Engineering Aspects of Earthquake Risk Mitigation

Lessons From Management of Recent Earthquakes, and Consequential Mudflows and Landslides

Proceedings of the UNDRO/USSR/UNDP Training Seminars

(Dushanbe, Tajikistan, USSR, 17 - 28 October 1988)
(Moscow, USSR, 23 October - 3 November 1989)
Engineering Aspects of Earthquake Risk Mitigation

Lessons From Management of Recent Earthquakes, and Consequential Mudflows and Landslides

Proceedings of the UNDRO/USSR/UNDP Training Seminars

(Dushanbe, Tajikistan, USSR, 17 - 28 October 1988)
(Moscow, USSR, 23 October - 3 November 1989)
FOREWORD

The Proceedings contain selected presentations given at the Second and Third UNDRO/USSR Training Seminars:

- on Engineering Aspects of Earthquake Risk Assessment and Mitigation of Losses, held in Dushanbe (Tajikistan, former USSR), October 1988;

- on Lessons from Management of Recent Earthquakes, and Consequential Mudflows and Landslides, held in Moscow (USSR), October 1989.

The annexes to the document provide information on the participants, the work programme and the resolution adopted at each of the seminars. UNDRO hopes that this material will assist disaster-prone developing countries to assess their state of the art in disaster management techniques.

1. Too often a serious analysis of disaster consequences is not carried out until after a destructive event, although experience proves the need for systematic pre-disaster planning and risk analysis. Careful land use zoning, disaster management planning and training together with education of the population, and co-ordination between all concerned - all these aspects are recognized and confirmed to be essential by the authors.

2. Disaster mitigation plans (particularly, plans to strengthen existing structures to resist identified hazards) can be substantially improved by providing the population with adequate information on how and what should be done to increase the resistance of settlements.

3. The need to fill the gap between theoretical findings and their practical application is an important task of UN institutions. But why have all such findings, by very competent specialists, not produced the most natural result i.e. the improvement of disaster management planning? Why have they been left without the necessary follow-up from the authorities concerned? Lack of public and political awareness is one of the explanations.

4. Several of the authors underline the need for new, more precise calculations of loads. However, the need for this is in most cases of lower priority than the need for drastic
improvement of building skills at the construction site, and also the need for improvement of design, the introduction of monolithic cast-in-place rather than pre-cast elements, and the classification of building materials.

5. The questions of the theoretical aspects of the prediction of earthquakes are mentioned by several authors. Although UNDRO is convinced that the problem of prediction is very important and may save lives it stresses the necessity of disaster preparedness and management planning as a more effective means of reducing human suffering and material loss.

6. The first response after a disaster always comes spontaneously from neighbours, volunteers, the general public and the emergency services. The government disaster-management service, however large, cannot physically provide the necessary immediate help to all the victims of a major catastrophe. The strategy of disaster management planning is that each family and community in the disaster-prone area should be self-reliant. Disaster management plans must rely on and complement this spontaneous activity of the population in emergency situations.

*) As the seminars took place in 1988 and 1989, references to country and citizenship reflect the political reality of that time.
# TABLE OF CONTENTS

## Texts of Formal Presentations

<table>
<thead>
<tr>
<th>SEISMIC HAZARD ASSESSMENT AND ZONING</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Hazard Assessment and Determination of the Largest Possible Earthquakes</td>
</tr>
<tr>
<td><em>V. Karnik</em></td>
</tr>
<tr>
<td>Seismic Microzonation</td>
</tr>
<tr>
<td><em>M.D. Trifunac</em></td>
</tr>
<tr>
<td>Selection of Earthquake Design Motions for Important Engineering Structures</td>
</tr>
<tr>
<td><em>M.D. Trifunac</em></td>
</tr>
<tr>
<td>Principals of Seismic Zoning in North China</td>
</tr>
<tr>
<td><em>C. Dasheng and S. Zhenliang</em></td>
</tr>
<tr>
<td>Features of the Seismic Effect in Platform and Orogen Areas</td>
</tr>
<tr>
<td><em>T. Rautian</em></td>
</tr>
<tr>
<td>An Integral Model for Earthquake Damage and Seismic Risk Assessment</td>
</tr>
<tr>
<td><em>Z. Milutinovic and J. Petrovski</em></td>
</tr>
</tbody>
</table>

## ECONOMIC AND VULNERABILITY ANALYSIS

| Hazards, Vulnerability and Risk - A Commentary | 67 |
| *V. Karnik* |
| Indirect Loss and Damage Caused by Earthquakes: A General Treatment | 73 |
| *H. Tiedemann* |
| Building Damage Classification and Loss Assessment for Risk Mitigation | 85 |
| *J. Petrovski* |
| Priorities in Earthquake Damage Reduction | 101 |
| *H. Tiedemann* |
| Lessons from the Mexican Earthquake 1985: Quantitative Evaluation of Damage and Damage Parameters | 119 |
| *H. Tiedemann* |
DESIGN STANDARDS

Improvement of Design and Construction Standards
F. Aptikaev

Design and Construction Standards in Earthquake-Prone Areas
V. Olizerman

CASE HISTORIES

The Japanese Earthquake of 1978
S. Suyehiro

Data on Natural and Induced Seismicity in Brazil
J.A.V. Veloso

Seismicity and Seismic Risk in and Around Egypt
E.M. Ibrahim

Gissar Earthquake - Facts and Figures

The Gissar Earthquake (23 January 1989) Seismotectonics, Seismology and Engineering Geology
S.K. Negmatullaev et al

F. Niyazov

Gissar Earthquake, 1989 - Economic Analysis
S.V. Kozharinov, F.G. Garvrikov

Spitak Earthquake - Facts and Figures

Spitak Earthquake, 1988: Engineering Consequences
J. Burgman, V. Rzhevsky, T. Markaryan

Spitak Earthquake, 1988 - Engineering Analysis
A. Khachiyon

Emergency Management and Relief Experience Following The Spitak Earthquake (1988)
B. Chernichko

Spitak Earthquake, 1988: Organization of Rescue and Resource Management
A. Kapochkin

Spitak Earthquake, 1988: Seismotectonics and Seismology
I. Nersesov

Causes of the Catastrophic Consequences of the Spitak Earthquake
V. Udaltsov
PUBLIC INFORMATION AND EDUCATION

Public Awareness of Information Systems
S. Suyehiro

DISASTER MANAGEMENT AND THE ROLE OF CIVIL DEFENSE

Mitigating Natural Disasters
E. Lohman

Reorganization of a Civil Defense System
F.C. Cuny

Restoration and Strengthening of Damaged Buildings
A.I. Martemianov, S.G. Shaginian, T.G. Markarian

Reinforcement of Existing Buildings
T. Chachave

ANNEXES

Annex A - 1988 Seminar Programme
Annex B - 1988 Seminar List of Participants
Annex C - 1989 Seminar Programme
Annex D - 1989 Seminar List of Participants
SEISMIC HAZARD ASSESSMENT AND ZONING
SEISMIC HAZARD ASSESSMENT AND DETERMINATION OF THE
LARGEST POSSIBLE EARTHQUAKES

by
V. Karnik
(Czechoslovakia)

Summary

The estimate of the largest possible earthquake within a certain seismogenic region is the key
problem in any seismic hazard assessment because the upper threshold magnitude event determines
the level of hazard. Several approaches in estimating Mmax are reviewed below: N(M), historical
records, recent geological movements, fault length, strain release, Riznichenko's activity A, pattern
recognition, extreme values method by Borisov et al. None of these approaches, used alone, can
provide a reliable value of Mmax.

Principles of Seismic Hazard Assessment

The existing procedures for assessing different natural hazards are essentially similar. The
first step is always the definition of the source model in terms of the source location and the fre­
quency and size of potentially damaging events; the second step relates to the propagation of dis­
turbances from the source; and the third step, involves the development of an exposure model for
a site or for a region.

Standard methodology applied to seismic hazards is presented in Figure 1. It is obvious that
accuracy of assessment depends on the quantity and quality of input data. There are too many
examples of an uncritical use of published data for hazard mapping, all of which illustrate the need
for a careful revision and unification of the input information. (Processing heterogeneous data
leads only to misleading results.)

The first major problem appears when potential earthquake-source regions are to be defined
in terms of boundaries and activity. It cannot be assumed that the historical sample has revealed
the existence of all source regions or provided evidence on the largest possible events. The latter
problem is discussed in detail in the present paper.

Once the source is defined, empirical functions relating epicentral distance D, magnitude M,
focal depth h and the selected parameter of ground motion must be established. Macroseismic
intensity I is still widely used as a parameter because macroseismic observations are more abun­
dant than those of any other quantity; they also provide the basis for the assessment of events
which originated prior to 1900. Moreover, macroseismic intensity can be applied to estimating
probable losses or damage ratios of the basic types of buildings. However, defined physical parameters, such as particle velocity, acceleration or others, are needed if the hazard assessment is to be used in earthquake engineering. Strong-motion observations are increasing in number, but locally valid attenuation curves for parameters of ground motions are still rare and investigators are often obliged to apply the results from other regions.

Hazard calculations become less reliable if, because of a lack of detailed information, uniform radiation of seismic waves is assumed instead of azimuthally varying attenuation which is closer to reality.

By combining information on source regions with that on attenuation, hazard values can be calculated by applying one of the algorithms available in the literature. It must be pointed out again that the quality of results depends mainly on the quality of the input data. No matter how sophisticated the algorithms may be, they cannot compensate for inadequate knowledge of the physical parameters defining the source and the attenuation.

Although there are long and short-term variations of earthquake activity, hazard calculations assume a stationary activity resulting from the observational sample. This assumption is likely to be valid for estimates made for the next 25 years, but the estimates required in terms of return periods of the order of $10^2$ to $10^4$ years, e.g. for sites of nuclear power plants, are extremely uncertain. The application of non-stationary models may remove some of the uncertainty; however, a non-stationary model of earthquake occurrence must be based on a long observation interval which is rarely available.

Hazard assessment methodology as a whole was treated in the paper by Karnik and Nersesov presented at the first Training Seminar in Dushanbe in 1986 (see Proceedings, Vols.1, p.29, UNDRO 1987). The purpose of the present paper is to discuss all existing approaches to estimating the largest possible earthquake within an earthquake region.

Initially it is necessary to clarify some basic expressions.

The terms ‘maximum earthquake’ or ‘maximum magnitude $M_{\text{max}}$’ are used, and understood, in different ways. Often adjectives such as ‘expectable’, ‘credible’, ‘possible’ or ‘probable’ are added. We can say that the ‘maximum credible earthquake’ means the extreme event that, by judgement of competent experts in seismology, tectonics, geodesy, etc., appears capable of occurring under the conditions of the presently known seismotectonic environment. Although the time interval between two occurrences may be very long, it is still the maximum believable event corresponding to the physical limits of a given region, i.e. to the upper threshold of the strain energy that can be stored in the given volume of the crust or upper mantle without fracture. The ‘maximum probable earthquake’ is usually defined on a statistical basis as the event that will occur with a certain high probability during a given time interval, i.e. it is not an assured occurrence.

Thus, there are different approaches to the assessment of an extreme event ranging from purely deterministic to probabilistic methods.
The assessment of maximum magnitudes has been frequently debated but no fully satisfactory solution has been found as yet. The pressure of practical requirements, however, remains great since seismic hazards cannot be assessed without the introduction of magnitude thresholds for all earthquake source regions.

Definition of Potential Earthquake Source Regions

Basic questions posed to any investigator at the beginning of a seismic study are:

• where to expect the future foci of earthquakes, e.g. above magnitude M = 4;
• how frequently earthquakes of different magnitudes (M > 4) may occur and;
• what may be the upper threshold magnitude during the period of interest (t = 50, 200, etc., 10,000 years).

The whole process of defining earthquake source regions is a mixture of deterministic and probabilistic approaches and of experts' judgements, but, so far, no standard, well-defined method exists.

1. **The spatial delineation of a source region** is based first of all on the distribution of earthquake foci located by instrumental or macroseismic evidence. This step is fairly easy in regions of high seismicity; however, there are other regions with dispersed foci and with long recurrence intervals of large events. There are also gaps in regular alignments of earthquake foci and some of these gaps can be considered as the most potential places of future earthquake occurrence. In areas of medium and low seismicity, the period covered by instrumental and pre-instrumental observations is often too short to reveal all possible sources in the crust and in the upper mantle. Under these circumstances, the results of other disciplines must be employed; in particular, geological, geophysical, geochemical and geodetic observations are invaluable. These applications are based on simple analogies between the earthquake occurrence and associated geodynamic phenomena.

2. **Frequency of earthquakes, N(M) relationships.** It is well known that the earthquake activity is not stationary and that intervals of low and high activity alternate in an irregular way. Only in some large active areas (e.g. the Mediterranean) can one find a general constant rate in seismic energy release during the 20th century. This observation is, however, unhelpful in hazard investigations which must consider individual source regions and not large areas.

Our knowledge of a particular source region is based on a sample of instrumental data which covers a relatively short time window, usually ranging from 20 to 80 years. In continental areas with historical record, a good data base (non-instrumental) of destructive events going back for several centuries can often be established.

It may happen that a short observation interval coincides with either the active or the quiet period, respectively, i.e. the prediction of future occurrence of events may be either exaggerated or underestimated.
So, it is always advisable to investigate the time variation of activity either by a simple plotting of \( M \), intensity \( I \) or square roots of seismic energy \( E \) with time. Some magnitude-frequency graphs \( N(M) = \log N = a - bM \) are much dispersed and indicate incompleteness of the observational data or inadequate delineation of the region. Sometimes, \( N(M) \) graphs must be constructed by combining several observation periods of different homogeneity, i.e. by correcting the values \( N \) for incompleteness. For standard hazard calculations, the average \( N(M) \) is used over the whole observation period, i.e. stationarity is assumed.

Thus the activity level of a certain source region is defined by the magnitude-frequency relationship and by the values of \( M_{\text{max}} \). Some plots of \((\log N, M)\) show a bending at the end of large magnitudes, so that they seem to indicate natural limits. Some investigators use the asymptote to such curved distributions as \( M_{\text{max}} \). It is true that real changes of the slope of \((\log N, M)\) distributions may occur; however, the bending at the upper magnitude end may be simply caused by the saturation of the magnitude scale used. It is known that \( m_b \) (short-period) scale saturates at \( m \approx 6 \), \( M_L \) scale at \( M \approx 6 \, 3/4 \), \( M_S \) scale at \( M \approx 8 \, 3/4 \), \( m_b \) (medium period) at \( m \approx 7 \, 3/4 \). Thus, particularly the magnitude scale based on short period waves \((P,S_g)\) saturates at medium magnitude and the curvature of \( N(M) \) may be misleading.

There is no doubt that for physical reasons every volume of rock must have its \( M_{\text{min}} \) and \( M_{\text{max}} \) when stress is applied. \( M_{\text{min}} \) can be detected within a relatively short interval of observation; however, \( M_{\text{max}} \) can have a very long return period. It is not necessary that an empirical \((\log N,M)\) distribution should be concave (see e.g. Karnik 1968, Karnik, Prochazkova 1976), it can show local deviations from linearity.

**Historical Earthquake Record**

Earthquake catalogues compiled for individual earthquake provinces cover different time intervals. Usually, the homogeneity threshold increases when we go back in history, i.e. a set is complete only above a certain magnitude (or intensity) threshold within a given time interval. In regions where the earthquake activity has remained high for several centuries, one can assume that the largest earthquake observed so far is the maximum possible earthquake which is likely to occur in the future as well. There is, however, no rule which would be instrumental in deciding the validity of this assumption. ‘Seismological surprises’ have been reported in some regions, i.e. large earthquakes originating within a region considered so far to be a low seismicity one according to historical records. This ‘historical record approach’ must be therefore applied with caution and additional evidence sought.

For some practical applications, the maximum expected intensity is estimated by adding one degree to the value of the largest observed intensity.
**Geological Approach, Fault Rupture or Length vs. M\text{\text{max}}**

Geomorphic, stratigraphic and soil-stratigraphic studies can furnish information on the surface rupture parameters, i.e. surface rupture length and displacement during past earthquakes. Displacement can also be determined by digging trenches across faults; this method seemed to be quite effective in the western USA.

