential for great earthquakes to rupture that zone. Figure 4 compares the hazard estimates for rock and soft soil site conditions for exposure times of 10 to 300 years.

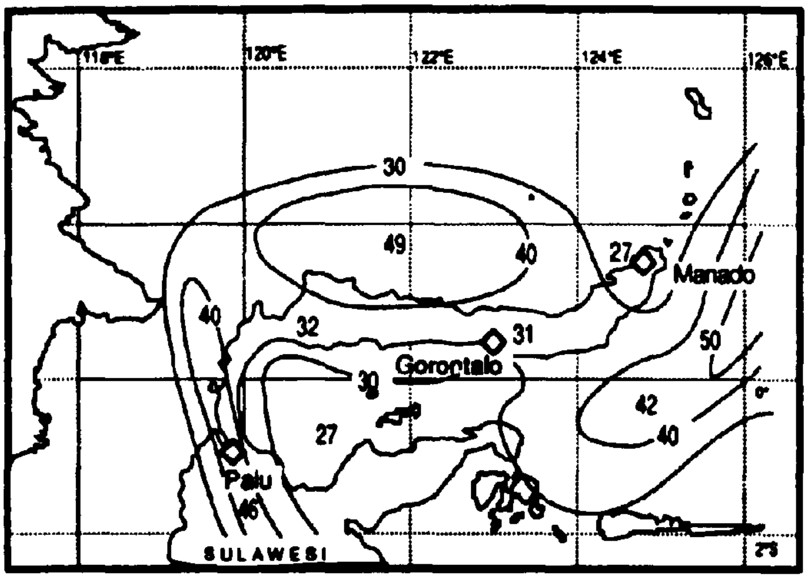

Figure 3. Peak horizontal acceleration in soft soil having a 90 percent probability of not being exceeded in 50 years. Contours are in percent of gravity (%g).

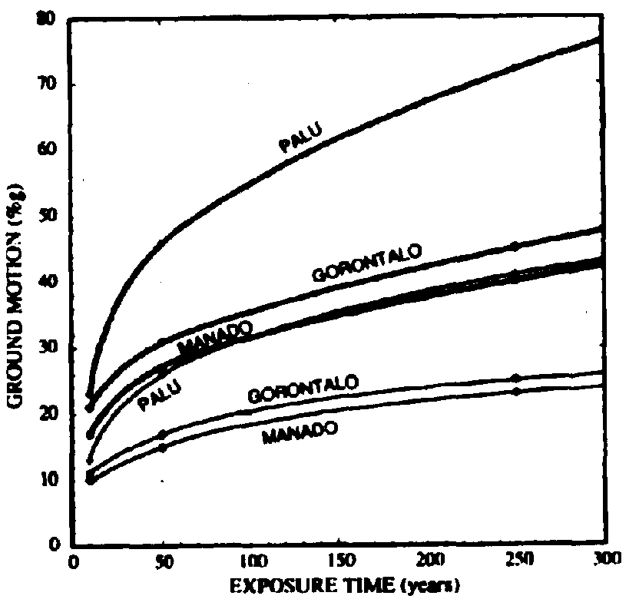

Figure 4. Comparison of the ground-motion hazard at Palu, Gorontalo and Manado in North Sulawesi Province. Ground motion values have a 90 percent probability of not being exceeded in their corresponding exposure times. Bold lines indicate the hazard for soft soil site conditions. Light lines indicate the hazard for rock site conditions.

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SEISMICITY, AND ITS RELATION TO THE VOLCANIC'S ACTIVITY IN INDONESIA

Gunawan Ibrahim and M. Ahmad
Dept. of Geophysic and Meteorology
Institute of Technology Bandung, Indonesia

ABSTRACT The seismicity in Indonesia related with the Benioff zones which subduct in different directions with heterogent dip angles. Many trenches found in this region indicated subduction zones. The tectonics structure in the eastern Indonesia more complex than in the western Indonesia.

The Indonesia region is an active seismic area, it is recorded that on the average about 460 earthquakes per year with the magnitude equal and greater than 4 in Richter scale.

The subduction zones of 180 km depth are founded from Sumatra to West Java, and begin of Central Java to Flores reaching 665 km depth. In the Banda Sea region, the subduction zones are face to face and have convect form with decreasing depth from west to east: from 650 to 96 km. In the other case, around Molucca area the tectonic plates descend to the west reaching 658 km depth, and to the east into the depth of about 275 km forming a concave. The aseismic zones of 80 - 282 km width can be found between Central Java and Flores, and about 223 km width the south of Mindanao.

In the simple subduction of Sumatra, the distribution of volcanoes correspond to the end subduction plate. In Java, up to Banda Sea area, the volcanoes have nor direct relation to discontinued zone. In South Molucca the activity of volcanoes correspond to the end of subduction zone, but in Central Molucca it is found that the volcanoes are above the aseismic area.

INTRODUCTION

The Indonesia archipelago lies in the end of three main tectonic plates, that is, Eurasian plate, the Indo-Australian plate and the Pacific plates. The boundary of those plates in Indonesian region mostly consist of Benioff zone with different angles and various directions.

In the western Indonesia, the Indo-Australian plate subduct from the south side to the north beneath the Eurasian plate, recognized as a Mediterranee earthquake zone. Meanwhile in eastern Indonesia, Pacific plate, Philippines plate, Eurasian plate, Indo-Australian plate are joined together called as Pacific belt earthquake zone.

In the Indonesia archipelago can be found many trenches, indicates the subduction zones such as Sunda trench, Banda region trench, Molucca trench, and North Sulawesi trench. There are 129 active volcanoes which are distributed along Sumatra, Java, Nusatenggara, Banda area, North Sulawesi and Molucca.

Based on study in Central America, Tonga-Kermadec, Hanus and Vanek (1985) find the direct relation between the seismic gap in subduction zone and volcanoes distribution. This study discuss the relation between seismic gap in subduction zone and volcanic's activity in Indonesia.

SEISMICITY IN INDONESIA

The Indonesian region is an active seismic area; it is recorded that on the average about 460 earthquakes per year with the magnitude equal or greater than 4 in Richter scale. They are shallow, intermediate, and deep earthquakes because most of hypocenter are situated in the subduction areas. Mostly, the earthquakes occur of magnitude between 5.0 and 5.5.

The shallow earthquakes was located along the west side of Sumatra, Southern Java, Nusatenggara, Molucca, Banda area and Irian Jaya. The intermediate earthquakes distributed in West Sumatra, Java, Nusatenggara, Banda area and Molucca. The deep earthquakes occur begin in northern Central Java, north of Nusatenggara till western Molucca. The focal mechanism in the Banda area and in Molucca areas more complex than in the Sumatra and

Java area. It is because the tectonic structure in the eastern Indonesia is more complex than in western Indonesia.

The distribution of earthquakes in Indonesia is concentrated in the area of subduction zone. In Sumatra, the depth maximum of earthquakes is 180 km, it means the plate subduct till 180 km. Beginning of Central Java to Flores, the Benioff zone have a depth of 665 km with aseismic zones between the depth of 260 to 542 km in Central Java and between 280 to 360 km in Flores area.

In the Banda Sea area, the subduction zones are complex. The trenches make a circle near Weber basin and face to face each other. In this region, the subduction have a convect form with decreasing depth from west to east. The depth of the subduct plate in East Flores reaches 650 km, and in Tanimbar is about 96 km. In the south side, the length of the subduction decreased from 902 to 282 km, while in the north side from 562 to 368 km.

Around the Sangihe archipelago in Molucca area, the tectonic plates descend to the west reaching 658 km in depth, and to the east into the depth of about 275 km forming a concave. In general, the subduction zones in the west side are deeper than the east side.

Figure 1 shows the maximum ground acceleration in Sumatra based on McGuire's formula for the period 100 years: Bengkulu area is 100 gal, South Aceh and along western Sumatra are 60 gal. In Selat Sunda the maximum ground acceleration is greater than 80 gal, meanwhile in southern Central Java, and Bali are 60 gal, figure 2.

SUBDUCTION ZONES AND VOLCANIC ACTIVITY

Subduction zone is an active tectonic area, many earthquakes and volcanoes are found in this zone. Indonesia lies in triple junction plate, most of them create subduction zones.

In Sumatra, the Benioff zone subduct till 180 km, the location of volcanoes correspond to the earthquakes depth of 100 - 140 km. The aseismic zones in Java found between the depth of 260 - 542 km, and the volcanoes are located in the region which relates to earthquakes epicenter between 100 to 200 km, figure 3.

The volcanoes have not direct relation to discontinued zone in Banda Sea area. In South Molucca, there is no seismic gap, the volcanoes distribution correspond to the end of subduction zone, but in Central Molucca it is found that the location of volcanoes are above the seismic area.

CONCLUSION

The maximum earthquake depth in Sumatra is 180 km, and the distribution of volcanoes correspond to the end of subduction plate. In Java, the seismic gap found between the depth 260 - 542 km, the volcanoes was located in the regions which relates to earthquakes epicenter between 100 to 200 km.

In Banda Sea area, the volcanoes have not direct relation to aseismic zone. In South Molucca, the activity of volcanoes connected are to the end of subduction zone, and in Central Molucca it is found that the volcanoes are above the aseismic area.

The Indonesian zones of seismic hazard mostly lies in the western Sumatra and southern Java. In eastern Indonesia, even though the earthquake frequency is higher than the one in western Indonesia, its intensity is smaller. The reason for this, the eastern Indonesia mostly covered by the ocean and less populated. Nevertheless tsunami still becomes a threat.

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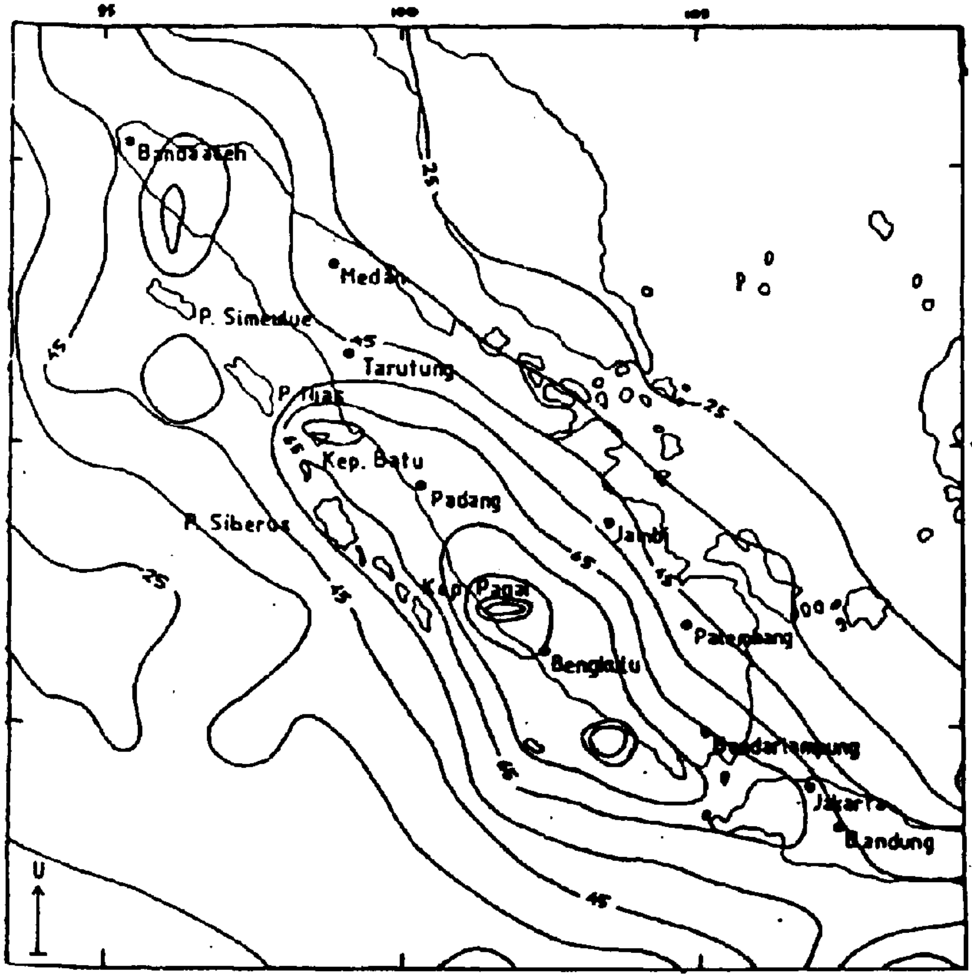


Figure 1. Maximum ground acceleration in Sumatra, return periode 100 years.

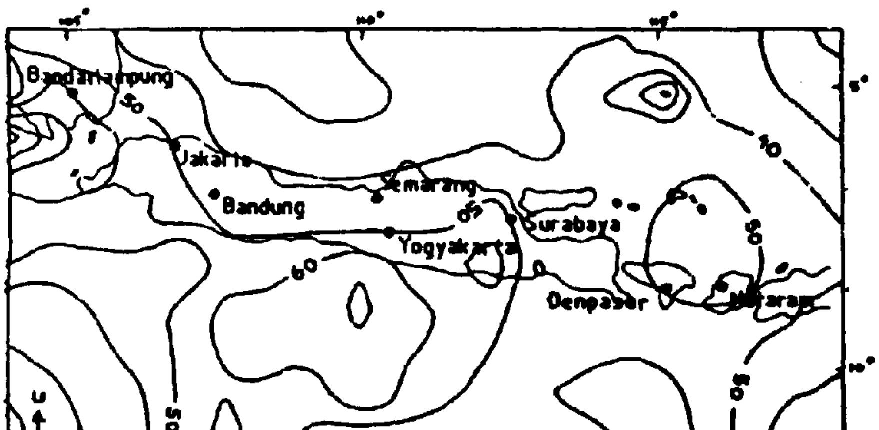
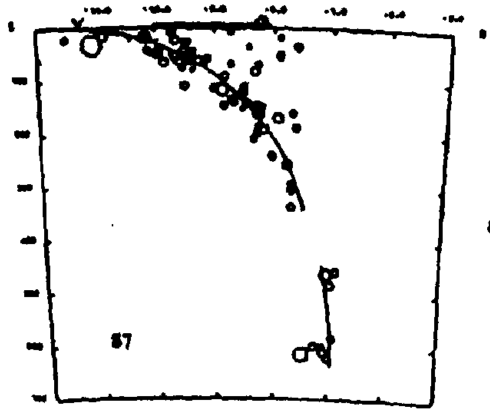
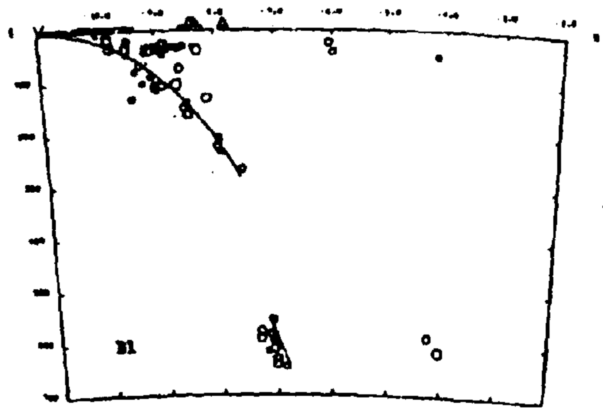


Figure 2. Maximum; ground acceleration in Java, return periode 100 years.



a)



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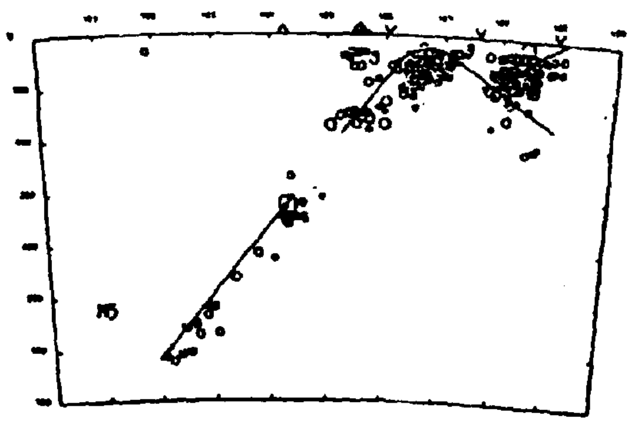

c)

Figure 3. The distribution of earthquakes epicenter and volcanoes:
a') Java b) Banda Sea area c) Molucca area.

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TECHNOLOGY TRANSFER IN EARTHQUAKE ENGINEERING RESEARCH

George C. Lee[1], and Ian G. Buckle[2]
1. Former Acting Director
2. Deputy Director
National Center for Earthquake Engineering Research
State University of New York at Buffalo, USA

ABSTRACT There is a widespread view in the United States that a major impediment to earthquake hazard reduction is not the lack of new knowledge but rather the delayed application of available knowledge. Technology transfer is therefore being given greater emphasis by those federal and state agencies responsible for mitigating the earthquake hazard.

The National Center for Earthquake Engineering Research was established by the National Science Foundation in 1986 for the express purpose of conducting systematic research in earthquake engineering and to improve the rate of transfer of research results in practice. Accordingly, the Center is actively engaged in technology transfer by a variety of means. These activities may be grouped under "traditional" and "special initiative" headings.

Traditional technology transfer mechanisms include: (a) publication of technical reports, (b) conduct of conferences, workshops and seminars, (c) conduct of short courses for the professions and (d) development of design aids and user-friendly computer software.

However, traditional methods are passive by nature and need to be supplemented by special initiatives. Pro-active strategies, which the Center has found to be successful, include methods that improve the usefulness of the

research product and providing improved access to research results and related information.

Techniques for improving the usefulness and quality of research include using multidisciplinary research teams, placing end-users on research teams and sponsoring demonstration projects. Improved access is provided through a new Information Service, the development of expert systems and the placement of qualified researchers on code committees.

This paper summarizes these activities and illustrates their potential benefits by describing a case study in the seismic vulnerability of a water delivery system.

INTRODUCTION

There is a widespread view in the United States that a major impediment to earthquake hazard reduction is not the lack of new knowledge but rather the delayed application of available knowledge. Technology transfer is therefore being given greater emphasis by those federal and state agencies responsible for mitigating the earthquake hazard.

However if the rate of technology transfer is to catch up and keep pace with the generation of new knowledge, traditional methods of transfer must be supplemented by special, non-traditional, initiatives.

The National Center for Earthquake Engineering Research (NCEER) was established by the National Science Foundation in 1986 for the express purpose of conducting systematic research in earthquake engineering and to improve the transfer rate of research results into practice. Accordingly, the Center is actively engaged in technology transfer by a variety of means.

Headquartered on the campus of the State University of New York at Buffalo, the Center is a consortium of academic and professional institutions, financially supported by the National Science Foundation and the State of New York. To date it has funded 166 projects at 44 institutions in 20 states. Its mission is to reduce the earthquake hazard in the United States through research and technology transfer. The Center considers earthquake engineering to be more than just theoretical modeling and structural testing. It has established research programs ranging from the seismicity of the eastern United States to water delivery systems, from structural testing of nonductile lightly reinforced concrete joints to buried pipelines, from intelligent structures to fire following earthquake and from base isolation to the societal impact of earthquakes.

Technology transfer is sometimes confused with implementation, but implementation of research results, in the strict meaning of the word, is not a function of this Center. Instead the Center plays a supporting role by helping others who are responsible for implementation such as designers, building officials, and constructors. Various mechanisms are being used to improve the efficiency of this process and these may be grouped under traditional and non-traditional headings.

It is to be noted that no single method of technology transfer will, on its own be universally successful but that a mix of strategies, tailored according to topic and audience, will be necessary for optimum results. It is also true that the Center is still experimenting with various options and will no doubt revise its program in time, as it accumulates experience in this endeavor.

TRADITIONAL TECHNOLOGY TRANSFER

Traditional methods of technology transfer include:

- publication of technical reports
- conduct of workshops, seminars and conferences
- conduct of short courses on specialized topics
- dissemination of computer software

NCEER is using all of these techniques to disseminate its research results. Some of this activity is described below.

Technical Reports and Special Publications

To date, 172 technical reports have been published which describe the results of research projects funded by the Center. Since the Center was established in 1986, over 35,000 copies of these reports have been distributed, worldwide. The Center maintains an exchange program with other universities, corporations, and Government agencies. Currently over 30 institutions are members of this program.

In addition, 500 copies of the English translation of the Japanese research report, *Manual for Repair Methods of Civil Engineering Structures Damaged by Earthquakes*, have been distributed by NCEER. Other special publications produced by the Center include a commentary to the New York City Seismic Building Provisions and a reprinting of Myron Fuller's book *The New Madrid Earthquake*.

Workshops, Conferences and Seminars

NCEER schedules a large number of workshops, conferences, briefings and seminars each year for the purpose of educating the practicing community and the general public as well as for sharing the state-of-the-art of research knowledge. The Center's objective here is to:

- Conduct and participate in workshops, conferences and meetings, to bring together researchers and practitioners to discuss issues related to seismic hazard mitigation, for application to earthquake-related problems in engineering, urban planning, geosciences, and education
- Conduct seminars on earthquakes to educate an audience about the hazards of earhtquake and related research, and to promote interaction between NCEER researchers and other earthquake experts
- Provide opportunities for practicing professionals to benefit from the expanding knowledge base in earthquake hazards research, by offering professional development seminars and short courses.

Examples of the topics covered by these meetings include:

- earthquake hazards and the design of constructed facilities in the eastern United States

- expert systems which can be shaped and used by practicing engineers for the seismic design of strucutres
- post-earthquake reconnaissance findings
- innovative technical developments in engineering and seismology
- geotechnical impact of earthquakes on lifelines and other buried structures
- post-earthquake serviceability of water delivery systems
- socioeconomic problem areas which result from earthquakes and other natural disasters and evaluation of potential solutions

<u>Computer Software</u>

To date, approximately 40 computer programs have been developed under NCEER sponsorship. Not all of these programs are suitable for immediate use by the practicing profession, but a significant number were written specifically for the end-user. These are currently being disseminated through the National Information Service in Earthquake Engineering (NISEE), at the University of California in Berkeley. Some examples are:

IDARC	-	Inelastic damage analysis of reinforced concrete structures
DYNA1D	-	Nonlinear site response analysis
3D-BASIS	-	Nonlinear dynamic analysis of three-dimensional base-isolated structures
GEOBASE	-	Strong-motion data base interface

Additional efforts related to the development of computer-based expert systems, are described below.

NON-TRADITIONAL TECHNOLOGY TRANSFER

The preceding section has presented a number of traditional mechanisms for technology transfer which although widely used, are seldom sufficient to assure the transfer of research results into practice. The Center has therefore supplemented these "passive" activities with a number of pro-active, special initiatives. These can be grouped broadly under two headings:

- improving the quality and usefulness of the research product

and
- improving access to the research product

Techniques used by the Center to improve the quality and usefulness of research results include using multidisciplinary research teams, placing end-users on the research teams and sponsoring demonstration projects. Improved access to existing knowledge and research products is being provided through an Information Service, the development of expert systems and the placement of knowledgeable researchers on code committees. Some of these activities are described below.

Multidisciplinary Research Teams, End-users and Demonstration Projects

Multidisciplinary teams which include seismologists, engineers and social scientists, not only improve the quality of work done, but they also facilitate the implementation process. For example, the involvement of the servo-hydraulic industry, a construction company and university researchers in a coordinated research team has led to the construction of the first full-scale intelligent structure (an actively controlled six-story building). This demonstration building will be used to not only advance the state-of-the-art in active control, but also to identify and overcome impediments in the application of this technology in real-world structures.

The technology transfer process can also be accelerated if the research team includes a member of the user community. This is particularly true if this end-user has on-line responsibility for the consequences of earthquake-related damage, such as a hospital owner, a fire department chief or a city building official.

It is well established that a demonstration project can be extremely useful as a final phase in a major research program. Such a project can provide a focal point for those researchers who are working on various aspects of the same problem, and at the same time, the field experience can enhance the quality of the research products. However, the Center has also found that these projects can stimulate the implementation of research findings in the real-world by demonstration to the profession and the public at large, the applicability of the research findings. Furthermore, the active involvement of the end-users of the research in these projects also facilitates the transfer of these technologies in a remarkably effective manner.

Examples of NCEER demonstration projects either planned or underway at this time include:

- Active control of the Takenaka Building and the Akron Tower
- Equipment evaluation in hospitals
- Vulnerability study of the electric, gas, water and oil lifelines in Shelby County, Tennessee
- Vulnerability study of the Auxiliary Water Supply System (AWSS) and the Municipal Water Supply System (MWSS) in San Francisco
- Response and retrofit of a typical Eastern highway bridge
- Regional study of the East Bay Municipal Utility District and the Hetch Hetchy Aqueduct System

Information Service, Expert Systems, Code Committees

The Center established an Information Service in 1987, with a two-fold objective; first to provide researchers with access to current literature and second to provide a service to practicing engineers and other professionals who are unfamiliar with the earthquake literature and who need assistance with finding publications and other information relevant to their interest.

Housed in the Science and Engineering Library of the University at Buffalo, the Center's Information Service responds to about 200 reference queries per month from around the world. A circulating collection of tens of thousands of books, journals, technical reports, maps, and audio-visual media is maintained, and new acquisitions are constantly added. The Service publishes a monthly newsletter, the *NCEER Information Service News*, which is sent to hundreds of selected researchers and practitioners in many countries. The Service also develops and maintains the QUAKELINE database.

Quakeline

The Information Service established the QUAKELINE database in mid-

1987 in response to a need for more thorough indexing and abstracting of the literature in the field.

It was considered that many publications of great interest were not being indexed in traditional bibliographic resources, such as COMPENDEX. As a result, the Service adopted the task of indexing and abstracting a variety of items, most of which are not indexed elsewhere and are often considered to be fugitive literature. These items can include technical reports, domestic and foreign journals, books and book chapters, professional papers, theses and dissertations, and other formats.

Materials chosen for inclusion in QUAKELINE must be directly related to earthquakes, earthquake engineering, seismic design, the dynamic properties of materials, natural disaster hazard mitigation, disaster preparedness, response and recovery, wind engineering, or closely related topics. Abstracts are provided for all items in the database. Many times the abstracts are taken directly from the publication itself and may be edited for space considerations. When not provided in the original document, abstracts are written by the Service staff.

QUAKELINE now contains over 14,000 records, and about 400 records are added monthly. The database is mounted on the BRS system, a product of BRS Technologies, Inc. The association with BRS provides three distinct advantages: BRS offers simple telephone access throughout much of the world; BRS provides sophisticated search software, with Boolean search strategies and the capacity for complex and field-specific searches; and BRS' search language is well known to many librarians and researchers in the field.

Potential QUAKELINE users need only a terminal, modem, and telephone line to take advantage of remote searching of the database; no special equipment or software is required. The sophisticated BRS search software is accessed online along with the database. As an alternate method of accessing QUAKELINE for those who do not wish to do their own searching, the staff at the Service perform online searches on request, for a modest fee. It is clear that many users of the database prefer to have an information specialist perform their searchers.

Computer-based expert systems for the seismic evaluation, design and retrofit of structures offer a potentially powerful mechanism for accelerating the transfer of expert knowledge from experienced professionals and researchers to any engineer with a personal computer and a telephone. NCEER has funded the development of two expert systems in this field that have innovative architecture and which are abased on good engineering practice. These are EVAL (at Carnegie-Mellon) for the evaluation of existing buildings, and STRAKE (at Cornell) for the design of new buildings. Both expert systems, which run on engineering workstations, are now available for trial use to engineers experienced in earthquake-resistant design of buildings. Feedback from these trial sites will greatly strengthen and expand the knowledge base in these programs making them powerful tools for students and engineers with less experience in seismic design. This will be particularly important as attention to seismic design issues increases in the eastern United States.

Another strategy adopted by the Center, has been to encourage prominent researchers to actively participate in the development of design and construction standards. To date, the Center has placed researchers on code development committees for buildings (for both New York City and the State of New York); for highway bridges (for the American Association of State Highway and Transportation Officials); for seismic (base) isolation; and for the National Earthquake Hazards Reduction Program (NEHRP) Revisions to Recommended Seismic Provisions (national building provisions).

Some of the above-mentioned methods for technology transfer are illustrated in the following case study.

CASE STUDY

One of the Center's current research projects concerns the seismic vulnerability of water supply systems. Ground failure due to an earthquake can rupture buried pipelines and the consequential loss of water can have serious consequences for the local population not just for consumption but also for the control of fire following the earthquake.

This study is a cooperative venture between researchers at Cornell University, consultants at EQE, San Francisco and end-users in the Water and Fire Departments of the City of San Francisco.

Computer codes for the hydraulic analysis of pipeline systems have been developed and used to develop evaluation methods for existing pipeline systems. Strategies for design, operation and rehabilitation of such systems have also been developed and correlated against historical performance in past earthquakes. Workshops have been held and a strong working relationship developed between Center researchers and officials of the San Francisco City Water and Fire Departments. This led, for example, to the passage of a $46.2 million bond issue to improve the City water supply with protective features against earthquake damage. It also led to the City retaining one of its two fireboats at a time when both were up for sale. As it turned out, the availability of the fireboat Phoenix on the night of October 17, 1989 was crucial to the eventual suppression of the fire in the Marina District which followed the Loma Prieta earthquake.

These events are considered evidence of an effective and rapid transfer of technology which had an immediate and obvious benefit to the City of San Francisco. The methods used were a mix of traditional (a workshop) and non-traditional means (a multidisciplinary research team involving end-users). This example is significant because it shows that technology transfer can be improved by the use of special initiatives.

CONCLUSION

A significant impediment to earthquake hazard mitigation is the relatively slow rate of application of new knowledge in the real-world.

If the rate of information transfer is to catch up and keep pace with the generation of new knowledge, traditional methods of transfer must be supplemented by special measures which may be thought of as "non-traditional". This paper has summarized some of the traditional and non-traditional methods that the National Center for Earthquake Engineering Research is using to accelerate the transfer of research results into practice. It is concluded that no single method will, on its own, be universally successful, but that a mix of strategies will prove to be more effective. Notwithstanding its modest success so far, NCEER is continuing to look for new and improved ways to transfer research results into practice. Only by applying innovation to the technology transfer process, will the gap between knowledge generation and knowledge application be closed in a reasonable time frame.

ACKNOWLEDGEMENTS

The work reported in this paper is the result of several years of continuous effort by many people at the National Center for Earthquake Engineering Research headquartered at the University at Buffalo, New York. In particular the authors wish to recognize the contributions of the late Dr. Robert Ketter, the first Director of the Center, and Dr. M. Shinozuka, the present Director.

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DAMAGE ASSESSMENT OF EXISTING BRIDGE STRUCTURES WITH SYSTEM IDENTIFICATION

A. H-S. Ang and W-J. Kim
University of California
Irvine, California 92717 USA

ABSTRACT A method for estimating the damage of existing bridge structures is developed using results of system identification. For damage assessment, structural properties must include the nonlinear parameters, which may be evaluated through system identification. Dynamic behavior of damaged structures is represented by a nonlinear hysteretic moment model. To incorporate the variability of the structural properties and the effects of stochastic excitations, response statistics are obtained through random vibration and damage is represented as random quantities. A numerical example is illustrated for a bridge structure subjected to earthquake excitations.

INTRODUCTION

Bridge structures deteriorate with age under varying loadings and environmental conditions. Consequently, the degree of deterioration and thus the precise determination of the existing structural properties and corresponding load-carrying capacity are difficult and subject to considerable uncertainty. This determination, however, is important and necessary in the assessment of structural resistance of an existing bridge structure to natural hazards, such as earthquakes.

A method for bridge damage assessment is proposed consisting of the following:

-- a technique for identifying the existing structural properties, including the parameters defining the nonlinear behavior;

-- a method for assessing the damage of a bridge structure subjected to specified earthquake loading, taking into account the uncertainties in the loading and structural parameters.

The method is illustrated for a bridge that was damaged during the 1971 San Fernando earthquake.

risk, we must find vehicles by which knowledge generators and knowledge users are communicating. Earthquakes can affect large urban areas in developed or developing countries. Some of the largest urban regions of the world are in such countries with relatively high seismic risk. Government planners, economic decision makers do not have appropriate tools to help them understand the extent and relevance of seismic risk. There are ways in which they could manage that risk if they were properly informed. The knowledge base on which they currently make their decisions is not consistent with the state of knowledge currently existing in research (and to a lesser degree in practicing engineering) community. This lack of integration must change.

Another problem that exists is the problem of technology transfer between the developed and the developing regions of the world. Massive reliable data bases about earthquake occurrences, source modeling, source mechanism, instrumental recordings, and attenuation, exists in few central locations around the world. A global and regional network of such centers could help many developing countries who cannot afford to maintain their own data bases. The challenge is not only to develop new models for source mechanism or new attenuation or new occurrence model; the most pressing challenge is to assist the world in utilizing the current know how. The challenge is to develop IRAS type systems that any informed user can utilize for improving the quality of seismic know how. Such an effort would provide opportunities to assess earthquake risks in regions of the world where historically many have died in past earthquakes. Such an awareness and technology would help in identifying factors that increase or decrease seismic vulnerability of communities or regions.

The International Decade for Natural Disaster Reduction (IDNDR) could be an excellent vehicle to achieve the above stated objectives. Amongst the ten most important programs that the EERI Committee on IDNDR has identified as candidate projects for the decade, at least four projects address the issues of international cooperation, technology transfer and reduction of seismic risk through intelligent development of mitigation strategies, and through integrated risk management procedures.

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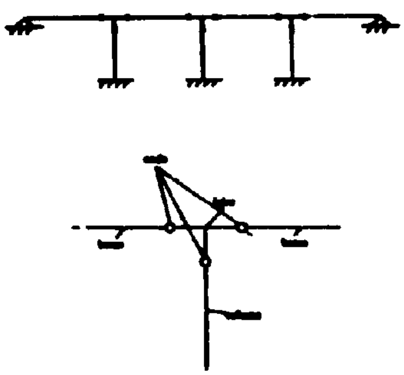

Fig. 1. Bridge model for damage analysis

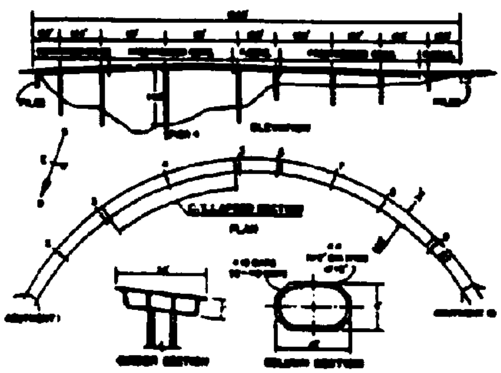

Fig. 2 Description of highway 5/14 overcrossing

Response Analysis

To calculate the damages of the nodes, the means and variances of the respective maximum curvatures and dissipated hysteretic energies are needed. For this purpose, the following are required: a restoring force model for bridge structures, a ground motion model, and a method for response analysis.

Nonlinear restoring force model -- The equations of motion including the nonlinear restoring forces can be written as

$$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + \{F_{NL}\} = -[M]\{J\}\ddot{x}_g \tag{5}$$

where, [M] = mass matrix; [K] = stiffness matrix; {J} = direction vector; and $\ddot{x}_g$ = ground acceleration. The nonlinear force vector, $\{F_{NL}\}$, is defined from the moments at the nodes to incorporate the damages and given as,

$$\{F_{NL}\} = [K]\{X\} + (1.0-\alpha)[T][K_{el}](\{z\}-\{\phi\}) \tag{6}$$

where, [T] is a matrix transforming moments in local coordinates to forces in the global coordinates, and $[K_{el}]$ is the element stiffness matrix. Observe that α = 1.0 in Eq.6 indicates linear restoring force and that nonlinear forces are calculated from the locations where damage hinges (nodes) are expected to occur. The curvature at each node, $\{\phi\}$, can be obtained from the modal displacement vector, and the hysteretic component, z, can be described as

$$\dot{z} = \frac{A\dot{\phi} - \nu\{\beta|\dot{\phi}||z|^{n-1}z + \gamma\dot{\phi}|z|^{n}\}}{\eta} \tag{7}$$

where, α, β, γ and n are constants related to the hysteretic restoring force characteristics; A, ν and η are parameters related to degradation and are functions of the dissipated energy (Suss and others, 1985).

In this study, the state vector approach is used to solve the equations of motion and mode superposition is adopted to reduce the number of variables. In such a case, the equations of motion are transformed to,

$$\{\ddot{W}\} = [-2\xi\omega]\{\dot{W}\} - [\omega^2]\{W\} - (1-\alpha)\frac{[\Phi^T][T]}{[\Phi]^T[M][\Phi]}[K_{el}](\{z\}-\{\phi\}) - \{\Gamma\}\ddot{x}_g \tag{8}$$

where, [Γ] is the modal participation vector; ξ is damping ratio and ω is the natural frequency of the structure.

Ground motion model -- Earthquake ground motion is modeled as a zero mean filtered Gaussian shot noise with a Kanai-Tajimi spectrum. To model the non-stationarity in the ground motion, the intensity is modulated by the Amin-Ang type envelope function (Amin and Ang, 1968).

Random vibration analysis -- Using equivalent linearization (Baker and Wen, 1981), the underlying nonlinear-hysteretic random vibration problem is reduced (Kim and Ang, 1992) to the solution of the following stochastic differential equation:

$$\frac{d}{dt}S = GS + SG^T + B \tag{9}$$

where: $S = E[y(t)y(t)^T]$ is the covariance of the response,

$B_{ij} = 0$ except $B_{22} = I(t)$

$I(t)$ = intensity function of excitation, and

$$G = \begin{bmatrix} 0 & 1 & 0 & 0 & 0 \\ -\omega_g^2 & -2\xi_g\omega_g & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 \\ \omega_g^2 & 2\xi_g\omega_g & -\alpha\frac{K}{m} & -\frac{c}{m} & -(1-\alpha)\frac{K}{m} \\ 0 & 0 & 0 & C_{eq} & K_{eq} \end{bmatrix}$$

Also,

$$\frac{d}{dt}y = Gy + F \tag{10}$$

in which,

$$\begin{aligned} y &= \{x_g, \dot{x}_g, w, \dot{w}, z\}^T \\ F &= \{0, \ddot{x}_g, 0, 0, 0\} \end{aligned} \tag{11}$$

and, x_g, $\dot{x}_g$ and $\ddot{x}_g$ are displacement, velocity and acceleration of the soil, respectively; w and $\dot{w}$ are modal displacement and velocity of the structure.

Evaluation of response statistics -- The mean and variance of the maximum curvature can be obtained assuming that the nonstationary peak has a Weibull distribution (Yang and Liu, 1981). The mean hysteretic energy is obtained by solving Eq. 9, whereas its variance requires the solution of another differential equation (Pires, et al, 1983). Approximation can also be obtained (Kwok and Ang, 1987) showing that the C.O.V. is fairly constant around 0.2.

Global Damage Determination

The global damage is necessarily a function of the damages of the nodes. In general, however, this functional relationship may be expressed only in terms of probability. Specifically, the event that the global damage is greater than damage level d can be defined as

$$(D_T > d) = \bigcup(D_i > d) \tag{12}$$

where, D_T is the global damage of the structure; and D_i is the damage index at node i, in which $\cup$ stands for the union of events.

The probability of the global damage exceeding the damage level d is then expressed as

$$P(D_T > d) = P[\bigcup(D_i > d)] \tag{13}$$

The required probability may be evaluated using the geometric mean of its second-order bounds, which can be written as,

$$P(E_1) + \sum_{i=2}^{k} max[\{P(E_i) - \sum_{j=1}^{i-1} P(E_iE_j)\};0] \le P(D_T > d)$$
$$P(D_T > d) \le \sum_{i=1}^{k} P(E_i) - \sum_{i=2}^{k} \max_{j<i} P(E_iE_j) \tag{14}$$

where,

$$P(E_i) = P(D_i > d) \tag{15}$$

in which i denotes the ith node where damage hinges are expected to occur.

To calculate $P(E_i)$, the performance function is defined considering the maximum curvature and hysteretic energy at node i. Using the damage model defined in Eqs. 1 and 3, the performance function can be written as

$$g_i(X) = D_i - d \tag{16}$$

where D_i is the damage at node i as defined by Eq. 3, and d is a given damage ratio relative to the ultimate damage capacity of the corresponding node.

The safety index for each node is calculated on the basis of Eq. 16 and the correlation coefficients between nodes are also obtained from which the probabilities of the global damage can be obtained through Eq. 14. Assuming log-normal distribution for the global damage, the two parameters λ and ζ of the log-normal distribution can be evaluated using any two probability levels obtained above. On the basis of the derived lognormal distribution, the median and logarithmic standard deviation of the global damage can then be evaluated.

IDENTIFICATION OF EXISTING STRUCTURAL PARAMETERS

In order to assess the damage or potential damage of an existing structure under a given earthquake clearly requires information on the current properties of the structure. Different methods may be used for identifying these properties, depending on the prior information available and/or nondestructive tests that may be performed. Since damage invariably involves nonlinear structural behavior, the required structural properties must include the parameters that define the nonlinear behavior of the structure. In this light, the identified linear properties may be augmented by the necessary nonlinear properties of related structural elements. However, when the strong-motion earthquake excitation and corresponding structural response records are available, the Kalman filtering (Yun and others, 1989) is an effective method for identifying the linear and nonlinear parameters of a structure.

ILLUSTRATIVE APPLICATION

The damage of Highway 5/14 overcrossing during the 1971 San Fernando earthquake is assessed to illustrate the application of the proposed method. This bridge is a typical highway overcrossing constructed with piers and curve girders. The structure collapsed during the San Fernando earthquake; a 1/30 scale model study was performed (Williams and Godden, 1976) to examine the seismic behavior of this bridge. The dimensions of the actual and model structures are shown in Table 1 and the description of the original actual structure is shown in Fig. 2. Structural properties of the model structure are obtained through extended Kalman filtering (Yun and others, 1989) using the measured time histories, and the results are converted to those of the actual structure using appropriate scaling relations suggested in Williams and Godden (1976).

The properties of the actual structure needed for damage estimation were thus obtained as summarized in Table 2.

TABLE 1: DESCRIPTION OF MODEL AND REAL STRUCTURE

	Real Structure	Model
Total Length	636 ft	254.5 in
Radius of Curvature	270 ft	108 in
Column Height	90 ft	36 in
Deck Section	30 ft x 7 ft	8.5 in x 2.5 in
Column Section	10 ft x 5 ft	4 in x 2 in

TABLE 2: STRUCTURAL AND SITE PARAMETERS OF HIGHWAY 5/14 OVERCROSSING

α	β	γ	ξ	ω(Hz)	ξ_g	ω_g(Hz)
0.05	6.09×10^6	-2.03×10^6	0.10	0.88	0.8	2.63

Damages are calculated from the expected maximum curvatures and dissipated energies at the nodal locations shown in Fig. 3; the results are summarized in Table 3. The global damage of the structure is determined as a function of the damages at the nodes according to Eq. 13, and the results are summarized in Table 4.

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TABLE 3: CALCULATED MEAN DAMAGE INDEX FOR EACH NODE

node \ peak accel.	1/6g	1/3g	1/2g	2/3g	5/6g	1g
1	0.0107	0.0428	0.0963	0.1714	0.2680	0.3856
2	0.0361	0.1446	0.3257	0.5794	0.9059	1.3032
3	0.0090	0.0361	0.0812	0.1445	0.2259	0.3249
4	0.0050	0.0200	0.0449	0.0800	0.1250	0.1798
5	0.0074	0.0298	0.0671	0.1193	0.1866	0.2684

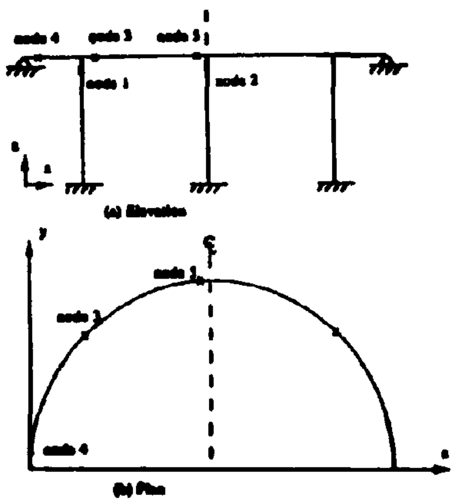

Fig. 3 Analytical model of highway 5/14 overcrossing

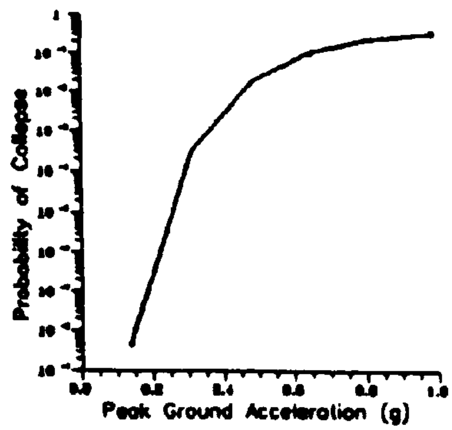

Fig. 4 Fragility curve of structure

The coefficient of variation of the global damage is fairly constant at a value of approximately 0.62. This structure collapsed at a ground excitation of around 0.87g during the 1971 San Fernando earthquake. Collapse appears to be primarily caused by the concentration of damage at the top of the center pier (node 2 in Fig. 3). In accordance with this observation, the structure can be considered to collapse at approximately a global damage index of 0.8 and the probability of exceeding different damage level d can be obtained. From these cumulative probability functions, the median and C.O.V. of the global damage are calculated for different earthquake intensities; the results are shown in Table 4. Assuming that the probability of collapse is the probability of exceeding damage level 0.8, the resulting fragility curve for this bridge would be as shown in Fig. 4.

TABLE 4: GLOBAL DAMAGE STATISTICS UNDER DIFFERENT INTENSITIES

	1/6g	1/3g	1/2g	2/3g	5/6g	1g
Median Global Damage	0.0307	0.122	0.277	0.493	0.770	1.11
C.O.V.	0.620	0.620	0.620	0.620	0.621	0.621

CONCLUSIONS

A quantitative method is presented for damage assessment of existing bridge structures using results of system identification to determine the current structural properties. As structural damage invariably involves nonlinear behavior of a structure, the parameters defining the nonlinear and hysteretic characteristics of its load-deformation relationship are also required. The Kalman filtering technique of system identification has been shown to be effective for this purpose. The proposed method is illustrated for seismic damage of a reinforced concrete bridge that was severely damaged during the 1971 San Fernando earthquake.

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THE USE OF NATURAL RUBBER BEARINGS TO PROTECT A SMALL APARTMENT BLOCK FROM EARTHQUAKE DAMAGE

K. Muniandy[1], K.N.G. Fuller[2] and H.R. Sidjabat[3]
1. Rubber Institute of Malaysia, Kuala Lumpur
2. Malaysian Rubber Producers Research Association
Hertford, England
3. Research Institute of Human Settlements
Bandung, Indonesia

ABSTRACT Seismic isolation is a novel technique of earthquake protection which involves mounting the building or structure on laminated rubber-steel bearings. The horizontal stiffness of the bearings is designed to give the mounted structure a horizontal natural frequency of about 0.5 Hz. This is below the frequency range in which most of the energy of earthquakes for rock and stiff soil sites is typically concentrated. The building is thus detuned from the ground motion occurring during an earthquake, and the accelerations it experiences are much reduced. Furthermore, the mounted structure will behave predominantly as a rigid body with little amplification of the base acceleration at other levels. Seismic isolation is superior to conventional methods of strengthening because not only is damage to the primary structure minimised, but secondary structural features, building contents and occupants are protected.

The technical and economic feasibility of applying seismic isolation to a small building in a country such as Indonesia is currently being assessed by means of a project involving the construction of a small four-storey building on a site near Pelabuanratu in S.W. Java. The area has a reasonable degree of seismic activity thus providing the possibility of a direct assessment of the technical performance of the building. The paper gives details of the building and outlines the principles of

seismic isolation and how the design of the isolation system is approached. As well as technical aspects, economic factors and appropriateness of this technology for countries such as Indonesia are discussed.

INTRODUCTION

The traditional method of protecting buildings and other large structures from earthquake damage is by strengthening, with no attempt to reduce the forces entering the structure. Although this approach can prevent collapse of a building, it provides no protection to the occupants or contents, both of which will generally experience amplified accelerations from the earthquake shock.

An alternative method of protection is to isolate the building from the ground disturbance (Derham and others, 1977). The method has been termed 'base-isolation' or more specifically 'seismic isolation' and it has slowly gained widespread acceptance among structural engineers. There are several such buildings in the United States and Japan. A number of these have now experienced earthquakes and their response has confirmed the expectation that a base-isolated building will perform better than a conventional building in moderate and strong earthquakes (California Dept. of Conservation, 1985).

Seismic isolation involves supporting a structure on flexible bearings that allow horizontal movement during earthquake shaking. The support system is designed to make the natural frequency of the mounted structure below the predominant frequency content (2-5 Hz for rock or stiff soil conditions) of the earthquake. The frequency mismatch results in the isolation of the mounted structure from the earthquake motion, and instead it responds by oscillating as a rigid body at its natural frequency. Because this frequency is low the acceleration imposed on the building is small. Furthermore response as a rigid body results in the acceleration being constant throughout the building, and hence the elimination of the amplification of the accelerations that occur in the higher parts of structures built with conventional rigid foundations (Derham, 1983).

The need to develop effective, but relatively low-cost earthquake resistant construction techniques in developing countries is of paramount importance. Much of the damage and the associated loss of lives occurs to smaller structures such as private dwellings, schools and offices. The use of laminated, high damping rubber bearings appears to offer the best approach to the provision of seismic protection to such small structures, particularly in developing countries. The main purpose of the current project is to establish the technical and economic feasibility of this by the construction of a demonstration building in Indonesia.

LOCATION OF DEMONSTRATION BUILDING

Over 10% of the world's earthquakes occur in Indonesia, many of these along the Sumatra-Java-Timor area which represents the boundary between the Indian-Australian and Pacific plates (Bentley and others, 1979). Given that some areas experience significant level of seismicity, Western Java, with its relative ease of access from Jakarta appeared to be a suitable region for the site of the demonstration building. A government-owned crop estate in one of the earthquake prone areas expressed willingness to donate a site. This had to satisfy requirements as regards location, topography, geology, seismicity, access and suitable existing services. After consideration of these, a preliminary geotechnical assessment suggested a site on the Pasir Badak crop estate, very near the town of Pelabuanratu, as suitable for the demonstration building.

A borehole was drilled at the site down to 68 metres to establish the soil depth and underlying rock structure. The results indicate a sand and gravel layer of only 0.5 metres, and below this moderate to high strength rock. Strength test results and the rock description given in the borehole log suggest that if the building foundations were all set into this relatively fresh rock then the earthquake vibrations would be unaltered by surface soils or weathered rock. This finding will be incorporated into design spectra for the site currently being produced by Beca Carter Hollings and Ferner Ltd.

DEMONSTRATION BUILDING DESIGN AND STRUCTURE

The type of building chosen for the project is a low-cost apartment block of four storeys. The minimum appropriate size is influenced by two considerations: the cost of the isolation system should not add a significant proportion to the overall cost, and designing low-cost isolation bearings to support small loads is difficult.

The general method of construction in Indonesia for buildings of two to five storeys utilizes a reinforced concrete frame. This technique, with masonry infill, was chosen as the most appropriate for the demonstration building, a overall impression of which is shown in Figure 1. The total ground plan area is 19.8 x 9 metres, and the height is 12.8 metres, excluding the roof. The fundamental natural period of the building is 0.4s. It is estimated that its total weight is approximately 600 tonnes (dead load) plus 120 tonnes (lives load). The building and the ground outside will be instrumented so that the performance of the isolation system can be assessed should any earthquake occur.

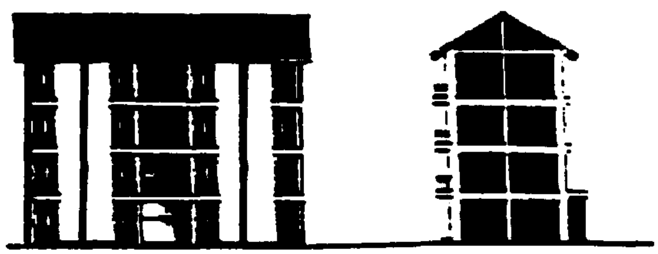

Figure 1. Front and side elevations of building.

The superstructure of the building has been designed to satisfy the seismic loading requirements in the current Indonesian Earthquake Code without taking the provision of base-isolation into account. Because isolation will lower the seismic forces, the structural frame need not be as strong as designed. A slightly weaker frame could produce savings to off-set the additional costs associated with the isolation system.

DESIGN METHODOLOGY OF SEISMIC ISOLATION BEARINGS

The simplest way, from a structural point of view, of incorporating the isolation system into the demonstration building is to locate one rubber bearing under each load bearing column. This method will require 16 isolation bearings for the building as indicated in Figure 2. The vertical load supported by each bearing ranges between 22 and 60 tonf.

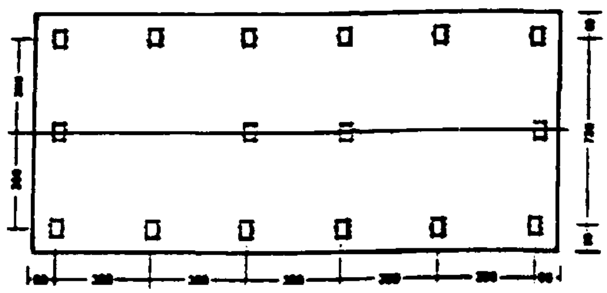

Figure 2. Ground-plan of building indicating the position of load bearing columns.

Earthquakes are simply ground oscillations of very large amplitude and rather low frequency, the predominant mode of excitation generally being horizontal. The isolation bearings are designed so that the horizontal natural frequency of the mounted structure is decreased to a value where the magnitude of the induced earthquake excitations are small.

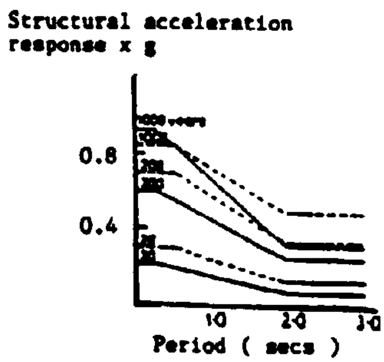

Figure 3. Structural response spectra (5% damping) for various return periods around Pasir Badak region. (—— stuctures on hard ground ---- on soft ground.)

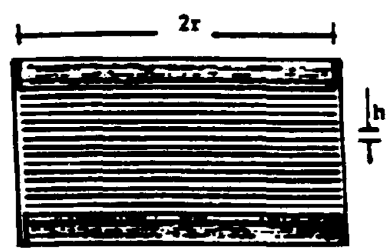

Figure 4. Sketch of seismic bearing. Thickness of internal metal plates is 1mm. For 60mm design displacement, 2r = 266mm and h = 6.3mm. For 210mm design displacement, 2r = 350mm and h = 10.8mm. Supported mass 30000kg.

As the design spectrum for the site is not yet available the final design of the seismic bearings is not possible. Nevertheless, some preliminary design of the bearings can be undertaken using the simplified design spectra for the Pelabuanratu area available in the Indonesian code for earthquake resistant building design (Bentley and others, 1979). Such spectra for varying earthquake return periods and 5% critical damping are shown in Figure 3. Assuming a structure with a natural period of 2 seconds, the maximum displacement, estimated from the acceleration, is about 71 mm for a 20 year return period and 243 mm for a 200 year return period at 5% damping. At 10% damping, the typical level for a high damping natural rubber system, it is estimated that the displacement would be 61 mm and 207 mm respectively.

Bearing Design

The bearings need to give a horizontal natural oscillation period T of the order of two seconds. The required horizontal stiffness of the bearing k_s can be calculated using

$$k_s = (4\pi^2 m/T^2) \tag{1}$$

where m is the total mass supported by the bearing. From the horizontal stiffness, the total height of rubber in each bearing, assumed a cylinder of radius r, may be calculated from

$$nh = \pi r^2 G/k_s \tag{2}$$

where G is the shear modulus of the rubber at a strain corresponding to the maximum design displacement and n the number of rubber layers each of thickness h. The total height of rubber must be large enough to keep the shear strain imposed on the rubber at the maximum horizontal bearing displacement less than the permitted maximum (typically 100-200%). The radius of the bearing has to be sufficient to prevent roll-out (if the bearing is not bolted to the structure) or cavitation within the rubber (Thomas, 1983) at the maximum horizontal displacement, and to support the vertical load without imposing too high a strain on the rubber. Another important design consideration of the bearing is its stability. The load P_c at which a bearing in compression becomes unstable can be expressed (Thomas, 1983) as

$$\frac{P_c}{mg} = \frac{8\pi^4}{\sqrt{2}} \cdot \frac{mn}{GgT^4} \tag{3}$$

where m is the mass supported by the bearing and g the acceleration due to gravity. (P_c/mg) is the safety factor, generally taken as 3. The minimum number of rubber layers may well be determined by this criterion.

The compressive stiffness k_c of each bearing is given by

$$k_c = 3G\pi r^2 (1 + 2S^2)/nh \qquad (4)$$

where S is the shape factor of the rubber layer equal to (r/2h). The stiffness k_c is required to be large enough to ensure that rocking of the isolated structure is minimized and that the vertical earthquake components are not amplified by the isolation bearings.

Using 0.5 MPa for G, bearings supporting 30 tonf were designed for maximum horizontal displacements of 60 mm and 210 mm. The dimensions of these are indicated in Figure 4 above.

Choice of Rubber Compound

Because of its strength properties and resistance to stiffening at low temperature, natural rubber is generally preferred in laminated bearings. It has a low degree of inherent damping, however. Nevertheless, natural rubber-based compounds with sufficient damping and non-linearity have been developed that make the provision of auxillary damping and wind-restraint devices for the isolation system unnecessary (Derham and others, 1985). Such an approach offers advantages of simplicity, cost benefits and long term reliability without continuous maintenance.

DISCUSSION AND CONCLUSION

It is apparent that costs are regarded as quite critical; the provision of superior earthquake protection should not involve a significant increase in costs, otherwise the innovation may not gain acceptance. Because of the lower strength requirements of the base-isolated building, preliminary study suggests possible savings of US$15,000 could be made on the superstructure for the demonstration building. This constitutes a significant but partial offset for the cost of the bearings and the necessary modifications to the structure and the service connections.

Base-isolation by means of rubber bearings is very appropriate to those tropical Asian countries like Indonesia, located in an earthquake-prone region. Both the raw material and a similar bearing technology exists. Indonesia is one of the largest producers of natural rubber in the world, and this is the most suitable elastomer for seismic bearings. Their method of manufacture is quite similar to that used in bridge bearings for which sufficient expertise and resources already exist for local production. With some extra care and control of materials and production techniques, it would be within the capability of local manufacturers to produce quality seismic bearings. The use of a local material and technology would have added cost benefits.

In conclusion, it is hoped that the construction of the demonstration building in Indonesia will help establish the advantages and feasibility of base-isolation as a method of earthquake protection and encourage its adoption by structural engineers in Asia.

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ZONING METHOD : ONE OF TECHNOLOGY TO LIVE WITH GEOLOGIC HAZARD AREA

Djoko Santoso
Department of Geology
Bandung Institute of Technology and IDMC
Indonesia

ABSTRACT Tectonically, Indonesia is situated in the active area. It is called an island arc, as indicated by an arcuate shape of the island chains, trenches, active volcanoes and earthquake activity and the chain of islands. On the other hand some part of the eastern Indonesia region is more complicated. Consequently, the area will be vulnerable to some hazard such as earthquake, volcanoes and landslide. The third one is also caused by thick soil due to strong weathering processes, and heavy rain in the tropical region.

Zoning is technology which divide an area in some region according to the hazard level. Earthquake zoning map will divide the area based on possible damage or ground motion criteria, volcanic zoning map will divide the region based on the hazard level due to volcanic products from the crater of volcano. Landslide zoning map will delineate the area according to possible landslide hazard. Indonesia is the fourth country in the numbers of population in the world. Because of this, in some places people have to live in the geologic hazard area. Population condition force the people to live in these hazards areas. Therefore the zoning map should be used as much as possible in any landuse planning of the area. Even in the danger area, if there is some advantage the region could still be utilized with the same boundary condition.

INTRODUCTION

Indonesia is located in the area called island arc. It contain of arcuate form of island chains with deep sea trenches in the ocean side (Uyeda 1978). The "active" island arc is an anomalous area in the earth having the major characteristics as accurate continuation of islands, prominent volcanic activity at present, deep trench on the oceanic side and shallow tray shaped seas on the continental side, gravity isostatic anomaly, seismic activity (shallow intermediate and deep), earth movement in progress, and coincidence of arc with recent orogenic belts (Sugimura and Uyeda, 1973).

The population of Indonesia was 179 million people in 1979, with the growing rate of 2 % yearly. The density are 200-1000 people/sq km in Java Island and Bali, and 20-200 people in Sumatra, Sulawesi and Nusa Tenggara Island. Other areas are less than the above. This condition force inhabitant to live in or to utilize the hazardous area. In order to minimize the loss of life or economic losses some technology should be adapted. Zoning methods is one of technology that could be used and to be utilized in the area with special geological condition as in Indonesia.

ZONING MAPS

A. Seismic Zoning Map

The seismic zoning map is a map which is related to some earthquakes parameters such as magnitude intensity, acceleration, active faults, probability occurrence and so on. Housner and Jennings (1974) described seismic zoning map for engineering purpose as a map that specifies the level of force or motion for earthquake resistance design, thus it differs from seismicity map that provides information only on occurrence of earthquakes. Srivastava (1974) defined seismic zoning map to delineate areas of probable maximum intensity of earthquake and to indicate basic acceleration for design. On the other hand Medvedev (1968) roughly described that zone in the seismic zoning maps are delineated zone on the basis of seismic and geological data. Those maps can be distinguished into three groups, seismicity map, faulty map, or seismotectonic map or seismic probability map, and engineering map.

1. <u>Housner and Jennings classification</u>

   a. <u>Seismicity map</u> - the simplest seismicity maps are the plot of magnitude-rated epicenters of the past earthquakes and judgement enter only to the extent of interpreting the pre-instrumental history of the region. In this map, if data are available adequately to define the frequency of occurrence vs magnitude of earthquake over region, the earthquake risk map can be constructed.

Sometimes the seismicity data are also well presented in the term of strain energy density associated with historical earthquakes.

b. Fault map, seismotectonic map and seismic probability map - seismic fault map is designed to show all fault on which movements have taken place within certain specified period of time, e.g. in historic time or in the last 10.000 years.

Seismotectonic map is used to describe maps which are essentially fault map augmented by other geologic information, such as tectonic processes, local geology and so on (Evseev and others, 1968).

A Seismic probability is constructed from a fault map by assigning probabilities of occurrence of earthquake of different magnitudes to each active fault, and assigning areal distribution of intensity to earthquake of different magnitudes.

c. Engineering map - the maps which are discussed above present basic data combined with professional seismological and geological judgement. They are generally not directly useful to engineers, who need a quantitative guidance regarding seismic loads to be resisted within certain allowable stresses, strain and so on. The engineering map vary depending upon the intended applications and on the interest of the individual constructing the map.

2. Karnik & Algermissen classification

Karnik & Algermissen (1978) differentiate several types of zoning maps according to the data and assumption used in their preparation. According to the content they classified the earthquake zoning maps to four categories.

a. Maximum intensity map - this type of map can be either map of maximum observed intensity with smoothed contours of integrated isoseismals or map of maximum expected intensity. Theoretically, this type of map can be developed based on the definition of source region, past earthquake statistics and attenuation functions.

b. Engineering zoning map - some zoning maps are classified according to other quantities. The most common is related to building code. This type of map will be specified as the coefficient according to type of structure and ground condition.

c. Maximum ground measurement map - this type of map show ground motion parameters such as maximum acceleration, particle velocity, or displacement. Actually the basic data for this map is intensity. In several countries this parameters can be recorded by special seismograph. The great majority of instrumental strong motion data have been recorded in California and Japan. Another way of finding this parameters is by using the attenuation model from another earthquake parameters. For any particular earthquake area the attenuation data based on correlation of observed intensity will have more accurate result.

d. Seismic risk map - there are a number of definition for seismic risk map. For instance, in the former USSR a risk includes economic and other effect of earthquakes during long time interval. Donovan (1973) stated that seismic risk should be in the term of return period of intensity or magnitude. Vere-Jones (1973) stated three point as concept of earthquake risk; they are geophysical risk referring to probability of recurrence of damaging earthquake at specific region; the engineering risk referring to the probability of failure of a particular type to structure; and the insurance risk referring to claim being lodged to specific amount. The Working Group on Statistical of Natural Hazard (Paris, 1972) (Karnik & Algermissen 1978) defined seismic risk as the probability of loss, from earthquake and a natural hazard itself is a state of risk due to the possibility of occurrence natural disaster. Karnik & Algermissen (1978) state that risk must be referred to the losses of certain object.

3. Seismic Zoning Map in Indonesia

Several Macro zonation maps has been made by several authors, the oldest one is Earthquake Hazard Map of Indonesia (Brest van Kempan, 19?). This map consist of epicenter distribution, intensity in the term of gravity acceleration and tsunami data. Sutadi (1962) produced a seismic zoning map based on the surface ground acceleration calculated using Kawasumi empirical formula.

Wiratman (1971) delineate similar map by using the same formula. The same map is also presented by National Working Group on Engineering Seismology and Earthquake Engineering (1976). Becca Carter Holling & Ferner Ltd. in 1977 presented the frequency map for certain magnitude.

At present several institutions made zoning map based on their needs. For example the map produced by Directorate of Water Research (1982). This map shows the zone delineate the earthquake coefficient for Water Structure. The data included in this map are based on soil/rock types, acceleration and period. The Directorate of Public Work (1980) published the seismic zoning m in for building structure.

Another maps are shows calculation of b value from Ritcher - Gutenberg formula (Sudarmo, 1977 and Santoso, 1980).

B. Volcanic Zoning Map

The types of volcanic hazards are diverse and vary from one volcano to another. The numbers of casualties also differ widely according to the nature and location of activity. The most significant aspect of volcanic hazards are : the distribution and pattern of different type of processes, the velocity, the temperature, the length of warning, and the frequency of occurrence (Blong, 1984 c.f. Situmorang and Sudrajat, 19?). The potential hazard process is usually related to volcanic product such as flows, outfalls, slide or avalanches, gas emissions, ground deformation and tsunami. Volcanic hazard zoning map is made based on the past and present character of the volcano, the topographic condition, prevail wind direction and so on (Sudrajat, 1991).

Some maps of volcanic zoning in Indonesia have been published by Directorate of Volcanology in Bandung, such as Merapi, Soputan, Semeru, Ciremai, Merapi Lamongan, Mahawu, Ganalama, Karangetan, Sundoro Banda Api, Makian, Awu, Lewotobi, Kerinci, Batur, Sangeyang Api, Papandayan, Krakatau, Kala, Dukono, Ilibulong, Rohatenda, Ebulobo and so on. This map is usually called Hazard Area Map.

C. Landslide Zoning Map

Landslide zoning map is usually made based on the past occurrence of events combined with the geologic and rainfall condition. In Japan as for example, the Japanese Islands are divided into eight geologic units for engineering purpose, based on geologic constitution and the landslide distribution (Japan Society of Landslide, 1980). On the other hand Koide distinguished Japan into three region as tertiary landslide, fracture zone landslide and hot spring landslide (Shinjo Construction Work Office, 1985). Identification of the landslide vulnerable area could be done by identification of steep slope, cliffs or banks being undercut by stream or wave action, area of drainage concentration and seepage zone, area of hummocky ground, and area of fault and fracture concentration (Rib and Liang, 1978). The landslide zoning in Indonesia is made based on the frequency of events, intensity and geological environment (Elifas, 1988).

DISCUSSION

The result of zoning method is a map which shows the vulnerable area to natural disaster, e.g. earthquake, volcanic eruption and landslide. Therefore in the Indonesian region where so many people have to live in the hazard area, zoning map could delineate the hazard level, consequently if this map is used the danger area could still be utilized.

The earthquake danger as a ground motion parameter in a certain zone could be mitigated by a proper seismic coefficient for a certain structure. In the high risk area such as fracture zone or fault zone the landuse planning should be adjusted only for agriculture, forestry or other activities with no settlement or industrial area.

Volcanic area is always a very fertile zone. Traditionally, this region has a high population density. The adjustment of landuse in certain region is necessary. The people should be trained that vulnerable area for volcanic eruption should be used only for agriculture, forestry or other activities of no settlement or industrial zone. In other word they should build their settlement in the area outside the hazard zone or low hazardous zone. They should also be trained not to put their valuable things in the hazard zone.

Landslide is a small scale disaster, although individual slope failure generally are not so spectacular or so costly as certain other natural catasthropes as earthquakes, floods and tornadoes, but total or cumulative financial loss of landslide is probably greater than any other single geologic hazard to mankind (Schuster, 1978).

The effort to manage this kind of natural phenomena could be done by avoiding or mitigating. If there is financial value people could still live in the danger area by providing mitigating effort.

CONCLUSION

Based on the above discussion the conclusion is as follows :

- In the future people would be educated to live with natural disaster.
- Natural Disaster could be managed by avoiding or mitigating the impact.
- Zoning is one technology that could be used in avoiding or mitigating natural hazard, therefore micro zonation of each natural hazard zones should be done.

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SECOND US-ASIA CONFERENCE ON
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EARTHQUAKE PREDICTION AND ENGINEERING SYSTEM FOR SEISMIC DISASTER REDUCTION IN CHINA

Ma Zongjin and Zhao Axing
The Institute of Geology
State Seismological Bureau of China
Beijing, China

ABSTRACT

In this paper, it is outlined that the earthquake hazards, earthquake prediction stages and the countermeasures for its reduction works in China. The future in this aspect, an engineering system for seismic disaster reduction which is composed six major disaster reducing measures: monitoring and survey, forecast, resistance, prevention, rescue and relief, is proposed.

EARTHQUAKE HAZARDS IN CHINA

China is one of the seismically most active country in the continent of the world. Since the beginning of this century, there are more than 1200 earthquakes Ms$>$7 occurred in the world, among them 1/10 took place in China. In the past 40 years, in average 6.6 seismic events per year with Ms $>$6.0 occurred in China, resulting in about 276,000 person dead and 763,000 person injured; the property loss caused by earthquakes, only by incomplete statistics for 11 earthquake events Ms $>$7 since 1949 shows: the buildings with 60 million rooms and about 100 million square meters collapsed, and they worth over 10 billion RMB; other direct loss of installations for industry and agriculture were over 30 billion RMB. Table 1 shows the losses and casualty caused by some disastrous earthquakes.

The seismic activity in the continental regions, in comparison with other places of the world, have such features as: high frequency, large magnitude and extensive distribution. Most domain was suffered from earthquakes with basic seismic intensity of 7 to 8, making up 32.5% of the gross land area. There are 136 cities located at that regions, of which there are 30 cities with a population over 0.5 million, roughly holding 45% of the overall cities in China.

The earthquakes in China mainland are typically intraplate events with complex structural background. Most of these earthquakes occurred within the middle crust, about 10-25 kilometers in depth. Because of shallow focal depths, high density of population and most of buildings built before 1950s' were no anti-seismic structure, so all large earthquakes occurred in the thickly inhabited districts have caused very severe disasters. For example, the Tangshan earthquake with Ms = 7.8, occurred on July 28, 1976, the seismically destructive region were over 30,000 square kilometers, about 242,000 people were killed and 180,000 people severely injured. Tangshan City, a modern city with 1.05 million inhabitants and over 100 years history, based on mining and ceramic industry, was razed.

The seismic hazards have brought about grave losses and tremendous threats to the Chinese people. Over a long period of time, the Chinese have been making unremitting efforts for reducting the seismic disaster and have got some experiences and achievements in this aspect: earthquake prediction, preventing, reliefing and against seismic hazards.

Table 1. Disaster produced by some disastrous earthquake

Date	Location	Magnitude	Casualty	Property loss
1556	Huaxian Shaanxi	8	830,000(dead)	
1668	Tancheng Shandong	8.5	100,000	
1679	Sanhe-Pinggu Hebei	8	100,000	
1920	Haiyuan Ningxia	8.6	230,000(dead)	
1966	Xingtai Hebei	7.2	8,000(dead)	
1970	Tonghai Yunnan	7.8	18,000(dead) 27,000(injured)	
1975	Haicheng Liaoning	7.3	1,300(dead) 4,300(injured)	800 million RMB of direct loss 450 million RMB for rebuilding
1976	Tangshan Hebei	7.8	240,000(dead) 180,000(injured)	9.6 billion RMB of direct loss 600 million RMB for relief 2.5 billion RMB for rebuilding

THE EARTHQUAKE PREDICTION AND ITS ROLE IN MITIGATING SEISMIC HAZARDS IN CHINA

Earthquake prediction involves estimating or forecasting the time, place, magnitude of a destructive earthquake and the damage or hazard with the losses of property caused by it. Table 2 gives some schemes about stage division of earthquake prediction . By comparing these schemes and considering present knowledge level about preparatory process of earthquake, state of data, and convenince for working program and international exchange, we think the sixth scheme in table 2 is better. Earthquake prediction in China is divided into long-term (years to several decades or even longer), intermediate-term (months to years), short-term (days to months) and imminent (within several days) predictions.

The opinions determining the preparatory period of several decades before earthquake as the long-term are coincident. The research objects and methods of long-term prediction all depend on recurrence of strong quakes, palaeoseismicity and tectonic comparison of strong quakes, revealing the long-term regularity of strong quakes in a point, a belt and a region. The research results are used for earthquake zoning and evaluating of seismic risk.

The opinions determining the period from several months to occurrence of strong earthquake as the short-term are also coincident, for the abrupt anomalies and rapidly-changing anomalies of several geophysical and geochemical fields before strong earthquake, in features and time scale, are very similar between themselves and to some experimental and theoretical results. Some experiences tell us that the more imminent the occurrence of strong earthquake is, the more abundant the abrupt anomalies are, and the higher the ratio of different

Table 2. Comparison of schemes about stage division of earthquake prediction

time \ author \ data	1 earth deformation: Mecharikov, J. A. 1968	1 earth deformation: Fujii Yoichiro 1974	2 examples of quakes: Ma Zongjin 1972	3 experiment: Scholz, C. H. 1973	3 theory: Mjachking, V 1975	4 example of quakes: Rikitake, T 1976	5 antiseismic request: Mumrpe, T 1978	6 summarization: Ma Zongjin 1986
-100 years	α deformation slow deformation		Super long-term seismic period	I-strain slow accumulation	I-strain uniform rupture	long-term strain accumulation (decades)	long-term 10 years	long-term
-decades	β deformation 20—25 years	β_1 obvious deformation	long-term seismic episode (20 ± years)	II mainly dilated	II Microrupture avalanche	intermediate-term dilatation 6—7 years	intermediate-term 6—10 years	intermediate-term early
-several years	4—5 years	β_2 unstable deformation	intermediate-term slow precursor					late
-several months		β_3 slip one year — tens minute	short-term rapid precursor	III mainly soaked	III concentration of crack	short-term 3—1 years	short-term 6 months — several days	short-term
-several days before		γ_1 dislocation	imminent sudden precursor	IV rapid deformation		imminent chief rupture pre-developed	subshort-term	
shock	γ sudden deformation	γ_2 dislocation	dislocation	V dislocation				shock time
after -several days -several months -several years		γ_3 continuous slip inverse recovery	aftershock-dislocation adjustment	aftershock adjustment	IV chief rupture unloading			strong aftershock adjustment

macroscopic anomalies including unusual behavior of animals and climatic anomalies. People all expect to distinguish an imminent-term prediction several days before strong quake from the short-term stage, but several peaks of abrupt anomalies often appear before a strong earthquake. Thus, in most cases, it is difficult to determine definity the coming of imminent stage.

Nevertheless, the earthquake prediction, as one of the forward position in geoscience, is still a scientific puzzle currently in the world. Compared with developed countries, there still is a distance in the observing technique, instruments, data processing and communication apparatus. Nevertheless among all the methods and measures for reducing seismic hazards, earthquake prediction (including intermediate-term, short-term and imminent predictions, and post-quake tendency diagnostic judgments as well) is the basis. In addition, there are many scientific and technological difficulties in making accurate predictions. Furthermore, earthquake prediction is practically affected by social, economic, psychological, political, legal and religious factors. Although earthquake prediction is a critical approach in mitigating seismic hazards, it still has some limitations. So only organically combining and integrating various measures for seismic hazard reduction together, can the optimal efficiency be attained in reducing seismic hazards.

THE EARTHQUAKE COUNTERMEASURES AND ENGINEERING SYSTEM FOR SEISMIC HAZARDS REDUCTION

The action of natural disaster reduction is not only the exclusive action of science and technology, but also the popular action of whole society. For the purpose of natural disaster reduction, it is necessary that the whole international society would work, cooperate and connect together with this aim. Based on the above mentioned, this paper suggests six major disaster reducing measures: monitoring and survey, forecast, resistance, prevention, rescue and relief, for the centralized state engineering system of seismic disaster reduction in China.

1. The monitoring and survey of seismic disaster

The monitoring and survey of natural disaster serve are the foundation of natural disaster reduction. By the monitoring and survey, data and information of disaster can be provided. Therefore, the hazards could be warned and forecasted. Even the emergent commence actions for disaster prevention and reduction can be taken directly. At present, the nets of monitoring and survey for the seismic disaster have been set up in China. In general these nets are composed of three scales: the statewide synthesis station, the regional monitoring and survey station and the local monitoring station, as well as wide spreaded unprofessional observatories.

At the present, as the result of efforts over 20 years, the earthquake monitoring and prediction research systems have been set up in major seismic areas of China. The observations and researches of earthquake precursors have been developed. The nation-wide network consists of Beijing, Shanghai, Chengdu, Shenyang, Kunming and Lanzhou 6 regional telemetry networks, 12 local radio telemetry networks and 9 Sino-USA cooperative digital seismographic stations. There are 970 professional seismic stations and observatories distributed over China, of which 379 stations run by local governments and enterprises. Every year, roving measurements of the gravity, geomagnetism and crustal deformation profiles are carried out, with the measuring routes as long as 20,000 kilometers and up to 4,000 observatories points. Except for the professional, there are a widespread unprofessional seismic observatories. So a relatively complete, nation wide monitoring network has been built.

2. The forecast of earthquake hazard

Earthquake prediction involves estimating or forecasting the time, place and magnitude of a destructive earthquake and the damage or hazard with the losses of property caused by it. Earthquake prediction in China is divided into long-term (years to tens of years or even longer), intermediate-term (months to years), short-term (days to months) and imminent (within several days) stages. The large scale studies for the earthquake prediction, after the Xingtai earthquake event,were began in 1966. Since the establishment of The State Seismological Bureau of China in 1971 (abbreviated to SSB), a unified leadership and management has been proceeded over the national seismological work, The stipulation of earthquake prediction release ratified by the State Council have made a rule for the contents and sequences to release the earthquake prediction. The earthquake prediction, must be proposed by the local seismological branch, approved by the local province government and at the same time reported to the State Council and have to be released to society by the local government at the right moment.

Over the unremitting probe in the past 20 years, the research of earthquake prediction have got great achievement in China. The successful prediction of the Haicheng earthquake of M = 7.3 has elucidated that at present, it is possible to make to a certain extent and even successful prediction of some large earthquakes before their occurrence through meticulous and serious effort. However, the earthquake prediction is an extremely complicated scientific puzzle, with the Tangshan earthquake and other ones failed to be predicted. This fact indicates that we are now still basically under an empirical stage of earthquake prediction, the physical mechanism of seismogenic processes and intricate precursory phenomena are not yet well understood. The empirical knowledge is very often challenged by various problems arising in the course of earthquake prediction practices. The most salient problem encountered is the nonuniquenness of the relation between the precursory anomalies and the earthquake occurrences.

Disaster forecast, including the regime for the disaster and the preassessment for the hazards, is the scientific base for various disaster reduction preparation. At present, although there are some empirical and theoretical bases for earthquakes forecast, the overall level of forecast is still relatively low. The successful prediction level could hardly be risen unless the capacity, contents and analysis methods of monitoring and survey are greatly improved. It is specially worth to emphasize the great earthquake which will be occurred in heavy populated areas.

3. The resistance of seismic disaster

The resistance and prevention of the seismic hazards have underwent a tortuous road in China. In the 1950s', with the limited of knowledge of seismology and the state financial support, it was clearly stipulated that the buildings should not be designed in anti-seismic structure in the areas where the basic intensity is lower than 8. In the areas with the basic intensity over 9 the buildings would be cut down in height and adjusted in plane to improve the building properties to reach the aim of seismic disaster reduction. So, except some very important buildings, there was no capability of anti-seismic property for most buildings. Undergoing Xingtai earthquake (Ms = 7.2, 7,938 persons dead, 8,613 persons injured and 1,200,000 houses collapsed) and Tonghai earthquake (Ms = 7.8, 18,000 person dead, 27,000 person injured and 338,000 houses collapsed), the seismic hazards impelled us to study the anti-seismic measures for buildings and installations and to search for optimal anti-seismic design in engineering and earthquake resistant measures in seismic risk region. The Chinese Government, soon after that, promulgated the stipulation requiring the design of the buildings and installations to make "structures would not collapse during large earthquakes and not be damaged during small ones". The works on engineering anti-seismic designing and earthquake resistant measures impelled the resistance and prevention of the seismic hazards and have got great achievements in China.

By the case of China, the Chinese Government decided that the key of the resistance and prevention for the seismic hazard reduction is the urban one and pointed at 38 cities as the focal points of it. Since 1979, the important buildings, installations and engineering for the earthquake resistance and prevention of the seismic hazard reduction have been built according to the standard of basic intensity 7 in the urban area, where the basic intensity is 6. Since 1986, it was stipulated that the area with the basic intensity 6, making up about 27% of the domain, should be optionally reinforced and anti-seismically defended.

All of the above works have greatly improved the capability of the resistance and prevention for the seismic hazards and got some achievements. For example, after adopting the above measures, an earthquake with Ms = 6 in 1981 occurred in the same place of 1966 Xingtai earthquake, no houses was collapsed and no person died or injured. In the same cases, the buildings reinforced in Haichen earthquake (1975), Daofu earthquake (1981), Heze earthquake (1983) have got obvious benefits and the seismic hazards were reduced. It is obvious that the capability of resistance and prevention for the seismic hazards will raise with the increase of the state finance and the investment.

4. The prevention of seismic disaster

Disaster prevention includes two aspects, one is that sufficient attention should be paid to disaster prevention in planning constructions and selecting their sites in order to avoid potential disaster, another is that shielding type measures for disaster reduction should be taken for mobile factors including personnel, machinery, equipments, etc. For the former, considerations based on standards and specifications have already been given in planning large national projects. For the latter, it is related to the knowledge and techniques of disaster prevention for popularization. When the disaster prevention consciousness of the whole people has been risen, it would be a social measure of great potential for disaster reduction. The propagation of natural disaster knowledge has been strengthen in order to improve the awareness of disaster prevention among the whole Chinese nation. Such extensive propagation would be greatly helpful for the public to raise the self-preparation and self- defense abilities, enable them to cooperate with the government in front of disaster and take positive attitude towards mitigating hazards.

5. The rescue of seismic disaster

Since the beginning of history, compared with other countries, the seismic hazards in China are very grievous both in casualties and property losses. For instance, up to the end of 1988, there are about 335 events with the casualties over 10,000 persons caused by earthquakes in the world, in which 88 events occurred in China and the casualties reached up to 40% of the total; especially 7 earthquakes occurred in the world with casualties over 200,000, among them 4 events in China, which is 57% of the total. So that the Chinese Government has to take great pains and mobilize the whole aspects to deal with various social and economical problems brought by the seismic hazards. Therefore the earthquake relief is not only arduous but also strenuous.

In the light of specific conditions of China, the fundamental policy of earthquake relief by Chinese Government are: depend upon the masses and collective efforts, self-saving and help each other and with the necessary relief and support by the state government. The relief experiences of over the years proved that the fundamental policy is very effective. According to this policy, the Chinese Government derived the historical lessons and experiences, with multiple measures and methods to prepare for preventing and reducing seismic hazards of pre-quake stage; for the co-earthquake and post-quake the urgent task is to rescue the victims and injured, to return the normal social life, rebuild the seismic hazard region and eliminate the seismic aftermaths.

6. The relief of seismic hazards

The relief of seismic hazards, including the recovery of production and social life is also a very effective measure for disaster reduction. Once a serious disaster occurred, the urban construction and public wealth would be damaged, the industrial production and financial activities would be stopped, and even the structure of society and family would be destroyed and thus induce large derivative losses. Therefore, to shorten the time for the recovery of production and the reconstruction of home is an important measure for disaster reduction. The Chinese Government reserves 1 billion RMB for the relief of natural hazards every year; except that, once the heavy hazards occurred, it will be supported by the social insurance, domestic help and international support as well.

The basic policy for the seismic hazard relief in China are: to put the prevention first, to maintain organization and command under the leadership of the administrative regions and to utilize the army, the militias and the professional relief teams. Although great progress and some experiences have been got in the rescue, relief works and rebuilding the lifeline engineering after earthquake, in preventing the occurrence and enlargement of the secondary seismic disaster. Because of the limited property storage, the social weakness in disaster preventing, and the imperfect disaster legislation in all of the Chinese society, it is more complex and arduous for the relief of seismic hazard in China.

In the past 40 years, according to the incomplete statistics, there were over 10 million people, over 10 billion RMB of the state financial support involved in the seismic relief, rescue and rebuilding in China. The Chinese Government has basically accomplished: to complete the urgent tasks of rescure the life and injured within a couple of days after the earthquake, to reduce the casualties and property loss to the minimum and to make a preliminary arrangement for the livielihood of victims in seismic calamity; to take about 3 years to relief and rebuild the seismic calamity areas upon general destructive earthquake regions. For great earthquake, such as Tangshan earthquake, until 1986, the city's economic production level just return to the prequake level and the Chinese Government took 10 years to relief and rebuild the urban installations, this work will take a long time.

According to the above mentioned, the six measurements can not be dispensed in the system engineering for the seismic disaster reduction. They are essential for the aim of the International Decade for Natural Disaster Reduction.

Seismic disaster reduction must be a system engineering and must rely on a unified information system. The overall disaster reduction countermeasures should be worked out beforehand on the basis of an overall understanding of the disaster. Meanwhile, the system should provide scientific information extensively to various departments and branches of science, coordinate the disaster reduction work in various departments, rise the research levels of seismic disasters synchronously, promote and accelerate the heightening of the disaster reduction efficacy.

We are confident that the loss caused by natural disasters will be minimized by the whole international society would work, cooperate and connect together with this aim.

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FLOOD HAZARD

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TO IMPROVE HEAVY RAINFALL FORECAST : TAIWAN AREA MESOSCALE EXPERIMENT (TAMEX)

George Tai-Jen Chen
Department of Atmospheric Science
National Taiwan University
Taipei, Taiwan ROC

ABSTRACT TAMEX is a research program to improve, through better understanding, the forecasting of heavy precipitation events that lead to flash floods. The program was proposed to the National Science Council and meteorological community in Taiwan in 1983. After about 4 years planning, the field phase was carried out successfully in May ~ June 1987. A 5-year follow-up research plan to cover both the basic and applied researches started right after the field phase in 1988. The Post-TAMEX Forecast Exercise, be carried out in May ~ June 1992, is planned to complete this 10-year TAMEX program. The objective of this Forecast Exercise is to apply the scientific results and forecast techniques generated by the TAMEX program and to develop nowcasting (0~3 h) and very-short-range forecasting (3~24 h) capabilities in the heavy rainfall forecast in Mei-Yu season. An overview of this 10-year program is given in this paper.

INTRODUCTION

A transition period between the winter NE monsoon and summer SW monsoon regimes occurs over subtropical East Asia in the late spring and early summer. During this transition period, a front (Mei-Yu front in Taiwan and China, Baiu front in Japan) tends to form in the deformation wind field between a migratory high to the north and the subtropical Pacific high to the south. Although the individual Mei-Yu front often moves slowly southeastward after its formation, the mean position of the front undergoes sequential northward shift between May and July depending upon the intensity and the position of the large-scale circulations

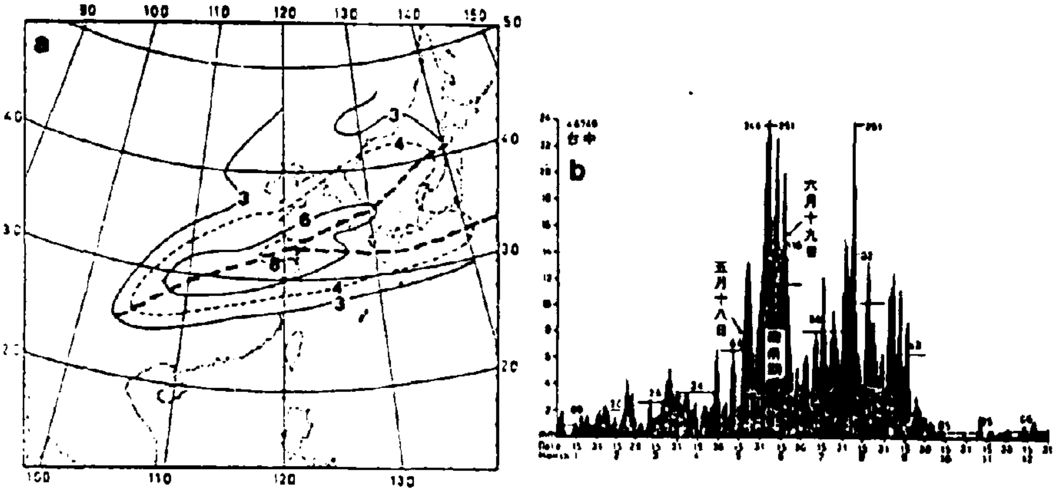


Fig. 1 (a) Annual mean (1975-1986) frequency distribution of 850 hPa front in Taiwan Mei-Yu season (15 May - 15 June). Front frequency is counted at 12 h intervals and analyzed at 1° lat × 1° long grid intervals. Heavy dashed line indicates maximum axis. (b) Climatological daily rainfall (mm) at Taichung in 1956 - 1975 and the monthly mean daily rainfall (mm) in 1951-1970. The Mei-Yu season is indicated.

in the subtropical latitudes. Fig. 1a shows the annual mean frequency distribution of 850 hPa front in the Mei-Yu season of Taiwan (mid-May to mid-June). The axis of maximum frequency, indicating the mean position of the Mei-Yu front, is oriented approximately in an east-west direction extending from southern Japan to southern China. The seasonal rainfall distribution in Taiwan reaches a maximum during the Mei-Yu season primarily due to the repeated occurrence of the Mei-Yu front. The mean daily rainfall at Taichung, which is located in central Taiwan, is presented in Fig. 1b to show this feature.

Fig. 2a shows the smoothed topography of Taiwan and the mean rainfall distribution during the 1972-77 Mei-Yu season. The Central Mountain Range (CMR) runs through Taiwan in a north-south direction with an average terrain height of about 2000 m and a peak of 4000 m. The topographic influence is clearly shown by the much higher values of rainfall on the windward slopes of the CMR than on the lee slopes under the prevailing southwesterlies in the lower troposphere. Among the many mesoscale features observed near the front, the most important are the organized mesoscale convective systems (MCSs). These convective systems tend to move along the front from west to east. As they move across the Taiwan Strait, they are often affected by the steep orography of Taiwan and produce locally heavy rainfall of up to a few hundred millimeters per day. Fig. 2b shows the spatial distribution of heavy rainfall events in the Mei-Yu season of May and June. The pattern reflects the influence of the CMR and local topography. The property damage caused by heavy rainfall and the associated flash flooding in the Mei-Yu season

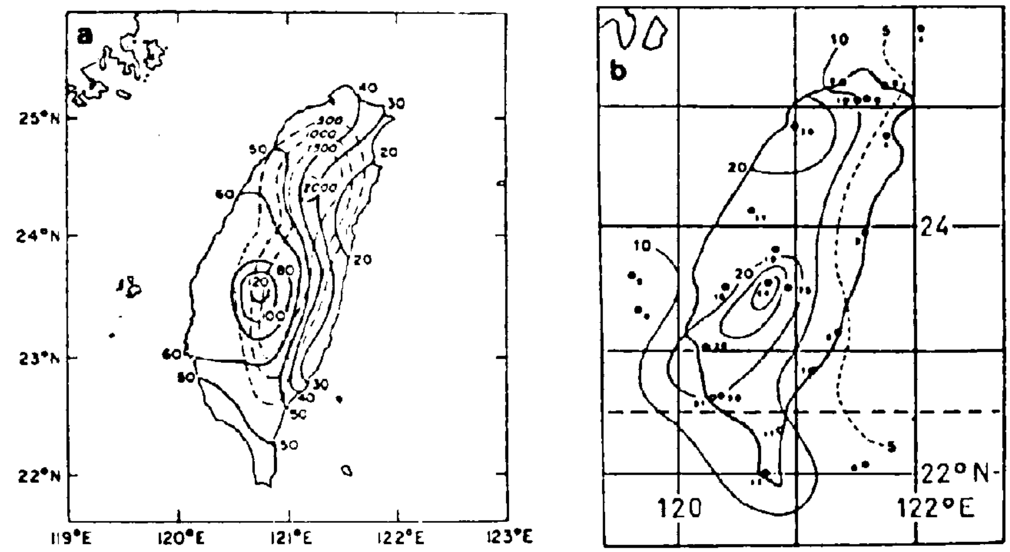


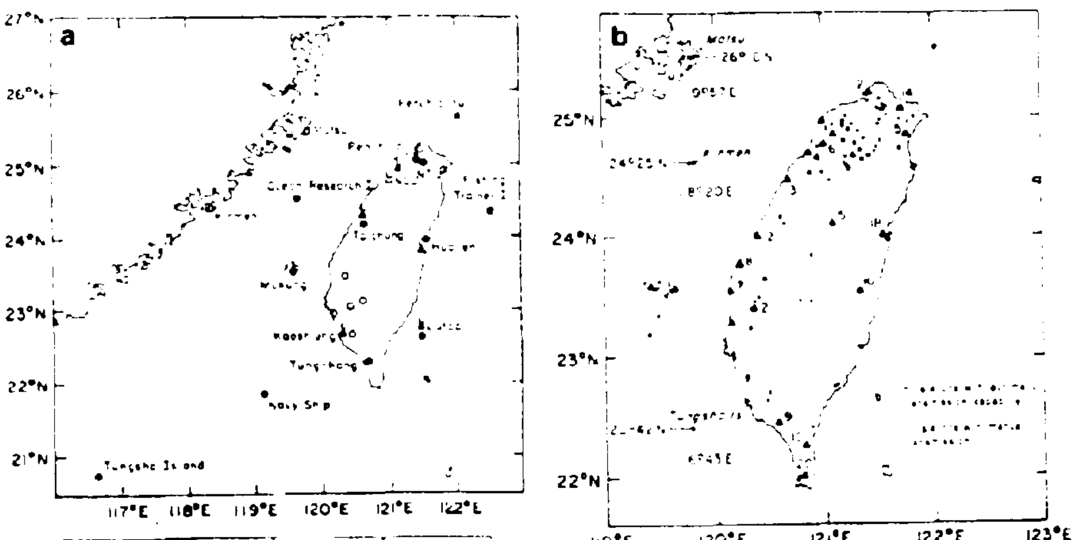
Fig. 2 (a) The Mei-Yu rainfall (solid, cm) in May 15-June 18, 1972-1977 and smoothed topography (dashed, m). (b) Distribution of the 326 cases of heavy rainfall events in May-June, 1975-1984.

becomes much more serious in recent years due to the rapid economy growth in Taiwan. Each of the heavy rainfall/flash flood events, such as the May 28 case of 1981, June 3 case and June 10 case of 1984, caused US $400-600 millions in damage. With the awareness that the prediction of flash floods is greatly hampered by a lack of understanding of the mesoscale processes responsible for producing heavy rain, the National Science Council (NSC) of the Republic of China (ROC) in Taiwan established a mesoscale meteorological research program --- the Taiwan Area Mesoscale Experiment (TAMEX).

EXPERIMENT DESIGN

TAMEX is a research program conducted jointly by scientists of Taiwan, the ROC, and the United States to improve, through better understanding, the forecasting of heavy precipitation events that lead to flash floods. In order to achieve this objective the field phase was launched to collect the data necessary to study 1) the mesoscale circulation associated with the Mei-Yu front; 2) the evolution of the mesoscale convective systems in the vicinity of the Mei-Yu front; 3) the effects of orography on the Mei-Yu front and on mesoscale convective systems. The observational program of TAMEX consisted of five components: an upper-air network, a surface network, a radar network, an aircraft program, and a satellite program.

The upper-air network was composed of conventional and special rawinsonde sites and pilot balloon stations. Nine of the 12 rawinsonde stations were land-based systems and three were located on ships, covering an area of ~ 500 km X 500 km contered over Taiwan (Fig. 3a). The surface network consisted of 75 surface stations (Fig. 3b), 126 raingauges (Fig. 3c), 21 wind towers, and 3 shipboard stations. The radar network consisted of five conventional radars and three C-band Doppler radars. Periods during which high-resolution observations were made were referred to as "intensive observing periods" (IOPs). From 11 May through 20 June 1987, the NOAA P-3 aircraft flew ten missions in support of eight TAMEX IOPs. The aircraft was based in Okinawa, Japan, though all flights were carried out in the TAMEX area. The meteorological satellites in operation during TAMEX were the GMS-3, NOAA-9, and N[illegible]A-



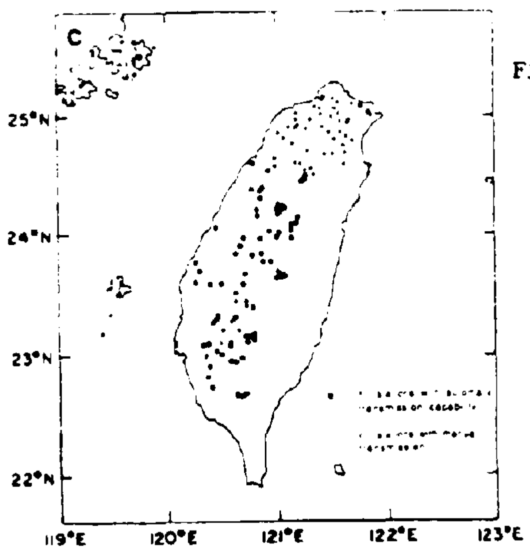


Fig. 3 (a) The locations of the TAMEX rawinsonde (solid circles) and pilot balloon (open circles) network. The locations of conventional radar stations are marked by the radar symbols. The VHF wind profiler is indicated by an open triangle. (b) The locations of the TAMEX surface observation network. Solid circles are stations with automatic real - time transmission capability , while open circles are stations with manual transmission only. The triangles with numbers indicate the wind tower stations. (c) The locations of the TAMEX raingauge stations. Solid circles are stations with automatic real-time transmission capability, while open circles are stations with manual transmission only.

10. During IOPs, routine observational systems increased their frequency of observations. "Fixed" special observing systems, such as Doppler radars, were operated in selected modes. Mobile observating systems, such as aircraft and ships, were deployed into areas of special interest. In order to provide continuous measurements to study mesoscale circulations during undisturbed, as well as disturbed periods, soundings were taken continuously at 6-h intervals during a special observing period (SOP) from 15 May to 15 June 1987.

FIELD PROGRAM

The field phase of TAMEX extended from May 1 to June 29, 1987 covering thirteen IOPs and ten P-3 aircraft flight missions. The participants in the field phase included the ROC and the US components. The ROC component consisted of 80 scientists and 1000 research associates, students, and technicians from 4 universities and 11 government agencies. The US component consisted of 50 scientists in Taiwan and 25 scientists, research assoicates, and students in Okinawa from 11 universities, NSF, NCAR, NOAA and NRL. The operation cost was estimated to be about US $ 5 million, consisting of 4 million from the ROC side and 1 million from the US side.

The TAMEX field program was an operational success, and quailty datasets were collected in support of all TAMEX scientific objectives. The 13 IOPs extended over 23 days of the 2-month operation. The P-3 aircraft flew a total of 82 h, three vessels participated a total of 92 days, and about 2000 soundings were launched, including 1200 launches at 6-h intervals, and 800 launches at 3-h intervals. In addition, nearly continuous data were collected at 3 ground-based Doppler radar sites, 75 surface stations, the VHF wind profiler, and 21 micrometeorological towers. A total of 739 h of ground-based Doppler-radar data were recorded. An excellent data set was available for studying various scientific problems relevant to heavy rainfall events. Mesoscale meteorological phenomena on which the TAMEX field observations were collected include: the Mei-Yu front, low-level jet (LLJ), pre-frontal squall lines, open-ocean MCSs, mountain convection, terrain-induced mesoscale circulation, frontal deformation due to topography, and land-sea breeze. An overview paper of TAMEX field program was presented by Kuo and Chen (1990).