Another approach uses dimensions of fault segments that have ruptured during previous earthquakes. These sections may be bounded by sections of roughness (asperities) controlling the segmentation process as well as fault branching. The mechanisms are not yet fully understood; it is generally assumed that one-sixth to one-half of the total fault length can rupture (Slemmons 1983). The application of the above-mentioned approaches requires reliable fault mapping and the availability of empirical relationships linking magnitude \( M \) and fault parameters. Different approaches and results have been described in the literature. Several examples are given below.

1. As stated by P.E. Wallace (1970) the recurrence interval \( R_x \) for earthquakes of a different magnitude at a given point on the fault follows the relationship:

\[
R_x = \frac{D}{S-C}
\]

\( D \) = displacement accompanying an earthquake of a given magnitude \( M \)
\( S \) = long-term strain rate estimated from offset of geologic units (2cm/year in California)
\( C \) = tectonic creep rate for recurrence interval \( R_y \) along the total length of the fault

\[
R_t = \frac{DL}{(S-C)L_t}
\]

\( L_t \) = total fault length
\( L \) = length of break proportional to \( M \)

2. I. Ozawa (1972) introduced a formula defining \( M\text{\text{max}} \) in terms of the size \( A(\text{cm}^2) \) of an 'active block': \( \log A = 0.9 \text{\text{max}} + 6.5 \), or of the length (km) of an 'active' fault: \( \text{\text{max}} = 0.76 \log L + 6.35 \). In principle the boundaries of 'active blocks' separate regions of uplift and subsidence.

3. Other empirical formulae were published by Chen (1984):
M = 3.3 + 2.1 log \( L_r \),
M = 6.43 + 0.66 log \( L_r \),
M = 6.72 + 0.48 log \( L_r \),

4. There is a great variety of this type of relationship; a list can be found e.g. in a paper by V. Schenk and Z. Schenkova (1983) who averaged a family of \( M(L) \) formulae from different regions and got \( M = (1.93 \pm 0.08 \log L (\text{km}) + (2.62 \pm 0.62) \). The value of such averaged formula is, however, questionable.

5. A.A. Nikonov (1988) published a set of formulae for Soviet Central Asia:

\[ M = 7.26 + 0.32 \log L_z \]
\[ M = 6.61 + 0.55 \log L_r \]
\[ M = 7.09 + 0.79 D \]

\( L_z \) - length of the surface rupture zone (km)
\( L_r \) - length of individual ruptures (km)
D - fault offset in metres

He also gives formulae derived by Solomenko and Khromovsky (1978):

\[ M = 6.0 + 0.6 \log L \]
\[ M = 6.0 + 0.8 \log L_r \]

\( L \) - length of faults (normal, shear)
\( L_r \) - length of fracture zones

Nikonov's aim, however, was to estimate magnitudes from dislocations found in Central Asia and caused by past earthquakes.

**Mean Focal Depth and \( M_{\text{max}} \)**

Two important observations which can be used in \( M_{\text{max}} \) assessments can be formulated as follows:

1. The proportion of large and weak events is increasing if deeper layers of the crust and upper mantle are considered, i.e. the slope of the magnitude-frequency distribution (\( \log N = a - bM \)) is decreasing with average focal depth.

2. \( M_{\text{max}} \) is varying with depth, it increases with depth within the lithosphere (Fig.2), then it drops within the 'low-velocity layer' and increases again. Each earthquake region has its specific distribution \( M_{\text{max}} = f(h) \). The explanation is in the variation of the lithospheric material and in the increasing stress or stress rate with depth. N.V. Shebalin (1971) found an empirical relation, \( M_{\text{max}} = \log h_{km} + 2.5 \), for the crust (\( h < 40 \text{ km} \)) in the Caucasus, the Crimea and Central Asia. In the same paper he published a relation between the length \( L(\text{km}) \) of a seismoactive zone (fault) and \( M_{\text{max}} 1.8 \log L + 1.4 \leq M_{\text{max}} \leq 2 \log L + 2 \) and \( M_{\text{max}} \leq \log H_{km}(\text{km}) + 1.8 \), linking \( M_{\text{max}} \) with the thickness \( H_{km} \) of the 'seismoactive layer', i.e. of the layer containing earthquake foci.
Housner's Method

G. Housner (1969) suggested an approach which is based on a good knowledge of \( M_{\text{max}} \) in one region. \( M_{\text{max}} \) in the second region can be estimated from the formula:

\[
M_2 = \frac{\beta_2}{\beta_1} M_1 - \beta_2 \ln \left( \frac{N_1(M)}{N_2(M)} \right), \quad \log N(M) = a - \beta M, \quad \beta = \ln 10.
\]

Fluctuation of Strain Release Curves

The behaviour of an earthquake source region with time can be well described by a cumulative plot of \( E^1 \) (\( E \) - seismic energy) with time \( t \). The original idea of 'strain release' stems from H. Benioff who in his study of aftershocks (1951) introduced a simplified assumption that deformation during each aftershock is proportional to \( E^1 \). In the plots \( (\sum E^1; \log t) \) he interpreted two types of a deformation process in accordance with laboratory studies. Later on, graphs \( (\sum E^1; t) \) were widely used in depicting the time development of earthquake activity within a certain area. Examples are numerous (see Fig.3) and all of them show features that could be exploited for assessing \( M_{\text{max}} \).

All graphs show periods of activity alternating with those of quiescence. The oscillations of most curves can be framed in two parallel lines; their span defines the maximum seismic energy that can be released within a relatively short time, i.e. one to say three years. (Other quantities than \( E^1 \) can also be plotted, e.g. \( M, E, E^2, M_0 \); they are inter-related, \( M \sim 2/3 \log E \sim 2 \log L; M_0 \sim 3/2 M \)). The value of \( M_{\text{max}} \) can be calculated from: \( 1/2 \log E = 5.9 + 0.75 M_0, M_0 = 1.33 (1/2 \log E - 5.5) \); e.g. for the eastern part of the Aegean region (Fig.3) the distance between the parallel lines is \( 43 \times 10^4 \) erg., i.e. \( M_{\text{max}} = 7.9 \), which is quite a reasonable value. For W. Greece \( M_{\text{max}} = 7.6 \), for the Crete area \( M_{\text{max}} = 7.1 \).

The method is quite promising because the estimate is based on actual limits. The graphs also permit us to see whether there is a tendency to seismic energy release in many smaller events or in a few large events. This behaviour can be checked by the slope of the \( N(M) \) graph.
Riznichenko's Method

An interesting and well founded approach was introduced by Yu. V. Riznichenko (e.g. 1985). It is based on the correlation between the activity number A and Kmax (\(K = \log E\) in joules, \(K = 4 + 1.8 M\), \(A =\) annual number of events corresponding to a certain \(K\), e.g. \(K = 10\)).

In practice, A values for several unit areas (cells) are plotted against Kmax observed. Some of the Kmax values can be expected to be below the possible upper threshold, some may be quite close to it and some may reach that level. The resulting picture is a cloud of points; however, the envelope at the side of the largest events may correspond to the real Kmax threshold. This curve (or line) can be used in other parts of the area to estimate Kmax (Mmax) if we know A.

For example, for the Altay-Sayan (Central Asia) and Vrancea (Romania) areas the empirical formula \(\log A = 2.84 + 0.21 (Kmax - 15)\) was found.

For Japan \(\log A = 2.84 + 0.39 (Kmax - 15), (K = 15 - 19)\). It must be noted that this method based on A, can be used only within an area where the magnitude-frequency (or energy class-frequency) relationships have identical or very close slopes.

Pattern Recognition

It has been stated earlier that the accuracy of M could be increased only by combining all available information on earthquake related phenomena. The first attempt in developing a defined procedure was made in the USSR (see e.g. Borisov et al. 1975). The method is based on the classification of geological and geophysical phenomena known to be connected with earthquake occurrence. For each phenomena \(x_i\) an empirical function \(Y_i(x_i)\) relating \(x_i\) and \(\partial Mmax\) is found in a region with well known geological and geophysical characteristics (for an example see Fig.4). Functions \(Y_i(x_i)\) must be calibrated in such a way that the summation of contributions \(\partial Mmax\) by individual functions gives the expected Mmax:

\[
M_{max} = f(x_1, x_2, ..., x_n) = \sum_{i=1}^{n} Y_i(x_i).
\]
Functions $Y$, have been compiled for the following phenomena:

- recent tectonics
- amplitudes of vertical movements during the Neogene-Quaternary
- young volcanism
- longitudinal volcanism
- longitudinal deep faults recently active
- transverse deep faults recently active
- transverse deep faults
- fault crossing
- seismic activity $A$
- gradient of isostatic anomaly
- amplitudes of recent movements.

Once $Y$ and $Z$ functions are calibrated for a section of a tectonic unit they can be applied in other parts of the unit. The calibration must be made in a region where geophysical and geological parameters as well as experts estimates of Mmax are known. The method was also applied by Gitis et al (1978) in the area extending from the Crimea to the Caucasus and to West Turkmenia in the USSR. Experts' estimates of Mmax for each cell were made on the basis of available seismological information. Five characteristic parameters $x_i$ were selected for empirical curves, i.e. horizontal gradient of the basement morphology, horizontal gradient of the topography, gravity anomaly reduced to a subsurface level, structural heterogeneity (according to the USSR Map of Tectonic Regionalization) and active fault intersections of different sizes (Fig.4). The method has been recommended for defining earthquake source regions in the USSR.

There is, however, at least one weak point, namely the experts' assessment of Mmax for individual cells. Moreover, empirical relations $Y_i(x_i)$ calibrated in one area cannot be simply transferred to another one; sound verifications are always necessary. This method is being further developed.

Theory of Extreme Values

The probability of the largest events can be calculated with the use of the theory developed by E.J. Gumbel (1958). It is founded on three basic assumptions:

- the conditions prevailing in the past must be valid in the future;
- the observed largest events in a given interval are independent;
- the behaviour of the largest events in a given interval in the future will be similar to that of the past.

The theory, which is also called the asymptotic theory of extremes, requires no knowledge of the initial distribution. The theoretical background leads to three distributions of
extremes; in the treatment of earthquakes the first and the third distributions are used. Since in the first distribution the variable is unlimited, the third distribution, which has an upper limit (M_{\text{max}}), is more suitable for seismological data. The basic assumption of the theory is that the earthquake magnitude M = x is considered to be a random variable with a negative exponential distribution. F(x) = 1 - \exp(-x), x > 0 and the largest magnitude in a given interval is then described by the cumulative distribution function H(y) = \exp [-\exp(-y)], y = c(x-u), where c is a parameter, u is the characteristic largest value, H(u) = e^u; this formula defines the first distribution. The third asymptotic distribution is defined by:

\begin{align*}
G(x) &= \exp \left[- \left(\frac{w-x}{w-u}\right)^K\right], K > 0, x \leq W, u \leq w,
\end{align*}

where w is the upper limit of the largest values (= M_{\text{max}}), k is the shape parameter, u is again the characteristic largest value, G(u) = e^u, G(w) = 1.

The return period of earthquakes with M equal to or greater than a given threshold value is defined as the reciprocal of the relation [1 - G(x)] for both distributions. In practice, the plotting position \( P_m = G(x) \) of the m-th observation is defined by:

\begin{align*}
P_m &= \left(\frac{m}{N+1}\right), m = 1, 2, ..., N
\end{align*}

N = number of observation arranged in the order of increasing maximum magnitudes \( M_1, M_2, ..., M_N \) observed in a given time interval (1, 2, 5, ... years); a special extremal probability paper is used. Estimates w, u, k can be obtained by the method of least squares. Examples of distributions are contained in Fig.5. The probability G(x) and the return period T(x) are plotted along the horizontal line. The graphs provide estimate of T(M_{\text{max}}) or of M_{\text{max}} which will be exceeded with a given probability in the future. The value defines the asymptote to the distribution and corresponds to \( T = \infty \) or zero probability of exceedence and can be considered as the largest possible magnitude, however, a more realistic estimate should be based e.g. on \( T = T^3 \) years (example see Table 1).

In actual observations, the curvature of the approximating curve depends on the largest of the extreme annual (or decade) events which are rare. Sometimes the curvature leads to \( M_{\text{max}} - M_{\text{max}} \), obs > 1, i.e. to exaggerated values above \( M = 9 \). It should be also noted that probability estimates for return periods exceeding half of the total observation interval of the sample are not considered reliable.
Conclusions

The above methods provide the possibility of estimating one of the most significant parameters in hazard assessment, i.e. the largest expected magnitude $M_{\text{max}}$. Every approach described above has some limitations and it is advisable to apply as many of the approaches as possible. Large $T_{\text{max}}$ are beginning to play an important role in the designs of large dams or nuclear reactors. $M_{\text{max}}$ estimated only by one or two methods will always involve a large degree of uncertainty. According to Esteva and Villaverde (1973) errors in estimating $M_{\text{max}}$ have little influence relative to high probabilities but show significant influence at low probabilities.
Fig. 1. Scheme of probabilistic hazard calculation

A) Delineation of earthquake source regions
B) Magnitude-frequency relationships $N(M)$
C) Intensity attenuation curves
D) Cumulative conditional probability of intensity $I$
E) The extreme probability $F_m(I)$ for various intensities $I$ and exposure times
Fig. 2. Distribution of largest magnitudes with depth, Aegean region
Fig. 3. Examples of the $\sum E(t)$ graphs for the E and W parts of the Aegean region
Fig. 4. Examples of functions $Y(x_j)$ used by Gitis et al.

1 - horizontal gradient of the basement relief
2 - horizontal gradient of the topography
3 - gravity anomaly reduced to the basement
4 - structural heterogeneity
5 - intersections of active faults
Fig. 5. Examples of Gumbel III distributions, Aegean region (two focal zones)
Table 1

Comparison of magnitudes M exceeded with given probabilities for N and SW Greece (regions 14-17, 20-24 see Karnik, Schenkova 1978) as determined from the Gumbel III distributions.

<table>
<thead>
<tr>
<th>Region No</th>
<th>N events</th>
<th>M $p=1%$</th>
<th>M $p=2%$</th>
<th>M $p=10%$</th>
<th>M max, obs</th>
<th>$t_{1/2}$ max, obs</th>
<th>T (years) M = 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>14-17</td>
<td>939</td>
<td>7.05</td>
<td>7.59</td>
<td>6.80</td>
<td>7.4</td>
<td>9.8</td>
<td>29</td>
</tr>
<tr>
<td>20-24</td>
<td>1365</td>
<td>7.08</td>
<td>6.92</td>
<td>6.30</td>
<td>7.0</td>
<td>8.0</td>
<td>71</td>
</tr>
</tbody>
</table>

(N = number of events from 1901 - 1970, W = M value corresponding to the asymptote).

References

8. G.W. Housner (1969): Engineering estimates of ground shaking and maximum earthquake magnitude. Proceed. 4 WCEE.


SEISMIC MICROZONATION
by
M.D. Trifunac
Los Angeles, California, U.S.A.

SUMMARY
Seismic microzonation method using Uniform Risk Spectra (URS) is reviewed and illustrated. It provides continuous probabilistic spectral amplitudes which can be used in design, in probabilistic estimation of response, or to generate synthetic accelerograms for non-linear analyses. The method does not require new or difficult steps to gather data and all required calculations can be performed on a personal computer.

INTRODUCTION
Earthquake resistant design provisions contain information on the areal distribution of the horizontal force coefficient, and thus present maps showing how this coefficient varies geographically. These seismic zonation maps typically do not show much detail and are meant to reflect the general distribution of the expected future earthquake shaking. Thus a large city or a metropolitan area may be covered by one constant value of the seismic design coefficient.