FOLLOW-UP RESEARCH PROGRAM

A follow-up research program was established right after the TAMEX field phase to cover 5 year (1988-1992) ROC-US collaboration research activities. At the NSC, the TAMEX basic and applied researches were supported by the Division of National Science and the Division of Planning, respectively. The financial supports for the applied research and related activities were also provided by the Central Weather Bureau, Ministry of Transportation and Communications, and Ministry of Education.

To provide a forum for scientific exchanges of TAMEX research results, the annual TAMEX scientific wrokshop was held in Taipei in June 1989 and at NCAR in February 1988 and September 1990. In addition, the International Conference on Mesoscale Meteorology and TAMEX, to serve as a forum for the disposition and discussion on all aspects of mesoscale meteorological research and the scientific results of TAMEX, was held in Taipei in December 1991.

More than 200 papers were presented at the TAMEX and other Workshops/Conferences, and more than 30 TAMEX papers have been published in the refereed journals including the TAMEX Special Issue of Monthly Weather Review/American Meteorological Society in November 1991. Some of the mesoscale research results in the pre-TAMEX era as well as those derived from TAMEX program are presented by Chan (1992). It is believed that through the combined theoretical, modeling, and observational approaches as demonstrated in TAMEX research, these scientific results will contribute directly and indirectly toward improving heavy rainfall forecast not only in the Taiwan area but also in other part of the would. To apply the scientific results and forecast techniques generated by the TAMEX program and to develop nowcasting (0 - 3 h) and very short range forecasting (3 - 24 h) capabilities in the heavy rainfall forecast in the Mei-Yu season, a Post-TAMEX Forecast Exercise is scheduled for the period of 1 May - 30 June 1992.

POST-TAMEX FORECAST EXERCISE

As an effort to put the research results into operational use, the NSC has sponsored a TAMEX Forecast Exercise in May and June 1992. Ten working groups have been established to develop new forecasting techniques and new conceptual models based on the scientific results of TAMEX. The goals of this experiment are : (1) to introduce new forecasting concepts into the operational forecast systems and to reduce weather hazards by utilizing new nowcasting systems and new forecasting methods, and (2) to establish a baseline for future forecasting improvements (such as new forecast target areas and new forecasting parameters).

Fig. 4 shows the operational flow chart for the Forecast Exercise. The routine operation period (ROP) and intensive operation period (IOP) are defined in terms of the observation and / or expectation of heavy rainfall event over Taiwan area. Besides the heavy rainfall forecast, the quantitative percipitation forecast (QPF) will be made at 3-h intervals over 6 regions as shown in Fig. 5a in ROP (0-24 h) and IOP-Alert stage (0-12 h). In the IOP-Warning stage, 0-3 h QPF will be made at 1-h intervals over 15 counties as shown in Fig. 5b. There are 5 experimental forecast groups (EFG) on the shift for the two-month Forecast Exercise from May 1 to June 30, 1992. Each EFG consists of a workstation expert, a lead forecaster, and 2 assistant forecasters. The Forecast Exercise will be operated at the Central Weather Bureau using the newly installed Weather Integration and Nowcasting System (WINS). Besides, more than 40 scientists from various research and operational communities of ROC, USA, Canada, and South Africa will participate in the Exercise.

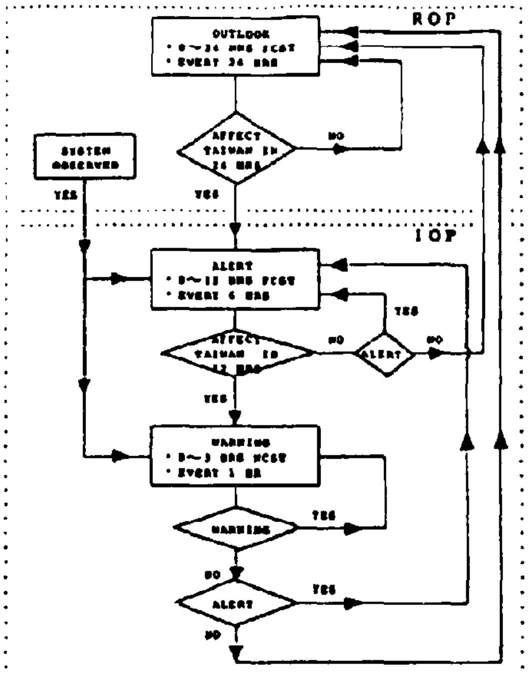


Fig. 4 The operational flow chart for Post-TAMEX Forecast Exercise in 1 May - 30 June 1992.

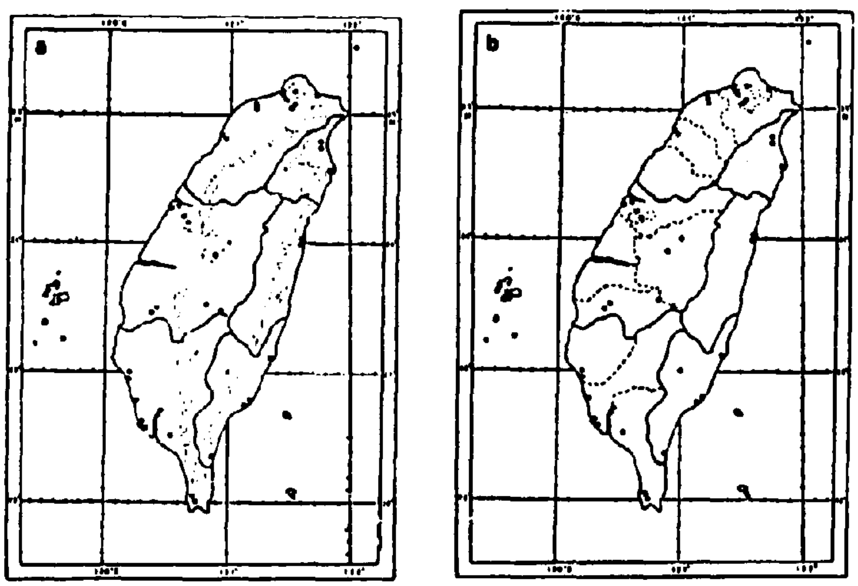


Fig. 5 (a) Six regions for QPF and heavy rainfall forecast in ROP and IOP-Alert stage. (b) Fifteen counties for QPF and heavy rainfall forecast in IOP-Warning stage.

CONCLUDING REMARKS

TAMEX is a research program to improve, through better understanding, the forecasting of heavy rainfall events that lead to flash floods. The field program of TAMEX was an operational success and an excellent data set was collected for studying various scientific problems relevant to heavy rainfall events. The follow-up research program of TAMEX, again, was a success with fruitful scientific results which will contribute directly and indirectly toward improving heavy rainfall forecast. The end of the ten-year TAMEX program will be reached by 1992. The National Science Council plans to conduct a meeting in 1993 to review the scientific accomplishments of TAMEX and to assess the effectiveness of the 1992 TAMEX Forecast Exercise. This review may serve as a starting point for the planning of a future field program.

ACKNOWLEDGEMENTS

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SECOND US-ASIA CONFERENCE ON
ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
Yogyakarta, Indonesia
22-26 June 1992

AN AUTOMATED RAINFALL AND METEOROLOGICAL TELEMETRY SYSTEM IN TAIWAN AREA

Ching-Yen Tsay
Central Weather Bureau
64 Kung Yuan Road, Taipei, Taiwan 10039

ABSTRACT

Most rivers and streams on Taiwan are short and steep. During the Mei-Yu and typhoon seasons, floods develop rapidly when storm rainfall is concentrated for short time periods. Such heavy rainfall induced by meso-scale and micro-scale convective systems cannot be effectively monitored due to the sparsity of weather stations. To strengthen the regional observation of heavy rainfall, the CWB has been proceeding on a project to establish an automated rainfall and meteorological data collection system in the principle drainage basins in the Taiwan area. The system will cover seven sub-regions spread over the region west of the Central Mountain Range(CMR), and four sub-regions in eastern Taiwan. A total of 318 stations is proposed.

The completion of the system at the end of 1992 will greatly enhance Taiwan's meteorological observing capability. The real-time availability of meteorological data will ensure the protection of western Taiwan from pouring-rainfall and flooding events through increased warning time and better data for improved flood management and control.

1. BACKGROUND

Most rivers and streams on Taiwan are short and steep. There are three rainfall periods in Taiwan, including northeast monsoon period in winter, Mei-Yu period from mid-May to mid-June, and typhoon period from July to September.

Northeast Monsoon Rainfall Period

In winter, the northeast monsoon picks up moisture over Yellow Sea and East China Sea before reaching Central Mountain range of Taiwan. Therefore, the moist northeast monsoon will cause rainfall in northern Taiwan, but not in central and southern part of west Taiwan. Fig. 1 is the rainfall distribution in February. It is seen that the winter northeast monsoon period is a rainfall period in the north, but not in the central and the south to the west of the Central Mountain Range. In this rainfall period in northern Taiwan, meso-scale heavy rainfall cases may occur occasionally. Fig. 2 is a distribution of daily rainfall of an example case.

Typhoon Rainfall Period

In summer, typhoons are formed to the south of the Pacific high and steered by the easterlies. A large portion of typhoon tracks passes the area of Taiwan (Fig. 3). Typhoon rainfall period is one of the three major rainfall periods in Taiwan. There are about three typhoons, on the average which pass the vicinity of Taiwan each year. Because of the orientation of the Central Mountain Range, special rainfall distribution occurs in Taiwan. Typhoon tracks in the vicinity of Taiwan may be classfied into six categories (Fig. 4). The distributions of averaged rainfall for each category are shown in Figs. 5 to 10.

Fig. 5 shows the distribution of the averaged precipitation for the first category. The precipitation is located mainly to the west side of the Central Mountain Range.

Fig. 6 shows the precipitation distributions for the medium to strong type (a) and weak type typhoon (b) of the second category. For the medium to strong type, there are three maximum centers in the regions of Yi-Lan to Hua-Lien, north side of Yang-Ming mountain, and the Ali mountain area. For the weak type, there are only two maximum centers in the regions of Yi-Lan to Hua-Lien and north side of Yang-Ming mountain.

Fig. 7 shows the pecipitation distribution for the third category. The precipitation as mainly located to the east side of central mountain range.

Fig. 8 shows the distribution for the fourth category. This category is subdivided into type a (occurred in August or earlier) and type b (occurred in September or later). For type a, the precipitation is mainly located in the eastern Taiwan from Hua-Lien to Heng-Chueng. For type b, the increases of percipitation in eastern and northern tip of Taiwan were possibly enhanced by the northeast monsoon which prevails in late fall throughout the winter.

Fig. 9 shows the fifth category for type a (occurred in August or earlier) and type b (September or later). For type a, the precipitation is mainly located to the east of the Central Mountain Range and to the south of Hua-Lien. For type b, the fact that more rainfalls were observed over norhtern Taiwan than for type a was again possibly due to the presence of the northeast monsoon.

Fig. 10 shows the sixth category for type a (occurred in August or earlier) and type b (September or later). For type a, the precipitation is mainly located to the east of the Central Mountain Range, especially to the south of Hua-Lien, in the region of Kao-Shiung and Ping-Tung. For type b, northeast monsoon may again play an important role to enhance the precipitation in the region of Yi-Lan and to the north of Yang-Ming mountain.

Mei-Yu Rainfall Period

As time goes from winter to summer, the Mongolian high weakens and the Pacific high develops. From mid-May to mid-June, the stationary front between these two high pressure systems stays in the vicinity of Taiwan and causes heavy rainfall in Taiwan. It is called Mei-Yu rainfall period in Taiwan. In the stationary type of large-scale Mei-Yu rainfall system, meso-scale convective systems are always found. An example of heavy meso-scale rainfall is shown in Fig. 11. As a result of frequent torrential rainfall, flash floods occurred almost every Mei-Yu period in recent years.

During the Mei-Yu and typhoon seasons, floods develop rapidly when storm rainfall is concentrated for short time periods. Such heavy rainfall induced by meso-scale and micro-scale convective systems cannot be effectively monitored due to the sparsity of weather stations. To strengthen the regional observation of heavy rainfall, the CWB has been proceeding on a project to establish an automated rainfall and meteorological data collection system in the principle drainage basins in the Taiwan area. The system will cover seven sub-regions spread over the region west of the Central Mountain

Range(CMR), and four sub-regions in eastern Taiwan. A total of 318 stations is proposed.

2. PURPOSE

Upon the completion of the project, the intensive automated rainfall and meteorological data collection system well be able to add greatly to the observing data source, and to monitor localized heavy rainfall. In addition, the data collected can be used to forecast flash floods and to issue timely warnings to the public. Better forecasts for the water discharges of reservoirs are also expected.

3. SCOPE

The number of stations of this system in west of the CMR will reach 248. Figure 1 illustrates the overall design of the network.

The installations of the system began in July, 1986. Up to May 1992, five sub-regions of 160 stations will be completed. They cover the drainage basins of Tam-shui, Cheng-wen, Ta-an, Wu, Tseng-wen and Ta-chia Rivers, and the Tao-chu, and Miao-li regions. (Fig. 12). These installations have produced the expected results. Real-time precipitation data (Fig. 13) are shared with water conservancy, reservoir, and electricity management groups. These data are used for the operation of reservoirs, regulation of water supply, and prediction of flash floods. Before September 1992, two sub-regions of 35 stations, spread over the central region west of the CMR, will be installed. They cover the drainage basins of the Chio-shui Rivers, and Chan-hua region. In March 1993, another 33 stations will be established in the Chia-nan region. By fiscal year 1997, 90 stations will be deployed in four sub-regions in eastern Taiwan covering the drainage basins of the Lan-yan, Hua-lien, Shiu-ku-luan, and Pei-nan Rivers.

In this system, approximately 78 percent of the stations record rainfall; 22 percent of the record wind speed, wind direction, air temperature, and sunshine duration. Statistical methods have been utilized to determine the number of stations needed in each area under the requirements of reliability and relative accuracy to achieve maximum economic efficiency. The network density is 60-100 km^2.

4. TECHNICAL DESCRIPTION

4.1 Computer System

The discussion of the computer system is divided into three parts. They are the network architecture data handling software and databases, and user interface software.

4.1.1 Network Architecture

The goal of the computer system design is to make instantly available for both real-time and historic rainfall and meteorologic data. In order to accomplish this goal, a network of minicomputers is utilized. Data from each major drainage basin in Taiwan is routed to a regional data processing station. Small Digital Equipment Corporation Micro VAXII or Micro VAX 3100 minicomputers at each regional data processing station are used to store data locally. The small Micro VAXes are connected by dedicated telephone lines and networking software to a larger central processing station in Taipei. A DEC VAX 8350 is used as a central station computer.

4.1.2 Data Handling Software and Databases

The rainfall monitoring system makes use of licensed VAX/DMS real-time software written by the Sutron Corporation of Herndon, Virginis, USA. The software consists of realtime, interactive, and batch software.

Data from the rainfall monitoring stations enter the computer network through serial data ports on the Micro VAX computers. Radio receiving and demodulating equipment turns the incoming data into a serial data message. Real-time software on each Micro VAX decodes the incoming messages, identifies stations, and stores the data in a daily files database for an easy local access. Every 10 minutes special batch processes transfer the data from the regional data processing centers to the central processing station in Taipei. The central data processing computer automatically updates an archive database.

4.1.3 User Interface Software

At the regional data processing stations, data are made available to users through Chinese Character terminals with printers. Users access the data through menu-driven software which allow selection of current data or historic data. Figures 14 illustrates a typical user menu in English.

At the central processing station, users can also access data through Chinese Character terminals with printers. Map graphics software provides real-time or summary presentations of rainfall and meteorological data. Figure 15. illustrates a typical map with meteorological data superimposed.

4.2 Field Monitoring Stations

Field stations are fully automated. Each station consists of an instrument tower and a telemetry tower. Some stations have instrument tower tipping bucket rain gauges only, some are equipped with a tipping bucket rain gauge, a wind speed and direction monitor, a sunshine sensor, and a temperature sensor. The telemetry tower contains a microprocessor-controlled data collector and transmitter along with lightening protection equipment and the transmitter antenna. Weather data are transmitted hourly. Rainfall data are transmitted each time when the bucket in the rain gage tips. In periods of no rainfall the rainfall, stations report at least once every 6 hours.

5. BENEFITS AND FUTURE EXPANSION

The fully automated monitoring system will benefit many areas of water resources management. The benefits stem from the following new data provided by the system:

- o Near-instantaneous rainfall data island-wide;
- o Hourly meteorological data island wide;
- o Automated, instantly available daily reports;
- o Automated long-term statistics for future planning use;
- o Automated monitoring of large rainfall amounts and high speed of winds;
- o Very short-range weather forecast island-wide;

Specific initial benefits which result from the newly available data include:

- o Improved forecasting because of increased data availability;
- o Improved warnings because of rapid data availability;
- o Easy access to data in both Chinese and English;
- o Minimum personnel required to operate the minicomputer system;

Future benefits will depend on the application of the data to real-time modelling of Taiwan's hydrology. The potential exists for:

- o Real-time rainfall/runoff modelling of Taiwan's major river basins;
- o Integration of water level monitoring to enhance the

ability to model rainfall/runoff hydrology.

- o Improved climatological and hydrological research because of the greatly improved database.

6. CONCLUSION

The initial phases of the automated meteorological data collection system for Taiwan have been in operation for over five years. The integrated system of meteorological instruments, radio communication, and computer hardware and software has been proved to be reliable and effective. The completion of the system at the end of 1992 will greatly enhance Taiwan's meteorological observing capability. The real-time availability of meteorological data will ensure the protection of western Taiwan from pouring-rainfall and flooding events through increased warning time and better data for improved flood management and control.

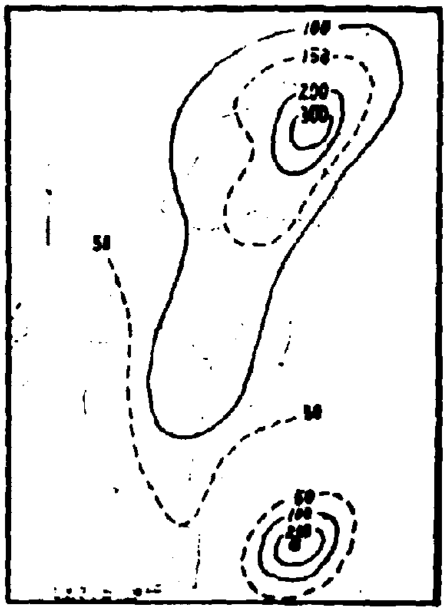


Fig. 1 Distribution of mean rainfall (mm) in February.
(After Chen Lai-Fa, 1984)

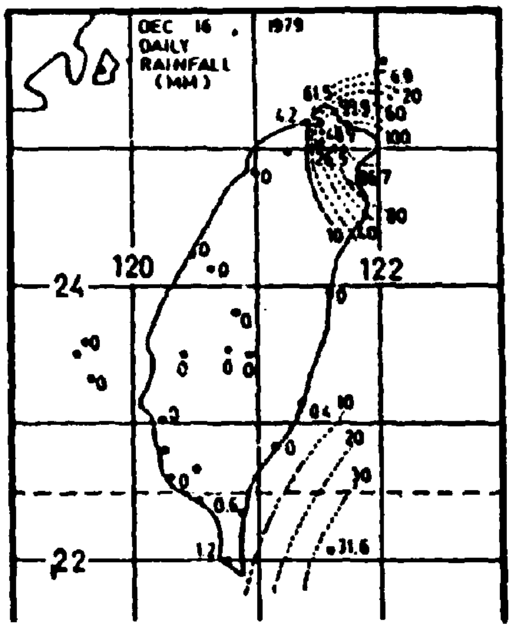


Fig. 2 Daily rainfall case (mm) on 16 Dec. 1979. (After Chen and Tsay, 1984)

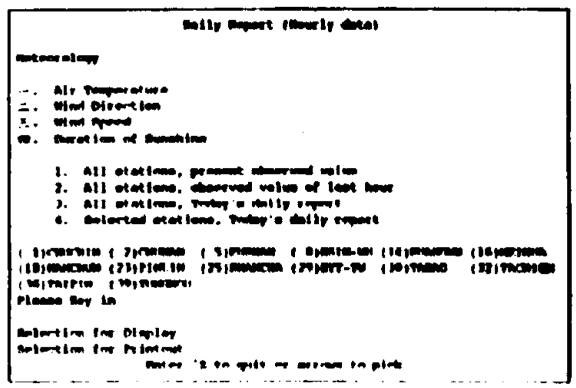


Fig. 15 Typical Operator Menu in English

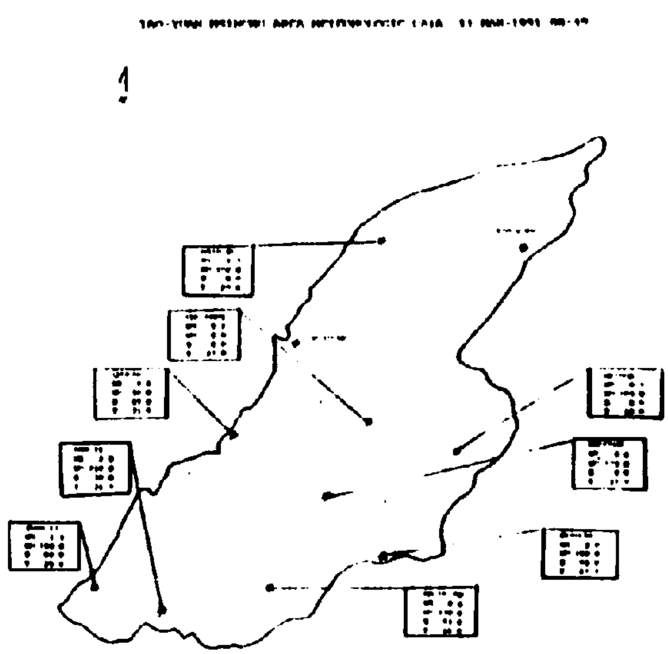


Fig. 16 Meteorological Data Map Display

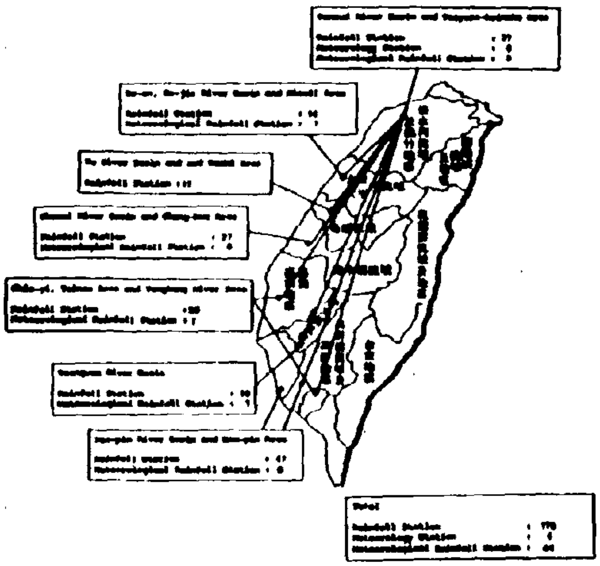


Fig. 12 Computer Network for Rainfall Monitoring System

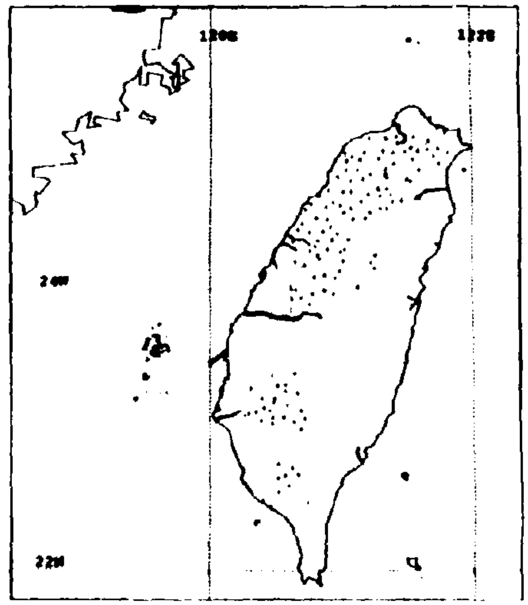


Fig. 13 Location of rainfall station over the drainage of Tam-Shui, Chang-Wen, Ta-Chia Rivers.

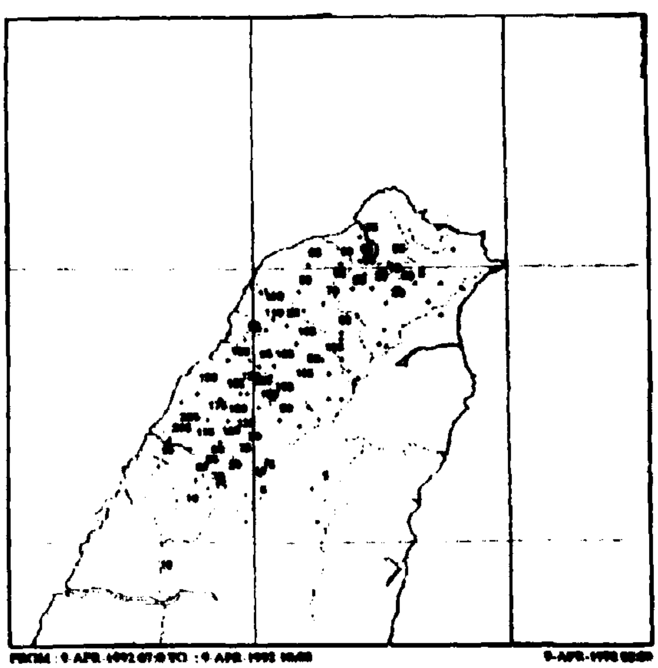
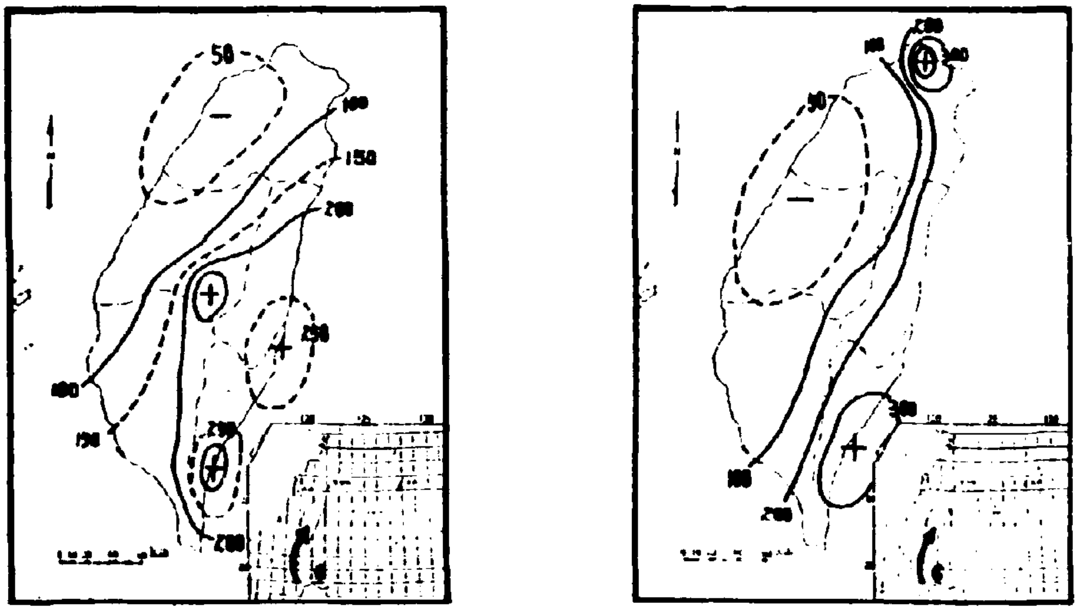
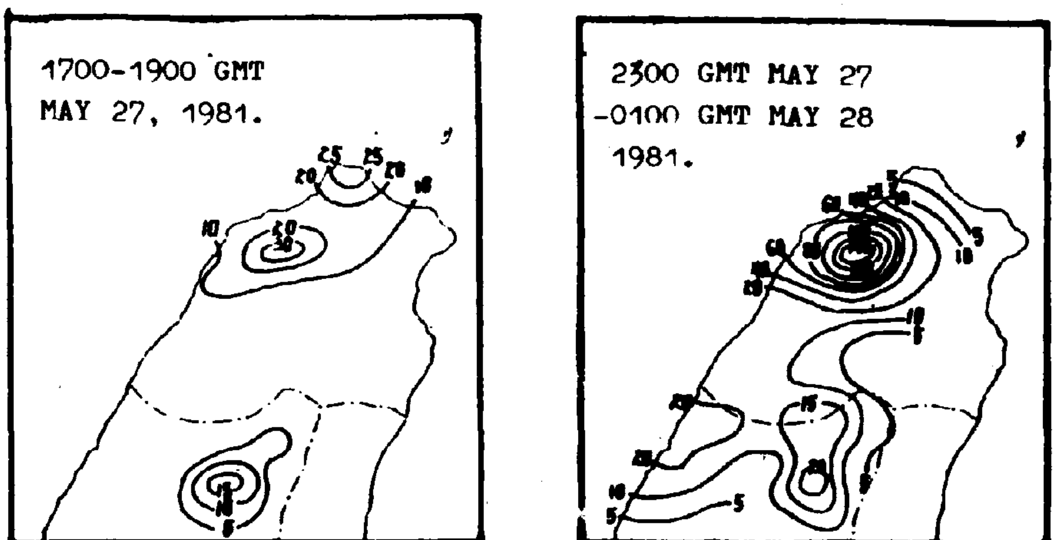


Fig. 14 Example of real-time precipitation data(0.1mm) shown in monitor.



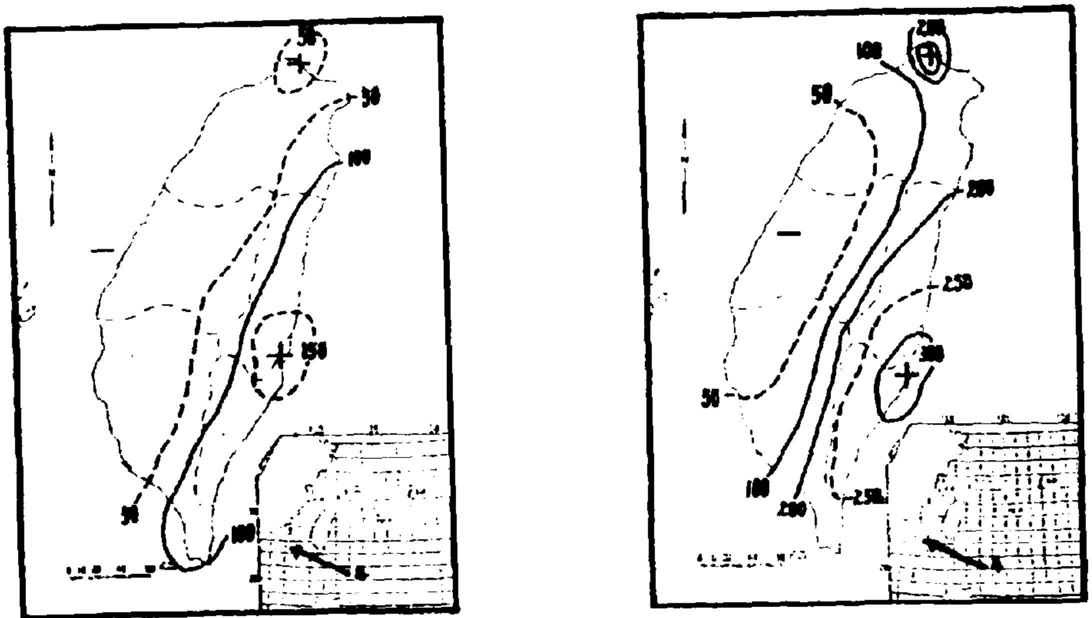
(a) (b)

Fig. 10 The distribution of the composite average precipitation for the sixth typhoon track category.
(a). August or earlier.
(b). September or later.



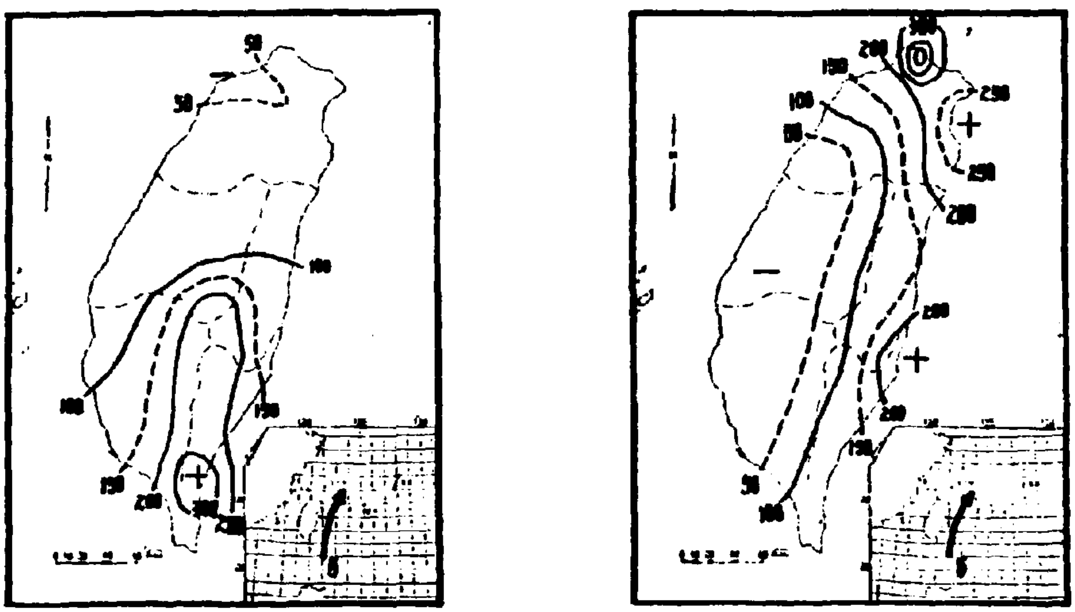
(a) (b)

Fig. 11 Distributions of three-hourly rainfall for a Mei-Yu case.
(After Chen and Tsay, 1984)



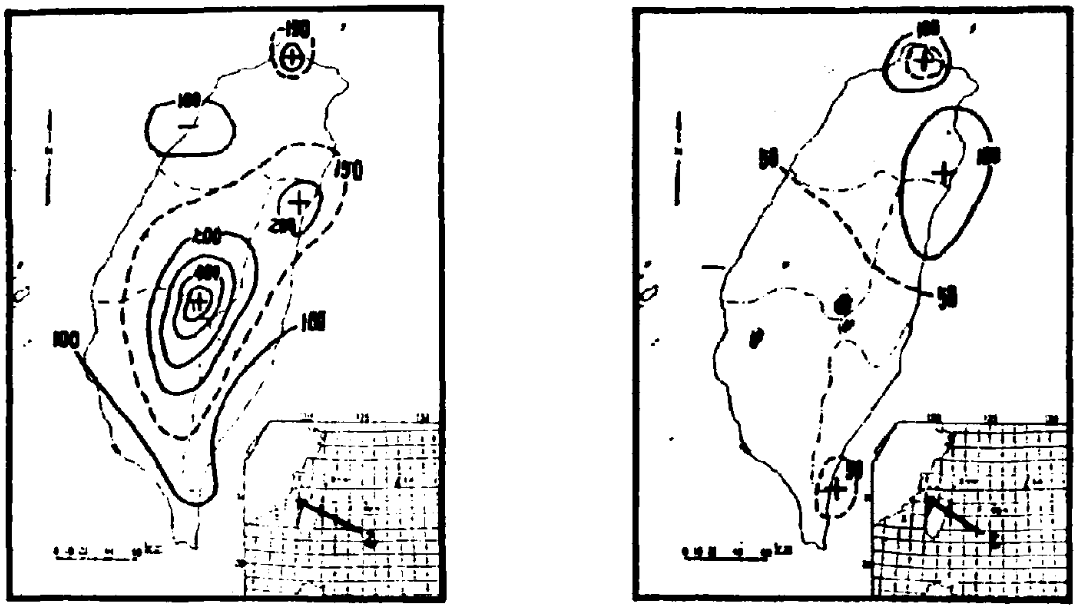
(a) (b)

Fig. 8 The distribution of the composite average precipitation for the fourth typhoon track category.
(a). August or earlier.
(b). September or later.



(a) (b)

Fig. 9 The distribution of the composite average precipitation for the fifth typhoon track category.
(a). August or earlier.
(b). September or later.



(a) (b)

Fig.6 The distribution of the composite average precipitation of the second typhoon track category.
(a). medium to strong typhoon intensity.
(b). weak typhoon intensity.

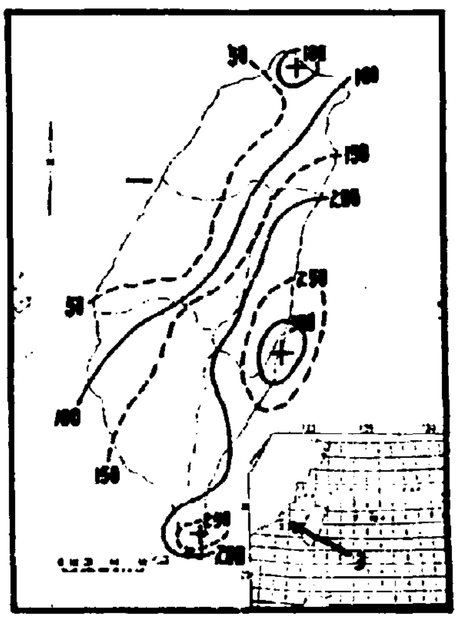


Fig.7 The distribution of the composite average precipitation for the third typhoon track category.

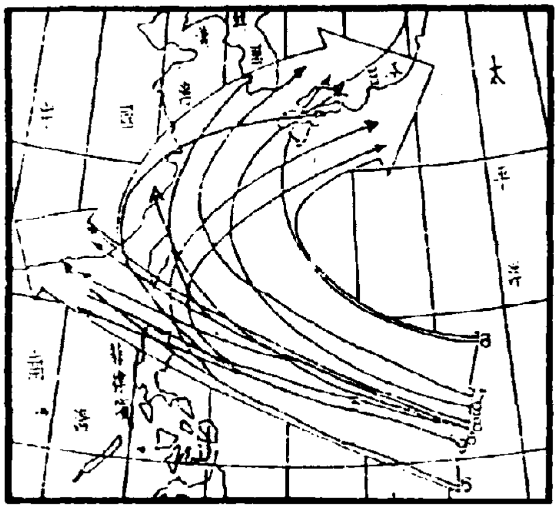


Fig. 3 The most commonly seen typhoon tracks from May to September, the numbers represent months of the year.

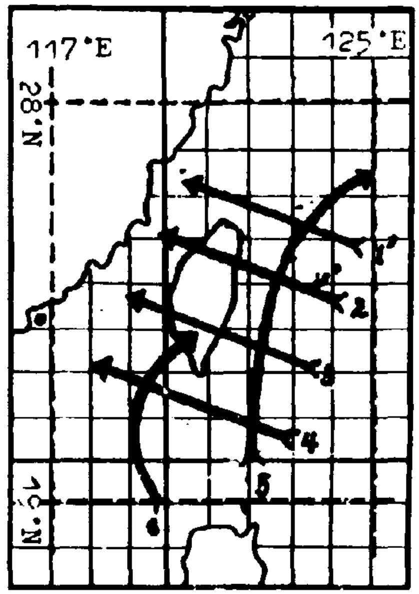


Fig. 4 A schematic diagram for the six typhoon track categories.

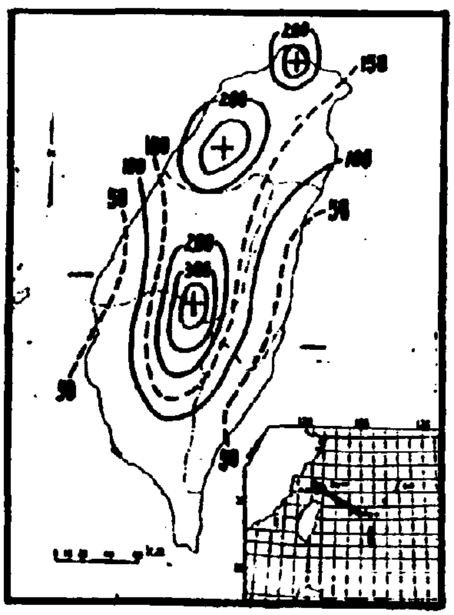
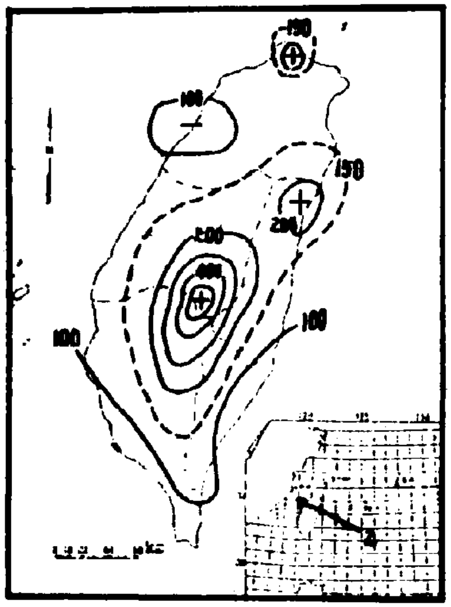
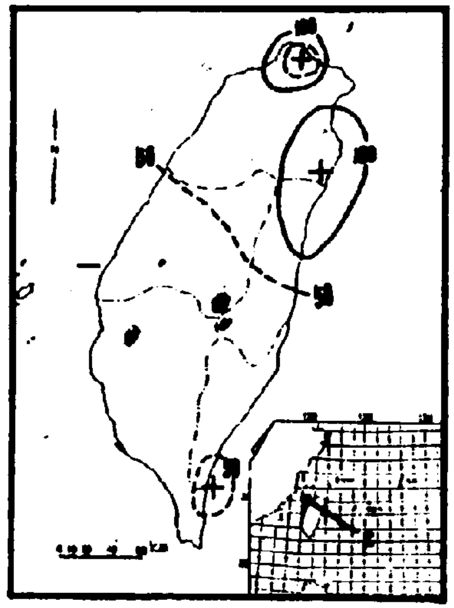


Fig. 5 The distribution of the composite average precipitation for the first typhoon track category.



(a)



(b)

Fig. 6 The distribution of the composite average precipitation of the second typhoon track category.
(a). medium to strong typhoon intensity.
(b). weak typhoon intensity.

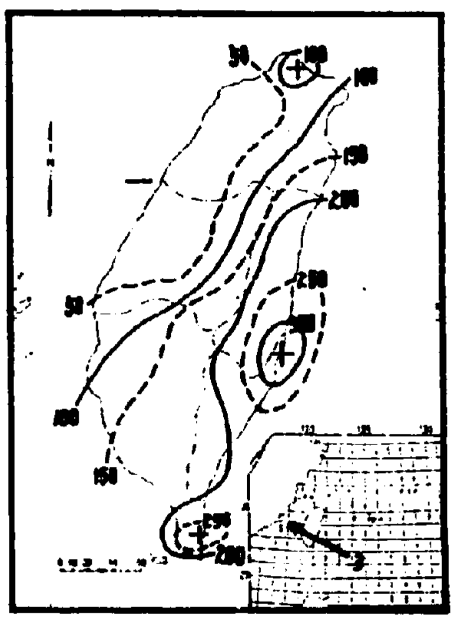


Fig. 7 The distribution of the composite average precipitation for the third typhoon track category.

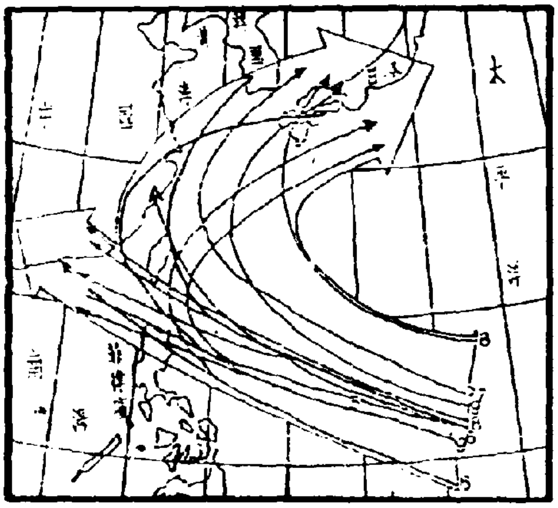


Fig. 3 The most commonly seen typhoon tracks from May to September, the numbers represent months of the year.

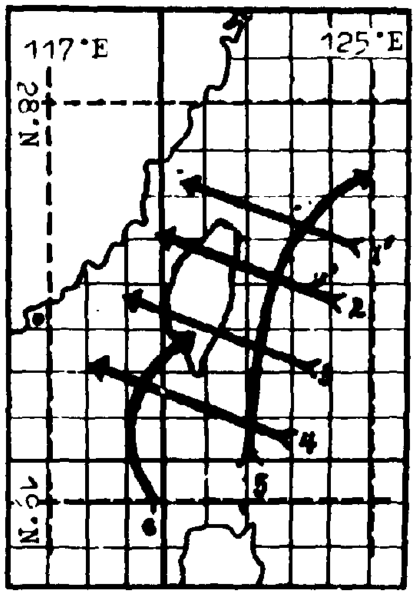


Fig. 4 A schematic diagram for the six typhoon track categories.

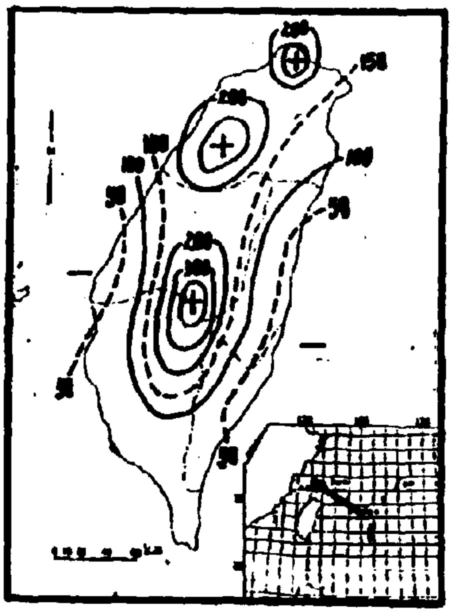


Fig. 5 The distribution of the composite average precipitation for the first typhoon track category.

SECOND US-ASIA CONFERENCE ON
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22-26 June 1992

PROCESSES OF RIVER BANK EROSION DURING FLOODS

Yuichiro Fujita, Yoshio Muramoto[1], and Yutaka Yahiro[2]
1. Disaster Prevention Research Institute
Kyoto University, Goka-sho, Uji, Kyoto 611, Japan
2. Chubu District Construction Bureau
The Ministry of Construction

ABSTRACT: Bank line retreat of a low water course caused by hydraulic scour and collapsing/slipping process has been observed since 1980 at a site in the Uji River, a middle reach of the Yodo River, in Kinki District, Japan. The bank erosion processes are discussed with a local bed scour at a toe of side slopes and its stability is evaluated by a simplified Junbu method of slope stability analyses. Local bed scour near the bank of the low water course causes bank slope instability and the slip failure is predicted to take place after overbank floods.

INTRODUCTION

Many houses and wide areas of agricultural land have often been lost by river bank erosion during floods, damaging human activities and social stocks. For the mitigation of social losses caused by river bank erosion, its processes must be clarified to pursue effective countermeasures. Bank erosion processes have mainly two sub-processes, hydraulic transports and geomechanical failure. In the former process, soil particles are derived from bank slope and washed away to make the slope steep, and the slope loses stability to fails and collapse or slip down onto the lower part of the slope. In this latter process, large part of the bank is supplied directly into the flow with high transport capacity, therefore bank retreat rate is accelerated.

OBSERVATION SITE

Bank line retreat of a low water course caused by hydraulic scour and collapsing/slipping process has been observed since 1980 around the 43 km site in the Uji river, a name of a middle reach of the Yodo river, the most important river in Kinki District, middle-western Japan. As shown in Fig. 1, the 38-45 km reach of the Uji river is a compound cross sectional channel. Its low water course has fairly constant widths of 80-100 m, while the floodplain widths change from 40 m to 350 m, being restricted with dikes of both sides. Along banks of the low water course, willow shrubs grow above the ordinary water level and the flood

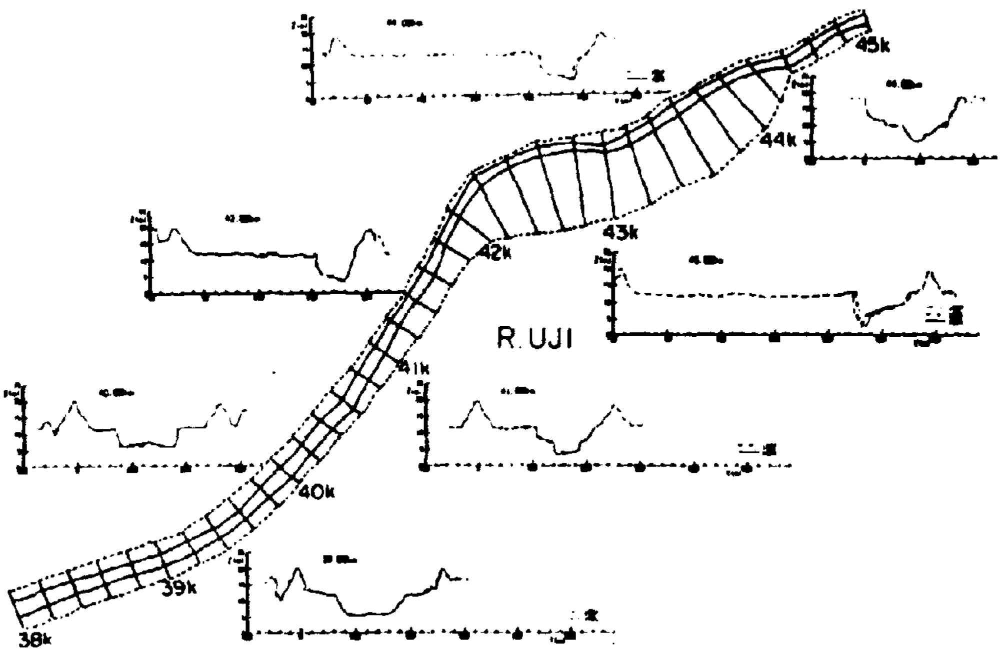


Fig.1 A sketch of the 38-45 km reach of the Uji River.

plain is covered with tall reeds. Cross sectional shapes at the 43 km show distinctive bank erosion on the left hand side of the low water course, where the willow shrubs were washed away with bank materials. Except there, however, no severe erosion has been recognized on both side banks though the low water course has almost same geometry. Texture and size distribution of the bank was described previously, as well as hydrologic and hydraulic conditions (Fujita and others, 1988).

BANK EROSION PROCESS

Retreat of the left side bank edge line of the low water course around the 43 km site is described in Fig.2 from surveys by plane table from 1980 to 1988 and by electro-optical distance meter after that. Erosion rate is higher in downstream part than in upstream part. Typical changes in bank shapes are illustrated in Fig.3 and bed topography in this reach is delineated in Fig.4. Figs. 3 and 4 present that bank heights at the 43 km are always about 10 m, almost twice at the 43.2 km. Contour lines in Fig.4 indicate that bank heights and water depths along the eroded side increase in the downstream direction and that high bed elevation area like a point bar along the inner bank of this slight bend has been advancing downstream according to the opposite bank erosion, which is demonstrated by changes in 7 m contour lines. Such variations

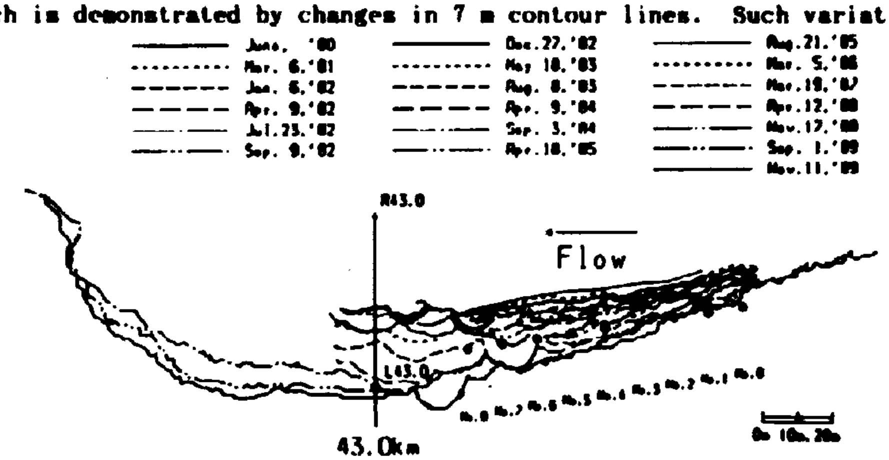


Fig.2 Retreat of bank edge line around the 43 km section.

rather than continuous grain dislodgement from the slope. It corresponds to bank edge line variations that indicate large-scaled bank failures of slip type are restricted within downstream part of the observed reach where bank is higher than 8 m and edge lines leave several concave figures of circular arcs. Bank slope failure conveys a large amount of bank materials directly into high capability zone of sediment transport and intensified bank erosion.

STABILITY ANALYSIS OF BANK SLOPE

Stability of the bank slope is investigated with a simplified Junbu method formulated by Yamagami and Ueta 1986. Near the 43 km site, a diluvial gravel layer with N values of 21-50, standard penetration test, is found from about 10 m depth below the floodplain surface. This diluvium is covered with an alluvium that consists of two sand-gravel layers and two clay layers lying alternately. Accordingly, soil parameters used in the analysis should be varied for each layer, but for sake of the simplicity, the bank materials are assumed to be homogeneous and soil parameters, such as the unit weight γ, the internal frictional angle ϕ and the cohesive force c, are presumed to be represented by a single value respectively. Representative values are sought parametrically, comparing analytical results of slip surface shapes and minimal safety factors with the survey data.

Acceptable values that $\gamma = 1.7\text{-}1.9$ tf/m^3, proved to have only slight influence on the results and fixed to be 1.8 tf/m^3. Values of $\phi = 10\text{-}40^\circ$ were examined and found reasonable to be 30-35. These values' coincided with those evaluated from N values of 5-21 in the sand-gravel alluvium by an empirical relation for ϕ. Then, cohesion c is determined parametrically below, changing values from 0.5 tf/m^3 to 2.5 tf/m^3.

Two sites, the 43.0 and the 43.25 km sections, the highest and the lowest bank respectively, were chosen for the analysis to distinguish the influence of bank height on slope stability. The water stage conditions in the river and in the floodplain ground water are simplified of the stream channel implies a mutual interaction between river bank erosion and bed variations.

Fig.5 shows a relation between cross-sectional areas eroded annually and bank heights above a point of abrupt changes in side slope. Though a positive correlation is clear between them, two different trends can be recognized in it. One of them can be expressed by a function that $A = 3.2(H_B-5)^2$ near the upper envelope line, where

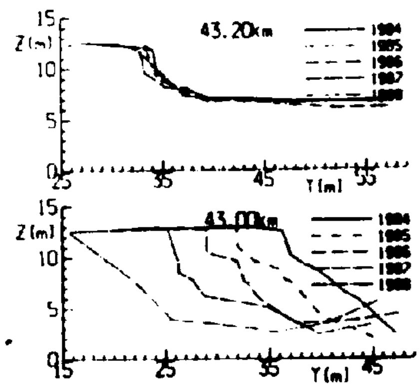


Fig.3 Typical change in the bank shapes of the low water course.

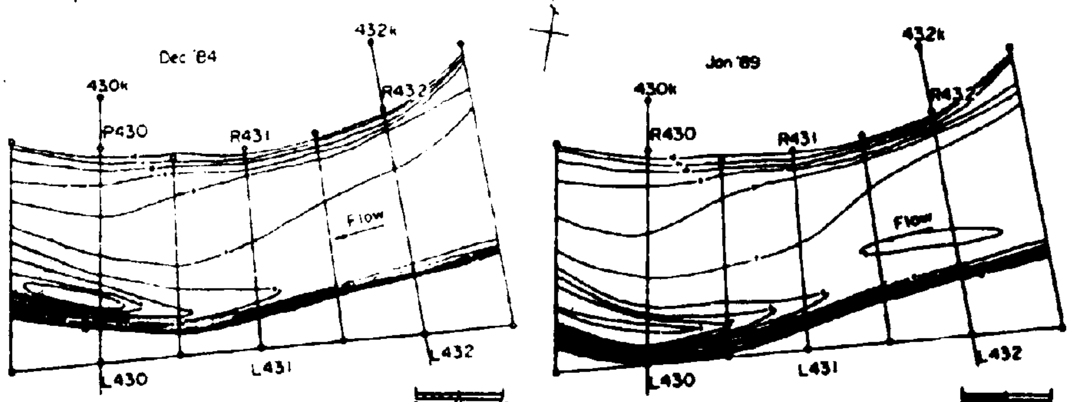


Fig.4 Changes in bed topography in the reach of eroded bank.

A is the cross sectional area eroded annually and H_B the bank height. This relationship corresponds to slope changes with complete removal of slipped bank materials. The other is denoted approximately as an equation $A = 3.2(H_B-7)^2$ near the lower envelope, suggesting the partial removal of failed materials or small scale bank collapse. These quadratic increases in erosion areas with bank heights implies that slip failure is common and its widths are proportional to bank heights exceeding 5 m,

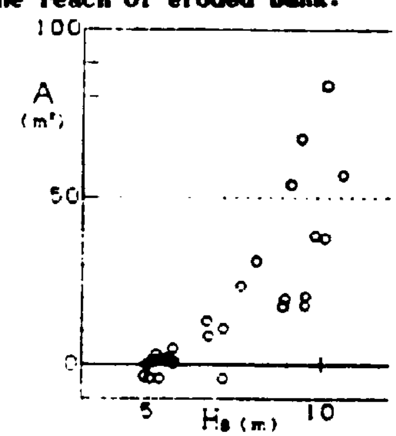


Fig.5 Correlations between annually eroded areas and slope heights.

into three combinations, considering typical flow phases, that is:

a) annual mean water level both in the river and in the ground, for the ordinary river bank condition or that long after floods

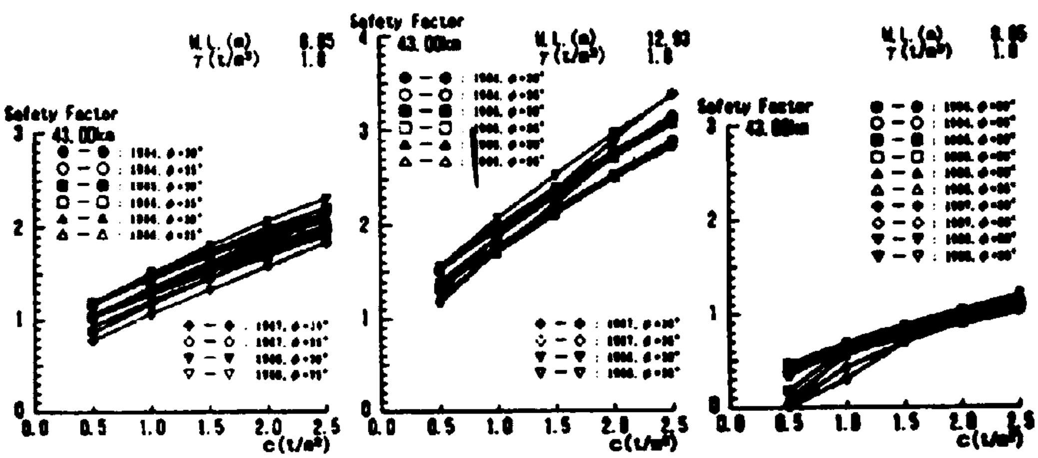
b) floodplain level both in the river and in the ground, for the condition during floods.

c) annual mean water level in the river and floodplain surface level in the ground, for the condition just after flood recessions

Some analytical results of slip surfaces and the minimal safety factors, F_s for a slope shape in 1986 are depicted in Fig.6, being compared with that in 1987. According to increase in c, slip surfaces are enlarged and move to deeper part, while values of F_s become large. Fairly similar slip surfaces for different bank height imply a quadratic increase in failed mass corresponding to the trend in Fig.6.

Values of F_s calculated under three conditions above are plotted in Fig.7(a)-(c) respectively for bank slope surveyed in 1984 -1988 at the

Fig.6 Some results of slip surface and minimal safety factors.



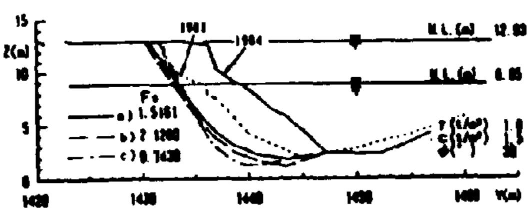
(a) (b) (c)

Fig.7 An diagram of calculated minimum safety factors for the 43 km.

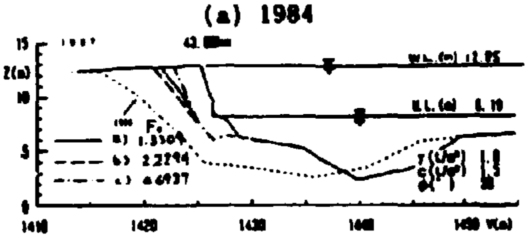
43 km. Changes in F_s values are little for both variation of ϕ values and annual change in slope shapes, while large for that of c. F_s have the highest values in case b) of whole bank submerged while the lowest in case c) of bank containing a large amount of pore water just after floods. In case b), effective weight of bank body which is a cause of instability is reduced by submergence to make the bank more stable. In contrast, the bank becomes unstable in case c) because pore water left in the bank produces a high hydraulic sliding force. Values of F_s at 43 km were much smaller than those at 43.25 km though they show almost same bank steepness. Such a clear distinction of F_s values can, therefore, be ascribed to a great difference of the bank heights which is subsequent to bed scour near the bank. It agrees with the observed variation of the retreat rate along the bank mentioned above.

Under the condition c), Values of F_s are smaller than unity for almost all combinations of c and ϕ at the 43 km, whereas the unstable results are restricted for several cases that c = 0.5 tf/m^3 and c = 1.0 tf/m^3 at the 43.25 km. Since only a few cases of c = 0.5 tf/m^3 yields F_s smaller than unity in case a), slip failure of bank is interpreted to occur mainly after flood recession at the 43 km section.

Inspecting result of slip surface shape, as shown in Fig. 6, for unstable conditions in case c), a combination that ϕ = 30° and c = 1.5 tf/m^3 proved to give the closest to the slope shapes measured in the subsequent years. One of them is that delineated in Fig. 10(a) by a chain line in case c) with F_s = 0.743 for 1984 data at the 43 km. Solid and broken lines in Fig.10 illustrate slip surfaces analyzed under a) and b) conditions respectively. These lines for a-c)



(a) 1984



(b) 1987

Fig.8 Changes in slip surface shapes with the minimal stability factors compared with slope shapes in subsequent years.

are fairly similar to each other in spite of the great difference of water stage conditions and values of F_s. The shape of bank slope in 1984 at the 43 km showing entire removal of failed mass gave good analytical results. On the other hand, a shape in the 1987 survey implies that remained part of failed mass on the bank slope toe. The analytical results shown in Fig. 10(b) are shapes of small-scaled slip surface much different from those measured in the subsequent year, indicating that large-scaled slip occurs when a flood rises after the river flow has conveyed bank materials failed previously.

CONCLUDING REMARKS

The slope stability analyses assured the occurrence of the bank failures occurs according to the scour near the banks. Spatial changes in flow velocity and bed shear stress are the causes of severe local bed scour. Distributions of flow velocity and shear stress changes may be evaluated through a 2-dimensional calculation of flood flows of various stages. We are now trying to develop a 2-D calculation model suitable for such a stream channel conditions and for the prediction of flow patterns and bed variations, though abrupt changes from the floodplain to the low water channel yield computational instability.

ACKNOWLEDGEMENT

We are grateful to Yodogawa Local Construction Office, the Ministry of Construction for the help of river channel survey. This study has been supported by Grant-in-Aid for Scientific Research "Natural hazard", the Ministry of Education, Science and Culture.

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SECOND US-ASIA CONFERENCE ON
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Yogyakarta, Indonesia
22-26 June 1992

EFFECTS OF THE WAR ON FLOOD DAMAGES IN HIROSHIMA DUE TO TYPHOON 4516

Yoshiaki Kawata[1], Taro Oka[1] and Yoshito Tsuchiya[2]
1. Associate Professor 2. Professor
Disaster Prevention Research Institute
Kyoto University, Goka-sho, Uji, Kyoto 611, Japan

ABSTRACT:Just after the second world war, typhoon 4516 which landed on Makurazaki, Kyusyu Island on 17 September 1945 with the atmospheric pressure of 916.4mb generated severe damages in Hiroshima. The nationwide loss of lives due to the typhoon was 3128 and a two-third of the dead was counted in Hiroshima. The factors which enlarged the damages in Hiroshima are 1)a lack of proper meteorological observation systems, 2)delay of debris and flood control works and 3)A-bombed wide area. They were all influenced by the war which declined disaster prevention potential. Due to the war, the government cut down the budget of land development and draft of civil engineers made the public works discontinuous and discontent.

1. INTRODUCTION

After the second world war, we had severe storm disasters in almost every year before 1960. Although they were due to strong typhoons and concentrated heavy rainfall, the effects of the war can not be neglected. During the war, the natural environments as well as social ones were much changed. Typhoon 4516 landed on 17 September 1945 and passed near Hiroshima as shown in Fig. 1. The pressure on landing was 916.4mb which is the second lowest pressure in Japan. At that time, our meteorological networks were much destroyed and confused by the war so that the typhoon track could not be traced well. As well known, Hiroshima where an atomic bomb was dropped on 6 August 1945 was still in ruins, therefore, the typhoon damages were enlarged. The number of the loss of lives due to the typhoon in Hiroshima was more than 2000.

The damages occurred at (1)flooding in the basin of the Oota river which flows

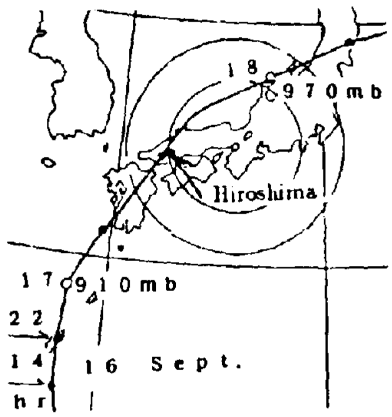


Fig.1 Track of Typhoon 4516

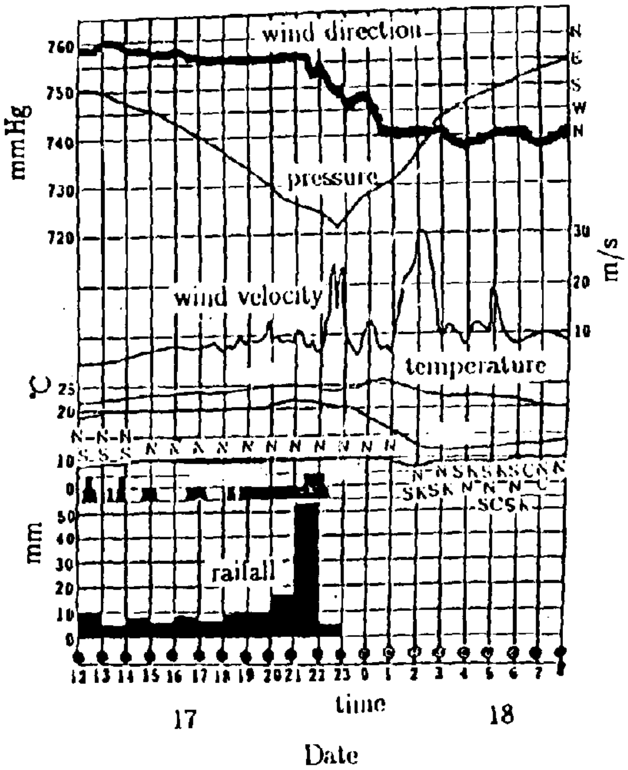


Fig.2 Meteorological conditions

through Hiroshima city, (2)debris flow in Kure city and (3)debris flow around country sides in Hiroshima prefecture. We discussed on the effects of the war on the damages through the data analysis with newly developed tools.

2. METEOROLOGICAL CONDITIONS IN HIROSHIMA

Fig. 2 shows the changes of meteorological conditions in Hiroshima. The maximum wind speed(10 min averaged) was 30.2m/s and its direction was north. We had unseasonable weather(much wetted due to a long rain) late August in 1945. The total rainfall in the disaster was 218.7mm and the rainfall intensity was 57.1mm/hr. The daily maximum and the four-hours maximum rainfalls in Kure are 185.1mm and 113.3mm respectively. The maximum total rainfall due to this typhoon 886mm was recorded at the Shikoku mountains. Therefore, the rainfall observed in Hiroshima area was not remarkable in comparison with other places located in nationwide.

3. DAMAGES DUE TO DEBRIS FLOW

In Hiroshima, the weathering granite widely distributed and its area was the largest in our country. When trees in the granite area are cut down, afforestation and recover of forest are very difficult. Moreover, the coefficient of permeability of the weathering granite is large, so that the weathering has easily penetrate into a deep layer. The large sediment discharge accompanied with heavy rainfall depends on this characteristics. The occurrence of debris flow was also accelerated by man-made environment as revealed in this section.

3.1 Single Debris Flow In Oono Village

Oono was located 20km west from Hiroshima city as shown in Fig. 3. In this village, we had the Oono Military Hospital in which the number of inpatients was about 800 including atomic bomb injured. Due to the debris flow, about 180 lives were lost. In the central part of the hospital area the Maruishi river whose catchment area was $0.617km^2$ flows. Usually the river discharge was very small and at the mountain site it changed to wadi. The debris flow has repeatedly occurred in 1804 and 1886. The longitudinal slope of the river is divided into three portions as shown in Fig.4. We have many small rivers whose slope is larger than that of the Maruishi river, but the volume of sediment yielded by the debris flow reached to $2.6 \times 10^5 m^3$. The discharge volume per unit area $4.3 \times 10^5 m^3/km^2$ is nearly maximum(Mizuyama,1989). The damages of buildings and the width of debris flow can be reconstructed with aerial photograph analysis in Fig. 5. The kinematic wave method can hindcast the river discharge as shown in Fig. 6. The dynamics of the debris flow can be analysed with recent contribution on this

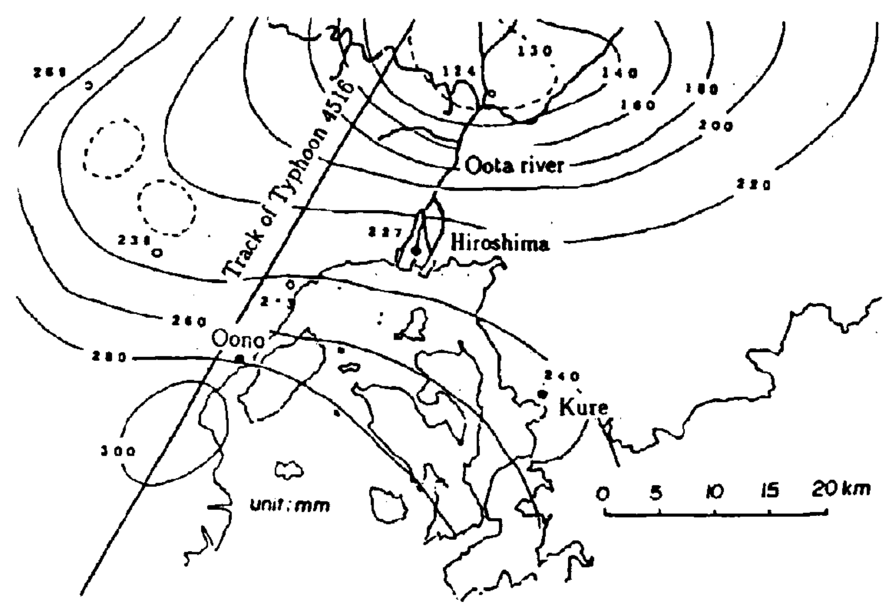


Fig.3 Location and total raifall around Hiroshima

problems by Ashida and Egashira(1989). Fig. 7 shows an example that the theoretical prediction of occurrence of d^bris flow is good agreement with the field data.

3.2 Debris Flows In Kure City

Kure was the second largest city in Hiroshima prefecture. During the war, naval base and dockyard were located in this city. The resident area had been developed on

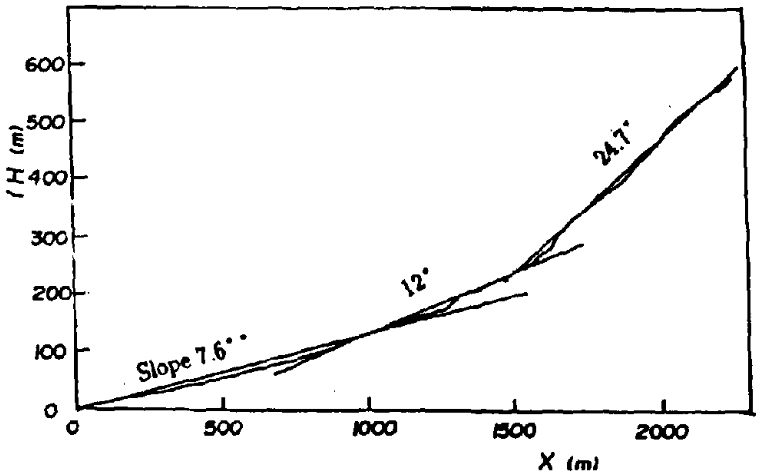


Fig.4 Longitudinal slope of the Maruishi river

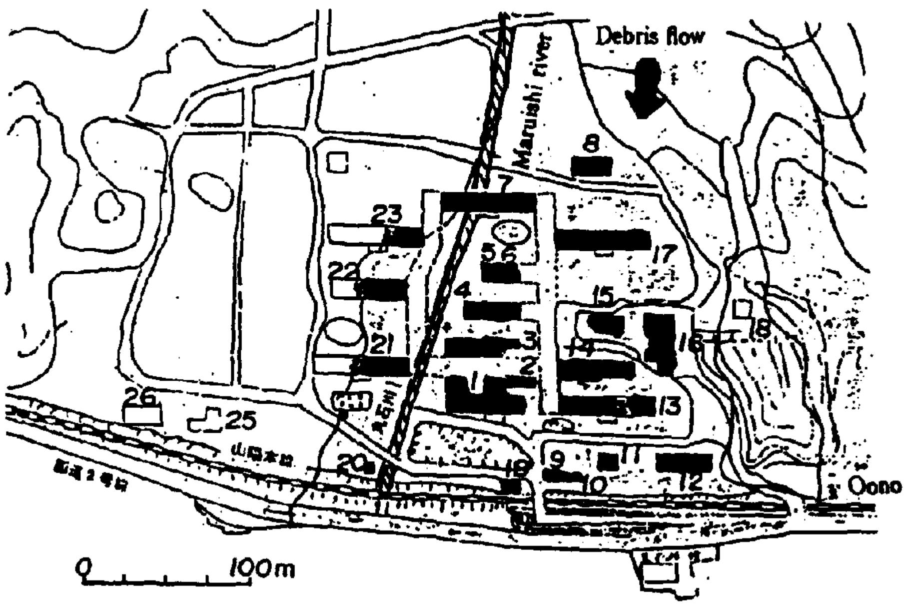
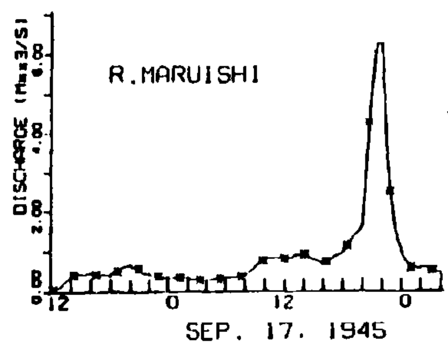


Fig.5 Debris flow and damaged buildings(black ones)



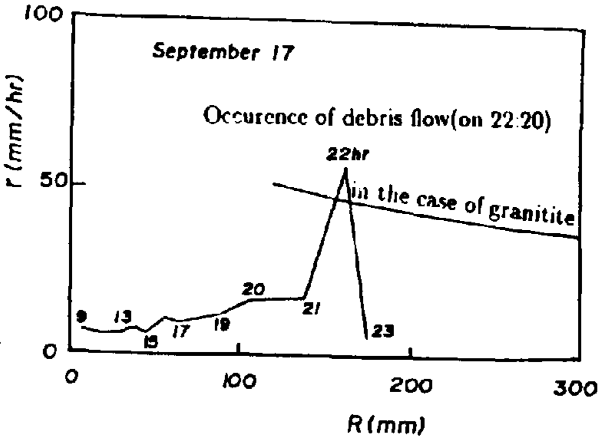


Fig.6 Discharge of the Maruishi river

Fig.7 Prediction of occurrence of debris flow

the slope of the mountains along the coast and so was very narrow. Rapid development due to naval demand of man-power and increase of military force make the population large. Although we have not accurate census of this city around 1945 due to military secret, the number of resident seems to reach more than 300,000. Therefore, there were no additional space to build living facilities in the city area so that some rivers were covered to build houses over them or the mountain slopes were newly developed. Moreover, at the end of the war, naval headquarters constructed some roads on a steep slope of the mountains. These roads were used to construct anti-aircraft emplacements. Due to the debris flow and flooding, the number of the dead were 1154.

In the mountains, a large number of landslide and mass movement were occurred around the mountains (from the visual survey of aerial photographs they were found at 591 points.). They almost played triggers of the debris flow. Fig. 8 shows the flooding area and distribution of the number of the dead in the old city area. The disaster report(1951) described the process of the enlargement of the damages. The large amount of sediment discharge buried river courses and debris control dams, and overflow water with large velocity carried away or destroyed the wooden houses in the midnight.

4. FLOODING OF THE OOTA RIVER

The riverhead of the Oota river is located at Mt. Kanmuriyama(1339m in height) and the area of watershed is 1690km^2 and its length of major river course is 104km. In the Edo period(1603-1867), Hiroshima had developed as a castle town in the lowerer course of the river. The local government had promoted to get newly reclaimed rice field, so that the occurrence of flooding had increased. As shown in Fig. 9, the flood disasters occurred about 90 since 863A.D.

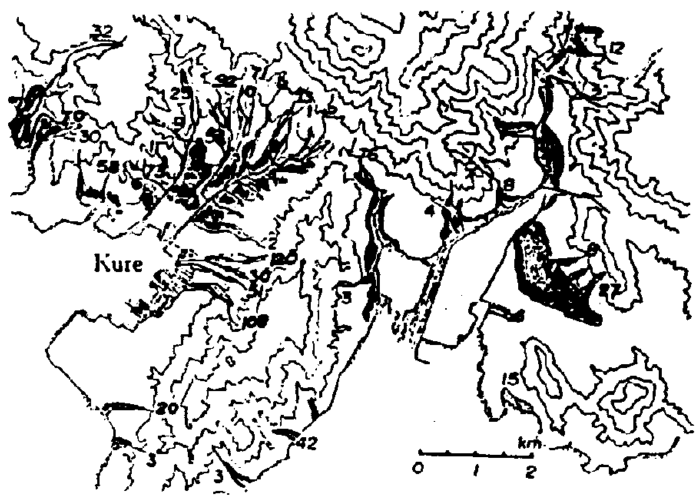


Fig.8 Flooding area of debris flow and the number of loss of lives

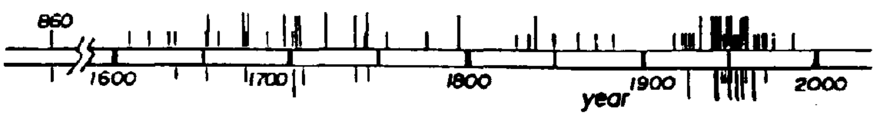


Fig.9 History of occurrence of flood disasters in the Oota river(long bar shows occurrnce of victims)

In the kinematic wave method, the governing equations are well known. The numerical calculation gives the hydrograph of the flooding as shown in Fig. 10. In the Oota river, we had severe flood disasters in 1988. In the run-off analysis, the various coefficients and constants included in the governing equations were already authorised by Oka(1989). The maximum flood discharge 6700m^3/s at Nishihara was recorded in 1943. In the 1945 flood, however, the estimated peak discharge was 5024m^3/s as shown in Fig. 10. The area inundated by flood water was very large in comparison with the scale of flood discharge as shown in Fig. 11. After the 1943 flood, the river improvement works had not also done due to the war as shown in Fig. 12. The cut-off budget and the lack of river engineers were very severe problems to continue the works. About 45,000 people escaped from north part of Hiroshima city lived temporarily on the riverbed and the neighboring of the Oota river at that time. Unfortunately, it is impossible to classify the victims due to atomic bomb or river flooding. For example, the number of the victims due to atomic bomb surveyed on 10 August 1946(one year after) were 122,338. In the flood plane, the estimated mean inundation height was 3.3m which is sufficient to carry out poor housing materials set temporarily. Therefore, this number surely include the victims due to the flooding.

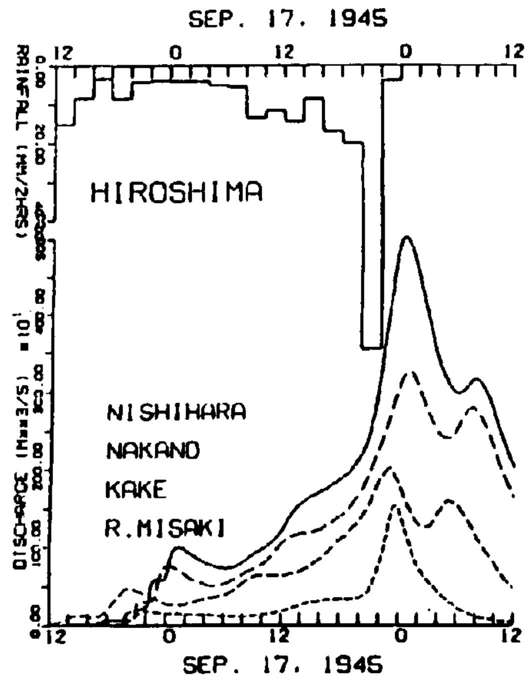


Fig.10 Some examples of flood discharge in the Oota river

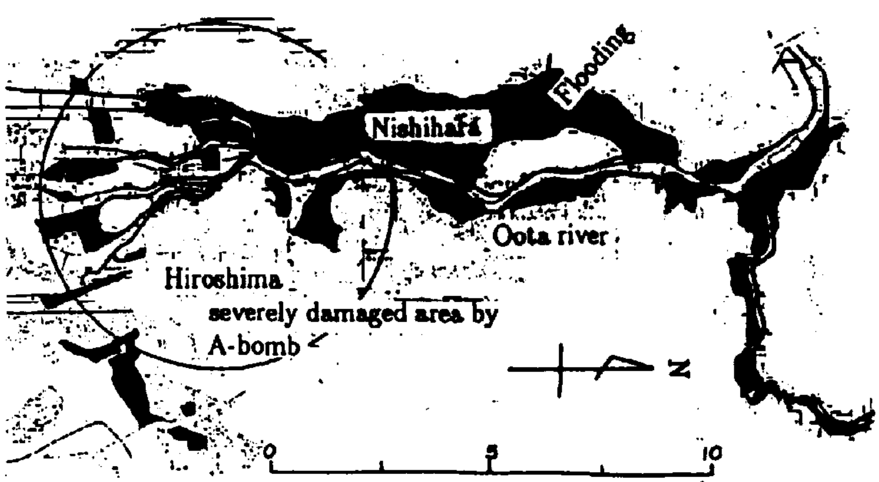


Fig. 11 Inundated area due to flooding in Hiroshima

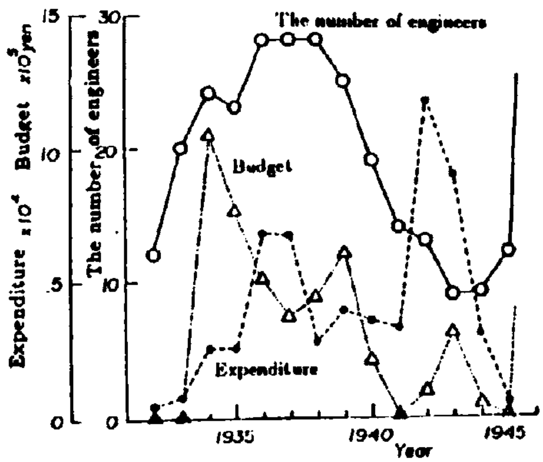


Fig.12 Changes of the number of civil engineers and budget at Oota Construction Office, Ministry of Home Affairs

5. OTHER FACTORS INFLUENCED

Our meteorological observation systems were almost out of use in nationwide due to damages of the war. Especially, Hiroshima Local Meteorological Observatory was severely destroyed by an A-bomb. Practically, no one knows that big typhoon came nearer. Sudden violent wind and heavy rainfall at night in Hiroshima must made people hopeless.

6. CONCLUSIONS

Through data analysis of the disaster caused by typhoon 4516, the damages in Hiroshima were enlarged due to the effects of the war. The major factors are pointed out as follows:1)a lack of proper meteorological observation systems, 2)delay of public works in the field of debris and flood control due to budget cut and draft of civil engineers , and 3)atomic bomb-devastated areas and occurrence of many refugees.

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SECOND US-ASIA CONFERENCE ON
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FLOOD HAZARDS MITIGATION IN MALAYSIA

Ir. Dr Hiew Kim Loi and Ir. Law Kong Fook
Department of Irrigation and Drainage
Kuala Lumpur, Malaysia

ABSTRACT Flooding is a significant natural hazard in Malaysia. Some 29,000 sq km or 9 percent of the total land area of Malaysia is flood prone affecting about 2.7 million people. The average annual flood damage is estimated at M $ 100 million. After the disastrous flood of 1971, the Government has taken positive steps to deal with the flooding problems. The strategies adopted comprise institutional development, implementation of structural and non-structural measures and a pro-active approach of comprehensive catchment planning and management. The Government's commitment is reflected in the increasing public expenditures on flood mitigation works. All these measures are aimed at creating a favorable environment to support and promote socio-economic development in the country.

INTRODUCTION

Malaysia covers an area of 330,400 sq. km, comprising of two regions, namely Peninsular Malaysia and States of Sabah and Sarawak. Situated just north of the equator, it experiences a tropical Monsoon climate. The average annual rainfall is estimated at 2,420 mm for Peninsular Malaysia, 2,630 mm for Sabah and 3,830 mm for Sarawak. The bulk of the water resources are derived from the South-west Monsoon (May to August) and the North-east Monsoon (November to February).

The topography of Peninsular Malaysia is characterized by a central spine (with ground elevations of up to 2000 meters above mean sea level) which slopes steeply to the relatively flatter undulating coastal plains on the eastern and western sides. In the States of Sabah and Sarawak, a similar terrain exists but the higher grounds are found in the interior along a northeast-southwest direction, bordering the boundary with Indonesia. There are more than 150 river systems in the country. The river courses are relatively short with steep gradients in the upper stretches and comparatively flat and meandering stretches in the lower reaches. Flood flows are therefore transient in the upper reaches but increase in duration and intensity towards the coastal plains. The bulk of the population are concentrated in towns and villages situated in riverine valleys and coastal plains and hence are prone to flood damage.

Flooding is the most significant natural hazard in Malaysia. The country is fortunate that it does not experience the problems of earthquake or typhoon as in her immediate neighbors such as Indonesia and Philippines. The severity of the flooding problem has escalated in recent years as the country becomes more developed. Very sizable flood damages have been experienced in many parts of the country as a result of the major flood events in 1967, 1971, 1973 and 1983. Though some 29,000 km or only 9% of the total land area are flood prone, more than 2.7 million people (18%) are affected by floods. Figures 1 and 2 show the flood prone areas in the country. The average annual flood damage is estimated at M$ 100 million.

FLOOD MITIGATION STRATEGIES

In Malaysia, the Government is the main body responsible for the provision of infrastructure facilities of which flood mitigation constitutes an important component. The problem of flooding is, however, a historical and complex one and hence a systematic and rational approach is required in order to ensure cost effectiveness. The Government is fully aware that the solution of the flooding problem requires multi-pronged strategies such as those listed below :-

(a) Establishing appropriate and workable institutions for implementing flood mitigation works and flood relief operations.

(b) Implementing flood mitigation measures in existing problem areas, comprising structural and non-structural measures.

(c) Adopting sound watershed development and management policy for future development to avoid aggravating existing or generating new flooding problems.

Establishment of Workable Institutions

The Government of Malaysia has established appropriate institutions or working arrangement to cope with the various aspects of the flooding problem. The most notable of these institutions are :

(a) The Permanent Commission on Flood Control

(b) The National Disaster Relief and Preparedness Committee

In addition, the Federal Government has also entrusted the Department of Irrigation and Drainage (DID) with the responsibility of implementing all flood mitigation programmes and the associated task of hydrological data collection for water resources planning and other applications.

The Permanent Flood Control Commission was first established by a decision of the Cabinet in 1971 and currently, it is headed by the Minister of Agriculture with DID serving as the secretariat. The Commission is entrusted with the functions of taking measures for flood control and to reduce the occurrence of floods and in the event of unavoidable flooding, to minimize the damage and loss of life and property.

The National Disaster Relief and Preparedness Committee, headed by the Minister of Information with its secretariat at the National Security Council is responsible for coordinating relief operations at the Federal, State and District levels so that assistance can be provided to flood victims in an orderly and effectively manner. The machinery is activated when floods occur in several states or when a state experiences a massive flooding which cannot be adequately handled at the state level. Relief operations are then carried out by the Police, the Armed Forces, the Ministry of Health, the Ministry of Social Welfare, the Ministry of Trade and Industry and voluntary organizations such as the Red Crescent. A flow chart showing the operation of this Committee is given in Figure 3.

Implementing Flood Mitigation Measures

Over the years, many studies have been carried out to address the existing flooding problems affecting many of the larger river basins and population centers. Based on these studies, various structural and non-structural measures have been proposed. Structural measures include channel improvement, bunding, flood bypass, poldering, flood storage dams and flood detention basins. Non-structural measures include flood forecasting and warning, flood zoning and flood risk mapping and resettlement of affected population.

The commitment of the Government to flood mitigation is reflected in the steady increase of government funds for flood control works (Figure 4). For the period from 1970 to 1990, a total of about M$ 500 million was spent on flood mitigation activities. Under the Sixth Malaysia Plan (1991 - 1995), the financial allocation for flood mitigation programmes has increased to about M$ 700 million. Flood mitigation programme has become the largest component of engineering activities in the DID which has a traditional role of implementing irrigation and drainage works in the country.

The future scope or demand for flood mitigation works in the country up to the year 2000 has been identified by the National Water Resources Study completed in 1982. In brief, the study proposed the improvement of 850 km of river channels, the construction of 12 dams, 82 km of floodway and the resettlement of about 10,000 people. These works are aimed at providing protection to 50 % of the population living in flood prone areas by the year 2000. However, due to limitation of funds, only a small percentage of the recommended flood mitigation programmes has been implemented even though 10 years have passed since the completion of the study.

Sound Watershed Development and Management Policy

The long term solution to unacceptable flooding problem requires addressing the issues at its source, which is the watershed. The clearing of forested catchment for agriculture and urbanization is generally acknowledged as the main contributor of flooding in many localities, a typical example of which is the capital city of Kuala Lumpur.

As the population increases and coupled with the projected expansion of agricultural and industrial development, one would expect urbanization to continue at an even greater pace in the years to come. Hence it is prudent to practise early, a sound policy of comprehensive catchment planning and management to ensure that future development activities do not aggravate existing flooding or generate new flooding problems. The formulation and periodic review of Structure Plan for urban centers, drainage masterplan, flood risk mapping are examples of such effort to minimize the flooding problem typically associated with urban development.

At present, about 60 % of the land area of Malaysia is still under natural forest cover. Development activities in the forested watershed is carefully monitored and controlled by the Forestry Department. A National Forest Policy was adopted in 1977 and embedded in this policy is the concept of sustainable utilization of forest resources with due consideration to flood hazard mitigation and other benefits.

CONCLUSIONS

Since 1971, Malaysia has embarked on a systematic long term programme to cope with the natural hazard of flooding which constitutes a major hindrance to socio-economic development. Workable institutions or framework has been formulated to implement programmes for flood control and to carry out relief operations during major or catastrophic floods. For existing flood prone areas, the Government has committed to large expenditures to implement structural and non-structural measures aimed at alleviating flood damage. Equal emphasis is also given to the pro-active approach of comprehensive catchment planning and management to avoid aggravating existing flooding or generating new flooding problems. Through this multi-pronged approach, it is hoped that the flooding problem can be effectively controlled and the water resources of the country can be harnessed for the maximum beneficial use of the people.

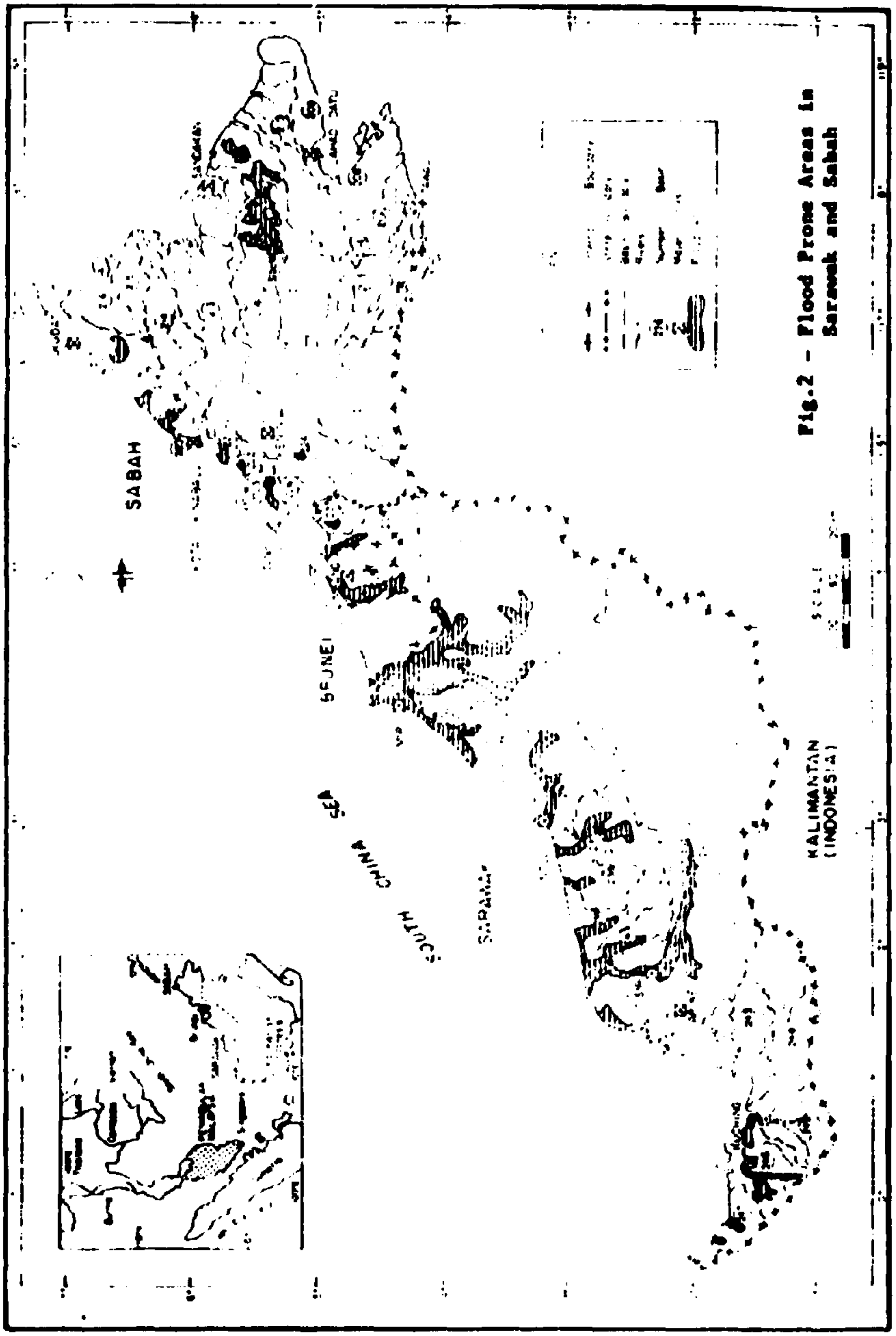


Fig.2 - Flood Prone Areas in Sarawak and Sabah

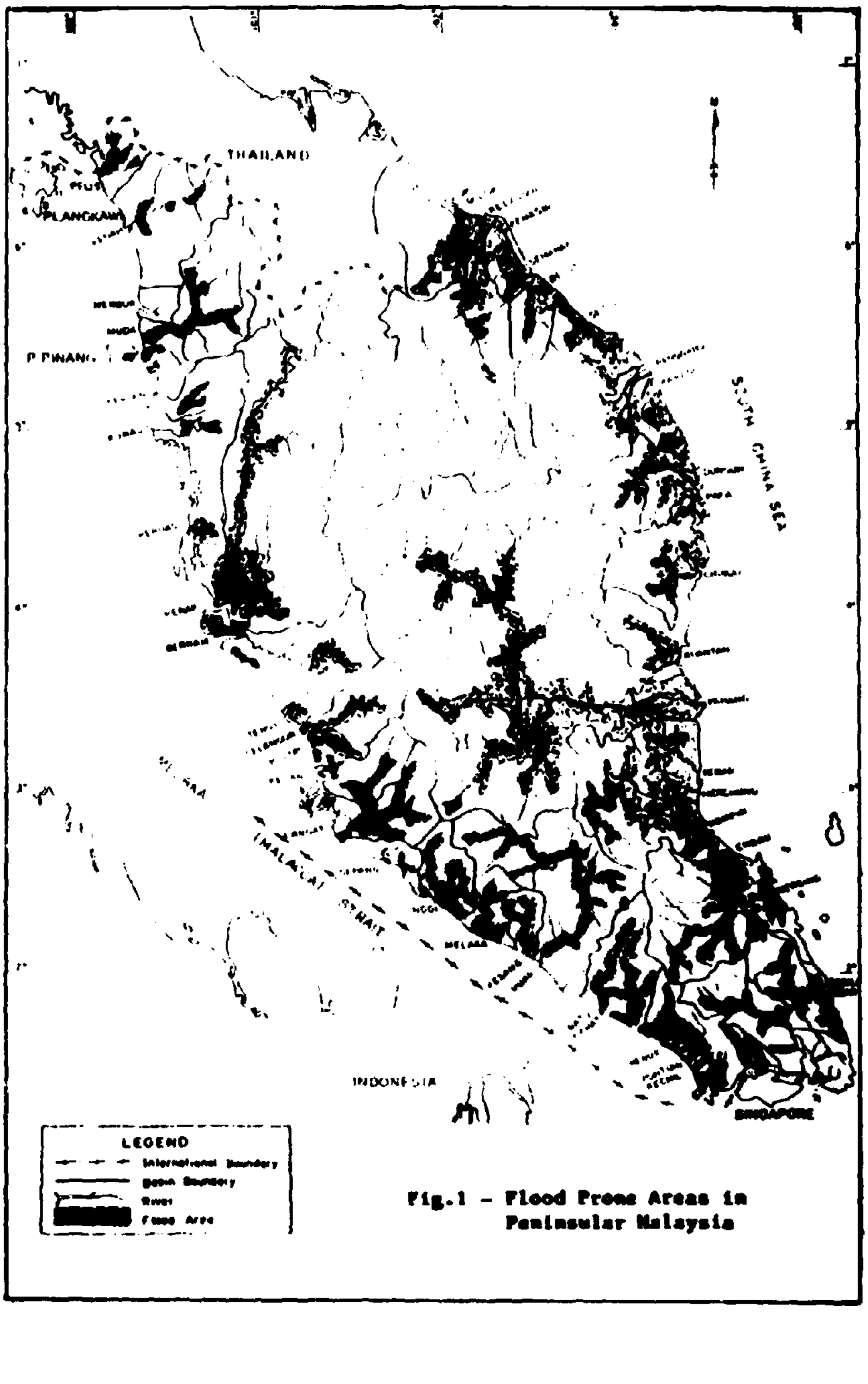


Fig.1 - Flood Prone Areas in Peninsular Malaysia

FIG. 3 - FLOOD DISASTER RELIEF CONTROL MACHINERY

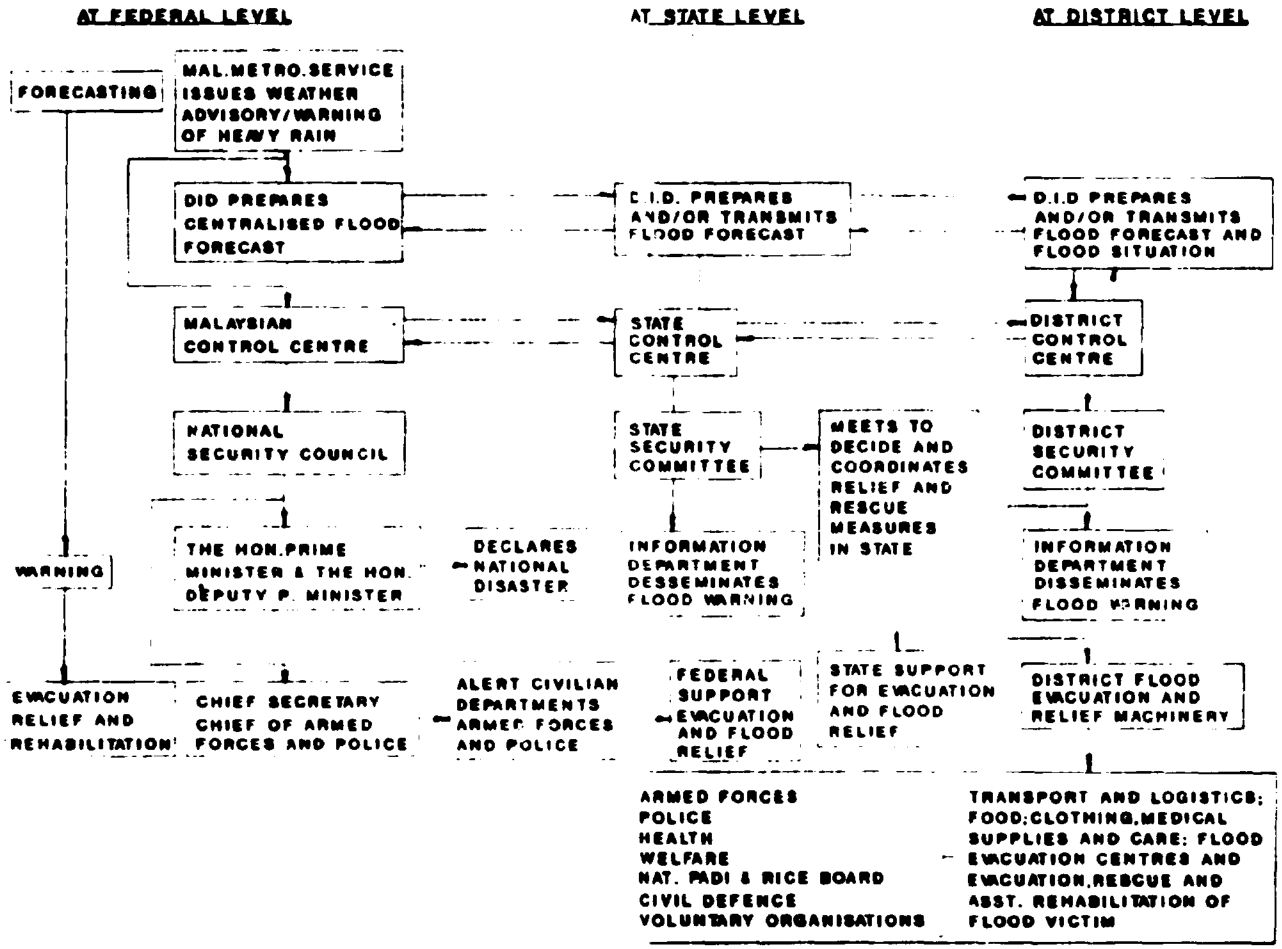
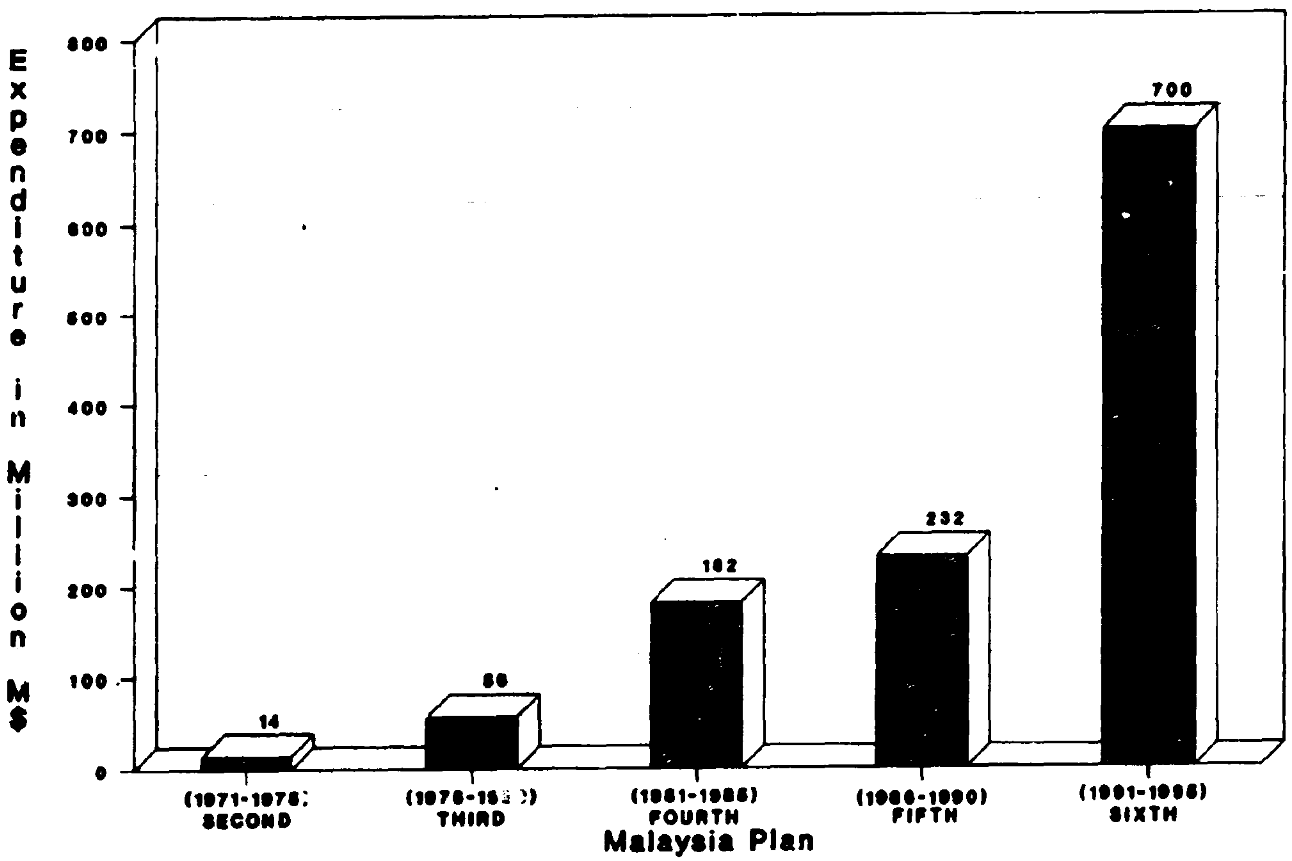


Fig. 4 - Financial Allocation for Flood Mitigation Works Under Various Malaysia Plans



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INTEGRATION OF NONSTRUCTURAL MEASURES INTO FLOOD CONTROL PLANNING

L. Douglas James
Utah State University
Logan, Utah 84322 USA

ABSTRACT The nonstructural approach to flood control was introduced in the United States to supplement costly dams, levees, and channels to control floods. When flood damages were still increasing despite an extensive structural program, the use of nonstructural measures was seen as a way to cut costs and reduce the environmental disruption of constructing large facilities. However, nonstructural methods shift much of the cost burden to the private sector by making land less productive and construction more expensive. The structural/nonstructural balance is particularly important in developing countries because both large flood losses and large development costs can retard economic growth.

The middle way is to combine measures in programs that help people improve their lives despite flooding. Each floodplain is unique, and each situation requires a separate analysis to chose the best measures and provide for their effective implementation and efficient operation. This paper discusses the measures, their assessment, and their combination. Of the nonstructural measures, flood proofing helps where the floods are shallow; land management is more appropriate with deeper flooding; and contingency programs provide backup during major disasters.

INTRODUCTION

Floods are large events; and society long lacked technology to build dams, levees, and channels to contain major floods. By 1900, construction capability reached the point where governments could hire cadres of engineers and build projects that they identified with a pride that was reinforced as flood-control infrastructure enhanced national economic development. The rub is that politicians and engineers who concentrate on economic gain find their projects causing environmental and social impacts that turn "beneficiaries" into antagonists. Opposition mounts when people experience increasing flood losses and other adverse social and environmental impacts and then face a greater financial burden to repair the aging facilities.

In the last 30 years, these increasing impacts and costs have favored a turn to nonstructural measures, and yet many people now wonder whether the reduced use of flood control infrastructure is neglecting an essential ingredient of national development. When planning to find the best balance, we can think of flood damages as a tax imposed by nature on economic development and of flood control as a program of human intervention to reduce a tax whose payment is divided among government budgets for flood control, government costs of keeping transportation and communication lines open, flood losses suffered by private parties, and expenditures people make to protect themselves. Since a heavy tax burden retards economic development, the planning goal is to find the combination of measures (ways to pay that tax) that minimizes the payment. However, it is important to recognize that economic development increases all taxes over time. It is not realistic to expect to be able to find flood control programs that will reduce the total tax.

CHARACTERIZATION OF FLOOD PROBLEMS

Planning begins by defining the problem. Flood waters from rising rivers, storm-driven ocean waves, or intense local rains and disrupt human activities and damage capital invested for economic growth or by people seeking better lives. They inundate and destroy crops; buildings and their contents; and roads, utilities, and other infrastructure. These disruptions and damages increase with water depth, sediment content, velocity, and other parameters. Some areas are flooded frequently, and others only experience rare events. While damages may be repaired after a flood, the cost of repeated repairs takes available resources away from economic development. In summary, a flood problem can be characterized by depth, source, type of property inundated, frequency, and long-term impacts; and flood control measures should be selected to perform well for the given characterization.

ALTERNATIVE CONTROL MEASURES

Planners can next look at the options. Their goal is to convert ideas into implementable programs, and innovative people through the ages have experienced flooding and concocted a wide variety of response approaches. Each has its strengths and weaknesses. Each works better in some situations and worse in others. Here, we will briefly outline the chief options and their major advantages and disadvantages.

Governmental flood control programs have primarily relied on containing floods through the engineered construction of reservoirs, channels, and levees. Reservoir storage damps flood peaks, and larger reservoirs are often gated for positive flow control; channels convey water downstream; and levees ring areas with high property values or compartmentalize floodplains to protect highly-developed areas. These large programs have the advantages of economies of scale, installation by experts, and an institutional presence with continuing responsibility. They have the disadvantages of high cost, large potential for environmental and social disruption, and imposing solutions rather than working with the people with the problems.

People commonly blame upstream development for floods and sedimentation. Watershed management can restore rural uplands to natural cover with greater soil water storage and resistance of the soil surfaces to erosion. Detention storage can be used in urban uplands to compensate for greater runoff from paved areas. These land treatments provide a fixed storage capacity that reduces smaller but is relatively ineffective against larger flood events and are generally helpful for sediment control. Rural measures must be carefully planned to sustain the productive use of uplands for agriculture or forestry. Urban measures must be reasonable in terms of the total cost of land development and be periodically cleaned to remove accumulating debris. Effective programs are costly to sustain against developmental pressures.

Nonstructural measures broadly divide between land management and flood proofing. The goal of land management is to make the best use of land commensurate with the flood hazard, economic needs, and the unique contributions of floodplain environments. Land management can be coordinated with levees to control spatial inundation-depth relationships. The goal of flood proofing is to protect buildings and facilities by using elevated or more flood-resistant designs for new buildings or by raising or modifying existing buildings to reduce future damages. Dry flood proofing protects people and their possessions by keeping interiors dry, and wet flood proofing uses construction materials and designs that reduce damages after water enters the buildings.

A well-prepared contingency program 1)maintains hazard centers where people can obtain accurate information on the factors contributing to their flood risk (the goal for the flood hazard modeling recommended in the last joint conference (Burges and Gong, 1988)) and technical and financial help in arranging land use and designing buildings to meet their personal needs; 2)activates emergency centers, when floods or failures threaten, to give people current information in terms they can understand on evolving flood events; forecast flood peaks; coordinate flood fighting, evacuation, and relief activities by the public and private sectors; and act with authority to keep people working together in times of danger; 3)provides flood proofed infrastructure that is designed and maintained to minimize disruptions to communications, transport, commerce, water supply, and sanitation during flood events and recovery periods; and 4)gives people the security of having safe and secure places to go for refuge in the worst of circumstances and of having insurance or other sources of financial security against catastrophic losses. Refuge programs generally operate more effectively if they are locally-run facilities; for this, they need to serve other purposes between floods (secure storage, schools, hospitals, police, fire, and other critical functions) to be well managed and fiscally sound. All four prongs of the contingency program are important.

All four measures should be combined. A good total program uses structural measures and watershed management to contain floods and nonstructural measures and contingency programs to reduce human vulnerability. People are most vulnerable when their livelihoods are threatened by hazards they poorly understand and they have little institutional support. Experts can by more effective if they put more effort into contributing to the understanding and resources available to support people. Many actions promoted by experts, particularly in the nonstructural and contingency programs, do not work as well as the ideas generated by people living with the problem. However, these people need better risk information and supplemental resources to make their innovations more effective. During major floods, it is particularly important to have infrastructure that preserves communications and trade and to protect the livelihoods of the population with sources of income during the recovery period. The ultimate goal is to help people escape from a demoralizing status quo by implementing innovations that reduce flood vulnerability and help enhance social and economic development.

PERFORMANCE EVALUATION

The third step is to conduct performance evaluations to determine which options can be implemented and operated to alleviate the defined problem, individually or in various combinations. The performances must be evaluated from engineering, legal, financial, environmental, and social perspectives. The evaluation of engineering performance investigates whether the proposed design will function (contain the flood water) with low risk of failure. The legal check is to ensure that the action can be conducted within the framework of existing constitutional authority, to make sure that there is no discrimination when taking property and regulating human activity, and to blunt unnecessary legal liability. Financial performance is evaluated in terms of an ability to generate revenues to pay the bills. The environmental evaluation must cover ecologic, agronomic, and geomorphologic issues to make sure that the uses planned for the floodplains and the land treatments planned for the uplands will indeed deliver. Social performance is commonly overlooked and may be the most important because people have to use the land and water in ways that will make the program succeed. The performance evaluations are made by experts in these respective fields, and a proposal that fails the test in any of these five domains should be immediately dropped.

WORTH EVALUATION

The alternatives that pass these performance checks must then be evaluated to determine whether the sacrifices required to make them work are justified by worthwhile results. The analysis is complicated by multiple interactions. People interact in deciding what they want individually and in reaching community consensus. Projects interact within the hydrologic cycle, with other processes in their large environment, and with the national economy and possibly world trade. People interact with projects in matching desires and capabilities. The government program for flood control must be coordinated internally and integrated with private investments for economic development for overall balance. The assessment of worth is used to make decisions on the four tradeoffs in Table 1 and can use the six objectives for programs in public works listed in Table 2 as criteria (for further reading see Patton and Sawicki, 1986). The five feasibility tests shown in Table 3 weigh good impacts against the bad in projected future scenarios.

STUDY GUIDELINES

The studies for problem definition, program formulation, and performance and worth evaluations are best done by an interdisciplinary team supported by sound information and visionary officials and giving support to public participation and decentralized decision making (James, 1975). The decisions on the issues listed in Table 1 are best make by government officials. Programs stumble when people with preconceived preferences, whether discipline specialists or in government, are able to sway actions by making statements on principles that do not apply to local situations. The goal of planning is to find solutions not to endorse popular ideas.

However, politicians and bureaucrats are more apt to plan from prior visions that they are reluctant to expose to objective assessment. People in authority are reluctant to open planning to others, whether they are common people who may not understand some large issues or specialists whose models are too esoteric. Officials fear losing control over programs that build them popular support, having to deal with greater complexity than they can explain to the public, getting surprise results, confrontations that make them uncomfortable and uncertain, and having to reverse prior positions in times of change.

Economic optimization would combine measures in a program that minimizes the flood tax using the objective function (James, 1967):

$$\text{Min: } SMc + WMc + FPc + LMc + CPc + D \qquad (1)$$

TABLE 1

ISSUES IN FLOOD CONTROL PLANNING

Budget issue: Whether the benefits of government action justify taxes that reduce the production by the private sector.

Service issue: Whether it is less costly for government to pay the flood tax by providing new flood control infrastructure or for the private sector to pay reducing the income earned from land. The comparison should also consider the environmental value of the land and the social cost of depriving poor landowners of higher incomes.

Investment issue: Whether declining operating efficiency and the growing cost of maintaining aging facilities justifies change or replacement.

Allocation issue: How much limited public funds to take from other governmental programs and give to water resources management and how much of those funds should be given to flood control programs.

TABLE 2

GENERAL PLANNING OBJECTIVES

National income: Increase the net value of production for the country as a whole as determined by benefit-cost analysis (Greeley-Polhemus Group, 1991). Preserve people's incomes and jobs through flood events.

Equity: Distribute benefits fairly over the population. People suffer when wealth is concentrated as well as when people are not able to gain personally by working harder. Equity requires a balanced distribution of benefits among individuals, social groups, and regions as well as during project implementation and for the long run.

Freedom: Respect individual sovereignty to judge their personal best interest. However, individual rights conflict with public welfare when people benefit by using floodplains in ways that hurt others, cause public expense for relief programs, or aggravate flood emergencies. Government planners may feel that their better information and greater expertise allows them to tell people what to do, but they must be respect basic human rights.

Environmental quality: Preserve the floodplain ecologic and amenity values (James and others, 1978) found in unusual habitats, river corridors, pollution control, and green space.

Stability: Shield people from varied and uncertain situations. The protection that flood control provides against large losses should be planned to foster social change as it is no benefit to preserve a static society where people have no hope for better times.

Health and Safety: Protect people from drowning, injury, and water-borne and other diseases. While the value of life cannot be expressed in economic units, government funds to protect life are limited.

TABLE 3

FEASIBILITY TESTS

Economic Feasibility: Will generated economic benefits exceed the costs required to keep the project going? Benefit-cost analysis is generally applied for this purpose.

Financial Feasibility: Will sufficient funds be available to implement and maintain the project? Governments have long had difficulty securing adequate maintenance for structural projects, and funding flood proofing or income protection with land management in the private sector pose more difficult issues.

Environmental Feasibility: Will the environmental gains exceed the environmental losses? The results of environmental impact assessment need to be reviewed to give normative guidance to project planning.

Social Feasibility: Will the benefits to people that cannot be expressed in monetary terms exceed the disruptions to their lives? This is done through social impact assessment.

Political Feasibility: Will responsible government officials grant the necessary approvals and provide the sustained support required for the measures to continue to be effective over time? The needed preservation becomes an issue with each change in administration.

TABLE 4

ECONOMIC IMPACTS OF FLOOD CONTROL MEASURES

Measure	Initial Cost	Continuing Cost	Residual Damage
Storage	Large public for construction.	Small public. for maintenance.	Reduced losses for large floods.
Channels and Levees	Large public. for construction.	Medium public. for maintenance.	Increased losses for large floods.
Watershed Management	Medium to landowners.	Relatively large for landowners.	No impact on larger floods.
Land Management	Hazard mapping.	Increases with development.	To conforming uses.
Flood Proofing	Large to building owners.	Often large to bldg. owners.	Lives risked during major events.
Contingency Program	Small preparation cost.	Large during events.	Property difficult to protect.

where the respective terms are the costs of structural measures, watershed management, flood proofing, land management, and contingency programs and finally the economic flood damages. Each term sums the costs of initial implementation and the present worth of the costs of continuing operation and maintenance (Table 4). The evaluation process must then be broadened to balance the good and the bad in the other dimensions of feasibility as illustrated by such issues as those outlined in Table 5. A study integrating the various measures generally selects measures that take advantage of their primary strengths, and these are highlighted in Table 6.

Typical programs in the United States (L.R. Johnston Associates, 1991) and Europe (Handmer, 1987) select design frequencies based on public policy and employ structural measures to protect against the selected event, watershed management to reduce sedimentation, land management to keep people out of more hazardous areas, and flood proofing where people build in areas where the flooding is shallower or occupancy is otherwise justified. Experts and officials work with local people in flood prone communities (Flood Loss Reduction Associates, 1981) and have a contingency program of flood warning and evacuation and flood fighting, relief, and repair to use when all else fails.

In an integrated program, the advocates of each measure will find their problems and needs better met through the cooperative effort. For example, the supporters of contingency measures to reduce losses when the embankments are overtopped will gain by working with levee builders to protect their common interests. People coming from diverse viewpoints will understand each other better, and their cooperation will add to the credibility necessary to sell the total program as needed to resolve the issues on Table 1. Cooperation is much better than letting rhetoric on environmental protection or some other favorite cause exacerbate fears and obstruct the use of facts in objective decision making. In this case, the environmental groups can gain by overcoming the spreading feeling that environmental studies are largely delaying actions or that they serve the elite more than the common people or poorer nations.

Thorough planning is likely to favor nonstructural measures early and structural measures later in the process of economic development. The continuation of an effective nonstructural program throughout the period of urbanization or economic development will reduce structural

TABLE 5

WORTH FEASIBILITY ASSESSMENTS

	Financial	Environmental	Social	Political
Storage	Init. cost.	Environmental disruption.	Displacees.	Budget.
Channels and Levees	High cost, maintenance.	Floodplain ecology.	Imposed solutions.	Budget.
Watershed Management	Continuing cost.	Debris cleaning programs.	Upland livelihoods.	Developer pressures.
Land Management	Resist pressures for development.	Environmental impacts of development.	Keep public support for the concept.	Resist development.
Flood Proofing	Money for building owners.	Discourage building in sensitive areas.	Deployment by poor and elderly.	Enforcing building codes.
Contingency Program	Holding funds in reserve.	People forget in emergencies.	Disseminating warnings.	Preserve in good times.

TABLE 6

SETTINGS OF PRIMARY ADVANTAGE

Storage	Reservoir sites available. Feasible multipurpose projects.
Channels and Levees	River deltas and other flat areas. Smaller streams flowing through established urban areas or as part of the infrastructure built with development. Rural streams at a lower level of protection.
Watershed Management	Above areas subject to frequent flooding. Farmlands, urbanizing areas, and sediment source areas.
Land Management	Lands subject to deep flooding or having low economic potential. Regional layouts for urban and industrial development. Parks and green space. Lands reserved for later development.
Flood Proofing	Buildings subject to shallow, frequent flooding with slow rates of rise. Wet flood proofing of buildings in areas of greater hazard. Infrastructure.
Contingency Program	Serve and assist private parties who cannot avoid flood-prone areas. Where people can use reliable information to help themselves. Where needed to keep basic support infrastructure functioning and where the hazard can be life threatening.

costs, and the people living in the floodplain will feel comfortable in making personal sacrifices if the planning finds ways for them to improve their lives by living more productively with the hazard.

A good nonstructural program is not a way to protect preserve present conditions but a way to help people find a better life. Nonstructural measures require a major effort by common people who will not work hard unless they can better themselves. The poor of the world are more interested in their personal welfare than in saving government the cost of buying structural flood control. Countries around the world will gain if people can do this without moving to the cities and causing increasing population concentrations in urban centers.

CONCLUSION

A working balance among the available flood control measures is essential. In that balance, flood proofing generally helps most where floods are shallow or unavoidable; land management works best in areas of greater hazard; and contingency programs are essential where the risks are large enough for people to take flooding seriously and where they need extra security so as not to be overwhelmed by extreme events.

The program as a whole should combine these measures in a way that helps people improve their lives in flood-prone environments because these will always be with us. Particular attention should be given 1)to preventing floodplain development that initially benefits the developer but then brings irreversible change to the natural ecological balance and cannot be sustained and 2)to dedicating flood-prone lands to uses that condemn local people to perpetual poverty. Program implementation needs to be particularly well organized in developing countries where continuing large flood losses retard economic growth no matter which way the tax is paid. Society must have self confidence to keep going.

Each floodplain is unique, and each situation must be carefully evaluated to determine the best measures. Provision must be made for their effective implementation and efficient operation and to make adjustments in their design and maintenance as conditions change over time.

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PREDICTION OF FLASH FLOODS DUE TO AN ASSUMED BREACHING OF BANG LANG EARTH-ROCKFILLED DAM, THAILAND

Tawatchai Tingsanchali[1] and Naris Sinthananopakhum[2]
1. Professor 2. Former Graduate Student
Division of Water Resources Engineering
Asian Institute of Technology
Bangkok, Thailand

ABSTRACT The Bang Lang dam is an earth-rockfilled dam built across the Pattani river in South Thailand. The dam is 85 m high having a crest length of 422 m and a reservoir of a maximum storage capacity of 1,403 million m^3. The dam is susceptible to breaching due to overtopping caused by heavy rainfalls in the upstream catchment. In this study, a breach erosion model is used to predict the breaching erosion of the Bang Lang dam caused by flow overtopping. The subsequent downstream flooding due to the dam breach outflow is predicted by the MIKE-11 model. The relevant predicted results are the arrival time of the flood wave front, the maximum flood depth and the extent of flooding at various stations downstream of the dam to Pattani province, a reach length of 110 km. The results of this study are useful in drawing up an emergency plan for evacuating people living in the flood hazard area to a safe location. Sensitivity analysis is carried out to determine the effects of the model parameters of the two models on the predicted results.

INTRODUCTION

The purpose of this study is to predict the breaching erosion of the Bang Lang dam due to flow overtopping and the resulting flash flood downstream of the dam. The Bang Lang dam is a multipurpose dam built in 1976 across the Pattani river in South Thailand (Fig. 1). The dam is located at Bannang Sata district. It is an earth - rockfilled dam with a height of 85 m, a crest length of 422 m and a maximum storage of 1,403 million m^3. The catchment area above the dam is 2,080 km^2 and the annual inflow is 1,460 million m^3.

A semi-analytical model is developed (according to Singh and Scarlatos, 1988) and applied to predict the breaching erosion of the Bang Lang dam. The computed dam breach outflow from the breach erosion model is routed further downstream along the Pattani river for a distance of 110 km by using the MIKE-11 flood routing model developed by the Danish Hydraulics Institute (DHI, 1981). The arrival times of flood wave front and flood peak levels at Bannang Sata District (KM 16 from the dam) and at Yala province (KM 72 from the dam) are calculated.

MATHEMATICAL MODELS

Dam Breach Erosion Model

The governing equations are the reservoir water balance equation and a relationship between the erosion rate and the flow characteristics (Singh and Scarlatos, 1988).

The water balance equation can be expressed as follows:

$$A_s \frac{dH}{dt} = (I - Q) - Q_b \qquad (1)$$

where H = reservoir water level, I = reservoir inflow, Q_b = outflow discharge from crest overtopping, spillways and power house and A_s = reservoir water surface area. Also,

$$Q_b = u_b A_b \qquad (2)$$

and

$$u_b = \alpha_1 (H - Z)^{\beta_1} \qquad (3)$$

where u_b = mean outflow velocity, A_b = wet breach cross-sectional area, α_1 = empirical coefficient, β_1 = 0.5 for critical flow over a broad-crested weir and Z = breach bottom elevation. For the case of dam breaching (I-Q) << Q_b, combining Eqs. 1, 2 and 3 gives

$$A_s \frac{dH}{dt} = \alpha_1 (H - Z)^{1/2} A_b \qquad (4)$$

The erosion of breach cross-section dZ/dt is given by:

$$\frac{dz}{dt} = -\alpha_2 u^{\beta_2} \qquad (5)$$

where α_2 = empirical coefficient to be determined by model calibration. In this study a linear erosion case is considered, i.e., β_2 = 1. For a trapezoidal breaching

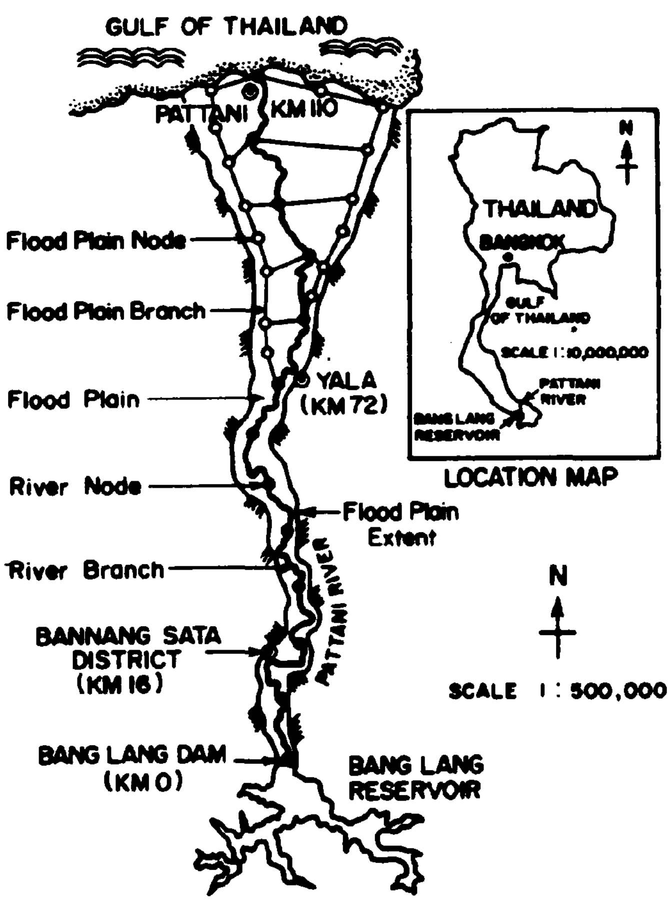

Fig. 1 Bang Lang dam and Pattani river.

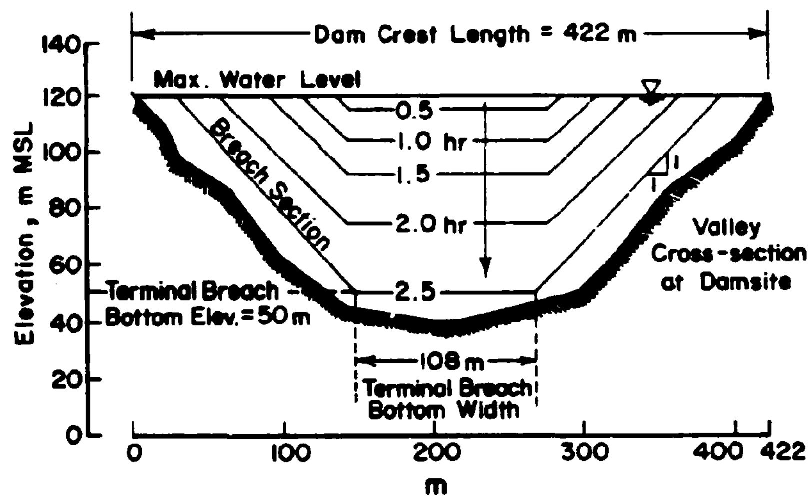

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Fig. 2 Valley cross-section and trapezoidal breach formation of Bang Lang dam.

shape with a constant bottom width b and a constant side slope s (Fig. 2), the following two equations can be obtained by integrating Eqs. 4 and 5 and inserting the initial conditions $H = H_o$ and $Z = Z_o$ at time $t = t_o$ (Sinthananopakhun, 1991), i.e.,

$$2s(H-Z) = \frac{[A_6 - b]\,[A_6 + b + 2s(H_o - Z_o)] - [A_6 - b - 2s(H_o - Z_o)]\,[A_6 + b]\exp[F]}{[A_6 + b + 2s(H_o - Z_o)] + [A_6 - b - 2s(H_o - Z_o)]\exp[F]} \quad (6)$$

$$\ln\left\{\frac{A_7 + i[s(H-Z)A_7]^{\frac{1}{2}}}{A_7 - i[s(H-Z)A_7]^{\frac{1}{2}}}\right\} - 2i\left(\frac{A_7}{A_8}\right)^{\frac{1}{2}} \tan^{-1}\left(\frac{s(H-Z)}{A_8}\right)^{\frac{1}{2}} = \frac{-i\alpha_1 t A_7^{\frac{1}{2}} A_6}{s^{\frac{1}{2}} A_s}$$

$$+\ln\left\{\frac{A_7 + i[s(H_o-Z_o)A_7]^{\frac{1}{2}}}{A_7 - i[s(H_o-Z_o)A_7]^{\frac{1}{2}}}\right\} - 2i\left(\frac{A_7}{A_8}\right)^{\frac{1}{2}} \tan^{-1}\left(\frac{s(H_o-Z_o)}{A_8}\right)^{\frac{1}{2}} \quad (7)$$

where $A_6 = (b^2 + 4\alpha_2 s A_s)^{0.5}$, $A_7 = 0.5b - 0.5\,(b^2 + 4\,\alpha_2\, s A_s)^{0.5}$,

$A_8 = 0.5b + 0.5(b^2 + 4\alpha_2 s A_s)^{0.5}$ and $F = A_6\,(Z - Z_0)\,/\,(\alpha_2\, A_s)$

The values of H and Z can be solved by trial and error from Eqs. 6 and 7.

<u>MIKE-11 Flood Routing Model</u>

The model is a one dimensional flood routing model and can be applied to the network of cells and links of the Pattani river and its flood plains as shown in Fig. 1. The basic equations are:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} - q = 0 \quad (8)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{\alpha Q^2}{A}\right) + gA\frac{\partial H}{\partial x} + \frac{g n^2 Q|Q|}{AR^{\frac{4}{3}}} = 0 \quad (9)$$

where A = channel flow area, n = Manning roughness coefficient, g = gravitational acceleration, H = water level, Q = discharge, R = hydraulic radius, α = momentum coefficient and q = lateral inflow. Eqs. 8 and 9 are transformed to a set of implicit finite difference equations using the well-known 6-point Abbott numerical scheme. The double sweep algorithm is used in solving the linearized finite difference equations for H and Q.

<u>Stability and Accuracy of MIKE-11 Model</u>

In order to obtain a stable solution of the finite difference scheme, the two following conditions have to be satisfied:

1) velocity condition, i.e. $V\Delta t/\Delta x \geq 1 \sim 2$, where V = velocity, Δt = time step and Δx = distance step between two computational locations. The velocity criteria expresses that Δt and Δx are to be selected so that the water will not be transported more than one space step per time step.

2) Courant stability: Though the 6-point Abbott finite difference scheme is an implicit scheme, the accuracy of the model will be poor if the Courant number, C_r is too large. In the MIKE-11 model, the following value of C_r is used, i.e., $C_r = (V+\sqrt{gd})\Delta t/\Delta x \leq 10 \sim 15$, where C_r = Courant number and d = depth. The Courant number expresses how many space steps a wave caused by a minor disturbance will move during one time step.

In this study the space step or distance step Δx = 1 km and the time step Δt = 2 minutes are used to obtain the required stability and accuracy of the results.

MODEL INPUT DATA

<u>Breach Erosion Model</u>

The required input data for the breach erosion model are the initial reservoir water level H_o = 119.5 m MSL, the initial breach bottom elevation Z_o = 118.5 m MSL, the terminal breach bottom elevation Z_b = 50 m MSL, the side slope of breach section s = 1:1, and the terminal breach bottom width b = 108.1 m. The discharge coefficient α_1 and the erosivity coefficient α_2 are determined by trial and error during model calibration.

<u>MIKE-11 Flood Routing Model</u>

For the MIKE-11 flood routing model, the input data are the cross-sections of the Pattani river measured at every 10 km interval. The flood plain cross-sections along both sides of the river are obtained from the contour map of scale 1:20,000 with a contour interval of 1 m. The river has many meanders between KM 0 (Bang Lang dam) and KM 70 and is located within a narrow valley. Between KM 70 and KM 110, the river is less meandering and having flood plains on both sides of the river. The upstream boundary condition is the computed breach outflow hydrograph from the Bang Lang dam and the downstream boundary condition is the constant mean sea level. A steady state initial flow condition is assumed. The distance step of 1 km and the time step of 2 minutes are used. The input model parameters are the Manning n of the river and the flood plains.

The Manning n, of the Pattani river obtained by the model calibration using flood data in 1969 (Sinthananopakhon, 1991) is found to be between 0.02 and 0.03. For the flood plain areas, the Manning n is estimated according to the vegetation condition (Chow, 1959), and it ranges from 0.8 to 1.0.

RESULTS AND DISCUSSIONS

Results and Sensitivity Analysis of Dam Breach Erosion Model

The discharge coefficient α_1 and the erosivity coefficient α_2 in the dam breach erosion model are determined by model calibration based on the previously estimated peak breach outflow of 145,000 m^3/s (EGAT, 1985) and the time to peak outflow of 2.5 hours (Mc Donald and Langridge-Monopolis, 1984). It is found that α_1 = 1.7 and α_2 = 0.00098. Sensitivity analysis show that when α_1 is decreased the peak outflow is reduced and its time of occurrence is delayed. When α_2 is reduced, the breaching rate is also reduced. The increase in the terminal breach bottom width b or the breach side slope s produces a higher peak outflow discharge. The reservoir water level depletion and the breach bottom elevation at various times are shown in Fig. 3.

Results and Sensitivity Analysis of MIKE-11 Flood Routing Model

The discharge hydrographs at every 20 km stations computed for the MIKE-11 model are presented for a period of 24 hours in Fig 4. The flood peak elevation envelope and the arrival time of flood peak and flood wave front along the river are presented in Fig. 5. It can be sen that the flood peak elevation at KM 0 reduces significantly in the first 10 kilometers. The flood peak elevations are mostly above the river bank elevations except within the reach from KM 85 to KM 110 where almost no overbank flow occurs. At Bannang Sata district, the arrival times of wave front and flood peak after the dam breaching starts are found to be 1.5 hours and 5 hours respectively. At this station, a maximum flood depth of 19 m could be expected with a flooding duration of 26.5 hours. At Yala province which is 72 km downstream of the dam, it is found that the flood wave front and the flood peak will arrive at the province 6.5 hours and 12.5 hours approximately after the dam breaching starts respectively. The overbank flow duration is found to be 25.5 hours. Fig. 6 show the transverse water surface profile of cross-sections at KM 72 at various times respectively. As can be seen, during the rising flood, the water level in the river is higher than in the flood plains. This means that the flow spreads from the river into the flood plains. The reverse

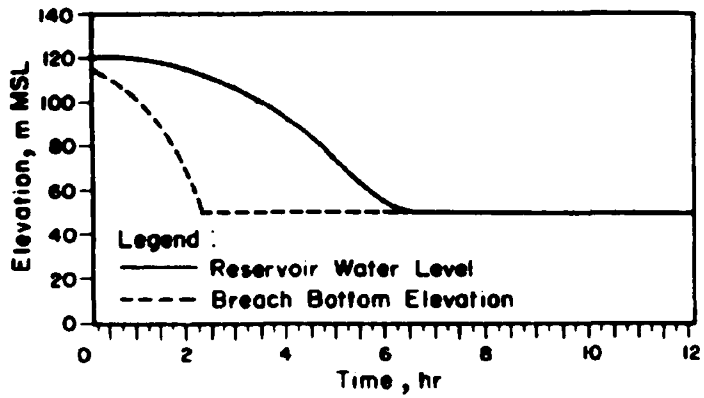

Fig. 3 Computed lowering reservoir water level and breach bottom elevation.

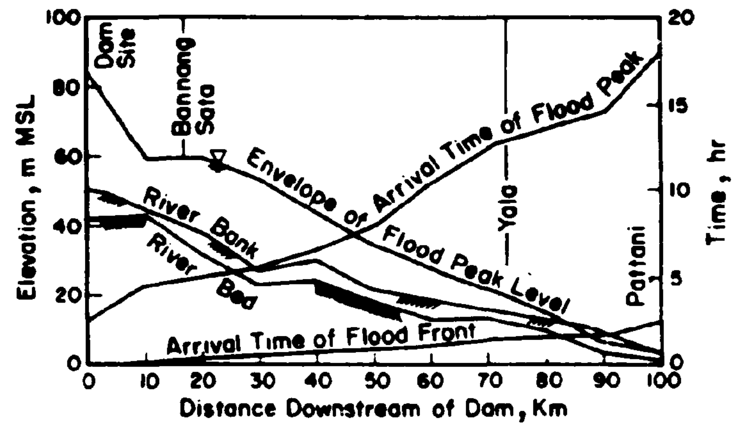

Fig. 5 Envelope of flood peak level, river bed and bank profiles, arrival times of flood peak and flood front.

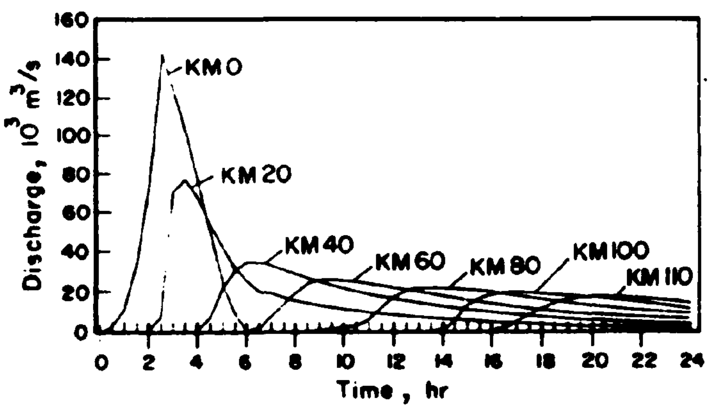

Fig. 4 Computed discharge hydrographs at various stations along Pattani River.

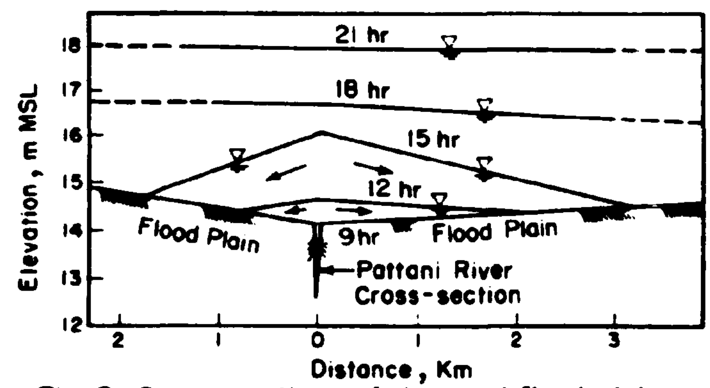

Fig. 6 Cross-sections of river and flood plains and transverse water surface profile, at Yala 72 km from the dam.

condition occurs during the flood recession period. The relevant parameters of flood routing model are the Manning roughness coefficients of the river and flood plains. According to the sensitivity analysis, it is found that the effect of the river Manning n is more significant than that of the flood plain Manning n. When the Manning n of the river and flood plains are increased by 20%, the peak flood levels are increased by about 0.05 m to 0.1 m.

CONCLUSIONS

The results of this study can be concluded as follows:

1. The breach outflow hydrograph of the Bang Lang dam is calculated based on a trapezoidal breaching shape with the discharge coefficient α_1 = 1.7 and the erosivity coefficient α_2 = 0.00098. The two coefficients are determined by calibration based on the peak outflow of 145,000 m^3/s (EGAT, 1985) and the time to peak outflow of 2.5 hours (Mc Donald and Langridge-Monopolis, 1984). Sensitivity analysis shows that the relevent model parameters are the discharge coefficient α_1, the erosivity coefficient α_2, the breach side slope s, and the terminal breach bottom width b.

2. The arrival times of wave front and flood peak after the dam breaching starts are found to be 1.5 hours and 5 hours at Bannang Sata (KM 16 from the dam) and 6.5 hours and 12.5 hours at Yala province (KM 72). The maximum flood depth at Bannang Sata is 19 m. However, there is no flooding at the Yala province because it is located above the river banks. Sensitivity analysis is carried out to determine the effects of Manning n of the river and flood plains.

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SECOND US-ASIA CONFERENCE ON
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POST-CYCLONE RELIEF AND REHABILITATION

Haroun Er Rashid
POUSH, 5/7, Sir Syed Road Block-A
Mohammadpur, Dhaka, Bangladesh

ABSTRACT The coastal areas of Bangladesh are frequently battered by cyclones and their associated storm surges. With a rapidly increasing population large numbers of people are now settling on land which are hazardous to habitation. This has increased human and livestock deaths, and damage to property. In the absence of any meaningful Land Use Policy the government has not been able to restrict habitation in the hazard zone. As a result post-cyclone relief and rehabilitation requirements have increased beyond the capacity of the local administration, and every time there is a major storm the central government has to play a major role and often international effort is required to save the situation. Relief usually reaches 12 to 48 hours after the storm. This causes immense suffering because food stocks have been washed away or damaged, and sources of potable drinking water are grossly inadequate. Aerial relief has been tried out but this is very expensive and sometimes counter - productive. The only viable solution seems to be to strengthen local administration and local NGO's to distribute relief materials immediately after the storm has abated. Similarly with rehabilitation, outside effort is usually expensive and the solution may be a system by which donors and international relief agencies use the District administration and Local NGO's as their ground-level partners.

INTRODUCTION

Over the past three decades, cyclonic storms and surges have killed more people in Bangladesh than in any other country. In terms of human misery this disaster-prone country suffers more from cyclonic storms than from floods and droughts. Yet, because the coastal area is remote from the centres of power and the people are very poor and socially marginalised, they receive less attention than those affected by riverine floods. In the unprecedented riverine floods of 1987 and 1988 a couple of thousand person died, but within the space of a few hours on the night of 29th - 30th April 1991, one hundred and forty thousand persons perished from the surge accompanying a huge cyclonic storm.

Definite record of large cyclonic storms go back to 1592, and many such storms are known to have hit the coast in the past two centuries, but detailed records and reliable meteorological data seem to be available only from 1960. In that year two severe cyclonic storms of Hurricane intensity (with wind speeds over 118 km. per hour) hit the Chittagong coast. Facilities in Chittagong port were badly damaged and ocean-going vessels were driven up on to the beach north of the city. Incidentally dismantling of these cast-up ships was the begining of a profitable ship-breaking industry which continues till today.

During the period 1960-1991 sixteen severe cyclonic storms have crossed the coast of Bangladesh of which ten have caused very extensive damage (Table 1). In this period some 500,000 to 700,000 persons perished in these storms, mostly due to the accompanying storm surges which can be up to nine metres above mean sea level (IEB 1991, Matin 1992).

Table 1

MAJOR CYCLONES WHICH AFFECTED BANGLADESH SINCE 1960

Date	Max. wind speed (km./hr)	Surge height in metre	Human deaths
09 Oct. 1960	162	3	3,000
30 Oct. 1960	210	5-7	5,149
09 May 1961	146	3-4	11,466
30 May 1961	146	7-9	N.A.
28 May 1963	203	4-6	11,520
11 May 1965	162	7-8	19,279
12 Nov. 1970	223	7-9	300,000
25 May 1985	154	3-5	11,069
29 Nov. 1988	162	2-4	2,000
29 April 1991	225	7-9	139,000

The population of Bangladesh has doubled over the past twenty five years and intense economic competition has forced millions of people to settle on natural disaster hazard zones along rivers and the coast. In the absence of any meaningful Land Use policy the Government has not been able to restrict settlement in these high risk areas. Despite the great increase in the number of people living in risky areas the proportionate numbers of deaths may have come down because of measures taken over the past two decades. However there is a need to reduce damage from these storms through "programs that control the type, density, and location of coastal settlements" (Clark 1990).

The big difference in the less of life in 1970 and 1991 is being attributed to a much improved cyclone warning and evacuation system. There were new 232 cyclone shelters, built by the Bangladesh Red Crescent Society or by the Government with World Bank assistance, with having saved the lives of a quarter of a million people during the cyclone of April 29-30th 1991.

RELIEF AND REHABILITATION MECHANISM

Over the years the Government of Bangladesh has evolved a mechanism for reaching relief to the affected people immediately after the storm winds subside (GOB 1985). There is a Ministry of Relief and Rehabilitation (MORR) which is probably not unexpected in a country so prone to natural hazards. This Ministry usually has on hand foodstocks assigned to it specifically for relief. These are kept in Ministry of Food storages (godowns) and can be released either as grants to individuals or sold (usually at a subsidy) to local shopkeepers. MORR also keeps on hand small quantities of lentils (pulses) and edible oil for emergency distribution. As soon as there is notification of a severe storm in the Bay of Bengal MORR allocates funds for eventual purchase of other relief material, such as clothing, corrugated iron sheets for roofing and bisciuts. The Government also relies heavily on the Red Crescent Organisation on disseminating storm warning, evacuating people from hazard zones, and distributing relief immediately after a damaging storm. The Bangladesh Red Crescent Society (BRCS) which is affiliated to the International Federation of Red Cross and Red Crescent Societies, has a good track record. It is relied on both by the Government and the people of the coastal areas to provide pre-cyclone warning and, in some cases, shelter, and post-cyclone relief, and in some cases, rehabilitation. The BRCS has 20,000 volunteers in the coastal areas, and they look after, to the extent that their resources allow, no less than 2 million people. It has been estimated (MCSP 1992) that there are over 4 million people in the High Risk Area of the coastal zone, and therefore half of the people at risk have no BRCS coverage.

Due to the very magnitude of the problem, with up to 20 million people likely to the affected by wind damage and over 4 million more to be severely affected by wind and surge damage, post-cyclone relief and rehabilitation cannot be handled by the Government alone. In fact hundreds of NGOs rush workers and material to the affected areas after a severe storm, but most of them are daunted by the destruction they see, the lack of roads, the shortage of boats, the inclement weather and the difficulty of living in an area strewn with dead humans and animals. Most of them quickly dispose of whatever they bring by the roadside and the stricken people gather along the major paved roads knowing that relief is unlikely to reach the interior during the first few days. Those who live along the roads often receive far more than they need, and those in the interior areas, difficult of access, barely survive in an environment vitiated by polluted drinking water and rotting bodies. Relief usually reaches them 12 to 48 hours after the storm, and this obviously causes immense suffering because food stocks have been washed away or damaged, and sources of potable drinking water are grossly inadequate (GOB 1991).

The Civilian authorities usually call in the military to help in these situations. The Air Force flies in helicopters to reach places completely cut off and air-drops food and clothing to people stringed on islands. Unfortunately these air-drops are often garnered by groups organized by a few families, leaving the majority to fend for themselves. This situation can be countered only if the civilian authorities have adequate transport vehicles and funds to place officials to handle relief operations within hours of the storm. This is usually not the case, and after the April 1991 cyclone some communities did not receive any assistance for as many as five days.

ROLE OF LOCAL NGOs

Very early assistance is absolutely essential because the drinking water supplies are affected and large numbers of people are totally shelter-less. It is reported that many die after the cyclone due to diarrhoeal disease and exposure to the rains which continue after the storm has passed. In such a situation only the local administration and local NGOs can be of help. Local administration consists of elected representatives (union chairman and ward members) and minor officials of various ministries. Local NGOs are very few, because most of them are daunted by the physical conditions and extreme difficulty of access. There is, for example, only one proper jetty (landing terminal) along one thousand kilometres of coast (excluding Chittagong City). Local NGOs therefore tend to avoid the High Risk Area, which extends between five to ten km. from the coast. However a few medium-seized development NGOs, such as Nijera Kori, HEED and POUSH, have gone into these areas, and the bigger NGO such as BRAC, PROSHIKA and Gono Shasto Kendra (GK) are begining to take an interest. A network of local NGOs and local administration, backed by the Government (particularly MORR) and International NGOs, could assist the BRCS in drawing up and implementing both emergency relief and rehabilitation work and community-level development work. In fact, on a long-term view, the rehabilitation work, which consists of land reclamation, re-stocking of fish ponds, re-building of huts, repair of roads etc. is development work, and as such it should form part of a coastal zone regional development plan (NEMAP 1991).

FUTURE NETWORK

Looking ahead it seems certain that BDRCS will have to continue to play a major role. It has a Disaster Preparedness Programme, but this is not adequately funded. As pointed out earlier they are providing pre-cyclone warning, evacuation and shelter and post-cyclone relief and rehabilitation assistance to, at best, one-third of the affected population. Whether they can cover the entire population in the High Risk Area just through an increase in funding is not clear, because there are problems of internal management and also coordination with Government officials, NGOs and local administration. More funding for BDRCS is absolutely essential, but even then, local administration and local NGOs will have an important role to play. Surveys carried out in 1991 by POUSH (a local NGO) and in early 1992 by the World Bank/UNDP funded Multipurpose Cyclone Shelter Programme, showed that elected representation at the grassroots level (ward members) provided prompt access to information and a channel to relief and rehabilitation material. They have not been trained in warning dissemination, evacuation procedures or post-cyclone handling and recording of the flow of relief and rehabilitation goods and services. During the 1980s the military establishment played an overwhelming role in the distribution of relief material. After the 1991 cyclone they still played an important part but elected they still played an important part but elected representatives and district-level officials had the major role. However, lines of responsibility were not clearly demarcated and this created some confusion. In particular it was seen that some International NGOs, in tandem with Bangladeshi NGOs with no experience in the coastal area, distributed relief without providing any information to the district administration, thus imbalancing the distribution equity, and promised expensive cyclone shelters and housing as part of the rehabilitation effort, but ultimately failed to come through with their promises.

CONCLUSIONS

It is not being argued that the Government should be they only channel of assistance, but that there should be much better coordination of all relief and rehabilitation effort. To this end coordination cells may be set up, more or less on a permanent footing, at central, district, thana and union levels. These cells should consist of representatives of BDRCS, locally based NGOs, elected representatives and government officials. One of their major functions would be to impart training and carry out awareness campaigns for post-cyclone access to relief and planning of rehabilitation so that it forms part of long-term development investment. Development efforts in the coastal area must take into account natural hazards. As many Anderson has aptly put it : "Disasters occur most often in poor countries and cause the most suffering among poor people. These are precisely the societies for whom development is most urgently needed. Yet by ignoring likely disasters, many development efforts do nothing to decrease the likelihood of disasters, and many actually increase vulnerability to them" (Anderson 1990). If Post-cyclone relief and rehabilitation is meshed with local development needs the benefits would possibly be far more lasting.

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SECOND US-ASIA CONFERENCE ON
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FLOOD ACTION PLAN OF BANGLADESH : A CRITICAL REVIEW

Ainun Nishat
Department of Water Resources Engineering
Bangladesh University of Engineering & Technology
Dhaka, Bangladesh

ABSTRACT Flood is a recurring phenomena in Bangladesh. Recently Bangladesh developed an approach of formulating a long term plan, which would provide a comprehensive solution to the recurrent flood problem and create a climate for sustainable program for economic and social improvement. The approach called the Flood Action Plan (FAP) is to be technically sound, socially acceptable, environmentally sustainable and economically feasible and financially implementable. The key elements of the FAP are the concept of control flooding which would allow the desired level of inundation over flood plain but prevent damages, the approach of integrating structural and non-structural flood mitigation measures, and the concept of compartmentalization for effective flood management. The Action Plan's present activities, covering the five year period 1990-1995, is the first of several stages in the project formulation process. Over the last two years regional plans are being formulated at prefeasibility level. A number of supporting studies are being undertaken to improve data base, develop understanding of the beneficial as well as adverse impacts of flood control projects through evaluation of completed projects and conduct pilot level activities to try out the new concepts and approaches. The FAP is taking a cautious approach in project formulation and their subsequent implementation which will lead towards development of an effective flood disaster mitigation program.

INTRODUCTION

Flood is a recurring phenomena in Bangladesh. While the monsoon dominates the rainfall pattern, flooding in the country is the result of a complex series of factors. Most of

Bangladesh is located within the flood plains of the three great rivers, the Ganges, the Brahmaputra and the Meghna and their tributaries and distributaries. These rivers systems drain a total catchment area of about 1.72 million square kilometers lying in India, China, Nepal, Bhutan and Bangladesh. Only 8 percent of the catchment area lies within Bangladesh. As a result huge inflows of water enter into the country, and Bangladesh has no control over it.

The various factors that singly or in combination create flood in Bangladesh are the huge monsoon inflows of water that come from upstream catchment areas coinciding with heavy monsoon rainfall over Bangladesh, low floodplain gradient, congested drainage channels, and the influence tides and storm surges. In Bangladesh, four major types of flood occur, namely monsoon floods from major rivers, local flooding due to heavy and intense rainfall, flash floods in the eastern and northern rivers, and floods due to tidal surges in the coastal areas. In fact two thirds of Bangladesh is vulnerable to flood and almost every year between one four to one third area goes under water.

FLOOD MITIGATION STRATEGY OF BANGLADESH

During pre-colonial days extensive flood protection efforts mostly through embankment were practiced in Bengal (eastern half of which is now Bangladesh). During the colonial rule of about 200 years there were no effort to maintain the embankment and people were led to accept flood as a part of life and suffer with it. After the major floods of 1954 and 1955 the first systematic study of the flood problem was carried out by a UN-sponsored team who recognized the importance of flood control and major works were recommended as a pre-requisite to advances agricultural production. A water Master Plan was prepared in 1964 which propose a number of large projects combining the function of flood control, drainage and irrigation. Subsequently, some major embankments were build along the main rivers. Polders have been completed covering the entire coastal belt which is now safe from tidal inundation (not from cyclonic storm surge). A few large flood control projects as recommended in the Master Plan have been completed and some projects are still under implementation.

However, by the early 1970's, a shift in development strategy based on small-scale, quick-yielding projects rather than major flood control works led to ground water development through tubewells, introduction of low lift pumps to harvest available service water and spread of dry season cropping. Thus, the flood control debate receded into the background until the floods of 1987 and 1988.

Strategies and option for flood control in Bangladesh have been debated for many years. On one side the argument that periodic flooding in Bangladesh is largely unavoidable because the works needed to eliminate flood from the rivers are technically and economically unfeasible and, moreover, tend to create as many problem as they solve. Those who support this argument believe there is considerable scope to built on the ability of the Bangladeshi farmer to cope with and recover from the annual floods. On the other hand is the widely held view that the country cannot be at the mercy of floods forever and that all the major rivers must eventually be contained so that the floods are safely passed through Bangladesh to the ocean (UNDP, 1989). Those who support this argument believe that this would reduce the risk associated with economic activity on the flood plains and increase the economic growth of the country.

The severity of the recent flood led the Government of Bangladesh to look for a flood plan which would, in the long term, provide a comprehensive and permanent solution to the recurrent flood problem and thus create climate for sustainable program of economic and social improvement. The plan is to be technically sound and implementable, socially acceptable, environmentally sustainable, economically viable and financially justifiable.

Soon after the 1988 flood, the government in collaboration with UNDP undertook a comprehensive review of the planning approach of on going activities related to flood mitigation and work began on a flood policy study and a flood preparedness study. A set of 11 guiding principles was developed (shown in table 1) which now forms the basis of flood mitigation strategy of Bangladesh.

Table 1. THE GUIDING PRINCIPLES

(Source : UNDP, 1989)

1. Phased implementation of a comprehensive Flood Plan for :
   - protecting urban and rural infrastructure,
   - controlling flooding to meet the needs of agriculture, fisheries, navigation, urban

flushing and annual recharge of surface and ground water resources;

2. Effective land and water management in protected and unprotected areas through 'controlled flooding' as against flood control and compartmentalization;
3. Measures to strengthen flood preparedness and disaster management;
4. Improvement of flood forecasting an early warning;
5. Safe conveyance of the large cross border flows to the Bay of Bengal by channelling it through the major rivers with the help of embankments on both sides;
6. Effective river training for protection of embankments, infrastructures and towns;
7. Reduction of flood flows in the major rivers by diversion into major distributaries and flood relief channels.
8. Improvement of conveyance capacity of river network through channel improvements and appurtenant structures to ensure efficient drainage and to promote conservation and regulation of flows;
9. Development of flood plain zoning where feasible and appropriate;
10 Coordinated planning and construction of all rural roads, highways and railway embankment with provision for unimpeded drainage; and
11 Encouraging maximum possible popular participation by beneficiaries in the planning, design, and operation and maintenance of flood control and drainage works.

Notable features of these guiding principles are effective differentiation between "inundation" and "flooding" (Nishat, 1989); recognition of different level of flood protection for urban and rural areas; introduction of the concept of "controlled flooding" and compartmentalization; integration of structural and non-structural option of flood mitigation inclusion of river training and bank protection as part of future flood plans, incorporation of environmental considerations in engineering and planning; and participatory approached planning and project management.

THE FLOOD ACTION PLAN

Almost parallel to the flood policy study three major studies were also completed by professionals from Japan, France and the USA (a study by Chinese experts has recently been completed). The World Bank in association with national and expatriate experts who had carried out the aforementioned studies prepared a Flood Action Plan (FAP) which is now under implementation (World Bank, 1989). The FAP has been based on the 11 guiding principles and adopted a stage approach, following pilot trials, in plan formulation. The FAP has been endorsed by a general meeting of donors, held in December 1989, in London. The present phase Action Plan, covering the five-year period 1990-1995, is to be seen as the first of several stages in the development of a comprehensive system of flood control and drainage works designed to meet the long term objectives of sustainable development of water resources and flood management. It would be carried out in parallel with agricultural development and a program of non-structural measures such as flood forecasting, flood warning, flood preparedness and disaster management. The main elements of the Action Plan are regional studies for the six regions, into which the country has been divided, based on the concept of control flooding aimed to prevent damaging level of flooding but trapping the benefit of normal level of inundation and compartmentalization for effective water management. In addition a number of supporting activities had been initiated for improvement of database essential for planning including mapping, development of better understanding of impact of floods and flood control works, development of appropriate hydro-dynamic models and carrying out of pilot level activities to evaluate and finalize new concepts and approaches in flood management.

The FAP projects to be recommended through the regional studies are to ensure that all the proposed embankment and other physical works take into account the environmental consideration. All regional studies are to consult and fully take into account the interests and views of the intended beneficiaries and those affected by the project, at various stages of design and implementation to ensure that they are committed to the projects; this commitment will help secure the long-term operation and maintenance of the investments.

The FAP's overall approach of the various regional studies now aim for integrated water management program designed to address not only flood mitigation but also drainage and irrigation requirements. Institutional development program forms part of the Action Plan. This program would attempt to strengthen the implementation capacity of the relevant government agencies, the local consultants and contractors. The FAP has established simple but effective coordination and follow-up mechanisms both within the Government and among donors.

The elements of the FAP are summarized in Table 2. Broadly the activities are divided into 11 main components and 15 supporting studies and pilot projects. In addition to these studies, a " Guidelines for Project Assessment " to provide a common agreed basis for evaluation of all project components have been prepared.

Table 2. COMPONENTS OF FLOOD ACTION PLAN

FAP No.	Activity	Donor(s)
Main Component		
1.	Brahmaputra River Training Study	IDA
2.	North West Regional Study	UK, Japan
3.	North Central Regional Study	EEC, France
3.1.	Jamalpur Priority Project	France, EEC
4.	South West Area Water Management Study	UNDP, ADB
5.	South East Regional Study	UNDP, IDA
5B.	Meghna Estuary Study	Netherlands, Denmark
6.	North East Regional Study	Canada
7.	Cyclone Protection Project	EEC
8A.	Greater Dhaka Protection Project	Japan
8B.	Dhaka Integrated Town Protection Project	ADB
9A.	Other Towns Protection Project	ADB
9B.	Meghna Left Bank Protection Project	IDA
10	Flood Forecasting and Warning Project	UNDP, Japan, ADB
11.	Disaster Preparedness Project	UNDP
Supporting Studies		
12.	FCD/I Agricultural Review	UK, Japan
13.	Operation and Maintenance Study	UK, Japan
14.	Flood Response Study	USA
15.	Land Acquisition and Resettlement Study	Sweden
16.	Environmental Study	USA

17.	Fisheries Study and Pilot Project	UK
18.	Topographic Mapping	Finland, France, Switzerland
19.	Geographic Information System	USA
20.	Compartmentalization Pilot Project	Netherlands, Germany
20/21	Bank Protection and Active Flood Plain Management Project	Germany, France
23.	Flood Proofing Pilot Project	USA
24.	River Survey Program	EEC
25.	Flood Modelling and Management Project	Denmark, France, Holland, UK
26.	Institutional Development Program	UNDP, France

PROGRESS SO FAR

Works on all the 26 components have commenced but sound are lagging behind in progress mainly as a result of late start (mostly due to procedural wrangles). Among the late starters are South West Regional Study (FAP-4), Flood Preparedness Study (FAP-11), Fisheries Study and Pilot Project (FAP-17), River Survey Program (FAP-24), Bank Protection and Active Flood Plain Management Project (FAP-21/22) and Institutional Development Study (FAP-26). Works on Brahmaputra River Training Study (FAP-1), Greater Dhaka Protection Project (FAP-8A), Dhaka Integrated Town Protection Project (FAP-8B), Other Town Protection Project (FAP-9A), Meghna Left Bank Protection Project (FAP-9B), FCDI Agriculture Review (FAP-12), O & M Study (FAP-13), Land Acquisition and Resettlement Study (FAP-15) and Flood Proofing Project (FAP-23) have either been completed or nearing completion and follow-up activities/studies are being planned. Studies and activities North West Regional Study (FAP-2), North East Regional Study (FAP-6), Flood Forecasting and Warning Project (FAP-10), Flood Response Study (FAP-14), Environmental Study (FAP-16), Topographic Mapping (FAP-18), Compartmentalization Pilot Project (FAP-20) and Flood Modelling and Management Project (FAP-25) have made significant progress in their work.

Regional studies are being carried out at prefeasibility level. However when a project component is firmed up it progress to a feasibility level study. In March 1992 a review of up to date progress of work was made and further need of synchronization of the supporting studies with the main project components was emphasized. Flood Plan Coordination Organization (FPCO) of the Ministry of Irrigation Flood Control and Water Development is supervising the implementation of the FAP. The FPCO is advised and guided by a Panel of Experts which includes both national and expatriates. Progress of work and coordination between various components of the FAP is reviewed regularly by an interministerial review committee.

ISSUES BEING RAISED BY CRITICS OF FAP

Ever since launching of the FAP, many experts and professional have been critical of various aspects of it. Even a government appointed Task Force to review the Flood Action Plan sounded skepticism about the success of the exercise (Task Force, 1991). The Task Force was highly critical of the conventional approach of flood control projects because of many adverse environmental impacts and frequent failure of the structural measures. While they agreed to the approach of the pilot level trials of new approaches of the FAP, they were apprehensive that all lessons may not be incorporated into the project components. The March 1992 Conference which was participated by representative of donors, professionals and experts in relevant fields including engineering, sociology, economics, geography, fishery, environmental and disaster management, and representative of government and non government organizations and agencies. This was an opportunity for critically reviewing the approach of the FAP and put it on a revised track when necessary.

This conference as well as the Task Force raised the following key questions :

- are embankment desirable technical options ?
- should Bangladesh not learn to live with flood ?
- will not the damage to open water fisheries be irreversible ?
- are environmental issues being properly addressed to ?
- are the non structural options being given due emphasis and priority ?
- can Bangladesh cope with the flood problem from within her boundaries ?

- is synchronization of work of various components being received ?
- how the huge funds required for project implementation and O & M will be met ?
- are FAP activities being properly guided ? and
- is FAP a nationally accepted program ?

In the Conference, FPCO and the Panel of Experts responded to these issues and it may be said that the approach of the FAP in finalizing a flood plan for Bangladesh received due support and activities under the FAP are continuing. Construction of embankments as the major mode of containing flood waters within the river channel faces the following main criticism : loss of land due to land acquisition, breaches in embankment due to river erosion and thus total failure of the investment, adverse impact on fish growing in the flood plain and possible higher incidents of flooding at downstream locations. Failure of embankments during high floods is common phenomenon as construction of embankment is often of poor quality and losses are higher in such situation compared to no embankment condition. However, embankments are found as the cheapest as well as locally accepted technology. It is expected that practice of controlled flooding will be able to overcome the environmental adverse effect including the problems regarding open water fisheries. Scope of floods reservoir, flood bye-pass and detention basins as alternative options are not feasible in Bangladesh (except for Chittagong Region) as the country is almost flat.

Living with floods and doing nothing is not an option acceptable to the people and the national government. However, about one third of the regularly flooded areas will remain without any protective measures and for those areas adaptation of non structural measures are the only choice. In the regional studies it is expected that proper integration of structure and non structural options will be made.

If Environmental Impact Assessment (EIA) (for which FAP has already prepared a manual and a guideline) is carried out properly then most of the questions pertaining to environmental adverse concerns will be addressed to. Inclusion of environmental impact assessment has been made mandatory for all the individual plan components.

Most of Bangladesh being part of a flat land and since most of the potential reservoir site are far away, and in other countries, and other structural options are not viable inside Bangladesh, it has been the approach of FAP that the main effort for flood mitigation will be carried out inside Bangladesh. However, efforts in improving flood forecasting by making available hydrological data from upstream areas tieing up of embankments at the border, through regional cooperation, are under progress.

Synchronization of the various components of FAP has not been possible and is admitted to be a major drawback of the program. Ideally all supporting studies should have been taken up together with the pilot project and then the regional studies should have proceeded. However it's expected that by the time the regional studies progress to feasibility level most of the supporting studies will be complete and appropriate modification in the plans will be possible. Need for the effective flood control in Bangladesh need not be established again but what is being debated is the planning approach. Total flood control is neither feasible nor desirable. In fact the aim of flood control projects already completed and presently under implementation, based on the Master Plan of 1964, set their approach to a goal of total flood control and these are the projects that are being criticized. These projects will be remodel and rehabilitated under the FAP.

The question of availability of funds for implementation of projects that will emerge from regional studies has not been worked out. In fact the outlay of fund that will be required is not known as studies are still in progress. However the present level of funding now available for water development sector may not adequate. One of the aims of the FAP is to involve beneficiaries in O & M and possible mobilization of local resources for maintenance work is being worked out.

The questions of coordination of FAP projects components are valid. The conference observed that coordination as well as the level of guidance provided to the various groups responsible for project components are not satisfactory.

CONCLUDING REMARKS

Water resources development including flood management has been and will continue to be a key factor in the economic development of Bangladesh. Failure to utilize the water resources in an integrated, balance and comprehensive manner will not only cause stagnation in growth, specially in agriculture sector but also will give rise to many environmental problems.

A rhythm in the annual water cycle dominates life in Bangladesh : excessive water during the monsoon causing floods and insufficient water during the dry season creating drought like situation. These two extremes influence the planning for water resources development in Bangladesh requiring effective measure in flood control, irrigation and drainage. Development in irrigation sector together with flood control and drainage infrastructure, in areas where it has already been completed, has created a regime when other agricultural inputs may be effectively utilized to enhance the yield rate.

Though FAP initially had put their main focus on flood mitigation it is now shifting towards development of an integrated water management plan for the country. Two words in FAP have been wrongly coined; they are "action" and "plan". In fact based on a policy (contain in the guiding principles, which however need to be reviewed in light of the insight gathered in the initial phase of FAP) a plan is under preparation. It will then be organized into a phased implementation program. Present action is mostly limited to pilot level activities aimed to refine and establish the planning strategy for proper incorporation of the guiding principles. It is expected that FAP will establish many new dimension in the approaches of effective flood management.

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F10-1

SECOND US-ASIA CONFERENCE ON
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FLOODS EXPECTED DUE TO GLOBAL WARMING

Ricardo Harboe and Zahoor Ahmad
Division of Water Resources Engineering
Asian Institute of Technology
Bangkok, Thailand

ABSTRACT Global warming will affect future floods therefore, careful consideration of all parameters involved is necessary. Changes in temperature, precipitation and solar radiation are given by three global circulation models (GFDL, GISS and UKMO) assuming a scenario of doubling of carbon dioxide (CO_2). The effects of these changes on runoff can be predicted using a rainfall-runoff model like the Sacramento Model which considers all these parameters. Calibrating of this models present nevertheless large difficulties so that results can only be considered as preliminary.

Some results show increase in floods of a certain annuity due to increased precipitation. In other cases the increased temperature, solar radiation and potential evapotranspiration decrease runoff and soil moisture. Definite conclusion as to weather floods are increased or decreased by global warming cannot be drawn at this stage.

A small catchment (Maetaeng River) in northern Thailand served as a case study. This tropical catchment (1765 km^2) is in natural condition, without human influence. The maximum flow is about 700 m^3/s in rainy season; average flow is around 22 m^3/s. The mean annual precipitation is 1350 mm.

Future studies should try to predict more accurately the meteorological changes and the dates when they will occur. Further, improved rainfall-runoff modelling should yield more reliable prediction of floods.

INTRODUCTION

The effect of global warming on floods can only be studied through detailed distributed or lumped rainfall-runoff models. This paper presents such a study based on given information on global warming effects on the atmosphere. This information is extracted from three projection models: the Geophysical Fluid Dynamic Laboratory (GFDL) model, the Goddard Institute for Space Studies (GISS) model and the United Kingdom Meteorological Office (UKMO) model.

The changes in temperature are given by the three Global Circulation Models (GCM's) as well as changes in precipitation and solar radiation. With these information, the changes in potential evapotranspiration were calculated using the Penman method.

The Sacramento rainfall-runoff model was selected for the study because it yields better results than other models in the calibration and verification phases, i.e., in the present conditions of carbon dioxide (CO_2) contents in the atmosphere (1 x CO_2). For the prediction phase, a doubling of CO_2(2 x CO_2) was assumed by all GCM's. Nevertheless no clear indication is given regarding the time span it will take for the earth's atmosphere to double it's CO_2 contents, since this will strongly depend on policies undertaken by the countries regarding CO_2 emissions.

THE SACRAMENTO MODEL

This model was developed by Burnash and others (1973) to forecast runoff in the Sacramento River Basin (California). It is a deterministic, lumped type parameter model, which treats the whole catchment as one unit. It contains four conceptual storage zones which include an upper free and tension water zones and lower tension, free primary and secondary zones. Transfer of water between the soil moisture zones is governed by the relative states of the zones (water contents) and the model parameters according to set algorithms. Precipitation is split between direct runoff, if the catchment has an impervious surface, and upper free and tension water. Only evaporation can remove the tension water but free water can either contribute to surface runoff or be transferred to the lower tension and free water zones by percolation. The lower free water zones combine to produce a non-linear base flow recession while the lower tension zone is depleted only through evapotranspiration.

Calibration of the model was based on 6 years of continuous record (1956-61), which include wet, dry and medium years. Verification was performed with 19 years of historical record (1962-1980).

THE MAETAENG RIVER BASIN

The Maetaeng river is located in northern Thailand (north of Chiang Mai) between 19° and 20° latitude and between 98° and 99° longitude. The area experiences tropical monsoon climate. The rainy season is brought about by a south-west monsoon from mid-May to October. The north-east Monsoon produces the winter season, generally from late October to early February. The summer season or dry season is characterized by relatively high temperature, increasing humidity and cloudiness. Approximately 90% of the annual rainfall occurs during the rainy season. Mean monthly relative humidity varies from 58% in March to 83% in August and September. The flow in the catchment is undisturbed by human activity.

GLOBAL CIRCULATION MODELS

The output of three global circulation models for the coordinates of the Maetaeng Catchments include:

- mean monthly temperature shift (Table 1)
- precipitation scaling factor (Table 2)
- solar radiation scaling factor (Table 3)

All these outputs correspond to the doubling of CO_2 scenario (2 x CO_2) with respect to the present situation (1 x CO_2).

Table 1 Mean Monthly Temperature Shift

Temperature Shift (°C)												
GCM	Jan	Feb.	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
GFDL	2.20	3.31	4.66	4.41	3.06	2.78	1.53	2.36	2.55	2.36	2.82	0.98
GISS	6.55	5.03	5.33	5.09	4.81	3.60	2.96	2.94	3.96	3.68	4.14	5.29
UKMO	2.57	4.30	6.15	4.00	3.11	2.59	2.84	2.84	3.36	4.39	7.05	3.86

Table 2 Mean Monthly Precipitation Scaling Factors

Scaling Factors												
GCM	Jan	Feb.	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
GFDL	0.47	0.32	0.49	0.69	0.88	0.96	0.90	0.90	1.34	1.26	0.79	0.50
GISS	0.65	0.56	0.68	0.84	0.95	1.12	1.38	0.73	0.89	0.90	0.86	0.91
UKMO	0.14	0.13	0.55	1.05	1.28	1.35	1.48	0.72	0.77	1.18	1.18	2.01

Table 3 Mean Monthly Solar Radiation Scaling Factors

Scaling Factors												
GCM	Jan	Feb.	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
GFDL	1.05	1.08	1.13	1.24	0.99	1.18	1.16	1.09	0.96	1.07	1.14	1.15
GISS	1.06	1.06	1.02	1.06	1.05	1.08	1.03	1.04	1.05	1.02	1.04	1.04
UKMO	1.08	1.15	1.15	1.05	0.94	0.95	0.80	1.38	1.11	0.99	1.09	1.02

ANALYSIS OF FLOODS

Using the predicted changes in meteorological factors due to three GCM's and the calibrated Sacramento model, a prediction of flow for the doubling of CO_2 scenario was attempted. Since many parameters (Lettenmeier and others, 1988) could be of interest regarding floods, only the annual maxima plotted on probability paper (In-na and others, 1989) are shown here (Figure 1) in order to estimate their magnitudes and the respective probabilities of occurrence. Table 4 shows the increase and decrease of annual daily runoff maxima (in m^3/s) predicted for all three GCM's. In general, it can be seen that GFDL Model predicts an increase in floods for all return periods, which GISS and UKMO models predict a decrease in floods. The reason for this result is that GFDL projections indicate higher rainfall in September and October, the peak flow period. The percentage change in Table 4 is also shown graphically in Figure 2.

Table 4 Annual Daily Runoff Maxima

Return Period (yrs)	Runoff (m^3/s)				Percent Change		
	1 x CO_2	2 x CO_2					
		GFDL	GISS	UKMO	GFDL	GISS	UKMO
	Annual Daily Runoff Maxima						
2	184.10	198.30	145.80	215.73	7.71	-20.81	17.18
5	314.74	364.41	226.48	315.94	15.78	-28.04	0.38
10	412.45	488.65	286.83	390.89	18.47	-30.46	-5.23
20	515.58	619.78	350.52	470.00	20.21	-32.01	-8.84
50	664.23	808.79	442.33	584.03	21.76	-33.41	-12.07
100	788.03	966.21	518.80	679.00	22.61	-34.17	-13.84

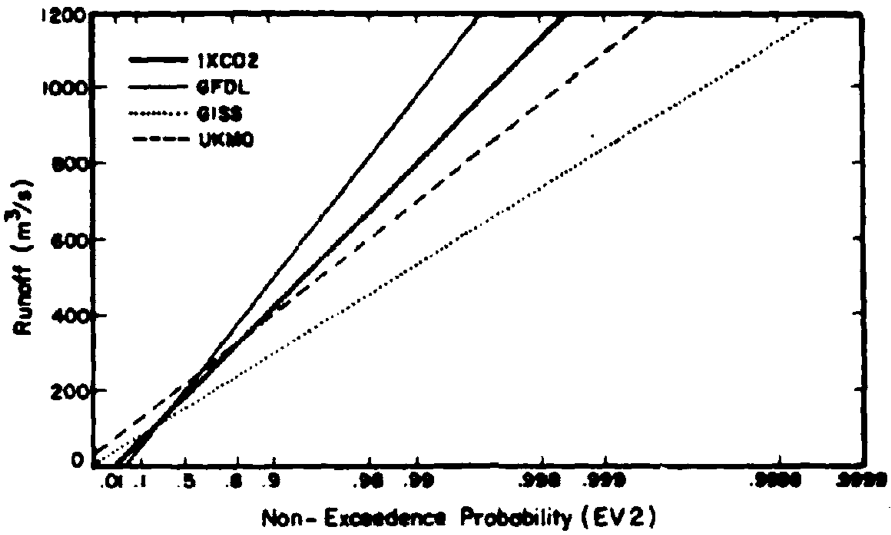

Figure 1 Annual daily runoff maxima simulated under 1XCO2 2XCO2 scenarios plotted on EV2 probability paper ($\gamma = 0.2$)

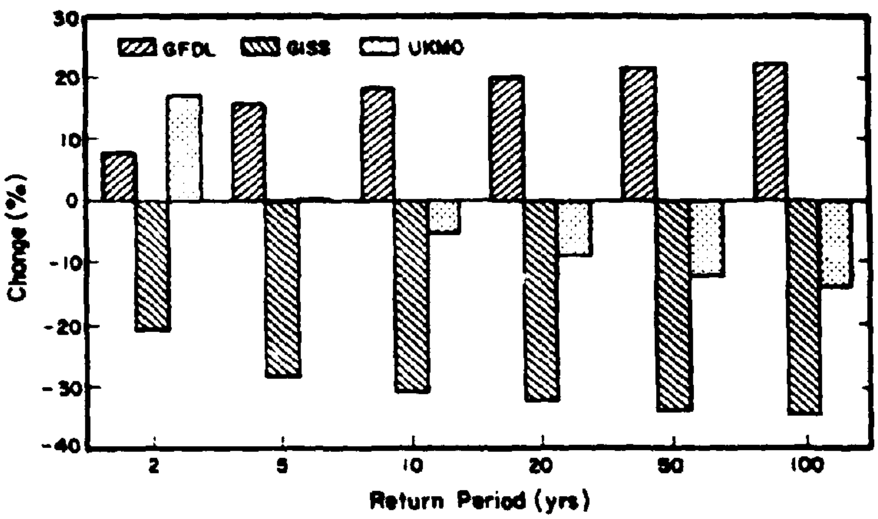

Figure 2 Change in peak flows due to Global Warming

CONCLUSIONS AND SUMMARY

The studies conducted regarding the global warming effects on floods cannot yield a unique conclusion. Some global circulation models yield higher floods and some lower for most return periods. For example, for a 100-year flood one model predicts a 22% increase while another predicts a 34% reduction.

The potential evapotranspiration (PET) plays a major role in tropical hydrology and even more so when the issue involved is global warming. Therefore it is imperative to model the effects of climate change on PET accurately so that the chance of getting a reliable estimate of the hydrologic impact of global warming in the topics becomes more probable.

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F12-1

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UNDERSTANDING FLOOD FORMING MECHANISM FOR BETTER FLOOD HAZARD PREVENTION AND MITIGATION

Lesmana Polo
Geological Department
Faculty of Mathematics and Natural Sciences
Padjadjaran University, Bandung, Indonesia

ABSTRACT. Flood hazard causes deaths and destroy properties, usually is encountered by dams, river normalization, etc., but compensated with very high costs. Therefore, flood occurences in developed countries, where funds are available, raise ambiguity in the methods used for flood preventions or mitigations.

Flood disaster occurs when accumulated river water from rainfall exceeds its channel discharging capacity. The streamflow is affected by rainfall characters, basin hydrological conditions, and basin physical parameters; which vary in time and space. However, all discharge calculations disregard of detailed basin physical parameters involvement. The basin is approached as a unit, to simplify the complexity of flood flow calculations, hence provides limited information on flood flow mechanism which determine flow discharge at a particular time and location.

The basin segmented "finite-element" approach, provides flood flow forming mechanism, moreover can defining the less from greater influential basin physical parameters on flood flow forming, hence providing better solutions to prevent or mitigate flood disasters.

1. INTRODUCTION

Further improvement to dissolve flood hazard problem is required since this natural disaster occurs everywhere all around the world, including in Indonesia. Some rivers associated with floods have been successfully controlled or mitigated, e.g. by dams, river normalisation, etc. However, similar treatment technique works less at other rivers, the river basins are still vulnerable to flood hazards.

Flood experts in public works, universities, etc. work hard to provide better flood hazard preventions or mitigations. Nevertheless, up to now flood of much less destructive than "Noah's Flood" that submerged the valley of Euphrate and Tigris always causes disasters, raising ambiguity in the existing methods on flood prevention or mitigation compatibility. In fact, various studies have failed to improve the understanding of basin responses to rainfall (Corbett and Sopper, 1975). More works are required for better understanding on flood forming mechanism, from which flood preventions or mitigations will be obtained.

2. METHODS TO UNDERSTANDING FLOOD HAZARDS

Heavy rainfall usually complements high stream flow or forms flood hazard which is also affected by the following factors:

- Basin physical parameters : area, slope, shape, river length, etc.
- Basin hydrologic conditions: groundwater, baseflow, antecedent moisture, no rain days, etc.
- Streamflow hydraulic system, i.e. insitu streamflow velocity, flow depth, discharge, etc.

The intermingling of these factors which vary in time and space, including of rainfall characteristic, distinctly forms an incredibly complex of flood forming mechanism and hard to understand; challenging man's knowledge and ability.

The methods usually are based on climatological and/or hydrological data, but less concern in basin physical parameters. Three methods, associated with (flood) flow discharges, are the "climatic method" that heavily relies on storm/rainfall characters, the "black box method" such as in the Rational Method, and the "unit hydrograph method" that provides calculation of (flood) peak flow discharge.

2.1 The Climatic/Rainfall Method

Three types of graphical correlations, the Co-axial Graphical Correlation, Rainfall vs Storm Runoff Curves, and Graphical Representation of Initial Moisture Conditions, are used to estimate runoff from rainfalls. The differences between these methods, in sequence, are on the factor involvements which are the monthly surface runoff, current and antecedent monthly rainfall, initial moisture content and deficiency, and storm characters (intensity, distribution and duration); see also Varshney (1977). These methods contribute very limited information on flood flow forming mechanism.

2.2 The Black Box Method

The black box method in the Rational Method (Mulvaney, 1851; in Pilgrim, 1978), commences the understanding of flood flow property (i.e. flow discharge) in a river basin. It involves basin area (A, km^2), rainfall intensity (i, m/sec) and basin conversion factor of rainfall to runoff (c,-), in order to calculate (flood) peak flow discharge (Q_p, m^3/sec); mathematically: $Qp = c\ i\ A$ (in Pilgrim, 1978).

The value of "c" becomes a major constraint, mostly intangible works even by using tables. Ward (1978) mentioned that the value of "c" varied from basin to basin, moreover from storm to storm even in a single river basin. (French and others, 1974; Raadsma and Schultze, 1974, and Pilgrim, 1978; in Polo, 1985, Unpublished Dissertation).

Hundred of Rational Method equations have been founded, and others will arive. Consequently, there is no reason to apply one of those equations for other river basins. The black box approach stands for probable flood flow design by involving the so-called return periods. It is not associated with (flood) flow forming mechanism, but figures that higher rainfall intensity relates to higher flow discharge.

2.3 Unit Hydrograph Method

Unit hydrograph represents information on the relation of flow discharge, rainfall, and time. By definition, unit hydrograph is a graph of direct runoff resulting from one centimeter of effective rainfall of a specified duration generated uniformly over the basin are at a uniform rate (Varshney, 1977).

On its development, hydrograph describes basin baseflow conditions, effects of different rainfall character or distribution on hydrograph shape, etc. From unit hydrograph it is possible to calculate peak of flood design (see Table 6.2 in Varshney, 1977). Several methods that involve different basin conditions and rainfall characters are described in the Van Te Chow method, Collin's method, etc. Another type of unit hydrograph is S-hydrograph which is a summation hydrograph, produced by a continuous effective rainfall at a constant rate for an indefinite period. Then the graph is converted into its unit hydrograph, as in Bernard method, Clark's method, etc.

The unit hydrograph approach has been well developed and gives information on the relations of flood flow discharges and rainfall characters, but on flood flow forming mechanisms. Anyway, the direct use of measured flow discharge at a stream gauge recorder shows very limited involvement of basin physical parameters which really affect the river streamflow characters.

3. GLOBAL AND FINITE-ELEMENT APPROACHES

Both, the black box and unit hydrograph methods are based on global approach, only of different scale. In the black box method, the river basin is treated as one unit, including to the rainfall intensity which is assumed uniform (but later is graded, e.g. isohyte method). In the unit hydrograph flow discharge from the whole basin or from a particular segment of the basin, is measured at a particular time and location. Improvements of this approach, either to solve any encountered constrains or in order to understand the basin responses to rainfall, heavily involving statistics. However, very limited information are available for the understanding of flood forming mechanism.

Global approach avoids complexity caused by the inter-relationship of basin physical parameters, basin hydrologic condition, and stream flow hydraulic, due to any direct rainfall or in the stream flowing systems. It delays the early stage complexity of calculation to be left behind for the final analysis. Yevjevich (1972) mentioned that the inadequacy of the available technology regarding flood treatment is as a result of the simplification of flood properties and descriptions. Nevertheless, the continuing hardship in flood stricken countries to day represents that less have been understood on flood forming mechanism.

Progress in science and technology leads to the founding of new hence completely different problem approach. Computer technology solves computing time consumptions which can be done in seconds. Therefore, finite element or finite difference approach favors the involvement of every part of basin conditions, to count every part of basin responses to rainfalls. Thousand to million calculations are representing the relationship of various basin parameters and basin (river) water conditions, in the form of flood flow discharges, at any part along the river length in a very short time, as desribed below.

The involvement of basin physical parameters within finite-element approach was described by Polo (1985, Unpublished dissertation). It involves "finite" basin area, topography (elevation, etc.), river length, its pattern, slope, width, etc., in order to determine streamflow characteristics at any location along the river length. Rainfall characters (intensity, duration and distribution) and basin hydrological conditions (stream depth, baseflow, etc.) are included in the calculation as well.

Figure 1 and 2 show the (peak) flow discharges according to time, on the account of basin local physical parameters response to rainfall characters and also of existing hydrological conditions. Then, simulations on factors favor to differentiate greater from less influencial factors for peak flow discharge, alternative solutions of flood hazards can be done especially at any particular prone sections. In other words, by manipulating the value of great (and small) influencial factors, flood flow properties can be calculated before any proposed treatment for reducing high peak flow discharge, i.e. to prevent or at to mitigate flood flow potentials.

4. CONCLUSIONS

Global approaches, i.e. black box and unit hydrograph methods, are less effective than finite-element approach for understanding flood flow properties. The global approach provide only limited information on flood forming mechanism, what is the influencial factors, time of concentration, etc., as provided by the finite-element approach.

Running "finite" computer program (Bruno in Polo,1985, Unpublished dissertation) enables us to understand on basin responses to storms, hence providing better understanding on flood flow forming mechanism. Consequently, appropriate flood hazard prevention or mitigation, on the account of simulated basin parameters, can be obtained.

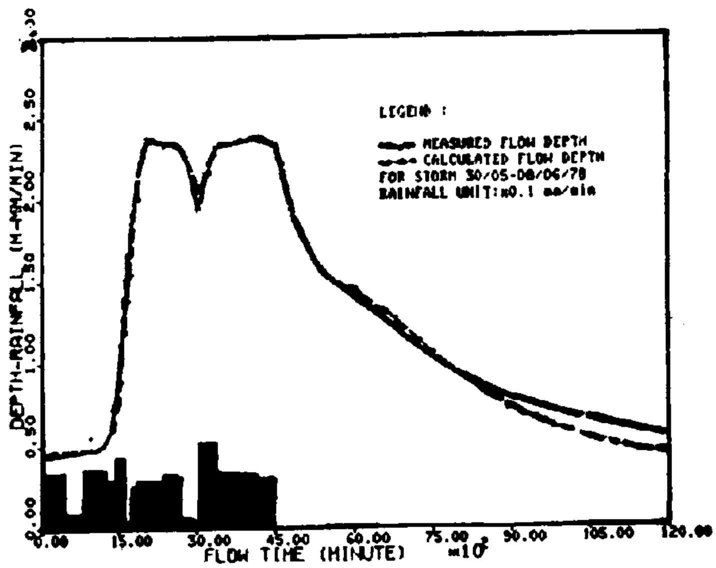

Figure 1. The fluctuations of flow depth at the Mulgoa Road stream gauge station, for storms dated 05/30/78 to 06/08/78. (source: Figure 5.1 in Polo, Unpublished dissertation).

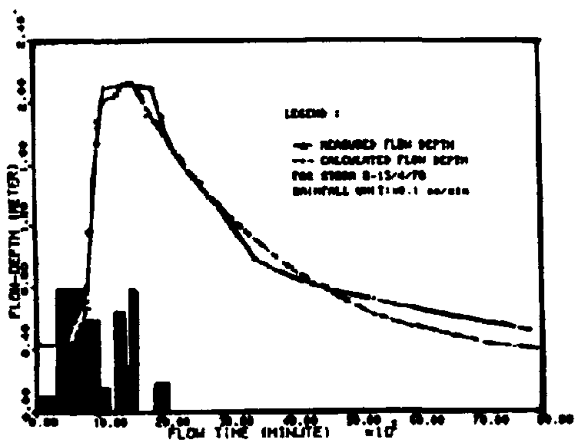

Figure 2. The fluctuations of flow depth at the Mulgoa Road stream gauge station, for storms dated 04/08/78 to 04/13/78. (source: Figure 5.2 in Polo, Unpublished dissertation).

5. ACKNOWLEDGEMENTS

The author would like to appreciate the Geological Department of Padjadjaran University, the Dean of Faculty of Mathematics and Natural Sciences of Padjadjaran University, and the Head of the Research Institute of Padjadjaran University, for providing opportunities and/or funds that enable the presentation of this paper in the Second US-Asia Conference on Engineering for Mitigating Natural Hazards Damage.

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FLOOD HAZARD IN THE COASTAL LAND RECLAMATIONS OF HONG KONG

M.R. Peart and W.W.-S. Yim
Department of Geography and Geology
The University of Hong Kong

ABSTRACT. The distribution of the flood hazard in Hong Kong including reclaimed areas is described and case studies are used to illustrate the severity of the hazard. Some observations on the major causes of flooding which include rainfall and storm surges are made. The current flood warning system operated by the Royal Observatory is described and future developments discussed. Engineering practices in relation to flooding are outlined and current projects reviewed. This includes modifying drainage systems to cope with reclamation.

INTRODUCTION

Hong Kong is located on the south coast of China to the east of the Pearl River Estuary. Due to its natural shortage of flat land for development out of its total present day area of about 1,100 km^2, close to 10 per cent consists of coastal reclamation (Figure 1). Currently, the rate of expansion into the sea is greatly increased because of two mega-projects. They are the Chek Lap Kok Airport and West Kowloon Reclamation involving 1,200 and 330 hectares respectively. The territory has a flood hazard.

SPATIAL DISTRIBUTION OF THE INCIDENCE OF FLOODING

The spatial distribution of reported incidences of flooding for the period 1970-1990 are shown in Figure 2. Mapping was based on secondary data sources, primarily the Royal Observatory annual and monthly weather reports, along with newspapers such as the South China Morning Post. The map does not portray all reported incidents of flooding as some were too general in the description of area to allow them to be plotted. Cartographic constraints also influenced the mapping. Nevertheless, the map does provide a useful guide to floodprone areas.

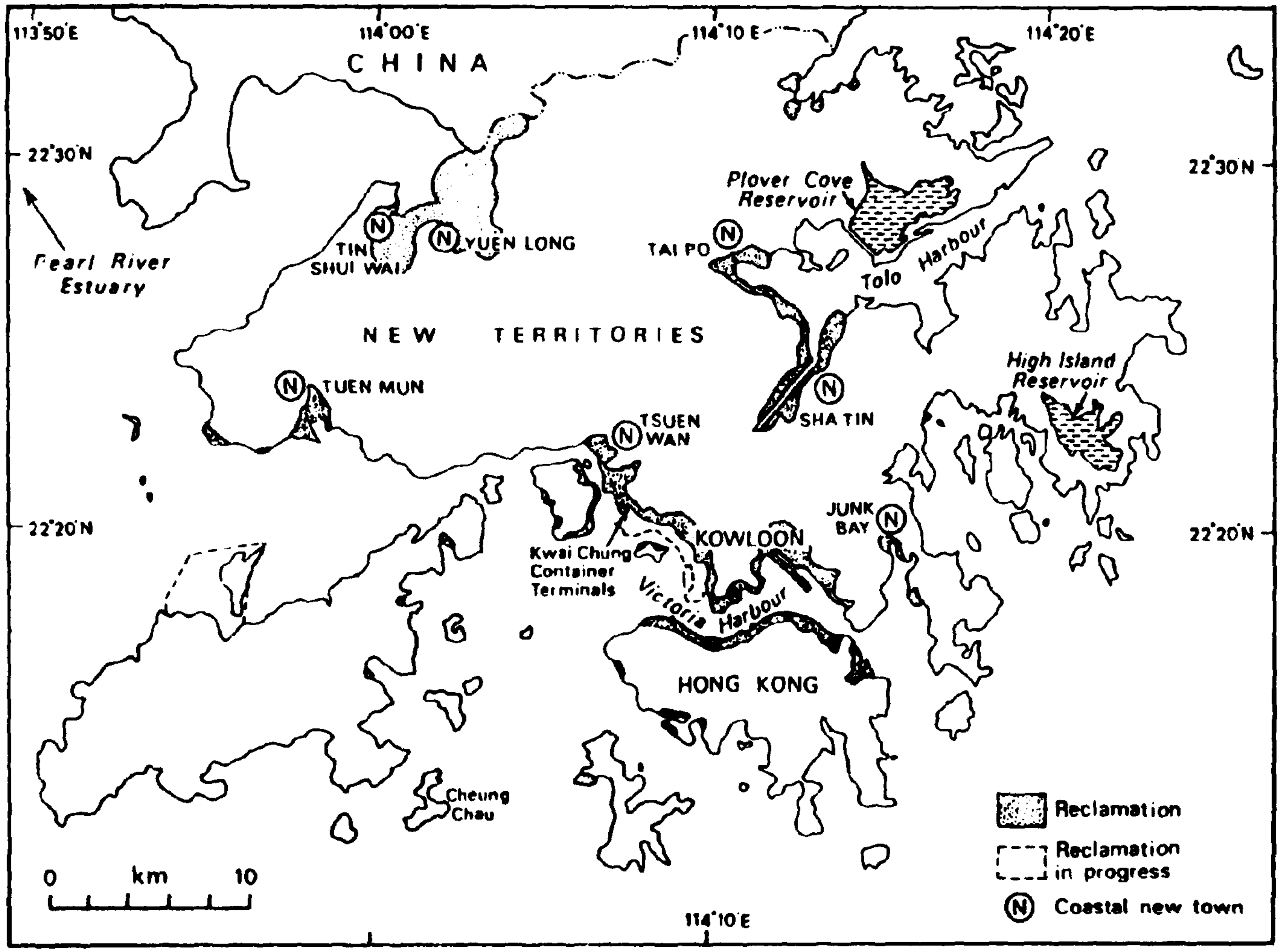

Figure 1. Location of reclamation areas.