It has been recognized by many earthquake engineers that local soil and geologic conditions influence the level and nature of earthquake shaking and the degree of the observed damage. Many code provisions incorporate such effects into the shape of the design spectrum or allow further investigations to modify the average code coefficient and incorporate local areal variations reflecting the local site conditions. The extent of permitted departures from the average code amplitudes and the procedures recommended for estimating their fluctuations vary from one code to another. In most currently employed methods, these variations are determined from consideration of soil and geologic site conditions, which are assumed to “amplify” the average incident strong motion amplitudes. The average amplitudes are determined from the maps of horizontal design coefficients, from assigned “representative” intensity for use in design (usually associated with different return periods), or from some amplitude controlling parameter (e.g. peak acceleration), determined via simplified seismic risk analysis.
The key assumption in the above approach is that the local soil and geologic conditions lead to amplification patterns which do not change from one earthquake shaking to the next, and that these patterns can be determined experimentally or by analysis of the local site conditions. In Japan, Kanai and his co-workers investigated microtremors as possible source of excitation to "measure" the distribution of the local amplification patterns. Later studies of their analyses procedures and of the nature of microtremor waves, and of the differences in their wave propagation paths relative to the paths of earthquake waves, have discouraged many investigators from further considering this approach. So far no one has developed a satisfactory and physically justified method for using microtremor vibrations to characterize the patterns of amplification of ground motion during earthquakes. Observations of repeated strong motion shaking from different earthquake at the same recording station show large differences which depend on the angles of wave arrival, on the source mechanism of the earthquake, and on its proximity to the site. Simple analytical studies of wave scattering, focusing and diffraction, by soft soil and alluvial deposits show that the peaks of the spectral amplitudes of recorded motions at the ground surface can shift with changes in the incident angle of the arriving waves so much that the concept of "predominant period," as used in older microzonation studies, can no longer be justified.

Some older proposals for microzonation methods included the local soil and local geologic site characteristics, but more recent work tends to favor the local soil conditions only. Our present understanding of the changes in strong motion amplitudes along the wave propagation path, from the source to the recording station, indicates that one must include both the soil and the geologic site conditions, if correct average amplification is to be determined. This means that a larger volume (1 - 10 km) surrounding the site must be included in the analysis. Typical soil investigations involve smaller (10 - 100 m) volume and include shallow soil properties only. Clearly the state-of-the-art microzonation mapping of the amplitudes of strong shaking may be complex and costly for an individual project and should involve entire cites and metropolitan areas.

The time involved in the development, in the implementation and in the long range benefits from proper use of physically sound microzonation maps in earthquake resistant design, all suggest that it will take many years before the current individual, subjective and often incomplete or specialized microzonation studies, and the structures designed by the current procedures, are all surpassed and phased out. The future advanced society will have to overcome first the short term, often post facto, and the incremental improvements of earthquake design codes, before the full power of the multidimensional economic advantages are not only recognized, but also included in future microzonation procedures. Thus, at present, we can focus our attention on the physical aspects of this problem, hoping that through measurement and analysis at least sound principles can be developed for later inclusion into more integrated and general optimization framework.
MICROZONATION BASED ON U.R.S. METHOD

The method for preparing microzonation maps based on Uniform Risk Spectra (U.R.S.) has been discussed elsewhere in detail. Here only some general features and advantages of this approach will be summarized. The method is based on the computer program NEQRISK which computes U.R.S. for selected probabilities of exceedance, for a chosen duration of exposure (e.g. \( Y = 50 \) years), for a combination of random independent (Poisson) and literal (predicted) earthquake occurrence at sources which can be defined to reflect all the details of the faulting geometry surrounding the site. The seismic activity at all seismic sources can be described by the occurrence rates, versus magnitude or intensity, and in terms of the geological slip rates using time rate of change of the seismic moment. When output is required in the form of acceleration time histories, the program NEQRISK is used to compute the Uniform Risk Fourier Amplitude Spectra, which then serve as input for computer program SYNACC, which calculates synthetic strong motion accelerations. This program includes the site specific wave dispersion analysis and computes two horizontal, one vertical and two rotational (torsion and rocking) components of acceleration.

In the following we illustrate typical microzonation maps computed via URS amplitudes and point out several advantages of this method. First, over an extended area of a large city, the seismic risk cannot be described by a constant (e.g. peak acceleration, or maximum intensity), unless all sources of earthquakes are at considerable distance (e.g. further than say 200 km). For example, Figure 1 illustrates the setting of the metropolitan Los Angeles area and shows the major qaternary faults in Southern California. Earthquakes occurring at all these faults have been included in the calculation of microzonation maps illustrated in Figure 2. Even if the geologic and the soil media in this entire area were uniform, the microzonation maps would still indicate high and low amplitudes, which would be governed by the proximity, geometry and activity of all sources (faults) \textit{``contributing''} strong motion amplitudes with different probabilities of exceeding spectral amplitudes at a site. In Los Angeles area, the seismic risk measured by the maximum site intensity or maximum \textit{``earthquake''} that could shake a site, slowly increases forwards northeast as one approaches the San Andreas fault, assumed to be the \textit{``major contributor''} of earthquake events during the next \( Y = 50 \) years. Without San Andreas fault in this model the amplitudes of seismic risk, throughout Los Angeles, would be more variable and the amplitudes would reflect the proximity of a site to one or to several of the active faults in the model (Figure 1).

Second useful feature of the U.R.S. approach is that it incorporates the site soil and geologic conditions into the calculations directly. Figure 2 illustrates the Uniform Risk Pseudo Relative Velocity Spectra (in/sec) for oscillator periods \( T = 0.04, 0.34, 0.9 \) and \( 2.8 \) seconds, for probability of exceedance equal to 0.5 and for exposure during the next 50 years. The large amplitude elongated zone near the central part of the metropolitan area reflects the influence of the deep sedimentary basin (~ 30,00 feet) which tends to amplify more the longer period ground motion. A detailed manual with many such maps can be prepared for probabilities...
of exceedance \( p = 0.99, 0.90, 0.50, 0.1 \) and 0.01, for example, and for amplitude scaling in terms of earthquake magnitude\(^{28-30}\) or local intensity.\(^{33,34}\) From such a manual, at any point in the area covered, one can construct U.R.S. of Pseudo Relative Velocity (PSV), for the above given probabilities of exceedance, to obtain the site specific URS of PSV amplitudes.

Third, the shape of the U.R.S. changes continuously as the site moves. This reflects changing contributions to the shaking amplitudes which come from faults at different distances, with different earthquake activity and for wave arrivals through different site soil and geology. Unless the risk is governed by the proximity of a fault where large earthquakes may occur, the shape of URS will usually be quite different from the shape of a response spectrum associated with one earthquake. The shape of U.R.S. will also reflect the relative proportion of small to large earthquakes occurring near a site, and the largest magnitude earthquake, \( M_{max} \), associated with one or several near sources.

Other details on URS microzonation maps, including conversion from horizontal to vertical components of ground motion, different scaling alternatives and forms of mapping can be found elsewhere.\(^{10,23,28}\) The general methodology, definition and the methods of calculating URS have been available since 1977.\(^{3,4}\)

In the above described microzonation method it is assumed that the objective is to determine the amplitudes of strong earthquake shaking in elastic and linear local site environment. These motions can be used as input to evaluate the liquefaction potential, landslide hazard, and various non-linear phenomena of response and their effects on distribution of structural damage, for example.

**CONCLUSIONS**

The purpose of this brief paper has been to review the current state of art in preparation of seismic zoning maps via U.R.S. method. This method enables one to incorporate many factors contributing to the seismic risk, in a balanced way, and to forecast the distribution of seismic risk for use in earthquake resistant design, using an independent Poisson sequence of earthquake events, earthquake prediction, or a combination of these two.

The calculations using NEQRISK and SYNACC computer programs can be performed on a small personal computer. The specification of earthquake sources, their geometry, number, and source activity, usually can be prepared from the existing seismicity and geologic data, by a coordinated group of geologists with total effort rarely exceeding a one man year. Following careful regional investigation of representative scaling and attenuation of recorded strong ground motion,\(^{33,34}\) NEQRISK program can be modified to work with regionally representative empirical scaling functions for strong motion amplitudes.
Figure 1

Figure 2
REFERENCES


This paper reviews selected recent publications\(^4\) on physical and observational bases for quantitative description of earthquake occurrence rates, on selection of maximum earthquake and on the concept of uniform risk spectrum.\(^2,21,24\) It reviews the empirical description of strong ground motion for generating artificial accelerograms for use in engineering design.\(^24,34\) Using the computer program SYNACC an artificial accelerogram\(^22,23\) can be derived, starting from the uniform risk spectrum computed by the NEQRISK program,\(^21\) or by using independently determined Fourier amplitude spectra\(^25\) of the postulated strong ground motion.

When the seismic risk at a site is dominated by the presence of an active fault nearby, the analyst may wish to compute the waves in terms of a physical model which produces ground motion in terms of the assumed slip and faulting characteristics at the earthquake source.\(^26\) References are cited which summarize the state of the art in forward modeling procedures of ground motion from kinematic description of earthquake sources.

For long structures,\(^20,30-32\) which are sensitive to relative motions of different multiple supports, the local soil and geologic conditions call for detailed modeling of the wave scattering and diffraction.\(^28,29\) For input ground motions computed either in terms of synthetic accelerograms via empirical scaling and uniform risk spectra\(^21,24\) or in terms of deterministic physical modeling\(^26\) the cited papers on wave scattering show how, by using transfer function approach and Fourier analysis and synthesis, one can derive motions at multiple support points of long structures.

I. INTRODUCTION

Since 1933, March 10, in Long Beach, California, when the first strong motion accelerogram was recorded, many important advances in earthquake engineering design have been accomplished. From 1940's, when the first earthquake resistant design codes were introduced, to the present, almost every country situated in a seismically active region has adopted some form of earthquake design provisions. In this sense the earthquake resistant design is now en-
tering a “mature” state in which it is performed routinely and according to some specifically defined design codes.

The methodology, and the computer programs which are outlined here, have been calibrated in terms of specific empirical equations, to work in the Western United States, from where most of the data we use comes from. While we have extended some of these empirical equations to other parts of the world, it should be noted that the methodology we present is general, while the examples of scaling equations are not. Thus, for example, to use our approach for estimation of the ground motion characteristics in Europe or in Japan, a sequence of regionally specific empirical scaling equations must be developed.

Most of the available literature on strong ground motion is scattered in many reports which, if available, may not represent systematically arranged material for easy study. Therefore, we have selected here sample sections of the available material, and arranged it into a sequence, as outlined in the flow chart of Fig. I. In this form, so grouped and cited material will provide an earthquake engineer with basic introduction on the steps that are required for selection of the design ground motion, and with the starting background for further study.

Section II will briefly summarize the methods for empirical scaling of ground motion, and will end with a response or Fourier spectrum of the representative ground motion, or with a synthetic accelerogram characterizing the expected future ground motion. No detailed site specific wave propagation analyses will be considered, but only the overall site specific spectral considerations.

In contrast, Section III (Fig. I) will cite literature on physical modeling approach for computing the ground motion, in those instances, when the causative fault is very close to the site of the structure being analyzed. Here, it will be assumed that the geometry and the motions on the fault have been specified, so that the task to be accomplished involves only the forward computation of ground motion at desired locations near by.

In Section IV we cite selected papers which deal with the procedures for calculation of the site specific effects. The input for these calculations will come from either empirical (Section II) or physical (Section III) modeling of ground motion. The need for this work comes from the requirement to select proper ground motion input for long structures, which may be sensitive to the propagating wave effects.

Recently, there were some attempts to reconcile the old, essentially linear, approach for earthquake resistant design, with more realistic non-linear analyses. The outcome of these attempts consisted of some “equivalent,” “effective” and “anchoring” “design” ground motion parameters, providing for the required “conversion” between simplified analyses and the reality. To avoid any confusion, in this work, even though the word DESIGN is used in the title, we will address only the problem of estimating the “actual” ground motion that may take place at the site during future earthquakes. Of course, we cannot predict the true details
Describe earthquake occurrence rates for all sources surrounding the site, using geological and seismological data.

Many sources contribute to the hazard at the site?

Choose physical modeling path (only one or two sources dominate)

Specify the source parameters: length, width, orientation, slip amplitude, slip direction, dislocation velocity, stress drop, moment, etc...

Select the physical model and compute the ground motion at selected points.

Site specific modeling required?

Choose empirical estimation path

Select attenuation equations specific to the area of investigation

Compute Uniform Risk Spectra: NEQRISK

Construct site specific synthetic acceleration time histories: SYNACC

Yes Site specific modeling required?

Select the site specific model involving detailed geology, soil geometry and material properties.

Using input ground motion from empirical or from physical models compute motions in the site specific model at required locations

Output

No

Output

Figure I
of the motion before it occurs. By "actual" we mean that no reductions, modifications, or changes will be imposed on the ground motion estimates which result from our knowledge and experience with specific features of the structure which is to be analyzed.\textsuperscript{10-14}

II. EMPIRICAL ESTIMATION OF DESIGN EARTHQUAKE MOTIONS

Detailed seismic risk analyses begin with description of the seismic environment of the site and characterization of the levels of earthquake activity, distance of the activity from the site and selection of the procedures for introducing the identified sources into the method of analysis,\textsuperscript{1,2,4,4} Once the seismic sources have been specified, the second important step involves description of the attenuation equations.\textsuperscript{33,38,42} This work has the objective of translating the description of the nature and of the size of the earthquake into the preferred mode of presenting the ground motion at the site where the analysis is to be carried out. The third important step in the analysis combines all contributions from different sources into one joint presentation, for example, involving distribution of spectral amplitudes, so that one can select the final amplitudes on the basis of some desired level of confidence that those will not be exceeded. In some instances, this may be the end result of the analysis representing the spectrum of the "design" motions. In other instances it may also be necessary to produce artificial time histories,\textsuperscript{22,23} so that non-linear response analyses can also be carried out.

To organize the tasks that have to be performed in the above steps, one may divide this work into three different parts addressing: sources of earthquakes, uniform risk spectra and synthesizing realistic strong motion accelerograms.

Sources of Earthquakes

This part is concerned with describing the physical and observational bases for a quantitative relationship between earthquake occurrence rates and geological deformation rates.\textsuperscript{1} Such relationships are founded on the principles of the elastic rebound theory,\textsuperscript{26} and are supported by observations.\textsuperscript{44} The role of a seismic creep, the width of the seismogenic zone, the shape of the earthquake magnitude – occurrence rate curve (including maximum earthquake), and the limitations of geological observations to define deformation rates are the most important sources of uncertainty in the process.\textsuperscript{44}

Uniform Risk Spectra

First methods to determined the seismic risk at a site were discussed by Cornell.\textsuperscript{9} The results are often presented in terms of one ground motion parameter, such as peak acceleration, and the return period may be calculated versus that parameter. Other ground motion parameters like magnitude, some peak amplitude of ground shaking, and Modified Mercalli Intensity at the site have also been used. In such studies, the spectral nature of ground motion is not considered. The probability that a spectral amplitude will be exceeded
during future earthquakes does depend on wave frequency\textsuperscript{27,30} and so must be considered in all modern analyses.\textsuperscript{44}

The concept of uniform seismic risk\textsuperscript{2,21,24,44} generalized this work to a functional of shaking, $S(\omega)$, which can represent any functional of strong ground motion at frequency $\omega$ (like Fourier amplitude, response spectral amplitude, peak response amplitude, or duration of strong shaking). This work\textsuperscript{21} also incorporated a realistic model for describing the seismicity and proposed two independent methods to obtain uniform risk functionals: one assuming that the seismicity, which is the input to the model, is treated as the mean of a Poisson sequence, and the other one assuming that it can be taken literally.

There are several ways of describing seismicity. Depending on the method used, the outcome of a seismic risk analysis may be very sensitive to this description. In a large region, the occurrence rate of earthquakes is often known. The analysis for a specific site is more difficult and depends on the conditions near the site, particularly within 25 to 50 km.\textsuperscript{21} For small regions, "historical seismicity" based on felt reports is often incomplete and may not represent the true seismicity of the region. In such cases, the knowledge of fault slip rates or regional strain rates\textsuperscript{1} from plate tectonics theory may be used to estimate "geological seismicity." This helps to increase the reliability of the description of seismicity in a region.\textsuperscript{21}

The steps in the derivation of a uniform frequency dependent risk functional can be summarized as follows:\textsuperscript{21,24}

1. Specify the geometry of earthquake zones by point, line, areal, dipping plane and volume sources. For each of these source zones, the estimated number of events of each earthquake size, $N(M_f)$, is defined. The uncertainties in the estimation of seismicity and maximum allowed earthquake sizes are also defined. This is done by studying previous seismicity, by insights obtained from geological studies and plate tectonics, and by statistical inference and scientific judgement.\textsuperscript{21}

2. Divide each source zone into small source elements, and assuming the epicenter of each event of some magnitude $M_f$ is equally likely to occur any place in the source zone, distribute the seismicity to each element accordingly.