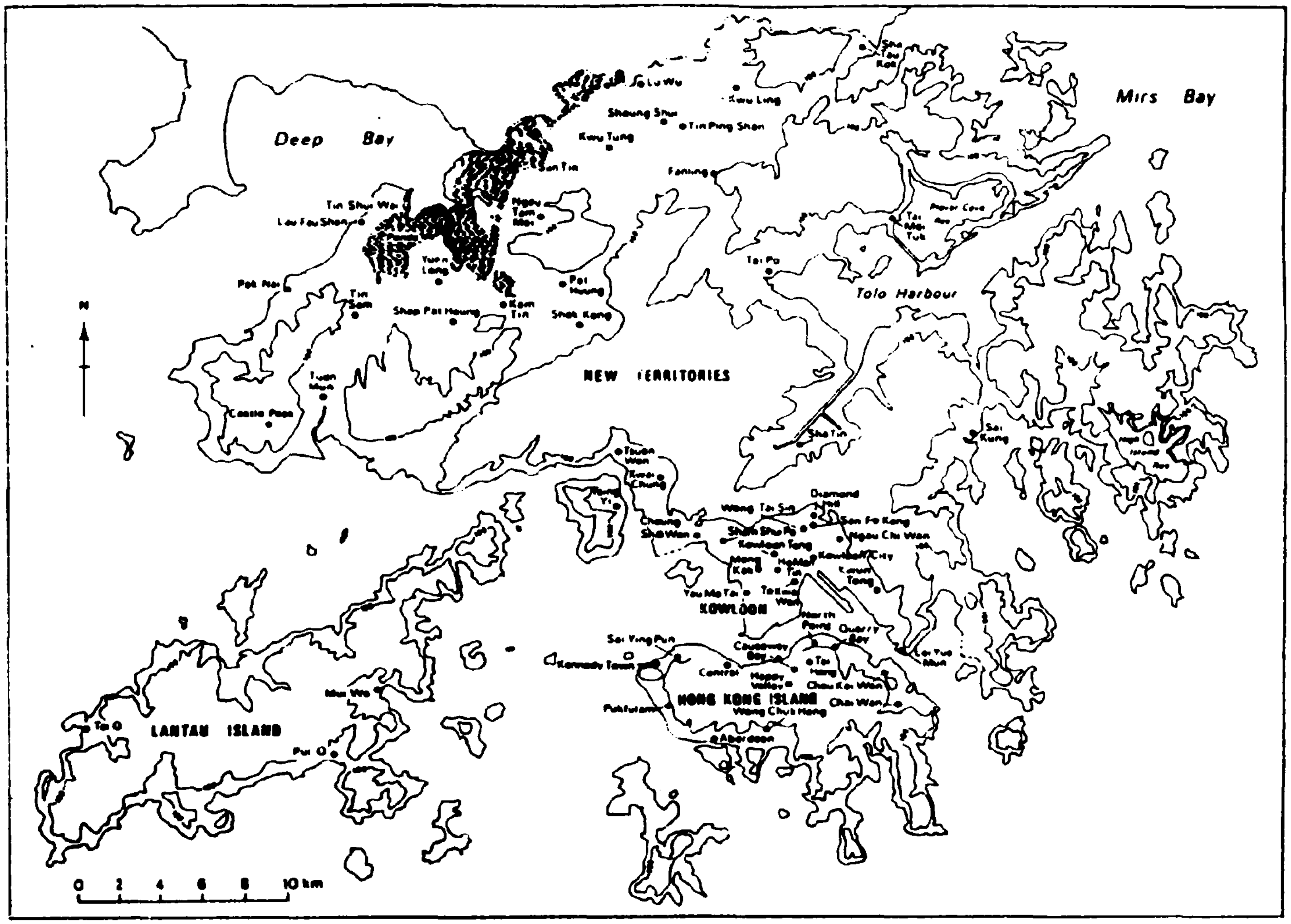

Figure 2 Spatial distribution of flooding.

Figure 2 reveals that the flood hazard is spatially restricted in occurrence and is confined to lowland areas, many of which are also near the coast and may also be on reclaimed land. The flood hazard affects both urban and rural areas of the territory but the perception of the hazard differs between the two environments. In urban areas flooding is not regarded as a serious problem. This is because of its temporary nature and the fact that drainage systems have been designed and emplaced so as to minimise the flood hazard. Furthermore, in the New Towns such as Tsuen Wan, Tai Po and Sha Tin where much of the development is on reclaimed land, engineering practice has attempted to reduce the risk of both sea and river flooding.

In rural areas there has been a change in perception of the flood hazard. Until relatively recently when paddy formed the main agricultural practice little was heard of the flood hazard in rural areas. Paddy required a plentiful supply of water and with an intricate and well maintained drainage system excess water could rapidly be drained away. However, with the abandonment of agricultural land, associated with the decline in paddy farming, and increased development of the rural areas for housing and other non-agricultural uses a growing awareness of the flood hazard has developed. Since the coastal lowlands may receive considerable runoff from inland rural areas the Hong Kong Government has had to devote considerable attention to drainage and flooding in rural areas.

CAUSES OF FLOODING

Some major causes of flooding in Hong Kong are summarised in Table 1 along with examples. In the coastal land reclamations, flooding may be exacerbated by the long term ground settlement (Yim, 1991a). Old reclamations which have settled are particularly susceptible to flooding because they create a "trough-effect" between the inland areas and the new reclamations. Changing hydraulic gradients consequent upon reclamation may also increase the flood hazard.

CONSEQUENCES OF FLOODING

Flooding causes a number of problems in Hong Kong one of which is the loss of agricultural crops, livestock and fish from ponds. The Agriculture and Fisheries Department has an emergency relief fund to aid farmers who have suffered severe losses due to flooding. However, the funding given to farmers is intended to help them re-establish their businesses

Table 1 CAUSES OF FLOODING AND EXAMPLES

CAUSE	EXAMPLE
1 Rainfall	17th August, 1982
2 Rainfall associated with a typhoon	Typhoon Brenda, 20-21st May, 1989
3 Storm surge	Typhoon Wanda, 1st sept., 1962
4 Rainfall and storm surge	Typhoon Rose, 16-17th August, 1971
5 Other causes, e.g. burst watermain, failure of sewerage system	Kwai Chung, 1st July, 1977

rather than as full compensation. Flooding also causes disruption to transport and damage to infrastructure. For example, Typhoon Gordon in July 1989 caused flooding on the New Territories Circular, Man Kam To and the Shau Tau Kok roads. Property and lives are also put at risk during flooding and squatter settlements may be particularly susceptible. For example, Typhoon Wanda in September 1962 resulted in a storm surge which caused severe flooding around Tai Po and Sha Tin and resulted in 130 casualties. 3000 huts and 5 village type houses were destroyed in Sha Tin and 72,000 people were registered as homeless. Further back in time, two un-named severe typhoons in 1906 and 1937, both of which were associated with severe storm surge flooding, have estimated death tolls of 10,000 and 11,000 respectively.

THE FLOOD WARNING SERVICE

The Royal Observatory of Hong Kong operates a flood warning service. Whenever, flooding is expected to occur a warning will be issued by the Royal Observatory and sent to the Information Services Department for distribution to the relevant government departments and organisations. The flood warning is also issued to the Hong Kong Telephone Company who operate a special calling service. Local radio and television stations also receive A Flood Special Announcement for broadcasting to the public. This is updated hourly until rain, sufficient to cause flooding, is no longer expected. In deciding to issue a flood warning consideration is given to both the intensity and expected duration of rainfall. This information is obtained from conventional meteorological observations, satellite and radar and a network of raingauges connected to the Royal Observatory by telemetry. For the period 1983-1990 inclusive the Royal Observatory has issued, on average, 15 warnings per year. Since 1983 the Royal Observatory has operated a thunderstorm, flood and landslip warning service, the latter being in conjunction with the Geotechnical Engineering Office of the Civil Engineering Department. The service evolved from the thunderstorm and heavy rain warning service which began operation in 1967.

Currently there is no flood warning service in Hong Kong based upon streamflow or water level in rivers. This may be because the small length of Hong Kong rivers would give little advanced warning. It may also be due to the fact that there is no streamflow forecasting model calibrated to Hong Kong conditions. However, consultants are currently examining real time flood forecasting for the Indus Basin based upon both routing and rainfall runoff models.

Much of the development in the territory is in the lowland areas or on reclamations and in these areas ground settlement or a rise in sea level can exacerbate or cause flooding. In consequence of this "hazard" the Royal Observatory runs of storm surge warning programme. This service issues warnings of an impending sea level rise during the passage of a typhoon. Because of the influence exerted mainly by the coastal configuration (Yim, 1991b), the Tolo Harbour area, including the two New Towns of Sha Tin and Tai Po is affected by the highest flood water levels.

CIVIL ENGINEERING AND FLOODING

Sound and effective drainage design plays an important part in combating the flood hazard in Hong Kong. This has been aided by the development of a design rain storm profile for Hong Kong (Peterson and Kwong, 1981). The rainfall information, when used with an appropriate hydrologic model, can be used to predict runoff volume for specified return periods. In Hong Kong the rational method has often been used to calculate the optimum size of drains. Table 2 presents the minimum return periods for drainage design in Hong Kong. The design engineer can opt for higher return periods if site condtions necessitate. There also exist engineering guidelines for cross drainage structures on roads and highways. These standards and guidelines have been updated (Highways Department, 1983). Design standards and guidelines for all aspects of flooding are currently under review by the Government.

Table 2 MINIMUM RETURN PERIODS FOR DRAINAGE DESIGN (AFTER CEO 1978)

A.	For nullahs and main stormwater drains through developed areas and for important land drainage	200 years
B.	For stormwater drainage networks in developed areas	50 years
C.	For unimportant land drainage	10 years

In order to help prevent flooding from the sea considerable attention has been given to selecting suitable formation levels on reclamation projects and crest levels for sea walls. The minimum formation levels used at Junk Bay, Sha Tin and Tai Po are 4.5 metres above Principal Datum while at Tsuen Wan it is 3.9 mapd. The selection of the appropriate formation levels is helped by sea level records obtained by the Royal Observatory (e.g. Chan, 1983) and by the use of models developed by the Royal Observatory. The formation level is usually set at the height of the 100-200 year return period storm surges. This is because of the greater risk of rainstorm related flooding if the formation level is set too high. The use of numerical models to predict storm surge effects began with the Sha Tin New Town Development project in 1976.

Extensive reclamation has necessitated updating the storm drainage systems in many urban areas. This has been done by means of modelling and field studies. For example, Mott MacDonald are currently examining the impact of the West Kowloon reclamation on drainage and sewerage. The reclamation cuts across the drainage outfalls of 16 mostly urban basins and the new reclamation will further restrict flow in an area already prone to flooding. New stormwater drains are being designed and constructed and the project involves field monitoring and mathematical modelling.

Until recently the possibility of sea level rise due to global warming and the settlement problem on reclamations have received little attention in the territory. However, increasing attention has been focussed on these aspects including their implication on flooding (e.g. Butling and Chalmers, 1988; Secretary for Works 1990; Yim 1991a). Although Yim (1991c) found no evidence for a rising sea-level trend in Hong Kong, the North Point reclamation has been found to be subsiding at a rate exceeding crustal uplift. Therefore it is also necessary to consider the influence of long term ground settlement on flooding. Major new reclamation projects such as the North Lantau Replacement Airport (1200 ha) and the West

Kowloon Reclamation (330 ha) would require careful consideration on these matters.

CONCLUSION

There exists a flood hazard in Hong Kong. A variety of causes of flooding exist and man in both urban and rural areas may have acted to enhance the problem. To counter the flood hazard the Royal Observatory runs a flood warning service and a storm surge warning programme. The possibility of real-time flood warning is being investigated. Engineers have assisted in alleviating or minimising the flood hazard in Hong Kong by deciding upon appropriate design standards and investigating the hazard in the context of new reclamations. They are also involved in examining new methods of flood warning.

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DEBRIS CONTROL IN MOUNTAIN WATERSHEDS OF LOS ANGELES COUNTY

Zhida Song and Thomas H. Jackson
Utah Water Research Laboratory
Utah State University
Logan, Utah 84322-84200 USA

ABSTRACT The large discharges of water and sediment from steep mountain watersheds onto a densely populated floodplain are a major geological hazard in Los Angeles County, California. To reduce the damages, the County constructed Devil's Gate Reservoir, northwest of Pasadena, to trap sediment from the 31.9- mi^2 Arroyo Seco watershed discharges its flood and sediment load. The County extended its structural program in the 1940s. Faced with increased operating costs, the City later added a watershed management program for flood and debris control. Records of debris accumulation in the reservoir for over 50 years gave data on sedimentation that were used with the Pacific Southwest Inter Agency Committee Model to determine the effectiveness of such land treatment measures as improving land cover and preventing fires.

INTRODUCTION

Los Angeles County, California is one of most populous counties in the United States. About 9 million people live on an alluvial plain formed by sediments discharged from the geologically young San Gabriel Mountains rising to the north (Fig. 1). During the winter season that brings most of the rainfall in this semi-arid climate, occasional storms produce major floods from the small, steep drainage basins and can initiate debris flows. These events are a major geological hazard that have caused extensive damage and even taken lives in the urban areas that have expanded into the canyons and floodplains. Major storms in February, 1914, caused an estimated $10 million damage to residential property; power, telephone, and transportation lines; and other infrastructure. The storms of February and March, 1938, generated flows from the 39.1-mi^2 (77-km^2) Arroyo Seco watershed, northwest of Pasadena, that caused damages of almost $2 million (Cooke, 1984). A flood in the La Canada Valley on New

Year's Day, 1934, killed over 40 people and caused property damages reaching $5 million.

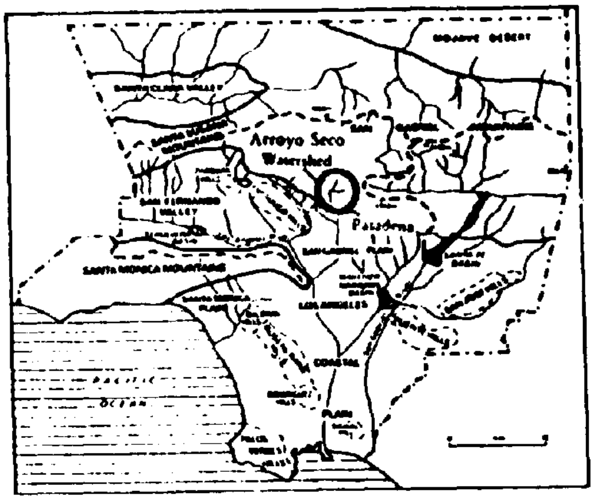

Fig. 1. Map of Study Area (after Cooke, 1984)

After these floods, Los Angeles County, in concert with the Corps of Engineers, greatly expanded its system of structural measures for sediment and debris control. Mountain slopes were stabilized, 30 dams were built at canyon mouths with reservoirs sized to contain design flood peaks and retain their debris and sediment, and the channels below were lined with concrete. Many more small crib dams were constructed in the mountains to reduce the amount of debris reaching these reservoirs. The County also cooperated with the U.S. Forest Service in programs to control fires and regulate recreation. However, program costs pose an enormous financial burden for the County. Debris must be removed from behind the dams after major storms at a cost from $4.5 to $10 per cubic yard depending upon the distance to disposal sites and opportunities for recuperating costs from the sale of sand and gravel. Present storage does not give adequate protection to the urban areas, and the costs of adding sediment control structures in the steep watersheds are excessive. For that reason, further land treatment in the watershed has attracted the interest of the County.

CASE STUDY: STRUCTURAL MEASURES FOR ARROYO SECO WATERSHED

Devil's Gate Reservoir at the mouth of Arroyo Seco canyon has served Pasadena as a flood and debris control facility for over 70 years. Sediment production from the Arroyo Seco watershed can be estimated from periodic field surveys of debris accumulation within the reservoir made by the Los Angeles County Department of Public Works (LACDPW). The total production is obtained by adding measured accumulations to the sediment volumes that have been excavated or sluiced through the reservoir. Fig. 2 shows the debris accumulation from 1919 to 1988. The rate of sediment generated has varied significantly during different time periods. Before 1935, flood flows and sediment production rates were generally small.

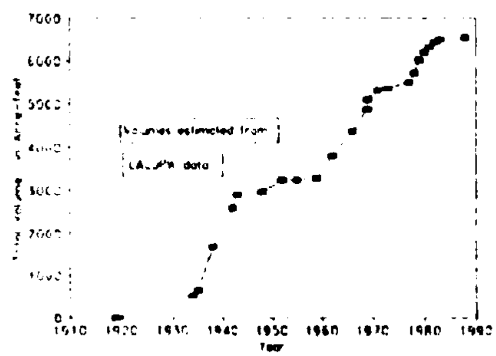

Fig. 2. Debris Accumulation at Devil's Gate

From 1935 to 1948, the peak sediment flows were higher as little was being done to contain sediment within the watershed, and the faster sedimentation was filling the reservoir. Annual sediment production was estimated as 177 acre-feet (AF) or 5.56 AF/mi^2. The period from 1948 to 1960 was drier, and sedimentation slowed. The years after 1960 once again brought larger floods and faster sedimentation, and the excavation program had to be accelerated to keep the reservoir from filling. Since 1969, the sediment accumulation has slowed to an average annual rate of about 100 AF, 3.14 AF/mi^2 per year, despite two major sediment-producing storms (Fig. 3). Thus the erosion control structures have reduced annual sediment production to about 57 percent of their former values. This figure approximately matches the 60 percent found by Simpson (1969) when comparing storms in 1969 and 1938.

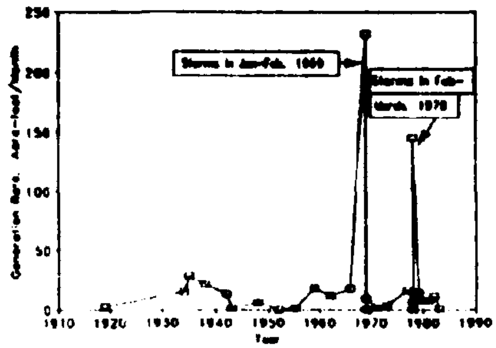

Fig. 3. Debris Generated from Arroyo Seco Watershed

NON-STRUCTURAL MEASURES FOR ARROYO SECO WATERSHED

Facing steadily increasing operation costs, the City is looking for a cost-effective way to reduce sediment and debris flow to the reservoir. Restoration of native plants and regulation of land use are primary measures. Although these measures will probably have little effect on large flood peaks, they are expected to reduce discharges of sediment and debris substantially.

Mountain sediment production comes through two processes: 1. surface runoff on hill slopes picks up soil particles causing sheet erosion; and 2. subsurface runoff along the top of impermeable rock layers causes pore pressures that induce landslides when flow rates exceed soil transmissivity. Vegetation inhibits landslides by roots that secure the soil mantle to the underlying rock, by providing transpiration drying the soil, and by faster pore drainage as roots maintain soil permeability. The influence of vegetation on microclimate reduces erosion by preventing soil breakup by cycles of freezing and thawing. Plants shield the soil surface from the erosive energy of raindrop impact and retard runoff velocities. Unfortunately, vegetative cover is difficult to maintain in a semi-arid climate. Long dry seasons make plant cover susceptible to forest fire, greatly increasing erosion rates and debris events. The watershed management plan is to restore conifers to the higher elevation, oaks to the valley, and mosaic chaparral by age groups. The mixed vegetation will be protected by regulating recreation and controlled burning.

EVALUATION OF THE EFFECTIVENESS OF WATERSHED MANAGEMENT

The Pacific Southwest Inter-Agency Committee Model (PSIAC) is a sediment yield rating model that provide a framework for evaluating potential watershed management measures (PSIAC, unpublished report; Nelson and Rasely, 1990). The Model defines ranges for nine variables important in determining sediment yield as shown in Table 1.

TABLE 1. PARAMETERS AND THEIR SCALES IN PSIAC MODEL

	Factor	Lower bound	Upper bound
F1	Geology	0: hard rock	10: friable rock
F2	Soil	0: rock surfaces	10: fine textured, easily eroded soil
F3	Climate	0: arid areas experience few storms or most of precipitation as snow	10: long duration or intense storms
F4	Runoff	0: lower flood peaks generated from base flow	10: high flood peak from overland flow
F5	Topography	0: mild slopes; moderate slopes (20 degrees or 32.5 percent) at 10	20: steep slopes
F6	Ground Cover	-10: completely protected by vegetation from erosion. 0: 40% cover	10: a ground cover protects <20% of the area
F7	Land Use	-10: areas with undisturbed natural vegetation	10: intensely grazed or cultivated, or frequently burned
F8	Upland Erosion	0: no evidence of erosion	25: > 50% the area is comprised of rills and gullies
F9	Channel Erosion	0: flat gradient, bedrock stream channel; structural protection	25: active areas with head cutting

Appropriate values for each parameter are estimated by reviewing conditions within the watershed, and the nine values are summed to give a Sediment Rating Factor (SRF) that indicates its sediment production potential.

The annual sediment yield (AY), in AF/mi^2, is estimated as:

$$AY = 0.0833 \cdot A \cdot e^{0.359 \cdot SRF} \qquad (1)$$

where A is the area of the tributary watershed in mi^2, and e is the base of natural logarithms.

The recorded sediment yield at Devil's Gate dam from 1969 to 1983 was used to calibrate the nine parameters. The value for each of the nine parameters was adjusted to give an overall annual sediment yield that matched the recorded total. The parameter values estimated for current conditions in the Arroyo Seco watershed, the total SRF, and the PSIAC rank are listed in Table 2, on the following page.

The rationale for the value selected for each parameter is now discussed individually. F1: In the Arroyo Seco watershed, most of the parent bed rock is highly fractured igneous rock that decomposes rapidly when exposed to the elements. These surfaces are the primary source of the sand and silt washed out of the canyon during storm events. F2: The igneous rock decomposes into a thin mantle of coarse sandy soil that tends to move down slope soon after being formed. The remaining soil is poorly developed and shallow. The coarse structure gives the soil a high infiltration capacity; but heat generated by fires often causes a chemical change that makes the soil water repellent. After a fire, storms on the watershed generate more runoff and carrying much more sediment and debris. F3: The Los Angeles area has a Mediterranean Climate in which most of the precipitation is in the form of winter rain. Many storms last several days and thoroughly wet the soil. Following lush growths of vegetation dry out and increase the fire danger. F4: Stream flows, measured in Arroyo Seco since 1911, vary greatly from one year to the next. Within the wet years, most of the runoff is concentrated during one or two major storms. One would expect sediment production to be even more concentrated in large storm events. F5: The San Gabriel Mountains above Pasadena are very steep with an average slope of about 50 percent. Some slopes reach 70 percent. F6: The main native vegetation on the hill slopes is chaparral which intercepts more rainfall than does grass and thus reduces runoff from lesser events; but the thick brush is combustible and a higher fire risk. Conifers areas are diminishing. F7: Except for a few roads and trails and some high-elevation recreation facilities, the lands in the watershed have not been developed, but the San Gabriel Mountains have been rated as having a high fire hazard with most fires caused by visitors during dry periods. F8: Dry erosion is extensive on the steep mountain slopes during rainless periods. During major storms, wet erosion occurs as raindrop impact loosens the soil surfaces, and the flowing runoff transports the detached particles. Landslides occur when the soils become saturated. F9: Channel erosion is a lesser problem than upland erosion because it is reduced by crib dams but is still an active sediment source.

TABLE 2 ESTIMATION OF CURRENT SEDIMENT YIELD BY PSIAC MODEL

	Factor	Comments	Score
F1	Geology	Heavily weathered Igneous	8
F2	Soil	Coarse, sandy, poorly developed and shallow; water repellent after fire	8
F3	Climate	Long dry season, winter storms	9
F4	Runoff	Small watershed, high peak flow	9
F5	Topography	Very steep, avg. slope > 50 degree	20
F6	Ground Cover	Incomplete chaparral cover	5
F7	Land Use	Recreation, some road & trails, high fire risk, small burn	4
F8	Upland Erosion	Both dry & wet erosion on slopes	22
F9	Channel Erosion	Crib dams reduced some erosion	16
Sediment Rating Factor (SRF) 101		PSIAC Rank	1
Calibrated Annual Yield	99.80 AF	Observed Annual Yield	100.08 AF

Watershed management schemes proposed for the Arroyo Seco watershed can be evaluated by estimating their effects on each parameter in the PSIAC model. The new sum give an SRF that can be used to estimate a new sediment yield. Watershed management will not alter geology, soils, climate and topography, thus these parameter values are held constant. It can directly change ground cover (F6) and land use (F7); and these changes alter runoff (F4) and upland erosion (F8). The change that can be achieved by watershed management is used to estimate new values for these four parameter, and the SRF and the annual sediment yield are recalculated (Table 3). The annual yield can be reduced to 64.87 AF, 2.03 AF/mi^2; accordingly, the PSIAC rank will improve to 2. As a direct benefit, the debris basin evacuation cost will be reduced by at least $255,620 per year, calculated at the minimum excavation cost of $4.5 per cubic yard. Working together, structural and non-structural measures can cut the total amounts of annual sediment yield from 177 AF to 65 AF.

TABLE 3 REDUCTION OF SEDIMENT YIELD BY WATERSHED MANAGEMENT

	Factor	Comments	Score
F4	Runoff	Reduce peak, increase baseflow	8
F6	Ground Cover	Restore conifers, mosaic vegetation, contour furrow planting	0
F7	Land Use	Regulation visiting, fire control	0
F8	Upland Erosion	Reduce wet erosion on slopes	20
Sediment Ration Factor (SRF) 89		PSIAC Rank	2
Annual Yield	64.87 AF	Reduce Annual Yield	35.21 AF

CONCLUSIONS

Watershed management is an effective non-structural measure for debris control on the steep mountains in semi-arid Los Angeles County. The case study of the Arroyo Seco watershed shows that restoration of native vegetation and strict fire control have a potential to reduce current annual sediment yield by an additional 35 percent. These measures are recommended for other watersheds. The Pacific Southwest Inter-Agency Committee Model is a useful management tool that obtains judgments from experts from several disciplines to help decision-makers evaluate the effectiveness of sediment control programs. Data collected on the accumulation of sediment in debris basins and the conditions in the contributing watersheds can be used to improve model calibration and accuracy in estimating the consequences of specific management practices.

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MITIGATING FLOOD IMPACTS : APPROACHES AND EXPERIENCES IN THE UNITED STATES

James M. Wright
Federal Interagency Floodplain Management Task Force
Tennessee Valley Authority
Knoxville, Tennessee 37902 USA

ABSTRACT Approaches to dealing with floods in the United States have changed dramatically during the latter half of this century. There are several reasons: changes in policies and attitudes resulting from the Nation's experience with actual flood events, development of better trained and more experienced professionals, improved analytical techniques and forecasting measures, and increased capabilities to assess the economic and natural resource losses associated with various uses of the floodplain.

The author reviews current philosophies, policies, and practices for reducing these losses in coastal and riverine floodplains in the United States. Some promising mitigation approaches are presented, based on an extensive 5-year study of progress towards developing a unified national program for floodplain management.

INTRODUCTION

Throughout the history of the United States, the prevailing view has been that humans should use and modify the natural environment to meet their needs. For centuries, people have been settling on the banks of the country's rivers, streams, and coastlines to take advantage of the water supply, transportation, energy source, wildlife habitat, and other benefits floodplains provide. The large-scale development and modification of riverine and coastal floodplains for economic use has exacted a high price annually in deaths, personal injury, suffering, economic loss, and damage to natural and cultural resources.

Floods account for more losses than any other natural disaster in the United States, with the exception of droughts during certain periods. About 7 percent of the country is subject to flood water inundation and average annual flood damages are estimated at $4 to 6 billion. Because of its size and geographic diversity, the United States experiences the full range of flooding forms. Examples include inland flash floods, ice jams, alluvial fans, channel migration, ground failure, and storm-induced coastal flooding and erosion.

THE EVOLUTION OF FLOOD MITIGATION IN THE UNITED STATES

The way floods and their consequences are dealt with in the United States today is the result of political actions and governmental and private measures that span nearly six decades.

As floods became more destructive because of the increasing capital investments that knowingly or inadvertently were being placed within flood-prone areas, there was a call for measures to prevent or limit damages to such investments. Assumptions of Federal responsibility for controlling flood waters started during the early part of this century primarily as a response to significant loss of life or property damage. In the ensuing decades, despite billions of dollars in Federal investments in flood control projects which averted billions in damages, overall flood losses and other costs continued to rise because of unwise use of the Nation's floodplains.

During the 1960s, several major steps were taken to redefine Federal policy to avert future flood losses by helping the 50 states and 20,000 flood-prone localities encourage wise use of flood-prone lands.

Broader approaches were studied and applied including ways to adjust future development and use to the flood risk, flood forecasting and response systems, floodproofing or retrofitting of existing flood-prone structures, relocation of property, and a national flood insurance program. The environmental values of floodplains received growing recognition and support. To implement these measures, an extensive program to map the Nation's flood-prone areas was also initiated.

A proposal for a unified national program for promoting wise use of the Nation's floodplains was prepared during the 1970s and later revised. It set forth a conceptual framework in which managing floodplains would become a true Federal/State/local partnership. The program entailed Federal assistance (technical and financial); state initiative, involvement, coordination and leadership; and local responsibility, decisionmaking, and management. Strategies and tools for flood loss mitigation and for the preservation and restoration of natural and beneficial floodplain resources also were presented.

During this period the 1-percent-annual-chance flood was adopted as a minimum national standard to be met in devising flood protection measures for permitted future development in flood hazard areas. State and local involvement in floodplain management increased substantially. Over the past two decades, increased capabilities for natural resource assessment and impact analysis have been developed.

EVALUATION OF THE EFFECTIVENESS OF MITIGATION MEASURES

In 1987, a Federal interagency task force undertook a study of the overall effectiveness of the Nation's efforts to mitigate the impacts of flood events. This national assessment (the first of its kind on such a comprehensive scale) was completed in late 1991 and published in 1992 in a two-volume report. The two volumes are entitled Floodplain Management in the United States: An Assessment Report, Volume 1, Summary Report, and Volume 2, Full Report. The study provides information on the nature and use of floodplains, along with an evaluation of the various strategies and tools being employed to reduce losses.

Listed below are some of the key findings of the study:

Individual risk awareness. Although substantial progress has been made in increasing institutional awareness of flood risk, individual awareness falls far short of what is needed, resulting in unwise use and development of flood hazard areas.

Migration to water. People are attracted to riverine and coastal environments for a variety of reasons, usually unrelated to economic necessity. In recent decades, the annual growth rate in these areas has greatly exceeded the Nation as a whole. This has exposed property and people to unnecessary risk.

Floodplain losses. Despite attempts to cope with the problem, the large-scale development and modification of riverine and coastal floodplains has resulted in increasing damages and loss of floodplain resources.

Short-term economic returns. In many instances, private interests develop land to maximize economic return without regard to long-term economic and natural resource losses. This increases public expenditures for relief, rehabilitation, and corrective actions.

Life, safety, and public health. Because of technological advances in flood warning and response, flood-related deaths are not increasing on a per-capita basis. Public health and safety resulting from flood consequences is not a pervasive problem because of the provision of health care and government and private flood relief.

Enhanced knowledge and technology. Institutions and individuals that deal with floodplain problems must have a broad range of information, a variety of technologies to deal with emerging problems, and standards to which they can refer for guidance. Research has enhanced our knowledge and provided new and better tools to deal with physical, biological, and social processes.

A national flood protection standard. Because of avoidance of high-hazard areas and changes in construction practices, most new floodplain developments now have improved flood protection. However, these controls over development within the regulatory floodplain, defined by the limits of the 1-percent-annual-chance-flood event, have resulted in a concentration of developments just beyond these limits or levels. Protection from the effects of greater, less frequent flooding is still needed in those places where such flooding will cause unacceptable or catastrophic damages.

Limited governmental capabilities. Some states and most communities lack the full resources necessary to bring about comprehensive local action to mitigate flood problems without Federal support. Local governments invariably misjudge their ability to deal with severe flood events. However, they are necessary partners to any successful solution.

Need for interdisciplinary approaches. Consideration of plans to solve flood problems has to encompass the entire hydrologic unit and be part of a broader water resources management program. A lack of familiarity with all the available techniques biases the investigation and selection of solutions for specific flood problems. Training in a variety of disciplines is needed in devising and carrying out mitigation strategies.

Application of measures. Nationwide mapping of floodplain areas has resulted in detailed studies of most community floodplain areas. A variety of strategies have been used to restore and preserve the natural and cultural resources of floodplains and to reduce economic losses by modifying flooding, by modifying susceptibility to flood damage and disruption, and by modifying the impacts of flooding on individuals and the community. Storage or control of floodwaters is still the preferred political approach at the local level.

Effectiveness of mitigation measures. Structural flood control measures have been effective in reducing economic losses to floodplain occupants. The application of additional structural measures is viewed as limited because of economic and environmental considerations. Land use regulations required by some Federal programs and implemented by state and local governments have reduced the rate of floodplain development but have not arrested it. Compliance with regulatory controls is a significant problem. New technologies and techniques associated with risk assessment, forecasting, warning, and construction practices have improved the application and effectiveness of these activities substantially. A national flood insurance program has not realized its full potential because less than 1/4 of floodplain residents have purchased and maintained policy coverage.

Role of disaster assistance. Liberal Federal assistance in post-flood relief and recovery has reinforced expectations of government aid if and when flood disasters occur. This mindset has resulted in limited mitigation planning and actions by communities and individuals.

Multi-hazard mitigation. There is a growing interest and need in integrating flood loss reduction strategies with those for other natural hazards. Dealing with simultaneous hazards in post recovery efforts is a problem.

National goals and resources. Despite great strides in that direction, the United States still lacks a truly unified national program for floodplain management. Ambiguity in national goals has hindered the effective employment of limited financial and human resources.

SOME PROMISING APPROACHES FOR BROADER APPLICATION

The application of mitigation measures is constrained by the availability of funds, familiarity with a measure, individual or community capability to implement it, and certain physical and technical limitations. However, in assessing the approaches and experiences of the United States in mitigating flood impacts, these measures hold promise for application and evaluation elsewhere:

Risk analysis. Public awareness of flood-prone areas and the flood risk must be improved by identifying hazard areas and employing other strategies. This information must be provided to individual decision-makers and policymakers in terms they can understand and act upon.

Avoidance of flood control biases. Engineers are expected to find solutions to problems using their specialized education and training. The temptation is to study and recommend measures with which we have experience or familiarity. It is becoming increasingly difficult to justify and obtain the means for capital intensive projects. Broader, interdisciplinary approaches to problem solving usually are needed.

Development of state and local capabilities. To meet the needs of affected communities, state and local interests must be involved in developing and applying flood mitigation measures. Engineers and other disciplines can assist in providing training concerning mitigation alternatives.

Improved warning and response systems. Fairly simple flood warning systems can be developed to help forecast the magnitude and timing of flood events. Temporary evacuation of residents of flood hazard areas can be accomplished. The degree of success depends on notifying the affected population, how they respond, lead time, and the availability of evacuation routes.

<u>Adjustments to individual structures</u>. Changes can be made to existing construction (for example, elevation in place) and to permitted new construction to minimize exposure to flooding. However, such adjustments must be consistent with individual resources and capabilities, local building practices, and availability of technical assistance if necessary.

<u>Public infrastructure.</u> Two goals can be accomplished by factoring exposure to flood risk in the design and location of public infrastructure. First, these systems are less likely to be incapacitated by flood events. Second, and perhaps more importantly, they can encourage or discourage development in flood hazard areas by their existence or absence.

CONCLUSIONS

Flood mitigation in the United States initially centered on structural measures to modify flooding. In response to changing situations, needs, values, and priorities, however, a new approach has evolved which allows the best mix of mitigation measures to be applied to unique local circumstances. A number of important opportunities are emerging for improving the effectiveness of those selected. They involve broadening the scope of the measures to encompass other water resource management activities and to integrate flood-related strategies and measures with those designed to mitigate other natural hazards. Technological advances also hold great promise for improving the application of existing strategies and tools.

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GROUND FAILURE HAZARD

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CURRENT METHODS OF SLOPE PROTECTION IN TAIWAN

HUNG Ju-Jiang
Department of Civil Engineering
National Taiwan University
Taipei, Taiwan
Republic of China

ABSTRACT

Taiwan has more than 75% of its land occupied by mountains, hilly lands, and lateritic terraces. Landslide, debris flows, and erosion of slope surface have long been the most frequent natural hazard in Taiwan.

Traditional methods of slope protection, such as reinforce-concrete retaining walls, earth anchors, gabion walls, drainage system, etc., remain popular in engineering circle. However, the high costs and environmental conflicts involved in use of these methods have caused concern and opposition from the general public and environmental-protection agencies.

Since 1983, the National Science Council has been sponsoring research projects on landslides. Emphasis is partly given to developing methods of slope protection which are not only effective and economical, but also in harmony with the environment. Vegetation, reinforced earth with geotextiles, surface treatment, soil improvements, and hybrid methods have been developed. Some full-size experimental slopes have been constructed in the field to demonstrate the superiority of the methods developed.

INTRODUCTION

The main island of Taiwan has more than 75% of its land occupied by high mountains, hilly lands, and lateritic terraces.

In the Central Mountain Ranges, There are more than 100 summits higher than 3,000 meters above sea level. The highest peak, Yu-Shan (Mt. Jade), stands 3,952 meters above sea level. These summits are, in general, highly fractures at the top. Loose screes can always be seen on slopes immediately below the crest. Some of the scree slopes have developed into colluvium deposits.

Unstable slopelands in the hilly region include dip slopes, talus deposits, and mudstone slopelands.

Lateritic terraces stands up from the Western Coastal Plain to heights from 100 meters to 250 meters. Although they are flat and stable at their top surface, their sideslopes are steep and basically unstable.

Many slopeland communities are developed partly on cut ground and partly on filled ground. Most of the filled ground are not properly compacted. Standard of slope protection are not even better than that of ground compaction.

Taiwan is in the collision belt between the Eurasian Continental Plate and the Philippines Sea Plate. Very high rate of uplifting has made Taiwan a mountainous country of very high relief. Geological processes are very active. Folding, thrusting and over-riding of geological formation are very well developed. Destructive earth-quakes occurred from time to time in the history of Taiwan. Earthquakes induced landslides are not uncommon. The most noted case was Tsao-Ling Rockslide (1941) which involved mass movement of more than a hundred million cubic meters.

Taiwan is situated in the subtropics. Heavy rainfall and typhoon attack the island several times a year. Maximum 1-hour rainfall can be as high as 120 mm. Maximum 1-day rainfall was 1136 mm. Landslides and debris flows of Various scales occur frequently during or after a heavy rainfall. Erosion of slope surface in the mudstone area can be more than 100 mm per year.

There are many traditional methods of slope protection in use of Taiwan. Some of them remain popular but some of them have some caused opposition from the general public and environmental-protection agencies.

Since 1983, the National Science Council (NSC) has been sponsoring research project on landslides and slope erosion. Emphasis is partly given to developing methods of slope stability and slope surface protection which are not only effective and economical, but also in harmony with the environment.

TRADITIONAL METHODS OF SLOPE PROTECTION

Bench Cut

Retreating of slope by bench cut has long been a most use methods of slope stability. Many cut-slope of freeways enjoy the success of this methods. Slope height can more than 40 meters. Slope surface are in general, protected with vegetation.

Placing Toe Weight

At station 63k of the cross-island Highway. There was a large scale unstable slope dipping into the Teh-Chi Reservoir. Toe weight was placed to increase counter-balance force for the slope.

RC Retaining Walls

RC retaining wall was once a very popular structure type of slope protection. Objection are mainly due to its conflicts with the environments. Nevertheless, people are still using this type of retaining wall if its not to high and if the land area for constructing other types of retaining structure is not available.

Earth Anchor

Earth anchor are very much over-used in Taiwan. They are very often used in conjunction with RC plates or free-frames for retaining high and steep cut slopes. They are very expansive and not in harmony with the environment although they are effective in slope stability.

Gabion Walls

Gabion walls have been used in Taiwan for more than 60 years. They remain popular in protecting slopes of talus deposits, lateritic soils, weathered rocks, etc. Due to their flexibility, free-drainage property and simplicity in construction, they are especially suitable for being used in mountainous and hilly regions.

Crib walls

Since 1950's when crib walls were introduced to Taiwan, they remain one of the most popular retaining structures of gravity type. Free drainage and flexibility are considered as their major merits

Drainage Systems

Water is considered as the number one killer for slope stability by geotechnical engineers in Taiwan. Most people believe that the trouble with a slope can be reduced to less than a half if water, surface or subsurface, can be got under control.

There are three approaches to deal with water in slope stability. One is to intercept the surface water and the subsurface water before it reaches the unstable slope. To intercept surface water using ditches is a practice which does not very much from country to country. To intercept subsurface water, geotechnical engineers in Taiwan often use the one shown in Fig. 1. It is called subsurface drainage trench. It is good for stabilizing a slope having a pervious-impervious boundary located at a depth of less than 3 meters.

Fig 2 shows a schematic section of drainage well. With a pump, it has been used, with great success, to stabilize colluvium deposits. Drainage wells of larger diameter have been constructed to lower the water table within a lateritic slope. Small diameter horizontal drainage holes are drilled through the vertical RC wall of the well to collect water from the surrounding ground. The water in the well can then be led, by gravity in general, to a drainage system of lower elevation through a RC pipe. Drainage tunnels have been used, with great success, to stabilize slips of very large scale. Fig 3 and Fig 4 show just an example at 62K of Taiwan Area N-S Freeway (Moh and others, 1977). Together with five large diameter vertical shafts, the drainage tunnel was able to stabilize the moving mass of lateritic terrace. Drainage galleries were also constructed to stabilize a slope moving into Teh-Chi Reservoir.

NEW CONCEPTS OF SLOPE PROTECTION

Slope Protection on Forests

Deforestation is now strictly forbidden in Taiwan. It is a general believe in Taiwan that forest provides bast protection for steep slopes in mountainous area and slopeland regions.

Plantation of trees are encouraged by the government and many private nonprofit-making organizations.

Vegetation

Ita is a well known fact that the surface of soil slopes can be protected against erosion by vegetation. Many engineers thought that soil and water conservation helped nothing in slope stability but reducing erosion. However, researches an field experiments have proved that vegetative methods can help not only reducing erosion but also increasing slope stability. The following vegetative methods are suitable for various slope conditions :

a. Spray method,

b. Transplanting method,

c. Vegetative belt method,

d. Fixed frame method,

e. Pile an fence method,

f. Soil bag method, and

g. Erosion prevention blankets

Experimental slopes protected by using vegetative methods have been set up in mudstone area as well as in lateritic terrace.

A recent study (Chen and Liu 1992) pointed out that matric suction existing in unsaturated soil can raise the shear strength of soil and keep the slope stable, even nearly steeper than 70°, and that in rainy days this matric suction will gradually be destroyed due to the rainwater infiltrating into the slope and reduce the stability of the slope. This study provides positive theoretical foundation for the ideal of slope protection using vegetative techniques.

Geotextile Reinforced Retaining Wall

Under the sponsorship of NSC, Chang an others (1986, 1987, 1988, 1989, 1992) have established an instrumented geotextile reinforced retaining wall at Highway No.3 (station 358 Km + 120 to 358 Km + 160). The vertical section of the wall is shown in Fig.5. It has two tiers. The lower tier has a height of 3 m while the upper tier has a height of 2 m. Two kinds of geotextile (G1, a composite local product and N1, a woven Geotextile) and two types of fill material (flyash-cement treated weathered mudstone and alluvial sand) were used to form 4 sections. Monitoring system and numerical analysis were undertaken to understand the behaviour and performance of the wall. The wall has withstood several severe attacks by typhoons and heavy. A brand-new full size demonstration site was established in the mudstone area early this year. It is hoped that this method will be used as one of the major methods for protecting cut slopes in Taiwan.

Surface Treatment

Cut slope of mudstone is protected by liquid asphalt and geotextile sheets on the surface. The asphalt stop water from going into the slope while the geotextile sheets provides strength for slope stability.

Hybrid Methods of Slope Protection

A demonstration project has been set up in the mudstone area of southwestern Taiwan in a hope to develop hybrid methods for slope protection. The methods integrate vegetative techniques and very light engineering structures.

CONCLUSION

Slope failures are very common in Taiwan due to unfavourable natural factors as well as human factors. Traditional methods of slope protection such as bench cut, toe weight, RC retaining walls, earth anchors, gabion walls, crib walls, and drainage systems, remain popular in engineering circle. However, some methods using massive concrete structures or spray concrete have been criticised as creating visual contamination and environmental change. New concepts of slope protection and new methods of slope stability are now being developed. Experimental slopes and demonstration project have shown preliminary success. It is hope that those methods will be proved effective, economical, and in harmony with the nature. It is also hoped that those methods will be available for all developing countries in the coming years.

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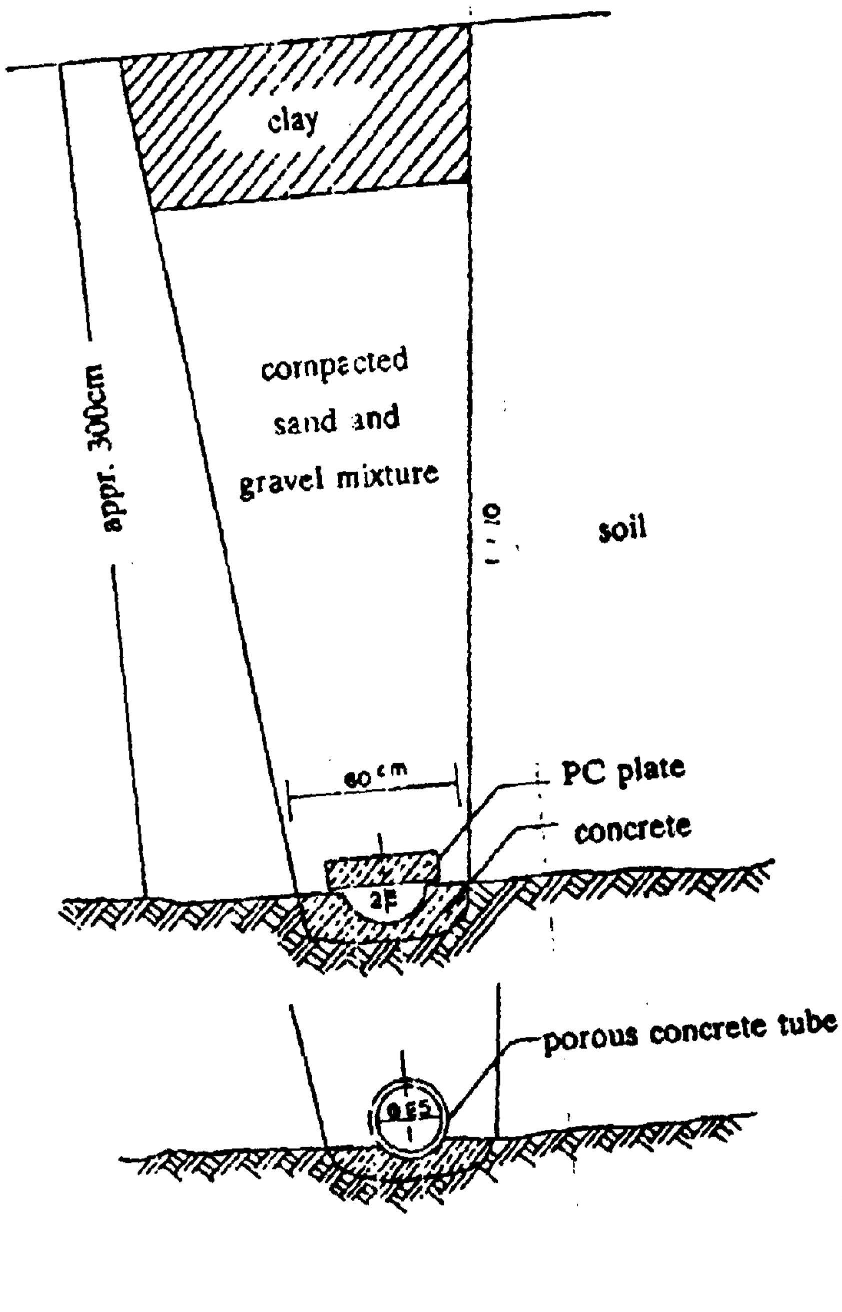

Fig. 1 A Schematic Section of Subsurface Drainage Trench

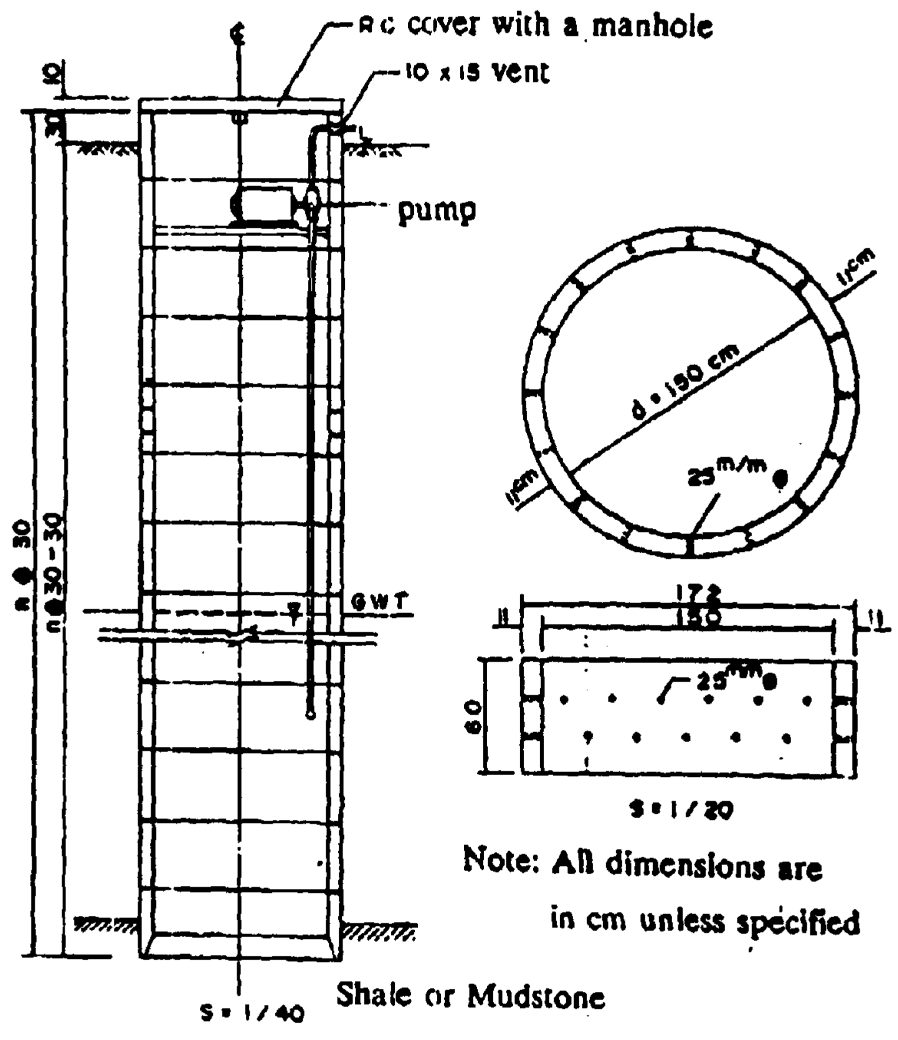

Fig. 2 A Schematic Section of Deep Drainage Well, by Pumping or by Gravity

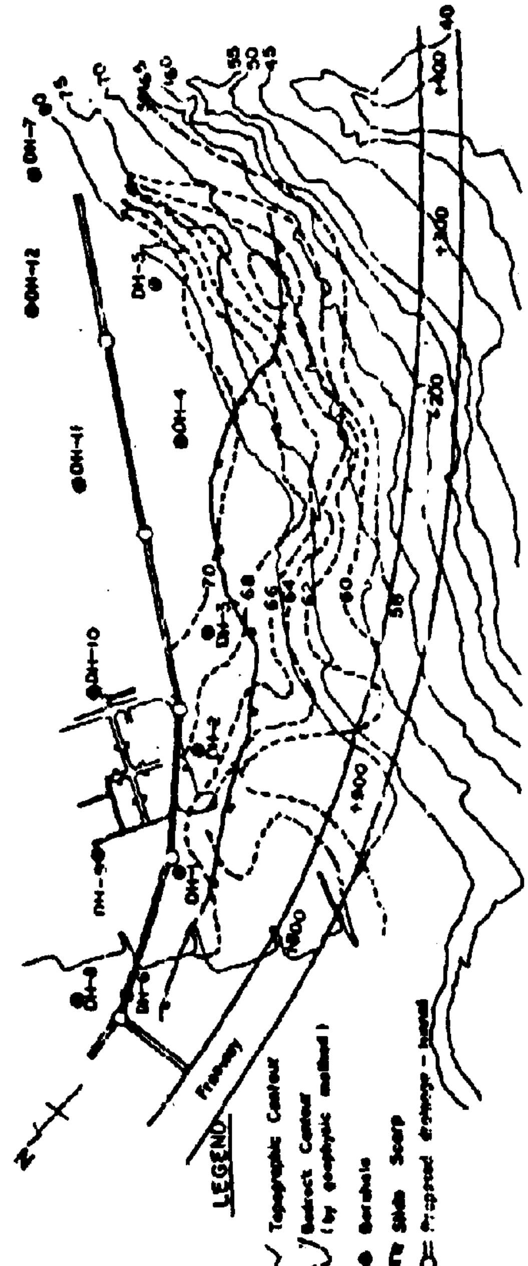

Fig. 3 Site Plan of Drainage Tunnel at 62 K of N-S Freeway (Moh and others, 1977)

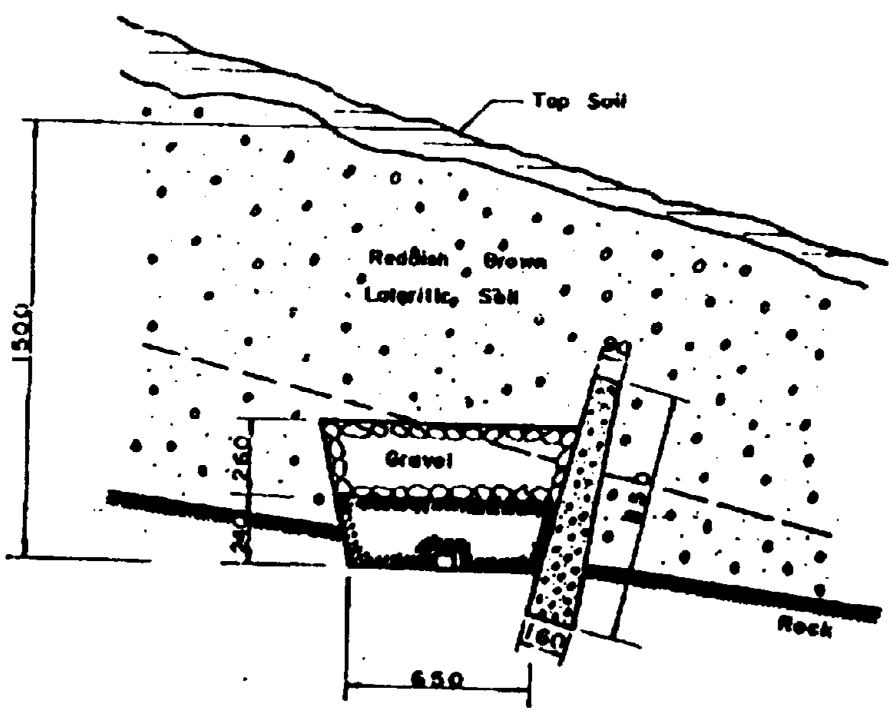

Fig. 4 Design of Drainage Tunnel at sta. 62 K of N-S Freeway (Moh and others, 1977)

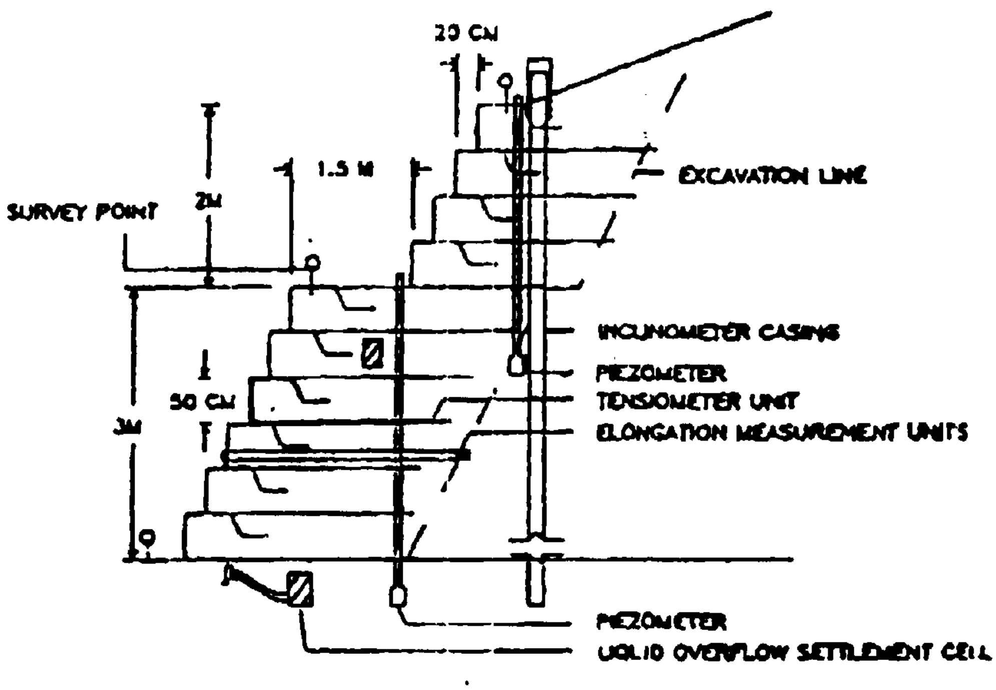

Fig. 5 Typical Cross Section of Geotextile-Reinforced retaining wall with Instrumantation System (Chang and others, 1992)

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RETENTION SYSTEMS FOR SLOPE STABILIZATION : ENGINEERING INNOVATIONS IN THE UNITED STATES

Robert L. Schuster
U.S. Geological Survey
Golden, Colorado, USA

ABSTRACT This paper reviews stabilization of rock and soil slopes in the United States by means of innovative earth retention systems. New computerized approaches are being used in design of nets, fences, walls, benches, attenuators, and ditches for rockfall control. Placed and in-situ internal reinforcement systems are used to stabilize soil slopes and embankments. New and waste materials are being used as lightweight backfills in slope-failure repair.

INTRODUCTION

Slope failures cause $1-2 billion in economic losses and 25-50 deaths annually in the United States (Committee on Ground Failure Hazards, 1985). However, effective management has restrained these losses by avoiding the hazards or by reducing the damage potential by means of physical measures that prevent or control slope failures. This paper discusses innovative earth retention methods in use in the United States as part of the overall scheme of physical control measures. The use of product, trade, proprietary, or company names is for clarity of expression, and does not imply endorsement or superiority of specific procedures or of the equipment used.

The most widely used physical measures for control of unstable slopes include:

(a) <u>Drainage</u> -- Because of its high stabilization efficiency in relation to cost, drainage of groundwater and surface water is the most widely used slope-stabilization method.

(b) Slope modification -- Increased slope stability can be obtained by removing all or part of the landslide mass or of the overlying earth that loads the slope.

(c) Earth buttresses -- Earth buttress counterforts placed at the toes of potential slope failures often are successful in preventing failure.

(d) Retention systems (often called retaining or restraining structures) -- Where methods (a) to (c) will not ensure slope stability by themselves, structural controls, such as retaining walls, piles, caissons, and rock anchors are used to prevent slope movement.

All of the above methods of physical stabilization of slopes are used in the United States. However, in the author's opinion, the greatest innovations in slope stabilization in recent years, in terms of technology, economy, and environmental characteristics, have been in rock/soil retention systems. This paper will accentuate recent advances in these methods. The discussion will be divided into use of retention systems on (1) rock slopes and (2) soil slopes, although there are obvious overlapping uses in these two categories of geologic materials. It also will deal with the use of lightweight backfill materials in control or repair of unstable slopes.

INNOVATIVE RETENTION SYSTEMS FOR ROCK FACES AND SLOPES

Recent advances have been made in stabilization of slopes that are subject to rockfall. Increasing traffic volumes on mountain roads in the United States have increased public awareness of the danger of rockfalls, resulting in significant ongoing research and development of innovative methods of rockfall control. The measures most often used to prevent rockfall from encroaching upon a highway, railway, or other structure or development are rock nets, fences, walls, benches, attenuators, and ditches. The most important rockfall input factors in design of these control measures include: (1) trajectory (height of bounce), (2) velocity, (3) impact energy, and (4) total volume of accumulation. Statistical analysis of rockfall behavior for slopes has been developed by computer modeling. One of the newest computer programs is the Colorado Rockfall Simulation Program (CRSP), which incorporates numerical input values assigned to slope and rockfall characteristics (Pfeiffer and others, 1990). The program provides estimates of probable bounce height and velocity at various locations on a slope. The Colorado Department of

Transportation is using CRSP for design of rockfall retention walls in rugged Glenwood Canyon in western Colorado (Barrett and White, 1991).

Highways and railways in the mountains of North America commonly have been protected from rockfall by traditional single-twist mesh fencing supported by fixed, rigid posts. This basic "chain-link" fence is relatively inexpensive and will effectively contain small rockfalls. A 1985 study by the California Department of Transportation (Caltrans) (McCauley and others, 1985) concluded that rolling rocks up to 0.6 m in diameter can be restrained by chain-link fence; however, this type of restraining device frequently is damaged when hit by rocks of this size and is inadequate to stop larger rocks. Thus, a field testing program of heavier "European-style" rock-restraining fences (Figs. 1 and 2) has been conducted by Caltrans (Smith and Duffy, 1990). Two types were tested by Caltrans: high-impact wire-rope net systems developed by Brugg Cable Products, Inc., (Switzerland) and Enterprise Industriele (France). Both systems rely on friction brakes; when

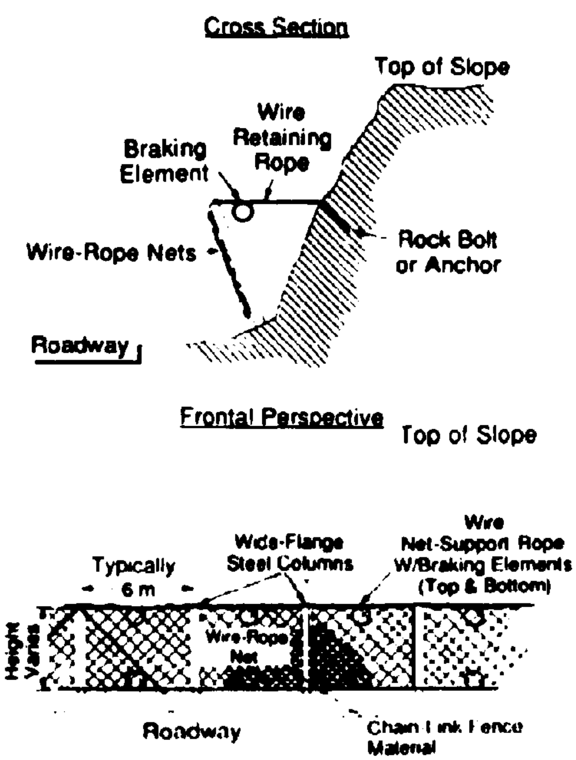


Fig. 1 Schematic cross section and frontal perspective of typical wire-rope rockfall barrier net (after Yarnell, 1991).

Fig. 2 Three-ton boulder impacting rock net in Caltrans field tests. (Photograph by J. L. Walkinshaw, Federal Highway Administration, U.S. Department of Transportation.)

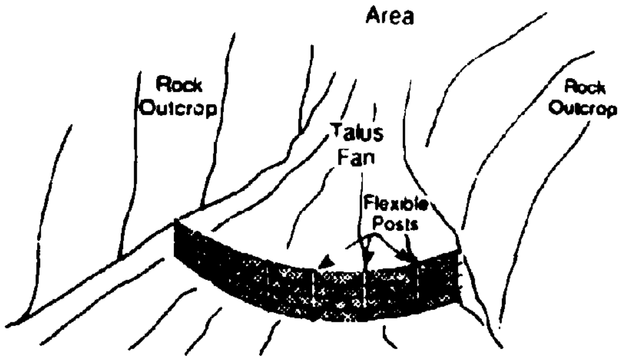


Fig. 3 Schematic diagram of Colorado Department of Transportation Flexible-Post Rockfall Fence (after Barrett and White, 1991).

bouncing rocks hit the fences, deforming the nets, energy-absorbing friction brakes are engaged, which significantly increases the capacity of the nets to restrain the rocks. This approach allows the use of lighter, less costly, fence elements.

A disadvantage of systems that use friction energy-absorbing brake systems is that the brakes require resetting after each significant rockfall, a factor in long-term maintenance costs. For this reason, the Colorado Department of Transportation has recently developed the Colorado Flexible-Post Rockfall Fence (Barrett and White, 1991) (Fig. 3). By grouting bundles of wire tendons into steel casings, posts are produced that are flexible, yet are rigid enough to support the mesh netting. In principle, the fence catches and redirects bouncing rocks to energy-dissipating

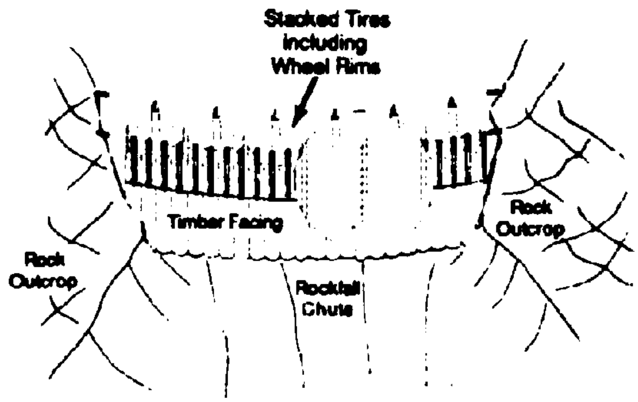


Fig. 4 Schematic diagram of Colorado Department of Transportation rockfall attenuator (courtesy of Colorado Department of Transportation).

collisions with the slope; immediately after each collision, the flexible posts rebound, leaving the fence ready for the next encounter without maintenance.

Another approach to controlling rockfall is to partially absorb or attenuate the energy of bouncing or rolling rocks without actually stopping them. The Colorado Department of Transportation has developed an attenuation system that uses columns of waste tires and rims mounted on vertical 75-mm-diameter steel pipes suspended from a large-diameter wire rope mounted across the rockfall chute (Barrett and White, 1991) (Fig. 4). Rock anchors are used to secure the ends of the wire rope to the bedrock walls of the gully. To address aesthetic concerns, a facade of wooden timbers is usually suspended from a wire rope immediately downslope of the hanging tires. The Colorado "rockfall attenuator" is designed to absorb most of the kinetic energy and to reduce rebounding heights from incoming rockfall. After a rock passes through the attenuator, the system returns to its original position without maintenance.

INNOVATIVE RETENTION SYSTEMS FOR SOIL SLOPES

Figure 5 summarizes current methods of soil retention. Externally stabilized systems (Fig. 5 a-d) rely on external structural walls against which soil forces act. Prior to the late 1960's, external walls, mainly gravity and cantilever walls, were the predominant types of retaining structures. External walls are well understood and will not be discussed here. Internally stabilized soil retention systems (Fig. 5 e-f)

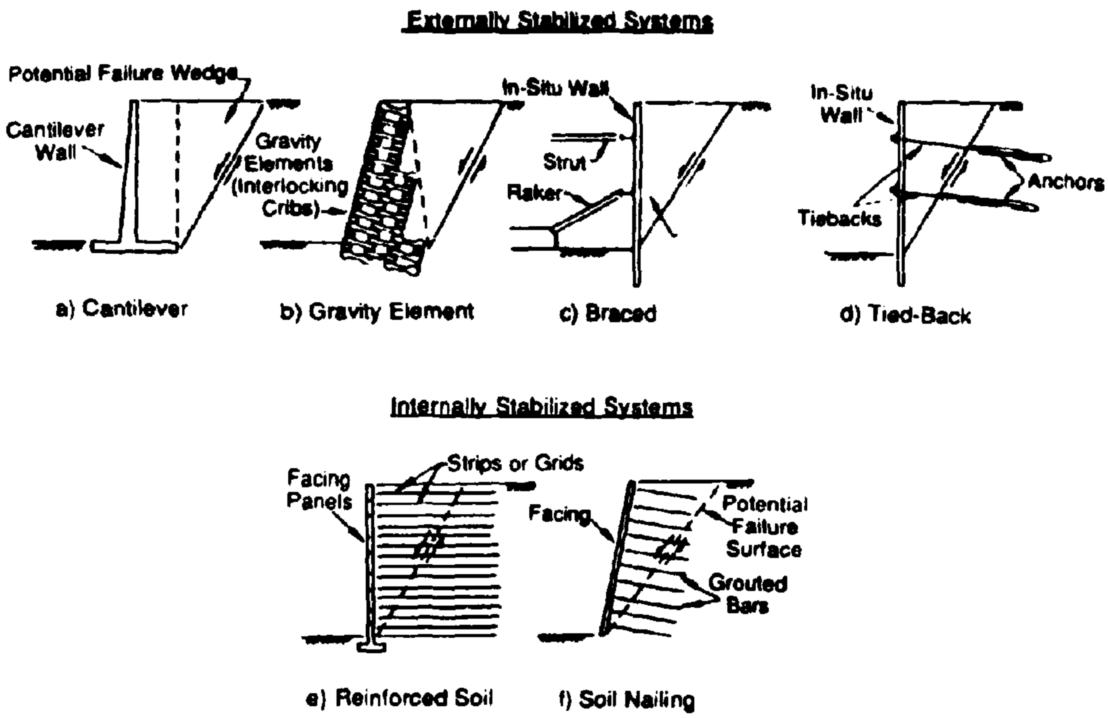


Fig. 5 Examples of externally and internally reinforced soil retention systems (after O'Rourke and Jones, 1990).

rely on reinforcement that is installed within the slope and extends beyond the potential failure surfaces into stable ground. This section will note advances in the use of these internally stabilized soil retention systems, which are known generically as "reinforced soil" (general references: Mitchell and Villet, 1987; Christopher and others, 1989, 1990; Mitchell and Christopher, 1990; and O'Rourke and Jones, 1990).

Reinforced soil (Fig. 5 e), which can be defined as the inclusion of resistant elements in a soil mass to improve its overall strength, has emerged over the past 25 years as a technically attractive and cost-effective technique for extending the use of soil as a stable construction and slope-forming material. Internal reinforcement can be used to stabilize natural slopes or the slopes of embankments, or to retain excavations. Reinforced soil structures have the following advantages over traditional external retaining walls: (1) they are coherent and flexible, and thus are tolerant of large deformations, (2) a wide range of backfill materials can be used,

(3) they are easy to construct, (4) they are resistant to seismic loadings, (5) the variety of available facing types makes possible aesthetically pleasing structures, and (6) they are commonly less costly than conventional retaining structures (Mitchell and Villet, 1987).

Internally stabilized soil retention systems rely on transfer of shear forces to mobilize the tensile capacity of closely spaced reinforcing elements, either by incremental burial to create reinforced embankment soils or by systematic in-situ installation of reinforcing elements, such as soil nails. The common types of inclusions are steel strips, steel or polymeric grids, geotextile sheets, and steel nails, that are capable of withstanding tensile loads, and, in some cases, shear and bending stress as well (Mitchell and Christopher, 1990). Internally stabilized soil reinforcement can be divided into placed soil reinforcement systems (in the United States the most commonly used are (1) strip reinforcement, (2) sheet reinforcement, and (3) grid, bar, and mesh reinforcement) and in-situ soil reinforcement systems (in the United States the most commonly used is soil nailing).

Placed Soil Reinforcement Systems

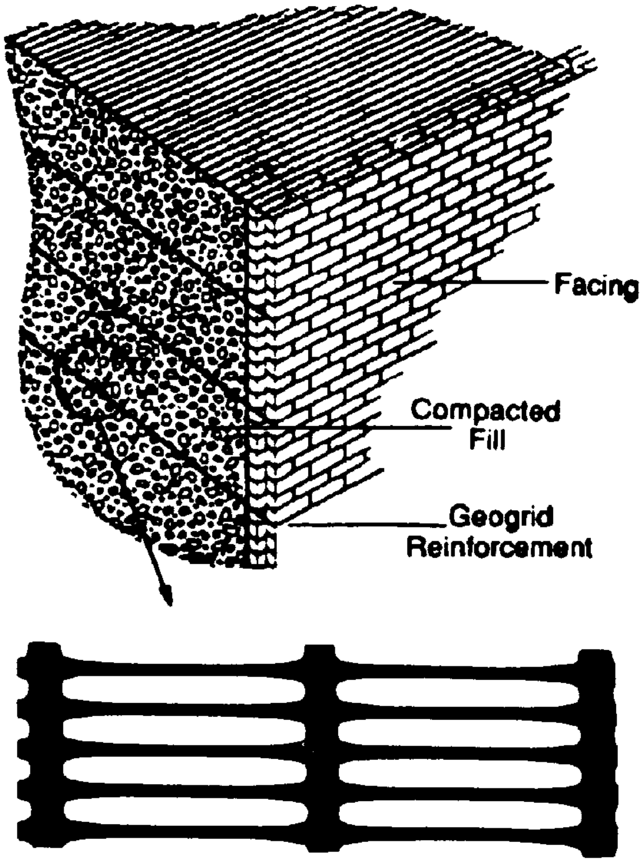
In strip reinforcement systems, a coherent strengthened material is formed by placing metal or geosynthetic strips horizontally between successive backfill layers. The modern concept of soil reinforcement by galvanized steel strips was introduced by the French engineer/architect Henri Vidal in the early 1960's. Vidal named his development "Terre Armee" or "Reinforced Earth." As of 1991, 16,000 Reinforced Earth walls with a total face area of 9,600,000 m^2 have been constructed worldwide (D. McKittrick, 1991, personal communication, Reinforced Earth Company, McLean, Virginia); about 34 percent of this total is in the United States (Schlosser, 1990). Early Reinforced Earth walls utilized stainless steel and aluminum strips. However, because of corrosion problems with these materials, all Reinforced Earth walls currently being constructed employ galvanized strips. However, even galvanized steel is subject to corrosion, and is thus restricted to use as reinforcement in cohesionless, granular, free-draining backfills to reduce the potential for chemical and water attack (Carroll and Richardson, 1986). In recent years, non-metallic reinforcing materials, such as geotextiles, fiberglass, plastics, and composites, have been used extensively for soil reinforcement. These materials do not corrode, but may undergo other forms of chemical and biological deterioration. The effects on many of these materials of long-term burial and exposure to the elements are not well known (Elias, 1990). For this reason, research currently is underway on their weathering characteristics.

Sheet reinforcement commonly consists of geotextiles placed horizontally between layers of embankment; the mechanism of stress transfer is mainly friction. A variety of geotextiles with a wide range of mechanical properties and environmental resistance can be used, including woven polypropylene and polyester and nonwoven needle-punched or heat-bonded polypropylene and polyester (Christopher and others, 1989). Granular soils ranging from silty sand to gravel commonly are used as backfill. Facing elements are formed by wrapping the geotextile around the soil at the face of the wall and covering the exposed fabric with gunite, asphalt emulsion, or shotcrete, or with soil and vegetation, for long-term protection from ultraviolet light.

Grid reinforcement systems consist of polymer or metallic elements arranged in rectangular grids, metallic bar mats, or wire mesh. The two-dimensional grid-soil interaction involves both friction and passive bearing resistance against the transverse members. The greatest advance in development of rectangular grids for soil reinforcement has been in the area of polymeric "geogrids." Geogrids are relatively stiff, netlike, synthetic materials with open spaces called "apertures" that usually measure 1-10 cm between the ribs. Manufacturing processes have evolved to the point where strong and durable geosynthetic soil-reinforcing elements can be mass produced. The most familiar products in earth retention systems are high-density polyethylene and polypropylene grids (O'Rourke and Jones, 1990). A commonly used example in the United States is Tensar (Fig. 6), a proprietary plastic grid reinforcement developed in the United Kingdom in the early 1980's.

In-situ Soil Reinforcement Systems

Soil nailing (Fig. 5 f) is the most commonly used in-situ soil reinforcement system in the United States. Soil "nails" are steel bars, metal rods, or metal tubes that are driven into in-situ soil or soft rock, or are grouted into predrilled boreholes. Together with the soil, they form reinforced soil structures capable of stabilizing slopes or of supporting temporary excavations. In usual practice, one nail serves each 1 to 6 m^2 of ground-surface area. Stability of the ground surface between the nails is provided by a thin layer (10-15 cm) of shotcrete reinforced with wire mesh, by intermittent rigid elements similar to large steel washers, or by



Plan View of Tensar Geogrid Reinforcement

Fig. 6 Schematic diagram of a geogrid reinforced soil wall and plan view of a Tensar geogrid reinforcing element.

prefabricated steel panels. The stability of soil-nailed reinforcement relies upon: (1) development of friction or adhesion mobilized at the soil-nail interface and (2) passive resistance mobilized at the face of the nail. Soil nailing is most effective in dense granular soils and stiff silty clays.

A new method of soil nailing uses geotextiles, geogrids, or geonets to cover the ground surface (Koerner and Robins, 1986). The geosynthetic material is reinforced at its nodes and anchored to the slope using long rods (soil nails) driven through the nodes (Fig. 7). When the rods are properly fastened, they pull the surface netting into the soil, placing the net ("spider netting") in tension and the constrained soil in compression.

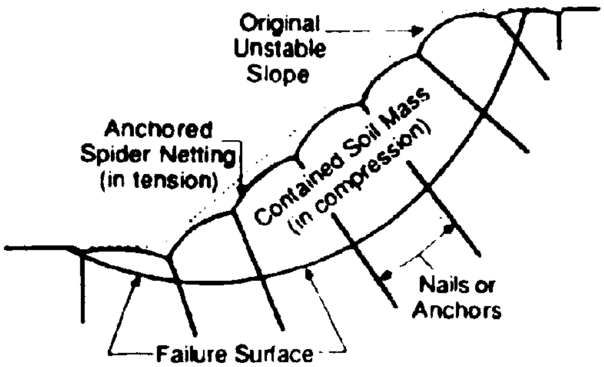


Fig. 7 Schematic cross section of anchored geosynthetic "spider netting" used with soil nails or anchors to stabilize a soil slope (after Koerner and Robins, 1986).

A recently patented (by Soil Nailing Limited, United Kingdom) soil-nailing technique inserts reinforcing nails into the ground by means of a compressed air "launcher" that was originally developed in the United Kingdom for military use in shooting projectiles into the air (Bridle and Myles, 1991). Under favorable conditions, the launcher can inject 6-m-long, 38-mm-diameter nails into a soil slope at a rate of one every 2-3 minutes.

USE OF LIGHTWEIGHT BACKFILLS TO FACILITATE SLOPE STABILIZATION

To reduce the gravitational driving force behind slope-stabilizing retaining structures and to replace slope-failure soils, various types of lightweight backfills have been employed. Sawdust, burned coal, and fly ash have been used where these waste products are available. In the past few years, two new types of lightweight materials have been used as backfill for slope stabilization in the United States: styrofoam blocks and shredded waste car and truck tires.

The introduction of superlight expanded polystyrene (EPS; styrofoam) blocks in 1972 allowed the construction of lightweight fills for highways, particularly in cold regions where it also served as road-base insulation. As superlight fill, EPS is used in the form of large blocks with a density of 0.02 t/m^3, a drastic reduction compared to other lightweight materials. The Colorado Department of Transportation has expanded the use of EPS to landslide repair. During the spring of 1987, an 8400-m^3 slide closed the eastbound lane of heavily traveled U.S. Highway 160 in southern Colorado. The slide was stabilized by means of a counterfort berm and by replacing the slide material in the highway embankment with EPS (Yeh and Gilmore, in press).

Another recently applied lightweight fill for slope-failure correction is shredded waste rubber car and truck tires. Nearly 300 million tires are discarded annually in the United States, creating a major disposal problem. Shredded tires have a compacted dry unit weight of about 0.64 t/m^3 (Humphrey and Manion, in press). About 580,000 shredded tires were used as lightweight fill in correction of a landslide that occurred in 1989 in a highway embankment on U.S. Highway 24 in the State of Oregon (Read and others, 1991). The force driving the slide was considerably reduced by replacing the slide material with the lightweight shredded tires.

FUTURE TRENDS AND NEEDS IN SLOPE STABILIZATION IN THE UNITED STATES

Research in analysis, design, and construction of measures for rockfall control and soil retention will continue to provide new approaches to the development and utilization of these slope-stabilization systems. Particularly important is the continuing development of strong, corrosion-resistant, economical, and environmentally acceptable materials that can be used as elements in slope-stabilization systems. In addition, further research and development are needed on lightweight fill materials to be used in repair of slopes or as retention-system backfill.

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SECOND US-ASIA CONFERENCE ON
ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
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MITIGATION OF GROUND FAILURES IN THIRD WORLD COUNTRIES WITH SPECIAL REFERENCE TO INDIA

Dr. Prabhat Kumar[1], and Dr. N.S. Bhal[2]
1. Scientist
2. Scientist Coordinator Structural Engineering Div.
Central Building Research Institute
Roorkee 247 667, India

ABSTRACT In view of the population explosion in the third world countries and in the name of development, the hazard prone areas will continue to be inhabited leading to a heavy loss of life during a disaster. It appears appropriate to embark upon a mitigation programme to reduce damage to public utilities and loss of human life. The disaster under examination is ground failure. Various forms of ground failures are classified and a link between environment degradation and ground failures is established. The Indian scenario on environment degradation, possible solution strategies and hurdles are described. A general mitigation programme against mass ground movement is outlined. Finally, cost-effective structural systems are described to minimize environmental damage and to discourage further mass ground movement.

INTRODUCTION

The ground failures may be classified as (1) Settlement of ground; (2) Loss of bearing capacity; (3) Mass movement of ground. Such failures occur in all countries of the world and cause mammoth loss of life and property. When such failures occur in any third world country such as

India, the loss of human lives is usually heavy. The reason is that in view of the population explosion and in the name of development even the hazard prone areas are inhabited. For example, in India, the hilly regions are being utilised for construction of dams and hydroelectric schemes for harnessing the power generation potential of rivers. The hill slopes are being utilised for exploitation of forest produce; boosting up of agricultural production; and construction of tourist complexes, defence and communication facilities.

The economic exploitation of hills and associated construction activities involve processes such as deforestation, grazing and may disrupt the natural drainage system. Several such man-made factors combine with other natural phenomena to cause environment degradation. Several forms of ground failures involving mass ground movement occur as a result of this environment degradation. The debris produced by these cause further environment degradation which in turn promotes further ground failures.

The environmental degradation and mass ground failures, thus, form a cause-and-effect circular relationship. The goals of environment rehabilitation and prevention of mass ground movement supplement one another.

INDIAN SCENARIO OF GROUND FAILURE PROBLEM

This problem can best be described by concentrating on Himalayan mountain chain because in addition to several exclusive features the associated mass ground movement problem there is also unique.

Extremes of Variation

- The Himalayan mountain chain is the tallest in the world and its sweep covers an area of about half a million square kilometers.

- Its geology is young and immature which is in a state of continuous dynamic modification.
- The rainfall is of the order of several meters in some parts while it is less than 10 cms. in some other parts.
- The temperature varies from about 30°C in summer to as low as -43°C in winter.
- The vegetation cover varies from very thick at the lower reaches to practically none at higher altitude.
- Earthquakes measuring upto 8.5 on Richter scale have been recorded.
- The size of population which depends on Himalayas for their survival and growth is far more than the total population of several countries.

Human Factor

The Himalaya is the birthplace of several mighty rivers which should have been source of water supply, irrigation, hydroelectricity and other natural resources. But unplanned and reckless human activities have turned them into threats to stability of hills as well as to survival of rich and fertile farmland which may be located thousands of kilometer away. It is generally believed that landslides invariably take place following heavy and prolonged rains. However, as pointed out by Bhandari (1987), "to name rainfall as the 'cause' would be as wrong as blaming the dynamite that rocked the building as the cause although the dynamite, the fuse, the match and the man behind the blast must all share the blame as Copartners".

Geographic Factor

A significant and strategically important factor is that Himalayas are shared by several countries-India, Nepal, Bhutan, China, Burma and Bangladesh. It so happens that the cause of a problem lies in one country while its consequences are faced by another. Ives (1985) has illustrated this point beautifully as follows: "Nepal continues to export to India and Bangladesh in ever increasing quantity the one commodity (top soil) which it cannot afford to trade & the one commodity at least in that mode of delivery that the recipients cannot afford to receive".

NEED FOR ENVIRONMENTAL REGENERATION

The stability of Himalayan ecosystem is of great importance in the economy of the nation and so is its exploitation. These two opposite views call for striking a balance which has so far been missing.

Unless the present altitude changes, many hilly regions of the world shall become graveyard of the environment. Lampe (1982) states it as follows: "without the joint efforts of all responsible groups many mountain areas of the world will be remembered in the history of twentieth century because they became man-made deserts".

The following strategies have been recommended, Singh (1985):

- Identification of geodynamically and environmentally vulnerable areas.
- Control of human intervention through multi-layered checks.
- Multiple options with implications and cost instead of present single option approach.
- Participation of local people in the process of evolving and evaluation of these options to ensure social acceptability.
- Education of local people to create awareness, to infuse confidence, to establish faith and to replace helplessness with optimism.
- Legislation against dumping of debris on hill slopes.
- Legislation against mining in sensitive areas.
- Inclusion of cost of environment rehabilitation in the cost-benefit analysis of hill area projects.
- Development of a body of professionally competent consultants independent of vested interests as well as bureaucratic pressure to monitor and provide data for long-term planning without causing ecological instability.

HURDLES AND CONSTRAINTS IN INDIAN CONDITIONS

The adoption and implementation of these recommendations are not as simple and straight forward as it may appear. The following constraints may have to be overcome :

* Honesty of poor people can be bought at a small price so that any legislation may never become fully effective.
* Most of the things are made to work perfectly only during a dignitary's visit. It creates a false impression as though the things are always the same.
* The slogan of environment protection is used for gaining popularity, vote catching and for winning elections.
* The economic viability of any project is measured on a short term basis.
* As a consequence of the above, the available funds are utilised in solving 'more important and more immediate problems'.
* In a seniority oriented system the expertise of a person is judged by his rank alone.
* The technical experts are often allured by the prospects of a foreign visit and salaries in foreign currency.
* It is not uncommon to find technical experts who adopt a negative approach and criticize everything.
* In the absence of a strict and honest supervision, the implementation of various remedial measures are faulty.
* The maintenance is virtually non existent and unreliable.
* The instruments deployed for monitoring either malfunction or fail to function at all. An automatic and remote monitoring is hampered due to frequent breakdown in power supply and severe voltage fluctuations.

A GENERAL MITIGATION PROGRAMME

It is seen that the problems of environment rehabilitation and ground failure prevention are complex and throw open a challange to the engineering profession.

It is generally believed that the mitigation programme against mass ground movement requires colossal amount of money. Such a belief is erroneous because the damage to environment can also be evaluated in monetary terms. This damage has been accumulating over the number of years and shall continue to accumulate further unless it is stopped. Coordination, planning, cooperation and strict as well as rigid monitoring of the mitigation schemes are the essential ingradients for their effective implementation.

Engineering aspects may be subdivided into pre-failure mitigation programme consisting of preventive measures and post-failure programme consisting of corrective measures. Both preventive and corrective measures are equally important. However, preventive measures may be adopted in the future activities only. The existing construction can only be strengthened as a part of corrective measures.

A typical mitigation programme against ground failures may consist of the following steps.

* Study of literature and past records
* Risk calculation
* Determination of geological conditions
* Evaluation of geotechnical parameters
* Symptom analysis and diagnosis
* Evaluation of preventive/corrective measures
* Implementation and monitoring of mitigation measures
* Documentation for future reference

Through the study of published literature, the following conditions are found to favour and promote ground failures. These may be used in the symptom analysis and diagnosis.

* Discontinuities dipping down the slope
* Alternate soft and hard rock formations
* Loss of strength on saturation
* Alternate permeable and impermeable beds
* Blocked drainage and accumulation of water
* High horizontal component of initial stress field.

STRUCTURAL SYSTEM TO CHECK GROUND MOVEMENT

A large number of preventive/corrective methods are available which have been tried and found to be successful to various degrees. Quite a few of these have been modified to suit local conditions in India, particularly the cost. An inventory is given by Bhandari (1987). The aim of these cost effective measures is to organize the construction activities to minimize damage to the environment and to discourage further mass ground movement. The cost-effectiveness is ensured by using debris of hill area construction as the construction material.

Organised Dumping of Debris Along Hill Slopes

The debris produced in cutting of hill slopes is usually allowed to roll down the slopes. This mode of debris disposal causes environmental degradation. This practice may be banned but it will add significantly to the cost of hill road construction. It may be possible to seek a compromise by constructing icelands of debris along the hill roads as shown in Fig. 1. The weight of the debris deposited adds to the stability of slope and the debris is always available for aggregate production and recycling.

Improved Construction of Drum Retaining Wall

An ingenious way of utilizing empty bitumen drums and the debris to construct a retaining wall was proposed by Bhandari (1988). Some modifications to make the drum wall more rigid and stable against sliding are proposed as illustrated in Fig. 2.

The above proposals are in conceptual form as field trials are still awaited. In both of these proposals, the debris produced by slope cutting is the chief construction material. Its proper utilization ensures that it is no longer available to cause environment damage.

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Fig. 1 Schematic representation of organised dumping of hill road construction debris

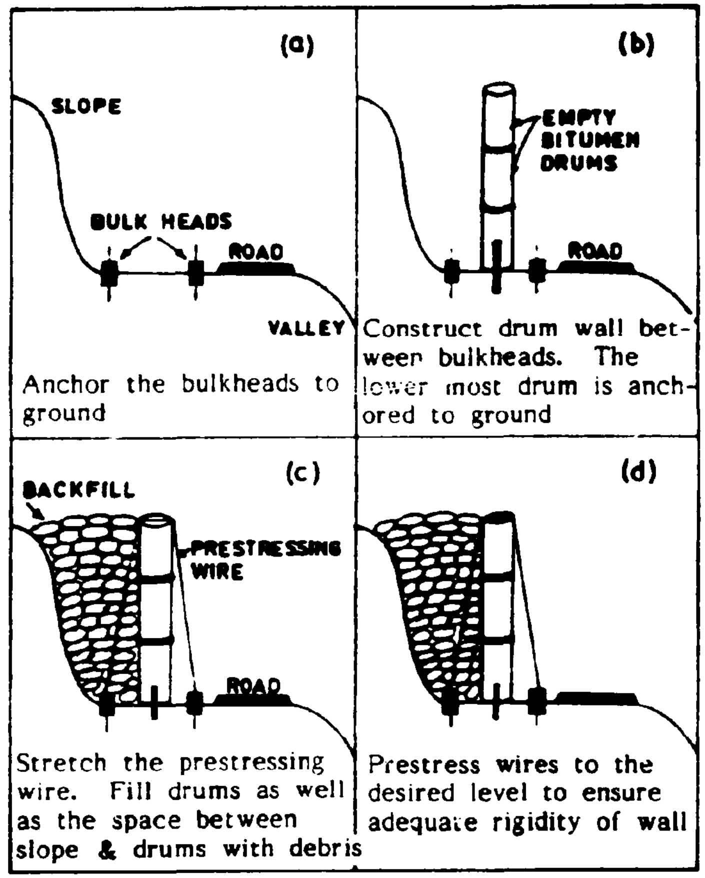

Fig. 2 Construction sequence of an improved drum retaining wall which uses road construction debris

SECOND US-ASIA CONFERENCE ON
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MECHANICALLY STABILIZED EARTH (MSE) FOR MITIGATION OF EMBANKMENT FAILURES

D.T. Bergado, H.O. Abiera and M.C. Alfaro
Division of Geotechnical and Transportation Engineering
Asian Institute of Technology
Bangkok, Thailand

ABSTRACT: Mechanically stabilized earth (MSE) consists of inserting tensile resistant materials in compacted soil materials to improve its mechanical properties. This study investigated the mechanisms involved in MSE construction with both extensible and inextensible reinforcements using laboratory direct shear and pullout tests. It was found that grid reinforcements made of 1/2" diameter steel bars generated the highest pullout capacity while Tensar SS2 grid has the least. The test results were applied to an embankment adjacent to canal excavation. The presence of tension cracks in the embankment and the lowering of water level in the canal proved to be critical for the stability of the embankment. A layer of either 1/4" or 1/2" steel grid reinforcements or 2 layers of Tensar SR80 grids or Bamboo grids are needed for stability purposes during critical periods.

INTRODUCTION

The main foundation problem for embankment construction in the Chao Phraya Central Plain of Thailand is the presence of thick and soft clay deposits which will lead to consolidation settlements and slope instability. Thus, the embankments cannot be constructed very high and the embankments are usually constructed with very flat slopes or berms (Fig. 1a). A more economic solution can be achieved by using a basal layer of geogrid reinforcements (made of bamboo, steel, or polymer materials), placed over the original ground before placing the embankment fill (Fig. 1b). If correctly designed and installed, the reinforcement will impart tensile strength to the base of the fill, thereby,

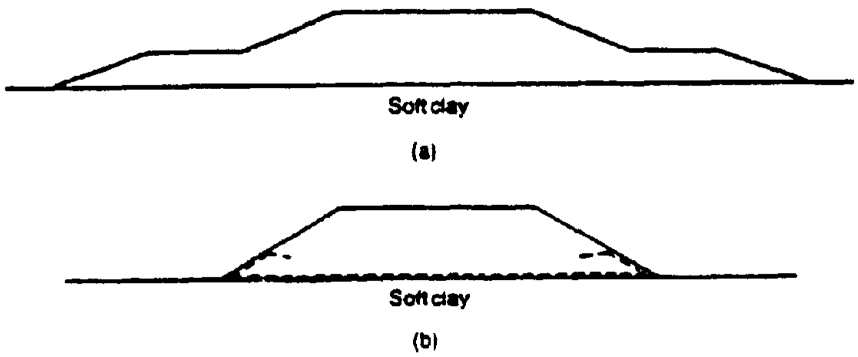

Fig. 1 Embankments over soft ground
a) with berms; b) with geotextiles reinforcement

lateral spreading, rotational failure or extrusion of the underlying soft ground are minimized. In addition, since the cost of granular and ideal material is very high, local materials excavated near the site are usually utilized for construction. Hence, canal excavations are usually located near earth embankment constructions. For such kind of situation, lateral spreading of embankments towards excavations and subsequent embankment stability failures are the biggest concerns.

Usually for embankment construction, the factor of safety against slope instability is quite low. Moreover, in cases wherein embankment cracks occurred due to the lateral spreading and subsequently filled with rain water, the embankment will reach criticdl condition. It is the aim of this study to investigate and apply the use of mechanically stabilised earth (MSE) embankment using grid reinforcements to remedy and mitigate the problem. Mechanical stabilisation by reinforcements allows for steeper slopes and consequent savings in volume and cost of embankment fill materials. The savings in the costs of backfill may exceed the costs of reinforcing materials.

Two master's theses researches have been done recently with topics directly related to the solutions of the dike instability problems. One research concerned partly with the evaluation of in-situ properties of the subsoil at the Bangpain site (Chen, 1991) and the other concerned with the pullout capacity of steel, bamboo, and Tensar grids reinforcements using surface deposits of the weathered clay as backfill materials (Abiera, 1991).

MECHANICALLY STABILIZED EARTH (MSE) EMBANKMENTS

Mechanically stabilized earth (MSE) is formed by inserting reinforcement which is strong in tension into compacted

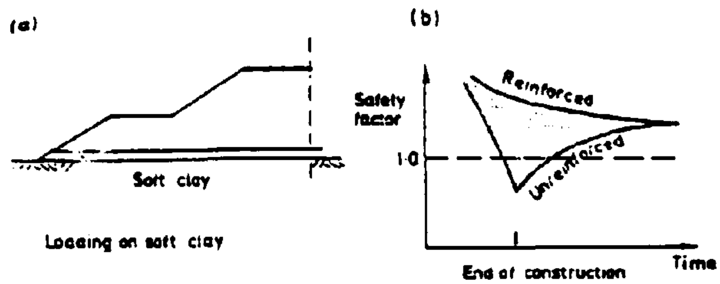

Fig. 2 Illustration showing the use of reinforcement on embankments

earth mass which is strong in compression forming a strong and semi-rigid composite material. The tension in the reinforcement is mobilized by the interaction between the reinforcement and the backfill soil in the form of friction and bearing resistances. MSE embankments can be used in conjunction with low cost, poor quality backfill materials. Moreover, in this construction, locally available and cohesive backfill materials together with light construction equipments can be used to reduce overall construction costs.

For an embankment on soft ground, the reinforcement in the MSE construction is only required to maintain stability during construction and during consolidation of the soft subsoil until the shearing resistance of the foundation soil has increased sufficiently to maintain stability without the additional benefit of the reinforcement (Fig. 2a,b). Figure 3 summarizes the mechanics of reinforced embankment on soft ground. The reinforcement hold the outward thrust of the embankment fill in equilibrium with the tensile forces. The reinforcement also restrain the surface of the foundation soil against lateral displacement. Thus, not only lateral spreading is prevented but also the slope stability and bearing capacity is increased.

SAMPLING, TESTING AND SOIL PROFILE

The testing and sampling sites were located in Bangpain Industrial Estate. The Bangpain Industrial Estate is located about 15 km north of AIT Campus along Highway No. 308. The AIT Campus is located 42 km north of Bangkok Metropolis. The "undisturbed" samples were obtained by thin-walled Shelby tube samplers in conjunction with the wash boring drilling technique. The sampling was done at depths of 0.50 m to 8.0 m. After sampling, the Shelby tubes were sealed with paraffin wax and stored in the moist room at AIT Soil Laboratory. The groundwater level was found to be at 0.50 m depth (December, 1990).

The disturbed weathered clay samples from Bangpain Site were also collected for tests as backfill materials in mechanically stabilized earth (MSE) construction. This weathered clay soils at the site were generally reddish-brown in color. It was excavated from 0.50 m to 1.50 m depth adjacent to the existing embankments. These weathered clay soils were used as backfill soils or construction materials for the embankments.

Laboratory tests were performed on the "undisturbed" samples consisting of consolidation tests, unconfined compression and triaxial CIU tests as well as index and classification tests. The disturbed weathered clay specimens were subjected to index and classification tests, compaction tests, direct shear tests, and pullout tests. Details of pullout test using grid reinforcements with low-quality

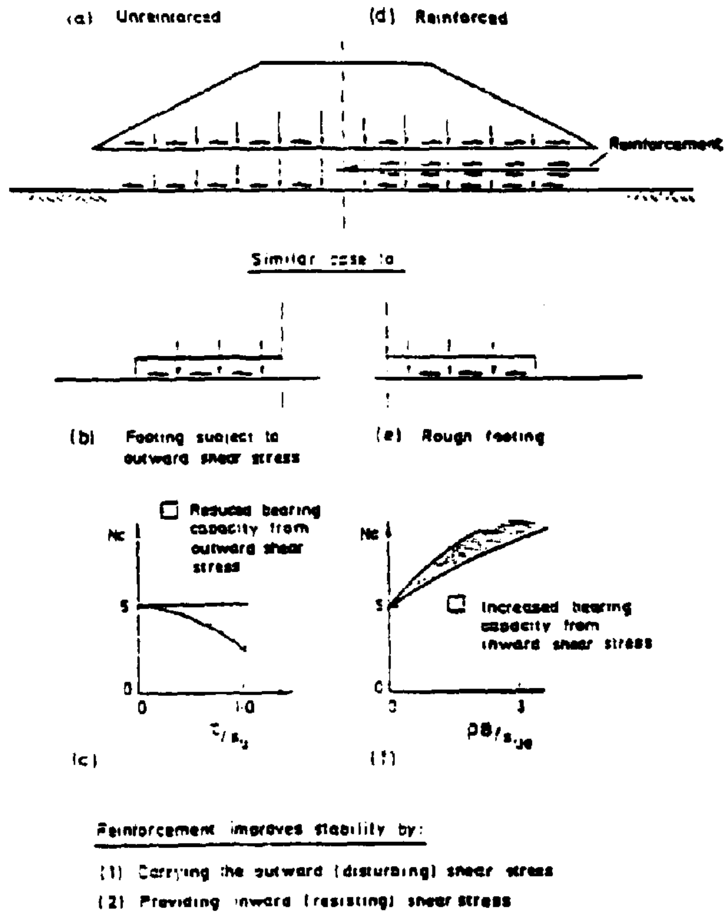

Fig. 3 Illustration summarizing the mechanics of a reinforced embankment on soft soil

backfill soils have been published elsewhere (Bergado et al, 1992). The specimens of the reinforcements consisted of bamboo, tensar, and steel grids were used in the pullout tests. The in-situ tests consisted of field vane shear tests, screw plate tests, and pressuremeter tests. These tests were made to obtain the compressibility and strength characteristics of the subsoil at in-situ conditions.

The subsoil profile together with the index soil properties are shown in Fig. 4. It can be seen that the subsoil consists of an uppermost 1.0 m thick weathered clay underlain by a 5.0 m thick very soft to soft clay and then underlain further by a stiff clay layer. The undrained shear strength with depth obtained from the different laboratory and field tests is also given in Fig. 4. Some typical load-displacement curves from pullout tests using bamboo, Tensar, and steel grids reinforcements is shown in Fig. 5. Subsequently, the relationship between the total pullout resistance and effective overburden pressure were computed and plotted in Fig. 6.