3. Specify a frequency-dependent description of the attenuation of strong-motion amplitudes in the region,\textsuperscript{42} plus a description of the distribution of the observed amplitudes\textsuperscript{33-43} $S(\omega)$ about the mean estimate $\tilde{S}(\omega)$. From this, define the function $Q_{ij}(S(\omega))$ which gives the probability that $S(\omega)$, will be exceeded at the site for an event of size $M_f$ in the $i$-th element of the source zone.

4. Calculate $N_E[S(\omega)]$, the expected number of times that $S(\omega)$ will be exceeded at the site from all source elements in all source zones and all allowed earthquake sizes. Then calculate $P[S(\omega)]$, the probability that $S(\omega)$ will be exceeded at least once at the site in the given period.
(5) Derive the frequency dependent uniform risk functional from the functions $P[S(\omega)]$.

Synthesizing Strong Motion Accelerograms

The Majority of proposed methods for generation of synthetic accelerograms fall into two categories: (1) methods that utilize random functions, and (2) methods that involve source mechanism and wave propagation models. Using the former methods, the resulting accelerograms do not always have a correct frequency content for engineering applications and the frequency characteristics of the time record are often uniform from the beginning to the end of the record. For a recorded accelerogram, the frequency contained in the earlier part is generally higher. Using the latter methods, a more physically consistent record can be generated, but it is impossible to model all the details of the source, as well as the wave path, for the complete frequency range of interest (e.g., 0.05 Hz to 30Hz). Because of the simplifications, the records generated often lack proper high frequency characteristics when compared with recorded accelerograms.

Trifunac and Todorovska present a brief review and a summary of a method based on computer program SYNACC for constructing synthetic accelerograms which have a given Fourier amplitude spectrum, $F(\omega)$, and a given duration. The Fourier amplitude spectrum and the duration can be obtained from correlation with the earthquake parameters. The times of arrival of the waves are derived from the dispersive properties of the site; i.e., the phase and group velocities for the lowest modes of surface waves. This method thus introduces the characteristics of each site into the resulting artificial accelerogram. The strong motion amplitudes are determined in terms of (1) earthquake magnitude and epicentral distance, or (2) Modified Mercalli Intensity at the recording station.

III. PHYSICAL MODELING OF NEAR EARTHQUAKE SOURCES

The computed strong ground motion near dislocation models of earthquakes in a uniform, elastic, layered half-space has been the subject of many investigations. The models include a fault surface, consisting of one or more planes, and involve distributions of slip and rupture velocity over the fault. The surrounding medium is represented as a multilayered elastic or viscoelastic half-space. The simplest three-dimensional dislocation model in an infinite space corresponds to constant dislocation moving with constant rupture velocity over a rectangular fault. For simple dislocation time functions, the time convolutions can be obtained analytically. The integrations over the fault plane are conducted numerically. Haskell calculated and plotted displacement, velocity and acceleration time histories for the motion in the vicinity of a vertical strike-slip fault. He also presented contours of maximum range, in the vicinity of the fault, for the parallel and normal components of displacement, velocity and acceleration.

More recently three-dimensional dislocation models of extended faults in a layered half-space have been utilized in the study of recorded near-source ground motion from a number
of earthquakes.\textsuperscript{4-8,20,45} One of the methods to synthesize ground motion is based on the frequency domain representation as a double integral over two horizontal wave numbers. Another approach is based on the use of the representation theorem in which the response of the medium is expressed in terms of spatial and temporal convolutions of Green's functions with the slip function. The use of this approach is made possible by the recent development of effective methods to evaluate Green's functions for layered media.\textsuperscript{3,7,28} The required discretization of the spatial convolution involving the Green's functions and the slip on the fault limits the application of this approach to low frequencies.\textsuperscript{10}

\textbf{IV. GENERATION OF SITE SPECIFIC MOTIONS}

In many areas of earthquake engineering analysis the ground amplification effects caused by surface and subsurface irregularities can be an important factor. Therefore, a detailed understanding of these effects is of obvious value to earthquake engineering work. Models with simple geometries are usually employed to study the wave amplification problems. For studies of inhomogeneities smaller than the wavelengths of the incident waves and waves with long periods, these models seem to be satisfactory. However, there are many cases in earthquake engineering where near field ground motions should be included, in which the waves with shorter periods become important.

In earthquake damage studies for example, evidence of wave amplification has been observed. The influence of irregular geological structure or topography may overshadow the effects of local site conditions.\textsuperscript{18} The sites with softer alluvium yield higher amplitudes and longer durations than the "stiffer" sites and the intensity of ground shaking can vary within a short distance.\textsuperscript{15,28,29} The importance of the amplification caused by soil deposits has been recognized in microzonation studies.\textsuperscript{44} Due to such effects of surface and subsurface irregularities, many models have been proposed and studied by different researchers for better understanding of the amplification phenomena.\textsuperscript{39,44}

In reviewing the methods of analysis in this subject area, one finds discrete methods and continuous methods. The discrete methods include finite elements and finite differences. Each of these methods have limitations. Due to large dimensions in geophysical problems, the application of discrete methods may be limited. On the other hand, the applicability of continuous methods is mainly restricted to linear, isotopic and homogeneous materials and simple geometries.

Moeen-Vaziri and Trifunac\textsuperscript{28,29} have illustrated the methods of solution and the effects of subsurface inhomogeneities and irregularities of arbitrary shape on the ground motion amplification. Their method can be applied to analyze cross sections along profiles of irregular geologic basins, to investigate the effects of subsurface inhomogeneities on scattering and diffraction of incident waves. The surface displacement amplitudes or displacement amplitudes along buried horizontal lines (tunnels) can also be considered.
This type of analysis can also include computation of transient motions. This can be carried out using the transfer functions obtained from harmonic analysis. The accelerograms from the 1971 San Fernando earthquake have been used, for example, to illustrate the effects of subsurface inhomogeneities upon transient motions along different profiles of the Los Angeles basin. The results can be presented in terms of amplitude spectra and/or accelerograms at several points on the surface of the inhomogeneity.

REFERENCES


PRINCIPLES OF SEISMIC ZONING IN NORTH CHINA
by
C. Dasheng and S. Zhenliang
(China)

Summary
In this paper, the principles as well as the method adopted in compiling the zoning map of North China are briefly illustrated. Characteristics of the inhomogeneous distribution of earthquake strength and frequency in space and time are considered, and a probabilistic seismic hazard analysis method and an elliptical attenuation model, which simultaneously considered both directions along major and minor axis, are applied.

Introduction
In China, most regions show characteristics of intraplate seismicity, i.e. a broad distribution of earthquake epicenters with great earthquakes being relatively concentrated in seismic zones; with high magnitude, shallow focal depth, serious risk, and long recurrence periods. Recently, research work has been studied, particularly that concerned with long-term earthquake prediction, analysis of seismicity trends and seismotectonic characteristics of great earthquakes. These results and experience have been applied in compiling a new seismic zoning map. Characteristics of inhomogeneous distribution of earthquake size and frequency in space and time were considered, and the probabilistic seismic hazard analysis method, initially proposed by C.A. Cornell [1] was applied with some modifications. In this paper, the North China region has been selected for a case study, and the compiling principles as well as the method adopted for the zoning map are briefly illustrated. The paper is mainly based on reference [2].

The Seismic and Tectonic Background of North China
Active faults due to crustal tectonic movements during the Mesozoic and Cenozoic eras were formed in the Pre-Cambrian crystalline basement of the North China platform. Its tectonic background was formed by the many fault structures, active since the Quaternary, such as the Yinchuan faulting basin, Fenwei gragen belt, Hebei plain en echelon fault zone, Tancheng-Lujiang fault zone, and the fault zone roughly EW or NWW of the southern part
of the Yinshan-Yanshan-Langshan region. Earthquakes with $M \geq 6$ usually occur in these fault zones.

The general trend of modern active faults in North China is in a NNE direction, but conjugate active faults of a NWW direction also exist. Source mechanisms of recent great earthquakes (during 1966 to 1976) indicate that most active faults exhibited right-lateral strike slip movement in a NNE direction, and a few showed left-lateral strike slip movement in a conjugate NWW direction. Ground fissures generated by earthquakes and shapes of isoseismals in earthquake meizoseismal area, provide evidence that these two conjugate systems of faults are the major active faults in the area under consideration.

In North China, strong earthquakes usually occur in the following special structural locations: (a) Boundary zones of large active tectonic blocks; (b) Quaternary active faulting basins; (c) Active Holocene faults; (d) Intersections of two or more groups of active faults; (e) Conjugate sites of networks formed by epicenters of medium to strong earthquakes.

Long-term seismicity indicates that small and moderate earthquakes occur almost everywhere in the region. Strong earthquakes of $M_s > 6$ are mostly concentrated in certain seismo-tectonic zones, and great earthquakes of $M_s > 7$ only occur in some particular tectonic structures. More than one thousand years of earthquake data is available for statistical analysis. The earthquake periodic diagram shows a cycle roughly equal to 300 years. The strain release curve of North China shows that a periodic cycle can be divided into relatively active and relatively quiet periods. Using these features, a tendency of seismic activity in future $T$ years can be analyzed with certain confidence. After the occurrence of a great earthquake of $M_s > 7.5$, relative seismic security can be assumed, (except if the earthquake is one with strong aftershocks or of the twin mainshock type). The phenomenon is called immunity, and it is applied in the seismic hazard analysis of certain seismic regions.

Seismic Hazard Analysis

The probabilistic seismic hazard analysis method was applied, with some modifications, in compiling the new seismic zoning map on the background to seismicity in China. The frame diagram of seismic hazard analysis is as follows:
1. **The seismic zone as a statistical unit for evaluating seismicity parameters.**

Several seismic zones exist in the region and can be identified by the characteristics of their seismic activity and their seismo-tectonic environment. A seismic zone is a relatively independent and complete tectonic unit; the time-space distribution of its seismicity has a good consistency and correlation, but seismicity is different for different seismic zones. Therefore, the seismic zone is taken as a statistical unit for studying the characteristics and parameters of seismicity.

In evaluating the total mean annual occurrence rate of a seismic zone, an assessment of seismic trends is considered. The seismic trend in the future, say, for the next century may differ during different stages, i.e. a very active stage, a quiet stage or a stage corresponding to a mean level of seismicity in historical time.

2. **Delineation of potential sources and determination of upper bound magnitude.**

By analyzing the seismo-tectonic environment of earthquakes of $M_s \geq 6$, potential seismic source areas in the seismic zone under consideration are delineated. The upper bound magnitudes $M_{ui}$ of each potential seismic source are evaluated, based on comparing the seismo-tectonic environments of potential seismic source with causality structural indexes of earthquake occurrence of various magnitudes, and also the historical maximum magnitude. Six magnitude ranks (i.e. 6, 6.5, 7, 7.5, 8 and 8.5) are considered for upper bound magnitudes. Areas of potential seismic source are considered as background seismic regions with an upper bound magnitude 5.5.
3. Determination of interval frequency of magnitude \( N(M_j) \).

In order to consider the spatial-temporal inhomogeneous distribution of earthquake occurrence in seismic zones, and to evaluate the contribution of high magnitudes to seismic hazard on site, the total mean annual-occurrence rate of a seismic zone is distributed to various potential seismic sources in the seismic zone by means of weighting coefficients of various magnitudes.

For the \( i \)th potential seismic source, with the mean annual occurrence rate at \( M_j \) magnitude is as follows: \( N(M_j) = \frac{[N(M_j) - N(M_j + 1)] \cdot W_{ij}}{T} \cdot \exp(\alpha) \cdot W_{ij} \cdot \exp(\beta \cdot M_j - \exp(\beta \cdot M_j + 1)) / T \) where, \( W_{ij} \) is weighting coefficient of the \( i \)th potential seismic source at \( M_j \) rank of magnitude; \( M_j + 1 - M_j = 0.5 \) is the interval of magnitude rank; and \( \beta = b \cdot \ln(10) \).

For rank of \( M < 6 \), \( W_{ij} = \frac{A}{\sum A_i} \), it means that earthquakes are randomly distributed in all seismic source areas, and \( A \) is the area of the \( i \)th potential source. For the range of \( M = 6-7.9 \), the following factors are considered in determining the weighting coefficients: (a) relative probability of event occurrence in potential seismic sources obtained by the pattern recognition method; (b) results of mid-long term earthquake predictions; (c) immunity function of great earthquakes; (d) ability of earthquakes to recur and small earthquake activity; (e) earthquake precursor of super-long term and earthquake tectonic gap. For \( M \geq 8 \), the recurrence interval of paleo-earthquakes is considered.

4. Attenuation relationship

In North China, ground motion attenuation apparently differs in different directions. In seismic hazard analysis, this kind of orientation characteristics are considered, and an elliptical attenuation model is applied. The attenuation relationships of earthquake intensity and horizontal rock peak acceleration with distance are applied.

The intensity attenuation relationship is as follows: \( I_{a,b} = A + B \cdot M - C \cdot \ln(R_a + R_{ao}) - C \cdot \ln(R_b + R_{bo}) + \sigma \) and if, \( R_a = 0, R_b = 0 \); if \( R_b = 0, R_a = 0 \). Where, \( I_{a,b} \)-earthquake intensity along major and minor axis; \( M \)-earthquake magnitude; \( R_a \) and \( R_b \)-epicentral distances along major axis and minor axis; \( \sigma \)-standard deviation; \( R_{ao} \) and \( R_{bo} \) are specific constants; \( A, B, C \) and \( \sigma \) are regression constants. A method for estimating the acceleration attenuation relationship proposed in reference [3] is applied to derive the rock acceleration attenuation.
Discussion and Conclusion

Zoning maps have been compiled of earthquake intensity and horizontal PGA in North China for the next 50 years, with 10, 5 and 0.5 percent probability of exceedance. In determining the parameters of seismicity, the characteristics of inhomogeneous distribution of seismicity in space and time were considered.

From the results, several features can be summarized as follows:
(a) Those places where large earthquakes have recently occurred have their intensity ratings obviously lower than the epicentral intensities of those earthquakes, such as at Xingtai, Hailchong, Tangshan, etc.
(b) The occurrence time of a large earthquake in the past will influence the results; the longer the occurrence time, the higher the intensity level.
(c) New high intensity areas appear in some places where no great earthquake has occurred in the past.
(d) High intensity ratings also appear in mid-long term prediction areas where large earthquakes could occur.

It should be pointed out that these results are only preliminary, and further investigation will be carried out.

References
FEATURES OF THE SEISMIC EFFECT IN PLATFORM AND OROGEN AREAS

by

T. Rautian
(USSR)

We know that the character of earthquake oscillations is controlled by three factors:

- the character of source radiation, which is in turn dependent on source dimensions and the distribution of stresses within the source;
- attenuation of seismic waves as they travel in the medium;
- resonant processes at a site where the oscillations are recorded and produce damage.

The first two factors involve seismic zoning. On average, they do not greatly vary within comparatively extensive zones, similar in geologic and tectonic features. The third factor is local in character and should be studied for each individual site (nuclear power station, dam, city) when determining the earthquake hazard and seismic risk for that site.

We shall examine the first two factors, using the extensive experimental material provided by investigations conducted over more than 15 years:

The following data have been obtained:

(a) Earthquake source spectra in the form of displacement, velocity or acceleration spectra in the magnitude range 2.5-3.0 to 6.8-7.0;
(b) Quality factor as a function of frequency in the lithosphere and locally within the upper crust.

In addition to this, data have been summarized relating to the character of attenuation with hypocentral distance for macroseismic intensity.

The area of study, to be called here Central Asia, extends from Armenia and the Caspian Sea on the west to Xingian and Djungaria on the east, and from the Kazakhstan platform in the north to Afghanistan in the south. Only a small part of the region (northern Kazakhstan) can be regarded as a platform; the rest is an orogen with a high level of seismic activity.

Investigations show that all engineering-geological characteristics vary considerably in different parts of the seismically-active portion of the region. From the viewpoint of seismic zoning, they should not be combined; on the contrary, it may be interesting to examine them individually. For this reason the data from these zones will be considered separately in analyzing earthquake source spectra, and distortions of the record for large amplitudes due to local geology.
Comparisons between spectra from different zones can only be made after parameterization and the incorporation of the dependence on energy. The parameterization adopted in our studies is determined by features in the spectral method using coda recordings by seismic analog-filtering multichannel (ChISS) station.