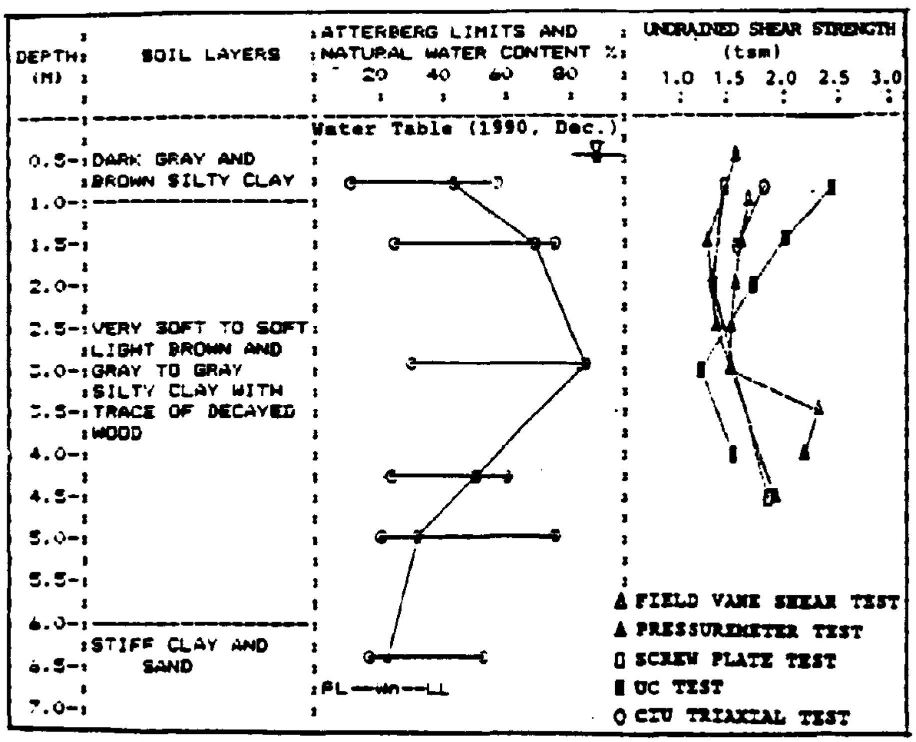

Fig. 4 Soil profile and general soil properties at Bangpain site

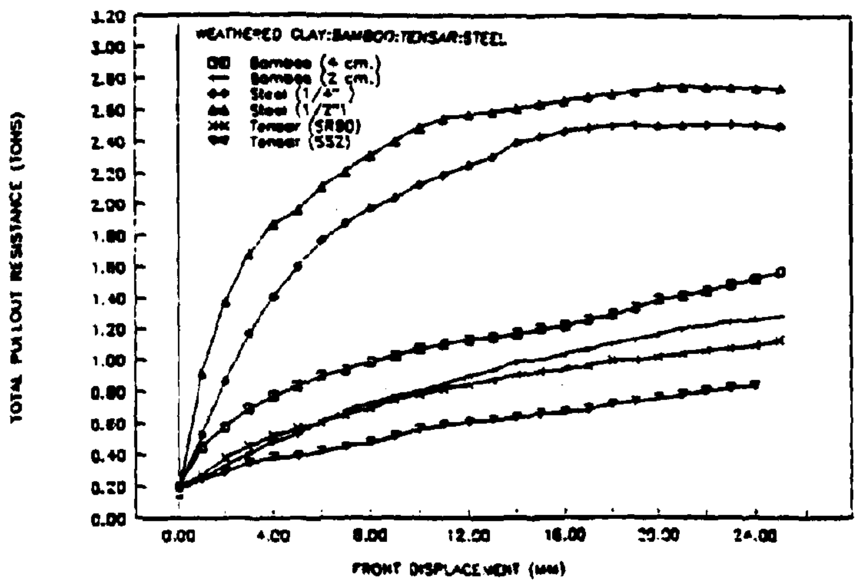

Fig. 5 Load-displacement curves for bamboo, Tensar and steel grids at 1 tsm normal stress

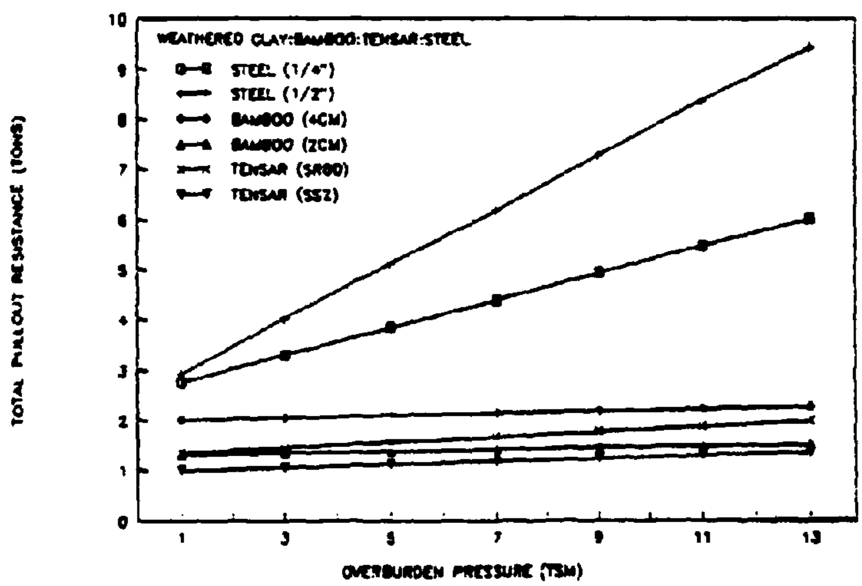

Fig. 6 Comparison of total pullout capacity of bamboo, Tensar and steel grids from the actual test results

SLOPE STABILITY ANALYSES OF UNIMPROVED EMBANKMENT

Slope stability analyses using micro-computer software, SB-SLOPE, have been done based on the existing geometries (see Fig. 7) of the unimproved embankment. The conditions of each case are described below:

1) The depth of canal excavation is 3.0 m and the water level in the canal at 1.0 m above the bottom of the canal. There is high water level behind the dike embankment such that the phreatic surface is 1.0 m below the top of the embankment. In this case, the lowest factor of safety is found to be 1.20.
2) The geometries and boundary conditions are the same as case 1, except that the water level in the canal excavation is 2.0 m above the canal bottom. The resulting factor of safety was higher than case 1 as expected.
3) The depth of canal is 3.0 m and the water level is at 2.0 m above the canal bottom. The phreatic surface is located at the natural ground surface. There is tension crack in the embankment down to 2.0 m depth. The factor of safety was computed as 1.11.
4) The same condition as case 3, except that the water level in the canal is lowered to 1.0 m above the canal bottom and the phreatic line is located 1.0 m below the natural ground. The factor of safety decreased as expected to failure conditions.
5) The same conditions as case 3, except that the canal is dry and the phreatic line is located at 2.0 below the natural ground. This time the factor of safety is 0.95 which is failure condition (see Fig. 7).

Thus, both conditions such as the presence of tension cracks in the embankment and the lowering of the water level in the canal excavation can have a devastating effect on the stability of the dike embankments.

SLOPE STABILITY OF IMPROVED EMBANKMENT

Improved embankments in this case means mechanically stabilized earth (MSE) embankments using grid reinforcements. Two essential cases were considered namely: one layer and two layers (Fig. 8) reinforcements. The reinforcements used in the analyses consist of steel, bamboo and Tensar grids. The results of the slope stability analysis of the improved embankments are given in Table 1. As shown, the factor of safety of the unimproved embankment is at failure condition at 1.01. In contrast, the values of the factors of safety for the improved embankment have increased to safe levels. As expected, the 2 layer reinforcement yielded higher factors of safety. Also as expected from the pullout capacity, the steel grids made of 1/2" welded steel bars

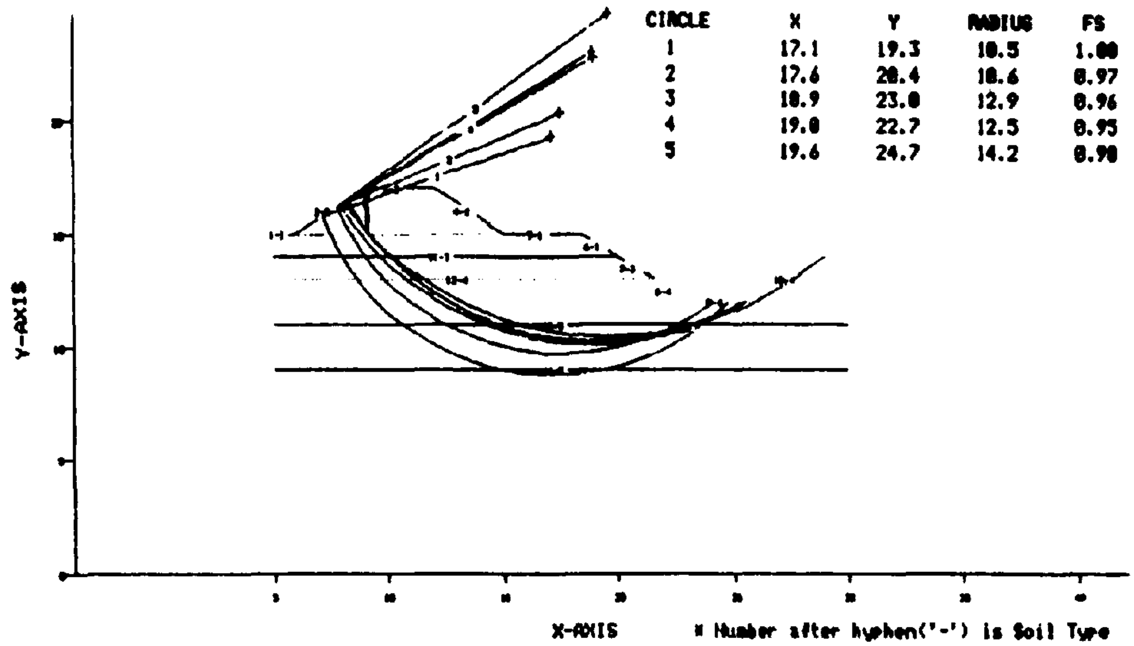

Fig. 7 Slope stability analysis of unimproved embankment (case 5)

TABLE 1 SUMMARY OF FACTOR OF SAFETY OF IMPROVED EMBANKMENT

Reinforcement	Factor of Safety	
	1 Layer	2 Layers
Steel (1/4")	1.286	1.435
Steel (1/2")	1.334	1.499
Bamboo (2 cm)	1.225	1.299
Bamboo (4 cm)	1.243	1.340
Tensar (SR80)	1.229	1.309
Tensar (SS2)	1.218	1.281

Factor of Safety without Reinforcement = 1.01

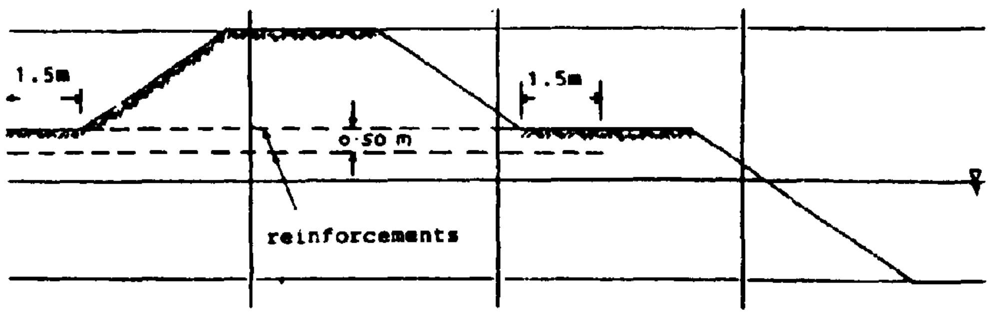

Fig. 8 Layout of two layers of grid reinforcements

yielded the highest factor of safety. The lowest factor of safety resulted from the use of Tensar SS2 geogrid as expected. This reinforcement is the weakest. However, all reinforcements being analyzed resulted in improving the factors of safety to safe levels.

CONCLUSIONS

From the results of this study, the following conclusions can be made:

1) The occurrence of tension cracks in the embankment, coupled with the lowering of water in the adjacent canal is critical for the stability conditions of the dike embankments.
2) The use of mechanically stabilized earth (MSE) embankment significantly increased the stability of embankments on soft ground especially near excavations.
3) One layer of steel grid reinforcements consisting of 1/4" or 1/2" diameter and 6"x9" mesh size is sufficient to stabilize the embankments. Two layers of either bamboo grids or Tensar SR80 grids are needed for embankment stability.

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SECOND US-ASIA CONFERENCE ON
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PREDICTION AND MAPPING OF LANDSLIDE HAZARDS

Tien H. Wu
Dept. of Civil Engineering
Ohio State University
Columbus, Ohio 43210 USA

ABSTRACT The paper outlines a methodology for prediction and mapping of landslide hazard in shallow soils on hillside slopes. The principal steps are estimation of infiltration and groundwater response, estimation of failure probability, mapping and updating with results of landslide inventory.

INTRODUCTION

Landslides constitute one of the major natural hazards that cause losses in lives and property. As a general principle, the choice among different management or mitigation options, including avoidance, evacuation, repairs or stabilization, should be based on cost, which should include the direct or initial cost, such as construction or removal, and the costs of lost opportunity and of potential failures. Because future events cannot be forecast with certainty, management decisions are made under conditions of uncertainty and with incomplete information. In pobabilistic decision theory, the optimum choice is the option with minimum expected cost. Expected cost is defined as

$$E\,[C] = C_0 + P_f\ C_f \qquad [1]$$

where C_0 = initial cost, P_f = probability of failure, and C_f = consequence or cost of failure, which can include lost of opportunity. In many publications, P_f is also called hazard and

$$R = P_f C_f \qquad [2]$$

is called risk.

Hazard prediction and mapping is the first step in mitigation. After the zones of different hazards and risks have been identified, various processes for mitigation can be considered. This paper focuses on hazard prediction; technologies for mitigation and costs are not covered.

The term landslide includes all mass movements of soil and rock that occur on slopes. This paper describes methodology for landslides induced by high porewater pressure in shallow soils on hillside slopes. According to the principle of effective stress, increase in porepressure reduces the effective stress in the soil and and reduces the shear strength of the soil. Shear failure occurs when the shear strength is reduced to the critical value required for limiting equilibrium. Hence, in humid regions with seasons of heavy precipitation, the first task in landslide hazard prediction is prediction of the porepressure response to precipation.

PREDICTION OF POREPRESSURE

Prediction of porepressure includes evaluating the precipitation and snowmelt, porepressure response to infiltration, and slope stability. Sources of information include soil survey reports, geological maps, and judgement and opinions derived from field surveys and in-situ measurements. Three levels of hazard prediction are made. The first is based only on average conditions for a region and uses objective information derived from maps and reports. The second level is a modification to account for spatial variations in properties and conditions. Observations on materials of similar geological origin and experiences within the region under study are used to derived the parameters. The third level introduces features observed in field inspection in specified locationss within the region. Predictions at each level is made with inputs, whose mean reflects the best estimate, without incorporating conservatism, and whose variance represents the engineer's uncertainty.

Average Conditions Several models of infiltration and groundwater flow are available (Beven,1981,Sloan and Moore,1984,O'Loughlin,1986, among others). We have used the kinematic storage model with a component for infiltration through the unsaturated zone (Reddi and Wu, 1991) summarized in Appendix A. This is used to compute the groundwater response under average site conditions, which denote the average values of slope (α), catchment shape (B), soil thickness (D), and storage coefficient (C), and permeability (K), that are given in soil survey reports. These conditions represent the best information available in the absence of specific site investigation. This may be considered as the reference state and serves as an indicator of the groundwater level in a slope.

Because the lumped parameter solution assumes a simplified groundwater profile, a better estimate of the gropundwater profile is obtained by calculating the saturated flow by the finite difference solution (Lee, 1986). This is used to identify the zone of high groundwater level within a basin.

Local Variations The second level is an investigation of the effect of local variations in site conditions on the groundwater response. Local variations in bedrock slope, soil thickness, and soil properties are introduced into a finite difference analysis (Lee and Wu, 1987). The spatial variations or departure from the mean trend can be expressed as a variance Var [.] and a correlation distance, δ (Vanmarcke,1977). The measures of local variations are estimated from available data from other sites plus observations in the region under study. The effects are added to the reference state.

Geological Anomalies In the third level effects of geological anomalies on groundwater response is investigated by the finite element method. Geological anomalies include all geologic features in the region that depart from the reference state. These include presence of weathered zone in bedrock, joints in bedrock, pervious inclusions in soil layer. This is derived from observations of geology, slope, and soil characteristics made in the field.The effects are also added to the reference state. The three levels represent progressive refinements in the estimation of groundwater response.

LANDSLIDE HAZARD MAPS

Landslide hazard is expressed as

$$P_f = P\ [\ H_w > H_c\ ] \qquad [3]$$

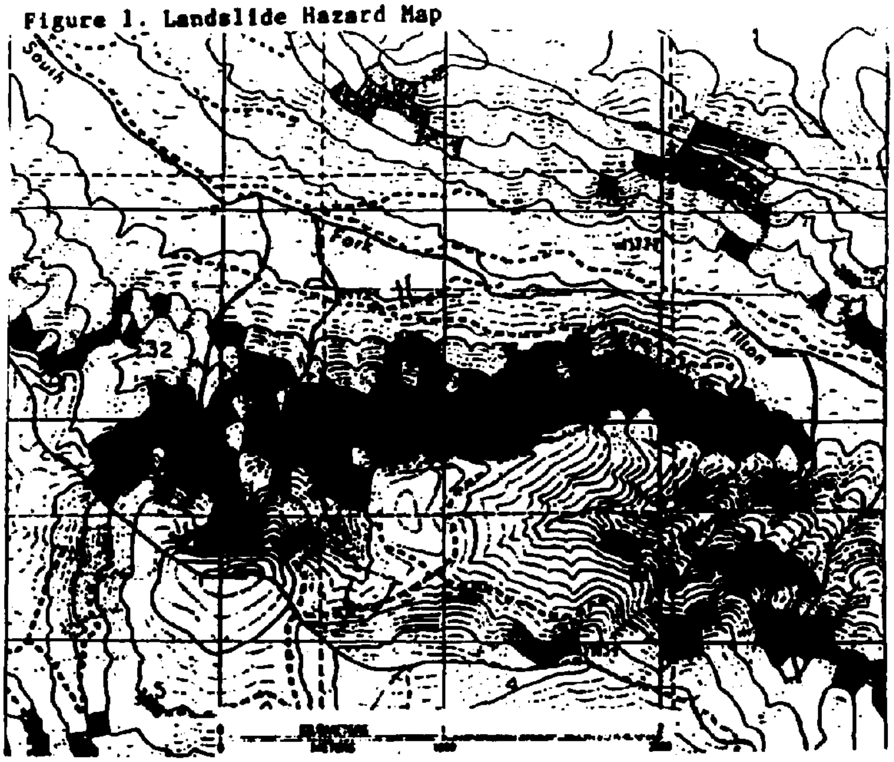
where H_w = height of groundwater level, (Fig.A.1), H_c = critical groundwarter level required to produce a slope failure. The value of H_c is determined by a stability analysis for an infinite slope. Uncertainties about input and model error are represented by N_i for the i th source. Then

$$P_f = P\ [\ N\ H_w > H_c\ ] \qquad [4]$$

where $N = \prod N_i$. The effect of local variations and of geologic anomalies are treated in the same manner.

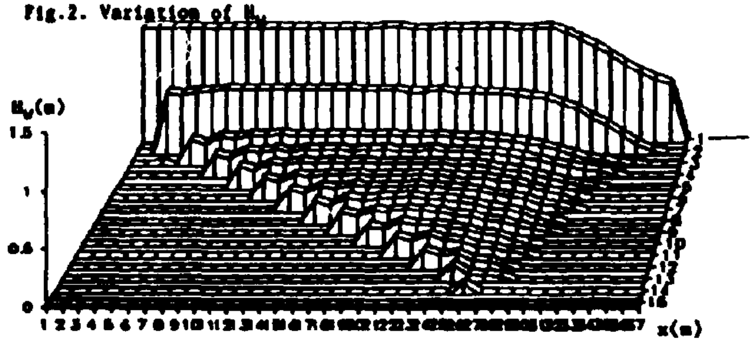
A landslide hazard map shows the region within which P_f falls in a certain range during a storm of given magnitude. Fig. 1 is an example of a landslide hazard map for Focus Township, in the Cascades Mountains of Washington, constructed from average site conditions. The shaded areas represent a failure probability 0.1 under a 10 year storm. This may be considered a macro-map as it represents site conditions averaged over a large area (10 km)$_2$. Fig.2 shows the variation of H_w within a catchment, which may be a portion of a slope. This is used to

Figure 1. Landslide Hazard Map



construct a micro-map, that covers an area of $(50m)^2$, and serves to indicate locations on a slope or within a catchment where failures are most likely to occur. Local variations and geological anomalies can be introduced into either the macro or micro maps depending on the scale of the variations and anomalies.

Fig.2. Variation of H_w



A geographic information system (GIS) is used to construct the various maps. The GIS is used to identify areas where combinations of input parameters (slope, soil depth, etc.) would result in failure probabilities, $P_f < 0.01$, 0.1, etc.

UPDATING

The computed hazard (Fig.1) is compared with the results of landslide inventory, in which landslides are identified from air photos and site inspection. The results of the landslide survey is used to update the computed hazard by means of Bayes' Theorem

$$f[\theta_i|z] = \frac{P[z|\theta=\theta_i]\, f[\theta=\theta_i]}{\Sigma P[z|\theta=\theta_i]\, f[\theta=\theta_i]} \quad [5]$$

where z = observed outcome = failure or no failure and θ = parameter used in the prediction model. $P[z|\theta]$ = the likelihood function

$$L = P[z|\theta_i] = \Pi P[z_j|\theta_i] \quad [6]$$

where z_j = observation in area j of region 1, where the inventory is made. The updated parameter θ may be considered to be calibrated against the observations in Focus Township and may be used to extend the hazard map to regions outside of Focus Township, provided the site conditions are similar.

SUMMARY AND CONCLUSIONS

The methodology for prediction and mapping of landslide hazard considers uncertainties about the input parameters to the infiltration and groundwater model and the slope stability model. It incorporates data from pubished sources and site investigations. Subjective opinions can be included and updating can be made after landslide inventories.

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APPENDIX. THE LUMPED PARAMETER MODEL

The lumped parameter model of Reddi and Wu (1991) considers infiltration through the unsaturated zone, Fig.A.1a, and the drainage by gravity flow in the saturated zone, Fig.A.2b. The governing equations for infiltration are:

$$v_j = 0.5K(\theta)\left[\frac{\Phi_j - \Phi_{j-1}}{z_j} + 1\right] + \left[\frac{\Phi_{j+1} - \Phi_j}{z_{j+1}} + 1\right] \quad (A.1)$$

$$v_0 = (q - \alpha E_e) \quad (A.2)$$

$$i = v_2 - \Delta\theta_3 z_3 \quad [A.3]$$

vj-1, v_j = velocity at the top and bottom of the j th layer (Fig. A.1), q = rainfall E_a = equilibrium evapotranspiration, κ = a coefficient , K = permeability coefficient, θ = volumetric moisture content, θd = drainable volumetric water content, i = infiltration into whe saturated zone. The drainage rate is

$$v = K_s \sin\alpha \quad [A.4]$$

where K_s = saturated permeability. The groundwater level at time 1 is

$$h_d^1 = \frac{h_d^0 [L\theta_d - v\Delta t]}{L\theta_d + v\Delta t} + \frac{2Li\Delta t}{L\theta_d + v\Delta t} \quad [A.5]$$

From this,

$$h_d^1 = h_d^0 \frac{1 + \lambda}{1 - \lambda} + \frac{2i\Delta t}{\theta_d(1 + \lambda)} \quad [A.6]$$

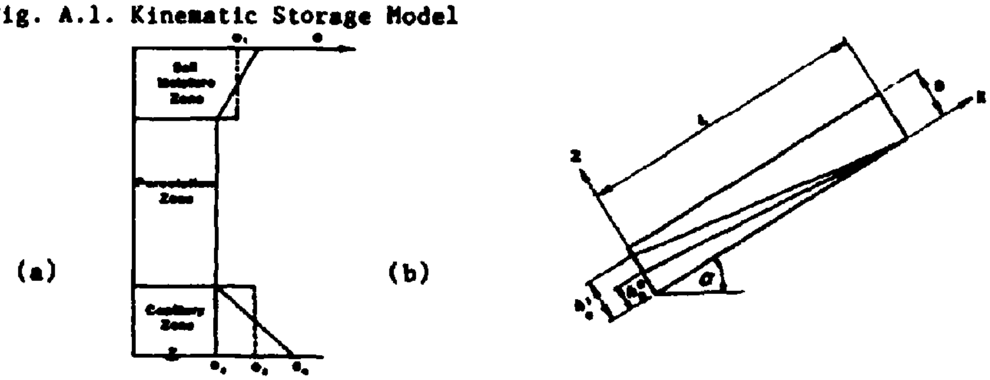
where

$$\lambda = \frac{v\Delta t}{L\theta_d} \quad [A.7]$$

For small values of v or λ ,

$$\frac{h_d^1 - h_d^0}{D} \to \frac{2i\Delta t}{D\theta_d} \quad [A.8]$$

Fig. A.1. Kinematic Storage Model



SECOND US-ASIA CONFERENCE ON
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MOUNTAIN RISK ENGINEERING FOR LINEAR INFRASTRUCTURES

Birendra B. Deoja
Deputy Director General
Department of Roads, HMG Nepal

ABSTRACT A simple and systematic technique for hazard and risk assessment at the pre-feasibility and feasibility stage of an infrastructural project, such as roads and canals in the mountain terrain, is a first step towards engineering for mitigation of natural hazards influencing the road or canal. Roads in the mountainous region of Nepal have indicated that traditional engineering in planning and design of roads result in (i) either very expensive rehabilitations from frequent failures, or (ii) massive enviromental deterioration from indiscriminate slide clearance, hill cutting, and spoils disposal due to cost and time constraints. Mountain Risk Engineering concepts and methods developed and applied so far have been presented with examples in Nepal.

INTRODUCTION

Traditional civil engineering is oriented to plain areas and transported soils and its application to mountainous areas tends to overlook mountain specifities e.g. residual soils, colluvium, rock types and structures, runoff and infiltration effects on the stability of slopes, uncertainties of behaviour of young rivers and streams, landslide daming, debris flow and mud flow, and land use and climate variability. The dynamin nature of mountain morphology and the spatial variability of the rock and soil properties render the reliability of engineering design and analysis under static conditions and limited space domain highly questionable. A combination of engineering-geological, geotechnical, enviromental, and economic investigations and analysis is essential for a sound engineering of infrastructures in the mountains.

Mountains in Nepals range from 4000 feet to 29000 feet in altitude, and highly erodible and geologically active Siwaliks in the south to snow covered massive rock peaks in the high Himalaya in the north. Infrastructures necessary for growing human activities must rely on either the sophisticated technologies and consequently high capital investments, or optimum utilization ot intermontane valleys, river terraces, and slopes with relatively higher stability. High initial investment is rarely possible for developing countries. Combination of avoidance and physical mitigations shoul lead to an effective investment strategies. This is possible only through the assessment of hazards, risks, and life cycle costs and benefits.

MOUNTAIN RISK ENGINEERING (MRE)

MRE is defined as science and art of engineering mountain infrastructures with due regard to natural and human processes, and the tolerable risks to and from infrastructures[1].

Risk assessment has benn treated differently by different persons or agencies. Geotechnical Control Office, Hong Kong has expressed hazards and risks in terms of instability score (potential for failure) andconsequential score (risk to life in the event of failure) respectively based upon formulae developed from discussions and calibrations from trial and error method[2]. Slope ranking was done using total score which is a sum of instability score and consequence score. Romana has suggested slope classification by SMR (slope mass rating) based on empirical relationships[3] which involve Bieniawski's rock mass rating and adjustment factors for dip direction and dip amount for slope and joint, and adjustment rating for method of blasting. Wagner A. has utilized subjective ratings for various natural factors. Hazard map is produced through overlaying of slope map, geological map, and morpho-structural map. These methods or maps provide indication or relative hazards but is silent on prediction of both physical or monetary loss and the time dimension of the occurrence of the damaging phenomena[4]. Einstein H. has suggested a systematic and formalized technique of risk assessment utilizing assesments of probabilities[5]. Expert system for hazard and risk assessment has been proposed by Thapa B. et.al [6]. All of these techniques, however, utilize, at varying degrees, experience based subjective judgements to arrive explicity and implicity at the probabilities of occurrence or rating. While it is possible to treat a given site within small areas with rigorous engineering-geological investigations, field and laboratory tests, and deterministic and relative hazard and risks analysis, these are justified for important and high cost structures at a specific site and normally at the detailed design stage (post feasibility stage) of a project. Linear infrastructures such as roads and irrigation canals, however, do not normally justify rigorous investigations and analysis for the reason of both costs and time at the prefeasibility stage and often at the feasibility stage too of project cycle. Investment decisions at prefeasibility and feasibility stage, nevertheless, require some indication of the likelihood of occurence of the damaging phenomenon and associated loss of life and property over specified time periods. A comprehensive, simple, and rapid assessments based upon desk study and walkover checks is necessary for the prefeasibility dan feasibility studies which would then, identify specific locations requiring rigorous investigations and analysis for the detailed design stage of the route identified from the feasibility studies.

HAZARD AND RISK ASSESSMENT

Risk, as defined by Varnes, is a function of hazard, element at risk, and vulnerability. H. Einstein defines Hazard as " the probability that a particular danger occurs within a given periode of time " and risk as " hazard times potential worth of loss."

Risk assessment discussed here utilizes the formal risk assessment procedure followed by Einstein except for the assessment of probabilities.

The method outlined in this paper is intended for route selection so that risk mitigation is primarily through avoidance of risks, and the residual risks are minimized by adoption of physical mitigation measures within the limits of resource availability and analysis period. For simplicity, only one time occurrence of hazard immediately after the construction of the structure is considered. Posterior probabilities requires updating such as Bayesian updating which requires many more statistics. Rainfall amd earthquake are assumedto be the main triggers of landslides. However, earthquake has not been considered in the method outlined here. Prior probabilities are obtained by ratings from 0 to 1 scale for various attributes existing and multiplying the total rating by rainfall factor. This is based upon the assumption that no matter whatever is the existing condition, landslide do not occur if there is no trigger. The total rating for the state of nature and the existing danger may be treated as the probability for a threshold value of rainfall.

State of nature is the description of the existing conditions in otherwise stable areas and danger is the existing landslide or mass movements. Table 1 presents an example of subjective ratings for hazard at threshold value of rainfall for the state of nature. Hazard for lower values of rainfall shall be obtained by multiplying this value by lower values (0 to 1 scale) for rainfall. Type of likely failure has to be judged from information on the state-of-nature and danger such as depth of soil, rock type and structure, ground water table, and type of existing failure. Hazard for dangers (existing failures) is treated as one for rainfall at the threshold value, and is equal to Ps+(1-Ps) x Rainfall factor. Risk is obtained by multiplying the hazard by the lenght of road likely to be affected times the damage factor (percent of road likely to be fully damaged, should the likely danger occur).

MRE APPLICATIONS IN NEPAL

Route selection based upon relative hazard assessment involving preparation of slope map, geological map, and morpho-structural map started since 1985 in Nepal. Hazard and risk assessment at the pre-feasibility and feasibility studies based on these informations have been employed for about ten road projects in Nepal. International Center for Integrated Mountain Development (ICIMOD) has prepared a three volume handbook on Mountain Risk Engineering covering an awareness book, subject background in the first volume, and application guide in the second volume. Use of MRE concepts and improvements on comprehensive hazard and risk assessment techniques are continually ongoing in the mountain road projects. Institutionalization of MRE approaches and practices for enviromentally sustainable infrastructures in the Hindu kush Himalaya countries is being facilitated by ICIMOD.

EXAMPLE OF MOUNTAIN ROAD FAILURES IN NEPAL

Amiko Highway :

Glacial lake outburst flood in 1981 in Tibet resulted in complete washout of 26 kilometre of the 114 kilometre long Amiko Highway linking Kathmandu with Chinese border near Khasa. Rainfaal and floods in 1987 washed out 10 kilometre and damaged 50 kilometre of this road requiring 8.2 million US dollar for reconstructions, landslide stabilization, and toe protections of the river banks; the cost almost being equal to half the cost of new construction of the road.

Lamosangu-Jiri Road :

Heavy moonsoon rainfall and cloudburst washed out a bridge, lowered the Chernawati River bed by more than 5 metre, and triggered numerous landslide in the 2 kilometer road section in the vicinity of the Chernawati River. Extensive investigations of landslides, design of mitigatory measures, stabilization of the river bed, protection of river banks, and stabilization of landslides affecting the road costed about 5 million US dollar, i.e. about 25 % of the total cost of this 110 kilometre road in the Mahabharat range to Lesser Himalayan range north-east of Kathmandu. Photos 1 and 2 illustrate the failures and the stabilization works.

Dharan-Dhankuta Road :

About 1 kilometre of a 50 km double lane road in eastern mountain region in Nepal was completely washed out following rainfall and earthquake in 1987. Photo 3 shows the debris flow damage in a section of this road.

Pokhara Road :

A 30 metre span plate girder bridge in the Kathmandu-Pokhara road (Prithvi Highway) collapsed from undercutting by Seti River during 1991 moonsoon. Rehabilitation of this bridge requires construction of a 150 metre single span new bridge. Besides, this event cast a panic among the people of Pokhara terrace due to the likelihood of sink holes, and subsidences from the caverns and tunnels formed by subsurface flow, and by the Seti riber flowing 15 to 20 metre below the surface through narrow gorges and appearing and disappearing at intervals. Figures 2 to 4 and photos 4 to 7 show the failure mode, area affected, and geology. Preliminary engineering-geological investigations have indicated that mapping of sink holes and caverns is required for the entire Pokhara terrace so that risk based zonation and infrastructural investment can be done without undue fear and uncertainities in the people of Pokhara.

It is estimated that, in Nepal, (i) 400 to 700 cu.m. of landslides occur per km per year in the mountain roads, (ii) 3000 to 9000 cu.m. of landslides occur per km every year during road constructions, and (iii) 10 to 25 per cent of mountain roads following river are completely washed out every 4 to 5 years.

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Table 1. Ratings for State-of-Nature(Ps) and Dangers, and Hazard and Risk Calculations.

Ratings for State-of-nature

Attributes	Description/ Measurement	Rating
1 Slope angle, degrees	0-5, 6-15, 16-25, 26-35, 36-45, .45	0, .05, .1, .14, .12, .1
2 Relative relief, metre	0-50, 51-100, 101-150, 150-200,>200	0, .03, .06, .09, .12
3 Drainage	simple, active, very active	0, .04, .08
4 Ground water condition	dry, wet, flowing(spring)	0, .04, .09
5 Landuse/Vegetation	thict vegetation, Moderate, sparse, barren or cultivated land	0, .03, .06, .09
6 Fault	<50 m, 51-100 m, >100m	.16, .08, .04
7 Soil type	alluvium, colluvium, loose alluvium, talus, till and debris and moraine	0-.04, .06-.08, .1-.12, .08-.1, .1-.12
8 Soil depth	<1m, 1-3m, 3-10m, >10m	0, .06, .12, .08
9 Fold, Syncline	<50m,>50m, >100m	.04, .02, .01
Anticline	<50m,>50m, >100m	.08, .04, .02
10 Rock type	massive resistant, soft, interbedding of soft and hard, weak	0, .02, .04, .06
11 Weathering grade	fresh, moderate, high, complete	0, .02, .04, .03,
12 Joint spacing	>1m, 51-100 cm, 10-50 cm, <10 cm	0, .03, .04, .06
13 Orientation of discontinuities	slope oblique to joint/bedding up to 30 degrees, dip slope of joint + 20 degree, dip slope of bedding, foliation + 20 degree	0-.04, .06, .08

Danger Classification

Old landslides	small, medium, large	slope length < 10 m, 10-50 m, > 50 m
Recent landslide	small, medium, large	slope length < 10 m, 10-50 m, > 50 m
Dormant landslide	small, medium, large	slope length < 10 m, 10-50 m, > 50 m
Landslide due to river bank under cutting	small, medium, large	Ht > 100 m from road, Ht 50-100m, Ht >100 m
Debris flow	small, medium, large	Depth < 0.5 m, .5-2 m, > 5m.

Rainfall Factor

Average annual rainfall, mm	Mean annual 24 hr max rainfall, mm	Rating
1000	<50, 50-100, 101-140, 141-170, >170	.3, .5, .8, 1, 1
2000	<80, 81-120, 121- 140, 141-170, >170	.4, .6, .8, 1, 1
3000	<130, 131-150, 151-190, 196	.5, .8, 1, 1
4000	<160, 161-190, 191-220, 221-260, >260	.6, .9, 1, 1, 1.

Road Damage Factor

Type of Likely Failure	Soil	Rock	Soil + Rock	Type of Likely Failure	Soil	Rock
Minor slide(1-3m deep)	.2	.3		Minor undercutting	.3	.1
Medium slide(3-6m deep)	.5	.6		Medium undercutting	.5	.2
Major slide(>6m deep)	.9	1		Major undercutting	.9	.5
Minor debris flow			.2			
Medium debris flow			.4			
Major debris slide			.9			

Hazard and Risk Calculation

Hazard for State-of-nature	= Rating for State of nature(Ps)*Rainfall factor
Hazard for Danger	= Ps + (1-Ps)*Rainfall Factor = >1
Risk, km (R1)	= Hazard*Length of road likely to be affected* Damage factor
Risk, monetary value	= R1* per km cost

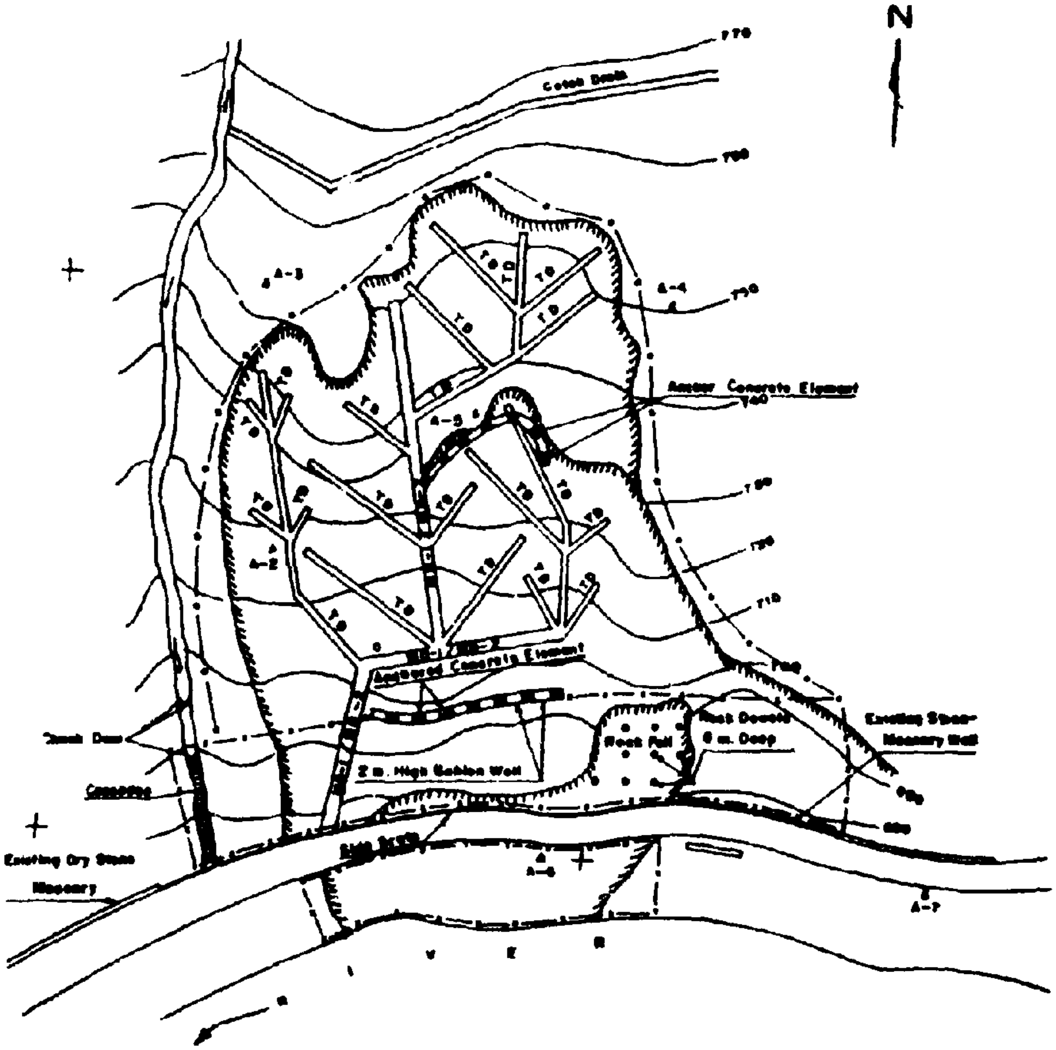

Fig. I Landslide Stabilization By Counterfort Drains ,Tributary Drains And Rock Dowels

Air Photo of Pokhara Valley

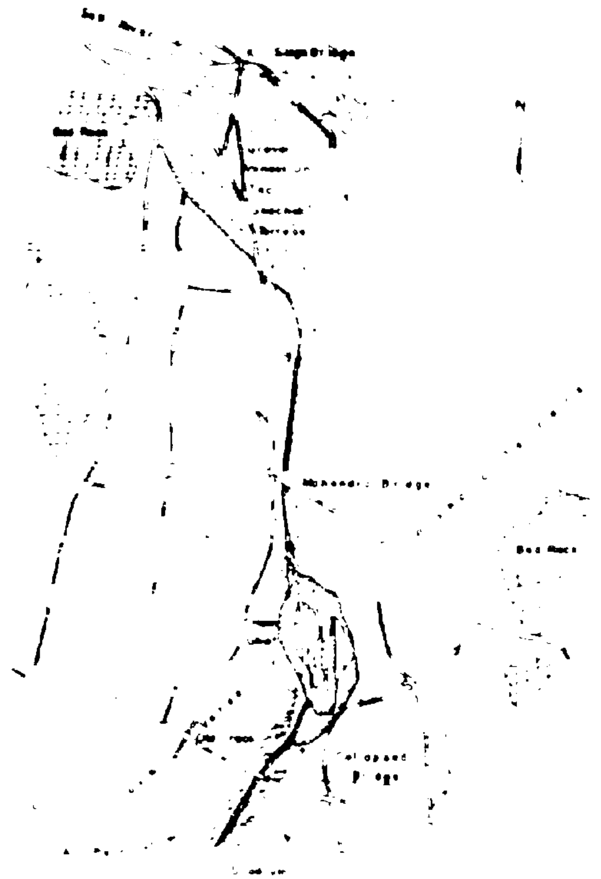

Fig. 2 ... Geomorphic Map Showing Bed Rock Terraces and Hazard Zone Around The Seti Gorge

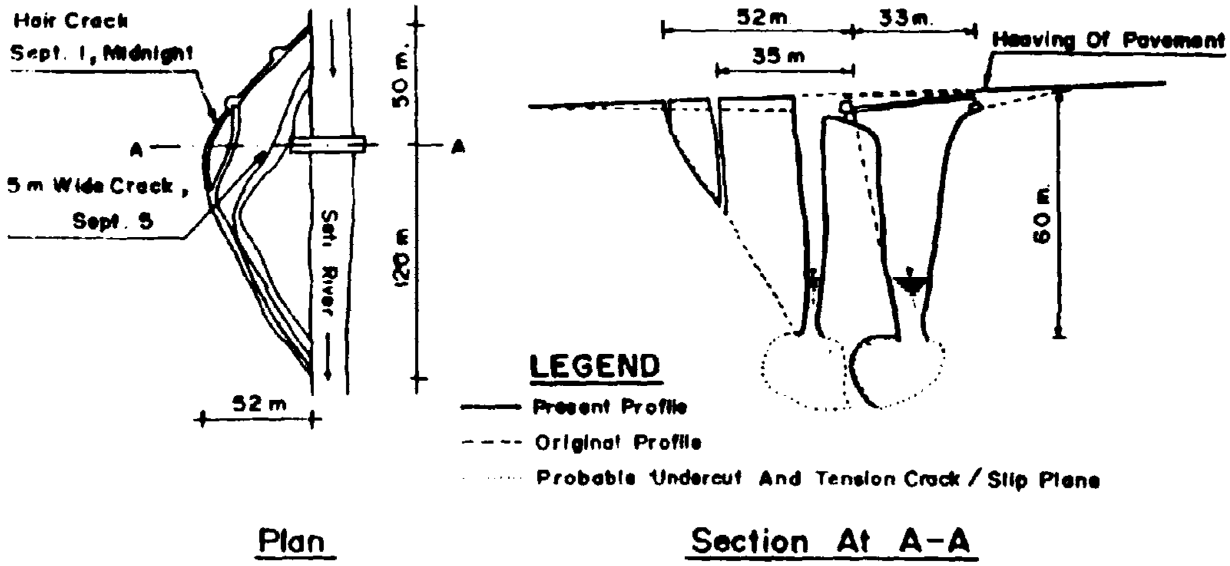

Fig. 3. Landslide Of Seti Bridge, Sept., 1991

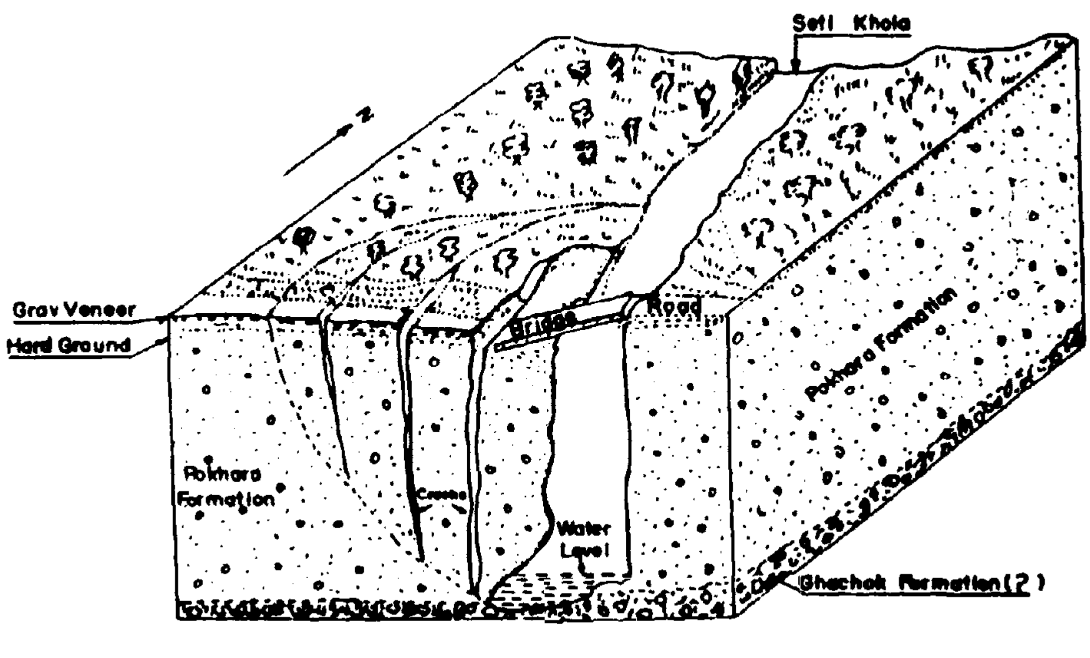

Fig. 4. Bank Failure, Seti Khola, Pokhara

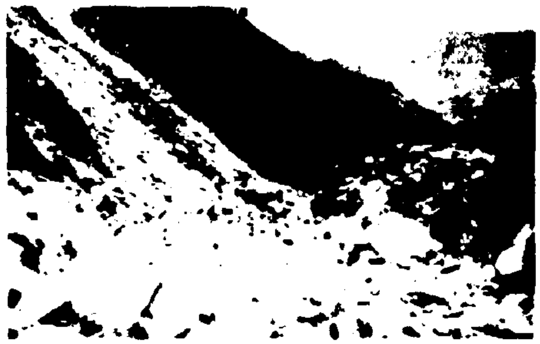

Photo 1 River Bed Stabilization by Tetrapods

Photo 2 Stabilized Slope

Photo 3 Car Cought by Debris Flow

Photo 4 [illegible] Bridge [illegible]

[illegible] River at 4 Km
[illegible] Bridge

Photo 6 New Crack Downstream
of Failed Bridge

[illegible] River at 2.2 Km
Downstream of Bridge

SECOND US-ASIA CONFERENCE ON
ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
Yogyakarta, Indonesia
22-26 June 1992

MONITORING OF BENDOWULUH LANDSLIDE IN BANJARNEGARA REGENCY, JAWA TENGAH

U. Sudarsono and S. Kartoatmodjo
Directorate of Enviromental Geology
Bandung, Indonesia

ABSTRACT Bendowuluh landslide is one of the landslide prone area in Banjarnegara Regency, Jawa Tengah. The landslide covers an area about 480,000 m^2, 800 m long and 600 m wide. The crown of the landslide is located at the western slope of Pawinihan Mt. and the toe is situated at Simpar river.

The landslide area is built up by rocks of the Merawu Series which are overlain by quarternary volcanic rocks. The Merawu Series consist of calcareous and marly claystones and the volcanic rocks are made up of lavas, breccias and lahar deposits.

Since 1989, the Bendowuluh landslide has been monitored using eight monitoring points which are measured from fixed points at Lumbung Mt. The result of measurement on 1991 shows that the landslide has moved the rocks horizontally as well as vertically. The horizontal movement range from 0.02 meter to 0.60 meter to the soutwestern direction and the vertical movement varies between - 0.17 meter and - 1.41 meters.

In order to study the movement of the landslide, in 1991, eight new points were added and fixed points at Bondan Mts. was established.

INTRODUCTION

The Bendowuluh landslide is one of the landslide-prone areas in Banjarnegara Regency, Central Java. The landslide is situated near the Bendowuluh Village, Banjarmangu District which is located some 10 km to the northwest of Banjarnegara (Fig 1). This landslide covers an area of about 480,000 m^2, 800 meters long and 600 meters wide.

The Bendowuluh landslide has caused damage on rice fields, farmlands, pine forest and cuts the road between Banjarmangu and Kalibening which makes it neccessary to repair this road every year.

In order to understand the factors that generates the recurring landslides in this area, especially the rate and the direction of movement, the change of the slope, the elevation and the depth of the slip surface, in this area eight monitoring marks were installed in 1989. These monitoring marks have been measured periodically. The result of the monitoring provides information about the process of the landslide, so that the input can be evaluated.

GEOLOGICAL SETTING

Bendowuluh area is situated in an area which can be distinguished into two geomorphic landform : a mountaineous and undulating landforms (Fig 2).

The mountaineous region is characterised by rough terrains with slopes exceeding 30 %. This region is located at the northeastern, southwestern and eastern part of the area; in the northeastern part there is 20 metres west facing high steep scarp.

The undulating region is characterised by slopes varying between 15 % to 30 % and it lies in between mountaineous region where the landslide area has taken place in this region.

The rock formations in Bendowuluh area belong to the Miocene Merawu Series (Tms) which consists of andesitic breccia and tuffaceous sandstone (Tpmv); and the Pliocene to Quarternary Ligung Series (Qtv); lava flows, flow breccia, pyroclastic breccia and lahar (Qjm and Qjo); and intrusive rock (Tpmi) (Condon and others, 1975) (Fig 3).

The Merawu Series (Tms) is mostly made up of calcareous and marly claystone. The residual soil varies between 1.00 to 2.00 meters thick and it forms highly plastic clay (CH) which has unit weight (g) 14.5 kN/m^3, cohesion (c) 4.4 kPa; and internal friction angle (0) 27^0.

The andesitic breccia and tuffaceous sandstone (Tpmv) cropout in the area between the Simpar River and the Kalidondong village.

The Ligung Series in this area consists of younger andesitic volcanic breccias (Qtv) which are exposed found in the mountaineous range to the west of the Simpar River.

The western slope of Mt Pawinihan is formed by volcanic rocks which consist of lava flows, flow breccias, pyroclastic breccias and lahar deposits. The residual soil is mainly made up of low plastic mud (ML) with unit weight (g) 17.8 kN/m^3, cohesion (c) 3.9 kPa and internal friction angle (0) 45^0.

The intrusive rocks which built up Mt Lumbung consist of dioritic rocks.

Structural geology in the Bendowuluh area is expressed by a low angle thrust fault. Evidence of this fault may be relocated when the regional features of the landslide has been analysed.

THE LANDSLIDE

The Bendowuluh landslide lies in the landslide prone area in Banjarnegara regency. This landslide area measures about 1,500 to 2,000 meters long and 20 to 50 meters in width with slopes varrying between 10% and 70% (Fig 4).

Van Bemmelen (1937), suggested that the slide in this area is caused by the overload of the Pawinihan volcanoes, resulting in a huge landslide of which the crown is located in the western part of Mt Pawinihan and the toe at the Simpar River generating a low angle "upthrust".

This "upthrust" is the result of the breaking off and sliding down of a portion of the Pawinihan volcano along a cycloidical plane (Fig 5). Therefore this up thrust is caused by gravitational movement.

The landslide has always been active, is proved by the fact that the road between Banjarnegara and Kalibening has to be repaired and the farmland is destroyed every year.

In order to recognize the behaviour of the landslide, in 1989 eight monitoring marks were installed. The monitoring marks were arranged in a grid system in the direction of the apparent movement. The purpose of this system is to periodically measure the horizontal and vertical movements and the change in elevation during certain intervals of time. These monitoring marks are referenced to geodetic bench marks which are situated on stable areas i.e. in Mt. Lumbung.

In order to measure the movement in the area to the south of Bendowuluh additional eight monitoring points and a reference bench mark in Mt. Bondan were established in 1991.

The monitoring marks were fixed to a depth of about 0,60 meter therefore primarily surface movements are observed, whilst deeper movements deeper based monitoring marks will be constructed.

The results of monitoring in January 1991 showed that the general direction of movement of the landslide is to the southwest whilst horizontal displacements range from 0.20 meter to 0.64 meter (Fig 6) and vertical displacement - 0.07 meter to 0.44 meter (Fig 6).

CONCLUSION

The results of the monitoring programme indicate that the Bendowuluh landslide is still active, moving in the southwest direction with a rate of horizontal movement for 1 year period ranging from 0.02 meter to 0.64 meter and vertical movements - 0.07 meter and - 0.44 meter.

The general movement is similar to the movement of Simpar upthrust, that is to the southest direction.

In order to minimize the landslide hazard in this area landuse should be planned appropriately because the land seems to move forever and the following steps are recommended:

1. Prevent infiltration of surface water in to the soil using proper drainage system.
2. Minimizing rainwash and avoid gullying by reforestation.
3. This area should not be used for housing and wet farming purposes.

ACKNOWLEDGMENTS

Acknowledgment is made to Mr. P.H. Silitonga, The Director of Directorate of Environmental Geology for his support and Mr. Suharsono for editing and clarifying the manuscript.

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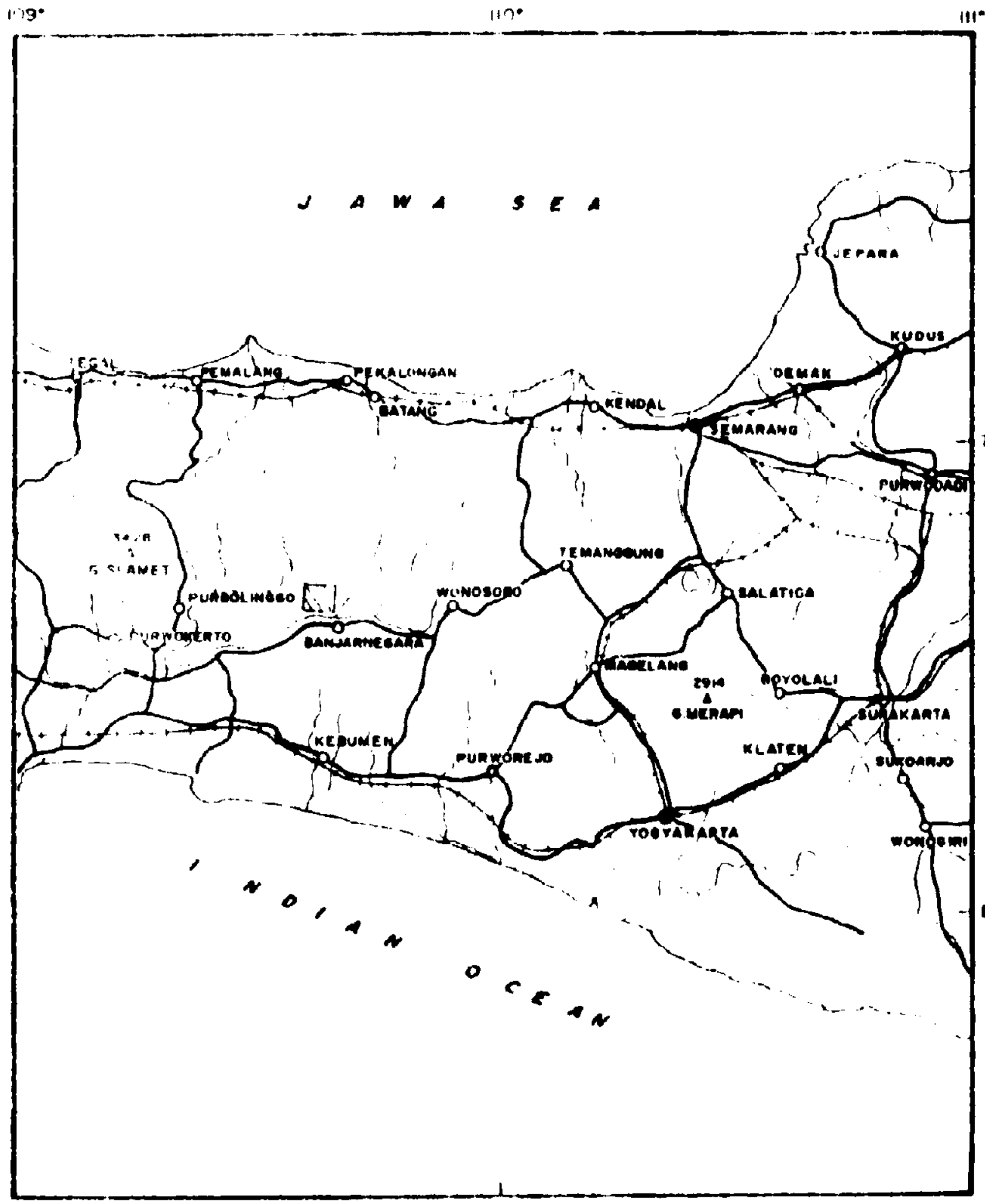

Investigation Area

Figure 1. Location map of the Bendowuluh area, Banjarnegara, Central Jawa.

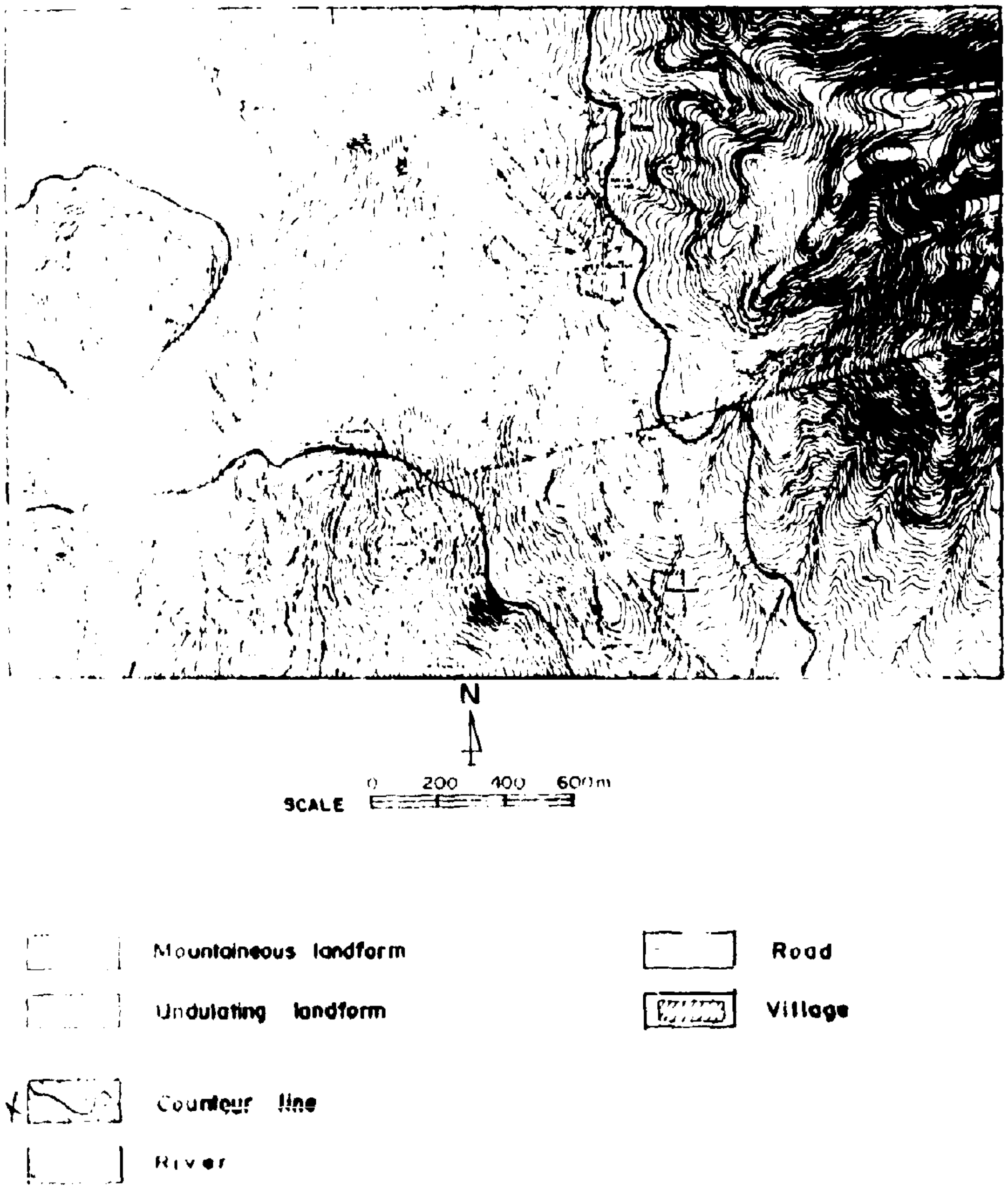

Figure 2. GEOMORPHIC MAP OF BENDOWULUH AREA, BANJARNEGARA CENTRAL JAWA

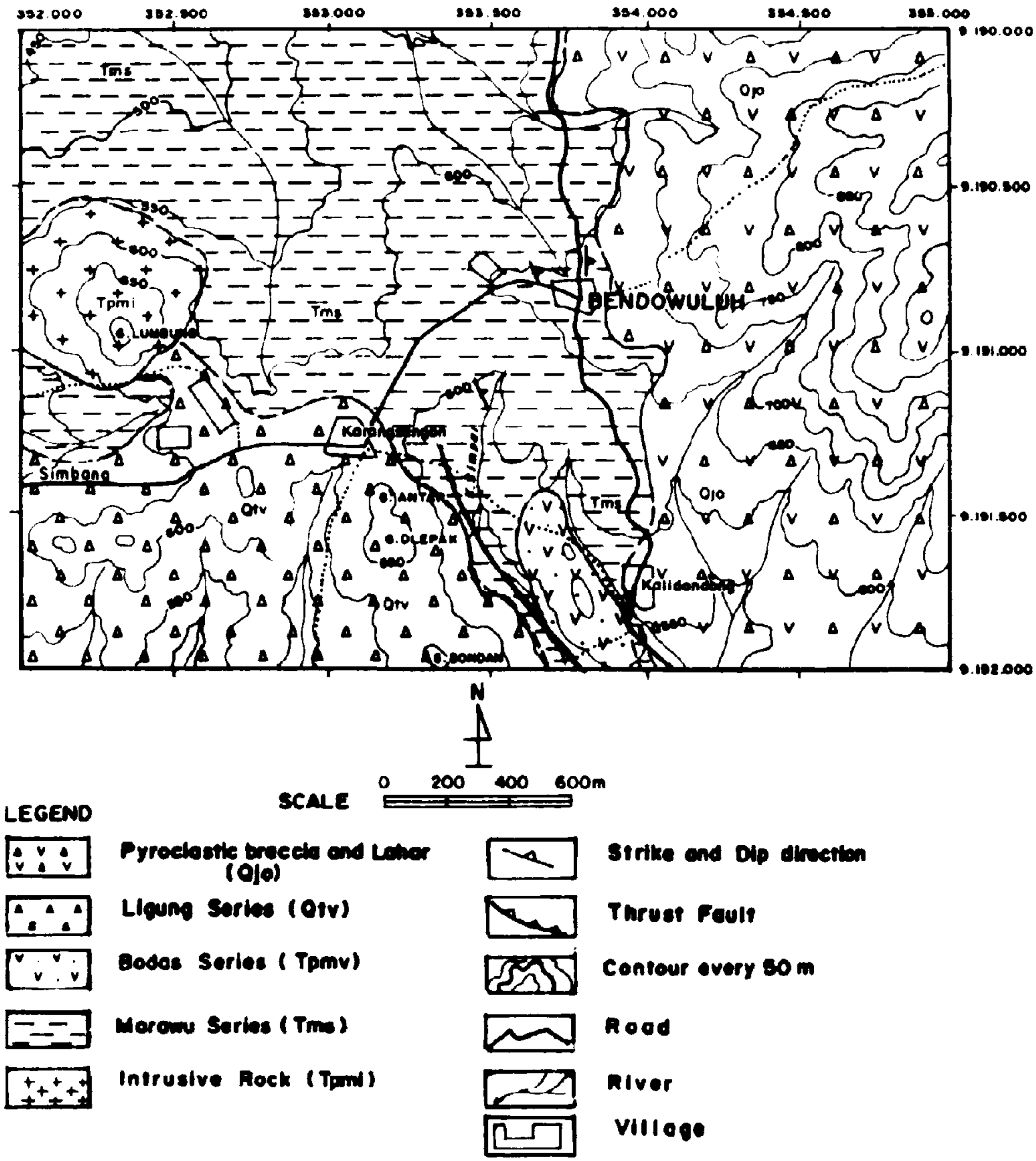

Figure 3 GEOLOGIC MAP OF BENDOWULUH AREA
BANJARNEGARA - JAWA TENGAH
(After Condon and others,1937)

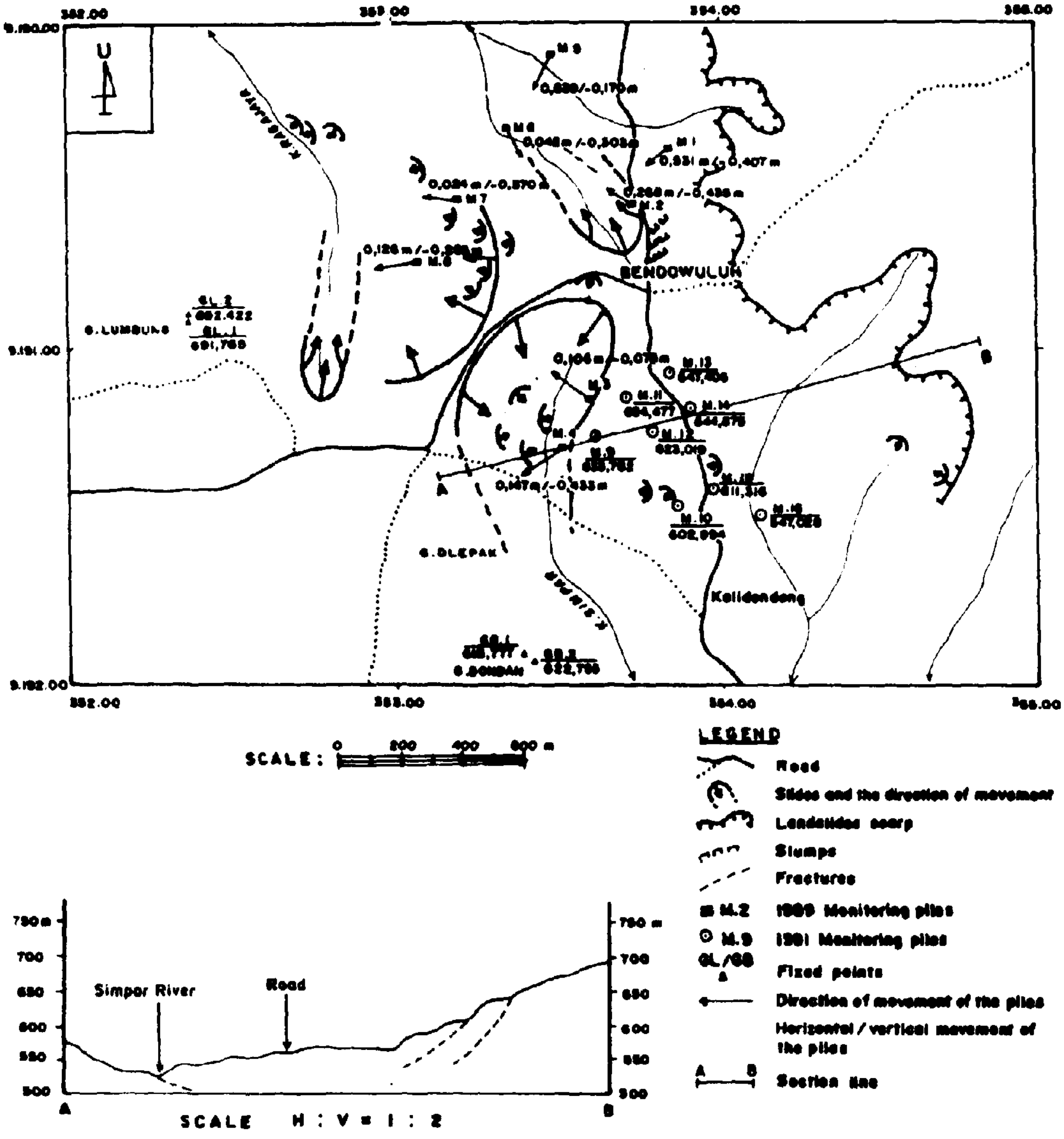

Fig. 4 **SITUATIONAL MAP OF BENDOWULUH LANDSLIDE PERIOD OCTOBER 1989 – FEBRUARY 1991 BANJARNEGARA – JAWA TENGAH**

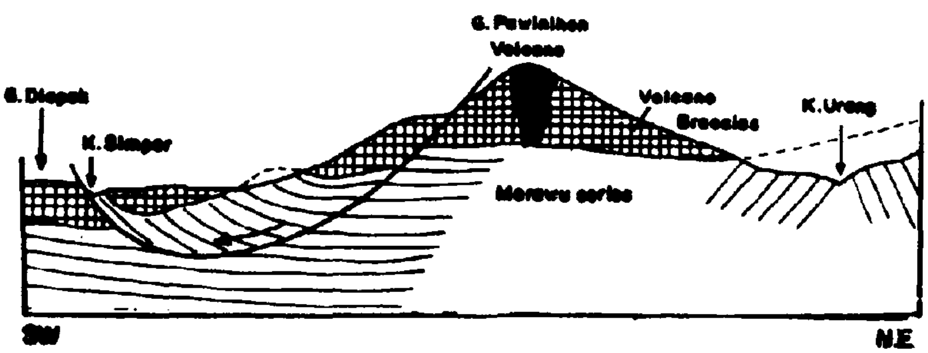

Figure 5 Van Bemmelen's representation of the large landslide in the [illegible] area.
(After Van Bemmelen, 1937)

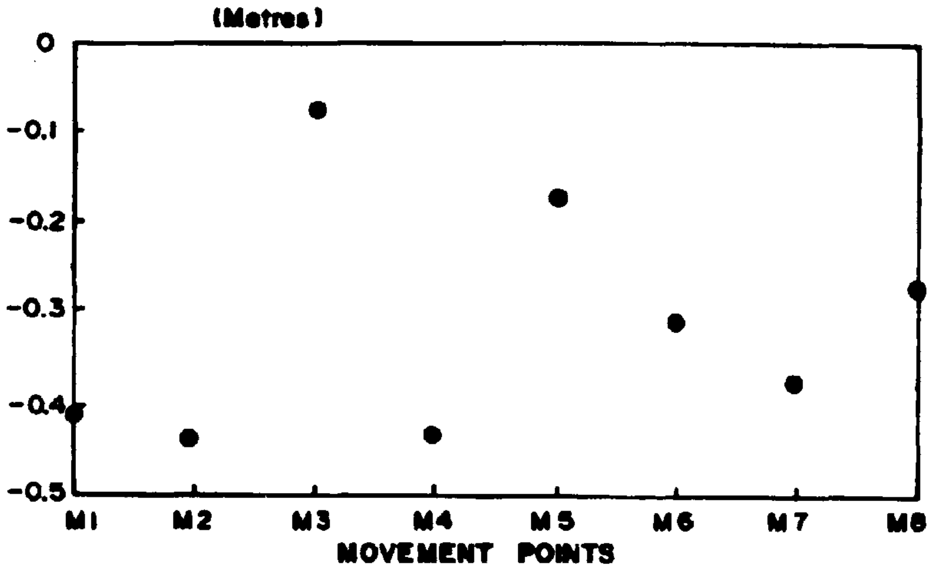

Fig. 6a VERTICAL MOVEMENT
OCTOBER 1989 – FEBRUARY 1991

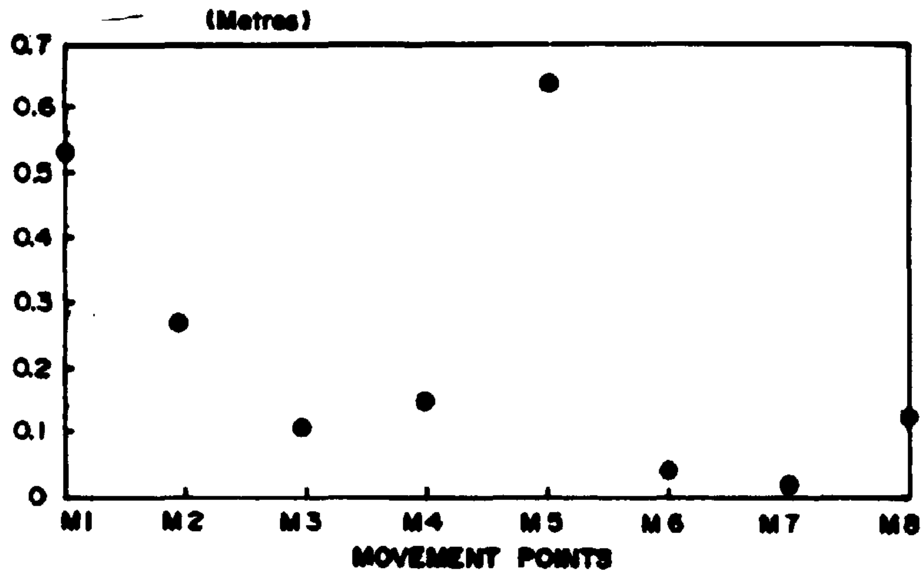

Fig. 6b HORIZONTAL MOVEMENT
OCTOBER 1989 – FEBRUARY 1991

SECOND US-ASIA CONFERENCE ON
ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
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THE SUSCEPTIBILITY TO LANDSLIDING IN THE ENREKANG AREA, SOUTH SULAWESI, INDONESIA

Sugiharto Nitihardjo
Directorate of Enviromental Geology
Bandung, Indonesia

ABSTRACT In order to mitigate the impact of landslide hazards it is important to determine its potential for sliding and to present the results into a map of the area.

A method for recognizing the potential of susceptibility have been introduced by the CTA-108 in Bandung. This scheme is based on factors and parameters of geology and especially lithology, slope inclinationn and landslide evidences, and other factors, such as rainfall, landuse and seismicity.

Based on the parameters and the safety factor of the soil, the Enrekang area can be devided into four landslide susceptibility zones: very low landslide susceptible zone, low landslide susceptible zone, moderate landslide susceptible zone, and high landslide susceptible zone.

The critical angles of the soil of various rocks are : 16° for shale, 34° for conglomerate, 35° for sandstone, 42° for metamorphic rock and 44° for breccia.

INTRODUCTION

Enrekang area is one of the landslide prone regions in Sulawesi. Commonly, landslide is influenced by man-made activity, such as road cuts and settlements on slope, also clearing up the vegetation resulting in bare land.

The study area is located around Enrekang in longitude 119°45' - 120°00' east, and latitude 3°30' - 3°45' south, covering about 800 square kilometres.

Enrekang area occupies a hilly landform with steep slopes. This area was selected for landslide study since it is located in the route between Ujung Pandang and the tourist centre of Tana Toraja.

One method for mitigating landslide hazard is to produce a landslide susceptibility map. The map can be used to predict the possibility for sliding, so that prevention mearures can be done.

MAJOR FACTORS CONTROLLING LANDSLIDES

Landforms susceptible to landslides area generally effected by the following factors: morphology, rock and soil, rainfall, landuse and earthquake.

1. Morphology

The study area comprises flat to mountaineous regions ranging in altitude between 100 m and 1900 m above sea level. This area can be divided to six units of slope steepness (see fig.1): Flat or almost flat 0%-5% (0°-3°); Gently sloping 5%-15% (3°-8°); Moderately steep 15%-30% (8°-17°); Steep 30%-50% (17°-27°); Very steep 50%-70% (27°-36°); Extremely steep 70% (36°).

2. Rocks and Soils

According of the geological map Majene and Palopo quadrangle (Djuri and Kastowo, 1974) the study area can be divided into 10 rocks units (see fig. 2):

Alluvial, consisting of : clay, silt, sand and gravels. These materials are typically loose, uncompacted soils.Conglomerate, medium to hard rock. The soil derived from the rock are silty sands of medium density, 0.5 to 1 m thick, unit weight ( ) = 1.67 ton/m^3, cohesion (c) = 0.80 ton/m^2 and angle of internal friction (0) = 26.55°.

Limestone, hard to very hard. The soil derived from the rock of clays to silts, soft, high plastisity, ±0.75 m thick, = 1.69 ton/m^3, c = 1.10 ton/m^2, 0 = 29.73°.

Lava, hard to very hard. The soil derived from this rock : clay, stif, medium plastic, 0.5 to 1 m thick, = 1.66 ton/m^3, c = 0.50 ton/m^2, 0 = 28.06.

Sandstone, hard. The soil derived from this rock : silty sands, sands and gravel, loose to uncompacted, 1 to 1.5 m thick, = 1.68 ton/m^3, c = 0.85 ton/m^2, 0 = 24.86°.

Marl, medium hard. The soil derived from this rock : clay, very soft to soft, highly plastic, 0.5 to 1 m thick, = 1.65 ton/m^3, c = 1.00 ton/m^2, = 26.55°.

Breccia, hard to very hard. The soil derived from this rock : silty clays, soft, medium plastic, ± 1 m thick, = 1.65 ton/m^3, c = 0.50 ton/m^2, 0 = 34.55°.

Shale, hardness: low to medium hard, with joints and cracks. The soil derived from this rock : clay, soft, medium to highly plastic, 0.50 - 1.50 m thick, = 1.58 ton/m^3, c = 0.30 ton/m^2, 0 = 26.33°.

Limestone, hard. The soil derived this rock : clays, soft to stiff, medium plastic, ± 0.50 to 1 m thick, = 1.57 ton/m^3, c = 0.70 ton/m^2, 0 = 35.32.

Metamorphic rock, hard to very hard. The soil derived from this rock : silty clays, stiff, medium plastic, ± 1 m thick, = 1.79 ton/m^3, c = 1.15 ton/m^2, = 21.25°.

3. Rainfall

According to Indonesian rainfall map vol II (LMG, 1973), the study area lies in the rainfall zone between 2000 and 2500 mm/year. The monthly rainfall during 10 years (1978-1988) at Enrekang Sta. amounted between 60 mm and 275 mm.

4. Landuse

Based on the landuse map (Badan Pertanahan Nasional, Enrekang), the study area can be divided in to eight landuses: villages (0.62%), paddy field (1.83%), vegetables fields (6.10%), forests (72.68%), mixed gardens (8.20%), scrub (11.00%), coffee estate (0.43%), and bare land (0.06%).

5. Earthquake

According to the earthquake map, the study area is included in the zone 4, which has a maximum acceleration with a return period of 20 years, between 0.13 and 0.20 g equivalent VII - VIII at the MMI scale. Epicentres are located on land, 0-65 km deep to the north of the study area (Beca Carter, Holling & Ferner, 1975).Evidently earthquakes have not had any influence such as triggering effects during landslide events.

LANDSLIDE SUSCEPTIBLE ZONES

To analyse the safety factor of the slope, soil properties are obtained from the laboratory: Unit weight ( ); Cohesion (c); Angle of internal friction (0).

The safety factor for translational and rational slide are analyzedusing the Fellenius and Bioshop methods respectively. The results of the analyses are compared with the field observation (table.1).

Ward (1976) compared the safety factor (Fs) and the degree of susceptibility of the slope (see table 2) :

$Fs < 1.2$ = High Susceptibility
$1.2 < Fs < 1.7$ = Moderate Susceptibility
$1.7 < Fs < 2$ = Low Susceptibility
$Fs > 2$ = Very low Susceptibility

Based on the critical slope observed in the field and the safety factor (Fs), with the range of the degree of susceptibility to landslide (Ward, 1976), four landslide susceptible zones may be distinguished (see fig. 3):

VERY LOW LANDSLIDE SUSCEPTIBLE ZONE

The degree of susceptibility to landslide is very low. The zone rarely or have never been subjected to landslides. Old and new landslides are uncommon, however, along river cliffs small landslides could be found.

This area is mostly flat or gently undulating with natural slopes less than 15% (5), and the slope was evidently not formed by landslide deposits, filling material or plastic and swelling clay.This zone is mainly covered by alluvial deposits (Qa) or conglomerate unit (Tmc).

LOW LANDSLIDE SUSCEPTIBLE ZONE

The zone has a low susceptibility to landslides. Landslides rarely occur unless the slope is disturbed, and old landslides have been stabilized during the past period. Small landslides may occur, especially on the river side or gully.

The natural slopes are gentle (5-15%) to steep (50-70%), depending on the physical and engineering properties of the rock and soil forming the slope. Generally, on steep to very steep slopes a thin soil of weathered rock may be formed covered by vegetation such as forestry or plantation.

The slope area is mostly composed of the weathering product of conglomerates (Tmc), lava (Tmpv), sandstones (Tmpss), breccia (Tol), silts (Tet), or marls (Tmb). The zone extends in Malino, Bali, Malaga and Enrekang.

MODERATE LANDSLIDE SUSCEPTIBLE ZONE

The Zone has a moderate susceptibility to landslides. Landslides may occur in this zone especially along river sides, road cuts and slopes which have been disturbed.

Old landslides may be activated, especially when induced by high rain fall and strong erosion processes.

The slope inclination ranges from slight (5-15%) to very steep (50-70%) and they depend on the engineering properties of the soil/rock. This zone is usually quite absent of vegetation.

The slopes are built up by the weathering product of conglomerate (Tmc) sandstone (Tmpss), marls (Tmb), silts (Tet), and Latimojong formation (Tkl). The zone is spread over Kp. Karang, Riso, G. Batopali, Nating and other steep area.

HIGH LANDSLIDE SUSCEPTIBLE ZONE

The zone demonstrates a high degree of susceptibility to landslides. In this zone landslides have occured very frequently. Old and new landslides are induced by high rainfall or strong erosion processes.

The natural slopes are moderate (15-30%) to very steep (more than 70%), depending on the physical and engineering properties of the rocks and soil forming the slope. The slopes usually are sparsely vegetated.

The slopes are composed of weathered silts (Tet), sandstones (Tmpss), marls (Tmb) and conglomerats (Tmc). The zone occurs in Kp. Kulinjang, Kp. Karang, Kp. Bullo, Kp. Talang Ridau, Kp. Paladang and Kp. Bulirang.

CONCLUSION

1. The mapped area is divided into four susceptibility zones: very low susceptibility to landslides, covering about 26.3%, zone of low susceptibility to landslides, approximately covering 40.2%, zone of moderate susceptibility to landslides, approximately covering 31.6%, and zone of high susceptibility to landslides, approximately covering 2%.

2. The critical slope angle of conglomerate (Tmt) = 41°, limestone (Tmpl) = 70°, lava (Tmpv) = 41°, marl (Tmb) = 48°, breccia (Tol) = 32°, silt (Tet) = 18 °and 35°,sandstone (Tmpss) = 40, sandstone (Tetl) = 75° and metamorphic rock (Tkl) = 51°.

3. In order to prevent sliding, it is suggested that man- made slope angles should not exceed the critical slope angle.

4. The present road through Kulinjang runs across a landslide area which is likely a landslide on prone area. It is proposed to undertake a detailed investigation on the engineering geological properties of the materials involved including the possibility of changing the road.

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Table 1 : Conparation Between Computed and Actual Critical Slope Angle

No.	RESIDUAL SOIL	Critical Slope From Calculated: Safety Factor (F = 1)	Critical Slope From Calculated: Safety Factor (F = 1.2)	Critical Slope From Data
1.	Aluvium (Qa)	-	-	-
2.	Satuan Konglomerat (Tmc)	40°	34°	32°
3.	Satuan Batugamping (Tmpl)	70°	58°	-
4.	Satuan Lava (Tmpv)	41°	38°	-
5.	Satuan Batupasir (Tmpss)	40°	35°	39°
6.	Satuan Napal (Tmb)	48°	40°	41°-43°
7.	Satuan Breksi (Tol)	52°	44°	55°
8.	Satuan Serpih (Tet)	18°	16.5°	15°*/30°**
9.	Satuan Batugamping (Tetl)	75°	60°	-
10.	Satuan Latimojong (Tkl)	51°	42°	56°

* Thickness of soil 1.50 meters

** Thickness of soil 0.50 meters

Table 2. Degree of Susceptibility to Landslides of Each Weathering Rock Within The Slope Classes

No.	RESIDUAL SOIL	0-15%	5-15%	15-30%	30-50%	50-70%	>70%
		0-30°	3-8.5°	8.5-17°	17-27°	27-36°	>36°
1.	Aluvium (Qa)	I	-	-	-	-	-
2.	Conglomerate (Tmc)	I	I	I/II	II	II/III	III/IV
3.	Lava (Tmpv)	-	-	I/II	II	III	III/IV
4.	Sandstone (Tmpss)	-	I	II	II/III	III	III/IV
5.	Marl (Tmb)	I	I	II	II	II/III/IV	III/IV
6.	Breccia (Tol)	I	I	II	II	II/II	III/IV
7.	Shale (Tet)	I	II	II/III*	II/III/IV*	III/IV*	III/IV
8.	Limestone (Tetl)	-	I	II	II	II	III/IV
9.	Methorphic rock (Tul)	-	-	II	II	II/III	III/IV

Explanation : I. Very low susceptibility

II. Low susceptibility to landslide

III. Moderate susceptibility to landslide

IV. High susceptibility to landslide

G13-08

Fig. 1 Slope Map of Enrekang area, South Sulawesi

N

LEGEND

[illegible] sloping, 5 - 15% (3 - 8,5°)

Moderately steep, 15 - [illegible] (8,5 - 17°)

Steep, 30 - 50% (17 - 27°)

Very steep, 50 - 70% (27 - 36°)

Extremely steep, > 70% (> 36°)

G13-09

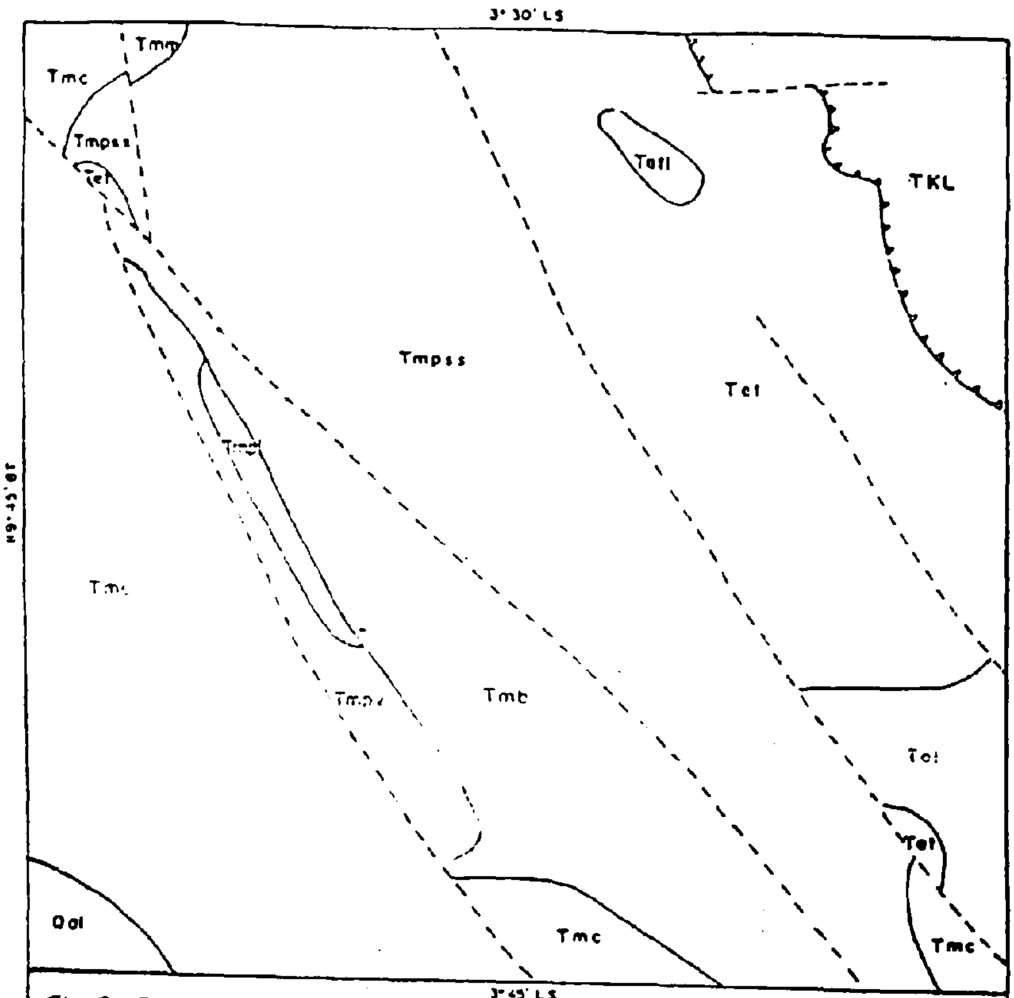

Fig. 2 Geological Map of Enrekang area, South Sulawesi

SCALE

0 1 2 3 Km

Legenda

Alluvium (Qal)
Conglomerata (Tmc)
Lava (Tmpv)
Sandstone (Tmpss)
Marl (Tmb)
Breccia (Tol)
Shale (Tel)
Limestone (Tefl)
Metamorphic rock (TKL)

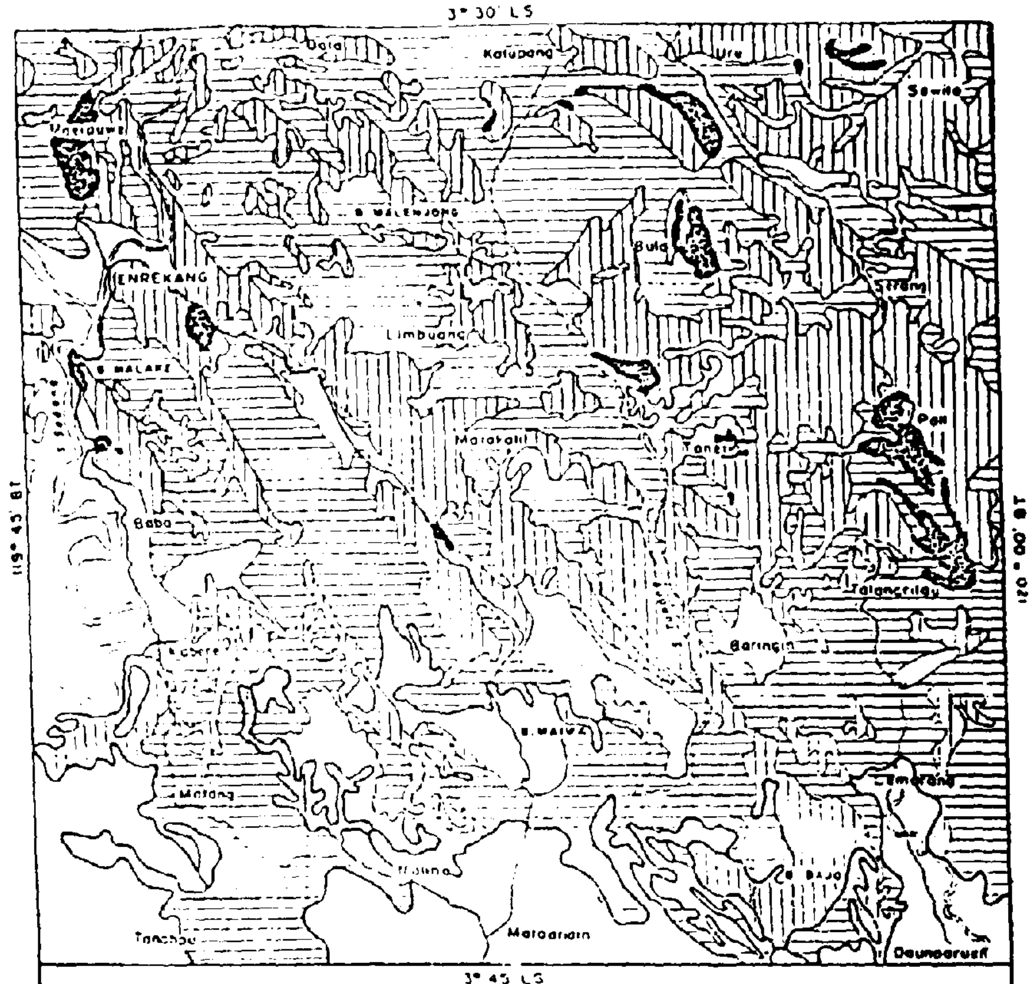

Fig. 3. Map of susceptibility to landslide, Enrekang area, South Sulawesi

Scale

0 1 2 3 km

Zone of very low susceptibility to landslide

Zone of low susceptibility to landslide

Zone of moderate susceptibility to landslide

Zone of high susceptibility to landslide

SECOND US-ASIA CONFERENCE ON
ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
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LANDSLIDE POTENTIAL OF THE HALANG FORMATION IN THE WALAD AREA, CIREBON REGENCY, WEST JAVA, INDONESIA

I.B. Sudjarwo, Suranta, and Y.O.P Siagian
Directorate of Enviromental Geology
Bandung, Indonesia

ABSTRACT The constituent rocks of the Halang Formation (Tmh) generally consist of turbiditic deposits with clear sedimentary structures. The upper part consists of claystone and marl, the middle part is built up by coarse limy sandstones, and the lower part consists of conglomeratic limestones.

Landslides inthe Halang Formation include debris slide and creep of 25 m to 45 meters width in an investigated area about 50 square kilometers. These landslides occur especially along the boundary with volcanic rocks, frequently forming and elongated landslide scarp reaching a scarp height of 25 to 75 m.

Landslides frequently occurr in clayey soils which originate from the weathering of claystones . These soils have characteristic properties such as swelling, sticky and crumbly in wet condition.

The test on dry samples in the Soil Mechanics Laboratory yielded the average effective cohesion (c'= 0.50) ton/m^2, density (wet=1.58) ton/m^3, effective of angle of internal friction (0= 2138'43"). The safety factor (SF =1.2), gave the angle value of slope in this area at 20^0 in the medium saturated condition (Rh=0.5).

The Halang Formation is widely distributed in the South western part of Waled Sub District, so that this area is considered to be highly potential in landslides.

INTRODUCTION

Failure of earth materials under stress may disrupt the equilibrium and causes movement of the earth material. The disruption of the equilibrium may be reviewed according to the factors that change the shear stress conditions, which may be due to the natural phenomena as well human activities (Varnes, 1976).

The Waled area, southwards from Cirebon is an interesting area for study as slope of various susceptibility to landslide can be recognized in soil formed by the weathering of the Halang Formation, when landslide have occured in the part.

The investigation of Halang Formation is aimed of recognition of the composition and engineering properties of substructure. This will be used as a basis of building up engineering geological information needed in the environ- mental development planning of this area.

Methods and techniques for assesing the potential landslide areas in the Halang Formation are based on quality and quantity approach. The quality approach involves observations of climatical condition (rainfall) and earthquakes. In the quantity approach we identify the geometric dimension, the physical indices and the engineering properties such as water content (Wn), unit weight (g), porosity (n), degree of saturation (Sw), effective cohesion (c'), angle of internal friction (0'), either of lithologies or of the known sliding planes.

GEOLOGICAL SETTING

Landforms in the Halang Formations are evidently in part the effect of the geological structure. According to the terrain analysis (Young, 1976), landforms in this area may be classified as moderate and high relief. The landform of moderate relief is charaterized by moderately steep to steep slopes (8^0- 17^0 or 15% - 30%) with heights varying in between 145 and 200 meters. The landform of high relief have steep slopes with angles varying in between 17^0 and 27^0 (30% - 50%) having elevation ranging from 75 to 195 above sea level.

The Halang Formation which is distributed in the southern part of the Waled area is mostly represented by its weathering products. It is spread for about 200 km^2 extending from the northwest to the southeast direction, 50 km^2 of which occupies hilly landforms where in landslides have occured.

The Halang Formation is charaterized by turbiditic sediments, indicated by evident sedimentary structures, such as parallel lamination, graded bedding, flute and load casts. Lithologically it is built up of three parts. The upper part consist of claystones and marls, the middle part is dominated by sandstones, whereas the lower part is formed mainly by conglomeratic sandstones. In the hilly area near Waled, the Halang Formation is represented by its weathering products, which consist of yellowish grey, soft to stiff (in dry condition), clay soils, which have low permeability, high plasticity and a highly expansive ratio (Merwe, 1975). The thickness of the soils vary between 1.50 to 2.00 meters.

The Geological map of the Cirebon Quadrangale (1:100.000) by Silitonga and Masria (1978), the studied area is charaterized by folding and faulting. Folding is represented by synclines and anticlines, whereas faulting consists of lateral, thrust and normal faults. The thrust fault that intersects clays and marls, this structure is known as the thrust system of Seuseupan which is located at the northeastern part of the studied area . In this hilly area modeıate slopes are typically built up of clays. Failure in these clays are quite common as indicated by frequent landslides.

LANDSLIDE CHARATERISTIC

Landslides are caused by instability of the slope when the forces of movement exceed the restraining forces. Result of the field investigation and laboratory analysis indicate that landslides in the Halang Formation are influenced by 4 (four) factor, as follow :

a. Geometric dimension such as angle and height of slope.

b. Geological setting including attitude of sublayer, geological structure and seismicity.

c. Physical properties of soil materials, which are made up of CH-soil; i.e. inorganic clays of high plasticity (according to USGS classification), showing very different on strengths during wet and dry seasons.

d. Climatic conditions, especially rainfall when water soaks the soil material, it would be decreased soil strength and increased the groundwater pressure (the lower part of sublayer is more permeable than the upper one).

Eighteen landslide scars have been observed, evidently according on slope greater than 19^{0} (34%). It has been shown that the slope are covered by clayey soils which have been derived from the weathered Halang Formation, it has also been noted that landslides have taken place during the raining season. Fortunately the landslidea are located far from any village. Therefore destruction was restricted to rice fields and eucalyptus plantations.

SLOPE TYPE AND DIMENSION

Considering the depth of slip, scarp shape, materials of layered slope and the slippage mechanism on weak plane (Varnes, 1976), the landslides in the Halang Formation are regarded as translational slides, with thickness about 2.00 - 2.50 m.

As a comparison landslides occurring in the claystone of Cijulang Formation, exposed to the north of Halang Formation, demonstrate 6 (six) slope failure within limited area of about 23 km^2, near the Seuseupan village, the slides vary in width between 10 and 25 meters and 15 and 35 meters in length with slopes ranging from 20^0 to 23^0. These slides are evidently controlled by the angle of slope, joints/joint sets and various physical properties. Since the materials involved in the slide composed of rock fragments embedded in clays, the landslides of Cijulang Formation are better classified as debris slides, 1.50 - 2.00 meters in thickness.

Based on size dimension, landslides in the Halang Formation may be classified into small and medium landslides. Measurement on the small landslides varying in 10 to 40 meters and 5 to 30 meters in lengths and widths respectively. The medium landslide was observed at Cikeusik (near to Halang Formation boundary), showing a scarp of about 40 meters in height and 700 meters in length.