Earthquake Source Spectra

Source spectra were determined by the method of seismic coda based on recordings of earthquakes by bandpass filters of approximately constant logarithmic width. The relative filter width for most of the channels is 0.5. For low frequencies, (0.14 Hz or lower), this is 0.7, while for high ones, (18 Hz or more), it is 0.22. The total range, from 40 seconds to 40 Hz, permits a complete description of earthquake source spectra in the magnitude range of interest.

The determination of source spectra is carried out through the spectral content of the coda, because this method is capable of high accuracy for very few stations. The set of coda levels $A_{100}(f)$ at a fixed instant of time ($t=100$ sec after the origin time for all frequencies) describes the spectral content of the coda for the earthquake under consideration. The displacement source spectrum $DIS(f)$ is calculated using the function $D(f)$:

$$D(f): \log DIS(f) = \log A_{100}(f) + D(f)(1)$$

The conversion to velocity and acceleration source spectra, $VEL(f)$ and $ACC(f)$ is straightforward:

$$\log VEL(f) = \log DIS(f) + \log (2\pi f)(2)$$
$$\log ACC(f) = \log DIS(f) + 2 \log (2\pi f)(3).$$

Spectra can be of two types. The first includes spectra with a single corner frequency $f_0$; the second type includes spectra with two corner frequencies: $f_1$ and $f_2$. They include an intermediate portion where the velocity spectrum either slightly increases or slightly decreases, or else does not vary, the displacement spectrum decreasing less strongly than the square of frequency.

Parameterization of the Source Spectra

Spectra can be parameterized by a comparatively small number of parameters. The minimum number of parameters for spectra of the first type is two. For example, a displacement spectrum of the first type is fixed by giving a corner frequency $f_0$ and spectral level $F_0$ for the low frequency portion: $F_0 = DIS(0 \leq f \leq (4f_0)$.

Velocity can be more easily parameterized. A spectrum of the first type is defined by fixing a corner frequency $f_0$ and the peak velocity spectrum $F$: $F_1 = VEL(f_0)(5)$. A spectrum of the second type being defined by two corner frequencies $f_1$ and $f_2$ and the values of velocity spectrum at these frequencies: $F_{11} = VEL(f_1)(6)$ and $F_{12} = VEL(f_2)(7)$. 

- 46 -
For our data it is also of interest to know the level of acceleration spectrum, $f_2 = \Lambda C C (f_2 > f_1)$.

We also used the seismic moment $M_0 = \mu F_0 (8)$ and seismic energy:

$$\log E = 2 \log F_1 + \log f_1 + \log \left\{ \frac{f_2}{f_1} \right\} \frac{1}{3} 0.35 (9)$$

The last formula in the case of a spectrum of the first type assumes the form: $\log E = 2 \log F_j + \log f_0 = 0.52 (10)$ while for a spectrum of the second type, if $f_j < f_2$ and $F_{j2} \approx F_{j2}$: $\log E = 2 \log F_{j2} + \log f_2 - 0.35 (11)$.

In addition to this, we compute the value of apparent stress $\eta \sigma$: $\eta \sigma = E/F_0 = E/M_0$.

The above-mentioned parameters characterize an earthquake source as a whole. However, many earthquake sources involve non-uniform processes in space and time. This non-uniformity should be taken into account both for understanding the relation of the non-uniformity to the geology and for practical purposes of zoning.

There is considerable evidence which tells us that earthquakes which involve a spectrum of the first type have simple motion at the source, while those with one of the second type have complex motion.

Earthquakes with a spectrum of the second type are here considered as a combination of two sources, subsources (SS), and Big source with a smooth process (BS). Accordingly, the spectrum is a sum of two simple spectra.

SS is a small part of the earthquake source (asperity) where comparatively high stresses have been concentrated prior to the earthquake. Rupture of SS initiates slip along adjacent portions of low strength (hence low stress), this process forming the BS spectrum. This interpretation enables one to compute seismic moments, energies and apparent stresses separately for SS and BS. Generally speaking, BS can also consist of several big subsources, but what is important for us is that the high frequencies and, hence, the bulk of energy, are radiated by the small subsource. The peak acceleration spectrum and motion duration with large amplitudes on the accelerogram are determined by the SS spectrum.

**Dependence of the Spectra on Energy (Magnitude, Moment)**

If the volume density of energy in the medium had been the same everywhere, the differences in earthquake energy would have been controlled by increases in source volume. In that case the sources of large and small earthquakes would have been similar. The geometrical dimensions of the source would increase (and the corner frequencies would decrease) in proportion to the cubic root of energy, while the seismic moment would do so in proportion to
the first power of energy. Consequently, apparent stresses must have been identical for large and small earthquakes, on average.

If this were so, the problem of seismic zoning would have been simple. However, observations show that the seismic moment increases less rapidly than energy within the magnitude range 2 to 7, approximately in proportion to the 0.75-0.8 power of energy. Corner frequencies, especially for smaller earthquakes, do not decrease as fast as power 1/3. Apparent stresses increase with increasing earthquake energy, particularly for earthquakes of lower magnitude. This is an extremely important feature which should be borne in mind when trying to draw conclusions about the spectra of large earthquakes from those of small ones in a particular zone.

Let us consider sets of large and small earthquakes in comparatively restricted zones; for example, the main shock and its aftershocks. It turns out that, instead of a smooth variation of frequencies, they all cluster around a few typical values, the transition occurring through variation of the spectral shape, while the corner frequencies vary in steps, from one typical frequency to another. This character of spectral distortion indicates a tendency of 'quantization' for block size and inter-block boundaries in the medium of a zone. The distributions of corner frequencies \( f_1 \) and \( f_2 \) are rather narrow within a wide range of magnitudes and are well separated.

**Regional Features of Source Spectra**

An analysis of apparent stresses at earthquake sources of different zones shows that zones of predominantly high and low values of \( \eta_0 \) can be identified in the region, hence those with high and low corner frequencies. Spectra from different zones differ greatly, as do the scatter of individual values of \( \eta_0 \).
<table>
<thead>
<tr>
<th>Source</th>
<th>Range of log E</th>
<th>Apparent Stresses</th>
<th>Number of Earthquakes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hindu Kush</td>
<td>14.0-15.9</td>
<td>30</td>
<td>1200</td>
</tr>
<tr>
<td>Pamir</td>
<td>12.0-13.9</td>
<td>28</td>
<td>1200</td>
</tr>
<tr>
<td>h &gt; 70</td>
<td>10.0-11.9</td>
<td>10</td>
<td>360</td>
</tr>
<tr>
<td>Gazli</td>
<td>12.5-14.5</td>
<td>50</td>
<td>1900</td>
</tr>
<tr>
<td>Northern Tien-Shan</td>
<td>12.0-13.9</td>
<td>80</td>
<td>280</td>
</tr>
<tr>
<td>Tien-Shan</td>
<td>10.0-11.9</td>
<td>5</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>8.0-9.9</td>
<td>1.8</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>7.0-7.9</td>
<td>1.1</td>
<td>3</td>
</tr>
<tr>
<td>West Hindu-Kush</td>
<td>12.0-15.0</td>
<td>1.6</td>
<td>130</td>
</tr>
<tr>
<td>Peter 1st Range</td>
<td>10.0-13.5</td>
<td>2.0</td>
<td>32</td>
</tr>
</tbody>
</table>

The apparent stresses at a subsourc $\eta \sigma^s$ and for the smooth component of the entire source $\eta \sigma^s$ as functions of energy ($E^s$ and $E^s_b$) vary widely. It turns out that there is a well-defined maximum value $\eta \sigma^s$ for each zone. This value reaches 5-10 kbars for the platform zone (Gazli), corresponding to the strength of hard rock. Different zones of the orogen have very different values of the upper strata:

- **Gazli**: 5-10 kbars
- **Northern and Western Tien-Shan**: 1-3 kbars
- **Southern Tien-Shan**: 600 bars
- **Alai Valley and Tadjik depression**: 200 bars
- **Armenia**: 50 bars

The values of the upper strata $\eta \sigma^s$, can be regarded as assessing the carrying capacity of tectonic features where tectonic stresses are concentrated, but which still rupture, producing earthquake subsources. The variations of $\eta \sigma^s$ and $\eta \sigma^s_b$ towards smaller values are very great. They indicate a perforated medium with many imperfections of the medium being nearly free of stress.

To sum up, identification of seismic zones with different characteristics requires the upper strata of $\eta \sigma^s$ as an essential parameter which can be regarded as a factor for mapping the conditions of earthquake occurrence. It is this parameter which controls the greatest possible value of corner frequency $f_2$ at a fixed energy of a large earthquake, as well as the level of acceleration spectrum $F_2$. 

- 49 -
Attenuation of Seismic Waves

The attenuation of seismic waves was investigated by two methods: summarizing the macroseismic data and assessing the $Q$ of the medium, $Q(f)$, as a function of frequency for different seismic zones. It is important to note that the attenuation of macroseismic intensity for platform zones is difficult to study because of absence or scarcity of information, since seismic activity in these areas is low, while the quality factor can be estimated from records of explosions. The use of the coda method enables one to achieve high accuracy for a comparatively small number of events and observations sites.

Attenuation was studied in such a way as to derive average characteristics for large zones. Local peculiarities associated with specific geologic features have not been considered. Such an averaging in the analysis of intensity attenuation was achieved by using many earthquakes and excluding from consideration the data for extended sources in the region of strongest isoseisms. $Q$ was estimated by using the seismic coda only, rather than by using direct waves.

Attenuation of Macroseismic Intensity

Macroseismic data were summarized for three regions: Soviet Central Asia, the Balkans, and the Caucasus. Attenuation is approximately identical at comparatively short distances (below 50 km) in different regions, only local differences were available and have been levelled off in summarizing the data. Beginning from 50 km upwards, however, one can clearly see patterns pertinent to comparatively large regions.

For Soviet Central Asia we have a well-defined decrease in the slope of attenuation curve $I(R)$ at distances of about 100-250 km with subsequent increasing slope of $I(R)$ with greater distance $R$.

For the Balkans the smaller slope portion is less strongly expressed and is shifted towards shorter distances. At greater distances the slope of the attenuation curve for macroseismic intensity is very large, much larger than that in Soviet Central Asia.

For the Caucasus there is practically no smaller slope portion. Here, too, macroseismic intensity is attenuated much stronger than in Soviet Central Asia.

We believe that the smaller slope portion in the curve is associated with the appearance of intensive waves reflected at the Moho. The larger distances corresponding to this portion in Soviet Central Asia compared with the Balkans are due to a greater crustal thickness in Central Asia. The strong attenuation in the Caucasus is caused by Quaternary volcanism in this area, while for the Balkans the presence of an asthenosphere and increased heat flow must
play a certain role. In contrast to these areas, the asthenosphere in Soviet Central Asia is not intermittent.

The well-defined association between the attenuation of macroseismic intensity and the presence of strongly attenuating volumes (asthenosphere, volcanism) beneath the Moho provides evidence that the seismic energy which is responsible for felt effects travels, not only within the crust, but also beneath the crustal bottom.

**Determination of Q by the Coda Method**

Recordings of earthquakes by using a set of bandpass filters (ChISS) enable one to determine Q from coda envelope shapes separately for each channel, thereby yielding the quality factor Q as a function of frequency.

Coda envelopes consist of several consecutive segments each of which gives its own Q value. Segment b is observed at short intervals and is here considered as a coda that forms in the upper crust. Segment c is observed at times of a few tens or hundreds of seconds to several hundred seconds. We believe that segment c forms in the crystalline crust and the lithosphere.

The quality factor $Q(f)$ increases with frequency approximately proportional to the square root of the frequency. However, values of $Q_c$ for a fixed frequency, (for example, $f = 1 \text{ Hz}$), are subject to strong regional variations; reaching 700 in Northern Kazakhstan, while being 560 in the southern margin of the Kazakhstan platform (Gazli). The quality factor for 1 Hz within the orogen zones decreases to 360-420 in Tien-Shan and to 200-250 in Kopet Dag and Armenia.

$Q_b$ as a function of frequency behaves in a different way. There is a range of frequency with an approximately constant $Q_b$, while for higher and lower frequencies this increases in proportion to approximately the square root of frequency, the range of constant $Q_b$ in different zones occurring at different frequencies. The values of $Q_b(f)$ themselves in the flat portion of the curve vary between 50 and 200. The values of $Q(f)$ at high frequencies (10 Hz) vary from 80 for the Peter the First Range to 1000 in Northern Kazakhstan.

We note the regional differences in Q as derived from instrumental data are consistent with the peculiarities of macroseismic attenuation in these regions.

Obviously, wave attenuation at short distances is controlled by the upper crust (segment b) which is locally variable. The attenuation at large distances, hence the area within isoseismals with a specified intensity, is controlled by the Q for segment c.

The coda method enables one to determine Q as a function of frequency using coda recordings from comparatively few seismic events.
AN INTEGRAL MODEL FOR EARTHQUAKE DAMAGE AND SEISMIC RISK ASSESSMENT

by

Z. Milutinovic and J. Petrovski
(Yugoslavia)

Summary

This paper presents an integrated prediction model for large-scale assessment of regional/urban seismic damage based upon seismological and instrumental data, regional and local studies, and damage records from past earthquakes. Damage predictions are made for masonry and reinforced-concrete frame low-rise Montenegrin buildings, by defining vulnerability functions, based on empirical data for 38,830 buildings (5,634,088 sq.m.) damaged by the 1979 Montenegro (Yugoslavia) earthquake and the strong motion data recorded from the same earthquake. A general long-run aspect of an overall regional earthquake risk-reduction programme is analyzed and presented through studying two hypothetical land-use scenarios in the same seismic environment, comprising the same regional, communal and zonal building gross area, but differing only in the adopted building typology.

Introduction

Although significant efforts have been made in the assessment of seismic hazard and the mitigation of its possible consequences, major earthquakes continue to cause enormous damage to the economy of affected regions and entire countries. Therefore, a specialized and comprehensive assessment of natural (and technological) hazards is required to solve this truly global problem of protecting orderly industrial development and accompanied urbanization patterns such as investments in regional and local infrastructure, life-lines, housing, urban furniture and other public and social activities against losses at all stages of their development.

Recent research and field surveys have shed a new light on the effects of natural disasters on technologically-organized society, thus indicating better approaches for providing more appropriate response of natural and local policy planners and other authorities of concern.
Outline of the Model and Outputs

An integrated large-scale prediction model for estimating regional/urban damage [refs. 1,2,3,4], Fig.1, involves a series of complex procedures which require a continuous and systematic approach to data collection, analysis and presentation. It incorporates four basic procedural steps that should be carried out consistently in the following sequence:

- Zoning of the regional/urban area with identification, inventory and mapping of existing and planned elements at risk - *Inventory Methodology*;
- Identification of the effects of local site soil conditions in modifying the severity of the event at a given location with prediction of ground motion determinants in terms of average effective response spectra ($S_{eff}$) - *Effective Response Spectra Methodology* [refs.2,3,5], Fig.2;
- Assessment of the vulnerability of identified elements at risk - *Loss Prediction Methodology* [refs. 1,2,3,4,5,6,7,8], Fig. 3; and,
- Seismic risk analysis and optimization of seismic losses (physical, functional and economic) for current or improved land-use scenarios [refs.1,3].

The spatial interaction of these factors determines the regional/urban loss-producing potential for the adopted seismic hazard scenarios or single earthquake event, and provides urban planners, public and social policy makers, scientists, engineers and other concerned authorities with:

(a) Regional/urban specific loss maps for selected elements at risk;
(b) Regional/urban damage distribution maps for each element at risk and superimposed maps providing information on cumulative damage distribution and spatial damage concentrations for all elements at risk;
(c) Cumulative figures on regional/urban losses pertinent to elements at risk adopted in the urbanization policy;
(d) Estimates on total physical, functional and economic losses that the region/urban area will suffer due to an earthquake event of predetermined magnitude or seismic hazard scenario justified by the level of economic development;
(e) Information on convenience, applicability and needs for improvement of existing construction standards, regulations, codes, etc.