STABILITY ANALYSIS

Analysis of slopes evaluates stability failure of the surface as compared with the movement of the pushing forces due to the weight of materials and pore pressure with the resisting forces provided by the shear strength of the slope.

The ratio of the maximum resisting forces which can be developed along a potential slip surface to the amount actually required for slope stability gives the Factor of Safety (FS) againts slope failure along the surface. Several potential slip surfaces should be calculated until the slip surface fields the minimum Factor of Safety obtained.

Where most of the slip surfaces are classified into translational sliding, the analysis is derived from the (modified) Fellenius method (1936), which is appropriate for the analysis of the slope stability of the Halang Formation.

$$Fs = \frac{c' + (Wt \cos - Ww.h) \tan 0}{Wt \sin}$$

Where :

Fs= Safety factor

c'= Effective cohession

Wt = Total unit weight

= Slope angle

Ww= Weight of water

h = Height of water 0'= Effective of internal friction angle

According to a limited data, particularly the ground water condition (i.e. hydrostatic pressure/ pore water pressure) is still unidentified, we should assume a ratio of water level due to the linear plane (Rh), namely Rh= 0.0 for dry, Rh= 0.5 for medium saturated and Rh= 0.9 for saturated conditions.

The factor of safety is calculated from analysed models on the basis of slope angle ranging from 0^{o} to 80^{o}, involving either or not of the earthquake acceleration (seismicity), as shown at the table as listed below :

Table 1. Critical slope angle of clay soil of Halang Formation, South of Cirebon, West Java.

Critical Slope Angle					
Fs = 1.2			Fs = 1.2 (influenced by earthquake)		
Rh = 0.0	Rh = 0.5	Rh = 0.9	Rh = 0.0	Rh = 0.5	Rh = 0.9
22^{o}	20^{o}	17.5^{o}	11^{o}	9^{o}	6^{o}
Note : The acceleration of the seismicity factor give 0.20 g, i.e. whithin the range between 0.13 to 0.23 g.					

SUMMARY AND CONCLUSION

a. The slope of the hilly area are located in South Cirebon are built up weathering product of upper part of Halang Formation. The materials are composed of inorganic clays of high plasticity, soft to stiff in dry condition, homogenous layer which having low permeability.

b. The morphology of the hilly area is characterized by moderate to high relief, having slope angles of 8^{o} to 17^{o} and 17^{o} to 27^{o}, and elevations varying between 145 to 200 meters and 75 to 195 metres respectively.

c. Landslides occur on slopes upon moist clayey sliding planes which occur approximately 2.00 to 2.50 m below the surface. These slides are regarded as translational slides of small to medium size.

d. A planned reforestation is necessary, trees should be channeled in rows parallel to the toe of the slope so that groundwater may flow freely to the surface through the parous media of the roots.

e. Springs and seepages along the toe of the slope should be channeled into a surface drainage system thereby lowering the groundwater table as much as possible, reducing the pore water pressure.

f. The analysis on slip on the clayey sliding plane wich is related to slope failure with strength parameters of c 0.5 tons/m^3 and '= 21.5^0 gave the factor of safety SF = 1.2. The angle of slope is regarded to be stable at about 20^0 in a medium saturated condition (R=0.5).

g. The slopes are unsuited for rice fields which would need a great quantity of rain water. It is recommended to afforest with broadleaf trees. Which consume a limited quantity of water but lower the water content of the soil.

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VOLCANIC HAZARD

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VOLCANO MONITORING AND ERUPTION PREDICTION : STRATEGY, TECHNIQUES, AND LIMITATIONS

Barry Voight
Departement of Geoscience
Pennsylvania State University
University Park, PA 16802 USA

ABSTRACT The capability to monitor volcanoes and to warn of impending eruptions has improved over the years. This paper presents a critical appraisal of the state-of-the-art. The most useful monitoring tool is seismic monitoring, the determination of earthquakes per unit time, their energy release, and source characteristics. Deformation monitoring has proven useful at some volcanoes, and gas monitoring has been conducted with variable success. Field observations must not be omitted. From these data, predictions are traditionally developed by pattern recognition, augmented by interpretation of evolving processes. Where data values accelerate prior to eruptions, the "materials science method" may be helpful. However, volcanoes are such extremely complex systems that with all techniques, warning of impending eruptions is difficult and not necessarily reliable, even under optimal circumstances.

INTRODUCTION

This paper is dedicated to the memory of Professor R. Mugiono of the Gadjah Mada University in Yogyakarta, a remarkable scientist and leader whose unfortunate death in 1991 leaves a tremendous void in Asian volcanological research. May his life inspire others to high achievement!

About 50 of the earth's 600 active volcanoes erupt each year, threatening the lives and property of millions of people. In a typical year, unrest (anomalous activity) occurs at about 18 large calderas worldwide, and eruptions occur within or near five of them (Newhall and Dzurisin, 1990). Disasters since 1700 A.D. have killed more than 260,000 people, a number that would be very much greater if today's population applied over the period examined. In the 1980's volcanoes killed more than 28,500 people and seriously disrupted local economies and social life in a number of instances (Tilling, 1989; Voight, 1990).

The mitigation or reduction of volcanic hazards involve the following issues and context:

1. Political and financial mandates for mitigation.
   - Regrettably, disasters are sometimes necessary to provide the impetus for mandates.
2. Development of mitigation or alleviation programs that can be sustained.
   - Considerations include the relative merits of self-sufficiency vs. foreign and/or regional support assistance.
3. Long-range hazards assessments for greatest-threat volcanoes. For effective mitigation -- **be prepared!**
   - Hazard maps, risk maps; hazard travel times; quantification of magnitudes and frequency of prior events; probability and uncertainty issues; use of Geographic Information Systems (GIS).
   - Literature search and study for analogues and lessons about anticipated unrest.
4. Monitoring.
   - To detect **unrest** -- a significant change from "background" values (or noise).

- For short-term prediction/warning.

5. Action by scientific team.
   - Internal decision-making issues on conclusions for presentation to civil officials.
   - Team dynamics: consensus, diversity of opinions.
   - Interactions with civil officials, and media.
   - Maintaining "credibility" -- coping with intermittent unrest, false alarms.
6. Action by civil officials
   - Public education, zoning, development and testing warning systems and emergency response plans; influences of culture; media issues.
   - Socioeconomic consequences of action vs. non-action.
   - Chain-of-command in decision making; **the will to act**; the implications of evacuation decisions; coping with intermittent unrest and false alarms.
7. Post-crisis investigations.
   - Lessons learned.

The remainder of this paper concentrates on monitoring, and its use for short-term prediction of hazardous events.

MONITORING AND SURVEILLANCE STRATEGIES

The following issues are involved:

1. "Baseline" measurements of background activity are desirable in advance of crisis, in order to enable the detection of a change from the background state.
2. Monitoring is normally carried out at volcanoes already recognized as high-risk. One problem is that some "highly-dangerous" volcanoes may not be recognized.
3. The tools:
   - Visual and sensory observations by trained observers (typically daily to monthly), supplemented by local resident information. Includes reports of explosive noise, felt earthquakes. **Field observations should not be underestimated.**
   - Seismic. Continuous operation of at least one

seismometer within 5 km of vent, preferably telemetered, or if not, smoked drum with observer in radio contact. Three seismometers the miminum for earthquake location.

- Deformation. Electronic Distance Measurement (EDM) of lines and networks, using infrared, laser, or 2-color laser systems; tilt, spirit level vs. electronic; extensometers; leveling; Global Positioning Systems (GPS).
- Gases and thermal activity. Fumarole sampling; ground or aircraft-based COSPEC; temperature; water or condensate chemistry; ash leachate analysis.
- Other. Magnetic surveys, gravity surveys; thermal surveys; barometers for explosion detection; wind profiles; hydrologic changes.

4. Data interpretation.
   - Attempting to understand the processes.
   - Comparisons with historical data base involving past eruptions, including those at other volcanoes.
   - Search for analogues.
5. Prediction/detection methodology and limitations.
   - Understanding the limitations -- lessons in humility.
   - Predictive capability does not guarantee successful mitigation.
   - Uncertainty does not preclude wise decision making.

MONITORING TECHNIQUES

Development of tools and interpretational/predictive methodologies have improved substantially over the years. Achievements have been substantial, but important failures have also occurred -- with the most notable example Ruiz in Colombia (Voight, 1990). **At Ruiz, preliminary hazard maps completed one month before the November 1985 eruption clearly pointed to Armero as being especially vulnerable to mudflows. However, emergency response measures taken before or during the eruption were inadequate to save more than 23,000 people killed when the mudflows struck the village. The eruption was not -- and could**

not have been -- predicted; but detection was feasible, and, because of the distance from the village to the crater, the lead time would have been sufficient to enable lives to have been saved -- with appropriate hazard management practices.

There is no justification for volcanologists to be complacent about the current state-of-the-art; a satisfactory level of predictive reliability has not been achieved with existing monitoring technology.

Relative merits/demerits of tools are summarized as follows:

1. Seismic Monitoring. The most successful monitoring tool, providing telemetered real-time information on the state of the volcano. An increase in earthquake activity generally (but not always) precedes the eruption (Figure 1).

The simplest but most fundamental data set involves **number** of daily (or weekly) earthquakes (but this depends on location, detection threshold). Other issues in seismicity include subdivision of observed earthquakes into particular categories, some more useful for prediction than others; calculation of seismic energy release (manual vs. computer-based methods with near real-time capability); definition of earthquake locations and tracking of systematic location changes (Figure 1); volcanic-tremor characteristics; Real-time Seismic Amplitude Measurement, RSAM (Endo and Murray, 1991), see Figure 2; reduced displacement and source characteristics. Computer-based data manipulation procedures are increasingly used to provide near-real time results. High-frequency seismic technology may also be used for lahar (mudflow) or water flood detection (R.J. Janda, personal communication; Brantley, ed., 1990).

Despite these advances, increase in earthquake activity can culminate in swarms with no eruption. Such seismic crises may indeed indicate actual pulses of magma movement underground (intrusions), but are "false alarms" for emergency management.

2. Deformation (Geodetic) Monitoring. Volcanoes typically expand before they erupt. Current techniques to measure this expansion emphasize EDM and tiltmeter technologies. EDM and "single set up" (dry tilt) leveling are useful but typically not

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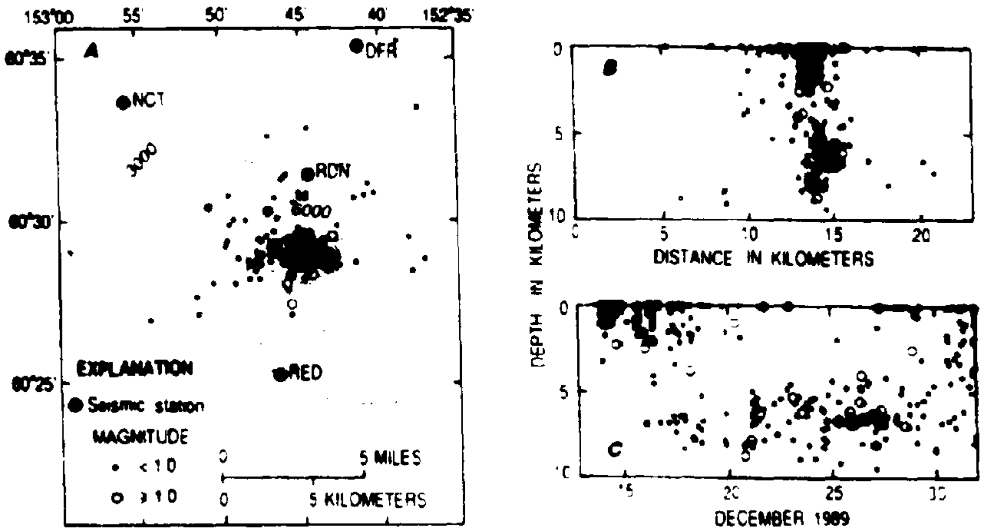

Fig. 1 Seismicity at Redoubt Volcano, Alaska, December 13-31, 1989 (after Brantley, ed., 1990). a. Location of seismic events. Light lines are topographic contours. b. East-west vertical cross section showing depth of seismic events. c. Variation of depth of seismic events with time. Note beginning of events >4 km on December 16, and increase in shallow earthquakes (and gradual shallowing of deeper earthquakes) after December 28. Low-frequency events, present earlier in December, returned on December 29. Two powerful explosions on January 2 sent tephra to heights >12,000 m and destroyed most of the dome.

real-time methods. Telemetered electronic tilt provides real-time data, but apparent tilt may also occur with electronic malfunction or instrument instability. Issues of precision and background noise may arise, depending on instrument locations and techniques, and the style and magnitude of deformations involved. In occasional use today, within a few years GPS will be widely used for monitoring.

Deformational monitoring has met with mixed success, being very useful at some volcanoes, but not at others. Lack of success in some cases may reflect volcano type and eruption style and process (e.g., small deformations difficult to recognize above background noise), and other cases, poor technique. Deformational methods should generally be used in combination with other methods, particularly seismics. Calderas may present unusual problems (deformation 10^2m in decades).

3. Gas Monitoring. Real-time monitoring of gas composition and emission rate is conducted at a few volcanoes. Results are mixed but techniques are improving. At Pinatubo between May 14 and 28, 1991, a 10-fold rise in SO_2 was measured, complementing evidence from seismology and deformation that magma was in fact rising into the volcano (PVOT, 1991).

4. Other Methods. A variety of methods, including thermal IR monitoring, and periodic or continuous gravity-field measurements, are used in an experimental basis at a few volcanoes but do not necessarily provide a basis for reliable warning. Wind profiles are useful for ashfall forecasts.

Besides the modern emphasis on expensive and increasingly sophisticated instrumentation, there remains the need for simple, low-cost tools. For example, the measurement of fault movements was carried out using nails and a steel tape in the crater at Mount St. Helens, and these data were useful for eruption prediction (Swanson, 1992). **Making measurements more sophisticated or more precise than required does not improve their value.**

Yet not all monitoring schemes attempted are successful, however promising they initially appear. Because manpower and resources are finite, excessive effort in unfruitful directions (usually confirmed only by hindsight) can damage the chances for success of a monitoring operation.

PREDICTION TECHNIQUES

The traditional way to predict the date and type of an eruption is to qualitatively compare its precursors to known precursors of previous eruptions. This is **pattern recognition** -- recognition of a particular pattern of unrest and its relation to an ensuing eruption (Tilling, 1989; Ishihara, 1990). Recurrence of the pattern suggests that another, similar eruption will occur. The principle is simple, but application is not necessarily straightforward.

The **materials science method** for eruption prediction is an attempt to systematize analysis of accelerating precursory data by application of a general law governing the failure of materials (Voight, 1988). This law can be applied using several procedures, of which the 'inverse rate' plot and 'linearized rate' plots seem most useful (Figures 2 and 3). The method can be combined with seismic or deformation monitoring, to provide a tool for (potentially) real-time eruption prediction. Advantages are claimed with Real-time Seismic Amplitude Monitoring (RSAM), and cumulative coda relations in near real-time prediction applications (Cornelius, and Voight, 1991).

However, success in obtaining reliable monitoring information on the state of the volcano does **not** necessarily mean that it is also possible to reliably predict the approximate date of eruption, or the type and size of the eruption. The balance is delicate between episodes of volcanic unrest and quiet, and between unrest and eruption. Applications discussed include volcanoes Mount St. Helens (USA) in 1980-1986, Ruiz in 1985, Redoubt (Alaska) in 1989-90, Merapi (Java) in 1990-1992, and Pinatubo (Philippines) in 1991.

LIMITATIONS

Existing predictive methods can provide valuable parameters for decision-making, but cannot guarantee success in every

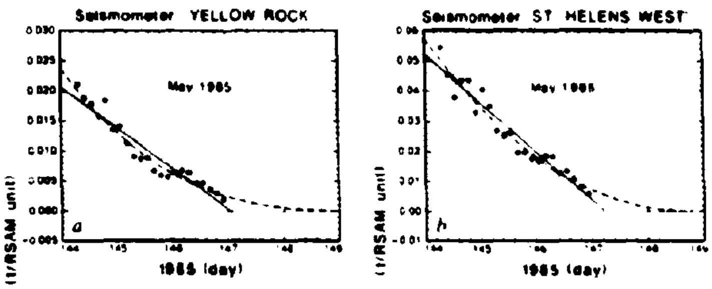

Fig. 2 **Inverse-RSAM** from two stations for the May 1985 eruption at Mount St. Helens (after Voight and Cornelius, 1991). Data are averaged over 3h. Time is GMT, Julian dates. Solid lines are linear fits to data; dashed lines are numerical fits for materials science constant α ($1< \alpha < 2$). Time of failure is estimated by extrapolation of inverse-rate versus time curve to a pre-determined intercept near the abscissa. A delay interval of about one day may separate "time of failure" from the "time of eruption" as indicated by the vertical line.

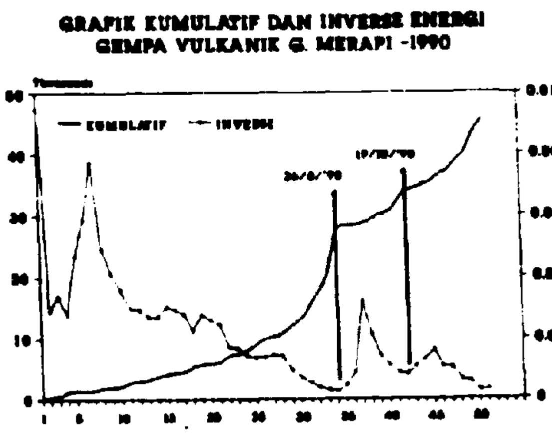

Fig. 3 Example of application of "materials science method" at Merapi volcano, 1990. Individual earthquakes were classified, and energy was estimated from coda length and amplitude. Energy released from volcanic earthquakes is represented by a cumulative energy curve (solid line) and by an inverse-energy rate curve (dotted line). Time of event occurrence is forecast by extrapolation of the inverse rate curve to a position "near" the abscissa. Application of the method enabled anticipation of a seismic crisis, accompanied by a gas burst, which occurred on 26-8-90. A second seismic crisis occurred on 19-10-90. In neither case did an eruption of magma products take place. (Unpublished data, Merapi Volcano Observatory, courtesy Pak Purbo 12-90).

application. **The results of predictive analyses with individual data sets should not stand alone, but should be incorporated within a comprehensive analysis of the given situation that considers all pertinent evidence and integrates the results of other forecasting methodologies. Use of the materials science method, for example, is not appropriate for certain situations -- e.g. for data types or instrument locations that are not consistent with an accelerating precursory trend, or where trends are highly irregular.**

The possibility of false alarms are not eliminated by any existing method. Included in this category is the "arrested eruption", in which the volcano displays the precursory symptoms typical of an eruption but culminates with an intrusive event. The 1983-1985 crisis at Rabaul Caldera, Papua New Guinea, provides one such example; the 1990 seismic crisis at Merapi provides another (Figure 3). Forecasting the outcome of unrest for caldera systems is more difficult than for small volcanic centers (Newhall and Dzurisin, 1990).

Prediction problems are almost inevitable, as the monitoring data may be consistent with several possible outcomes. If these outcomes occur with a known probability, and the probabilities are known to the forecaster, the forecast can be formalized mathematically. However, where the probabilities of these outcomes are unknown, less well known, or are not meaningful, the condition is one of uncertainty. Solutions in this case are less formalized.

These predictive problems of course carry over to civil officials, who are then forced to make optimal decisions under frequently non-optimal scientific, economic, and political circumstances.

Thus, at the same time that efforts are made to improve event prediction, event detection, and communications technology for early warning, efforts should also be made to improve education in facing uncertainty and false alarms, and to achieve improved understanding of policy science, so that the expectations of crisis management might be reduced to a tolerable level.

CONCLUSIONS

Because volcanoes are extremely complex systems, warning of impending eruptions is difficult under the best of circumstances (Swanson, 1991). Under the worst, as in areas for which monitoring is limited, ineffectual, or absent, warning is generally impossible. Hazard management must cope with these limitations in a broader context. Thus at Ruiz, authorities were unwilling to bear the economic or political costs of early evacuation or a false alarm; action was delayed to the last possible minute, with catastrophic consequences.

Predictive capability is best achieved by using a combination of data sets and methods, rather than by the reliance on any single procedure. Likewise we should not think in terms of field observations versus electronic measurements, but instead should attempt to provide a monitoring effort that uses the available manpower and expertise wisely, and incorporates the best of all observations into a consistent, unified interpretative approach. We should acknowledge that success is not inevitable despite technological advances. While ground deformation begins days to weeks before eruptive activity at some volcanoes, such as Kilauea, Mount St. Helens, and White Island, sensible deformation occurs only a few minutes before eruption at other volcanoes, such as Sakurajima (Ishihara, 1990). Deformation rates for some instrument placements accelerate before some eruptive periods at some volcanoes, but not at others (Mount St. Helens before May 18, 1980).

Other predictive problems include (1) recognizing the type and magnitude of an eruptive event; (2) timing the climax (rather than the initiation) of an event; (3) recognizing the end of the period of hazard; (4) optimizing monitoring approaches when funding or expertise is limited; (5) quantifying the degree of predictive uncertainty; (6) handling false alarms.

Laymen, the media, and public officials often have the unrealistic expectation of invincibility for modern science in volcanologic applications. It is well to keep in mind that a predictive capability is only one component of the more comprehensive activity of disaster prevention, and that despite important successes, volcanology has not reached the point whereby the timing, style and consequences of each eruption episode can be reliably foretold.

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VOLCANOES AND THEIR VOLCANIC HAZARD MAP PREPARATIONS

Sutikno Bronto
Merapi Volcano Observatory
Indonesia

ABSTRACT Indonesia is well known by having a lot of volcanoes which have been active since 1.8 million years ago (Quaternary in age). At first, only 128 volcanoes were considered active at present time. The total number of active volcanoes has become 129 after Anak Ranakah Volcano erupted in 1987. The youngest rock of pre 1987 eruption was dated in 14,570 320 years BP. In Philippines, Pinatubo volcano erupted in 1991 after over 600 years dormancy. The two volcanoes are not listed in the available Catalogue of active Volcanoes in the World. In addition, other destructive eruptions such as Mt. St. Helens (1980) and Unzen Volcano (1990-1991) have occurred after very long repose time (more than 200 years). These suggest to take inventory not only volcanoes having recorded eruptions and volcanic manifestations but all Quaternary volcanoes.

Volcanic eruptions vary from small (e.g. Slamet, Semeru and Gamalama Volcanoes) through moderate (Mt. St. Helens, Pinatubo Volcano) to large scales (Tambora in 1815 and Krakatau in 1883). So far we cannot predict whether a volcanic activity will produce small, moderate or big eruptions. In the mean time, available volcanic hazard maps in Indonesia are only for overcoming small scale volcanic eruptions, in which the danger zone cover an area with less than 20 km in diameter. In case of the 1991 Pinatubo eruptions, affected areas are about 50 km in diameter; and damaged areas caused by the 1883 Krakatau eruption reached over 100 km in diameter. These suggest that volcanic hazard maps for moderate and large scale eruptions must be provided besides small ones.

Damages caused by volcanic eruptions are not only in the ground but also in the air. Aircraft accidents occurred several times in the last decade. Galunggung eruptions in 1982

forced Boeing 747 Jumbo Jets of British Airways (BA09) and Singapore Airlines (SQ21A) to make an emergency landing on Jakarta International Airport. Another Boeing 747 (KLM 747-400 aircraft) entered a cloud of volcanic ash from Redoubt Volcano, Alaska, in 1989. Although the aircraft landed successfully, extensive repairs with very high costs were required. Recently, Pinatubo eruptions affected 14 big aircraft and caused the Manila International Airport closed. These also suggest that preparations of volcanic hazard maps regarding to aviation safety are necessary.

In addition, submarine volcanic eruptions might threaten sea transportation. Further detailed studies are needed in order to prepare hazard map of volcanoes.

INTRODUCTION

In Indonesia, there is likely no time gap of volcanic activities from active volcanoes at present time through Holocene (0,01 million years) to Pleistocene (1.8 million years) (as Quaternary volcanoes) and even with Pliocene volcanic activities (3 million years; Soeria Atmaja et al, 1991). In a general view, it is well known that the longer period of dormancy the more explosively some time at present or in the future. In the mean time population and land-use to increase and to move closer to the hazard source. People living in the surrounding area tend to forget past volcanic disasters.

Volcanic eruptions vary from non violence to very destructive events. These threaten not only people living in the surrounding area but also aviation and sea transportation passing nearby. Nearly all area surrounding Indonesian active volcanoes are densely populated and have very high economic values. So, in order to minimize loss of life and property damage, besides continuous monitoring systems are carried out, volcanic hazard maps must be prepared properly.

VOLCANO INVENTORY

Formerly, in Indonesia only 128 volcanoes were considered active. These have become 129 since Anak Ranakah erupted in 1987. Before the 1987 eruption Anak Ranakah was not listed as an active volcano because there was no indication of active volcano and the youngest rock was dated in 14,570 some 320 years BP (Abdurrachman et al., 1989) in Philippines, Mount Pinatubo that was also not listed as an active volcano erupted in 1991 after 635 years dormancy (Wolfe & Self, 1983). In addition, very destructive eruptions such as Krakatau in 1883, Mount St. Helens (1980) and Unzen Volcano (1990-1991) occurred after over 200 years repose time. These imply that long dormant volcanoes can erupt explosively at present time or in the future. In order to anticipate the volcanic disasters firstly it is suggested to take inventory not only volcanoes having recorded historic eruptions and volcanic manifestations but all Quaternary volcanoes. Thus volcanoes such as Telomoyo, Ungaran and Nuria in Central Java; Willis, Penanggungan and Baluran in East Java; Cikurai, Karacak and Mandalawangi in West Java must be registered. Furthermore, some basic studies on those volcanoes should be started.

VOLCANIC HAZARD MAP ON THE GROUND

Volcanic hazard maps are required for each volcano in order to minimize loss of life and property damage surrounding the volcano. So far, available volcanic hazard maps in Indonesia are only for overcoming small scale volcanic eruptions, in which the danger zone covers an area with less than 20 km in diameter. Table 1. shows zonation of volcanic hazard map in Indonesia (Kusumadinata, 1979). The validity of those maps is based on the following assumptions :

a. Eruptions occur in the main crater from which volcano has erupted in the past, and nor from other unexpected points such as flank eruptions.
b. The eruption column will be vertical
c. Eruption will not form a caldera
d. Topography of the volcano does not change considerably

In other countries such as USA (Crandell, 1980), and Ecuador (Miller et al, 1978) each volcano also has only a volcanic hazard map.

In facts, volcanic eruptions vary from small (recently e.g. Slamet, Semeru and Gamalama volcanoes) through moderate (Mt. St. Helens, 1980; Mt. Pinatubo, 1991) to large scales (Tambora, 1815 and Krakatau 1883). Volcanologists are expected to estimate whether an eruption will yield large, moderate or small explosions. If the available volcanic hazard map is only one type and it does not match with an eruption that is in progress, volcanologists may lose their credibility in the eyes of the public.

In order to anticipate this problem, it is suggested that volcanic hazard maps for moderate and large scales must be provided besides the small ones. To estimate the hazard degree which match with one of the volcanic hazard maps in the future, monitoring systems and estimations of explosion energy particularly the paroxysmal event are the most important factors.

Figures 1 and 2 show available volcanic hazard maps of Galunggung, whereas figures 3,4 and 5 are proposed volcanic hazard maps to the volcano for second, third and fourth degree hazards (Bronto, 1989). The first degree hazard is caldera forming event, thus the most explosive and destructive eruption. To prepare a volcanic hazard map for the first degree hazard, further research is needed.

VOLCANIC HAZARD MAP FOR AVIATION

Indonesia has very busy air traffics connection nearly all parts of the world. When a volcanic eruption occurs aircraft can be affected by highrising ash clouds. Explosive eruptions at some volcanoes may generate a column that will rise in minutes to the cruising heights of international aircraft, yet in the past some aviation authorities have been unaware that an eruption has taken place until hours after the event. Pilots may not be able to see the volcanic cloud ahead of them if it is concealed by normal weather clouds or if flights are at night. Pilots are unaware of the cloud until they enter it, engines begin to surge, and St. Elmo's fire is seen on leading surfaces.

Aircraft damages caused by volcanic explosions have occurred in many countries. In Indonesia, Galunggung eruptions in 1982 forced Boeing 747 Jumbo Jets of British Airways (BA09) and Singapore Airlines (SQ21A) to make an emergency landing on Jakarta International Airport. Both aircraft were flying at night and encountered ash at a height of 37,000 feet (Tootell, 1985; Johnson, 1991). Although the aircraft landed successfully, extensive repairs with very high costs were required. Other Indonesian volcanoes have also been hazardous to aircraft. A British Airways 747 aircraft on a diversionary route around Galunggung Volcano ran into an ash cloud from Colo Volcano in July 1985, and a Qantas Airways 747 aircraft on a night-time flight between Hong Kong and Melbourne ran into the drifting ash (Johnson, 1991).

In Japan, Kagoshima International Airport is only 24 km north of the repeatedly active volcano Sakurajima, and there were eight reported incidents from 1975 to 1986 of damage to aircraft that flew into ash clouds from the volcano (Japan Meteorological Agency, 1986).

Alaskan Volcanoes have also interfered with military and civil aviation movements since at least 1955 when an eruption from Mount Spurr damaged US Air Force Aircraft, and eruptions at Augustine Volcano affected aircraft in both 1976 and 1986 (e.g. Kienly et.al., 1986). In 1989 a Boeing 747 (KLM 747-400 aircraft) also entered a cloud of volcanic ash from Redoubt Volcano. The 1980 Mount St. Helens eruption in the United States also caused substantial disruption to airports and military bases downwind from the volcano, including engine failure on Lockheed transport aircraft (O'lone, 1982).

In 1991, Pinatubo eruptions in Philippines affected 14 big aircraft and caused the Manila International airport closed.

All the incidents require to provide volcanic hazard map for aviation besides volcanic ash warning system. In Indonesia there are three volcanic areas i.e., 1. Volcanoes located along island of Sumatra, Java, Bali, Nusa Tenggara Barat, Nusa Tenggara Timur and Ambon in Banda Sea, 2. Volcanoes in North Sulawesi, and 3. Volcanoes in Halmahera Islands. Based on their geographic position, these volcanoes are divided into several groups which are plotted on volcanic hazards maps for aviation. The volcanic hazard maps may be prepared at firs by studying degree of explosive eruption of each volcano which occurred in the past and a general understanding that a big explosion will produce high and widespread ash clouds.

VOLCANIC HAZARD MAP FOR SUBMARINE VOLCANO

Besides volcanoes appearing on the ground and island volcanoes there are many volcanoes which are under sea water. Examples of these volcanoes are Barren Island in

Andaman Islands, north of Sumatra island, Nieuwerherk and Emperor of China in Banda Sea, Banua Wuhu and 1922 submarine volcano in Sangihe Islands, north of Sulawesi, and unnamed Seamount in Solomon Islands.

The areas where Barren Island is located has a very high value in term of sea transportation. During eruption submarine volcanoes not only eject materials but also cause high sea wave or tsunami. Those kinds of danger threaten ships, people living in islands nearby, and aviation.

Unfortunately, there is very little data about submarine volcanoes. So further studies are required before preparing volcanic hazard maps.

CONCLUSION

In order to minimize volcanic risk efforts on volcano inventory and volcanic hazard mapping required. Volcanoes which are taken inventory are not limited for recent activities but all Quaternary volcanoes. Volcanological studies and hazard maps are prepared for mitigating volcanic hazards on the ground, in the air and sea.

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Table 1. Zonation of volcanic hazard map in Indonesia (Kusumadinata, 1979)

I. Established hazard maps:

1. Forbidden zone closed zone is an area closets to the danger source, that is easily affected by pyroclastic flows and ballistic blocks and bombs, and therefore should be permanently abandoned.
2. First danger zone is an area which was in danger during previous eruptions although it may not be affected by pyroclastic flows. During paroxysm, however, it may be destroyed by ballistic blocks and bombs.
3. Second danger area comprises the areas situated in or close to valleys originating from the summit, and which may be invaded by rain lahar. This zone may be eventually divided into " alert zone and " abandoned zone ". The alert zone is an area situated near topographically high, e.g., a hill which can provide an evacuation area in case of lahars.

II. Preliminary hazard map :

1. Danger Zone is an area that has to be absolutely abandoned in case of signs of increased activity. The situation may be afterwards investigated by a competent volcanologist.
2. Alert Zone is an inhabited area where people have to be on their alert, and evacuation from this zone may also be necessary, depending on the development of the volcano's activity.

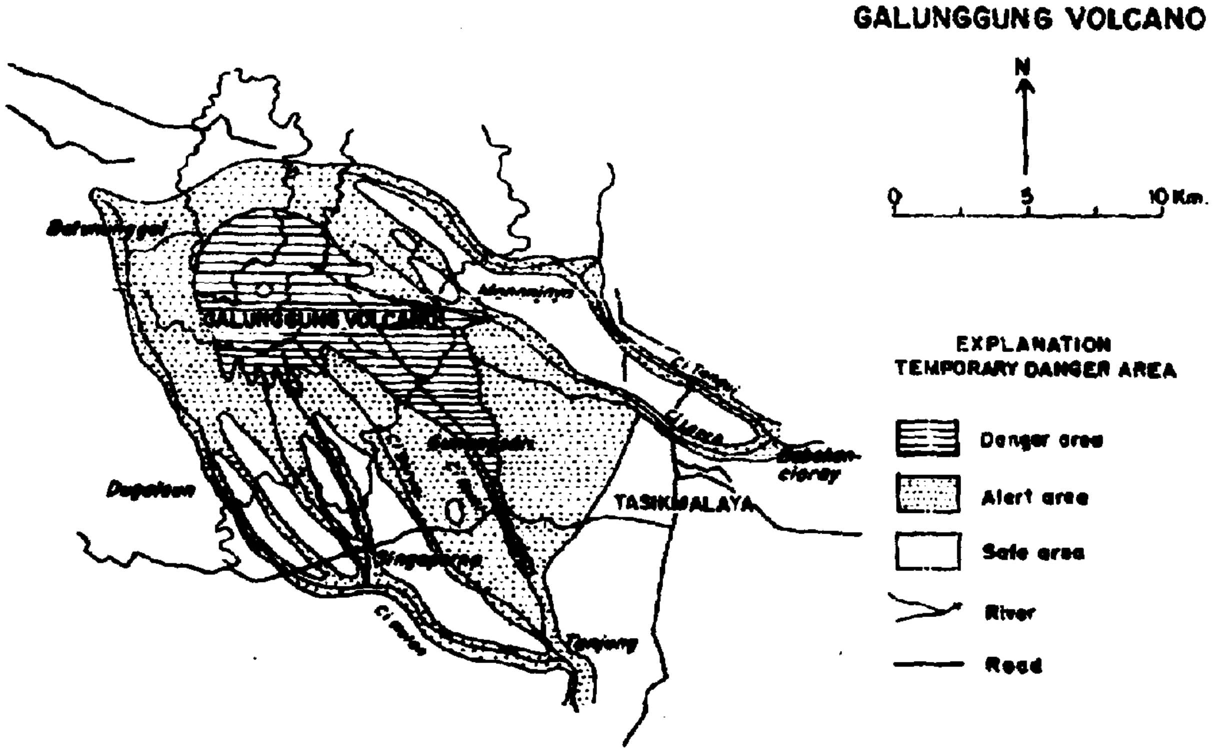

Figure 1. Preliminary volcanic hazard map of Galunggung Volcano (Kusumadinata, 1979)

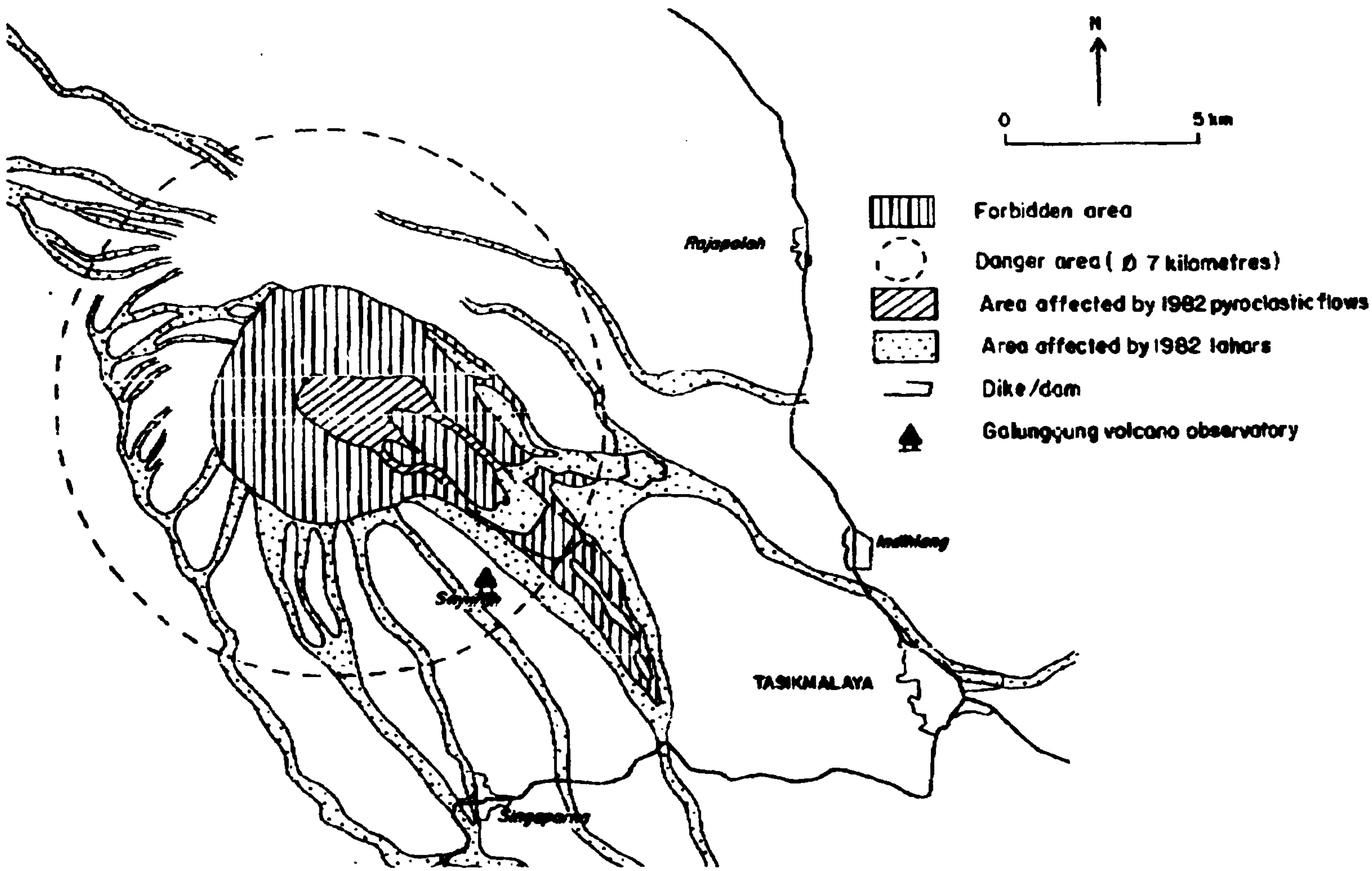

Figure 2. Preliminary volcanic hazard map of the 1982-1983 Galunggung eruption (VSI, 1985)

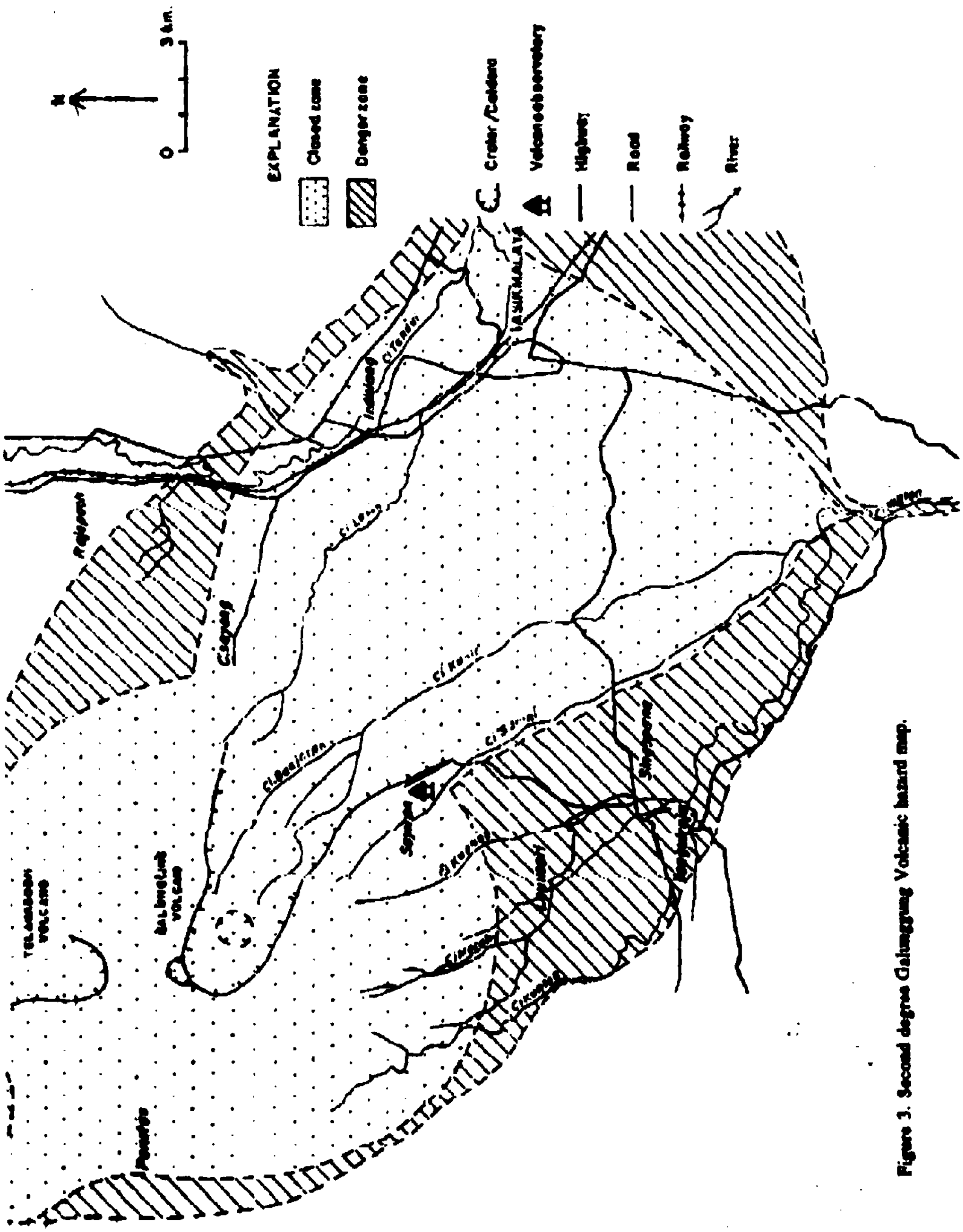

Figure 3. Second degree Galunggung Volcanic hazard map.

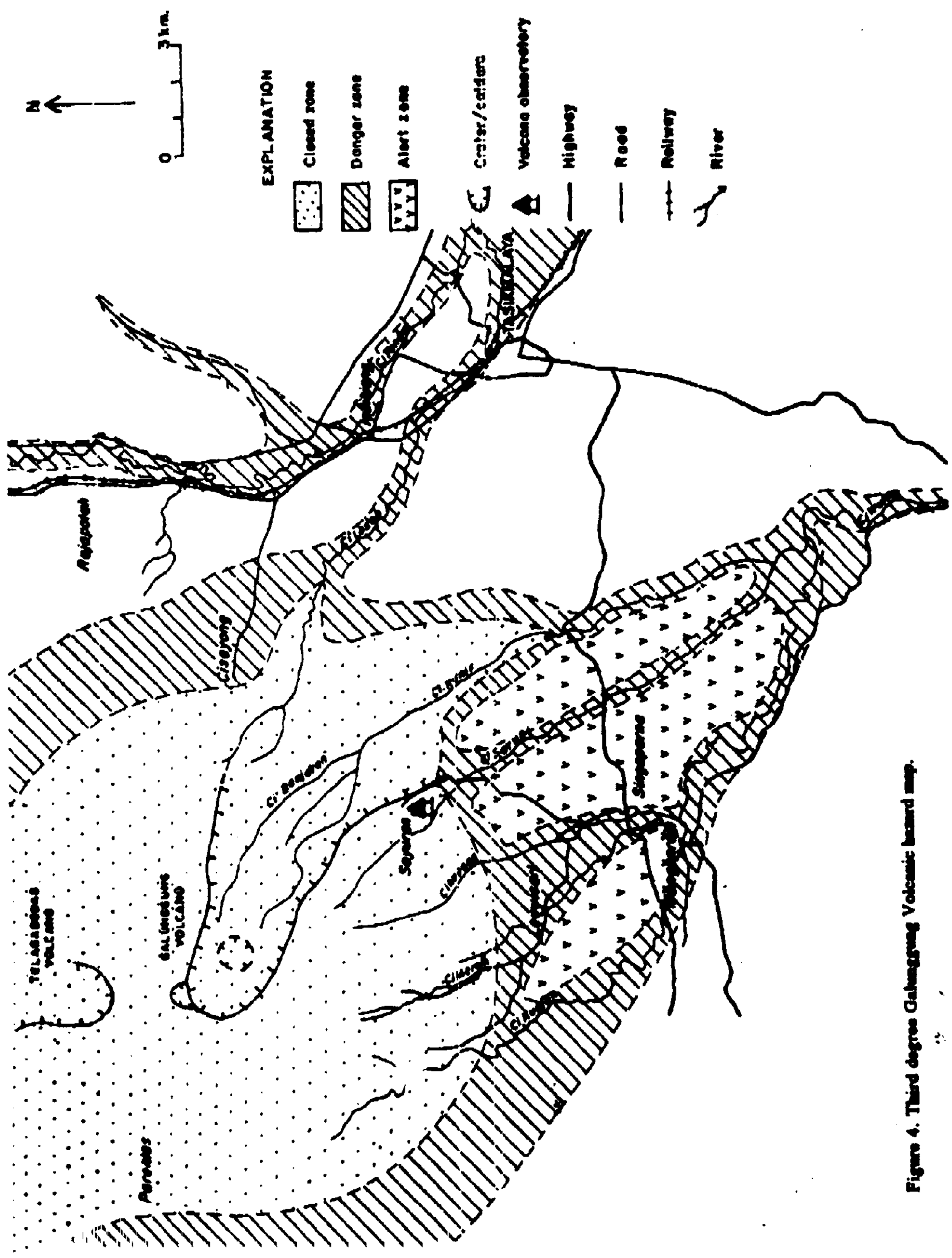

Figure 4. Third degree Galunggung Volcanic hazard map.

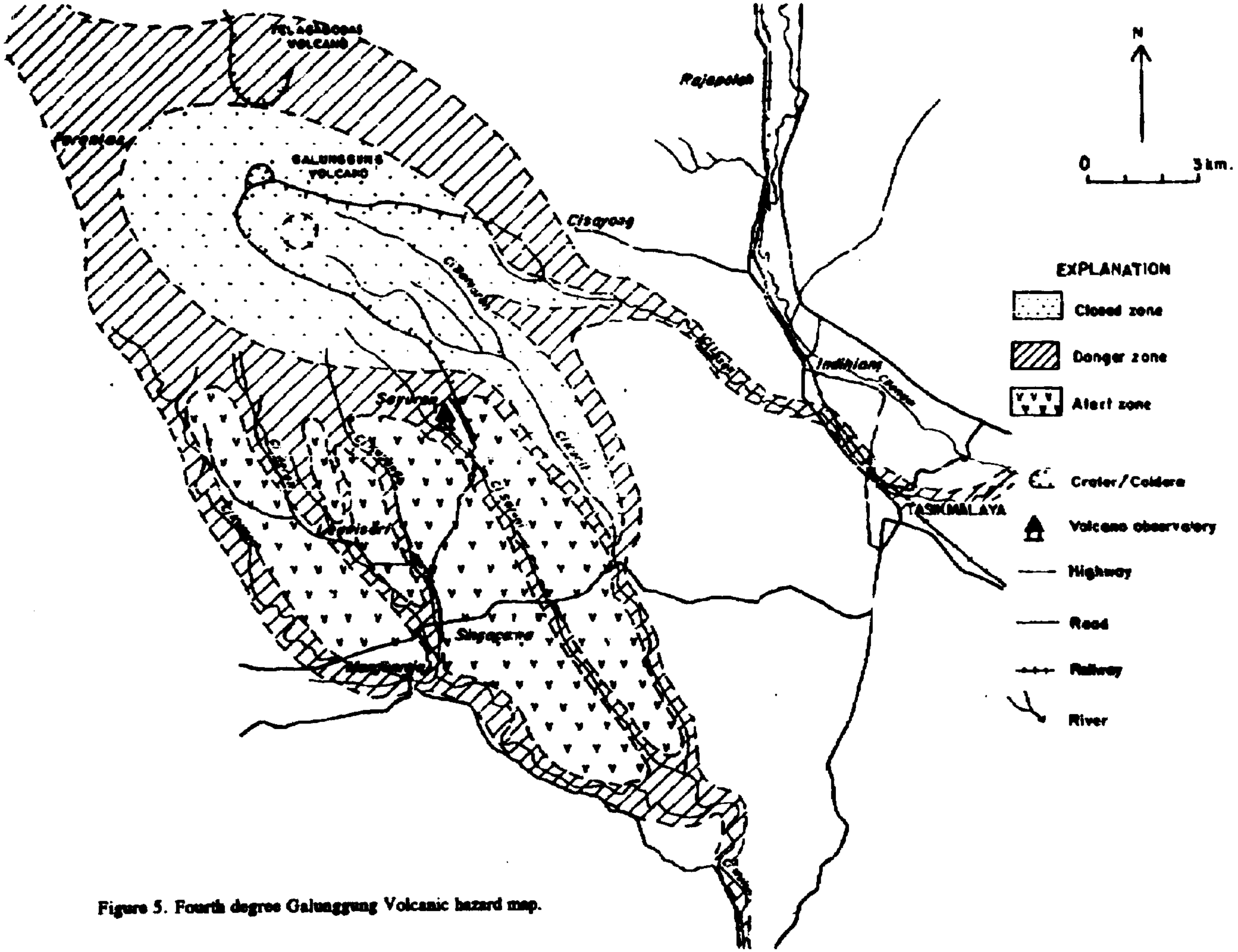

Figure 5. Fourth degree Galunggung Volcanic hazard map.

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THE VOLCANIC HAZARDS OF HYDROTHERMAL AREAS IN INDONESIA, AND MITIGATION EFFORTS

Muziel Alzwar
Volcanological Survey of Indonesia
Directorate General of Geology and Mineral Resources
Indonesia

ABSTRACT In Indonesia there are 21 fumarolic and solphataric fields related to the geothermal systems of active volcano. The disturbance of such hydrothermal system, e.g. by triggering tectonic activity, may result in phreatic eruptions or gas emissions. Occurrences of disastrous phreatic or gas emission events in historic time e.g. Mt. Papandayan (West Java, 1772), Suoh Antatai (Lampung 1933), and Dieng Plateau (Central Java, 1928, 1939, 1944, 1964, and 1979) and Mt. Gamalama (North Maluku; 1775). Hazard related to such events include primary lahars, phreatic surges, tephra fall and release of poisonous gases.

Some fumarolic fields have been developed for hydrothermal power e.g. Kawah Kamojang and Mt. Salak (West Java), Lahendong (North Sulawesi) and Dieng Plateau (Central Java). However, these areas are still dangerous. Integrated investigation, e.g. hazard mapping and zonation, monitoring, and risk assessment are needed to reduce risk. Such investigation include geologic, geothermal and volcanological mapping, geochemical and geophysical (including microseismic method), and tectonic analysis, with the aim to identify hazard-prone area caused by increasing hydrothermal activity.

INTRODUCTION

Based on the recognition of the volcanic activity stages visibility in Indonesia there are three types of volcanic stages activity related to the status of active volcano, namely (Van Bemmelen, 1941; see table 1)

- A-Type (stage of activity)

  Volcano still erupting of magmatic since 1600, 79 volcanoes.

- B-Type (stages of activity)

  Volcano not erupting of magmtic since 1600, the activity is limited to their emission of solphataric/fumarolic; 29 volcanoes.

- C-Type (stages of activity)

  Fumarolic emission field; 21 field.

These active volcanoes are located on the active volcanic belts, namely, Sunda Zone, Banda Zone, Halmahera zone, where these zones are strongly related to tectonics which are showing the active subduction zones between Australia, Indian Ocean and between Pacific Ocean to Eurasia plates, and between Pacific Ocean to Eurasia plates (Fig. 1).

Mechanically, based on the role of direct and indirect magma there are two types of external volcanic activity : magmatic eruption and semi-magmatic/semi-volcanic eruption. Magmatic eruption produces new volcanic rocks as silicate melt in the form lava flows, pyroclastic or hot glowing cloud, (or nuees ardante), fragments or tephra fall of various sizes (from ash to bomb).

Primary lahar flow rushing down the earth's surface in an eruption is a primary disaster to human life and the environment.

Limited semi-magmatic activity (in normal stage) shown by juvenile gas emission activity is strongly affected by the formation of water vapor, assuming the form of fumarole or solphatara. This increasing activity can develop into an eruption which does not produce new magmatic or volcanic rocks. It only releases volcanic gas or steam, or accompanied by hot volcanic mud flows.

Semi-volcanic eruptions particularly hydrothermal eruption which has caused violent disaster in Indonesia, in the years of:

1772	Mt. Papandayan Crater eruption, West Java; 2951 people killed and destructed arable land and villages.
1775	Mt. Gamalama, Ternate Island, North Maluku; 141 People killed.
1780, 1902, 1903, 1919	Ratu Crater, Mt. Salak (West Java); destructed arable land.
1933	Pematang Bata eruption, Suoh Autatai depression, Lampung, South of Sumatra; no recording of victims.

1928, 1938/1939, 1979 Timbang Dieng Mts, Central Java, destructed Timbang Villages (1928, 1938/1939) and 149 people killed (1979).

1944 Sileri, Dieng Mts, Central Java.

The above type of eruption, commonly occurs in the fumarolic field known as C-Type of activity.

MECHANISM OF SEMI-VOLCANIC ERUPTION

By the different mechanism there are two types of semi-volcanic eruption, namely "phreatic eruption" and "hydrothermal eruption".

a. Phreatic eruption, releases juvenile gas or water vapor mixed with volcanic gas. This steam is formed as a result of ground/rain water heated directly by the magma moving to earth surface. The vapor containing volcanic or juvenile gas is immediately released to the earth surface. This eruption is an early indication of increasing activity from the volcanoes of A-type activity stage. Phreatic eruption at this condition shows low energy. Volcanic gases and steam carrying chunks of rocks are scattered around the site of central eruption, but ashes carried by the wind can spread further.

b. Hydrothermal eruption of semi-volcanic activity type, which is the focus of discussion in this paper, because of its mechanism, eruption intensity and the potential disasters factor causes. Eruption mechanism is associated with the balancing disturbances of internal process (physical and chemical development) or external factor particularly by tectonic which affected to hydrothermal system.

The hydrothermal system is formed by controlling parameters (see Fig. 2) :

- the presence of a source of heat, in the form of the active magma or young intrusion body and hot rocks produced by heated (active-) magma.
- adequate ground water, usually as regional flows.
- the subsurface geological condition, particularly with the presence of source rocks, capping layers of impermeable rocks, and structure control (fracture, fissure or fault systems).

The volcanic body, or plutonic intrusion intruding sedimentary rock layers consisting mostly of marine sediment. Various types of marine sedimentary rocks would be source rock for the hydrothermal system, which is composed of hot-fluid consisting of saliferous solution in the marine sedimentary rocks.

The development of thermodynamic condition in the hydrothermal system can transform physical quality from water/liquid to steam, or saturated gas with high P and T (superheated steam).

The transformation to steam domination system (with gasic elements) can occur in shallow layers of rocks bed. Balancing disturbances, in the hydrothermal system, as a result of internal process or external control, would be able to cause "hydrothermal eruption", with blowing upon the steam (and gas) or hot water.

Hot fluid leak from hydrothermal system will reach the earth surface in the form of fumarole or solphatara activity or hot spring carrying a number of gasic elements.

In Indonesia there are 21 fumarolic fields occupying active volcano belts where intruded marine sedimentary/meta-sedimentary rock layers; e.g. geothermal areas of Prabakti (Mt. Salak, West Java), Manuk-Darajat (Mt. Papandayan, West Java), Karaha (West Java), Dieng Mountains (Sikidang, Sileri, Tumbang Condrodimuko, Central Java), Suoh Antatai (Lampung), Gayo-Lesten (North Sumatra), etc.

A number of active volcanoes of A and B types of volcanic activity stage grow on and intruding sedimentary rock layers will produce solution or fluid containing gasic elements. This type of volcano includes Mt. Sibayak (North Sumatra), Mt. Talang (West Sumatra), Mt. Sekincau Belirang (Lampung), Mt. Salak, Mt. Tangkuban Perahu, Mt. Papandayan (West Java), Mt. Pakuwojo and Mt. Butak (Dieng Mts. Central Java), Mt. Tampusu-Lahendong (Minahasa, North Sulawesi), etc.

THE HYDROTHERMAL ERUPTION HAZARD

Based on the physical and chemical condition, hot fluids which are coming from eruption will be able hazardous threat. This hazardous threat is characterized as :

a. Primary Hazard, or "direct hazard", caused directly by an eruption blowing upon the steam and hot gas, hot cloud, eruption lahar and poisonous gases.

b. Secondary Hazard, in the form of a landslide caused by the formation of hydrothermal alteration triggered by saturated hot water flowing or steam emission (fumarole/solphatara).

Hydrothermal eruption produces high pressured hot fluid flowing in great mass through the earth surface, as a result of the hot fluid emission, explosion holes or 'maar' are formed. In some active volcanoes in Indonesia are found a number of 'maars', they are formed and scattered in the lower mountain side, interesting example are Mt. Lamongan (East Java), Dieng Mountains (Central Java), Suoh Antatai depression (Mt. Sekincau Belirang, Lampung), Linau Lake (Mt. Tampusu, Minahasa, North Sulawesi), Prabakti crater (Mt. Salak, West Java) (Fig. 3, 4 and 5).

Areas located around the active geothermal areas with the 'maar' visibility tends to be vulnerable to the danger threat. A tremendous hydrothermal eruption causing disaster occurred at Mt. Papandayan (West Java) in the year of 1772, claiming 3,000 human lives and destroying thousand of acres of farmland and settlements. The landsliding of the products of the hydrothermal activity process can also occur to active volcano with great intensity visibility. Based on the visibility and the event currently happening at present, a number of volcanoes containing fumarolic and solphataric fields in Indonesia had ever occured big landslide. Big landslides causing disaster occurred at Mt. Semeru (East Java) and Mt. Talang (West Sumatra), damaging farmland, physical construction, settlement areas, and also a great number of human lives.

MITIGATING EFFORTS

Methods of active mitigating in an effort to reduce the hazardous threat of semi-volcanic activity associated with hydrothermal eruptions are very limited. Using seismic method is not so easy as to monitor normal volcanic activity, because the energy released from the hydrothermal system activity is too low. Monitoring hydrothermal system in depth of 2,000-3,000 meters could be applied by the microseismic method although it takes a long period; it should be done as in the exploration of geothermal potential. The activity of hydrothermal system in the form of up flowing of hot-fluid or out flowing of hot-fluid towards shallow depth may cause a weak surface explosion or gas-emission explosion.

Geoelectric method, a discipline using resistivity, support exploration of the geothermal potential, is applied to detect the background depth, the horizontal and vertical development spread and the hydrothermal system character. Geophysical method, a discipline using gravity, will confirm the status of hydrothermal system controlled by geological and lithological structure beneath the surface. Geochemical method, done by chemical analysis of steam/gas emission (fumarole/solfatara) and hot spring, is used to gain information concerning the origin of the water, the kind of rock layers passed by the ground water, hot fluid and rock beds and also gives information on the type of the hydrothermal system and the geothermic potential source.

Another mitigating activity is by geological mapping of the geothermal areas. This is done by observing the distribution of the geothermal/heat flow zone with genetic analysis related to volcanic activity active tectonic structure control. Tectonic structure fractures, fissures or faults which can control the development of volcanic structure is a weak area ideal for the formation and development of hydrothermal system or hot fluid migrating. (Fig. 6)

Based on the geothermal geology mapping including fractures, fissures or faults structure distribution, supported by the information from geophysical and geochemical methods, it may be applied to make the zoning map.

This map designate areas vulnerable to the dangers of :

- Spreading leakage of steam/gas emission containing poisonous gas which occur along the fractures fissures or fault.
- Primary-lahar distribution (if there is an eruption) through river valley.
- Wet landslide, particularly through valleys cutting down the mountain side.

Civil construction in the effort to mitigate the hazardous threat of hydrothermal activity and eruption may not be as ideal as mitigating for normal volcanic eruption. It is caused by the type of hazardous threat, natural condition and location of the geothermal zone in relatively flat area or "plateau" (Dieng Plateau, Suoh Antatai, Sibangor-Tenya etc.). Steam spread containing poisonous gases cannot be mitigated, by physical construction. Wet landslides and lahar flow through through the river valleys on the volcanic slope, can still be mitigated by physical construction in the downstream area.

Passive mitigating efforts can be done by drawing up local government regulation to close the hazard-prone areas and by informing the people living near the active geothermal areas (Fig. 7). Some geothermal areas in Indonesia have been treated as tourist spots. The tourists should be informed of any location that are off limit or need extra caution because of possible hot mud blowing and steam with poisonous gas content.

CONCLUSION

1. The number of 21 fumarolic/solphataric fields characterized as the C-type of volcanic activity stage in Indonesia, and a number visibility location which form the active geothermal areas.
2. Solphataric/fumarolic fields and geothermal generated from hydrothermal system is directly and indirectly related to volcanic activity which magma intruding marine sedimentary or meta-sedimentary rock layer. Hydrothermal system formed in marine sedimentary rock layers will be able to consist and develop poisonous gases.
3. The balancing disturbances in the hydrothermal system as a result of internal and external process (tectonics) may trigger hydrothermal eruption (steam and gas, or primary lahar). This is a primary hazard threat to the environment and human life. Fluid leak (steam gas and hot spring), and hot fluid from the hydrothermal system can assist the hydrothermal alteration process to the rocks. The hydrothermal alteration activity changes the physical quality of rocks, it triggered by the hot springs or ground/rain water, a wet landslide were occurred. This indicates the secondary danger threat.
4. The active mitigating efforts semi-volcanic activities associated with hydrothermal activity are limited and tend to be done as in the exploration of the geothermal potential geophysical, geochemical and geological mapping. Geothermal mapping shows the structure associated with the formation of hydrothermal system beneath the surface and the hot fluid leakage distribution in the form of steam/gas emission activity or hot spring sources on the surface. These methods can be used to compose a hazard zoning map, showing prone areas to the primary and secondary hazard. The closing of location considered vulnerable by drawing up regulation is a preventive measure. Passing on information to the people nearby, or to the tourists, is strongly advised.

ACKNOWLEDGEMENT

The author wishes to express his sincere gratitude to the Organizing Committee of EMNHD for the formal invitation and sponsership to participate the EMNHD-2 Workshop; particularly Prof. A. Chiu (University of Hawaii at Manoa, USA), Mr. Aspan S. Danuatmodjo and Mr. Hernowo (IDMC) for completing the manuscript.

And the author has also to thank the members of the Secretariat of Organizing Committee of EMNHD-Indonesia for all services rendered.

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Table 1 Zones of Active Volcanoes in Indonesia, With the Number of Volcanoes and Type of Volcanic Activity Stage

Zone of active volcanoes	Islands	Type of Volcanic Activity Stage			Total
		A	B	C	
Sunda	Sumatra	11	12	6	29
	Sunda Strait	1	-	-	1
	Java	20	10	5	35
	Bali and Nusa Tenggara	22	3	5	30
Banda	Volcanics Islands in the Banda Sea	8	1	-	9
Halmahera	Northern and Western part of Halmahera	6	1	-	7
Minahasa	Northern part of Sulawesi	6	2	5	13
	Sangihe Talaud Isla	5	-	-	5
		79	29	21	129

Table 2. Poisonous gases at Sinila Crater and Surroundings during the hydrothermal eruption in 1979, and gas leakage at Bakal village in 1992

Gases	Concentration in ppm; Hydrothermal activity in 1979					Hydrothermal Activity in 1992
	Sinila	Sigludug	Timbang	Jalatunda	Threshold	
CO	1.0	5.0	2.5	-	0.05	
CO_2	200,000	25,000	10,000	520,000	5,000	>
H_2S	125	50	less	-	10.0	>200
HCN	20	48.5	6.5	4.5	10.0	>200
H_3As	10	20	less	9.5	0.05	
NO_2	3.5	10	10	-	5.0	
Cl_2	-	-	125	-	1.0	
SO_2	-	-	-	-	5.0	
Victim	149 Person killed					1 Person killed

Table 3. Some numbers of volcanoes or volcanic activity in Indonesia which are showing the semi-volcanic activities or wet landsliding related to hydrothermal activities.

No.	Name of Volcanoe or Volcanic Activity	Type of Volcanic Activity Stage A	B	C	None	Years of Act. tendency Eruption	Landsliding
1.	We Island			*			
2.	Gayo Lesten			*			
3.	Sibayak		*				
4.	Sinabun		*				
5.	Heldtoba Tarutung		*				
6.	Sibual-bual		*				
7.	Sibangor Tonga, Sorik Merapi	*					
8.	Merapi	*					1985
9.	Talaud	*					1989
10.	Parek Sembilan (kerua) Lampung				*		
11.	Bukit Daun			*			
12.	Marga Bayan			*			
13.	Sekincau Belirang		*				
14.	Pematang Bata (Suoh Antatai)			*		1933	
15.	Hulubelu			*			
16.	Rajabasa		*				
17.	Prabakti-Salak			*			
18.	Salak	*				1780, 1902/03, 1919	*
19.	Domas-Tangkuban Perahu	*					
20.	Kamojang			*			
21.	Darajat			*			
22.	Papandayan	*				1772	
23.	Karaha			*			
24.	Guci (Slamet)	*			*		
25.	Timbang, Sinila Condrodimuko	*			*	1928, 1938/1939 1979	
26.	Pagarkandang, Sigludug, Sileri				*	1944, 1964, 1985 (Sileri)	
27.	Condrodimuko (Lawu)	*					
28.	Semeru	*					1986
29.	Merapi	*					
30.	Pocok Leok Calder			*			
31.	Murubaya			*			
32.	Larantuka						
33.	Inie Lake	*					
34.	Ili Lewotolo	*				1985	
35.	Poketanda	*				often	
36.	Inie Rie			*		1972	
37.	Lereboleung	*					
38.	Dukono	*				moore activity	
39.	Lahendong			*			

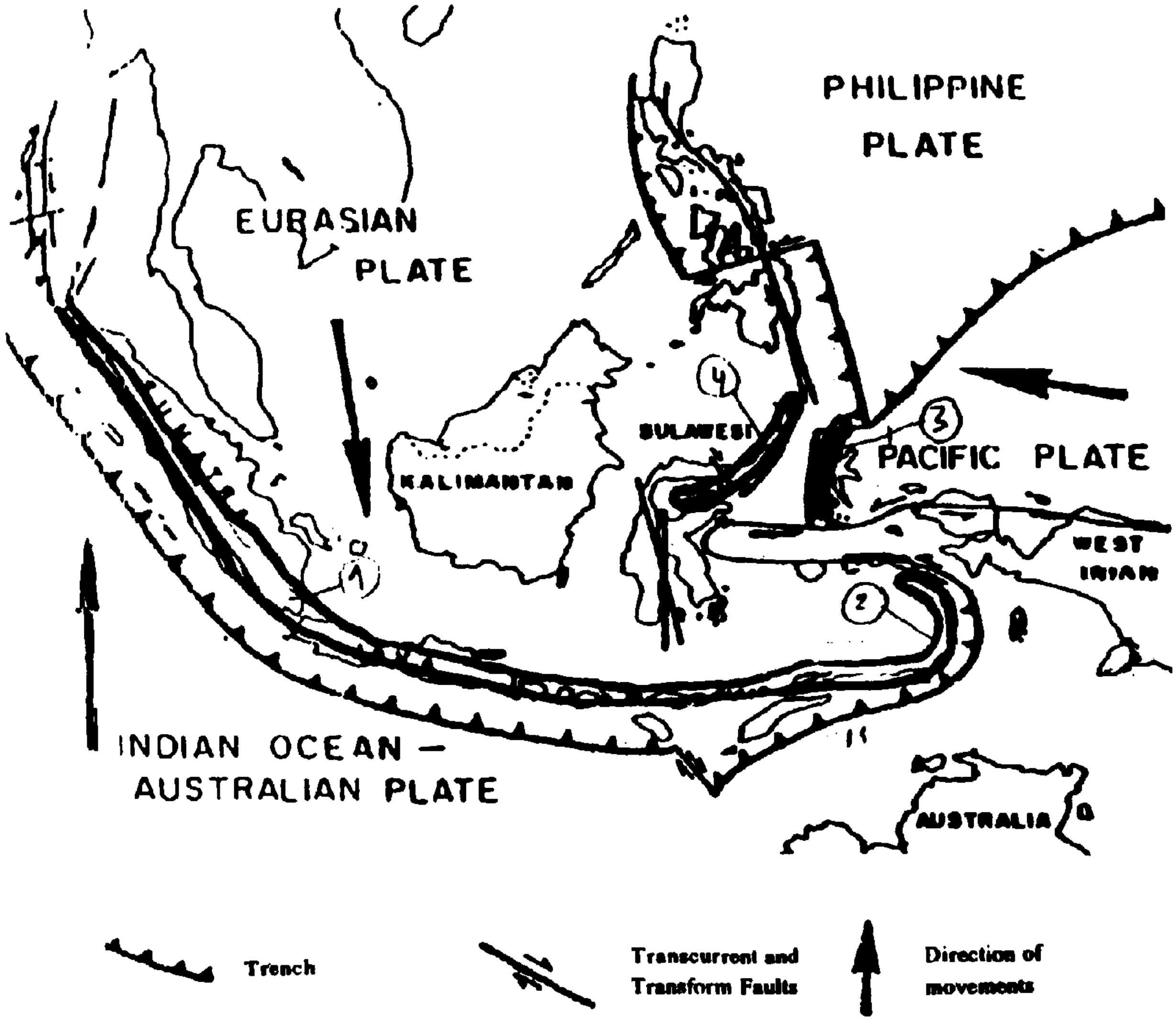

Figure 1. Plate baundaries in Southeast Asia (Katili, 1973)
Active Volcanic Zones :
1. **Sunda Zone**
2. **Banda Zone**
3. **Halmahera Zone**
4. **Minahasa Zone**

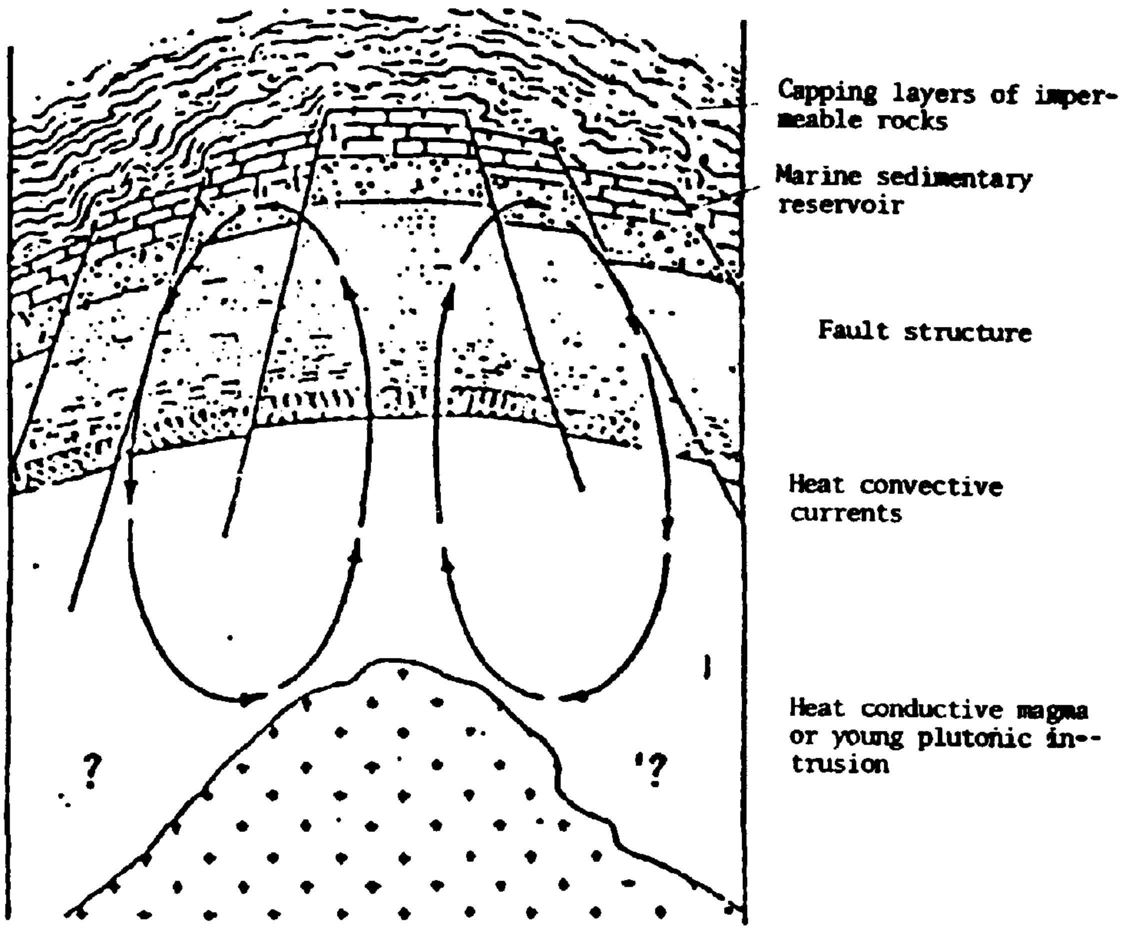

Figure 2. Mechanism of Hydrothermal System generating in marine sedimentary reservoir

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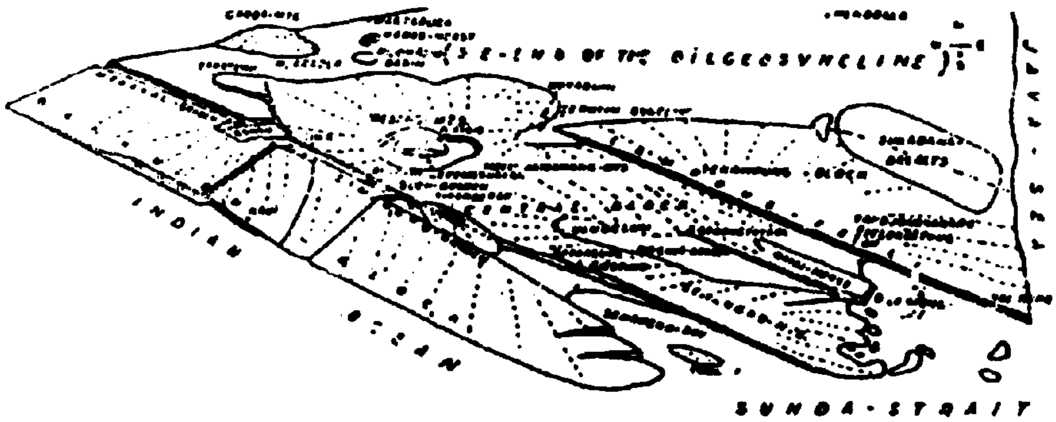

Figure 3. Tectonogram of South Sumatra, (Van Bemmelen, 1954)

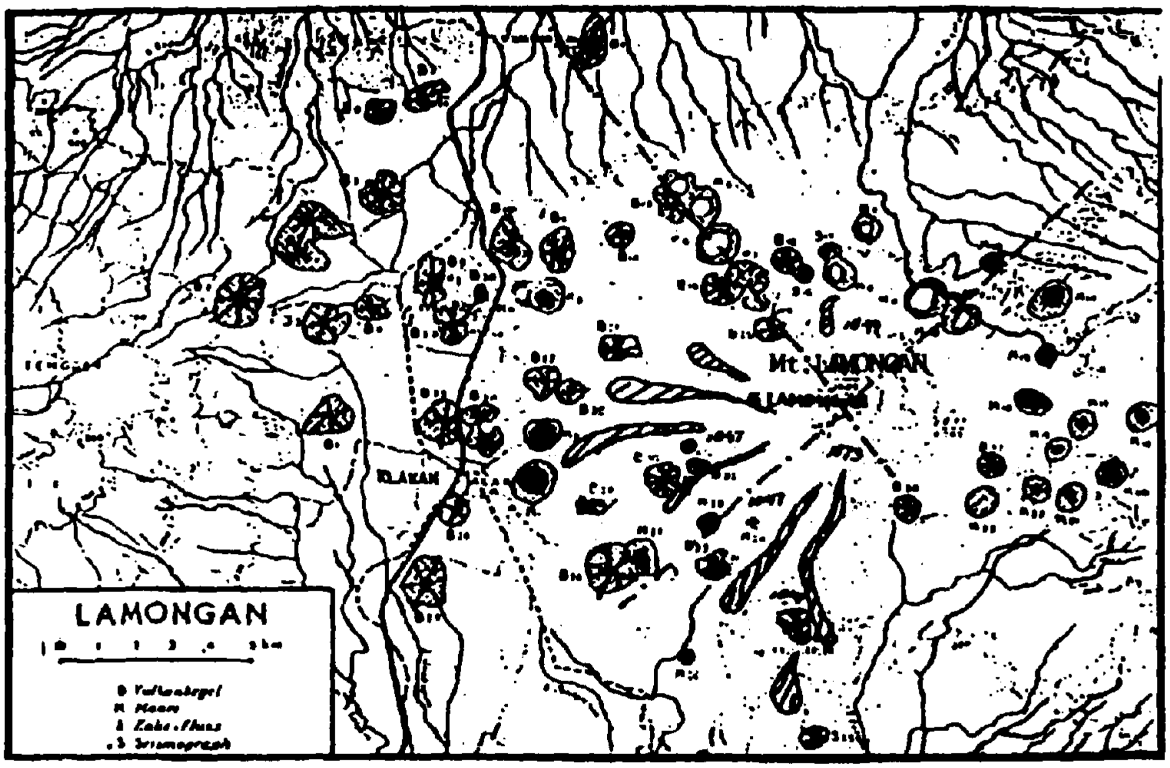

Figure 4. The distribution of "Maars" and cinder cones around the MT. Lamongan, East Java.

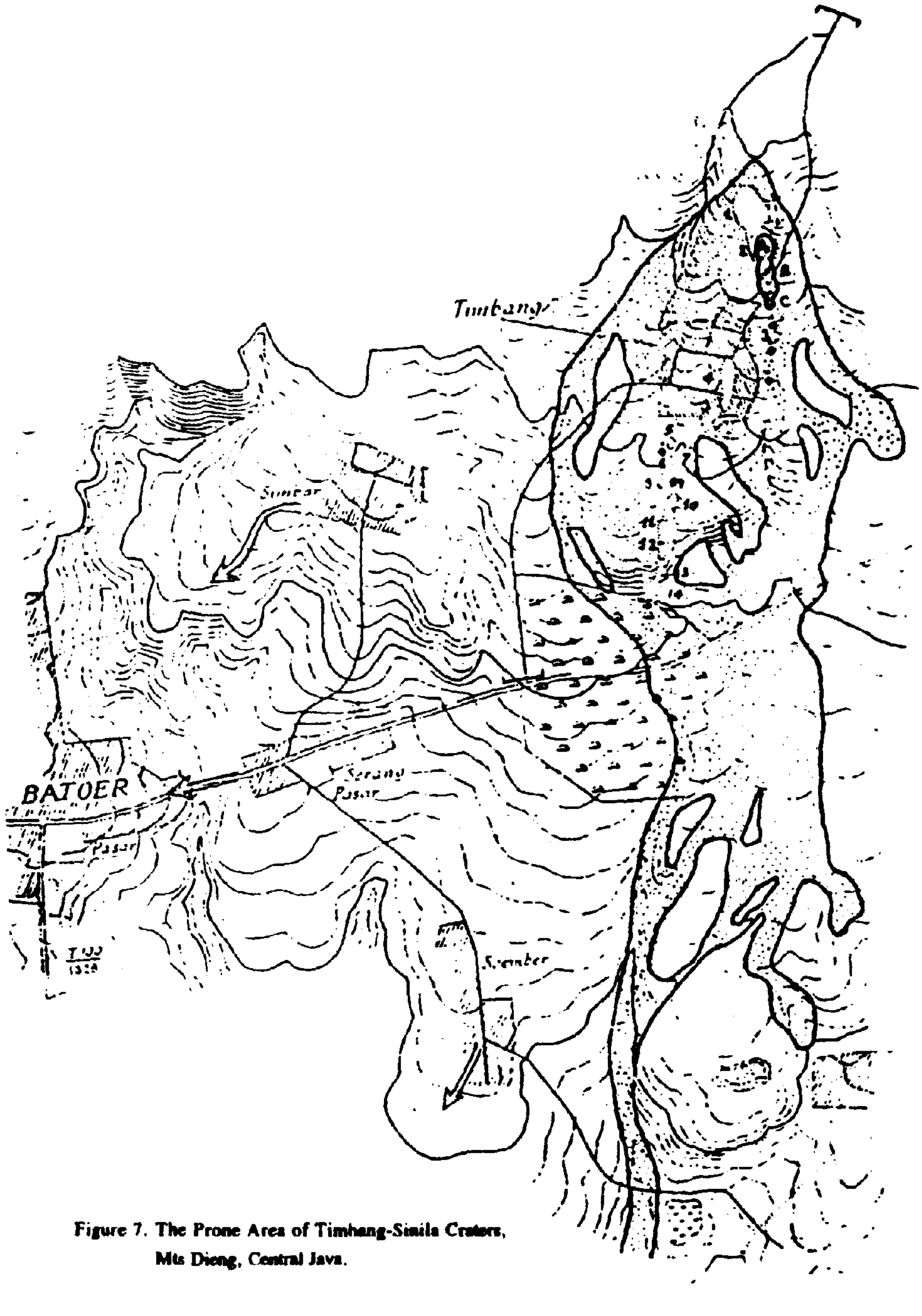

Figure 7. The Prone Area of Timbang-Sinila Craters, Mts Dieng, Central Java.

SECOND US-ASIA CONFERENCE ON
ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
Yogyakarta, Indonesia
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THE POTENTIAL HAZARD OF SECTOR COLLAPSE OF ALTERATION FROM MT. ILI LEWOTOLO, NTT

Asnawir Nasution
Volcanological Survey of Indonesia
Directorate General of Geology and Mineral Resources
Indonesia

ABSTRACT Some facts show that a sector collapse of volcanic flank is a potential hazard for the people who live in surounding of a volcano. It can be caused by lack of flank stability of active hydrothermal alteration. The alteration is now clearly shows by eastern peak and flank of Ili Lewotolo and as a weakness area. The volume of alteration materials (blocky lavas and piroclastic rocks) in a Lewotolo crater is about 400.000 cubic meters. These will become a potentia hazard if a collapse sector occurs in the future. The collapse can be trigerred by an earthquake, a crypto dome activity or a magmatic eruption.

INTRODUCTION

Mt. Ili Lewotolo is a quartenary active volcano of Sunda Zone, located to the north of Lomblen Island, eastern Flores (Fig. 1). It has erupted an extremaly wide range of lava and piroclastic materials which could be as a directed hazard to the people who live closed to the volcano.

Potential hazard from volcanic activity differs in some respects from one to another. It will drived from direct hazard (pyroclatic flow, tephra and lava flow) and indirect hazard (lahar and debris avalanche). By studying past history, present behaviour and the likely implication for its future behaviour, an hazardous of volcanic phenomena could probably be predicted.

Sector collapse does not always together with magmatic activity or intrusion (crypto dome), but it can be related to extensive hidrothermal alteration inside or on the flank of crater wall. It is characterized by no magmatic rocks in the deposits (mostly lithic fragment). Consecquently, the sector collapse of hydrothermal alteration probably due to by gravitation of weakness rocks by alteration processes (MacLeod, 1989).

Large slope failures may also occur indenpendently of this triggering mechanism, where stability is gradually lost through hydrothermal alteration and change pore water conditions (Hyde and Crandell, 1978).

The 1979 disastrous of Waiteba (southern part of Lembata Island) represented an alteration rocks as a caused lack of slope stability, produced massive avalanche to to the sea. Its effect yield tsunamy and killed more than 500 people (Sudradjat, 1979).

Based on the volume and altered rocks in the crater, distribution of the villages in the dangerous zone, the writer is trying to estimate the posible hazard of Ili Lewotolo volcano in the future.

THE GEOLOGICAL SETTING

Landsat and spot image interpretation of the Lembata Island represent the island geological view consist of Tertiary volcanics and sedimentary rocks, then covered by young volcanic products of lavas and piroclastic rocks of basaltic to dasitic composition. Two young active volcanoes are situated in the south island (Mt. Ili Werung) and in the north of Lembata Peninsula (Mt. Ili Lewotolo).

The Population

The factor which may lead to the disastrous or hazardous consequence in the future eruption or a tectonic effect is the increase of population and settlement location on the volcanic flank closed to the Ili Lewotolo eruption centre. They increase in a radius of 8 km from the crater (Fig.2). The people wholives in the hazard zone sorounding the volcano increases with the time. More than 7000 people settled, with their activities are agriculture and fishery.

Risk Assessment

Dangerous phenomena may rise from rock mechanical instability reached during the eruptive process. eg. the acumulation of large amount of deposits beyond their normal angle of rest under natural condition, such condition state which is quite easily revealed by external instabilities and which may cause lahar and/ or debris avalanches.

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ILI LEWOTOLO

Ili Lewotolo is an active volcano (1450 m above sea level) located on the north peninsula of Lomblen Island. It has two craters with 800 m and 75 m in diameters respectively. The big crater opens to the east, then the small one is located on the eastern part of big crater (Fig.3).

The flank of Ili Lewotolo is steep, especially on the north and eastern slope which have the angle ranges from 45-55^{0} (Fig.4) and extensive fumarole on eastern slope, while the western flank is relatively gentle. The upper part of the slope and the peak of Ili Lewotolo covered by pyroclastic fall.

The young lavas deposited in the big crater and spread out to the east flank of Ili Lewotolo then to the sea (Tokojaen and Lamawolo).They covered about 3/4 of the main crater, with 2-20 m thick and have the volume about 400.000 cu-meters. They also represent the dome like structur that situated on the eastern crater.

The Fumarole And Solphatar Activity

Ili Lewotolo emits a large permanent plume of gas from the summit area. Extensive solphatar and fumarole fields are present on the western flank of the cone structur inside the main crater and also along radially running fissures on the eastern-southern outer flank near the summit. Temperature measurement showed the maximum gas temperature on the hot vents closed to 490^{0} (Van Bergen et al.,1989). Strong hissing sound sometimes accompany the escaping of gas and a bluish light was visible from the vent.

The distribution of altered rocks from fumarole and solphatare activity spread out on the large areas. The rocks in summit area, inside the crater and the eastern steep crater wall are heavily altered, therefore the upper flank often eroded by gravitation and heavy rain water.

On the east flank down slope, the fumarole and hot water spread out in an area 300 x 500 sq. meters. The small hot spring has a temperature 40-43^{0}C, flow rate 0.3-0.4 l/sec with the sulphate deposit, silicification, pyritization and oxydation.

The Past Volcanic Activities

Past activities of Ili Lewotolo had been recorded by Neuman Van Padang (1951) and have periodically been monitored by Volcanological Survey of Indonesia. They have started since:

1660 : Central eruption

1819 : Central eruption

1849 : Central eruption

1852 :October 5 and 6. Central eruption destroyed sorounding area, then the small cone grew, followed by solphatare activity on the eastern flank.

1864 : Central eruption

1889 : Central eruption

1920 : Small explosion of the central vent

1939 : Increased activity

1951 : Increased fumarole activities to the eastern crater.

DISCUSSION

Two catastrophic events: Mt. Rainir (USA) during Holocene time (Crandell et al. 1979) and the 1772 Mt. Papandayan eruption (West Java, Indonesia) were massive avalanche of hydrothermal altered rock debris that produced mud flow or debris flow and lahar. These material could be a disastrous, if they extended into densely populated areas, eg. the 1772 Mt. Papandayan debris flow killed more than 3000 people (Van Padang, 1951).

The 1980 active crypto dome of Mt. Saint Hellen caused a sector collapse, producing debris avalanche volume 3 km^3, covering an area of 60 km^2 and moving about 25 km from the volcano (Voight et al., 1981; Crandell, 1984).

The collapse can be trigered by the changing of hydrothermal system, eg. the big tectonic earthquake. Some facts show that debris avalanche deposit of altered rocks do not show a new magmatic materials, as shown by the 1772 debris avalanche of Mt. Papandayan, probably trigered by a hydrothermal explosion or an earthquake.

The eastern cracks of young dome of Ili Lewotolo is one of the active vents, where the thick sulphur come out. The large rock alteration around the dome and eastern flank probably indicates one of the weakness and steep areas. It could be a potential hazard for future activity, if the intensity of the explosion or earthquake activity quite big enough.

The other factor that will affect rock weaknesses of Mt. Ili Lewotolo is probably ash, where hydrothermal process strongly attacked the old rocks or ashes, consequently they easily change to clay minerals. Therefore on a steep slope land slide will probably occur, because lack of slope stability.

The volume of materials inside the crater is big enough (400.000 cubic meters). It will be as heavy deposits to press and affect altered areas. They will weaken crater wall and slope stability. Therefore such masses could provide a source of large debris avalanche or mud flow if the eastern sector of the flank were disrupted by a crypto dome, a magmatic eruption or an earthquake, as shown on an example model (Fig.5).

If big debris avalanches were slide to the sea, they would cause a big wave or tsunamy which would be dangerous to settlements on the Bay of Lomblen Island.

HAZARD PREPAREDNESS EFFORTS

The posibilities to reduce victims from future hazard of Mt. Ili Lewotolo eruption; to evacuate the settlement areas inside the radius of 10 kilometers from the eruption point, to give information to the people who live in sorounding of volcano about volcanic activities and affects of the volcanic eruption and also to evacuate if an eruption is going to occur.

By studying the serious eruption of a volcano occured whitin recent time, pre-historic activities, geological mapping need to be done in order to find a figure kind of rocks and its distribution.

CONCLUSION

The possible hazard of Mt. Ili Lewotolo is debris avalanches or land slides, if an eruption or a big earthquake will occur in the future, based on the geological data, the altered rocks and flank stability. The direction of flow could probably to the eastern part.

Especially the ecentric location of the main centre of activity close to the rim of the crater, and the strong alteration of rocks may pose threats during future events higher (explosive) activity. It is feasible that the weakness of the altered part of the structure may lead to some kind of a sector collapse during more violent eruption.

Because there are many villages around the volcano, a further study is recomended to evaluate the need of a more permanent observation (including seismicity) near the volcano.

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Voight, B., H. Glicken, R.J. Janda, and P.M. Douglas, 1981, " Catastrophic rockslide avalanche of May 18," in Leipman P.W. and Mullineaux, D.R. eds., The 1980 Eruptions of Mount St. Helens, Washington : U.S. Geol. Survey Professional Paper, 250, p. 347 - 377.

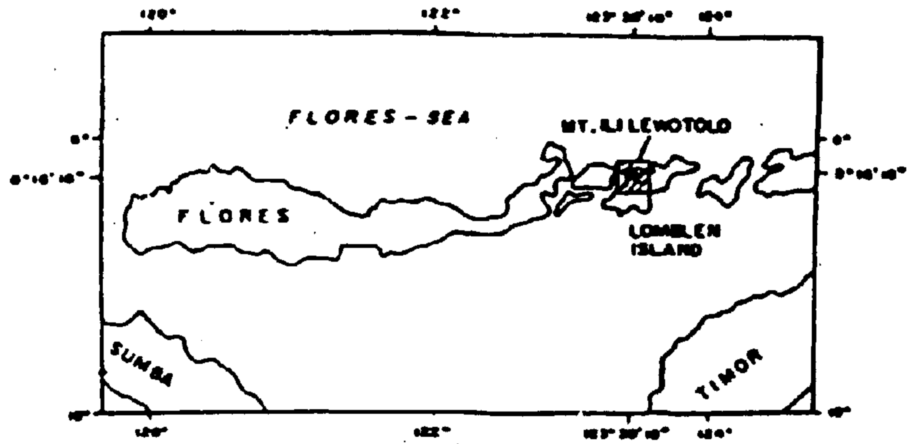

Fig. 1. Index map of Mt. Ili Lewotolo

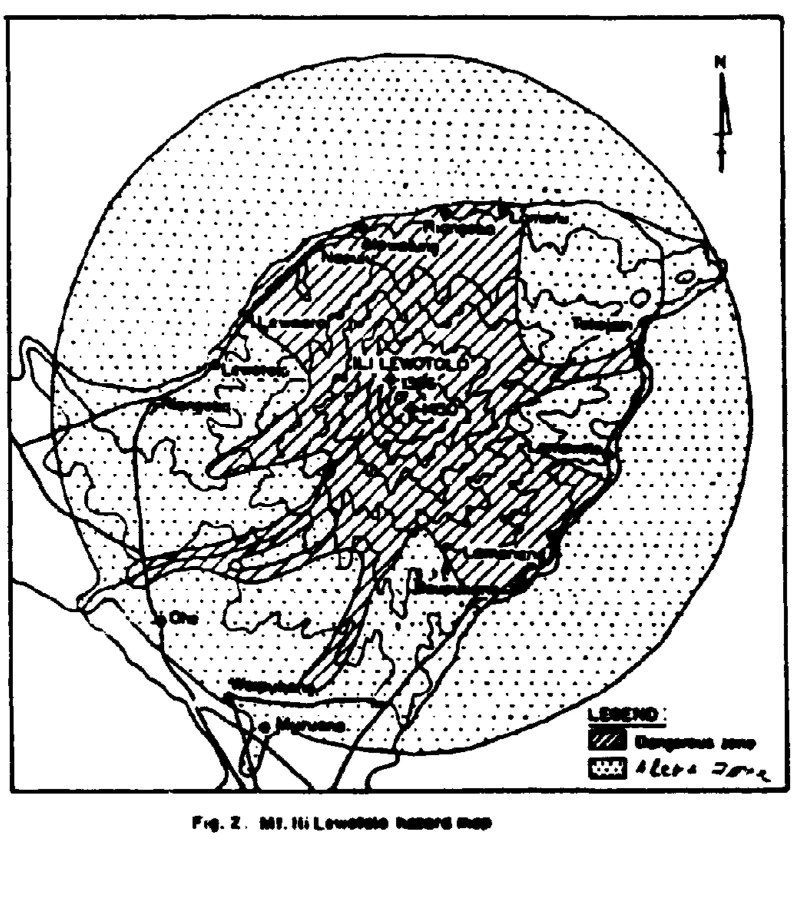

Fig. 2. Mt. Ili Lewotolo hazard map

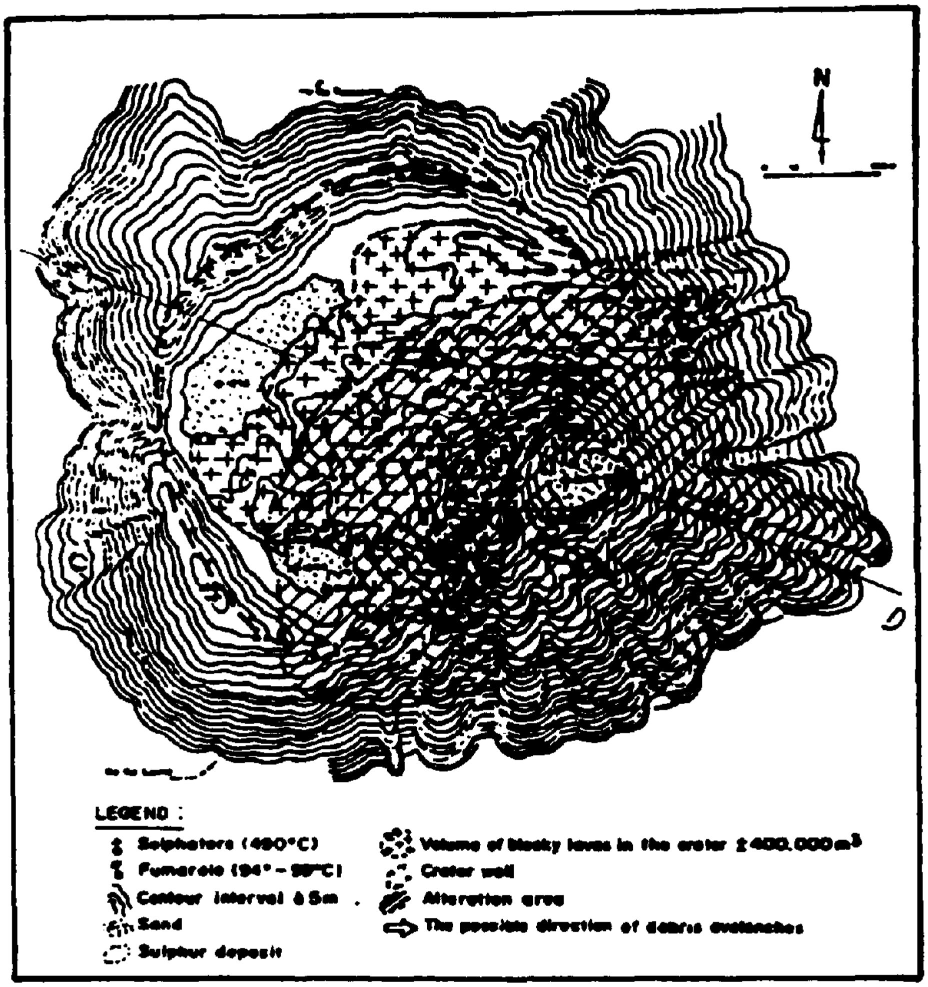

Fig. 3. The situasion map of Mt. Ili Lewotolo crater

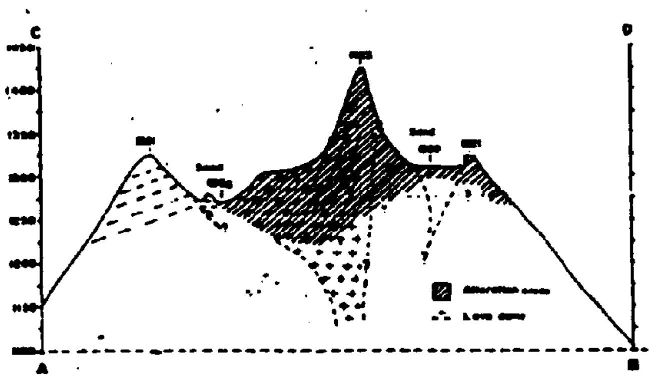

Fig. 4. The profile of Mt. Ili Lewotolo shows alteration areas

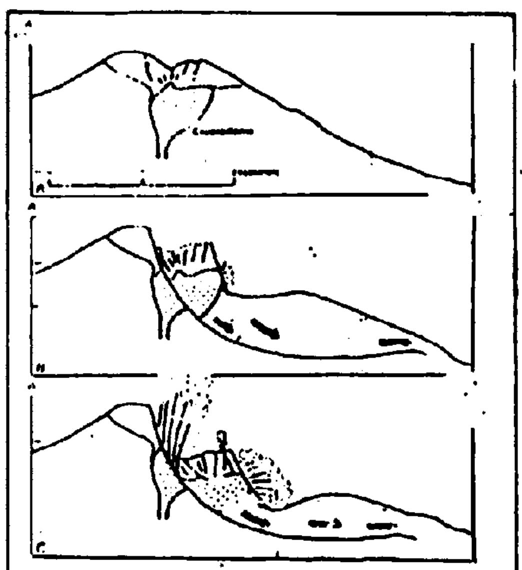

Fig. 5. Model debris avalanche processes

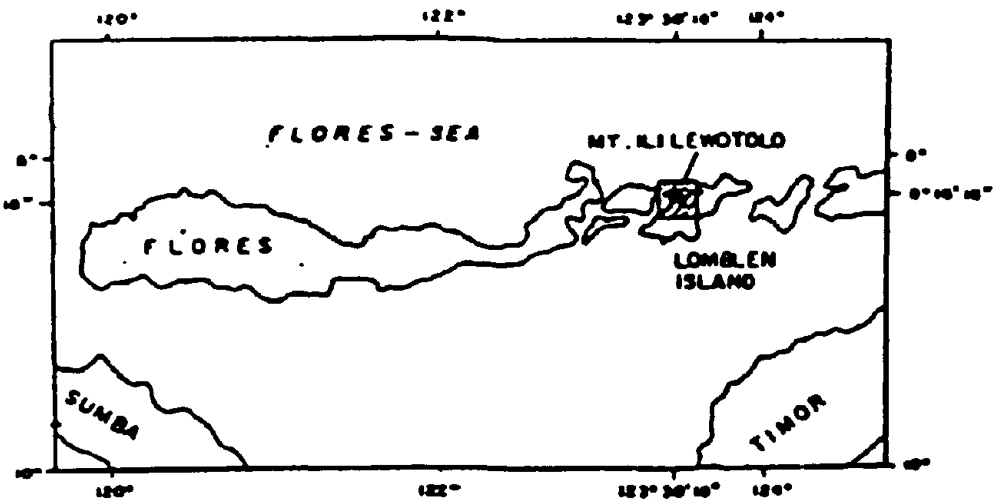

Fig. 1. Index map of Mt. Ili Lewotolo

V07

THE 1991 MT. PINATUBO ERUPTIONS: VOLCANIC HAZARDS AND IMPACTS

Raymundo S. Punongbayan
Philippine Institute of Volcanology and Seismology
Department of Science and Technology
6th Floor, Hizon Building
29 Quezon Avenue
Quezon City, Philippines

ABSTRACT

The June 1991 Mt. Pinatubo eruptions is considered as one of the biggest eruptions this century by world class standards. Its paroxysmal phase on 15 June 1991 produced an approximate volume of 7-11 cu. kms. of pyroclastic flow deposits and extensive air fall tephra, the thickest portions averaging 50 cms at 2-9 kms from the summit caldera. This caldera, formed during the 15 June 1991 eruption, measures about 1.5 kms and 300 m. deep. It is the site of numerous post-eruption ash ejections, the last on 02 September 1992. After almost a year from its June 1991 eruptions, volcanic activity have been mostly characterized by tectonic adjustments probably due to rocks adjusting within and around the vacuum caused by the removal of a big volume of magma beneath the volcano. However, equally threatening hazards are present and are still expected:

a. Lahars. During the 1991 rainy season, about 10-15% of pyroclastic flow deposits and most of the thick tephra fall deposits were washed down as lahars. Drainage systems where these flows occurred were: O'Donnell-Tarlac, Sacobia-Bamban, Abacan, Pasig-Potrero, Porac, Gumain, Marella-Sto. Tomas, Maloma, and the highly complex Bucau-Maraunot-Balin-Baquero river systems. Flows may have a highly erosive to channel-filling character while in the distal ends, may silt up or flood low lying areas. Our estimate is that the threat from lahars will continue until about 40% of the pyroclastic flow deposits have been washed out by annual precipitations. Worst-case scenario maps were prepared and distributed which delineated pyroclastic flow sources, areas already affected and/or buried and areas which will continue to be affected or at risk within the next several years.

b. Secondary Explosions. Temperatures of the very thick (maximum of 220 kms) pyroclastic flow deposits are expected to cool down within four to five years time. These deposits, when rained upon, caused secondary explosions whose heights could be as high as 10 kms and could cause light to heavy ashfall in nearby areas.

c. Volcano-tectonic Quakes. Tectonic adjustments are still occurring. Epicentral locations are along several areas around the volcano with maximum depths of 15 kms. Magnitudes vary from less than 1 to 4. The bigger and shallow ones are usually felt over a limited area.

d. Secondary Pyroclastic Flows. At least three significant secondary pyroclastic flows have been documented. These are when previously deposited pyroclastic flow deposits are remobilized by rainwater seeping into and generating the sliding block, occurrence of local and moderate magnitude earthquakes, or a combination of the two.