Vulnerability Assessment

To forecast damages that are expected to occur during future earthquakes, it is necessary to know how various types of structures (elements at risk) have behaved when exposed to ground shaking of different intensities, i.e., to develop vulnerability functions (VF's) for various elements at risk and incorporate them in the loss prediction methodology of Fig.3.
This study uses vulnerability functions derived purely on an empirical basis. However, the model presented (Figs. 1, 2 and 3) may efficiently incorporate any set of consistently developed VF's (theoretical, empirical or experimental).

Fig. 4 shows the VF model accepted [refs. 1, 2, 3, 4, 6] and developed on the basis of data obtained by IZIIS's uniform methodology for building inspection and damage and usability classification [refs. 1, 3, 5] that employ five damage-rating and three usability levels scheme (Table 1) to describe earthquake-induced damages. The three curves (denoted by I, II and III) refer to physical vulnerability functions defined through corresponding damage ratios: 

$$DR(\%) = 100 \frac{NB_D}{NB} \text{ (for } i = I, II \text{ and III)}$$

where $NB_D$ is the number of buildings where only slight nonstructural, but negligible structural damage has been observed (Usable, D/U-C-I); $NB_{II}$ is the number of buildings with reported extensive non-structural and moderate structural damages (Temporarily non-usable, D/U-C-II); $NB_{III}$ is the number of buildings destroyed, where 'destroyed' means collapse during or immediately after an earthquake, or buildings damaged to the extent that neither economic nor technical justification is found for their repair and strengthening (Permanently unusable, D/U-C-III); and, $NB$ is the total number of buildings.

For derivation of empirical vulnerability functions employed is effective response spectra $S_{eff}$ [refs. 2, 5] developed on the basis of 1979 Montenegro damage and strong motion data. The $S_{eff}$ incorporates the effects of: (1) ground motion intensity (PGA) and duration ($T_d$), as ground motion parameters; (2) fundamental natural period of the building ($T_o$) and damping ($h$), as parameters describing the dynamic properties of a structure; and, (3) ductility ($\mu$) and number of response reversals, as representatives of the group of parameters related to structural capacity (SC). For a given site soil condition $S_{eff}$ can be approximated as [refs. 1, 2]:

$$S_{eff} = S_{eff}(M, \Delta, s, T_d, h, T_o, SC).$$

Some of the proposed vulnerability models are displayed in Figs. 5 and 6. They were derived separately for both D + U (D/U = Damage and Usability category) categories (Table 1) by considering the local site soil conditions at the location of the building. A damage data set of 21, 837 buildings (2,614,402 sq.m.) has been used for derivation of number-of-storey and site-dependent (Fig. 5) SM (SM = one-to-three story stone masonry buildings) vulnerability models. Considerably smaller damage data sets were available for deriving the BM (1,594 buildings with total gross area of 245,582 sq.m.) and STM, Fig. 6, (STM = one-to-three story brick masonry buildings; 13,727 buildings with total gross area of 2,021,922 sq.m.) vulnerability models.

**Estimation and Mapping of Regional Losses**

Two urban forms comprising the same regional, communal and zonal building gross areas, Table 2, but differing in prevailing building typology only, are studied for seismic hazard levels corresponding to 50 and 200 years return period (refs. 2, 8 and 10). In total, 4,164,150 sq.m. gross building area has been considered for land-use scenarios A and B, out of which
1,890,900 sq.m. allocated to SM building class (LU-S-A) are relocated to BM (30%) and STM (70%) building classes (LU-S-B). Results, in terms of 200 years return period percent loss maps superimposed for SM, BM, STM and RCFS building classes, are presented in Figs. 7 and 8 for LU-S-A and LU-S-B, respectively. Cumulative figures on total loss estimates are estimated in Table 3 for seismic hazard levels and return periods of 50 and 200 years.

Concluding Remarks

The specific contribution of the model to land-use planning in seismically prone regions is an integral consideration and evaluation of seismic hazard, vulnerability and seismic risk expressed in terms of cumulative physical and functional loss figures (Table 3) or mapped as partial or cumulative seismic losses for different elements at risk and land-use scenarios (Figs. 7 and 8). The application margins of the model proposed are demonstrated by analyzing a general long-run aspect of a hypothetical regional earthquake risk reduction program.

It was shown that altering and replacing the gross area of highly vulnerable stone masonry building class by brick (30%) and strengthened (70%) masonry classes (LU-S-B, Table 2) decrease the total physical losses from 1,408,205 sq.m. to 518,145 sq.m. (or for 63.2%, Table 3) and from 1,590,857 sq.m. to 700,855 sq.m. (or for 55.9%) for seismic hazard levels related to specific return periods of 50 and 200 years, respectively.

The methodology and results presented provide a rational approach for creating an urbanization policy, estimation of convenience, applicability and needs for improvement of existing construction standards, regulations, codes, etc., as well as for estimation of existing or achieved level of seismic safety.
Table 1. Criteria for classification of damage and usability of buildings

<table>
<thead>
<tr>
<th>Damage/Usability Category</th>
<th>Usability Category</th>
<th>Damage State</th>
<th>Damage Category</th>
<th>Damage Description</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>D/U-C-I</td>
<td>Usable</td>
<td>Slight nonstructural damage, very isolated or negligible structural damage</td>
<td>1</td>
<td>Without visible damage to the structural elements. Possible fine cracks in the wall and ceiling mortar. Hardly visible nonstructural and structural damage.</td>
<td>Buildings classified in damage category 1 and 2 are without decreased seismic capacity and do not pose danger to human life. Immediately usable or after removal of local hazard (cracked chimneys, attics or gable walls).</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>Cracks to the wall and ceiling mortar. Falling of large patches of mortar from wall and ceiling surface. Considerable cracks, or partial failure of chimneys, attics and gable walls. Disturbance, partial sliding, slitting and falling down of roof covering. Cracks in structural members</td>
<td></td>
</tr>
<tr>
<td>D/U-C-II</td>
<td>Temporarily usable</td>
<td>Extensive nonstructural damage, considerable structural damage but yet repairable structural system</td>
<td>3</td>
<td>Diagonal or other cracks to structural walls, walls between windows and similar structural elements. Large cracks in reinforced concrete structural elements : columns, beams, R.C. walls. Partially failed or failed chimneys, attics or gable walls. Disturbance, sliding and falling down of roof covering</td>
<td>Buildings classified in damage category 3 and 4 are with significantly decreased seismic capacity. Limited entry is permitted, unstable before repair and strengthening. Need for supporting and protection of the building and its surroundings should be considered.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4</td>
<td>Large cracks with or without disattachment of walls with crumpling of materials. Large cracks with crushed material of walls between windows and similar elements of structural walls. Large cracks with small dislocation of R.C. structural elements : columns, beams and R.C. walls. Slight dislocation of structural elements and the whole building.</td>
<td></td>
</tr>
<tr>
<td>D/U-C-III</td>
<td>Unusable</td>
<td>Destroyed, partially or totally collapsed structural system</td>
<td>5</td>
<td>Structural members and their connections are extremely damaged and dislocated. A large number of crushed structural elements. Considerable dislocation of the entire building and delevelling of roof structure. Partially or completely failed buildings</td>
<td>Buildings classified in category 5 are unsafe with possible sudden collapse. Entry is prohibited. Protection of streets and neighbouring buildings or urgent demolition required. In case of isolated or typical buildings decision for demolition should be based on economical study for repair and strengthening.</td>
</tr>
</tbody>
</table>
Table 2. LU-S-A and LU-S-B Communal/Regional Distribution of Considered Elements at Risk

<table>
<thead>
<tr>
<th>Commune</th>
<th>Elements at Risk (Building Classes)</th>
<th>Total Gross Area Sq. m.</th>
<th>Land-Use Scenario A</th>
<th>Land-Use Scenario B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SM Masonry Gross Area Sq. m. %</td>
<td>BM Masonry Gross Area Sq. m. %</td>
<td>STM Masonry Gross Area Sq. m. %</td>
<td>Total Gross Area Sq. m. %</td>
</tr>
<tr>
<td>A</td>
<td>208,050 42</td>
<td>59,850 12</td>
<td>220,850 46</td>
<td>409,750</td>
</tr>
<tr>
<td>B</td>
<td>472,050 57</td>
<td>103,500 13</td>
<td>244,050 30</td>
<td>821,100</td>
</tr>
<tr>
<td>C</td>
<td>224,400 56</td>
<td>45,000 11</td>
<td>132,000 33</td>
<td>402,000</td>
</tr>
<tr>
<td>D</td>
<td>315,000 38</td>
<td>78,750 10</td>
<td>437,850 52</td>
<td>831,600</td>
</tr>
<tr>
<td>E</td>
<td>280,800 46</td>
<td>81,000 13</td>
<td>243,000 41</td>
<td>604,800</td>
</tr>
<tr>
<td>F</td>
<td>300,000 39</td>
<td>101,250 10</td>
<td>513,750 51</td>
<td>1,005,000</td>
</tr>
<tr>
<td>Total</td>
<td>1,890,900 45</td>
<td>470,250 11</td>
<td>1,803,000 44</td>
<td>4,164,150</td>
</tr>
</tbody>
</table>

Table 3. LU-S-A and LU-S-B Cumulative Total Losses (in %) Related to Seff Hazard Levels of 50 and 200 Years Return Period

<table>
<thead>
<tr>
<th>Building Class</th>
<th>Total Gross Area Sq. m.</th>
<th>D/U-C-II (50 Years)</th>
<th>D/U-C-III (200 Years)</th>
<th>Total Losses (in%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D/U-C-II 50 Years</td>
<td>D/U-C-III 50 Years</td>
<td>D/U-C-II 200 Years</td>
<td>D/U-C-III 200 Years</td>
</tr>
<tr>
<td></td>
<td>52.3</td>
<td>33.2</td>
<td>8.2</td>
<td>12.0</td>
</tr>
<tr>
<td>Stone Masonry SM</td>
<td>1,800,900</td>
<td>52.3</td>
<td>33.2</td>
<td>8.2</td>
</tr>
<tr>
<td>Brick Masonry BM</td>
<td>470,250</td>
<td>23.0</td>
<td>29.5</td>
<td>4.7</td>
</tr>
<tr>
<td>Strengthened Masonry STM</td>
<td>947,250</td>
<td>6.4</td>
<td>10.1</td>
<td>1.2</td>
</tr>
<tr>
<td>Reinforced Concrete Frame Buildings RCFS</td>
<td>855,750</td>
<td>7.3</td>
<td>0.4</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>1,037,520</td>
<td>22.9</td>
<td>29.2</td>
<td>4.6</td>
</tr>
<tr>
<td>Brick Masonry BM</td>
<td>2,270,880</td>
<td>6.3</td>
<td>9.8</td>
<td>1.2</td>
</tr>
<tr>
<td>Strengthened Masonry STM</td>
<td>855,750</td>
<td>7.3</td>
<td>0.4</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>1,114,400</td>
<td>16.6</td>
<td>14.5</td>
<td>3.5</td>
</tr>
<tr>
<td>Total for the Region Land-Use Scenario A</td>
<td>4,164,150</td>
<td>23.3</td>
<td>31.7</td>
<td>4.5</td>
</tr>
<tr>
<td>Total for the Region Land-Use Scenario B</td>
<td>4,164,150</td>
<td>16.5</td>
<td>14.5</td>
<td>1.8</td>
</tr>
</tbody>
</table>
Fig. 1. General Earthquake Loss Prediction Methodology for a City/Region

Fig. 2. Site-Dependent Effective Response Spectra Prediction Methodology
Fig. 3. Loss Prediction Methodology for Group of Structures
Fig. 4. Physical (Functional) Vulnerability Functions for Adopted Damage/Useability Categories

Fig. 5. Empirical Vulnerability Functions Generalized for SM Building Class

Fig. 6. Empirical Vulnerability Functions Generalized for STM Building Class
References


ECONOMIC AND VULNERABILITY ANALYSIS
HAZARD, VULNERABILITY AND RISK - A COMMENTARY

by

V. Karnik

(Czechoslovakia)

During the last decade new terminology has emerged in connection with the development of methods of seismic hazard and risk assessments. The first serious attempt to unify basic terms was made by a group of experts convened by UNDRO in July 1979 (ref. 1). The definitions were further clarified by E.M. Fournier d'Albe 1982 (ref. 2). However, some confusion still exists in using, for instance, expressions such as hazard and risk. The following review summarizes our present concepts and understanding of basic terminology.

The word most frequently used is "seismicity". Its meaning developed over a long period of time and at present it is defined as a function linking magnitude M and the corresponding number of events occurring within a certain earthquake region during a unit time interval.

The expected number N of events of M > M in an earthquake region per unit time in most cases fits the empirical Gutenberg-Richter relationship:

\[ \log N(M) = a - \beta(M) \text{ or } N(M) = \alpha \exp(-\beta M) \text{ where } \alpha = \exp(a \ln 10), \beta = \ln 10. \]

The cumulative probability function is \( P(M) = \text{prob}(M \leq M_t) = 1 - \exp(-\beta M) \) and the probability density function of \( M \) is \( p(M) = \beta \exp(-\beta M) \).

The mean interval in years between earthquakes having \( M > M \) ("return period") is:

\[ T_M = \frac{1}{N(M) \alpha \exp(\beta M)}. \]

The mean magnitude \( M = 1/\beta \) for all earthquakes of \( M > 0 \). (For further details see e.g. ref. 6, C. Lomnitz, 1974.)

If \( \beta \) is assumed constant over a large area with several earthquake regions, the \( N(M) \) distribution can be replaced by a single figure equal to the number of events \( N_a \) corresponding to \( M = M_a \). \( N_a \) then defines the level of seismicity. The information on seismicity is complete only if the upper threshold value \( M_{\text{max}} \) is also indicated for the \( N(M) \) distribution valid within a given region.

It must be pointed out that the \( N(M) \) function only provides the information on the mean probability of the occurrence of earthquakes of different magnitude and not on earth-
It must be pointed out that the \( N(M) \) function only provides the information on the mean probability of the occurrence of earthquakes of different magnitude and not on earthquake effects. Such meaning is conveyed by the term "seismic hazard". This is defined as the probability of occurrence, within a specific time interval, of a ground motion of a certain intensity \( i \). The example in Fig. 1 gives curves of expected intensity which will not be exceeded at the given site with probability \( F \) during different periods of time \( (t = 5, 10, \ldots, 10,000 \text{ years}) \) (ref. 7, Schenkova et al 1981). The curve in Fig. 2 gives a simplified approach to seismic hazard expressed in terms of annual probability of occurrence of intensity \( i \).

In describing the intensity of strong ground motion it is preferable to use a well-defined physical quantity, e.g. ground acceleration, particle velocity or displacement instead of the macroseismic intensity. However, in regions with very little or no instrumental observations of strong ground motions, the macroseismic intensity still serves for hazard mapping or site hazard assessments.

If we work with the extreme probability of \( F_{\text{max}} \) i.e. that a certain strong motion parameter \( i \) will not be exceeded, we can write:

\[
F_{\text{max}} = \exp \left( \frac{t}{T_i} \right)
\]

where \( t \) is the exposure time and \( T \) is the average return period of \( i \) in years. If \( t = T \), \( F = e^1 = 0.37 \), i.e. the parameter \( i \) has the probability of \( P = 1 - F = 63\% \). The term "return period" of \( i \) is sometimes confused with the recurrence period of earthquakes. If \( P(i) \) is small we have to consider occurrences of \( i \) with long average return periods, e.g. for \( P = 0.01, t = 50 \text{ years}, T = 5,000 \text{ years} \).

\[
P(i) = 1 - \exp \left( \frac{t}{T_i} \right),
\]

can be written in a simplified form \( P = \frac{t}{T} \) if \( \frac{t}{T} \ll 1 \).

Then for \( t = 1 \text{ year} \) \( P = 1/T \), i.e. the reciprocal of the average return period equals the annual probability of, e.g., \( T = 500 \text{ years}, P = 0.002 \).

Seismic hazard in the probabilistic sense is calculated in different ways, for review see e.g. ref. 4: Karnik, Nersesov (1986). Confusion, however, originates when some authors use the expression "risk" for figures calculated according to the definition given above for hazard. Before the terminology is stabilized it is imperative that every author defines his expressions in the introductory part of his paper.