SECOND US-ASIA CONFERENCE ON
ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
Yogyakarta, Indonesia
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VOLCANIC HAZARD MITIGATION IN INDONESIA

Adjat Sudradjat
Directorate General of Geology and Mineral Resources
Department of Mines and Energy
Republic of Indonesia

ABSTRACT Volcanic hazard mitigation program in Indonesia constitutes volcanic monitoring, hazard map preparation, public education, engineering construction and public awareness. In the last 200 years, 175 thousand people were killed by volcanic eruption. The number has significantly decreased due to intensive implementation of the program. Six eruptions that occurred in the last 10 years claimed 38 lives in comparison with 5870 persons killed in the previous eruptions at the same volcanoes.

The advances in volcano monitoring technology have also been important contributors to the success of the volcanic hazard mitigation program.

INTRODUCTION

Indonesia harbours 129 active volcanoes that erupted in historical time or since around 400 years ago. The volcanoes distribute along the volcanic belt of Indonesia, 7000 km long and 100 km wide. In the average, one major eruption occurs every 3 years and mild eruption almost continuously.

In the last 200 years approximately 175,000 people were killed and hundred of thousand acres of arable land were destroyed by the volcanic eruption. The most recent catastrophic eruptions of plinian type occurred in 1815 in Tambora volcano, Sumbawa and in 1883 in Krakatau volcano, Sunda strait. The first eruption killed 80,000 persons and the latter claimed 36,000 lives.

MITIGATION PROGRAM

Volcanic hazard mitigation program in Indonesia consists of :

a. Volcano observation and monitoring of the activity, therefrom early warning might be issued. Several methods are applied in volcano monitoring which include seismic, tilting, deformation, temperature measurement, gravity, magneticity, self potential, SO_2 and CO_2 measurement, radioactive gases etc. Volcano monitoring by means of satellites was also introduced in Kelut and several others volcanoes using ARGOS platform.

b. Geologic mapping of the volcano in order to understand the past and present character of the volcano.

c. Preparation of hazard zoning map based on the past and present character of the volcano, topographic condition, prevailing wind direction etc. The hazard zoning map includes also the inventarization of land use and vegetation coverage as well as villages and population.

d. Public education through information dissemination, in particular to the people living in hazardous area. The activity is carried out under the cooperation with related agencies such as Ministry of Home Affairs, Ministry of Information etc. Exercises are regularly implemented.

e. Engineering construction to mitigate the secondary volcanic hazard, consists among others of the construction of sabo dam, dyke and check dam and the construction of tunnel. Sabo dams were built in Galunggung, Merapi, Semeru and Kelut volcanoes to mitigate the secondary hazard caused by lahar. A tunnel was drilled in 1920 in Kelut volcano to drain crater lake water which generates hot lahar.

f. Civil Defence for Natural Hazard Mitigation encompasses related agencies such as Ministries of Social Affairs, Health, Education, Mines and Energy, Public Works, Agriculture, Transmigration, Home Affairs, Defence, Red Cross etc.

The organization is chaired by Minister Coordinator for People's Welfare in national level, and at the lower levels is headed by Regent or Mayor coordinated by the related Governor as a member of the National Coordination Board for Disaster Management.

CASE STUDIES

In the last 10 years, 6 maıor eruptions have occurred in Indonesia. The intensive volcanic hazard mitigation program has proven to be effective. The following figures show the decreasing number of victims caused by volcanic eruptions.

Table 1 Number of casualties caused by volcanic eruption

Name of volcano	Year	Number of victims	Number of victims at prev. erup.	Year of previous eruption
1. Galunggung	1982	5	4011	1827
2. Colo	1983	None	Many	1898
3. Merapi	1984	None	1369	1930
4. Kie Besi	1987	None	300	1861
5. Banda Api	1988	1	Many	1890
6. Kelut	1990	32	210	1966
Total		38	5890	

a. Galunggung (1982)

Galunggung volcano is located in West Java, in a densely populated area. The volcano has been prepared with a volcanic hazard map. When the eruption occurred in 1982, the evacuation took place applying the zoning map. Approximately 35,000 people have been saved before the bigger eruption that generated glowing cloud occurred. The intensive monitoring was able to predict the individual eruptions that came later during eleven months of crises.

b. Colo volcano (1983)

Colo volcano is located in a volcano island. The volcano had been jolted by strong earthquakes almost continuously for 2 weeks before the eruption. The earthquake intensity increased as time progressed and culminated approximately 40 hours before eruption. A short quiescent period was recorded 4 hours before the paroxysm occurred. Nuee ardente of St. Vincent type swept over the entire island. Thousand of coconut palms and clove trees perished. The last evacuation group was moved out of the island approximately 10 hours before the paroxysm, whilst the volcanologist and observers left the island 6 hours later when the quiescent period was recorded at the seismograph. In total, 7,000 persons were saved.

c. Merapi (1984)

Merapi volcano located in Central Java, is the most active volcano in Indonesia. The lava dome development takes place almost continuously. At present 6,5 million cubic meters of lava dome is estimated. The dome frequently slides down causing avalanche which usually accompanied by nuee ardente d'avalanche (of Merapi type). The materials deposited by the avalanche generates lahar when mixed with rain water.

The intensive monitoring has revealed several phases of the eruption characteristized by different types of seismic records. The deep tectonic quake usually is followed by the shallower volcanic and multiphase quakes related to the lava dome development. The multiphase quake in general may lead to the prediction of an eruption. The 1984 eruption was preceded by sharp increase of multiphase quakes several hours before the eruption.

d. Kie Besi (1987)

Kie Besi volcano is located in an isolated island in eastern part of Indonesia. The volcanic eruption was predicted based on the seismicity that increased significantly before the eruption. Some 5,000 people were evacuated.

e. Banda Api (1988)

The observer of Banda Api volcano was warned by the increasing seismic activity 3 days before the eruption. When the intensity reached 200 counts a day, the local government was contacted to evacuate the inhabitants. Lava flows destroyed the villages and the arable land located at the foot of the volcano.

f. Kelut volcano (1990)

The most recent volcanic eruption occurred in Kelut volcano. This volcano was known to be very dangerous because of the eruption lahar generated by the crater lake water. A tunnel was drilled in the crater in 1920 to drain the water. The water volume decreased from 40 million cubic meters to around 2.5 million cubic meters.

Six months before the recent eruption water lake temperature had increased progressively from normal temperature of 32°C. The highest temperature 41°C was recorded a week before the eruption. The temperature remained at this level until eruption took place. The increasing temperature was accompanied by seismic crises 4 months and several days before the eruption. The high acidity of the water lake was also recorded.

Kelut volcano was monitored continuously by means of telemetric device and satellite platform. The latest warning was issued 5 hours before the eruption, while alert was first announced 4 months before the eruption. Intensive public education and exercises were held during the alert period.

CONCLUSIONS

Volcanic hazard mitigation program including volcano monitoring, volcanic hazard mapping and public education has proven to be very important in minimizing the volcanic danger. The positive interaction between scientist and administrators is the key element toward the success of the program. In the Decade for Reduction of Natural Hazard, it is called upon to give more priority to the program for the sake of saving human lives.

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V08-6

WIND HAZARD

W02-1

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RELIABILITY-BASED WIND-RESISTANT DESIGN OF TRANSMISSION TOWERS

H.N. Cho[1], J.C. Shin[2] and S.J. Lee[3]
1. Professor 3. Grad. Res. Asst.
Dept. of Civil Engineering
Hanyang University, Seoul, Korea
2. Assoc. Prof. Dept. of Civil Eng.
Chungnam University, Taejon, Korea

ABSTRACT This study is intended to develop probability-based design wind loads as well as reliability-based LRFD criteria for transmission towers based on the investigation of the safety levels of various types of towers designed by the current design practice in Korea. In the study, the AFOSM reliability method and an Importance Sampling Technique are used for the element and system reliability evaluation of actual transmission towers subjected to weather-related loadings. Based on the selected target reliabilities, a set of load and resistance factors for the LRFD criteria are calibrated using the AFOSM and the code optimization technique.

INTRODUCTION

In Korea, there has been a number of reported incidents on the collapse of transmission towers due to storm winds, which had caused serious socio-economical problems at the affected area. These may be attributed to the fact that the current design wind loads for transmission line structures are not relevant. As such, the current design wind loads and wind-resistant safety provisions for transmission line structures are not reliability-based and thus, in general, do not provide rational balanced design.

Recently, the author developed probabilistic design-wind speeds and risk -based wind map based on simulated typhoons and short-term records[Cho, 1987 ;

Cho and Baik, 1991]. In the last decade, the developments in reliability-based design codes and methodologies for reliability analysis of various structures have been well established. Last year, ASCE [1991] published a report on "Guidelines for Electrical Transmission Line Structural Loading", in which a rational approach for the design of transmission line structures is recommended in the form of reliability-based LRFD procedure for a systematic reliability-consistent design of local transmission line components, subsystems and systems. However, the ASCE approach may not be appropriate in small countries like Korea and those where the reliability concepts are not filtered through to design practice. This study is intended to show a rational but practical approach for probabilistic assessment of design wind and ice loads, and to suggest a practical reliability analysis of transmission towers and a simple calibration approach for reliability-based LRFD criteria for wind-resistant design of transmission towers.

PROBABILISTIC DESIGN MODEL

LRFD Format and Limit State Model

LRFD Format In the paper, the following simple LRFD format for two representative weather-related loading combinations is proposed as a wind-resistent design criterion for transmission line structures. This format is well suited for the calibration of LRFD criterion based on the AFOSM reliability analysis and the Turkstra's load combination rule.

$$\phi R_n \geq \gamma_M (\gamma_D D_n + \gamma_S \gamma_W W_n) \qquad (1)$$

$$\phi R_n \geq \gamma_M [\gamma_D D_n + \gamma_S (\gamma_{W_I} W_{In} + \gamma_I I_n)] \qquad (2)$$

where ϕ = strength reduction factor ; γ_D = dead load factor applied to D_n ; γ_W, γ_{W_I}, γ_I = load factors applied to W_n, W_I, I_n ; γ_M, γ_S = importance factors of member, structure or line ; R_n = nominal resistance ; D_n = nominal dead load effect ; W_n = nominal wind load effect ; I_n = nominal ice load effect ; W_{In} = nominal wind-on-ice load effect.

Limit State Model For the development of the reliability-based LRFD criteria for transmission towers, a linear strength limit state function in terms of random variables of resistance R and load effects ΣS_i ($g(\cdot) = R - \Sigma S_i$) may be used for yield and elastic/inelastic buckling failure modes of axially loaded members. And the resistance random variate R may be modelled as the product of the nominal resistance R_n and the correction factor N_R which is the random variate to adjust any bias and to incorporate the uncertainties involved in the assessment of R_n. The random variate of the ith load effect S_i in each load combination may also be expressed as the product of the nominal load effect, S_{ni}, and the corresponding correction factor N_{si} which is the random variate similar to N_R.

Probabilistic Design Load

Wind load The wind load model for the probabilistic description of the design wind is based on the current Korean standard [KEPCO, 1987]. The basic velocity pressure $q_o(kg/m^2)$ and design wind pressure $W(kg/m^2)$ acting on a component or structure may be written as,

$$q_o = \frac{1}{2} \rho (G V_{10})^2 \tag{3}$$

$$W = C q_o \alpha \beta K_1 K_2 \tag{4}$$

where ρ = air mass density ($kg \cdot s^2/m^4$) : G = gust factor : V_{10}= basic 10-minute wind speed measured at 10 m above ground level (m/sec) : C = force coefficient : α = height coefficient : β = span factor : K_1 = structural & material importance factor : K_2 = shielding coefficient.

In this study, the extreme wind speeds with the Type-I distribution are based on the previous study on the probabilistic assessment of design wind in Korea[Cho and Baik, 1991], but for the assessment of the statistics of maximum wind load, the 18 major sites which hold records of more than 20-yr long-term wind spead are selected. For the practical probabilistic approach, only V_{10}, C, α and G are treated as random variables. Based on the above model, the distribution of wind load is determined by a Monte Carlo Simulation in the same manner used by Ellingwood and others [1980].

Ice load The icing on transmission lines is a random event, and the nature of the ice formed and its amount and shape deposited on the lines are controlled by the physics of the storm and the topographical features of the location. The wide variations in the size, density, and shape of the ice formation cause an equally wide scatterness in the ice load effect imposed on the transmission line structures. The assessment of precise ice loading on transmission lines are extremely difficult or even impossible due to the lack of the data available and because of the extreme difficulty and complexity in the data assessment. Thus, in this study the 5-year short-term data which is obtained from a recent report [KEPCO, 1988] on the ice load on transmission lines in a heavy icing region in

Korea are used as the representative statistics of the ice load for cold-weathered regions in Korea. It has been found that the probability distributions of the ice load as well as those of the wind-on-ice load described below, which are also obtained by Monte Carlo Simulations, can be fitted very well by the Type- I distribution.

Wind-on-ice load In the absence of the statistical data on wind loads acting on iced conductors, it is suggested that the basic wind-on-ice speeds are approximated by 50% of the basic design wind speeds, which is the traditional practice in Korea, whereas the ASCE Manual [1991] proposed to use only 40% of them. The extreme data of wind-on-ice are extracted from the wind speed records of the sites located in heavy icing regions during the winter season.

RELIABILITY ASSESSMENT

Reliability Analysis

At element level, the reliability of transmission towers is evaluated by using the AFOSM algorithm. But at system level, an IST(Importance Sampling Technique) simulation algorithm developed by the author as well as a 2nd-order bound method is used for system reliability analysis of transmission towers. The system reliability model adopted for the reliability analysis of transmission towers is based on a FMA(Failure Mode Approach) formulation with a series model [Thoft-Christensen and Murotsu, 1986].

Reliability of Existing Transmission Towers

In Korea, geneal steel transmission towers except special kinds are usually designed as the standard types in most cases. Tangent(A), angle(B), strain(C) and dead-end tower(D) are four basic distinct types of the standard transmission towers. In this study, at first, reliability levels of various current codes such as the Korean standard-WSD[1987], the NESC-LFD[IEEE, 1990] and the ASCE-LRFD[ASCE, 1991] are investigated to make a comparative study for code calibration. It can be found that the reliability levels of the Korean standard and the NESC are highly fluctuated over various load ratios and shows $\beta \approx 1.5$ in the extreme wind case which is much lower compared to other building or bridge design codes. In contrast, those of the ASCE-LRFD vary to a great degree ranging from $\beta = 2$ to 4 in accordance with the line reliability factor, LRF and the component reliability factor, CRF.

For the investigation of the structural safety of existing towers, at first , the element reliabilities of each type of tower are calculated and summarized in Table 1. Considering the desired hierarchy of the reliability levels of each component and each type of transmission towers, it may be stated that the reliabilities of existing tangent towers designed by the current practice are much lower than desirable, as it can be seen that the lowest β_e is 1.63. And it can be also observed that the reliabilities of arm members are too high such that maximum β_e is 9. This wide range of β_e's in tangent towers indicates that some members are overly conservative while some other members are too much under-designed, which means in Korea the safety of existing towers in terms of balanced design is totally not relavant.

Table 1. Element and system reliability of transmission towers

Type	Element Reliability, β_e			System Reliability, β_s		
	Member	$D_n + W_n$	$D_n + W_l + I_n$	Tech.	D_n+W_n	$D_n+W_l+W_n$
A	MP	1.85 ~ 2.73	3.53 ~ 4.86	2nd O. Bound	0.62	3.01
	DM	1.63 ~ 8.98	3.97 ~ 9.08			
	AM	5.76 ~ 8.96	5.35 ~ 8.93	IST	0.78	3.37
B	MP	2.99 ~ 4.50	4.04 ~ 7.03	2nd O. Bound	2.24	3.74
	DM	2.63 ~ 6.64	4.05 ~ 9.09			
	AM	5.52 ~ 6.66	4.91 ~ 8.90	IST	2.50	3.90
C	MP	4.41 ~ 5.44	4.96 ~ 7.83	2nd O. Bound	3.60	4.73
	DM	3.71 ~ 9.00	5.32 ~ 9.09			
	AM	5.90 ~ 8.87	5.56 ~ 8.91	IST	3.65	4.88
D	MP	4.27 ~ 6.96	4.53 ~ 7.41	2nd O. Bound	3.35	3.87
	DM	3.49 ~ 7.69	3.92 ~ 9.07			
	AM	5.92 ~ 8.91	5.64 ~ 9.07	IST	3.40	3.88

Considering that major parts of towers, in general, have relatively low β_e and are framed in statically determinate way, series system modelling for each tower type may be used in practice. Estimated results are summarized in Table 1. Note that the system reliability of tangent tower is much lower than those of any other types. It is mainly because in the case of tangent towers the current practice yields under-design of main members and thus the component reliability results in much lower than that those of any other types when subjected to the probabilistic design wind load proposed in this study. It may be argued that the target reliability of tangent tower should be adjusted to higher one considering the fact that the reliabilities of tangent towers designed by the current practice are unreasonably low.

RELIABILITY-BASED LRFD CRITERIA

Calibration and Target Reliability

Calibration The LRFD criteria described in the paper is calibrated by a practical method so that the reliability are nearly uniform for all load situations. In order to achieve a consistent safety level near the target reliability index using the Level-I LRFD code, the resistance factor, ϕ and the load factors, γ_i, reapectively, are calibrated by the well known code optimization procedure[Ellingwood and others, 1980].

Target Reliability No established procedure for the rational selection of target reliability indices, however, is available so far, although a number of different approaches have been reported in the LSD or LRFD code development for various kinds of structures. The practical approach suggested in this study is, therefore, based on the concept of the desired hierarchy of safety level along with the engineering judgement and experiences as well as foreign practices together with the trade-off between theory and practice. The selected target reliabilities based on the results of the investigation of the safety level of the current code and the reliabilities of existing transmission towers are summarized in Table 2.

Table 2. Target reliability

Type	Member	
	MP DM	AM
A	2.0	2.5
B	2.5	3.0
C, D	3.0	3.5

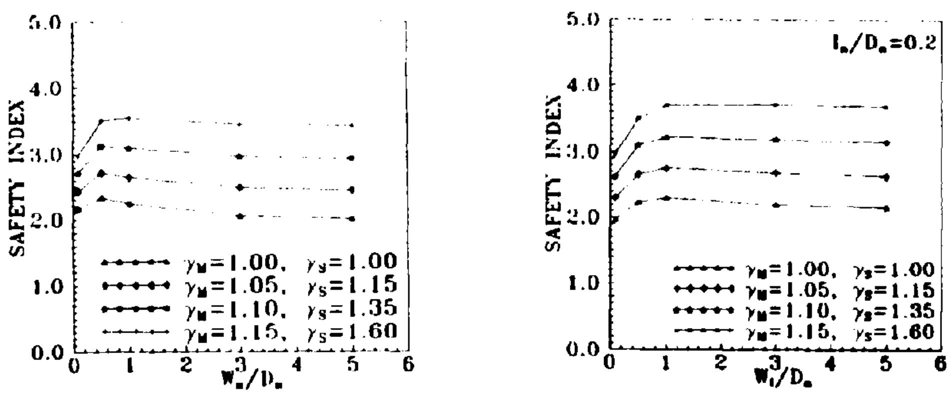
Table 3. Proposed LRFD criteria for transmission towers

Type : member	LRFD Eq. ◦ $0.85R_n=\gamma_M(D_n+1.2\gamma_S W_n)$ ◦ $0.85R_n=\gamma_M[D_n+\gamma_S(W_I+0.6I_n)]$	
	γ_M	γ_S
A : MP, DM	1.00	1.00
A : AM B : MP, DM	1.05	1.15
B : AM C, D : MP, DM	1.10	1.35
C, D : AM	1.15	1.60

Results and Discussions

Calibration results Table 3 shows the results of the optimum ϕ, γ_i corresponding to various target reliabilities for different types of members and towers. It has to be noted that the basic resistance factor ϕ_i(=0.85) for compression member fixed as the representative value is selected by considering the results of the AFOSM calculations and that of the AISC-LRFD specification, but the dead load factor γ_D(= 1.0) is selected as the truncated value based on the results of the calibration by considering the fact that there will be virtually no change in the dead load of transmission towers during the service life. And then, based on the selected ϕ_I, γ_d, the corresponding calibrated load factors γ_W, γ_{W_I}, γ_I as shown in Table 3 are determined from the code optimization technique.

Discussion In order to check the consistency of reliability of the proposed LRFD criteria in Table 3, the reliability indices are plotted for each target reliability, as shown in Fig. 1. As expected, it may be noted that the reliabilities are almost constant to the variation of load ratio except in the range where the load ratio is less than one. Therefore, the proposed LRFD design criteria can be successfully used for the balanced wind-resistant design of transmission towers in practice.



(a) D_n + W_n (b) D_n + W_I + I_n

Fig.1 Reliability variation of the proposed LRFD criteria for transmission towers : β vs. load ratio

CONCLUSIONS

In the paper, practical probability-based design loads and reliability -based LRFD criteria for wind-resistant design of transmission towers in Korea are developed on the basis of the probabilistic assessment of statistical load data together with the reliability assessment of existing towers using the AFOSM and an IST/FMA system reliability analysis.

It may be concluded that the proposed reliability-based LRFD criteria can achieve reliabilty-consistant and wind-resistant balanced design of all standard types of transmission towers in Korea. In addition, it has to be pointed out that the reliability of tangent towers designed and erected by the current Korean standard are too low and diverse compared to other types, and thus the safety factors for tangent towers should be upgraded to have consistent target reliability for the wind-resistant design of future transmission lines.

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ON THE QUESTION OF THE ROLE OF BUILDING CODES AND STANDARDS IN MITIGATING DAMAGE DUE TO HIGH WINDS

Dale C. Perry[1] and W.L. Beason[2]
1. Dept. of Architecture, College of Architecture
2. Dept. of Civil Engineering, College of Engineering
Texas A&M University

ABSTRACT Although our basic understanding of the nature of wind and its effects on buildings and structures has improved dramatically during the past three decades, the translation of this knowledge into codes of practice continues to present a formidable task. The aftermath's following the passage of recent hurricanes in the United States have served as reminders that we are not doing enough to address the wind threat. Wind damage in the U.S. on an annual basis now exceeds that induced by all other natural hazards.

Following each major wind event, the following statement is frequently heard:

"The wind climate was predictable and most of the damage was preventable."

What then, are we doing wrong? The answer is not a simple one, but is rooted in the complex manner in which building codes are promulgated, adopted and enforced. The issue of affordability vs. risk is always center stage and the political/economic systems in place in some jurisdictions do not always permit the adoption of proper strategies for mitigating damage.

The objective of this paper is to:

- review current practices in the United States with regard to the wind threat,
- discuss the adequacy of wind load provisions currently in place to mitigate damage, and
- suggest what new measures should be adopted to reduce wind damage to acceptable levels while at the same time safeguarding the economy.

INTRODUCTION

Many areas in the United States are vulnerable to extreme weather, with hurricanes, tornadoes, severe thunderstorms, and downslope winds exacting high tolls in human lives and property damage. Wind-related losses currently account for over $2 billion annually. An average of 96 lives are lost each year due to tornadoes alone. The insurance industry spent over $22.5 billion in wind-related losses during the period 1980-1989. Hurricane Hugo, which made landfall near Charleston, SC in 1989, produced $4 billion in insurance losses, approximately $10 billion in total losses, accounted for over 100 deaths, and disrupted the lives of millions. Hurricane Frederic (1979), Alicia (1983), and Elena (1985) exacted lesser, yet unacceptable levels of losses of lives and property damage (Sparks 1992). Of interest here is the fact that each of these events involved surface-level wind speeds at, or below, the design levels prescribed by the governing codes. This begs the question: "Are our building codes at fault or is it a question of adoption and enforcement by the responsible jurisdictions?" Certainly there exists no shortage of technical information regarding wind loads and the wind resistance of the vast majority of structural systems and cladding employed in high wind regions -- or is new knowledge needed to mitigate damage?

The answer to reducing wind-related damage is to be found by addressing the following inter-related factors:

- The increasing wind vulnerability of the populace
- Building code issues
  - legitimacy of process
  - technical context
  - economic and social factors
- Public awareness
- Adoption and enforcement of codes
- Alternatives to code adoption

Each of these issues is briefly discussed below and recommendations are made to affect a decrease in wind damage.

W03-3

WIND VULNERABILITY

Vulnerability to the wind hazard in the United States continues to increase. Changing demographics, increasing capital outlays for buildings and lifelines, and deteriorating infrastructure systems are the primary factors.

The east coast, gulf of Mexico and the islands of Hawaii are experiencing substantial increases in population density as more and more people move to the hurricane-prone areas and are potentially in harms way. To cite just a few examples -- the coastal county population of Texas has increased from 190,000 in 1900 to over 4,300,000 in 1990. Less than 44 thousand lived in Galveston when the 1900 Great Galveston Hurricane took over 6000 lives. The Galveston population currently exceeds 220 thousand. The population of Worcester County, Maryland (contains Ocean City - a major resort area) has experienced a ten fold increase in the same time frame. Similar increases are occurring at many other locations including Padre Island, Texas; Gulf Shores, Alabama; Panama City, Florida; the outer boundaries of North Carolina and northland through New England (Sheets, 1992). Barrier islands are particularly at risk during the tourist season when the population increases by a factor of 10 to 100 fold.

Another important factor concerns the aging of our population. The Bureau of Census estimates that by the year 2030, 22 percent of the population will be 65 or older, this represents a two-fold increase over 1987 (CND report, 1992). Millions of these people will retire in the coastal areas of the sunbelt states and many will live in manufactured housing that has proven to be extremely vulnerable to the wind hazard. Additionally, Sheets (1992) estimates that 80%-90% of the 44 million people now living in the coastal areas have never experienced a major wind event. The result is that many share a false impression of the threat to life and damage potential of these storms.

Adding to the wind-hazard potential is the increasing deterioration of the highways, bridges and overpasses that are essential to emergency evacuation during an extreme wind event. Nationally, county governments project their infrastructure needs to be over $18 billion annually (CND, 1992). However, actual funding for infrastructure maintenance and improvement continues to decline across the United States.

W03-4

BUILDING CODE ISSUES

Introduction

Building codes and standards exist to provide for the health, safety and welfare of the citizenry. In developing these standards, however, concern must also be given to the need to safeguard the economy. Otherwise major changes in the laws, rules and regulations governing the construction process may give rise to significant socio-economic problems within given communities or the country as a whole. In a perfect world, architects, engineers, constructors and other technical disciplines would enjoy complete freedom to apply their talents and judgments without the constraints of the myriad of building codes, referenced standards and other regulations imposed by federal, state, county, and city agencies. The presumption would be that responsible professionals, acting collectively, possess the required technical knowledge and responsibility to adequately safeguard the populace. Unfortunately, this presumption fails for a number of reasons as alluded to below.

Colwell and Kau (1982) suggest that the system that produces building codes and standards in the U.S. is so structured that "it does more mischief than good." They argue that there is no body of experience to indicate building codes add to health and safety in any way and that the cost of enforcement and compliance are growing more rapidly than the total cost of government. Sparks (1990a), in reviewing the adequacy of one of the model building codes in mitigating wind damage, claims that the code process suffers from "an overdose of democracy." Perry (1991) has suggested that the building code process in the United States can best be characterized as quasi-democratic in which all of the players exhibit some form of proprietary interest. Newly adopted code provisions frequently demonstrate only that the free enterprise system is alive and well, but they do not always reflect proper concern for the safety and welfare of society.

A pattern has emerged wherein there appears to be an ever increasing disparity between "what we know" and "what we do." Figure 1 depicts the hurricane resistance of structures if built in accordance with performance and prescriptive requirements set forth in building codes as compared with that of "as built" as a function of time period. More will be said about this later. Colwell and Kau (1982) go much further in stating: "the system of building codes and standards has been substantially diverted from the goal of protecting the public health and safety to serve the purposes of special interest groups." The Wind Panel of the Committee on National Disasters (CND, 1992), on the otherhand, recognizes that building codes and standards play an important role in the mitigation of wind damage. The report finds, however, that improvements are needed and attention must be given to their adoption and enforcement coupled with economically feasible land-use management practice.

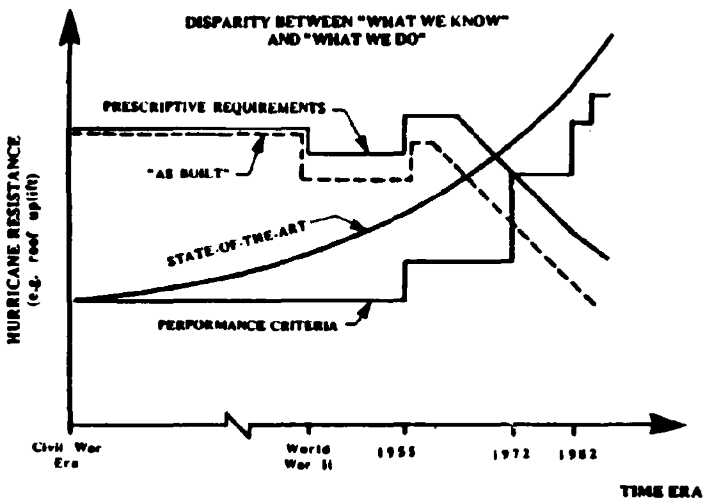

Figure 1. Possible Evolution of Hurricane Resistance With Time

The objective of the remainder of this section is to briefly review the processes by which building codes and standards are developed and address the issues which limit their efficiency in mitigating wind damage.

<u>Code and Standard Development</u>

The system followed in the United States to provide for the safety and serviceability of buildings and structures is somewhat unique. In most other industrialized countries, the national government oversees the regulatory development and enforcement processes. This results in a single "National Code." In this country, however, the development of building codes and standards has become, for the most part, private-sector enterprises involving a wide spectrum of federal, state, city, and local regulatory bodies, professional societies, trade associations and manufacturers, builders and developers, and the academic community.

It has been estimated (Perry 1987) that approximately 5000 building codes are in use in the United States at the present time. This figure is somewhat misleading, however, as Todd (1992) states that there may be up to 40,000 state, county, city and local jurisdictions that adopt and/or enforce codes. The wind load provisions adopted and enacted into law by the appropriate regulatory bodies are influenced by the provisions developed by nearly 500 organizations in the United States who write and maintain national standards as depicted in Figure 2.

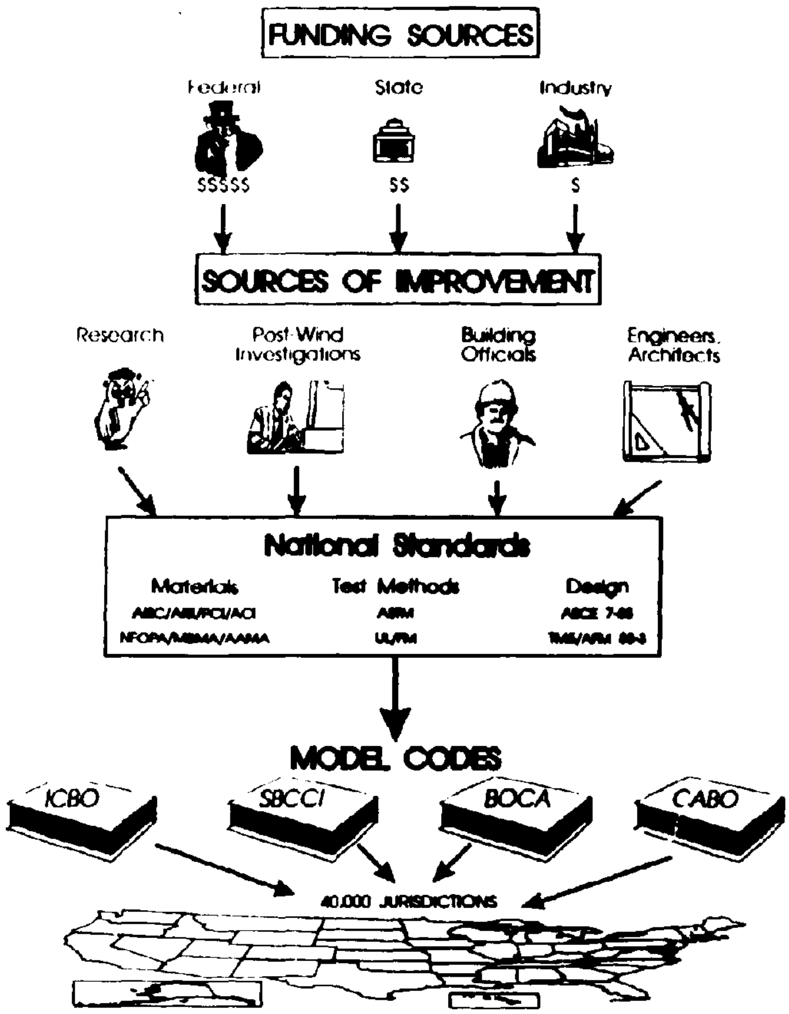

Figure 2. Linkages Between Organizations and Documents that Comprise Building - Code Community

The documents relative to providing proper wind resistance can be broadly classified as follows:

- The National Standard
  - ASCE 7-88 (formerly ANSI A58.1), Minimum Design Loads for Buildings and Other Structures (1988), the only truly "consensus document."
- Model Building Codes
  - Standard Building Code (1991) promulgated by the Southern Building Code Congress International (SBCCI).
  - Uniform Building Code (1991) promulgated by the International Conference of Building Officials (ICBO).
  - National Building Code (1990) promulgated by the Building Officials and Code Administrators International, Inc. (BOCA).
- Umbrella Codes
  - CABO One and Two Family Dwelling Code (1992) promulgated by the Council of American Building Officials.
  - CABO Manufactured Home Construction and Safety Standards (1987) promulgated by the Council of American Building Officials.
- Department of Defense Technical Manuals
  - Design Manual NAVFAC DM-2 (1970) developed by the Department of the Navy, Naval Facilities Engineering Command.
  - Load Assumptions for Buildings, Technical Manual TM5-809-1/AFM 88-3, Chapter 1 (1986) developed by the Departments of the Army and the Air Force.
- Industrial Standards
  - Low Rise Building System Manual developed by the metal building industry (MBMA, 1986).
  - AAMA Publications.
- Material Specifications
  - AISC, AISI
  - ACI, PCI
  - NF_0PA
  - ASTM, ASHRAE
- Testing Laboratories
  - Underwriters Laboratories, Inc. (UL)
  - Factory Mutual Engineering Corporation (FM)

Additionally, many of the largest cities (e.g., Chicago, New York, Boston, San Francisco, etc.) and some states (North Carolina, South Florida, and New York) promulgate and enforce their own codes. Disney World (EPCOT) also falls into this category although its wind load provisions are based on the 1973 version of the Standard Building Code (with 1974 revisions). Another notable exception is California State Title 24 governing the design of school buildings and hospitals.

Fortunately, the picture is not quite as disorganized as it might appear. A study conducted by an umbrella organization, the Council of American Building Officials (CABO), indicates that approximately 85 percent of all state and local governments have either directly adopted one of the three model codes or patterned their regulations based on the provisions of these documents. The key word here is "patterned." Additionally, each of the model codes designate the ASCE 7-88 National Standard as an acceptable alternate procedure for assessing wind design loads. The geographic regions of influence of each of the model codes are depicted in Figure 3. Note that there are many overlaps, a few state mixtures and many pockets of "local codes."

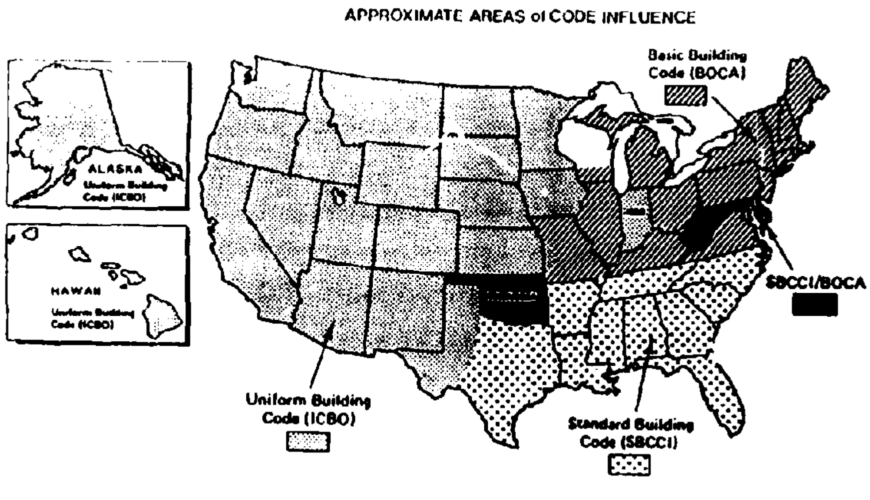

Figure 3. Geographical Influence of Model Codes
(After Perry, 1986)

It is important to note that the wind provisions set forth in the model building codes, the ASCE 7-88 document, and the CABO housing codes are only representations of possible regulations and do not become law until enacted by the authority having jurisdiction (state, county, city, etc.). Thus, these documents are usually modified (sometimes substantially) to satisfy local laws and ordinances and to reflect local building practices and political climates. As an example of the above, Manning (1987) notes: in the eleven coastal states and two territories that enforce state or territory-wide codes, seven different codes have been adopted and four of these states have elected to adopt different design wind speeds than those set forth in the codes they adopted. Add to this the number of local governments in the twelve coastal states that do not mandate a state-wide code, and local jurisdictions who do not adopt a code at all, and one can readily appreciate the variations in building requirements along the United States coastlines. It is also worthy of note that some jurisdictions tend to be "slow" in adopting new revisions. Thus, the codes of practice tend to lag far behind the state-of-art, by 15 to 20 years on an average.

Issues In Code/Standard Processes

Prior to entering into a discussion of the actual building code processes currently in place, it will be useful to focus on the specific issues involved. These may be broadly grouped into the following categories:

- Legitimacy of Process
  - degree of consensus
  - means for updating
  - influence of special interest groups
- Technical Content
  - adequacy/accuracy
  - performance vs. prescriptive (deemed-to-comply) criteria
  - material specifications/standards
  - product compliance (evaluation) reports
- Enforcement and Compliance
  - number and competency of inspectors
  - costs of enforcement
  - cost/benefit analyses
- Economic and Social Effects
  - affordability vs. risk
  - costs of code development and enforcement

In what follows, a discussion of how well the established processes followed by the three model code groups (SBCCI, ICBO, BOCA) and CABO lend themselves to addressing these issues. As noted earlier, the codes promulgated by these organizations account for the large majority of regulations adopted nationwide. It will be seen that the consensus process followed in developing the national standard (ASCE 7-88) cannot be strictly adhered to by the model codes for a variety of reasons, both political and economic. There are, however, distinct advantages to having the promulgation of code provisions in the hands of the model code organizations, perhaps the most notable being that code changes are possible on a yearly cycle.

Model Code Process

It will be seen expedient to first discuss the nature of the provisions and types of documents produced by the model codes, and second, to discuss the cast of players involved as the latter strongly influences the legitimacy and technical content of the documents produced.

In general, the publications of the model code organizations can be grouped into three categories:

- Building Codes/Standards
  - performance criteria
  - prescriptive (deemed-to-comply) requirements
- Material Specifications
  - incorporated by reference, or
  - reprinted by permission
- Product Compliance (evaluation) Reports

A PERFORMANCE CODE specifies the loads, material strengths and design processes. Material specifications are either referenced or included as part of the document. Registered engineers and architects then apply those provisions using technical judgment and experience to develop a design that can safely resist the prescribed loadings without collapse and excessive damage under extreme conditions. The design must also meet appropriate serviceability requirements (e.g., acceptable displacements) under lesser loads. Buildings and structures designed in accordance with a performance code normally receive a great deal of engineering attention and have been termed by Minor, et al (1979) as "fully engineered" as contrasted with "marginally engineered," "pre-engineered" and "non-engineered."

A PRESCRIPTIVE (or deemed-to-comply) CODE specifies the actual sizes and spacing of members, types of connectors and other structural details for achieving a specific level of performance (e.g., roof uplift resistance for 100 mph design wind speed). If the requirements are followed to the letter, a structure built in accordance with this code is **DEEMED-TO-COMPLY** (D-T-C) with the performance criteria. No further engineering attention is needed. Prescriptive provisions are based on empirical procedures which have evolved over a large number of years based on intuition and experience, tempered by economic factors. The methods and techniques are embodied in trade practices passed on from one generation of builders to the next. Industry and manufacturers are normally the "heavy hitters" in developing these provisions; and, as such, proprietary interests tend to dominate activity. Building officials prefer prescriptive requirements over performance criteria as they believe they minimize liability, involve little in the way of professional judgment, and require little in the way of professional training. Of late, both SBCCI and ICBO have funded their own committees to develop D-T-C provisions in an attempt to encourage the various industries to work together and produce more legitimate documents by minimizing special interest influences.

PRODUCT COMPLIANCE (EVALUATION) REPORTS are issued by each of the model building codes for the purpose of evaluating new products. The intent is to certify that a material, building component or perhaps a complete building system meets specific **performance** criteria. Examples include the flexural resistance of a particular type of metal door, the strength of fasteners, or the ability of some type of generic building system to meet the wind load provisions. As will be shown subsequently, much of the wind-related damage is due to the poor resistance of the components enclosing the building. Too often, roofing systems and other vunerable "non-structural" components are selected on the basis of color, texture, fire resistance, length of manufacturer's warranty, or cost. Little analysis is given to the products' ability to resist the wind and water loads which can be reasonably expected to be experienced during an extreme wind event. Sparks (1992) comments: "Why is it then that a country that could put a man on the moon in 1969, can't keep the roof on a house nearly a quarter of a century later?" The model code staff in attempting to evaluate a particular product is faced with the problem of having to translate the test results provided by testing laboratories into satisfaction of specific performance criteria. Adding to the complexities is the fact that the testing procedure frequently in no way replicates the time history of loading produced by a natural event such as the wind.

Cast of Players

The cast of players involved in the code process have been grouped into three categories (Perry, 1991):

- End Users
  - building code officials
  - engineers, architects, constructors
- Shakers and Movers
  - trade associations and manufacturers
  - professional societies
  - federal agencies
  - fire services
  - model code staffs
- Passive Participants
  - academic community
  - insurance interests
  - builders/developers
  - John Doe public

Perry (1991) noted that the actual involvement of the players varies widely for each of the model codes, and for some participants, depends almost entirely on the issues addressed during a given code cycle. His perceptions of the positive and negative influences of each in furthering the safety and welfare of society are given. Space does not permit a detailed discussion here, but reference is made to Table 1 in which it is noted that building officials account for 67% of the participation in model code activities. It has been suggested that less than 50% of the building officials possess university degrees in appropriate technical disciplines, much less, professional registration. This constituency comprises the only voting members in the code process.

Table 1: MODEL CODES PARTICIPATION AT ANNUAL PUBLIC HEARINGS*
(approximate average)

Category	Participation
Building officials	67%
Trade associations and manufacturers	17
Fire services	5
Engineers, architects, and code consultants	4
Federal agencies	1
Miscellaneous	6
	100%

* After Barris (1982)

Thus, the model code groups are basically **BUILDING OFFICIAL ASSOCIATIONS**. Public testimony on new advances in the state-of-the-art, adverse experiences regarding building performance and other forms of substantiation to support proposed code changes are given on a voluntary basis. In the final analysis, however, the building officials collectively serve as both jurors and judges in adjudicating proposed changes. Although it is argued that this process does have some "consensus-orientation," It can more aptly be described as quasi-democratic: quasi, as the privileges of the participants have been divided between those accorded voting rights and those relegated to advisory roles. This factor alone calls into question the legitimacy of the process.

Perry (1991) suggested the principal strengths and weaknesses of the model code process are as follows:

- **Primary Strengths**
  - Anyone can attend the public hearings and present evidence
  - There exists a mechanism for making amendments on an annual basis with a new edition produced every three years.
  - Members in the construction industry have an organization to direct and have answered promptly, their questions regarding specific code provisions.
  - Small manufacturers have the opportunity to have their products evaluated through product compliance (evaluation) reports
- **Primary Weaknesses**
  - The mechanism for advancing new code changes is not a consensus process
  - Although anyone can play, there are no monies to support the participation of individuals who do not align themselves with special interest groups
  - Voting members are restricted to building code officials, the majority of whom do not have the technical competency to critically evaluate all proposed changes.
  - Difficulties involved with enforcement may influence decisions by building officials as to proposed changes
  - Changes in code provisions which may significantly influence the economic health of a given community are feared and frequently rejected.

<u>Adequacy of Code Provisions</u>

To address the adequacy of the various documents produced by the model codes in mitigating wind-related damage, it is convenient to classify buildings into the following categories (Minor et al 1979):

- **Fully Engineered**
    - Buildings that receive specific, individualized design attention from professional architects and engineers. Usually the design is site specific. Examples include high-rise office and hotel buildings, hospitals, and public buildings. Most structures in this category are designed in accordance with performance criteria, while the building envelope (cladding, roofing materials, glazing, doors, etc.) selected on the basis of prescriptive requirements or product evaluation reports.
- **Pre-engineered**
    - Buildings that receive engineering attention in advance of a commitment to construction and are subsequently marketed in similar units. Examples include metal buildings and manufactured housing units such as mobile homes. Pre-engineered metal buildings are, for the most part, currently designed on the basis of performance criteria, but each building may not receive site-specific attention. The major weakness of this class of buildings is that certain critical components relative to wind resistance (doors, windows, foundations, etc.) may not be specified by the original manufacturer as the industry operates in a design-build mode (Perry, 1989). Pre-engineered manufactured housing (mobile homes) are designed in accordance with CABO's Manufacturer Housing Standard (1987). The loads specified are much lower than those set by most building codes.
- **Marginally Engineered**
    - Buildings built with some combination of masonry, light steel framing, open-web steel joists, wood framing, and wood rafters in which portions of the building receive engineering attention while others do not. Examples may include motels, commercial, and light industrial buildings. Design is based primarily on prescriptive requirements.
- **Non-Engineered**
    - Buildings that receive no specific engineering attention. Examples include most single and multi-family residences, most one- or two-story apartment units, and some small commercial buildings. Non-engineered structures are designed, for the most part, on the basis of prescriptive requirements contained in model codes and/or CABO's One- and Two-Family Dwelling Code (1992).

Numerous papers have been published calling into question the adequacy of building codes/standards in addressing the wind threat. Colwell and Kau (1982), e.g., argue that there is no body of experience to indicate building codes add to the health and safety. Fortunately, beginning with Hurricane Camille (1969), post-disaster teams have conducted extensive investigations following major hurricanes. Reports containing wind speed-damage correlations and assessments of the adequacy of the governing codes of record may be found for Hurricanes Camille (Marshall et al 1970), Frederic (Mehta et al 1983), Iwa (Chiu et al 1983), Elena (Sparks et al 1991a) and Hugo (Sparks et al 1991b). Numerous other post-disaster reports of damage caused by tornadoes and straight winds are also available. A review of the documents provides a clear indication of the strengths and weaknesses of the various code documents as they existed at the time of the event

PERFORMANCE CRITERIA set forth in the recent editions of the model codes (SBCCI, ICBO, BOCA) and the national standard ASCE 7-88 have been shown to provide adequate levels of safety insofar as main wind-force resisting systems are concerned. Most of the damage to fully engineered buildings has been sustained by the cladding. The current cladding loads set forth in the performance criteria appear reasonable, but unfortunately, much of the cladding comprising the building envelope is designed on the basis of empirical or prescriptive requirements. Experience suggests cladding requirements are, for the most part, inadequate and need revision.

PRESCRIPTIVE REQUIREMENTS of the model codes used for the design of marginally engineered buildings have been shown in many cases to be grossly inadequate (Sparks 1989, Sparks 1990a, Sparks 1990b, Sparks et al 1991). The masonry and wood industry must bear much of the responsibility for yielding to pressures from the market place and not providing proper requirements for the high wind regions of the country. The national standard 530/ASCE 5-88, **Building Code Requirements for Masonry Structures**, has been shown to be deficient for high wind speeds but is referenced by all model codes (Sparks 1989, Sparks 1990). On the positive side, the SOUTH FLORIDA BUILDING CODE (1957) and the NORTH CAROLINA RESIDENTIAL BUILDING CODE (1984) provide examples of adequate prescriptive requirements for non-engineered structures of wood and masonry. The use of the regulations set forth in these documents has been seen to significantly reduce the risk of wind damage (Sparks 1989).

HOUSING accounts for the highest percentage of wind-related property damage and loss of lives. This fact tends to support the statement made by Walker and Eaton (1983):

> **"Basically society has considered that housing does not warrant engineering analysis and design."**

Nonetheless, communities along the hurricane-prone coastlines have generally believed that the adoption of a code (any code) would protect them from the damaging effects of hurricanes. Until recently, insurers felt it unnecessary to impose further restrictions or even make inspections. Housing was considered to be an acceptable risk.

Most conventional wood-framed houses, with the exception of those sited in South Florida and North Carolina, are built in accordance with the CABO One and Two Family Dwelling Code (CABO, 1992). The prescriptive requirements of this code have evolved over decades based on experience and observed performance under gravity loads (erection, snow). Experience has shown, however, that houses "built to code" sustained maximum damage when subjected to high winds (Sparks 1989). The weak links have been identified as inadequate tie-down of the roof, inadequate connections securing the wall panels to the foundation and lack of racking resistance to lateral loading. Threshold damage to homes begins to occur at sustained speeds of 50-60 mph (roofing materials) and low pitched roofs may lose their entire structural integrity at gust speeds as low as 70 mph. It is important to note that one reason housing does not experience a higher damage level is that most homes today are sited in high density subdivisions and thus the bulk of the residences are sheltered and do not receive the high wind loads to which those on the perimeter are exposed.

MOBILE HOMES (manufactured housing) currently account for 50% of the new single-family homes purchased each year. First-home buyers and retirees satisfy the "American Dream" in this manner. Although certification of compliance with the Manufactured Home Construction and Safety Standards (CABO 1987) has been required by HUD since 1976, the wind loads specified are much lower than those set by most building codes for conventional housing in the same area. Requirements regarding properly designed anchoring systems are a local responsibility and may not be enforced. The damage threshold for mobile homes is less than a sustained wind of 70 mph (Minimum basic design wind speed set forth in ASCE 7-88 for the contiguous U.S.). Additionally, flying debris produced by mobile homes during wind storms may compromise other buildings and structures down stream.

The National Severe Storms Forecast Center provides statistics to indicate 37% of all tornado fatalities involve persons who are either in or fleeing from a mobile home residence. A policy enforcing evacuation of all mobile home dwellers under a major hurricane warning advisory appears advisable. Mandatory wind resistant shelters sited in mobile home parks should also be considered.

PUBLIC AWARENESS

Historically, the development of building codes has closely paralleled public outcry to unsatisfactory building performance attributed to unregulated construction. Unfortunately, however, the general public has virtually no knowledge of the building code process. Only in the aftermath of major disasters do they voice concerns and call for improvements in the codes of practice. Even then, their concerns are quickly forgotten outside of the immediate region affected.

The recurrence period of severe storms in any region is so long that economic considerations are more powerful than the bitter lessons--thus, disasters are repeated. Homes damaged by extreme winds are generally rebuilt to the same standards. One would think that the residents of Dauphin Island, Alabama, for example, would have learned the lessons of severe winds (Camille 1969, Frederic 1983, Elena 1985) - but reconstruction following Elena was not substantially more wind resistant than that following Camille (Sparks et al 1991a). Flesner (1992) raises the question: "if political and economic considerations permit code circumvention after a hurricane, how big a challenge do we have with code compliance in areas not experiencing a hurricane in recent years?"

Part of the problem may have to do with the misrepresentation of wind speeds by the news media. The public has come to believe that damage induced by hurricanes and tornadoes has been due to winds of incredible magnitude and as such, fall into the category of "Acts of God" They do not know that the vast majority of hurricanes and reported tornadoes (Mehta et al 1979) involve wind speeds at, or less than, design wind speeds specified by local codes. A notable example of disparity between reported speeds and those measured at ground level is found in a series of articles contained in *National Geographic* (1980). Wind speeds for Hurricane Allen (1980) are given as 215 mph, Camille (1969) at 155 mph, David (1975) at 175 mph and the Labor Day Storm of 1935 at 200 mph. To the authors' credit, some attempts were made to classify these speeds, where possible, as "peak gusts", "sustained", or "measured from reconnaissance aircraft." Unfortunately, these identifiers are all too frequently lost in translation. Perhaps the most flagrant example of recent history is the reporting of wind speeds for Hurricane Gloria which threatened the east coast for four days in 1985. As can be seen from Fig. 4 (after Powell and Black

1985), the disparity between wind speeds reported by the media and probable landfall surface speeds was as much as 100-150% . The probable landfall surface speeds shown are based on the "80% rule" (surface level wind speeds are roughly 80% of the values reported by reconnaissance aircraft penetrating the storm at elevations of 5-10 thousand feet, Bates 1977, Georgiou 1985).

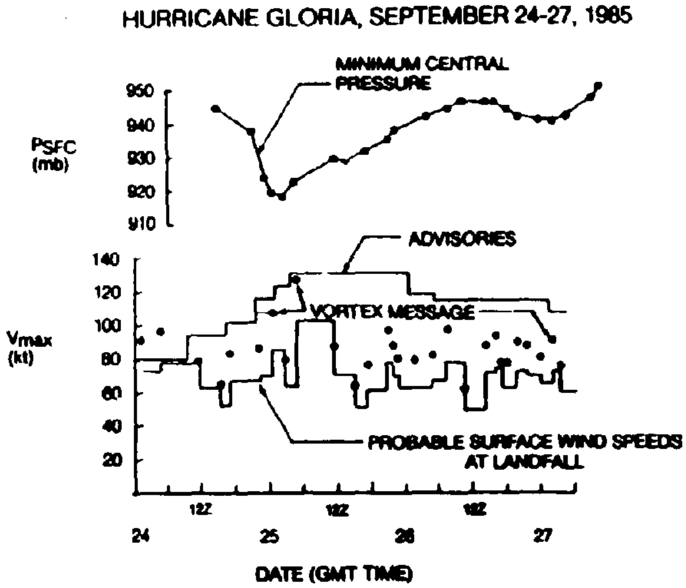

Figure 4. Comparison of NWS Advisories and Probable Surface Wind Speeds at Landfall as Gloria Progressed Northward Along Atlantic Coastline (after Powell and Black, 1989)

The consequences of overstating surface wind speeds have serious implications and could result in the following (Marshall 1992):

- Misplaced confidence in design and construction practices (for the structures which suffered no damage),
- Inadequate building code provisions left in place, and
- The stage is set for future disasters

In 1986, the Florida Legislature passed a law increasing the coastal design wind speed from 100 mph to 140 mph. Note that this would have almost double the wind speeds. It was subsequently rescinded. Interestingly, this same legislative body in 1989 passed a bill (CS/HB 1057) mandating that one and two family dwelling need not be designed for wind. Other attempts to require higher wind speeds have been introduced from time to time, but have been beaten back by the wind engineering community. The problem is that a higher wind speed is not required to mitigate damage but improvements in the codes are needed coupled with increased enforcement. If conscientious engineers were to heed the call for higher design wind speeds, the end result would be gross overdesign and needless waste of private and public resources.

Perhaps the current state of public apathy can be summed up by a statement made in a report by the Committee on Natural Disasters (CND-1992):

> **"The United States lacks the political will to develop long-term goals and objectives to deal effectively with wind-hazard issues. The threat from extreme winds is real and dramatic for all Atlantic and Gulf coast states, for Hawaii, Puerto Rico, and Virgin Islands, and for inland states as well. The nation's apparent indifference to this threat is astonishing and perplexing. . . . . . Advocates must arise from within the affected communities, and they must be augmented by a strong voice from the professional community that addresses wind hazard issues."**

At present, the federal government provides less than $1 million annually for research to mitigate the wind threat. States and industry have historically provided only paltry sums of monies (Figure 2).

ADOPTION AND ENFORCEMENT

In a previous section it was noted that wind load provisions set forth in a given code or standard do not become law until enacted by the authority have jurisdiction (national, state, county, city, etc.). Therein lies one of the basic weaknesses in the mitigation strategy. The Federal Government sets construction standards only when it has a financial interest in the property, or when construction is part of interstate commerce. Some individual states mandate state-wide codes, others leave the adoption and enforcement entirely to local jurisdictions. Many jurisdictions, even along the hurricane-prone coastlines, do not mandate any requirements (Sparks 1990).

Additionally, some political entities may circumvent code compliance to meet economic considerations. As an example, in many regions the construction of school buildings fall under the jurisdiction of State Departments of Education. Apart from compliance with appropriate fire regulations, the construction is required to meet only very minimal requirements (Sparks 1990). Note that these same buildings are frequently used as evacuation centers. Hurricanes Elena (Sparks et al 1991a) and Hugo (Sparks 1991b) and the East Coldenham School Building tragedy in Newburgh, New York in 1989 point out the seriousness of exempting school buildings from meeting acceptable building standards.

Ideally, building code enforcement should involve two steps:

1) Plans are submitted to a building department and approved, or approval delayed until modifications are made, and
2) On-site inspections are conducted during construction to ensure compliance with the approved plans.

The quality of enforcement varies widely across the US. depending on the commitment of the elected officials, political considerations, the salary level offered to personnel, and the number and qualifications of authorized personnel. A recent study conducted by SBCCI under the auspices of State Farm Insurance (Manning 1991), suggested that only 2 of the 12 jurisdictions surveyed along the Atlantic and Gulf Coasts were in compliance with the wind load provisions of the codes. More alarming is the fact that only 30% of the individuals taking the building inspector and plan review examinations received passing scores.

ALTERNATIVES TO CODE ADOPTION AND ENFORCEMENT

Colwell and Kau (1982) suggest that an important step to improving health and safety would be for the insurance industry to establish a set of standards of its own. The owner of a building could then select a standard to which his or her building would be built and pay the appropriate premium. Given the complexities of developing and promulgating building codes and standards, this approach may appear, at first, overly simplistic. Nevertheless, we may be heading in this direction.

The insurance industry has begun to recognize the adverse potential of the wind threat on their industry (AIRAC 1986, 1989). The National Committee on Property Insurance (NCOPI) is currently moving into a more active role in code and standard development. Insurance underwriters are supporting research in this area. Flesner (1992) suggests that history bears out the fact that if insurance coverage is not available in the voluntary market, eventually political pressure or regulatory pressure will mandate that coverage be provided. That is why Coastal Wind Pools have came into existence. The Texas Catastrophic Property Insurance Association (TCPIA) and New York Property Insurance Underwriters Association (NYPIA) are noteworthy examples. One of the questions to be asked is whether the entire state (or the country as a whole) should subsidize coastal residences who are at high risk that has happened in Texas where legislation mandates coastal and inland residents pay the same insurance premium.

The National Flood Insurance Program (NFIP), established by the Federal Government in the early 1970's, has been very successful in mitigating the risk of damage due to storm surge, wave action and flooding. The country may well need to consider a similar approach if the wind hazard is to be properly addressed.

CONCLUSIONS

Perhaps the most powerful and direct method available to mitigate wind-related damage is the adoption and enforcement of building codes and related standards. The performance codes currently in place have been found to provide an adequate level of safety when the provisions are properly applied by design professionals. Prescriptive requirements, on the other hand, require major attention. The performance of un-reinforced masonry and wood-framed buildings during extreme wind events has proven to be inadequate. Housing, both conventional and manufactured (mobile homes), should be the focus of a national agenda to improve performance while looking after the question of affordability. Cladding systems and materials have not received proper engineering analysis and account for much of the wind-induced damage.

RECOMMENDATIONS

Model Code Organizations

If the model codes are going to continue in the way they do business, the code development process will continue to be overly influenced by special interest groups. The following recommendations are offered to improve the end product:

- Appoint outside professionals (architects, engineers) to the Code Revision Committee and provide them with voting privileges.

- Encourage the participation of outside professional groups to monitor the code process. The past activities of SEAOC and the other Western States Structural Engineers Association working with ICBO staff are particularly noteworthy in this regard.
- Encourage the development and continual updating of D-T-C documents similar to those recently advanced by SBCCI and ICBO. Purge the publication lists of inadequate standards.

Adoption and Enforcement

A strong, national political base is needed to address the issues of code implementation and enforcement. This responsibility must be shared by federal, state, and local governments with private industry, professional societies, the insurance industry, and the general public. It is suggested that:

- Research is needed to address the socio-economic issues relative to code adoption and enforcement. As political will must be developed to deal effectively with wind-hazard issues.
- Extensive cost-benefit analyses must be properly structured and conducted to demonstrate the benefits of code implementation and enforcement.
- The national standards and model code associations should continue to encourage the insurance industry to become important participants in the code process.

Education

A coordinated effort is urgently needed to transfer wind engineering technology to design professionals, trade associations and manufacturers, building officials, the insurance industry, and the general public. Perhaps the most important target should be the participants of the model code processes and the appropriate individuals at the federal, state, and city levels who are responsible for legislation to provide for the health, safety and welfare of the populace.

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INTEGRATED APPROACH AGAINST CYCLONIC WIND HAZARD AND ROLE OF VOLUNTARY AGENCIES

K. Seetharamulu[1] and V.R. Sharma[2]
1. Principal, Regional Engg. College, Warangal
2. Research Scholar
Civil Engineering Department, IIT Delhi

ABSTRACT The natural disasters in India are mainly caused by earthquakes, floods and severe winds. In India the damage caused by tropical storms is very high as compared to tornadoes, local stroms and thunder storms. Herein an integrated approach is directed againts the hazards caused by cyclonic winds. The cyclones on the eastern coast of India occur in the months of April and May, and between October and December. The cyclones originated in the Bay of Bengal affect the coastal belts of Tamil Nadu, Andhra Pradesh, Orissa and West Bengal, whereas, those originating in Arabian sea affect Konkan and Saurashtra coast. The Cyclones on eastg coast are amongst the most destructive of all the natural disasters. These cause destruction to man made structures meant for housing and shelters. It is not feasible to economically design the housing and community structures that are fully resistant to cyclones and therefore safe against loss of life and damage to material assets. There are three categories of stgructures, (a) Storage structures for safe storage of household goods, (b) Community structures and shelters for livestock and local population and (c) Large span shopping and distribution complexes away from the cyclone affected areas. The structures belonging to categories (b) and (c) may be designed to safely resist the cyclones. Such a system with a proper warning system may result in practically fully safe against loss of life and damage to material assets. It is therefore equally important, if not more, to mobilize assisstance from voluntary agencies alongside the affected population and the government agencies for mitigating the hazards. The other measures may include (a) dykes and sea walls and (b) Shelter belts and Land use zoning. Herein, an integrated approach for cyclone disaster is discussed.

INTRODUCTION

The natural disasters are mainly caused by earthquakes, floods and severe winds. In India frequency and intensity of tropical stroms is very high as compared to tornadoes, local storms and thunderstorms. Herein, an integrated approach is directed against the hazards caused by the cyclonic winds.

Tropical cyclones originate between 5 and 30 degree latitudes on either side of the equator and affect islands and coastal regions in India occur in the months of April and May, and between October and December. Coastal belts of Tamil Nadu and West Bengal are affected by cyclones originated in the Bay of Bengal, whereas, those originating in Arabian sea affect Konkan and Saurashtra coast. The cyclones on east coast are amongst the most destructive of all the natural disasters like earhquakes and floods.

In India, tropical cyclones on east coast are frequent and cause destruction to man made structures menat for housing and shelter. The loss of lives and livestock, damage to material assets and crops are heavy. It is not feasible and economical to design the housing and community structures that are fully resistant and therefore safe aginst loss of life and damage to material assets. It is equally important, if not more, to mobilize assistance from voluntary agencies alongside the effected the effected people and the government agencies for mitigating the hazards. The role of the voluntary agencies is an integral component of the total strategy. Herein, an integrated approach for cyclone disaster is discussed.

TROPICAL CYCLONES

The diameters of the cyclones are usually of the order of several hundred kilometers. The depth of the atmosphere involved is about ten kilometers. In India, the average radius of "eye" is about 20 to 30 kilometers, but it can reach 40 to 50 kilometers in large mature storms. The temperature is the highest, whereas pressure is the lowest in this region with either clear or partly cloudy sky. Based on the measurements in the Indian subcontinent (1) the "eye" is surrounded by strong winds extending up to 30 to 50 kilometers beyond the centre. Beyond the 'wall cloud' region winds spiral in counter-clockwise direction in the northern hemisphere and extend outwards to a large distance with decreasing speed. In northern hemiphere the rotation of wind in vertical direction up to 7-8 kilometers is counter-clockwise, whereas beyond that it is clockwise up to 12-14 kilometers (1). A schematic diagram is shown in Fig. 1. (2).

The wind speed of 1977 cyclone that struck Andhra Pradesh were estimated to be as high as 269 kmph and most of the damage caused was due to inadequacies of the warning system and lack of mobilization of voluntary agencies and lack of awareness of the consequences on the part of people. The cyclone of 1990 was not any less severe but timely warnings to the affected people and mobilization of various agencies reduced significantly the human loss.

Tropical cyclones are accompanied by strom surge which could be quite high. Based on the previous data available, 'Probable Maximum Storm Surge' pertaining to India and Bangladesh heve been computed by mathetmatical modelling Gosh (1983). The computed values are shown in Fig. 2 (1). For India, maximum computed value is 12.5 at Contai, Orissa. The rise of sea level due to tide can be up to 4.5 meters above mean sea level. Storm surge and tide also create havoc with human life, cattle and other material assets. Most destructive cyclones during the last 100 years which hit east coast of India are listed in Table 1 (3).

TABLE 1

No	Place	Year	No. of People Dead
1.	Contai (Orissa)	1864	100,000
2.	Masulipatnam (AP)	1864	40,000
3.	Midnapur (WB)	1942	75,000
4.	Chirala (AP)	1977	10,000

Most of the lost is caused by coasstal inundadtion by storm surge which penetrated about 20 kilometers inland from the coast. Death and destruction purely due to winds are relatively small except in the zones over which eye of the cyclone moves. Heavy rains and floods combined with high winds close to the eye of cyclone result in damages due to structural collapse of buildings, falling of trees, electrocution etc. Disease from the contaminated water and flood constitute post cyclone disaster resulting in further loss of human lives.

MEASURES FOR DISASTER MITIGATION

It is not economical to design the structures to be fully resistant against this natural disaster specially in the path of 'eye' of cyclone. However, by taking long term structural and non structural measures the fury of the disaster can be reduced to a considerable extent. Structural measures like construction of cyclone shelter, embankments, dykes, reservoirs, and coastal afforestation are some of the long term mitigation measures against cyclonic disasters. Persuading people not to inhabit in major cyclone disaster prone areas, establishment of development projects away from cyclone prone areas, insurance cover, and proper education about the cyclone warning systems and disaster mitigation measures, are amongst the non-structural measures. Short term preparedness measures constitute timely warnings, effective rescue, relief and rehabilitation, at the time of disaster.

1. Structural Measures

Low Rise Residential Buildings

Assessment of pressure distribution due to wind on low rise buildings is important as it helps in arriving at wind resistant design and construction. Fig. 3 shows pressure distribution on low rise buildings due to wind (5). With the knowledge of exact behaviour of a structure subjected to wind, it can be made safe by proper design detailing and construction techniques. Proper roof to wall connection; wall to wall joints; adequate fastening between covering and roof frames; stability of partition walls; sound foundation and its proper connection to walls contribute significantly towards reducing the damage. Typical recommendation are (2) :

i. Height to width ratio should preferably be one and lenght ratio not to exceed 1.5
ii. For isolated dwellings circular plans should be preferred.
iii. Cement/lime mortar should be used in brick construction instead of mud mortar, and walls should be plastered on both sides.
iv. The linkages between foundations and walls/columns, wall and wall, walls and roof should be perfect.
v. Fastening between covering and roof frames shoul be adequate.
vi. Doors and window openings should be designed in such a way that they don't become free entrances for surge/flood water.

Some recomended details of connections of various components in a structure are shown in Fig. 5.

Large Span Shelters for Dwellings

An economical design system for membrane type large span shelters to provide community housing for the weaker section in cyclone prone areas can be evolved. These large span shelters can also be used as storage as well as shopping/market areas. For a relatively more permanent type of systems roofing consisting of shell modules resting on space frames may be preferred overtension membrane systems. The basic purpose of these shelters is to provide a large membrane covering, capable of resisting cyclonic force with minimum cost, Fig. (5). The interior can be used effectively by weaker section for community dwellings using indigeneous materials, which otherwise in the absence of shelters fail to withstand nature's fury.

On receiving cyclone warnings people generally leave their belongings in their dwellings and rush to cyclone shelters for protection. The belongings are likely to get damages and lost. It is difficult to recover their belongings and resume the normal work immediately on return to their dwellings. Ferrocement attic units, Fig. 6, developed by S.E.R.C. Madras, can be used for storing the belongings at the time of cyclone. These core units can be locked and anchored to the floor of the proposed shelters. This arrangement will prevent damage/loss of the belongings and people can immediately resume their professional work on return to the dwelling after passing off of the cyclone.

EXPERIMENTAL OBSERVATIONS AND RESULTS

Mean Winds

Thre cup generation anemometer "PRICOL" model WMG 100 fitted at the top of the mast on the roof of a three storey building at a height of 19.2 m abobe ground level is used for velocity measurements. The response time of the anemometer being 15 sec., output can be taken as 15sec. mean wind speed. The wind speed was recorded on the mast erected on the roof of a three storey building at a height of 19.2 m above ground level. The wind speed was recorded at different hours. The observations recorded in time domain were converted into frequency domain by using (a) Graphical Method and (b) Discrete Fourier Transform Method (9). The result obtained from the two methods have been found to be in agreement. The mean wind spectra has been obtained for different periods. One such spectra is shown in Fig. 7. The trend observed in all the cases is similiar. The highest peak of the spectra is noticed corresponding to a period of one day. The second highest peak corresponds to a time period of twelve hours. The other spectral peaks are of lesser magnitude.

The presentation of mean wind velocity in the form of spectra provides important information in respect of energy at different frequencies at which the spectral values are the highest. This form of presentation involving parent population is more direct than the existing mathematical models for extreme mean winds. The spectra obtained for low frequencies, therefore, results in a morel rational static analysis than the one based on extreme mean winds. The design mean wind speed to obtained can be used for computing basic wind pressure required in Gust Factor Method of analysis. The design mean wind speed is to be computed for a standard height of 10 m.

Gust

Gust measurements were carried out with KANOMAX Anamomaster Model 6611. It is a constant temperature type thermal anemometer with large size liquid crystal display and printer facilities. It has a response time of 0.2 sec. and the moving average value is displayed after eight cycles of sampling in a second (0.125 sec. sample cycle). There is a provision for linearised analog output also. By selecting suitable data interval and data number the signals are obtained in a desire format. For different sets of such measurements maximum, minimum, average and standar deviation values were obtained. Based on these values turbulence intensity, peak to mean wind ratio and peak factor were computed and these were found to be 41 percent, 2.0 and 2.7 respectively (10).

The analog ou put was recorded on TEAC cassette Data Recorder Model R-61 D, for varying time segments. A typical time history record is shown in Fig. 8. The recorded signal was fed to 'AND' FFT Analyser Model 3522. Based on the spectral ordinates obtained in high frequency range and frequency range of Civil Engineering Structures 10 Hz. range (20 sec. sample lenght) was selected. For this range instantaneous spectra as well as averaged spectra for varying duration ot time on various days at different hours were obtained. The spectral hujp has been consistently noticed in the frequency range of 0.16 Hz top 0.5 Hz (9). A typical averaged spectra is shown in Fig. 9 for a flexible structure, Fig. 10. The observations on the rotating tower, Fig 10 (b) are yet to be taken.

The gust spectra has a relatively broad hump as compared to the one for slowly varying velocities. The response spectra is obtained by multiplying free wind spectra with "aerodynamic admittance function" which is dependent on the structural properties. The effect of mechanical admittance is to create a new peak in the response spectra at the natural frequency of the structure. For Civil Engineering Structures it occurs to the right of the broad hump of free wind spectra.

The response of a structure to gust is taken as sum of two components: (a) area under the broad hump representing non resonant response due to back-ground turbulence, $RB^{1/2}$, and (b) area under the resonance peak representing resonance response around natural frequency of structure, $R (SE/\beta)^{1/2}$. The backround excitation factor, B, is a function of structural dimensions, turbulence lenght scale, longitudinal and lateral correlation constants. The size reduction factor, S, is a function of structural dimensions, its natural frequency, mean wind speed, longitudinal and lateral correlation constants. Gust energy factor, E, is a function of natural frequency of structure, mean wind speed, turbulence lenght scale and gust spectrum. The surface roughness factor, R, depends on terrain category. The peak factor, gp, is the ration of maximum dynamic response to rms value. The gust factor, G, is obtained from the expression $(1 + gp\ R\ (B + SE/B)^{1/2})$, and used in Gust

Factor Method for obtaining design forces on structures (19,1).

Community Shelters

During cyclones community shelters should be available in each village. The size of the shelter would be governed by population. These shelters can be single storey or double storey R.C. framed structures designed to withstand storm without damage. The present Indian Code for Wind Loads, IS 875 (Part3) 1987, recommends basic wind speeds for 50 years return period with 63 percent risk.

These shelters are to be designed as special structures with reduced degree of risk. The speeds considered are based on peak winds for specified return period and life of structure. The data pertaining to high winds close to the eye require a very close monitoring of the movement of strom and instrumentation within the core of the spiral movement of the winds. No data is available based on a systematic monitoring and measurements of speeds within the 'wall cloud'. Though the structures designed with reduced degree of risk would be uneconomical yet this penality has to be paid for saving human lives concentrated in such safety pockets as shelters. These shelters should be at elevated place with sufficient open space to accomodate cattle and property. The shelters besides being located at elevated sites should be as far as possible be away from high risk zones. This aspect is further highlighted elsewhere in the context of development zones. It is proposed that such buildings may not be erected specifically for shelters only. These buildings should be used as school and serve other community purposes so that round the year these are utilised.

Dykes and Sea Walls

The coastal areas frequently innundated by strom surge and floods can be saved by constructing dykes and sea walls. This kind of training work will not only save the coastal population from onslaught of storm surge and floods but also save the agricultural land from being spoiled by saline sea water.

2. Non Structural Measures

Shelter Belt and Land Use Zoning

The cyclones and associated strom surge and flood result in a wide scale coastal erosion. This can be checked by planting high growth varety trees along the coast as shelter belts. The belt of the trees act as cyclone breaker and enhances the decceleration process of the cyclone intensity. Experimental studies carried out in China reveal that plantation of the three rows of trees along the coast line are adequate. Highly risk prone areas need to be identified for an effective land use. The development projects are expected to be in areas not lying in the high risk zones.

Cyclone Warning System

Efficient warning system consists of (a) forecasts of movements of cyclones and their intensity in advance from the stage of formation over the sea (about a week) (b) rapid and reliable system of warning to the affected people (more than 24 hours), (c) prompt action of warnings by Government Agencies, Voluntary Agencies as well as the affected people.

Accurate prediction of severe storm requires adequate and reliable data and a sophisticated mathematical modelling. In the present age of the advancement in instrumentation, availibility of high speed computers, the complex mathematical models can be handled with ease. With the access to these facilitiesIndian Meteorological Department has been successful in forecasting the event twenty four hours in advance with sufficient degree of reliability. Cyclone warnings are provided through different " Area Cyclone Warning Centres " / " Cyclone Warning Centres " / " Coordination Centres " established in different parts of the country. Radio and television network in the country is used for broadcasting warnings to the affected people.

Rescue Operations

Immediately after occurrence of calamity there is an urgent need of large scale rescue operation for evacuating people to safer places and relief camps. In the event of collapse buildings, tehreis also a need of trained manpower to recover dead bodies and taking survivors for treatment. People should be trained for such works, Army personnel may be called only in exceptional circumstances.

Relief Operations

The supply of essential commodities to the affected people well in time is very essential. Voluntary agencies and other institutions can play major role for collection of the essential items (food items, clothes, medicines ect.) and distribution of the same among the affected people.

High priority should be given to restore cut off routes (roads, water transport) from other unaffected areas for bringing speedy normalcy.

Another important aspect of management is prevention of spread of disease in the affected area after disaster. All the necessary step should be taken to check breakout of any kind of epidemics by restoring cleanliness, putting sewerage system in order, supplying uncontaminated drinking water in cans till clean water supply is restored in the affected areas.

i. It is not feasible and economical to design structures which are fully resistant against the cyclonic winds. The designs of shelters with usual risk factor are not adequate against cyclonic winds.

ii. A systematic monitoring of the movements of the cyclones and measurement of the highly spiralling winds is needed to arrive at appropriate peak winds for design purposes.

iii. Design of low rise dwellings for recomended peak winds should lay emphasis on detailings of the joints of various structural components.

iv. Umbrella structures in the form of large span membranes as shelters for individual dwellings may protect the life and property. The ferrocement attic units and core units are recomended which help in fully recovering the valuable items of the assets.

v. The community buildings like schools are required to be designed for low risk values instead of the simply basing on codal provisions for normal structures, so that these could be used as shelters during cyclones. Buildings constructed exclusively to serve as shelters during cyclones can not be economical.

vi. The warning with reliability should give enough time for mobilizing voluntary and other government agencies. With sophisticated weather modelling based on satelite data it should be possible to give proper reliability to (a) forecasting and (b) warnings with sufficient time (more than 24 hours) before a cyclone strikes.

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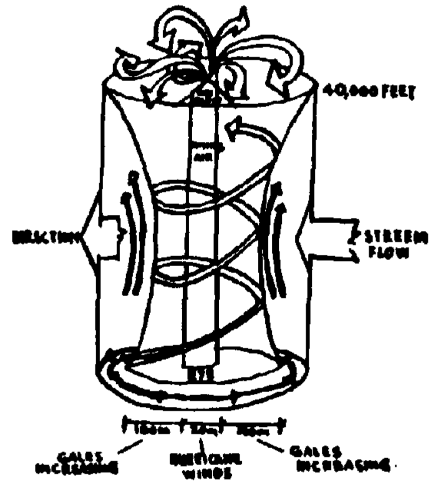

FIG:1. CROSS SECTION OF A CYCLONE.

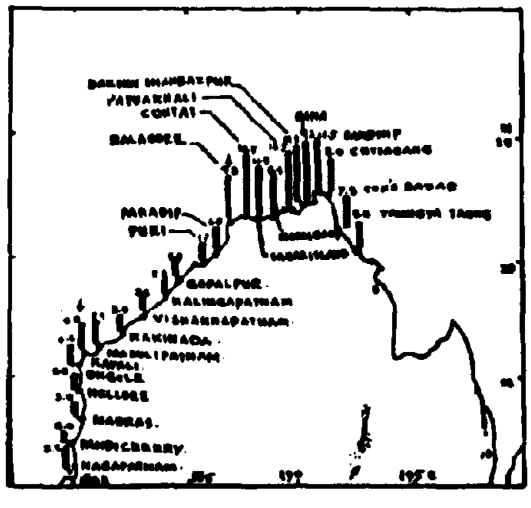

FIG:2. PROBABLE MAXIMUM STORM SURGE IN METRES.

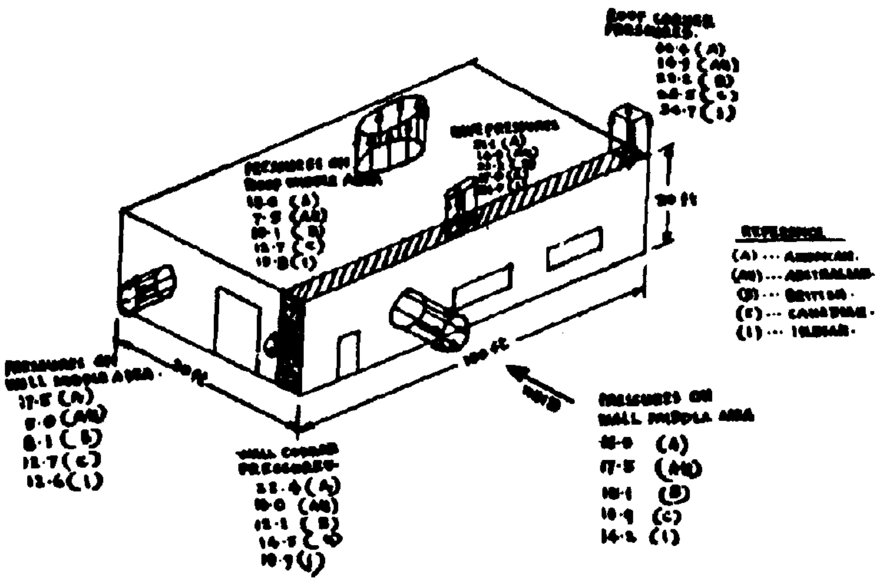

FIG: 5. DESIGN WIND PRESSURES OF CLADDING FASTENERS.

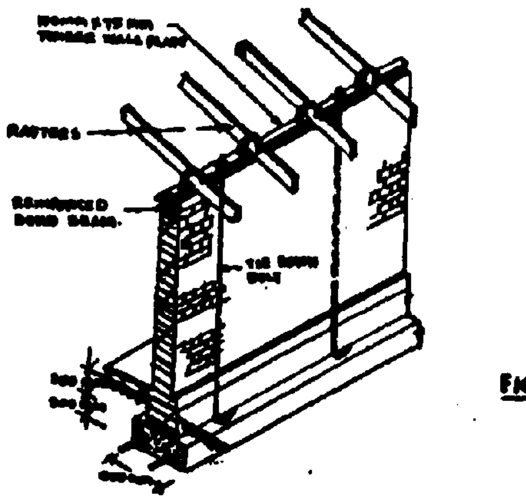

FIG: 4.(a). FIXING OF WALLS TO THE FOUNDATION TIE-DOWN BOLTS.

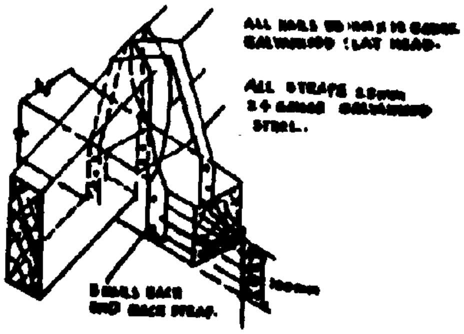

FIG: 4.(b). USE OF STRAPS AND NAILS TO TIE-DOWN RAFTER TO THE TOP PLATE.

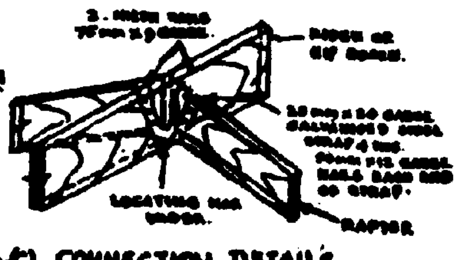

FIG: 4.(c). CONNECTION DETAILS AT THE RIDGE.

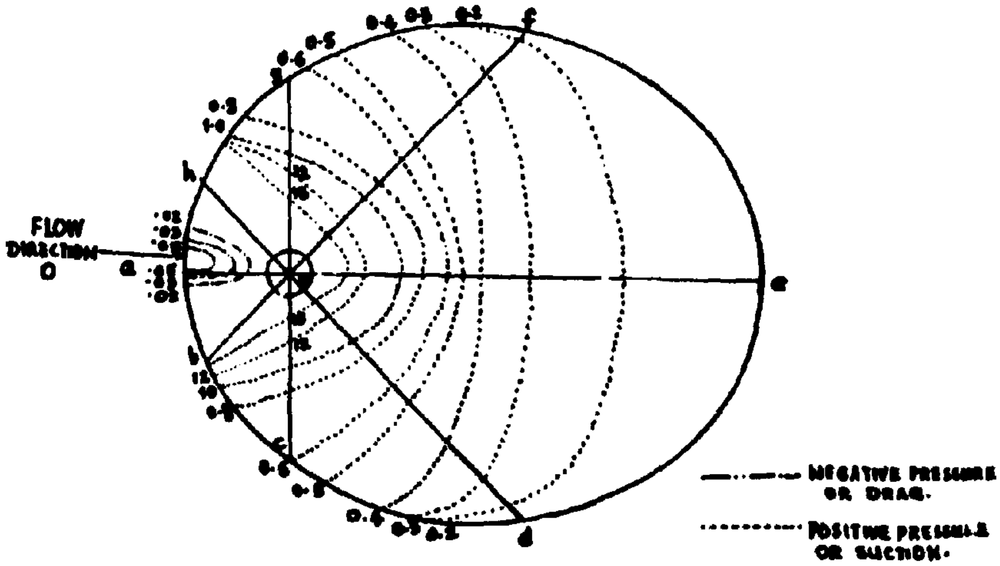

FIG: 5. WIND PRESSURE DISTRIBUTIONS ON A LARGE SPAN MEMBRANE STRUCTURE.

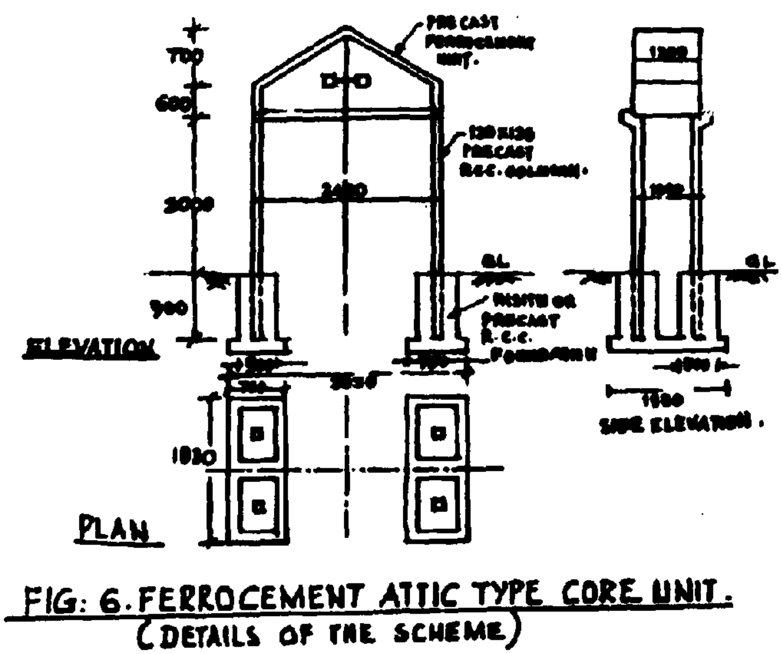

FIG: 6. FERROCEMENT ATTIC TYPE CORE UNIT.
(DETAILS OF THE SCHEME)

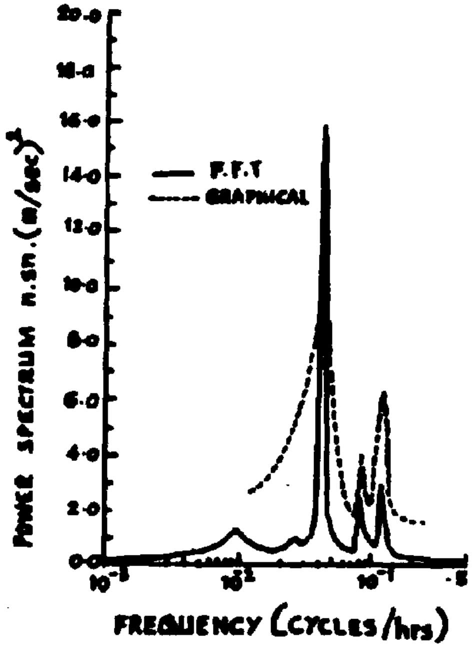

Fig: 7. MEAN WIND SPECTRA

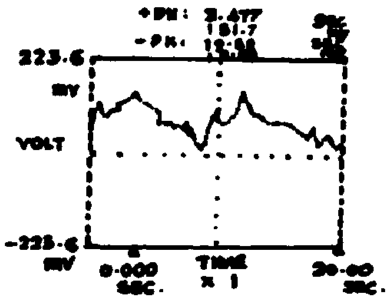

FIG: 8. A TYPICAL TIME HISTORY RECORD.

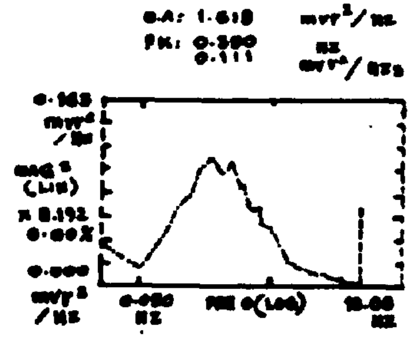

FIG: 9. AVERAGED GUST SPECTRA

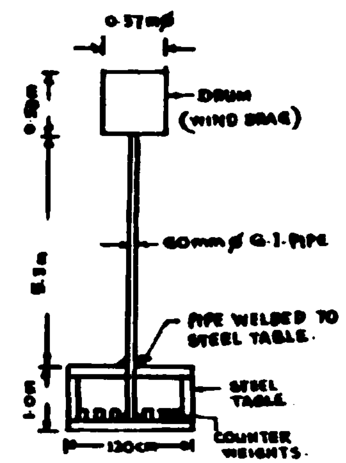

FIG: 10(a). DIMENSIONS OF THE FLEXIBLE STRUCTURE.

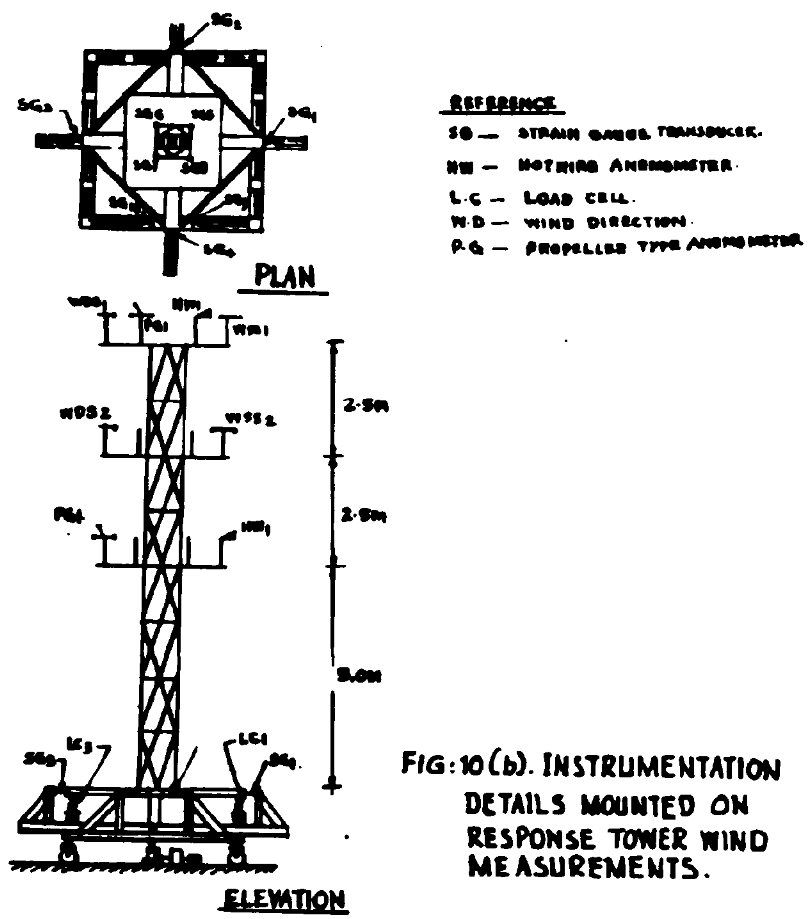

FIG: 10(b). INSTRUMENTATION DETAILS MOUNTED ON RESPONSE TOWER WIND MEASUREMENTS.

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NEEDS FOR DISASTER MITIGATION RELATED TO NON-ENGINEERED AND PARTIALLY ENGINEERED BUILDINGS SUBJECTED TO WIND STROMS

Dr. Prem Krishna[1] and Dr. A.K. Ahuja[2]
1. Professor of Civil Engineering
2. Lecturer in Civil Engineering
University of Roorkee, India

ABSTRACT Coastal zones of many countries namely Australia, Bangladesh, China, Hongkong, India, Philippines and U.S.A. frequently experience severe cyclones apart from high velocity inland storms. Cyclones are generally followed by heavy rain and storm surge resulting in loss of lives, crops, animals and severe damage to structures specially dwellings leaving thousands homeless. In most of the wind storms, the damage to non-engineered or marginally engineered buildings is considerably larger compared to engineered buildings. This is partly because scant attention is paid to details in case of the non-engineered buildings and partly due to the lack of comprehension and application of information available regarding wind effects on structures. Today there is great awareness for the need to mitigate disasters due to hazards including wind storms. The prime need is to coordinate the efforts being made to study the influence of storms on structures in wind tunnels as well as through post-disaster surveys in the field, innovations in design ideas and the technology being followed at the 'grass roots' level. The relief and rehabilitation work needed will reduce sharply, if disaster preventive measures are taken up in earnest. The present paper puts together some ideas on the developmental and training needs required for reducing wind disasters.

INTRODUCTION

Occurance of natural hazards is as old as the earth. Wind storms, earthquakes, floods, volcanic erruptions, landslides etc. have battered mankind since times immemorial. These may be considered as the wrath of God which cannot be prevented, except floods which can be controlled with varying degrees of success, disasters are indicative of the failure of mankind to mitigate them, being a result of system or structure failures. The disasterous effects of these hazards have gone on increasing with time because of more intense landuse.

While hazards are fairly disbursed over the entire earth, the disasters are greatly concentrated upon the developing countries. Amongst the different disasters, wind storms, earthquakes and floods account for the greater proportion of destruction. Table 1 gives an idea of the devastation caused by different natural hazards, in terms of loss of human lives, people rendered homeless and monetary losses.

Two factors that can influence the strategies for disaster mitigation and management are (i) type of hazard, (ii) whether the hazard is predictable. Land slides, earthquakes and volcanic erruptions often inflict loss by damage to structures, by burying populations and by causing fires. None of these are predictable to any degree of accuracy. In the event post disaster relief and rehabilitation is the only way unless disaster preventive measures are taken up. Wind storms in general cause destruction of buildings and structures, except in the case of cyclonic storms which are accompanied by heavy rain and a killer surge causing flooding and destruction thereby. Cyclones can, however, be predicted to a fair degree of accuracy in order that populations likely to be affected can be warned. Floods, perhaps the only hazard which is preventable, though perhaps at large expense, is also predictable.

Countries in the South-East Asian region and the Bay of Bengal region are affected by severe cyclones. Coastal regions of these countries, which are heavily populated and also categorised as developing nations, are hit by cyclones almost every year -- sometimes more than once a year. Although inland storms also occur in these regions, the problem is more acute in coastal regions which suffer enormous losses in terms of human lives, livestock, agricultural production, buildings and structures, and, industrial production.

The organisation and infrastructure in these countries for issuing timely warning during cyclonic storms and acting upon them to evacuate people, for provision of shelter and relief to those affected by the disaster, and, for rehabilitation is improving steadily. As a result, whereas, there is yet considerable loss and misery created by cyclones, loss of human life is now comparatively smaller. However, the same is not true for mitigation measures. In fact if mitigation activity is pursued in earnest, the extent of the disaster will reduce sharply and the relief and rehabilitation needed will reduce likewise.

DISASTER MITIGATION

Essentially the steps needed in disaster mitigation, which is a multi-faceted activity, are those which have to be taken (i) on a continuing basis to minimize or prevent the occurance of a disaster, (ii) just prior to and during the period the hazard strikes, and (iii) during the post-disaster stage. In this paper the pre-disaster building activity and the post-disaster reconstruction related to the dwellings of the poor are only being considered. This is not to suggest that engineered structures of different descriptions also need considerably greater effort in order to prevent their failures.

Disaster Preventive Measures

The main thrust of these measures would be to ensure that the structures do not get damaged when the storm strikes. Various reasons in general for the failures of structures during wind storms could be (i) wind speeds and loading phenomenon not being correctly known, (ii) a lack of comprehension of the design information as it exists, leading in one way or the other to a poor design, (iii) use of poor material, (iv) badly implemented construction, (v) in the context of the present paper, total (or virtually so) lack of engineering input. Whereas (i) is a subject of extensive attention separately, a majority of failures that occur can be avoided, if the situation vis-a-vis (ii) to (v) is improved through appropriate measures.

Non-engineered and partially engineered structures, examples of which are most of the buildings and houses of rural poor in the developing world, are near definite casualties every time a storm of even moderate strength strikes a rural establishment. These are rebuilt, more or less as they were, often with some subsidy being provided. The cycle of destruction and reconstruction goes on repeating. While efforts need to be made to shift rural habitats away from severely affected regions, there are in many instances, difficulties created by the pressure on space as well as socio-economic compulsions. There is nevertheless little doubt that with some effort this repetitive failure of dwellings is largely avoidable.