The term "risk" was recommended by the UNDRO expert group in 1979 to be used only in connection with expected losses. Thus, seismic risk \( R \) was defined as the probability of loss of value of a certain element exposed to seismic hazard, or in more general words, as expected loss due to a particular natural phenomena being a function of both hazard and vulnerability. Before writing the formula for risk it is necessary to introduce two other expressions, namely damage ratio \( DR \) and vulnerability \( V \).
Descriptions of damage degrees are part of the commentary to macroseismic intensity scales (MM, MSK). There are usually five or six damage degrees corresponding to different damage to a building of a certain category, ranging from no damage to collapse (Table 1). The damage ratio is a relative number expressed in % or as a number from 0.0 to 1.0 and is defined by the ratio:

\[
DR = \frac{\text{cost of repair (restoration)}}{\text{value of the building}}
\]

In various papers, this ratio is also called loss in value, degree of loss, specific loss or proportional loss but the meaning is the same. In fact damage ratio means the same as vulnerability which was defined by the UNDRO expert group in 1979 as "the degree of loss to a given element at risk or set of such elements resulting from the occurrence of a natural phenomenon of a given magnitude and expressed on a scale from 0 (no damage) to 1 (total loss)".

If we relate DR of a certain element at risk to the intensity \(i\) of ground motion we get a vulnerability function (example see Fig. 2). It is evident that vulnerability cannot be described by a single figure. An element, e.g. a building, may be vulnerable but not at risk unless it is exposed to hazard, i.e. to a potentially damaging ground motion. Since vulnerability is usually linked with a particular element the expression specific risk \(SR\) is used. It is obtained by combining the probabilities of vulnerability corresponding to each intensity, i.e.

\[
SR = \int_0^{i_{max}} V_i \cdot P_i \cdot di.
\]

In actual computations the hazard \(P\) is considered at fixed levels of \(i\).

An example of calculation of specific risk for category B buildings, defined according to the MSK scale, and for a site with hypothetical annual probabilities of recurrence of intensity \(i\), is given in Table 2. The individual values are read from Fig. 2; it follows that the specific risk \(SR\), i.e. the annual average loss, at the hypothetical site equals 0.011 = 1.1% for the buildings of category B.
TABLE 1

Relation between damage degree and damage ratios (1)
according to different authors

<table>
<thead>
<tr>
<th>Damage degree</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>MSK (Kárník et al)</td>
<td>2</td>
<td>10</td>
<td>30</td>
<td>80</td>
<td>100</td>
</tr>
<tr>
<td>Bulgaria (Paskalova)</td>
<td>5</td>
<td>20</td>
<td>40</td>
<td>80</td>
<td>100</td>
</tr>
<tr>
<td>China (Liu, person.com.)</td>
<td>20</td>
<td>40</td>
<td>60</td>
<td>80</td>
<td>100</td>
</tr>
<tr>
<td>USA (Petak, Atkisson)</td>
<td>1</td>
<td>2</td>
<td>8-65</td>
<td>65-100</td>
<td>100</td>
</tr>
<tr>
<td>Romania (Fournier d'Albe)</td>
<td>4</td>
<td>16</td>
<td>36</td>
<td>64</td>
<td>100</td>
</tr>
<tr>
<td>Yugoslavia (Fournier d'Albe)</td>
<td>0</td>
<td>6</td>
<td>25</td>
<td>56</td>
<td>100</td>
</tr>
</tbody>
</table>

TABLE 2

Calculation of the specific risk for buildings
of category B (MSK)

<table>
<thead>
<tr>
<th>I</th>
<th>VI</th>
<th>VII</th>
<th>VIII</th>
<th>IX</th>
</tr>
</thead>
<tbody>
<tr>
<td>V</td>
<td>0.02</td>
<td>0.07</td>
<td>0.13</td>
<td>0.26</td>
</tr>
<tr>
<td>P</td>
<td>0.036</td>
<td>0.022</td>
<td>0.013</td>
<td>0.009</td>
</tr>
<tr>
<td>R</td>
<td>0.0007</td>
<td>0.0014</td>
<td>0.0017</td>
<td>0.0023</td>
</tr>
</tbody>
</table>
Curves of the maximum expected intensity $I$ at a selected site which will not be exceeded with a given probability $F_{\max}(I)$ in different periods of time $t = 5, 10, \ldots, 10000$ years.

Figure 2
References

INTRODUCTION

So far one has nearly exclusively considered the physical damage but not indirect damage nor indirect loss or damage which we shall mostly call consequential damage. These losses very often weigh heavier than the direct damage proper. Moreover their effect on society and national economy can be profound and lasting.

Those trying to estimate the consequential losses of earthquakes will soon realize that reliable data are lacking on which to base estimate of the actual exposure. In addition many loss statements refer to earthquakes which happened several or even many years ago. This implies that one must allow for trends in values, in value-density and repair cost, and changes in the elements at risk. If mean damage ratios (MDR) for buildings which have been calculated from quite large samples harbour sizeable uncertainties it is evident that the few figures on indirect losses are associated with even larger error margins. The paper will also discuss some of the uncertainties associated with the important parameters used in the calculation of consequential losses from earthquakes.

We shall first briefly discuss aspects related to the probability of damaging earthquakes and in the following chapter those pertaining to indirect loss and damage resulting from earthquakes. The conclusions are summarizing important lessons and which further research would enhance the precision of estimates of the indirect consequences of earthquakes.

THE PROBABILITY OF DAMAGING EARTHQUAKES

A basic requirement for the probabilistic assessment of indirect damage and losses from earthquakes is adequate information on event probability, i.e. earthquake magnitude or intensity probability. It is appreciated that intensities, e.g. MM or MSK do not provide a precise yardstick for damage. Also acceleration is a very bad yardstick of damage. Acceleration-intensity correlations should not be used. Intensity scales render no assistance whatsoever in estimating indirect earthquake losses.

Ordinary earthquake maps do generally not offer much guidance regarding event probability because they give mostly only earthquakes or intensities observed in the past without stating frequencies or probabilities. We therefore developed a Seismic Index Map (SIM) many years ago. This map was refined repeatedly and eventually presented at the 8th World Conference on Earthquake Engineering in San Francisco in July, 1984 (1). A correlative evaluation of the latest versions also assists in locating seismic gaps, in addition to permitting intensity or magnitude probability assessments of the place. A simplified version of such a Seismic Index Map covering North America is shown in Fig. 1. Reference (2) also contains a descriptive catalogue of about 4,500 damaging historic earthquakes which assist assessments when addressing zones of low seismicity. It must, however, be noted that historical catalogues are bound to be incomplete for many reasons and therefore generally only indicate the minimum seismicity one should be prepared for.

The Seismic Indices (SI) are used to calculate for instance the return period \( R \) of selected intensities (MM) or MDR's with the aid of the following formula (cf. (1))

\[
R_{\text{MM; MDR}} = \frac{A_{\text{count}} n_g \text{ OP}}{f_G f_T f \text{ SI } A_{\text{eff}}}
\]
Herein $A_{\text{count}}$ is the area of the counting ellipse used when developing the attached SIM. In the cases presented here it is 125,000 km$^2$, $n_G$ is the global annual number of reference magnitude earthquakes (M 7 - 7.9), i.e. 17.74, OP is the observational period used (79 years) (the new maps to be published are based on 90 years), $f_G$ is a correction factor for seismic gaps and $f_T$ one for seismic trends, whereas $f$ takes care of the statistical uncertainty related to the observational sample and the confidence range. $S_I$ is the seismic index of the place studied and taken from the SIM. $A_{\text{eff}}$ is the effective area of intensities selected from Fig. 2. A general description of the model used is presented in (1), a detailed discussion with numerous examples is included in (2).

For a $S_I = 1$ and taking all correction factors equal to 1 we may calculate that the average return period for MM VIII and buildings of a quality similar to UBC 3 is about 622 years. From Fig. 3 it is apparent that for buildings which correspond about to UBC 3, which are of moderate irregularity and asymmetry and are founded on medium-hard alluvium, an MDR of about 12% and for such buildings similar to UBC 2 one of 25% is to be assumed (cf. also 2-9 & 21-23). With a different formula (cf.1) it is possible to calculate the probability of earthquakes of selected magnitudes or of certain acceleration levels (2).

An assessment must allow for the most salient uncertainties, like inadequate observations and seismic gaps. In seismic gaps the actual exposure is substantially higher than average. If no specific studies are available one may use the SIM to check whether there is a seismic gap in the area under study. It is, for instance, seen in Fig. 1 that the region of Los Angeles has a $S_I$ which is about 60% of those NNW and SSE of it. This would suggest a correction factor of about 1.7.

From an evaluation of our large catalogue of historic earthquakes we conclude that the non-randomness of energy release seen in seismic gaps also holds on a global scale resulting in what we call seismic trends. For instance, it may be assumed that part of the last global very active period lasting from about 1852 until 1911 is responsible for the high $S_I$ in the region of San Francisco. Consulting historic earthquake records (2, 10) and the average tectonic displacement one may estimate that the SIM does not show a realistic long-term $S_I$ for the areas neighbouring Los Angeles and that by a factor ($f_T$) of about 1.6 seems advisable.

The statistical uncertainty associated with the observational sample appears to be a fairly straightforward problem of stochastics, however, as shown in (2) an uncritical application of mathematics, e.g. calculating confidence ranges from very few earthquakes or intensities observed during, say 80 years produces correction (safety) factors which do not appear to be realistic.

We should mention here errors when estimating the maximum magnitude probable in a region. Many very large earthquakes which happen in the past were not associated with impressive large or visible fault systems. For instance, the largest earthquakes known from China or from the central or the eastern United States did not happen in regions with conspicuous fault systems or seismicity.

The economic consequences of earthquakes to a country, that is the consequential losses, are also related to the occurrence of aftershocks or of "multiple events" and to their geographic distribution. Tangshan (1976) or Mexico (1985) are only two instances in which very strong earthquakes followed the main event within hours. A detailed investigation shows (2) that multiple damaging events, which may affect a very large area, are not the exception but sometimes rather the rule, in particular in areas with seismic gaps or during phases of substantial global seismic activity.

This shows that it must become practice in seismicity evaluations to state the various uncertainties and to work with safety factors or margins.

**LOSS AND DAMAGE CAUSED BY EARTHQUAKES**

Reference (2) contains a detailed description of the method applied by us successfully for many years, of the statistical data and of the parameters controlling loss and/or damage. It also contains a large
number of worked examples of risk evaluations covering direct and indirect losses and dealing with buildings, factories, plants, life-lines, utilities and the infrastructure. References (11-13 & 21-23) cover this issue either in a more introductory manner or some special aspects of it.

The factors which determine the magnitude of earthquake losses are in particular:

- Size of the strongly shaken area which is predominantly controlled by the type of faulting, magnitude of the earthquake, the rupture history, and the attenuation which depends critically on the type of subsoil.

- Level of damage at selected intensity. In addition to the intensity and time history of shaking this depends on the quality/vulnerability/sensitivity of the exposed objects; each of which is influenced by several parameters. Damage to buildings from earthquake shaking may, for instance, be aggravated by foundation failure, quality of the subsoil, resonance between subsoil and building, soil liquefaction, asymmetry, orientation of the structure, fire and/or explosion, and tsunami.

- Level of investment. Here we consider types of risks, "density" of investments, and the general income standards (GNP, GDP).

- Non-property and indirect losses. One must allow, e.g., for loss of life and injury, business interruption and, probably increasingly in future, professional liability, contamination of the environment, etc.

We shall now briefly discuss the above mentioned parameters and also mention some of the uncertainties which must be considered.

The size of the strongly shaken area, as given by Fig. 2 or, in a correlation with earthquake magnitudes, in the following Table 1, has been derived for medium-hard alluvium from more than one thousand observations in many countries. Still, for any important evaluation of economic consequences of earthquakes the global values must be corrected allowing for local characteristics considering that each one has its own characteristics. The influence of bad (soft) subsoil and of site effects ("resonance") cannot be overestimated. The Mexican earthquake of 1985 did severe damage to buildings of more than 5 floors nearly 400 km from the epicenter and caused very large indirect losses. This also holds for other places.

For medium-hard alluvium some approximate average gross areas (km$^2$) experiencing selected intensities, for instance, MM VIII and above, are shown in Table 1. As pointed out above very circumspective use of such data is a must if the exposure is considerable.

<table>
<thead>
<tr>
<th>INTENSITY MM</th>
<th>VII</th>
<th>VIII</th>
<th>X - XII</th>
</tr>
</thead>
<tbody>
<tr>
<td>M 7.5</td>
<td>55,000</td>
<td>20,000</td>
<td>6,200</td>
</tr>
<tr>
<td>M 8</td>
<td>84,000</td>
<td>36,000</td>
<td>12,000</td>
</tr>
<tr>
<td>M 8.5</td>
<td>130,000</td>
<td>36,000</td>
<td>19,000</td>
</tr>
</tbody>
</table>

Note: Surface wave magnitudes are stated.

When discussing the level of damage we concentrate on buildings because it is virtually impossible to deal with the multitude of other objects in a short paper. For buildings of moderate irregularity which are founded on medium-hard alluvium the approximate mean damage ratios (MDR) in percent of their new value are as follows:
TABLE 2

MDR (%) FOR MODERATELY IRREGULAR BUILDINGS
ON MEDIUM-HARD ALLUVIUM

<table>
<thead>
<tr>
<th>INTENSITY</th>
<th>MM</th>
<th>VII</th>
<th>VIII</th>
<th>IX</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adobe, Rubble Masonry</td>
<td>55</td>
<td>84</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Brick, unreinforced</td>
<td>28</td>
<td>55</td>
<td>85</td>
<td></td>
</tr>
<tr>
<td>Reinf. Concrete 2-3% g</td>
<td>11</td>
<td>28</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>3-4% g</td>
<td>6</td>
<td>17</td>
<td>36</td>
<td></td>
</tr>
<tr>
<td>6% g</td>
<td>3</td>
<td>11</td>
<td>26</td>
<td></td>
</tr>
<tr>
<td>12% g</td>
<td>1</td>
<td>6.5</td>
<td>17</td>
<td></td>
</tr>
<tr>
<td>20% g</td>
<td>3</td>
<td>12</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The above MDR's are based on the statistical evaluation of damage caused by about 30 earthquakes to substantially in excess of 150,000 buildings.

It must be stressed that the average, the probable and in particular the possible maximum loss (MPL, PML) can be substantially higher than the MDR shown in the above table. This must be considered when evaluating the consequential losses caused by catastrophic earthquakes. One would be well advised to consider a factor of two or even three when working with MDR's below 30%.

The MDR's given in Table 2 do not allow for aggravating factors like fire following earthquake, liquefaction, tsunami, etc. It must also be noted that any subsoil softer than medium-hard alluvium will generally result in substantially higher MDR's.

In addition to the investment level in the earthquake area the general standard of prosperity and trends in values and repair expenses must be considered. One may work tentatively with a GNP (GDP) - yardstick, correcting, however, for local parameters. When assessing the cumulative loss potential it is also very important to screen the region for large elements at risks or others of medium size which incorporate the chance of an extreme loss level.

For non-property and indirect losses only general rules can be given here. Loss of life and injury depends critically on the quality and type of buildings in the area (25). Also the time of the day and the season when the earthquake happens play a role. The values given in Table 3 are very approximate death rates in percent of the population in the respective isoseismal zone. They do not allow for aggravating factors. Again it must be stressed that MPL and PML-death rates may be much higher.

TABLE 3

APPROXIMATE DEATH RATE IN PERCENT OF THE POPULATION

<table>
<thead>
<tr>
<th>INTENSITY</th>
<th>MM</th>
<th>VII</th>
<th>VIII</th>
<th>IX</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adobe, Rubble Masonry</td>
<td>20</td>
<td>40</td>
<td>80</td>
<td></td>
</tr>
<tr>
<td>Brick, unreinforced</td>
<td>2</td>
<td>10</td>
<td>33</td>
<td></td>
</tr>
<tr>
<td>Reinf. Concrete 2-3% g</td>
<td>0.05</td>
<td>1.5</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>6% g</td>
<td>0.08</td>
<td>0.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20% g</td>
<td>0.04</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

As regards injuries, as a rule of thumb one may multiply the death rate by about 3 to 6. The ratio is higher for low intensity shaking and settlements with high-quality buildings and lower for high intensities or if the buildings are of inferior quality. The loss of life and injuries may be worsened due to fire and explosion following the earthquake or by the release of toxic substances. Also tsunami may lead to large loss of life.