The relief and rehabilitation work needed which are part of post-disaster actions, will reduce sharply if disaster preventive measures are taken up in earnest. Therefore, in order to achieve this goal, specially with regard to non-engineered and partially engineered buildings, the authors believe that substantial degree of developmental effort is required in addition to training of junior level engineers and artisans.

Developmental work It will include development of strengthening measures for existing houses, and, development of inexpensive housing modules which can be adopted for construction by the local artisans and the people themselves with little supervision.

If the measures suggested for strengthening as well as the new modules give due congnizance to existing practices and keep them simple, the chances of adoption will improve. This is so because,

(i) design of such houses have evolved over a length of time to cater for local needs, climate and material,

(ii) modifications designed, if simple, can be implemented by the affected communities themselves.

The manner in which this exercise can be done is to demarcate regions where there is commonality in the existing designs and materials. Typical modified designs be then prepared and a pilot project be undertaken to demonstrate this design by building a few prototypes in a number of different villages with the local public works engineering organisations and tradesmen being involved. The expectation is that these models will then be adopoted. It can be made more effective by preparing simply-written manuals in local language. Further, the National Governments may consider encouraging the adoption of the modified designs by undertaking to provide loans/subsidy and to cover them with insurance.

Observations made by various researchers regarding failure of partially engineered and non-engineered buildings in wind storm disasters and recommendations for construction of wind resistant buildings are worth mentioning at this juncture.

Safety of partially engineered and non-engineered buildings generally depends upon plan of building, material and type of construction. These buildings fail during wind storms, showing foundation failure, part structural failure or complete structural failure. Such buildings can be made wind resistant by following simple, inexpensive strengthening measures.

1. Incase of thatch roof buildings, roof should be tied with straw ropes in different directions which will serve as wind bracings.
2. In case of tiled roof, continuous coat of mortar should be laid over tiles.
3. U-bolts should be used in place of J-bolts for connecting sheeting to purlins.
4. M.S. flat ties should be used in the eaves region along with sheetings.
5. Rafters and ridge poles should be properly tied/lashed.

6. Steep slopes should be prefered on roofs.
7. Overhangs should be minimised or provided with openings.
8. Proper footings should be provided for timber posts.
9. Ropes or coconut netting should be used as tie-back for weak dwellings.
10. Side openable windows should be avoided and top hinged windows should be prefered.
11. Trees should be grown surrounding the dwellings to reduce wind speeds.
12. Buildings should be provided with raised floor in order to check the entry of sea and rain water.

Figure 1 illustrates some of the recommended strengthening measures. These are picked up from existing literature and are only indicative of the possibilities. Details indeed will require to be evolved vis-a-vis identified building practices.

Rehabilitation of typhoon vistims in the Philippines is an example of an excellent piece of developmental work. Government of Philippines, with the co-operation of Asian Disaster Preparedness Centre (ADPC), AIT, Bangkok has built cyclone resistant houses on a "self-help" basis, pooling labour resources of the beneficiaries and utilising indigenous materials under the supervision of trained foreman ensuring that standards are maintained. The units constructed have simple design, but structurally strong, and material used are timber, G.I. sheet, cement-hollow block and split bamboo. Thousands of such dwellings have been built and are bound to cause a marked change in the socio-economic status of the users.

Another example of the developmental work for rehabilitation is that of Tonga. Though the level of technical inputs and specifications used is several notches higher as compared to the example from the Philippines. The islands of the Kingdom of Tonga in the South Pacific were hit by a tropical cyclone during March 1982 in which about 2000 houses were damaged. Main reason of failure was noted to be lack of engineering input. Building Research Establishment, U.K. developed new design on modular basis using timber and G.I. sheeting with special emphasis on connections. Both model and prototype of cyclone resistant house were tested at cyclone testing station, Queensland, Australia. 2000 new houses could be constructed within two years period. This project was an excellent example of the application of research work to alleviate the suffering caused by the disaster. Furthermore it demonstrated the benefit of carrying out a collaborative programme at more than one institute, and proved that a combined input from several countries could, literally, pick up the pieces after such a storm and put people back in houses.

Training programme As mentioned earlier, a majority of failures of buildings occur due to lack of comprehension of the design information as it exists, leading in one way or the other to a poor design; use of poor material; and badly implemented construction. Situation in this regard can be improved by giving proper training to junior level engineers and artisans who are responsible for building construction. Whereas, rural buildings and houses, which are typical examples of partially or non-engineered buildings, are generally constructed by the rural poor in the developing countries by themselves, properly trained engineers and artisans can take responsibility of passing knowhow to them.

Training courses need be organised in affected countries for junior level engineers and tradesmen involved in supervising/executing construction (in the coastal regions of the country in case of larger countries like India). A resource team consisting of experts drawn from within and outside the region should conduct courses which need to be repeated at short intervals over certain number of years to have a lasting impact. The leading/co-ordinating role could be undertaken by an institution with the necessary expertise for the purpose, however local institutions/organisations must join hands with the co-ordinating institution. The purpose of the training courses should be to barely introduce the design aspects but mainly to bring forth the construction and maintenance aspects. It is an earnest hope that failures that occur on account of ignorance can be reduced through the training envisaged above.

CONCLUSIONS

It is being recognised increasingly in the area of Disaster Preparedness, Management and Mitigation that "prevention is better than cure". Therefore, measures for reducing the disaster need intensification, without detracting from the efforts in relief and rehabilitation. Although all types of structures need careful attention, non-engineered and partially engineered buildings require special attention with regard to windstorms.

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Table 1 DEATH AND DESTRUCTION DUE TO NATURAL DISASTERS

Disaster	Deaths (1900-1976)	Homeless (1900-1976)	Losses (in Millions of US $)
Landslides	3,006	44,673	
Avalanches	3,059	150	
Volcanic-Erruptions	1,28,058	3,37,931	
TOTAL	1,34,123	3,82,754	10,000
Cyclones	4,34,894	1,76,48,463	
Hurricanes	18,513	11,97,535	
Typhoons	34,103	54,37,054	
Storms	7,110	34,32,641	
Tornadoes	1,175	3,42,459	
TOTAL	4,95,795	2,80,58,152	47,800
Floods	12,87,645	17,52,20,220	24,100
Earthquakes	26,62,165	2,88,94,657	49,300
GROSS	**45,79,728**	**23,25,55,783**	**1,31,200**

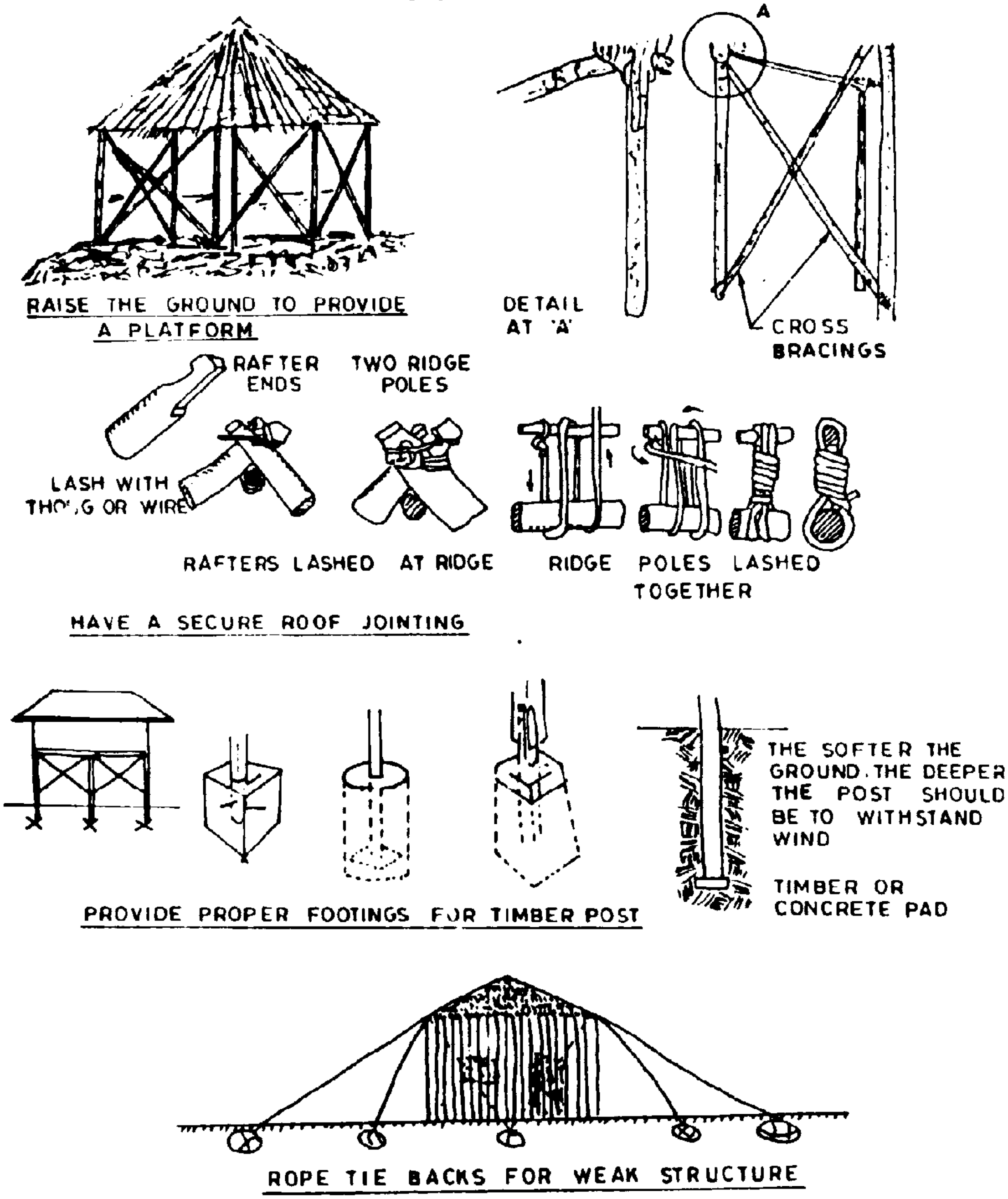

FIG. 1 — TYPICAL STRENGTHENING MEASURES

SECOND US-ASIA CONFERENCE ON
ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
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ENGINEERING OF STRUCTURES FOR WIND HAZARD REDUCTION

J. Shanmugasundaram, S. Arunachalam and T.V.S.R. Appa Rao
Structural Engineering Research Centre
Madras, India

ABSTRACT Every year a large number of buildings, industrial structures, and houses ranging from non-engineered type to well-engineered type, are severely damaged by tropical cyclones in many parts of the world. Mitigation of the effects of cyclone disaster is a problem of international importance. Assessment of wind loads due to cyclones and prediction of structural response of different structures subjected to such wind forces are highly complex to be dealt with using theoretical models alone. The present level of understanding of characteristics of cyclonic winds and causes of damage to houses, buildings, towers, and other wind-sensitive structures is far from satisfactory. Post-disaster field surveys on structural damage provide invaluable information that will help in understanding the modes of failure of structures and in developing cyclone-resistant designs. The paper deals with the details of the post-disaster surveys and assessment of structural damage caused by the two recent severe cyclones which hit Kavali and Guntur regions of Andhra Pradesh in India. Based on the

mitigation of damage to structures are inadequate. Further, it may be noted that there are complexities involved in assessing (i) probability of occurrence of extreme wind speeds during severe cyclones, (ii) wind pressure/load distribution on different types of structures (iii) dynamic response/strength of structures and (iv) types of failures/damage to structures and mitigation of damage. Hence studies based on theoretical models alone will not be adequate to understand the realistic nature of wind and to predict the behaviour of structures in cyclonic storms. Post-disaster field surveys on structural damage caused by cyclones will provide invaluable information in developing/validating analytical methods for determining structural response. Documentation of cyclonic wind data, typical failures of structures etc., enables in identification of various factors influencing the damage to structures by cyclonic forces. This paper deals with the post-disaster surveys conducted by SERC, Madras, on damage to buildings and other industrial structures caused by the two severe cyclones which hit Andhra Pradesh during November 1989 and May 1990. Based on the survey and analysis of data and also based on previous experience in this area guidelines for design of buildings and structures to resist cyclonic forces are outlined. Emphasis is also laid on training to be imparted to design and field engineers so that the results of R&D are translated to mitigate the damage to structures.

2. POST-DISASTER SURVEYS - CASE STUDIES

A post-disaster field survey on damage suffered by various buildings and structures during the severe cyclone which hit Kavali in Andhra Pradesh on 9th November 1989 was carried out by SERC, Madras. The above cyclone has caused colossal loss of life and property. An area of about 8 km radius in and around Kavali was severely battered. According to official sources, the cyclone caused 48 deaths and over 500,000 people in 545 villages were affected and over

100,000 people were rendered homeless. The gale wind speeds, when the cyclones crossed the land, as estimated by the India Meteorology Department was 187 kmph (52 m/s).

Direct measurement of maximum wind speeds during the occurrence of a cyclone, using the present day instruments, poses difficulty due to their limitations. On the other hand by analysing the nature and extent of damage suffered by typical structures, it is possible to reasonably assess the extreme wind speeds during the cyclone, that could have caused the failure. The failure of a R.C. pole in bending caused by the cyclone which hit Kavali in 1989, was analysed by the authors, taking into account the available moment carrying capacity of the failure section and the effects of along-wind forces due to the cyclone. The lower bound of the cyclonic wind speed that could have caused the failure was estimated to be 207 kmph (57.5 m/s) and this compares well with the cyclonic wind speed of 187 kmph (52 m/s) estimated by India Meterology Department, based on satellite/radar photographs, as mentioned earlier.

A similar field survey was carried out by this Centre on structural damage caused by the severe cyclone which hit the coastal areas of Guntur and Krishna districts in Andhra Pradesh, on 9th May 1990. The cyclone was also accompanied by high tidal waves and inundation by sea water, which led to colossal loss of agricultural crops and other livestock. It was reported that the cyclone crossed the coast near Machilipatnam with a wind speed of about 220 to 250 kmph (61 to 69 m/s).

2.1 Damage to Buildings and Structures due to Cyclones

It was observed from the above surveys that the cyclones caused much havoc to a large number of buildings and structures, ranging from non-engineered type such as huts, tiled houses and boundary walls to well-engineered structures such as elevated water tanks, and microwave towers. Typical damage observed during such cyclones include:

(i) blowing off of roof sheets due to poor connections between claddings and structural elements in pitched roof buildings

(ii) failures of tiled and A.C. sheet roofs due to uplift forces owing to their light-weight compared to R.C.C. roof system

(iii) collapse of boundary and gabled walls due to inadequate strength in resisting lateral high wind forces

(iv) collapse of low-rise industrial structures, due to lack of roof and wall bracing system, and

(v) failures of a few well-engineered structures due to design inadequacies

A few examples of the complete destruction of the well-engineered/semi-engineered industrial structures are given below:

a) collapse of a 91 m high microwave tower (Fig.1)

b) failure of a 12.6 m high overhead service reservoir (40,000 litres capacity) (Fig.2)

c) collapse of a 62 m long godown at the Port Trust in Machilipatnam (Fig.3)

3. DESIGN IMPLICATIONS

Based on the survey observations, it may be noted that by adopting simple, improved design and construction techniques, the above mentioned types of failures can be minimised and the resistance of structures against cyclones can be increased as described below.

It was observed that failure of roof sheet claddings in many of the low-rise buildings was primarily due to high uplift suction forces and low dead weight of cladding materials (185 N/m^2). On the other hand buildings with R.C.C. roof, due to heavy dead load (2400 N/m^2) performed well during the cyclones.

Fig.1 Collapse of a 91 m high microwave tower

Fig. 2 Failure of a 40 cu.m. capacity overhead service reservoir

Fig. 3 Collapse of a 62 m long godown at the port trust in Machilipatnam

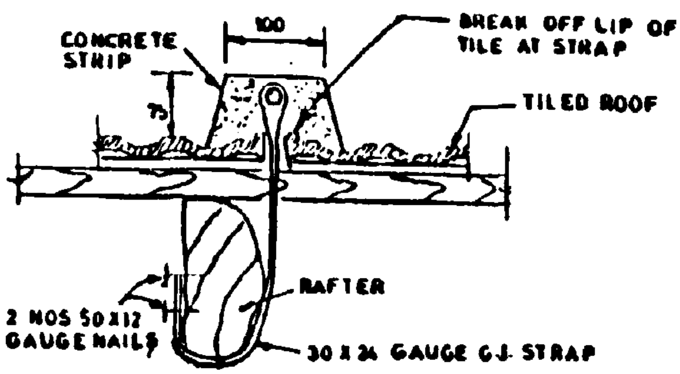

Fig. 4 Tieing down of tile roof cladding to the main rafter

In pitched roof residential and industrial structures 'J' bolts are conventionally used to tie down the roof cladding to the purlins. Due to dynamic action of cyclonic forces, the hook of the 'J' bolt is flattened, weakening the connection without effectively holding down the cladding sheets to the purlins. By providing 'U' bolts in place of 'J' bolts at closer intervals, failure of 'J' bolts due to stretching can be avoided and damage due to blowing off of cladding sheets can be considerably reduced.

It may be of interest to note that in some of the godowns and industrial sheds at Guntur where 'U' bolts in place of 'J' bolts were used for the connection between the roof cladding and the purlins offered adequate resistance to A.C. sheet roofs against uplifting cyclonic forces.

In the case of tiled houses, their resistance to cyclonic uplift forces can be improved by providing a continuous mortar bands over the cladding or by providing closely spaced concrete restraining strips in the critical regions of ridges, eaves, corners and by tying-down the roof tiles to the main rafter using G.I. straps as shown in Fig.4.

It was noticed that in most of the industrial structures, the failure was typically due to the failure of the gable walls due to poor lateral strength followed by progressive collapse of the roof system. Based on the observations, it is suggested that by providing a continuous RCC beam over the walls, the lateral resistance of the brick walls can be highly improved. Alternatively, the trusses can be supported on R.C.C. pillars with infilled brick walls. The above system ensures stability of the truss system, even when the infilled wall fails. Besides, trusses should be adequately strengthened with diagonal bracing in order to prevent progressive collapse..

Failure of compound walls during cyclones is very common. However, their resistance against cyclones can be increased by anchoring them to the concrete foundations through the use of 12 mm diameter tie-down bolts/rods.

Some of the well-engineered structures, such as, tall communication towers, chimneys, also collapsed due to severe cyclones. It was noted that these failures were due to inadequate design considerations and construction practices. In view of the heavy direct and indirect losses in the event of failure of such structures, it is stressed that for such structures a detailed analysis taking into account the dynamic response of the structure and the effects of height, terrain and topography on wind speed has to be carried out. Besides buffeting, torsional effects should also be considered in the design, wherever applicable.

As stated earlier, the maximum wind speeds experienced during these severe cyclones were between 52 m/s and 69 m/s and these values are higher than the design wind speed recommended in the Indian Standards on wind load criteria [Indian Standards IS- 875(Part 3)-1987].

In order to estimate maximum wind speeds, cyclonic wind speed data were collected from the India Meteorology Department. Using Monte Carlo simulation technique, a large number of data were generated and a statistical analysis was carried out using the extreme value distributions of Gumbel and Frechett type and log-normal distribution. Based on the Kolmogorov Smirnov goodness-of-fit test., the Gumbel distribution was selected to fit the test data well and the 63% characteristic wind speeds were computed for different coastal regions of India ["Venkateswarlu and others, 1989b"]. Thus, a regional multiplying factor for different coastal regions is recommended as shown in Table 1, to account for the increased intensity of cyclonic winds, for design of structures of considerable importance.

4. NEED FOR TRAINING COURSES ON CYCLONE DISASTER MITIGATION

The problem of cyclone disaster mitigation of damage to buildings and other structures, is only at the research level, due to the complexities involved in prediction and estimation of wind loads. Further, only limited organisations in our country have carried out post-disaster field damage surveys on buildings and structures. Hence, it is necessary that the expertise acquired and the

TABLE 1. RECOMMENDED REGIONAL MULTIPLYING FACTOR FOR CYCLONIC WINDS

Sl. No.	Coastal Region	Basic wind speed as per IS:875(3)-1987 (m/s)	63% characteristic wind speed (kmph)	(m/s)	Regional Multiplying Factor
1	2	3	4		5=(4/3)
1.	Tamil Nadu	50	254.85	70.8	1.42
2.	Andhra Pradesh	50	262.79	73.0	1.46
3.	Orissa	50	259.00	70.3	1.41
4.	West Bengal	50	215.48	59.9	1.20
5.	Konkan	44	223.51	62.1	1.41
6.	Sourashtra	50	223.51	62.1	1.24

results/information obtained/generated based on the work/research done at various organisations have to be transferred to the building industry, civil construction companies in public and private sectors, to realize construction of cyclone-resistant structures. Such dissemination of information and transfer of techniques may be carried out by conducting training courses and workshops in which a large number of design, and field practising engineers can participate. With the above objectives, the Centre has organised two international workshops and two in-house courses during 1985 and 1992, to practising design and construction engineers.

5. CONCLUSIONS AND SUMMARY

Failures of buildings and other structures due to cyclones result in heavy loss of life and property. At present, our state-of-knowledge on characteristics of severe cyclones, their interaction effects on buildings and structures etc., is far from satisfactory. Details of the post-disaster surveys carried out by SERC, Madras, on damage to different structures caused by the two severe cyclones which hit Andhra Pradesh during November 1989 and May 1990 are briefly described. Based on the survey observations, possible causes for failures of several buildings and structures were discussed and simple design and construction techniques have been suggested to improve the cyclonic resistance of various structures. Use of regional multiplication factor for different coastal regions of India is also recommended to account for increased intensity of cyclones for design of important structures. The need for conducting training courses on cyclone disaster mitigation for field and practising engineers is also emphasised.

6. ACKNOWLEDGEMENTS

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HOLISM, EUROCODES AND NATURAL HAZARDS IN THE PACIFIC RIM

Alan P. Jeary
Dept. of Building and Construction, CPHK
Kowloon Tong, Hong Kong

ABSTRACT This paper discusses the risk of damage to housing throughout the Pacific area, from the passage of Cyclones and Typhoons. Construction techniques throughout the area are considered and the experience of different communities are highlighted. The appearance of changes to the traditional paths of cyclones has severe implications for regions not traditionally associated with Cyclone activity. Interestingly the communities most at risk appear to be located in richer countries.

INTRODUCTION

The Pacific Ocean affords the greatest opportunity for the generation of Typhoons (Cyclones) on the surface of the Earth. Further, these cyclones, once developed, have greater opportunities for developing to the maximum potential, and for affecting populations in the area. There are two types of community that is affected in the Pacific area. These can be essentially divided into developed and partially developed countries, ranging from the type of community found in Japan or the USA, to those existing on isolated islands within the main body of the Ocean. The types of housing or dwelling are also divided according to the type of community. However, the ability of housing to withstand cyclones is dependant on the "community memory" of disasters. The communities carrying the greatest risk are those which have displaced, or who live at the extremity of an area of natural hazard, such that the rate of occurrence of that hazard is low enough for the community to forget a previous traumatic occurrence, and produce a new type of structure that does not take account of the lessons of history.

Another hazard is identified where regions previously in an area of frequent recurrence of cyclones are subjected to an altered climate which reduces their occurrence, and the preparedness of the community.

THE CLIMATE OF THE PACIFIC AREA

There are three major areas of cyclone activity in the Pacific region. These occur off the coast of Mexico, and in the western Pacific both north and south of the equator. Figure 1 shows these areas.

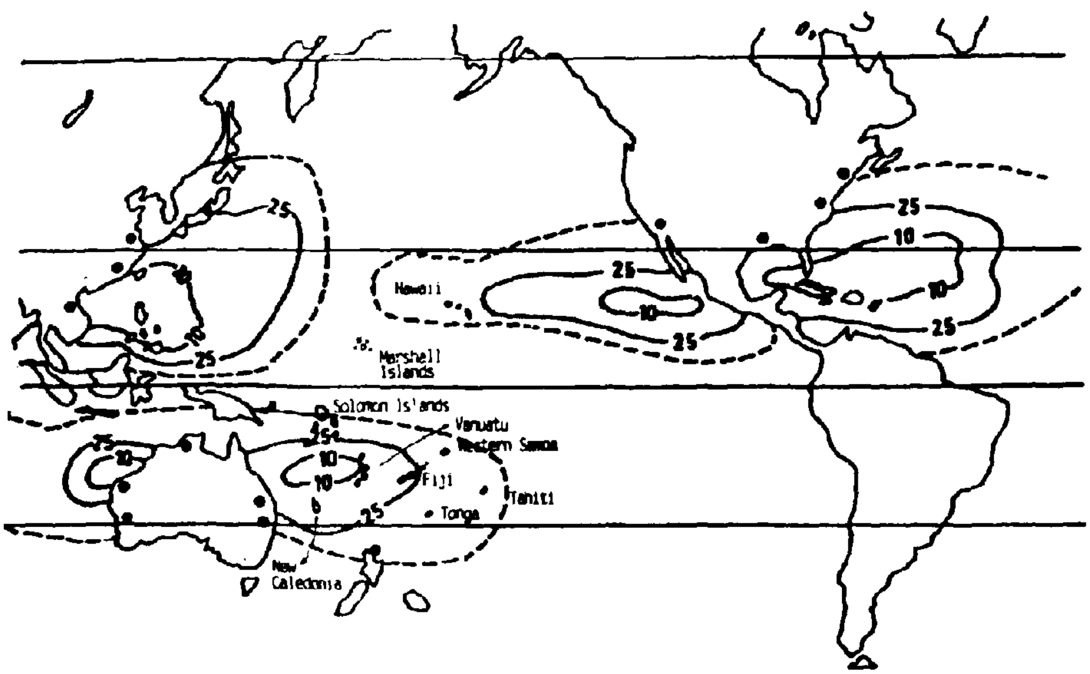

Figure 1 Zones of occurrence of Cyclones in the Pacific area

Since landmasses have a profound effect on the ability of a cyclone to maintain its momentum, the appearance of large landmasses can in some areas provide some shelter from the full effect of fully developed cyclones. Two such regions are the Northern Territories of Australia, and the Malaysian peninsula.

In general terms all Cyclones in the western Pacific move westerly, some with a curvature towards the pole, whilst in the central southern Pacific Cyclones move south easterly. Asia and Australia, block the natural Cyclone movement, resulting in areas of high risk close to these landmasses.

CONSTRUCTION IN THE PACIFIC AREA

There are two distinct types of construction throughout this area depending on whether the community is a rich or a poor one. The ability to withstand the ravages of wind forces, however, appears to be independent of the wealth of the community.

In the poorer regions naturally occurring woods are often used for construction. Coconut wood is particularly useful since it is quite hard and coarse. It is particularly commonly used for roof beams. However, because of the coarse nature of the wood, it does not lend itself to carving. Woodcarvings from hardwoods can be quite ornate whilst coconut woodcarvings tend to be of a more impressionistic nature.

This observation is significant, for the simple tieing of the roof to columns used for the structure of a building is a major factor in the ability of a building to withstand typhoon winds. In areas which put high value on the ability to carve well, such carvings are often incorporated into the structure of a dwelling, and in particular there are communities in the Pacific area which traditionally use carvings to provide the link between column and roof. How much this tradition has grown up because of necessity is unknown, but the survival of such structures when subjected to large wind forces may well have had a significant effect in establishing such a tradition.

Interestingly, similar types of construction exist in richer countries. In Japan, in response to a large earthquake problem a detail of such a connection in a pagoda is shown. The success of this detail for the response to earthquakes is already detailed, and a similar benefit for wind may also apply. Additionally, the Australian experience with Cyclone Tracy in 1974, convinced that country to adopt similar types of positive connection between columns and roof. Other factors which influence the ability of low cost housing to withstand cyclones is the preparation of the materials. Photos 1 and 2 show two extremes. In the former the wood and the walling are prepared with lacquer for protection, whilst in photo 2, this type of dwelling typifies a philosophy that an extreme event will destroy the dwelling and reconstruction will take place. In such areas it is common to expect that

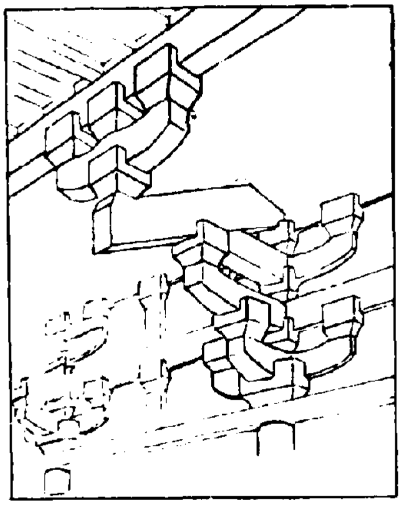

Figure 2 Detail of Pagoda in Japan

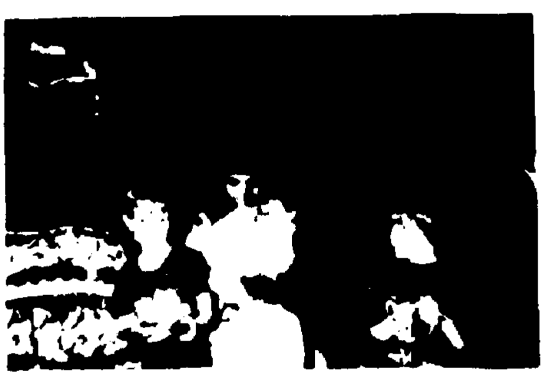

Photo 1 Well treated wood in low-cost housing

Photo 2 Basic tropical low-cost housing

PREPAREDNESS FOR TYPHOON HAZARDS

A recurring theme with damage occurrence is that the lessons of history are not learned. What is at issue is whether it is worthwhile for communities to put in the required effort to effect this design. There are, then four types of community, depending on their type of risk exposure. These are Short return period - High risk, Short return period - Low risk, Long return period - High risk, and Long return period - Low risk. In the following only the first three categories are considered.

The major communities of the Pacific region are divided into the three regions shown in Figure 1 which correspond to areas which are separated by lines of equal probability of occurrence of Cyclones (Typhoons). The dividing lines have been chosen as annual probabilities of occurrence of 0.1 and 0.04. More common terminology would insist that these represent average return periods of 10 and 25 years. These probabilities have been chosen because of the significance to the construction process.

For a twenty five year average return period under quite ordinary stationary random conditions, it should be expected that in any sequence of 2,500 years there would be 37 passages of 25 year periods in which none of these events occurred. It is also possible that there would be periods of 50 or 75 years without such events and this would not imply any change to the basic underlying statistic.

It is clear that a designer, noting the absence of such a probability 0.04 event for a period which spans his entire working life, and preceded by an period in which no such events were noted by his father, would be tempted to conclude that the risk had changed for the better. This would be an error since the basic underlying statistic would be unchanged. Periods of thirty and seventy five years are significant in that they represent approximately one and two working life-spans respectively. The occurrence or non-occurrence of a "design" wind during such periods profoundly affects the cultural view of the risk of damage caused by cyclones, and is accordingly fed into the style of construction.

For the Pacific region the following table categorises major communities in terms of these three risk areas:-

Zone 1 (Probability of occurrence > 0.1)

Vanuatu, Western Australia (North), Taiwan, Phillipines (North), Hong Kong, Hainan Island, Vietnam and South East China.

Zone 2 (Probability of occurrence between 0.1 and 0.04)

Japan (South), Korea (South), Phillipines (South), Queensland (North), New Caledonia, Fiji, Solomon Islands (South), Vietnam (South), and North East China.

Zone 3 (Probability of occurrence < 0.04)

Hawaii, Western Samoa, Tonga, Cook Islands, Tahiti, New Zealand (North), Northern Territory of Australia, Borneo (North), Irian Jaya (South), Papua New Guinea (South), Bali, Japan (North), Mexico (North), California (South), Guam, Korea (North), Java.

In general terms the communities listed above appreciate the cyclone risk well in zone 1, and somewhat in zone 2. However, it is in zone 3 that the major risks lie, since communities may have become unprepared through a loss of community memory. There are important exceptions to this general statement, both in zone 1 and in zone 3, which are discussed further below.

The response of a community, to the cyclone risk, by altering its' construction techniques is profoundly important to the actual risk incurred. This response can be manifested either by the use of traditional methods which have stood the test of time over longer than the living population's memory, or by the acceptance of standards from areas in which the actual risk is greater and which has altered its construction style.

Long Return Period - High Risk

Here we are principally considering those areas marked as zone 3. It is in this area that the risk of construction techniques not responding to the exigencies of the environment become extreme. However, the type of response in rich and in poor communities may well be quite different. This difference is caused by the weight put on tradition. The cases of Darwin in the Northern Territory of Australia and of Tonga are interesting to compare.

(i) Darwin, Northern Territories of Australia - zone 3

Darwin is situated on the northern coast of Australia. On the 25th December 1974 it was struck by Cyclone Tracy. A community of 46,000 people had established themselves in Darwin, and the local domestic dwellings largely reflected the desire for ventilation and cooling. Accordingly much of the housing was supported by wooden piles up to 2 or 3 metres long, allowing the passage of air underneath. Over 90% of domestic dwellings in Darwin were rendered uninhabitable by the passing of Cyclone Tracy.

Darwin had been previously struck by Cyclones in 1937 and 1897, and the local community which developed lacked any direct experience of the Cyclone risk. Undoubtedly, the Tracy event was traumatic for Darwinians, but the experience has been utilised to the full throughout Australia, and local construction practice has changed considerably through the efforts of many proponents. In particular the building regulations in Australia now fully reflect the experience of Cyclone Tracy, with Queensland being notable for the efficient enforcement of these new regulations.

(ii) The Kingdom of Tonga - zone 3

The Kingdom of Tonga spans a vast area from 15 to 23 degrees south of the equator in the region of the International Date Line. The northern parts of the Kingdom are more prone to the effects of Cyclone activity.

Tonga, although in zone 3, is close to the 25 year return period limit. The islands were hit by a cyclone in the late 1970's which caused extensive damage. However, in this case, the UK's Building Research Establishment became involved in the problem through the auspices of the Overseas Development Agency, and a system of low-cost cyclone resistant housing was introduced. In fact this style of housing had been developed for a similar problem in the Caribbean. A prefabricated housing system was designed and within one year all the houses destroyed had been replaced.

The housing in Tonga remains to this day. What has actually occurred in Tonga is a transfer of technology ultimately generated by one of the world's rich countries. This appears to have been a successful transplant, rather than that which took place in Darwin, which also involved a transplant of technology, although only geographic one.

(iii) The Solomon Islands

The Solomon Islands lie in the region of 5 to 12 degrees south of the equator in zone 3, although the southernmost islands extend well into zone 2 which is quite narrow in this area. Housing is traditional for the area with roofs composed of leaves, and the use of coconut wood is common.

On 18th May 1986 Cyclone Namu struck Honiara, the capital, causing damage estimated to be in the region of US$100 million. About 100 people were killed and over 90,000 were left homeless. Since that time numerous small cyclones have affected the islands. During 1992, two cyclones have struck the islands with one event on 2nd January causing extensive damage. Disruption to food supplies (grown locally) was also a considerable strain to the local population.

To take the case of the Solomon islands altogether can be a little misleading since the Solomons straddle all three zones. However, in such a circumstance it should be likely that the experience of the part of the community most affected would be likely to be transported to the other regions. The Solomon islands are clearly still undergoing a learning process. The fact that such a major disaster as Cyclone Namu, has not resulted in the vast reduction of damage when cyclones affect the islands, signifies a lost opportunity to learn from a previous experience.

Long Return Period - Low Risk

Identification of the extent of zone 3 is problematical. Firstly, the amount of information gathered on a world-wide basis is insufficient to give an accurate picture of the likelihood associated with very long return period areas. Secondly, the climate of the Earth is changing (Jeary), and these areas represent a potential for disaster.

In this category come those areas which are subject to cyclones very rarely, and which are at the extremity of the regions in which cyclones normally travel. It is probable then that the cyclones will have weakened considerably before reaching these locations. The movement of the ensuing cyclone, and its' possible intensifying, are not well understood. However, tentative conclusions about the movements of typhoons in the eastern Asia region suggest the following:

Deviations from a straight line of the track of a typhoon are caused by movements of the 500 hPa wind (Chan), the influence of nearby land, the influence of nearby weather systems or other cyclones, and differential sea surface temperatures in the area of the typhoon.

Additionally, typhoons reduce in intensity when they encounter a sea surface temperature less than 23 degrees Celsius or when they make landfall. The implication is that if sea surface temperatures are likely to rise then typhoons and cyclones will travel further than they have done previously, and areas that where previously outside zone 3 would be included inside.

There are some regions on the edges of zone 3, as it is currently assessed, where the consequences of the appearance of a typhoon could be rather severe. The following list gives an indication of where these occurrences could be :

Australia - Sydney and Perth - New Zealand- Auckland, Wellington and Christchurch - United States - Los Angeles and San Francisco - Marquesas Islands - Marshall Islands.

All of these locations are in areas in which the possibility of the appearance of tropical cyclones is largely discounted. This perception involves the occurrence of a risk that could be diminished greatly by some relatively simple measures. It is interesting to observe that these areas also largely occur in rich countries.

Direct evidence of a changing climate is difficult to establish, although some effects have been shown to be statistically significant, using the western north Pacific as the area for analysis, however, circumstantial evidence comes from a number of sources. Not least amongst these is the experience of Tahiti.

i) Tahiti - zone 3

French Polynesia consists of five archipelagos spread across 4.5 million square kilometers of ocean. The largest of these archipelagos is Tahiti, which covers an area of 1042 square kilometers. Tahiti lies in zone 3 near to the extremity of the zone (as currently listed). Experience of cyclones was non existent within the community memory until the end of 1982. Then between December 1982 and April of 1983 six cyclones devastated the area. Two years later, during 1985, 30% of all expenditure in Tahiti was for reconstruction.

Short Return Period - High Risk

Most of the areas with Cyclone wind return periods of less than ten years in the Pacific region are not heavily populated. The exception to this rule is the western pacific area which includes Hong Kong, the northern Phillipines (including Manila), and Taiwan.

Given that a stationary set of data exist then the occurrence of Typhoons should be sufficiently frequent that preventive measures such as design alterations will quite normally occur. Unfortunately, the changing climate noted previously seems to have a particularly severe effect in this zone. The effect manifests itself as a reduction of the probability of occurrence of typhoons in some areas (e.g. Hong Kong) and an increase in others (such as Japan and Korea). This effect is caused by the fact that many typhoons which previously would have crossed the coast of Southern China, are now recurving back into the Pacific and running up the coast to affect Japan, northern China and Korea. The significance of these movements has been shown to be statistically significant. Further, circumstantial evidence seems to show that the effect is deepening, since during the 1991 typhoon season Japan was affected by the passage of eight typhoons whilst the average number for one season is 2.6 for the period 1968 to 1988.

Unfortunately, the experience in Hong Kong suggests that there is an equally dangerous process which occurs, when the number of typhoons reduces. Observation of building sites in Hong Kong quickly leads to the conclusion that many potential projectiles are left in the open even when typhoons are very close and the population has been sent home as a precaution. Photo 3 shows a typical view of a construction site in Hong Kong, whilst Photo 4 shows the vulnerability of housing in San Francisco, although the damage shown was the result of the Loma Prieta earthquake, the same mechanism would undoubtedly be resent for wind actions.

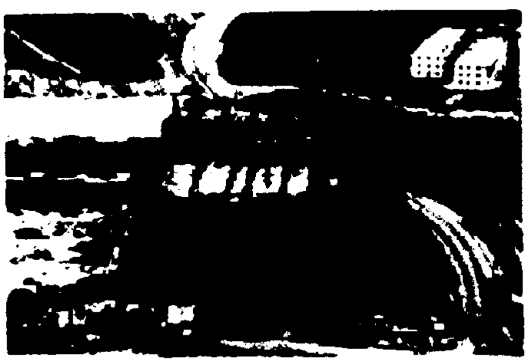

Photo 3 A construction site in Hong Kong

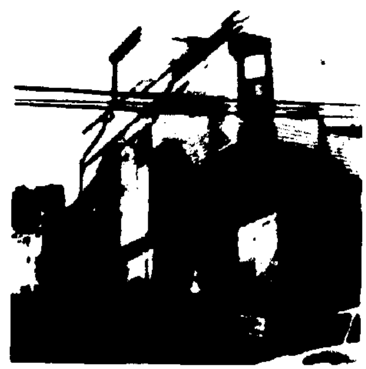

Photo 4 Damage to housing in San Francisco

WALKER's PAN PACIFIC CODE

Walker's suggestion that a universal simplified code of practice for the Pacific region could be adopted is basically a good one in that it may help to plan engineered buildings in the region better. However, as has been shown above, on closer examination, the potential for disaster appears to lie mainly in developed countries which do not consider the cyclone hazard to be particularly threatening. The introduction, in these regions of the new regulations introduced in northern Australia could avoid such problems. Walker's approach is far more appropriate to this region than that adopted, for example by the nations of the European Community. That experience suggests that all that results is an "average" code which is difficult for practitioners to use. If that is the experience of one multi-cultural environment then the suggestion is that the problems would be at least as difficult in the Pacific region.

In other areas of the Pacific region, low cost construction techniques can be very effective. However, the lessons of tradition and history must be combined, and in particular, positive connections between columns and roofs must be sufficiently durable, no matter what materials are used, to withstand the imposed wind forces or whether the country is rich or poor.

CONCLUSIONS

For the assessment of Cyclone activity areas of high risk in the Pacific area have been identified. These areas are generally unprepared for wind events that may become more common on a warming planet. The lessons of history and of tradition, derived from higher risk areas, have been suggested as suitable vehicles for the avoidance of disproportional damage.

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COMPUTATIONAL METHODS FOR ESTIMATING EXTREME WIND SPEEDS

Edmond D.H. Cheng and Arthur N.L. Chiu
Dept. of Civil Engineering
University of Hawaii at Manoa
Honolulu, Hawaii 96822, USA

ABSTRACT In order to provide a better method of estimating extreme winds at an ungaged site, computational methods of estimating 50-year or 100-year extreme winds from short-term data collected at various weather stations in a large region are being developed. In the current state-of-the-art, there are four approaches for estimating extreme wind speeds from short-term records. The first approach is based on the analysis of the largest monthly wind speeds of at least three years' data. The second approach is the determination of extreme wind speeds from a parent distribution. The third and fourth approaches are simulation models based on short-term continuous hourly wind speed records. Application of the methods will be presented.

1. ANALYSIS

In a region, extreme wind data obtained from locations with similar physiographic or meteorologic factors do exhibit some common characteristics. For example, if Type I distributions are assumed for the obtained extreme winds, similar slopes may be observed from the Gumbel lines. If the extreme wind speeds at these locations are reduced to dimensionless Gumbel lines, then these lines may be expected to be nearly coincident. This composite Gumbel line is then applicable throughout the entire region or in a subregion. The basic strategy of regional zoning for extreme winds in an area is to obtain a composite dimensionless extreme wind-frequency curve from wind stations in that area. With this dimensionless regional frequency curve, the extreme wind speed for a specific return period can then be obtained from multiplying the value from this curve by a mean annual extreme wind speed at a project site.

1.1 Basic Probability Functions

In the current state-of-the-art, there are four approaches for estimating extreme wind speeds from short-term records. The first approach is based on the analysis of the largest monthly wind speeds of at least three years' data (Simiu, et al., 1982). This method has been well tested at 36 weather stations situated in various locations in the United States, and it has proven to be practical and adequate. The second approach is the determination of extreme wind speeds from a parent distribution (Davenport, 1978). Using this method, extreme wind speeds can be estimated even from records shorter than three years. The third approach (Grigoriu, 1982) and the fourth approach (Cheng and Chiu, 1985) are simulation models based on short-term continuous hourly wind speed records. The selection of a proper approach is dependent upon the available data.

1.2 Homogeneity Test

The grouping of weather stations into regions or subregions was guided by a sequence of homogeneity tests. For each homogeneity test, permissible ranges of variation were determined by selecting $\pm$ 1.0 standard deviation of an appropriate probability distribution. In this paper, Type I distributions were assumed for representing the annual extreme wind speeds series. The Type I cumulative distribution function (CDF) of extreme winds may be expressed in reduced variate as:

$$F = e^{-e^{-w}} \tag{1}$$

with location and scale parameters equal to zero and unity, respectively.

Further,

$$w = \ln \ln \frac{1}{F(w)} = -\ln \ln \frac{T}{T-1} \tag{2}$$

and the standard deviation of the reduced variate w is

$$s_w = \frac{e^w}{\sqrt{n}} \sqrt{T-1} = \frac{e^w}{\sqrt{n}} \sqrt{\frac{1-F(w)}{F(w)}} \tag{3}$$

where w = reduced variate of Type I distribution of extreme winds;
$F(w)$ = CDF of w; s_w = standard deviation of w;
n = number of years of record; and T = recurrence interval.

1.3 Composite Frequency Function

A composite frequency function is constructed by taking median values from site specific extreme wind-frequency functions in a homogeneous region or subregion for the same return period. To make the composite frequency curve dimensionless, the median values are actually ratios of extreme wind speeds at various weather stations to the mean annual extreme winds at those stations.

2. APPLICATION

The proposed procedure has been applied to wind data collected on the Island of Hawaii. Long-term annual extreme wind records exist at the Hilo Airport, Honokaa, and Mauna Loa Observatory on Hawaii (Figure 1). For these stations, the annual series were constructed, and Gumbel distributions were fitted to these annual extreme data, respectively. However, most of the records collected at other locations at various times are of short periods or fragmented. Table 1 summarizes the wind data used in this paper. Due to the nature of the available data, the first and second approaches described in Section 1.1 were used for estimating extreme wind speeds from short-term records at various stations on the Island of Hawaii.

2.1 Assumptions and Validations

Type I distributions were assumed for representing both the annual and monthly maximum wind speed series. The validation of the first approach is measured by a standard deviation of the inherent sampling error of the historical records (Gumbel, 1958; and Simiu and Scanlan, 1986).

Predicted extreme winds of 25-year and 50-year return periods were compared with those obtained from the annual series in Table 1, columns (4), (5) and (7). Assuming the extreme winds from the annual series as the standard for comparison, column (7) shows that the deviations of the predicted 25-year or 50-year wind speeds are not more than 12 percent. This shows that good estimates of extreme winds can be confidently obtained from short records of monthly maxima.

In the second approach, we considered the parent distribution of the Rayleigh type, such as indicated in Figure 2. This rather crucial assumption was substantiated by an evaluation of the shape parameter of the Rayleigh distribution for long-term record stations. Thus, the extreme winds derived from the second approach are, of course, Type I in nature (Davenport, 1978).

In the second approach, a Rayleigh parent distribution has to be transformed to a Type I distribution of its extreme values. To accomplish this, first consider an extreme distribution derived from a parent Weibull distribution (Rayleigh distribution is a special case of Weibull distribution) as:

$$\text{CDF:}\ F(V) = \text{Exp}\left[mT(V/b^{*})^{c-1}\ \text{Exp}\left[-(V/b^{*})^{c}\right]\right] \qquad (4)$$

where m = a constant; T = return period

b^{*} = Weibull scale parameter; c = Weibull shape parameter

For annual extreme, $T = 1$ year, Eq. (4) becomes

$$F(V) = \text{Exp}\left[m(V/b^{*})^{c-1}\ \text{Exp}\left[-(V/b^{*})^{c}\right]\right] \qquad (5)$$

If Eq. (5) is approximated by Gumbel distribution, then Gumbel's parameters, α and u, can be obtained by solving:

$$m(u/b^{*})^{c-1}\ \text{Exp}\left[-(u/b^{*})^{c}\right] = 1 \qquad (6)$$

$$\text{and}\ \alpha = \frac{d}{dV}\left[-m\ (V/b^{*})^{c-1}\ \text{Exp}\left[-(V/b^{*})^{c}\right]\right] \qquad (7)$$

If Rayleigh distribution (a special case of Weibull distribution) is assumed, then $b^{*} = \sqrt{2}\ b$ and $c = 2$, Eqs. (6) and (7) become:

$$\left(m/\sqrt{2}\right)(u/b)\ \text{Exp}\left[-\frac{1}{2}\ (u/b)^{2}\right] = 1 \qquad (8)$$

$$\alpha = \frac{d}{dV}\left[-\left(m/\sqrt{2}\right)\ (V/b)\ \text{Exp}\left[-\frac{1}{2}\ (V/b)^{2}\right]\right] \qquad (9)$$

Solutions of Eqs. (8) and (9) were obtained by numerical methods.

The validation of the second approach was also tested on the Hilo Airport and Mauna Loa Observatory data (Table 1, columns (6) and (8)). Again, assuming the extreme winds from the annual series as the standard for comparison, column (8) in Table 1 shows that the deviations of predicted 25-year or 50-year extreme winds obtained from the second approach are within eight percent of those obtained form the annual series. These results show that the second approach gives very good estimates.

2.2 Results

All 14 stations were subjected to the initial homogeneity test. Four of these 14 stations (Upolu Point, Kahua Ranch, Waimea Airport and South Point) fell outside the range of one standard deviation of the reduced variate of the Type I distribution. Based on successive homogeneity test, three homogeneous subregions were formed for the Island of Hawaii (Table 2).

As shown in Table 2, the nearly overlapping feature of the Gumbel lines for the subregions of the Island of Hawaii indicates that a single region for the island may be considered. Excluding those stations which fell outside the permissible range of variation, the Gumbel lines for the Island of Hawaii as a whole confirmed this reasoning (Figure 3).

3. CONCLUSION

The preliminary results obtained from the application of this regional analysis procedure to long-term period, short-term period and fragmentary wind data collected on the Island of Hawaii are very encouraging. The dimensionless regional extreme wind-frequency curve serves as an effective means of estimating extreme winds for prescribed return periods at an ungauged project site.

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Table 1 Estimated extreme winds at various stations on the Island of Hawaii using the annual series, the first approach and the second approach (speeds in miles per hour and adjusted to 33 ft above ground).

Station (1)	Data Period* (2)	Return Period (years) (3)	V_a (4)	V_m (5)	V_R (6)	$\frac{V_a-V_m}{V_a}$ (%) (7)	$\frac{V_a-V_R}{V_a}$ (%) (8)
Hilo	1950-	25	43	-	42	-	2
Airport	1984	50	46	-	45	-	2
	1/57-	25	-	41	-	5	-
	12/60	50	-	43	-	7	-
	1/62-	25	-	46	-	-7	-
	12/65	50	-	49	-	-7	-
	1/72-	25	-	39	-	9	-
	12/75	50	-	41	-	11	-
Mauna Loa	1959-	25	67	-	62	-	7
Observ.	1984	50	72	-	66	-	8
	1/77-	25	-	-	-	-	-
	10/84	50	-	-	-	-	-
	1/60-	25	-	62	-	7	-
	10/68	50	-	66	-	8	-
	1/78-	25	-	59	-	12	-
	12/83	50	-	63	-	12	-
Bradshaw	1/64-	25	-	-	43	-	-
Army Air Field	12/68	50	-	-	47	-	-
Waimea	10/77-	25	-	56	-	-	-
	9/81	50	-	58	-	-	-
Keahole	3/79-	25	-	51	-	-	-
	10/82	50	-	55	-	-	-
	1/74-	25	-	-	48	-	-
	12/74	50	-	-	52	-	-
Honokaa	1920-	25	39	-	-	-	-
	1945	50	44	-	-	-	-
Kahua	10/77-	25	-	80	-	-	-
Ranch	10/81	50	-	84	-	-	-
Upolu	1/73-	25	-	-	60	-	-
Point	12/73	50	-	-	63	-	-
Waikoloa	1/77-	25	-	-	64	-	-
Mauka	3/78	50	-	-	69	-	-
Kapoho	1/77-	25	-	-	53	-	-
Beach	12/77	50	-	-	57	-	-
Kiolakaa	2/70-	25	-	-	51	-	-
	12/70	50	-	-	54	-	-
South	5/77-	25	-	-	76	-	-
Point	12/79	50	-	-	79	-	-
South	1/78-	25	-	-	44	-	-
Kona	12/78	50	-	-	47	-	-
Kailua-	1/66-	25	-	-	38	-	-
Kona	12/66	50	-	-	41	-	-

*Data periods begin with the first day of the starting month and terminate with the last day of the ending month.

V_a = wind speed from annual series.
V_m = wind speed from first approach.
V_R = wind speed from second approach.

Table 2 Dimensionless regional frequency curve for the Island of Hawaii

Sub-Region	Station	$\frac{V_{1.1}}{V_{2.33}}$	$\frac{V_{1.5}}{V_{2.33}}$	$\frac{V_{5}}{V_{2.33}}$	$\frac{V_{15}}{V_{2.33}}$	$\frac{V_{25}}{V_{2.33}}$	$\frac{V_{50}}{V_{2.33}}$	$\frac{V_{100}}{V_{2.33}}$
A	Honokaa	0.84	0.92	1.10	1.23	1.35	1.52	1.63
A	Hilo Airport	0.83	0.92	1.11	1.25	1.36	1.45	1.56
A	Kapoho Beach	0.82	0.92	1.11	1.26	1.32	1.41	1.49
B	Keahole Airport	0.89	0.94	1.11	1.27	1.33	1.43	1.53
B	Waikoloa Mauka	0.81	0.91	1.12	1.28	1.33	1.44	1.53
B	Bradshaw Army Air Field	0.76	0.89	1.15	1.34	1.41	1.54	1.65
B	Mauna Loa Observatory	0.78	0.90	1.14	1.38	1.49	1.60	1.74
C	Kailua-Kona	0.71	0.87	1.18	1.41	1.53	1.65	1.80
C	South Kona	0.77	0.89	1.15	1.33	1.43	1.53	1.64
C	Kiolakaa	0.81	0.91	1.12	1.27	1.35	1.43	1.52
	Median:	0.81	0.91	1.12	1.28	1.36	1.49	1.59

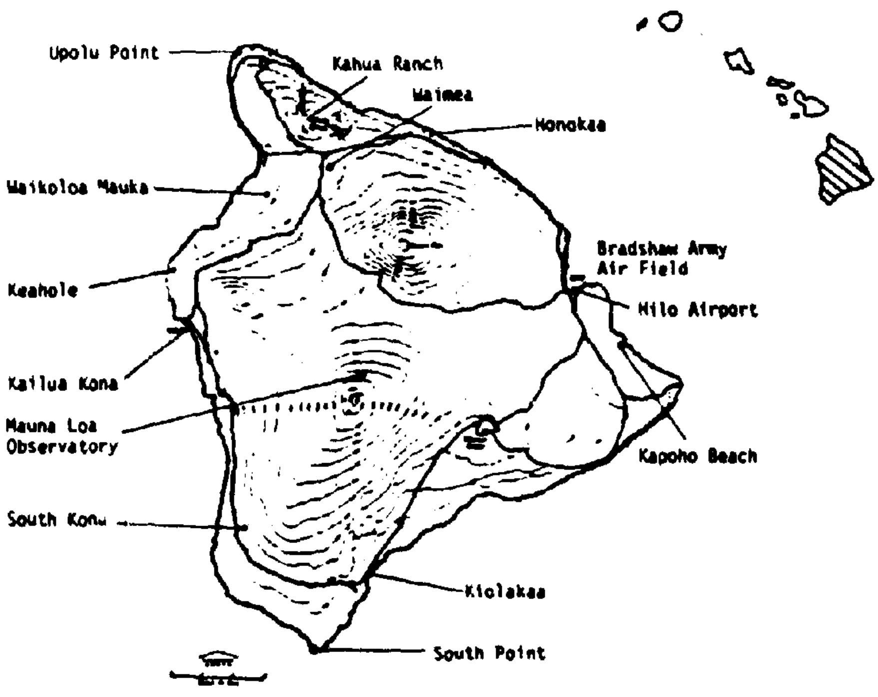

Figure 1 Location of Wind Stations on The Island of Hawaii, State of Hawaii

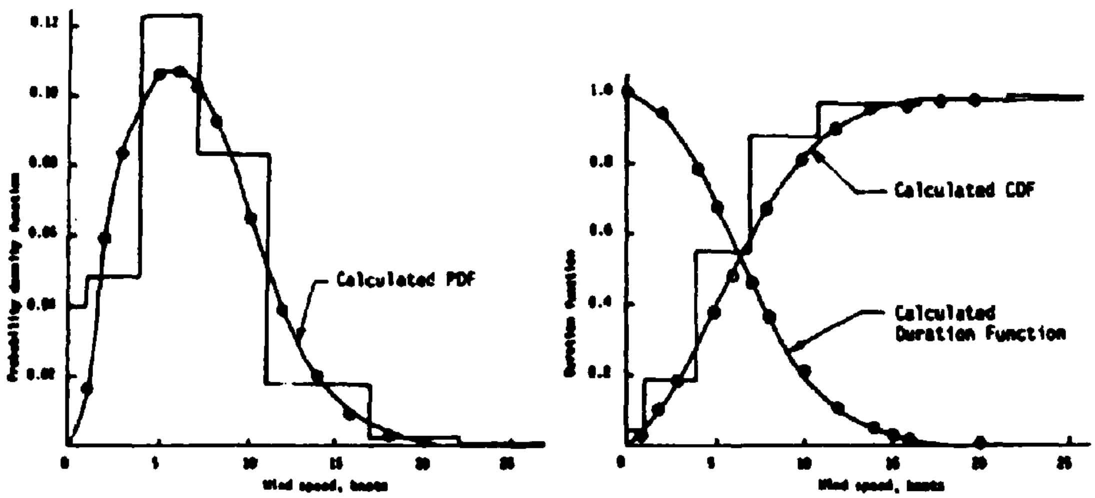

Figure 2 Rayleigh Distribution at Hilo Airport, Island of Hawaii

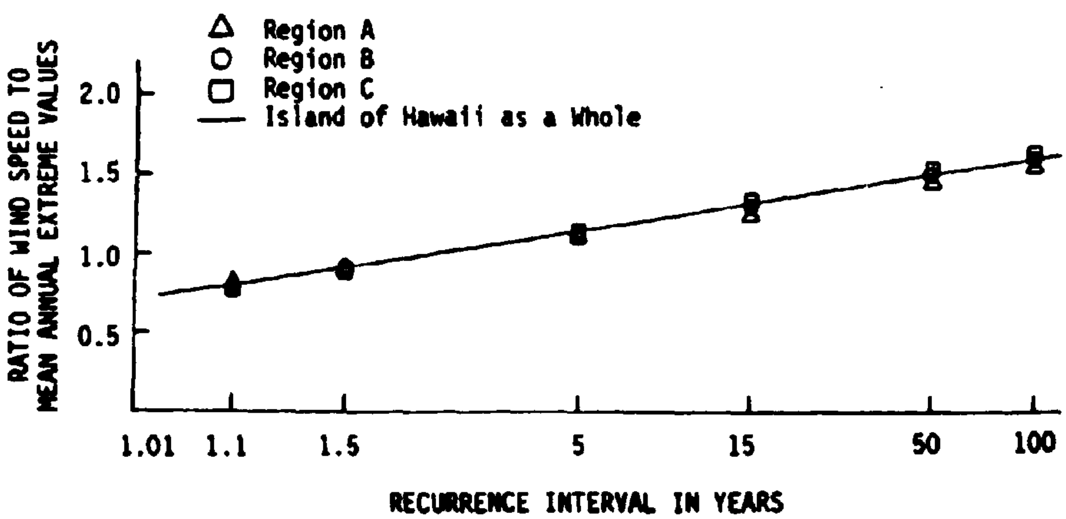

Figure 3 Type I Distribution of Regional Extreme Wind Speed Frequency Curves for The Island of Hawaii

SECOND US-ASIA CONFERENCE ON
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STRONG WIND DAMAGE TO HOUSES IN YANAGAWA CITY BY TYPHOON 9119

H. Ishizaki[1], J. Katsura[2], Y. Taniike[3] and T. Maruyama[4]
1. Professor emeritius, Kyoto University
2. Professor, 3. Associate professor, 4. Research associate
Disaster Prevention Research Institute
Kyoto University, Kyoto, Japan

ABSTRACT Typhoon 9119 struck the northwest area of Kyushu Island, Japan, during the afternoon of September 27, 1991. Significant damage occurred throughout the city of Yanagawa due to strong wind. Time-variations of damage to roofs were investigated using video camera records. The primary weakness of tile or sheet-metal roofing designs was clarified. It is estimated that the velocity of the maximum peak gust was at least 40m/s at 6m above ground level at the height of the eave of houses.

1. INTRODUCTION

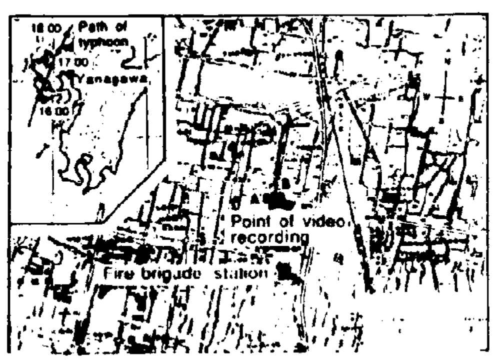


Fig.1. Location of Yanagawa city, the path of the Typhoon 9119 and detailed map of the center of the city.

Typhoon 9119 was first detected on September 16, 1991 near the Marshall Islands. During the afternoon of September 27, it came onshore at the northwest coast of Kyushu, the southern island of Japan taking its course as shown in Figure 1, and passed over to the Sea of Japan at a speed of approximately 100km/h. Despite the moderate rainfall, the typhoon was destructive due to the very strong wind that inflicted extensive damage throughout the country. Weather stations located at western and northeastern coastlines renewed their records of maximum wind speed and peak gust. The northwest part of Kyushu Island and Touhoku (northeast district of mainland Japan) were the two areas that suffered the most severe damage to houses and agricultural products.

Nowadays, many events are personally recorded on videotapes by the popularization of portable video cameras. In this particular incident, a three-hour video record taken from the top of a building in Yanagawa city located in the most severely attacked area of Kyushu provided a precious source of documentation of the progression of damage and its correlation with the time-variation of wind characteristics during the typhoon passage. This paper documents the analysis of damage caused to houses in Yanagawa city based on this video record together with field survey.

2. OVERVIEW OF DAMAGE

Yanagawa city is located on the west coast of Tsukushi-heiya in Kyushu, Japan. According to the report of damage by Typhoon 9119 published by the city office, 11,271 households comprising of 40,853 people received damage to their houses. The population of Yanagawa city was 44,480, meaning that 92% of its residents suffered. A total of 7,799 buildings were damaged, including 13 houses that were completely destroyed, 196 that were badly damaged, 7,546 that were partially damaged, as well as 44 non-residential buildings that were damaged to varying degrees. The damage was entirely due to strong wind and the cost of damage amounted to ¥6,665,319,000 ($51,272,000). Here, the word "completely destroyed" designates destruction of more than 70% of the total floor area or a destruction that costs greater

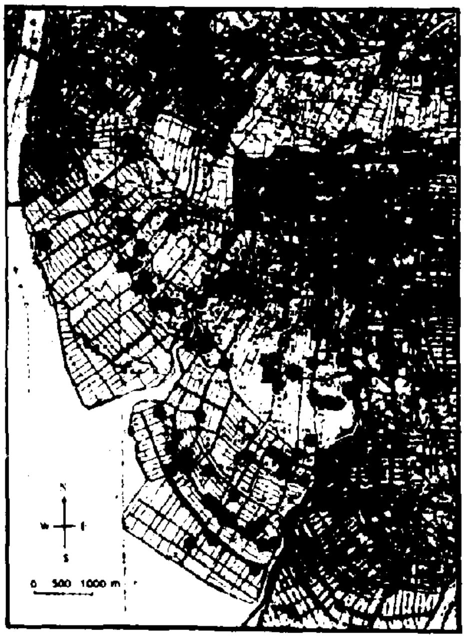


Fig.2. Distribution of damaged houses in Yanagawa city. ● shows a completely destroyed or a badly damaged house. (courtesy of Yanagawa city office)

than half of the initial construction cost; "badly damaged" if more than 20% of the total floor area is destroyed, and "partially damaged" if the damage is less than the aforementioned. The completely destroyed and badly damaged houses were spread over the city as shown in Fig.2. The data also includes an earlier damage caused by Typhoon 9117 which struck Yanagawa city on September 14. Typical damage to a house is shown in Photo 1.

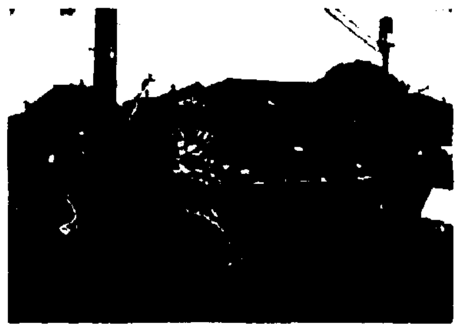


Photo.1 A completely destroyed house
(courtesy of Yanagawa city office)

3. METEOROLOGICAL ENVIRONMENT

The peak gust and the wind direction were measured by a propeller type anemometer set up at a 16m-high tower of Yanagawa fire-brigade station. The tower stands in the center of the city as shown in Fig.1, in the area surrounded by one- or two-storied houses and rice fields spread beyond.

The records of wind direction, wind speed and peak gust are shown in Fig.3. The eye of the typhoon was nearest to the city at around 17:00. Japan Standard Time (JST). The wind speed increased as the eye of the typhoon approached. The mean wind speed exceeded 20m/s at 16:20 JST. The strongest wind was recorded at 17:04 JST with a maximum wind speed of 27m/s and a peak gust of 49m/s. The wind direction was initially east and changed to south with the approach of the typhoon. At the time of the strongest wind, the wind direction was south-southeast. It changed to west-southwest after the typhoon left the city.

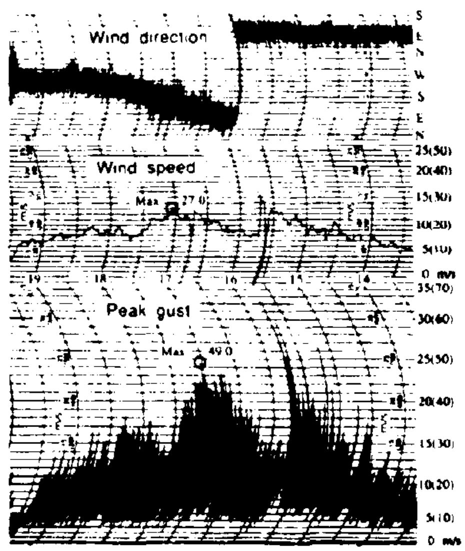


Fig.3. Records of wind direction, wind speed and peak gust at Yanagawa fire-brigade station. (The range of wind speed was changed in () at 15:44.)

4. MECHANISM OF DAMAGE TO ROOFING SYSTEM

In this section, the time-variations of the damage to roofs are examined based on the record by portable video camera. The video record was shot from the top of a thirteen-storied building located in the center of the city as shown in Fig 1. As this building was the highest structure, it was possible to record a bird's-eye view of the entire city during the three hours of the passage of the typhoon. Damage to a tile roofed house and a sheet-metal roofed house located at the foot of this building are analyzed.

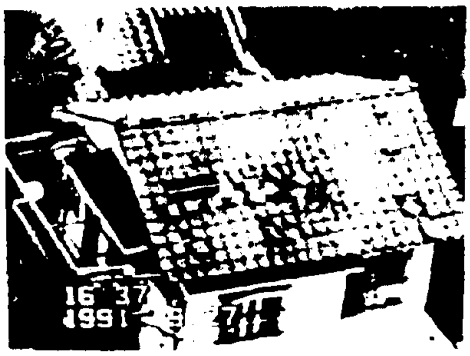
4.1 Damage to The Tile Roofing

The two-storied wooden house with tile roofing was located at point A in Fig.1. Between this house and the building where the video was recorded, there was a street and a canal together about 24m wide running north to south. The roof of the house was damaged by Typhoon 9117 two weeks earlier and it had just been renewed. The gable wall was about 4m wide, facing south and perpendicular to the canal. The height of the eaves covering the second floor was about 6m from ground level and overhanging 0.7m from the wall. The pitch of the roof was about 0.6. This house stood 800m northeast from the fire-brigade station. As the anemometer was set at a height of 16m, i.e. about twice the average height of the surrounding houses, the record of wind at the station were considered to represent the wind speed of the approach flow. The street and the canal were located windward of the house. The wind speed at the height of the eaves was calculated as 0.9 of the recorded value at the fire-brigade station, assuming that the ground surface was smooth and the exponent of the power law was 1/7. All of the wind data depended on the record at the fire-brigade station at the respective time points shown on the video records.

The sequence of damage to the tile roof is described in Photo.2a~d. At 15:15, the windward window was broken by a tile blown from elsewhere but the roof was still intact (Photo.2a). At 16:29, two or three tiles on the east-side roof (Photo.2b, lower left of the roof) were moved. At this time, the wind speed was about 21m/s, the peak gust about 34m/s, and the wind was blowing perpendicular to the eaves from east-southeast. After this, the tiles on the east-side roof near the windward eave and verge as well as some on the center portion were moved or blown off (Photo.2c at 16:37). Finally, about one quarter of the tiles at the windward region of the east-side roof was blown off and the

a. time : 15:15
wind speed : 10m/s
peak gust : 18m/s
wind direction : east

b. time : 16:29
wind speed : 21m/s
peak gust : 34m/s
wind direction : east-southeast



c. time : 16:37
wind speed : 24m/s
peak gust : 42m/s
wind direction : southeast

d. time : 16:45
wind speed : 20m/s
peak gust : 46m/s
wind direction : south-southeast

Photo 2 Time variation of damage to the tile roofing of a wooden house. The values of wind speed and direction are of the data at the fire brigade station. The left gable wall of the photograph faces south. (courtesy of Mr. M. Koga)

ceiling underneath was broken (Photo.2d at 16:45). At this point the wind speed was about 20m/s, the peak gust about 46m/s, and the wind was blowing vertical to the gable walls from south-southeast. Few damage occurred beyond this point and the roof on the west side remained intact.

The sequential events leading to failure of the tile roofing could be analyzed as follows. Prior to the attack of Typhoon 9119, the tiles had been fastened to the roof board at the eaves, verges and ridgeline to prevent similar damage received by the earlier typhoon 9117. Firstly, the unfastened tiles near the windward eave and verge began to move as it received a lifting load created by the increase in separated flow speed at the roof edges characterized by negative pressure. The damage spread out towards the center portion of the roof. As a consequence of the increase in the internal pressure caused by the broken windward window combined with the negative load on the tiles, the total lifting force on the roof increased and the roof board was blown off together with the ceiling. To prevent this kind of damage, the tiles should have been fastened not only at the edges of the roof but also every two or three rows at the center. The window shutter should be closed to prevent damage by objects carried by the wind.

The reason why the roof on the west side receive very few damage owes to the design of tiles and the way they were placed. As shown in Fig. 4, the typical design of a tile used in Japan is not symmetrical and the roofing is done by overlaying the left and bottom edges on the neighboring tiles. Because of this structure, the tiles are easily blown over by strong upward winds and/or winds from the left. Therefore, when the wind blows perpendicular to the gable wall, the roof on the right side is generally weak. This structural weakness should be alleviated by increasing the number of tiles that are fastened to the roof board or by filling the gaps between the tiles with plaster.

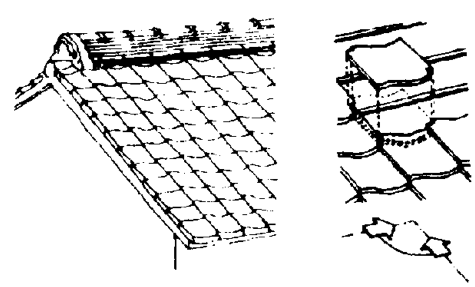
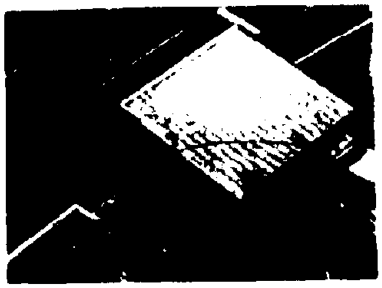


Fig.4 A typical tile roofing system in Japan. (Tiles are weak with the blows from the directions shown by the arrows.)

A wind tunnel simulation of a similar failure was conducted by "Okada, 1988". The results of experiments using a gable roof with tiles fastened to the roof board at the eaves, verges and ridgeline indicated that this design is not tolerant against wind forces greater than 40m/s average wind speed. The record at the fire-brigade station indicates that there actually was a peak gust stronger than 40m/s at the time the roof was blown off.

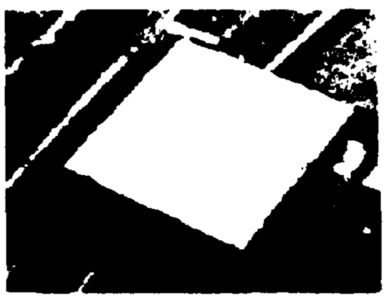
4.2 Damage to The Sheet-Metal Roofing

A two-storied wooden house with a sheet-metal roof (Photo.3) stood at point B shown in Fig.1. This house had a shed roof. The western eave was about 5.5m wide and about 7m high, facing the road and the canal. The pitch of the roof was about 0.2. The southern gable wall was 5.5m wide. The overhang of the verge was about 40cm from the

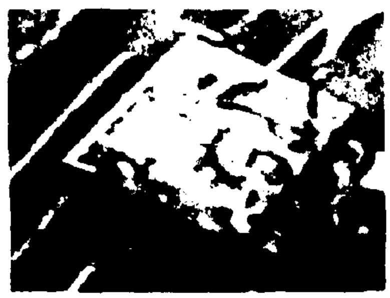


a. complete view

d. after 11/30 seconds

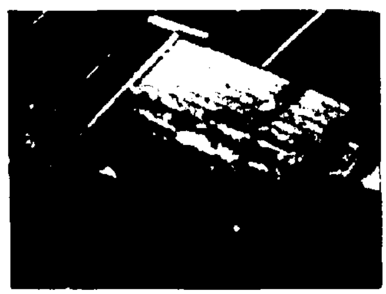


b. at 17 : 07



e. after 22/30 seconds

c. after 6/30 seconds



f. at 17 : 12

Photo.3 Time variation of damage to the sheet-metal roofing of a two-storied wooden house. The wind blows from the down left side of the photograph to the upper right. (courtesy of Mr. M. Koga)

gable wall as shown in Photo.3a.

The wind ripped away a part of the sheet-metal roof at around 17:07. starting from the southern windward side of the verge. Initially, the sheet-metal began to form corrugations and was slightly lifted up at the windward edge (Photo.3b). The next moment, about two-thirds of the roof began to rip off (Photo.3c~e). The entire incident lasted for about one second as recorded on the video tape.

As the wind velocity increased above the verge, it exerted a negative load on the windward surface and lifted up the verge, creating a gap between the sheet-metal and the roof board. The failure was triggered by the wind entering this windward opening. This kind of failure can be prevented by fastening the sheet-metal roof firmly to the edges of the eaves and verges.

5. CONCLUSION

The damage to houses in Yanagawa city caused by the Typhoon 9119 was surveyed. The damage to houses was spread out all over the city and was mainly caused by the strong wind. Time-variations of damage to roof were investigated using video camera records. The sequential events leading to failure of the tile and sheet-metal roofing system was clarified. The damage was estimated to be caused by a maximum peak gust greater than 40m/s. For the prevention of the damage to the tile roofing, it should be effective to increase the number of tiles that are fastened to the roof board or to fill the gaps between the tiles with plaster. The window shutter should also be helpful in protecting against breakage by objects carried by the wind. Sheet-metal roofs should also be fastened firmly onto the edges of the eaves and verges for prevention of failure.

ACKNOWLEDGMENTS

The authors are very grateful to Mr. M. Koga, president of Yanagawa High School, for his offer of the video tape recording, and Mr. T. Sinkai, manager of administration of Yanagawa city office, for his kind arrangement for our field survey. The information of Mr. T. Koga, senior director of NHK - Fukuoka Bureau, are also greatly acknowledged. Special credit is due to the individuals of Yanagawa city office, the fire-brigade station and other organizations for their contribution in gathering field data.

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W14-1

SECOND US-ASIA CONFERENCE ON
ENGINEERING FOR MITIGATING NATURAL HAZARDS DAMAGE
Yogyakarta, Indonesia
22-26 June 1992

NEW TECHNOLOGY APPLICATIONS FOR IMPROVED SEVERE STROM WARNINGS

J.H. Golden
NAA/Oceanic & Atmospheric Research
Silver Spring, Md. USA 20910

ABSTRACT We shall treat some of the current and future technologies in weather forecasting, especially as related to extreme winds and floods. The importance of accurate forecasting and the warning issued for mitigating damage and fatalities from these two natural hazards have often been underestimated. In the United States, flash floods account for more deaths and damage on average yearly basis than any other natural hazard, including tornadoes and hurricanes. The National Weather service during the deacade of the 1990's has begun an ambitious Modernization and Associated Restructuring Program. The cornerstone of this program is the field deployment, new underway, of some advances technological tools that will permit increased staff productivity and more accurate and timely forecasts and warnings. These new technologies include the Next Generation Radar (NEXRAD, WSR-88D Doppler), wind Profilers, Automated Surface Observing Systems (ASOS), and an interactive processing and display system, AWIPS-90. The capabilities and limitations of each these systems are assessed, along with examples of actual data sets from recent windstorms and flood events. Planned future enhancements to these observing systems will be described. Some specific potential applications to the improved tracking and warnings of typhoons and flash floods will be described. The importance of establishing "ground truth" networks of automated sensors, especially anemometers and rain gauges, along with trained storm spotters will be highlighted from recent experiences in the U.S. and elsewhere.

INTRODUCTION

As of early 1992, the Weather Surveillance Radar-1988 Doppler (WSR-88D) contractor, UNISYS Corp., has installed 10 radars, mainly in the SE and Central U.S. The same contractor has nearly completed installation of the 30-site Winds Profiler demonstration network which is also clustered primarily over the Central U.S. (additional sites are at Maynard, MA and Homer, AK). Finally, ASOS surface installations are proceeding rapidly, such that by the time of this Confrence, there should be about 80 systems installed (most of these at non-towered airports).

NEXT GENERA' /EATHER RADAR (NEXRAD)

NEXRAD is a replacement .. system for the current reflectivity-only weather radars, including the aging NWS network WSR-57's. The planned NEXRAD network over the continental U.S. will have 113 units; in addition, another 46 systems will be installed, some of these in Alaska, Hawaii, the Carribean and at the U.S. military bases in Asia and the western Pacific. Some of the limitations of the initially-deployed WSR-88D system were summarized by Golden and others (1990).

Some of the new operational and research applications of the NEXRAD system are illustrated in Figs. 1a, b and 2. (The original data is in color and similiar examples will be shown at the Confrence). Figure 1 shows an intense mesoscale winter cyclone that developed off the mid-Atlantic coast and moved slowly northwestward up the Chesapeake Bay on January 4, 1992. Figure 1a, b is a time-sequence of WSR-88D reflectivity and velocity, respectively, from the Sterling, VA, WSR-88D. The data are at 1 km resolution and the first pair shows the intense convective banding and even the suggestion of an "eye" near the circulation center which later moved directly over the Washington, DC metropolitan area (arrows). At the same time, Fig. 1b pair shows that peak radial velocities (at low levels) in the winter cyclone exceeded hurricane force; surface measurements earlier in the storm's lifetime indicated peak gusts of up to 80 mph over the Eastern Shore of Maryland. Flash flood, high wind and high surf warnings were issued by NWS for specific areas in Fig. 1 and most verified. Figure 2 (a,b) illustrates excellent examples of the WSR-88D hydrology product from the Norman, Oklahoma site during the 1992 severe thunderstorm season. The Precipitation Processing System software automatically sums rainfall estimates derived from reflectivity for one-hr, 3-hr, and storm-total accumulations. This is illustrated in Fig. 2a. which shows the state and county boundaries with the contoured estimated rainfall totals over an 8-1/2 hr period ending at 0150 GMT, April 18, 1991. Actual raingauge total rainfall amounts corresponding to the time-period sampled by the WSR-88D are plotted on the map as well and in general there is good agreement. Fig. 2b shows another case with Storm-Total estimates of nearly 10 inches over 12 hours.

The WSR-88D system appears at this juncture to be one of the most advanced weather radar systems currently being deployed in the world. The built-in fault isolation and diagnostics software and advanced modular hardware are designed to ensure ease of maintenance and high operational availability of the system. The primary limitations of the system appear to be the maturity of some of the meteorological algorithms, the need to develop others, and residual clutter from certain types of non-precipitation echoes. Research and development will be conducted to overcome these limitations. Potential future upgrades include the addition of polarization diversity for improved hail detection and rain/snow discrimination. Among the algorithms being considered for addition to the WSR-88D system include the Flash-Flood Potential (see Kelsch, 1989), tropical cyclone, improved wind analysis techniques to derived the total wind vector, improved hail and mesocyclone detection techniques, and snow estimation, and others.

WIND PROFILER DEMONSTRATION NETWORK (WPDN)

The wind Profiler is a ground-based remote sensing system designed to produce near all-weather, continuous measurments of wind. It is essentially a UHF clear-air radar with sufficient sensitivity to detect the backscatter from radio refractive-index irregularities caused by turbulance. The feasibility of using wind Profilers at various wavelenghts for a wide variety of meteorological applications has been pioneered by NOAA's Aeronomy and Wave Propagation Laboratories, and reviewed by Golden and others (1986). The vertical profile of the horizontal wind can be determined from Doppler frequency measurements on the backscattered signal. NOAA has just finished deployment of a demonstration network of 31 wind Profilers, concentrated in the central U.S. The primary performance specifications, as defines by NOAA, are given by Chadwick and Hassel, 1987. In addition to horizontal winds measured from 0.5-16 km heights, these 404 MHz Profilers also take measurements of the vertical wind component and reflectivity overhead. An excellent example of the hourly profiles of wind observed by the Profiler is given in Fig. 3. The data are plotted ini the form of a time-height section, derived from hourly averages on January 7, 1992 at Platteville, CO. These data were taken during a developing intense winter cyclone over Southern Rockies.

An operational and research evaluation of the Profiler data from the WPDN is underway. Early results are impressive for tornado and severe thunderstorm forecasting, as well as for optimum routing of aircraft. One of the few limitations of the present WPDN wind Profilers, partly due to the 404 MHz frequency, is that wind profiles cannot be obtained below 0.5 km.

AUTOMATED SURFACE OBSERVING SYSTEMS (ASOS)

The third major new technology being deployed to support the NWS Modernization is ASOS. The ASOS development to automate most surface weather observing functions was made possible by the advent of new reliable and sophisticated sensors and computer technology. Moreover, the three participating agencies, National Weather Service/NOAA, Federal Aviation Agency and Department of Defense are deploying from 900 to as many as 1,700 ASOS systems at airports across the U.S. during the 1990's. Every ASOS unit will contain the following basic set of sensors (in addition, other sensors, for example hail and snow-depth, are under development and may be added later) : (1) Ceilometer Sensor, one or possibly two, (2) Visibility Sensor, one or possibly two, (3) Precipitation Identification Sensor, (4) Freezing Rain Sensor, (5) Pressure Sensors (two or three, depending on airport size), (6) Ambient Temperature/Dewpoint Temperature Sensor, (7) Anemometer (F420-type rotating cup and vane for windspeed and direction), and (8) Precipitation Accumulation Sensor (Heated Tipping Bucket Gauge). The wind sensor will be mounted on 10 m towers in most cases. The ASOS wind algorithm uses a 2-minute period to obtain average wind direction and speed. The ASOS system measures these wind parameters <u>once every second</u>. ASOS will use <u>5-sec running averages</u> to compute gust speeds and direction. Current technology cannot replace all of the human observations which make up present Surface Aviation Observations (SAOs). Therefore, some of these may be provided by local manual augmentation or by using complementary technologies, such as WSR-88D and geostationary satellite data, with the ASOS observation.

CONCLUSION

When fully implemented, the ASOS will more than double the number of full-time surface aviation weather observing locations across the U.S. Moreover, many of the smaller airport sites with human observers now cease taking surface observations after dark, and ASOS will operate continuously. The ASOS network, coupled with other Federal and State surface sensor networks (such as the 110-site array being deployed across Oklahoma), will provide an unprecedented opportunity to perform mesoscale analyses for monitoring and "nowcasting" fast-evolving severe weather episodes. The combination of WSR-88D and wind Profiler data will provide continuous vertical wind profiles in nearly all weather regimes, ranging from clear to stratiform rain conditions (the WSR-88Ds obtain wind profile estimates every 5-6 min, see Golden and others, 1989). Taken together, these new technologies will as greater lead-times. Whereas most tornado warnings in the U.S. using current radar technology only have a few minutes lead-time, operational tests have shown that the WSR-88Ds will increase the lead-time to 20-30 minutes for larger tornadoes. Similarly, the detailed WSR-88D precipitation estimates and Storm Total estimate maps with detailed river/streambasin map backgrounds on the NEXRAD displays will allow more timely and precise flash flood/flood warnings.

ACKNOWLEDGEMENTS

The WSR-88D figures were kindly supplied by Don Burgess, WSR-88D Operational Support Facility, and the wind Profiler data from Dr. Russ Chadwick, NOAA Profiler Program Office.

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Sterling,VA NEXRAD reflectivity(left) & radial velocity(right), at 1400 & 1530 UTC, January 4, 1992.

Fig. 2a. Norman, OK NEXRAD Storm-Total rainfall map over 19 hrs,4/17/91

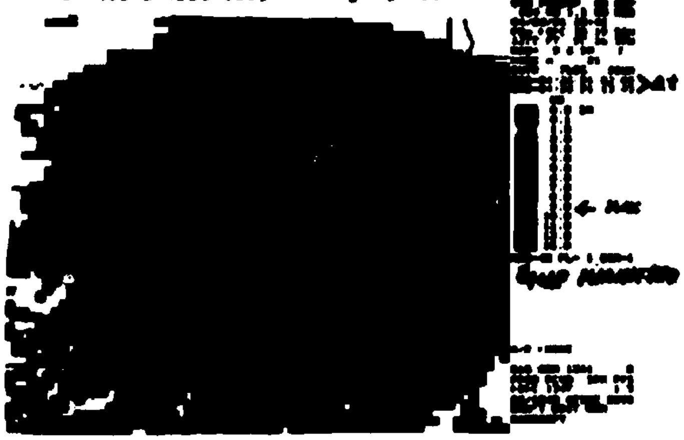

Fig. 2b. Norman, OK NEXRAD Storm-Total rainfall map (4X magnified) over 12 hrs, 4/26/91--peak rain est.~9.6 in.

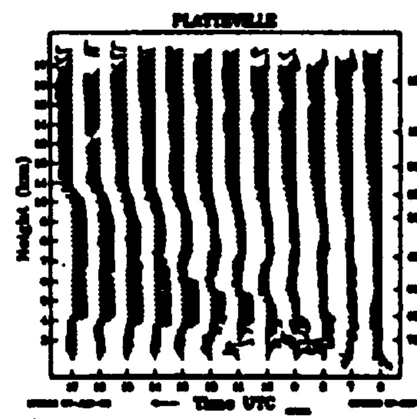

Fig. 3. Time-height section of Platteville,Colo 404 MHz Profiler winds, 06-17Z, Jan. 7,1992.

W15-1

SECOND US-ASIA CONFERENCE ON
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DETERMINATION OF WIND EFFECTS ON AND AROUND TALL BUILDINGS

H.F. Cheong, T. Balendra and S.L. Lee
Centre for Wind-Resistant Structures
Department of Civil Engineering
National University of Singapore

ABSTRACT In this paper, an aeroelastic model which simulates the shear flexure mode using distributed mass and stiffness is described. In order to obtain the distribution of loads along the height of the building, an experimental technique is presented herein where the fluctuating pressures from the aeroelastic model are sampled simultaneously from two tappings at a time. The measured data are later converted into the frequency domain in the form of auto and cross power spectral densities for the computation of modal forces. From which, the acceleration at any height and hence the variation of shear and moments along the height of the building are determined.

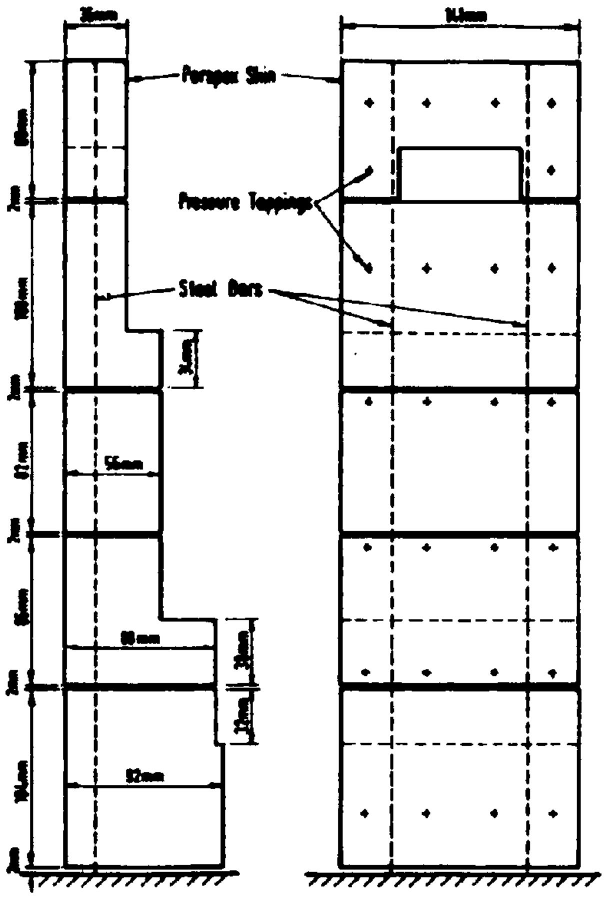
INTRODUCTION

For low rise buildings or other stiff structures, the wind induced structural loads may be determined by pneumatic averaging technique (Surry and Stathopoulos, 1977). However this method requires simultaneous sampling from several pressure transducers at a suitable scanning rate. A simpler method which requires measurement from only two pressure transducers at a time is the covariance integration method (Holmes and Best, 1981), which uses the measured covariance matrix containing the information on the statistical correlation between the fluctuating panel loads over the building. As both methods neglect any resonant dynamic effects, they are not suitable for tall and slender buildings which are sensitive for dynamic action of wind. For such buildings an experimental technique is presented herein where the fluctuating pressures from an aeroelastic model are sampled simultaneously from two tappings at a time. The measured data are later converted into the frequency domain in the form of auto and cross power spectral densities for the computation of modal forces. From which, the acceleration at any height and hence the variation of shear and moments along the height of the building are determined.

AEROELASTIC MODEL WITH SHEAR-FLEXURE MODE

Tall buildings generally comprise of walls and frames. Under lateral load, the walls deflect in a flexural mode while the frames deflect in a shear mode. When both frames and walls are tied together by rigid floor slabs, the building deflects in a shear-flexure mode. In this study an aeroelastic model with distributed mass and stiffness was constructed to simulate the shear-flexure mode of the prototype. As shown in Figure 1, the outer shell, made of perspex material, was supported by two steel bars through horizontal diaphrams. For the steel bars to deflect freely without any constraint from the outer shell, the latter was made of several segments instead of being a single unit. The gaps between the segments were covered by flexible viscoelastic tape which provided the damping to the model. The scale of the model to the prototype is 1 : 400. The mode shape and mass of the model along the height are given in Table 1.

In order to measure the fluctuating pressures on the model during along wind motion, pressure taps were provided at seven levels both on windward and leeward sides, as indicated in Figure 1. The positions of the pressure taps were judiciously selected to capture the representative pressures along the height of the model. To determine the frequency and damping of the model, an accelerometer was mounted inside the model.



(a) Side Elevation (b) Front Elevation

Figure 1 Aeroelastic model

Table 1. Mass and Modeshape of the Aeroelastic Model

Height (mm)	Mass (kg/m) per unit height	Mode shape	Height (mm)	Mass (kg/m) per Unit height	Mode shape
0	5.25	0.0	300	2.06	0.506
50	5.25	0.02	350	2.06	0.642
100	2.81	0.075	400	2.06	0.782
150	2.81	0.157	450	2.06	0.921
200	2.31	0.259	475	2.06	1.0
250	2.31	0.377			

SHEAR AND MOMENT IN THE MODEL

As shown in Figure 2, let M_1 (x_1, z_1) and M_2 (x_2, z_2) be two points on the model with tributary area $dx_1\ dz_1$ and $dx_2\ dz_2$ respectively. For along wind motion in Y direction, using only the first mode

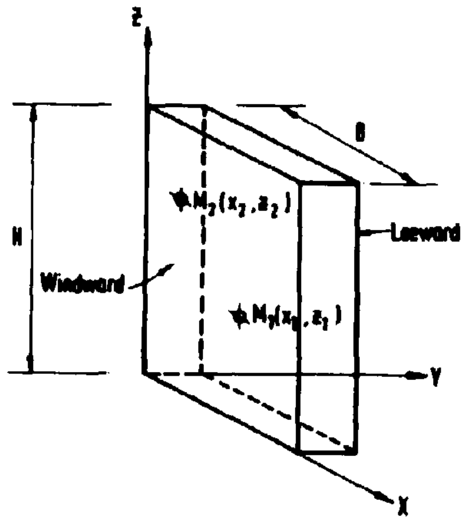


Figure 2 Position of two points on the model for the computation of the cross power spectral densities

$\phi(z)$, the modal force power spectral density is given by

$$S_{F^*}(n) = \int_0^B \int_0^B \int_0^H \int_0^H \phi(z_1)\phi(z_2) S_p(x_1, z_1, x_2, z_2, n)\ dx_1\ dx_2\ dz_1\ dz_2 \tag{1}$$

in which H and B are the height and width of the model, and $S_p(x_1, z_1, x_2, z_2, n)$ is the cross power spectral density of pressures between points M_1 and M_2, which may be expressed as

$$S_p(M_1, M_2, n) = S_p(M_1^W, M_2^W, n) + S_p(M_1^L, M_2^L, n) + 2S_p(M_1^W, M_1^L, n) \tag{2}$$

where the superscript W and L denote windward and leeward sides respectively. As the cross correlation between windward and leeward pressures are expected to be small, the last term in eq. (2) can be neglected. The background displacement σ_B and dynamic displacement σ_D are obtained from

$$\sigma_B(z) = \left[\frac{\phi^2(z)}{(m^*)^2\ (2\pi\ n_m)^4} \int_0^\infty S_{F^*}(n)\ dn\right]^{1/2} ; \tag{3a}$$

$$\sigma_D(z) = \left[\frac{\phi^2(z)}{(m^*)^2\ (2\pi\ n_m)^4} \left(\frac{\pi\ n_m}{4\ \xi_m}\right) S_{F^*}(n_m)\right]^{1/2} \tag{3b}$$

where n_m is the fundamental frequency of the model, ξ_m the damping ratio in the fundamental mode of the model and m^* the generalized mass.

The r.m.s. shear and moment at any height z_e of the model is obtained by integrating the inertia forces. Using an appropriate peak factor, the most probable maximum base shear and base moment can be determined.

EXPERIMENTAL RESULTS

The aeroelastic model was mounted on a wind tunnel which has a test section of 1 m height, 2 m width and 16 m length. The atmospheric boundary layer was simulated to match the velocity profile, turbulence intensity and the power spectral density of the longitudinal component of the turbulence of the natural wind (Balendra and Nathan, 1988). The experimental setup with surrounding building is shown in Figure 3.

From the free vibration response of the aeroelastic model, the fundamental frequency and damping are found to be 26 Hz and 2.5% respectively. As the frequency of the prototype is .183 Hz, the time scale is 142. Thus, for the chosen length scale of 400, the velocity scale is 2.82. The design hourly mean wind speed for the prototype at 500 m height is taken to be 34 m/s with a power law coefficient of .3.

As the computation of cross power spectra of pressure signals requires simultaneous sampling, sample and hold hardware device was used to sample two pressure tappings at a time, at a sampling speed of 200 Hz for a duration of 26 sec, which corresponded to approximately 1 hour in the prototype. Because of symmetry, the pressures were sampled from one half of the structure. Altogether 13 auto and 78 cross combinations of pressure tappings from the windward side and same number from the leeward side were scanned. The measured signals were corrected in the frequency domain for the distortion caused by the tubing system.

Figure 3 The aeroelastic model and surrounding buildings in the wind tunnel

The modal force spectra from one half of windward side and one half of the leeward side are depicted in Figure 4. The total modal force spectrum is also depicted in the same figure. It is found that the variance of the leeward force is about 60% of the windward force. By multiplying the modal force spectrum by the transfer function of the model, the displacement spectrum is obtained.

The variance of the measured modal force spectrum is found to be $4.06 \times 10^{-2}\ N^2$. Using the generalized mass of the model computed from the mass and modeshape given in Table 1, the background displacement of the model at the tip is found to be .042 mm which corresponds to 16.89 mm in the prototype.

The measured power spectral density of modal force at 26 Hz is $.912 \times 10^{-4}\ N^2/Hz$. From eq. (3b), for 2.5% damping, the dynamic tip displacement of the model is found to be .057 mm which corresponds to

22.8 mm in the prototype.

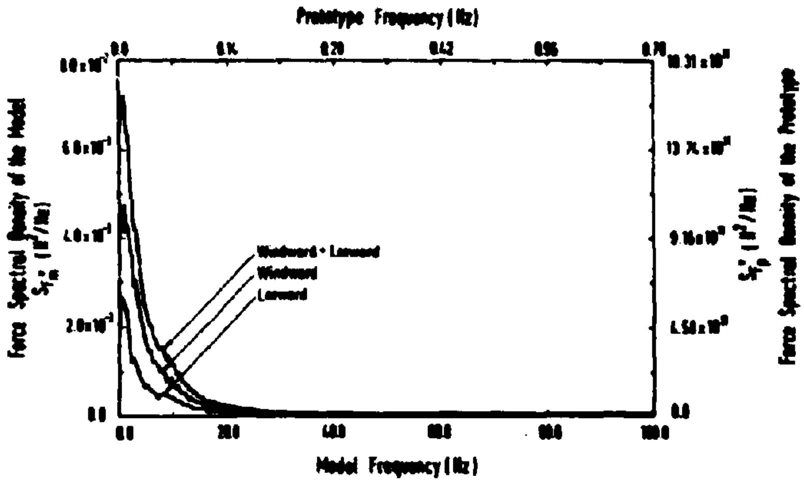


Figure 4 Power spectra of modal force in model and prototype scales

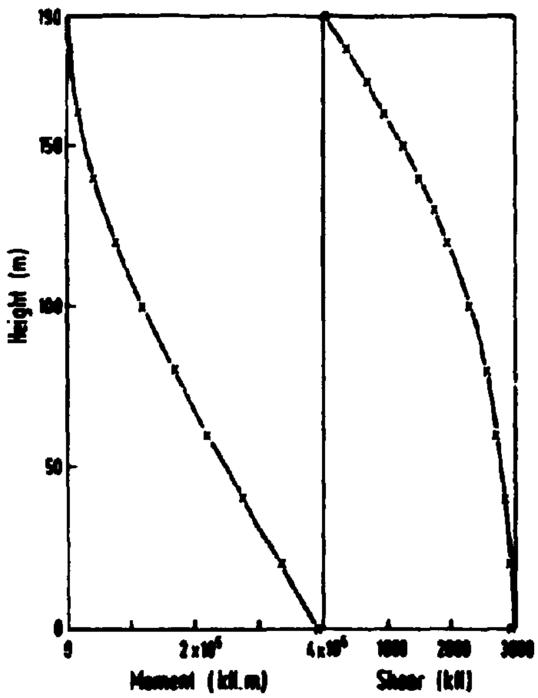


Figure 5 Variation of moment and shear forces along the height of the prototype

Using the peak factor corresponding to a narrow band excitation, the peak dynamic tip displacement and tip acceleration for the prototype are .086 m and .113 m/s^2. From the peak acceleration of the model and the data given in Table 1, the variation of moment and shear along the height of the model are obtained. The corresponding values for the prototype are presented in Figure 5, where the dynamic base moment and dynamic base shear are 393,012 kNm and 2,935 kN respectively. From the mean pressures at various pressure tappings, the mean base moment is found to be 596,842 kNm. According to the Canadian Code of Practice, the gust factor of the prototype at 2.5% damping is 1.85, which yields a dynamic base moment of 507,316 kNm. Thus the measured base moment from the aeroelastic model is 22.5%. less than the code value, this is within the expected range reported by other investigators.

CONCLUSIONS

The experimental technique proposed herein to determine the distribution of wind load along the height of a tall and slender building is found to give reasonable results. As the computation is done in the frequency domain, simultaneous measurement of pressures is required only from two tappings at a time. Thus, this technique can be easily implemented in any wind tunnel laboratory.

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