Indirect losses are so far mostly associated with business interruption. Indirect losses are, however, caused by many more indirect consequences. They range from the loss of lives and injury to the
lasting loss to economy because of the working force killed, expenses for those injured and
permanently disabled, emigration of people if the economic structure in the region suffers lasting
damage, loss of tourism, contamination of the environment, etc. It is to be anticipated that
professional liability or third party liability, will be involved increasingly.

We shall now present a simple example to illustrate a method which may be used to calculate the
economic consequences of earthquakes. The example illustrates a general assessment procedure but not
certain figures.

A town of about 100,000 inhabitants in a low-income level area has been selected which is founded
on medium-hard alluvium and has moderately asymmetric buildings of a quality corresponding to 2 -
3% g. The example covers only the MM VIII or MSK VIII zone; it is based on MDR's and not on
maximum loss levels. All values are US dollars.

1. DIRECT LOSSES

<table>
<thead>
<tr>
<th>Category</th>
<th>Description</th>
<th>Cost/Inhabitant</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential buildings</td>
<td>Constr. cost abt. 4,000 per inhabitant, MDR 30%</td>
<td>$1,200</td>
</tr>
<tr>
<td>Commercial bldgs. &amp; equip.</td>
<td>40% of working population of 30,000</td>
<td></td>
</tr>
<tr>
<td>Commercial bldgs. &amp; equip.</td>
<td>50 cubic metres per person, 150.- per cubic metre, MDR 30%</td>
<td>$270</td>
</tr>
<tr>
<td>Factories</td>
<td>50% of working pop. 150 cubicm./pers. 100.-/cubicmetre, MDR 30%</td>
<td>$675</td>
</tr>
<tr>
<td>Machinery</td>
<td>abt. 1/3 rd. of factory buildings</td>
<td>$225</td>
</tr>
<tr>
<td>Contents</td>
<td>Private 2,000.-/inhab., MDR 10%</td>
<td>$200</td>
</tr>
<tr>
<td></td>
<td>Merchandise 100.-/inhab in stock, MDR 10%</td>
<td>$10</td>
</tr>
<tr>
<td>Vehicles</td>
<td>prob. of damage = 0.2, 5,000/car, 1 car/4 inhab., MDR 12%</td>
<td>$30</td>
</tr>
<tr>
<td>People killed</td>
<td>abt. 1,300 people, 20,000 each</td>
<td>$260</td>
</tr>
<tr>
<td>Injuries</td>
<td>abt. 6,500 persons, 1,000 each (Casualties will be re-evaluated below)</td>
<td>$65</td>
</tr>
</tbody>
</table>

2. INDIRECT LOSSES

<table>
<thead>
<tr>
<th>Category</th>
<th>Description</th>
<th>Cost/Inhabitant</th>
</tr>
</thead>
<tbody>
<tr>
<td>General Loss of Production</td>
<td>abt. 15,000 people affected for 3 months, 15,000 each</td>
<td>$562</td>
</tr>
<tr>
<td></td>
<td>Interest, Depreciation, Continuing Expenses, Loss of Markets, Tourism, etc. (Depending on actual case)</td>
<td></td>
</tr>
</tbody>
</table>

3. OTHER LOSSES/DAMAGE

<table>
<thead>
<tr>
<th>Category</th>
<th>Description</th>
<th>Cost/Inhabitant</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transport, Power, Water, Sewage, Telephones, Roads, Bridges, Hospitals, Schools, Gov. Bldgs. &amp; Administration, Churches, Museums, Cultural Heritage, etc. (These losses can be very heavy but are difficult to state in a general way as they depend on the local setting.)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Even if no aggravating factors prevail the mean total loss from damage to buildings, commerce and
factories can reach or even exceed $10,000 per inhabitant in the strongly shaken area. It will be
about three times higher in a high-investment area. These figures are supported by many
observations.

We shall now return to the loss of human life because the above list of losses includes the cost of loss
of lives and of injuries only on the basis of a single expenditure, like a demand on life assurance or
single medical treatment. In fact the loss to society and national economy is very much greater, not to
speak of the human misery caused by the casualties.

The 1,300 people we assumed as killed by the earthquake are in fact a permanent loss to economy.
Applying actuarial methods and considering the average age in the region, the average production per
person, the age at retirement and the life expectancy one can calculate the additional total loss to
national economy. We shall not go for lengthy calculations but we shall be satisfied with a very simple example.

We shall also allow for indirect losses due to interest on the idling capital, loss of rent, wages and salaries and other overhead expenses which continue during the period of interruption of commerce and industry. Moreover, the cost of those permanently disabled will be considered.

In our example these indirect losses add more than $2,000 per inhabitant to the bill, i.e. more than $200 million for this town of 100,000 inhabitants. It should be noted that there are many other indirect cost items which we did not consider in this simple example. The most important item is the loss to national economy due to the lives lost. It amounts to more than 60% of the items considered here.

The next important item is rent, interest and overheads amounting to about 28%. As loss of life and business interruption and therefore general overheads are decisively determined by the vulnerability of the buildings it is evident that any improvement in this field will reduce the impact of an earthquake enormously.

If the town is a tourist resort at a beach the indirect loss could be much higher than mentioned earlier. Hotels are in general notoriously vulnerable because they are often founded on soft material next to the sea and because of their highly asymmetrical and irregular (showy) design. Therefor their MDR is substantially higher than the one of ordinary buildings and this means in the context of this paper more extensive and time consuming repairs. If, for instance, about 2,000 tourists cannot be accommodated for an average period of 6 months a loss of income of about $36 million will result. To this we have to add the interest on the investments lying idle. In this case we must be prepared for complete business interruption for most hotels because there will be not only some severe structural damage but cracks in the walls of most rooms. It is not possible to accommodate guests when bricklayers, plumbers and electricians swarm over the place. The loss of interest alone may be something like $7 million or more. Here to some wages, the salaries, bills for utilities and other overheads must be added.

The indirect losses can even be graver if there is a chemical plant next to the town which contains inflammable, explosive, or toxic materials, or if the area is hit by a devastating tsunami.

Considering now the damage level and other factors determining the economic consequences one must first of all stress that no great earthquake is required to produce loss levels and indirect losses as discussed in the examples (cf., e.g. (21)). A large earthquake happening near to a high-value region can cause much higher losses. The loss levels stated in the respective tables and used in the examples do, for instance, not allow for the adverse effect of damaging shaking of abnormally long duration. Moreover, we assumed a rather homogeneous mix of risks.

As discussed earlier, the levels of direct and indirect damage and losses can be substantially higher than those used in the examples if only because of the inevitable scatter. In addition there are a number of aggravating factors which are bound to have a negative effect on the economic consequences. The most essential parameters to be considered in this respect are:

- Bad subsoil, i.e. any foundation material which is softer than medium-hard alluvium.
- Liquefaction
- Buildings or structures which are more than moderately asymmetrical. (Buildings with a soft ground floor are literally deadly structures and they are often found in tropical earthquake zones.)
- Fire and explosions following earthquakes.
- Tsunami.
- Abnormally long business interruption.
- Strong aftershocks or multiple events.
- Trend in cost of loss and damage, i.e. one must consider that the former loss values may be soon out of date.
An error in the maximum loss level assumed or only the differences in loss levels possible within the confidence range selected can lead to a dramatic deterioration of the picture. This is illustrated by Fig. 4.

REDUCTION OF CONSEQUENTIAL LOSSES BY RISK OPTIMIZATION

Quite a number of damage parameters have been mentioned in this paper. Their intimate knowledge shows that "building stronger" by implementing a sweeping general "improvement" of an earthquake building code, is a simplistic approach indeed. A much more economic reduction of earthquake losses is possible if available knowledge is utilized and lessons learned in the past are heeded. We shall again revert to Mexico to highlight this.

It was known since the damaging earthquake of 1957 that subsoil of the Fondo del Lago region of Mexico City has a natural period of about 2 seconds. It was similarly known that tall buildings located in this zone are damaged selectively (19). The amplification of damped forced vibrations depending on the frequency of the force and those of the excited structure is, for instance, very much considered in mechanical engineering. Also in earthquake engineering, papers have been published on this subject (cf., e.g. (20)). Still, when a team of us spent weeks in Mexico after the 1985 earthquake inspecting in particular all buildings of more than 5 floors in the central part of Mexico City where once more selective damage had happened to tall buildings, we regretfully noted that repairs showed that still the lesson had not been learned. Broken columns or beams were jacketed restoring about the pre-earthquake strength or natural period of the building but the chance was missed to build shear walls replacing some of the shattered brick walls. The latter would have raised the stiffness and the natural frequency of these buildings thereby reducing the probability of damage, of casualties, and of indirect losses during future earthquakes. Like after the Mexican earthquakes of 1957 and 1979, also in 1985 stiff buildings suffered much less than soft ones even if the stiffness resulted only from extensive fill-in walls (9, 22).

A further parameter which can be improved very economically relates to asymmetry and irregularity. It is generally not appreciated that symmetrical buildings suffer a fraction of the damage otherwise comparable asymmetrical ones do, although both have received the same level of engineering (24).
Fig. 1. This illustration shows a somewhat simplified version of our Seismic Index Map (SIM). It covers part of the North American continent. Only instrumentally recorded earthquakes which occurred between 1897 and 1975 have been considered when developing this version. This sample is inadequate to assess the seismicity of regions where the average return period of earthquakes is long compared with the observational period. In such cases the precision of assessments may be enhanced by considering historic records of seismicity, tectonic features like faults, plate movement, etc. The map also helps to locate seismic gaps and to assess seismic trends. For instance, the seismic index (SI) near San Francisco is substantially above the one of the San Andreas Fault system to the south. This is due to the great earthquake of 1906. Disproportionately low seismic indices, like in the region of Los Angeles, however, suggest that a seismic gap should be assumed. If the two factors are combined it may be estimated that the SI near Los Angeles could be about 8. The SI may be used to calculate the average return period of selected intensities. With the formula described in the text one would, for instance, calculate an average return period of about 600 years for MM VIII if the SI of a place is 1, or 75 years if the SI is 8. According to Fig 3, this intensity may produce MDR's of 12 or 25% respectively for moderately asymmetric buildings founded on medium-hard alluvium and conforming to UBC3 or UBC 2 respectively. The return periods stated above do not yet consider a correction (safety) factor associated with the size of the sample and the confidence level desired.
Fig. 2. Graphs of the effective MMI-areas which result if the effect of depth distribution of hypocenters as regards numbers of events per depth-range and the corresponding reduction in isoseismal area is considered. The graphs have been calculated for the global average depth distribution, global average isoseismal area per one tenth magnitude steps, and lower cut-off magnitudes ranging about between \( m_b \) 4 to 4.5 and M 5.8 depending on building standards. The latter has been indicated as base shear strength in percent of gravity, i.e. ranging from 1% g to 20% g in this case. Therefore a number of corrections are needed if assessments are made in regions which deviate noticeably from averages as used here.
Fig. 3. Correlation between some quality levels of buildings and intensities showing the mean damage ratio (MDR) to be expected. The MDR-correlation shown here is applicable to moderately asymmetric buildings founded on medium-hard alluvium. Although these graphs have been developed from a total sample of in excess of 150,000 buildings uncertainties are still considerable for high-quality buildings and for intensities above MM VIII. Buildings with a reinforced concrete structure (RC) are differentiated here according to the base shear strength in percent of gravity. As described in the text, damage depends on many factors and careful corrections are therefore necessary before evaluating the cumulative damage and the economic consequences of large earthquakes.
Fig. 4. The schematic illustration on the left shows the correlation between the mean damage ratio (MDR) of a fairly homogeneous sample of, e.g. buildings and the average return period. Further, the differences which may be associated with two confidence ranges (C) have been shown by graphs as well as the range of return periods associated with the confidence ranges. It is seen that for an assumed maximum loss level (ML-level) one must allow for very different damage probability distributions (two illustrations on the right) depending on whether the actual loss ratio is substantially higher or lower than the MDR-R-graph. If the loss level should be much higher than the average assumed, e.g. because of soft subsoil, resonance, asymmetrical buildings, incompatibility of building material as regards brittleness in structural and non-structural members and items, long duration of shaking, etc., one can be certain that there will be far more buildings in high damage states than in the converse case. As discussed in the text, the loss ratio experienced by a homogeneous large sample may be some hundred percent above the MDR even if no aggravating factors are at hand. Such issues are of considerable importance in connection with estimates of the economic consequences of earthquakes.
REFERENCES
8. Swissre, 1982, Earthquake Risk Assessment, Swiss Reinsurance Co., Zurich, Switzerland
10. Swissre, 1977, Atlas on Seismicity and Volcanism, Swiss Reinsurance Co., Zurich, Switzerland
18. Tiedemann, H. 1980, A Statistical Evaluation of the Importance of Non-Structural damage to Buildings, 7th WCEE, Vo. 6, 617
BUILDING DAMAGE CLASSIFICATION AND LOSS ASSESSMENT
FOR RISK MITIGATION

by
J. Petrovski
(Yugoslavia)

Summary

The objective of this paper is to present a uniform procedure for examining and reporting building damage both in urban and in rural areas so that a data bank on earthquake effects might be established and used for the effective estimation of economic losses. Assessment of direct economic losses is illustrated by a summary presentation of earthquake damage classification performed on 16,478 residential buildings and on all 57,640 buildings damaged by the earthquake of July 26, 1963 in Skopje and April 15, 1979 in Montenegro earthquakes, respectively. Use of these methods and procedures will yield an adequate volume of data to assist community and national authorities in the elaboration and performance of effective seismic risk reduction programmes.

Introduction

During the last two decades natural disasters, and earthquakes in particular, have tended to become increasingly destructive as they affect ever larger concentrations of population and material property. Industrial development of seismic-prone regions that are ordinarily accompanied by urban expansion and increased population becomes prohibitive unless investments in infrastructure, housing, other public and social activities, etc., are protected against damage at all stages of their development.

Although significant efforts have been put into assessment and mitigation of the possible consequences of the existing seismic hazard, major earthquakes occurring in this period induced enormous damage to the economy of stricken regions and entire countries. Moreover, due to the high concentration of material property in seismically-active regions, a significant increase of damage might be expected in future major earthquake events.

A damaging earthquake provides an opportunity to acquire unique technical information about the physical effects of ground shaking, surface fault rupturing, earthquake-induced ground failures, regional tectonic deformation, wave inundation from seiches and tsunamis, etc. Technical information should primarily be acquired on the following scales:
• global, in order to obtain a large - overall picture of the global tectonic forces;
• regional, in order to define the physical parameters and the range of their values for providing a rational understanding of the spatial and temporal characteristics of the earthquake activity which the region is exposed;
• local, in order to determine the physical parameters and the range of their values that control the site-specific characteristics of the earthquake hazards; and
• engineering, in order to provide data that can be correlated with the spatial dimensions of specific structures, facilities, life-lines or any other element at risk being under the engineering relevance.

The information and lessons learned from post-earthquake investigations, provide a basis for identifying the present situation and give rise to necessary changes. The new information can be utilized in research studies, in assessment of earthquake hazards and risk for specific urban areas, in mitigation and preparedness actions, and in the implementation of new and improved loss-reduction measures.

**Earthquake Damage and Usability Classification**

The focus of this paper is therefore to present the developed uniform methodology for earthquake damage assessment through inspection, classification and reporting of earthquake damaged buildings in urban and/or rural regions, necessary for a reliable estimation of physical, functional and economic losses. The principal aim of developing this methodology and procedure for earthquake damage assessment is primarily to assure an adequate volume of data for the following needs:

• To reduce incidents of death and injury to occupants of buildings that have been seriously weakened or damaged by a strong seismic event and most probably will be exposed to a series of aftershocks immediately after the main shock;
• To obtain realistic information on the magnitude of the disaster in terms of the number of usable, damaged and dangerous buildings for the purpose of immediate protection of human lives, sheltering and housing of citizens, urgent revitalization of the basic life and social activities, etc.;
• To improve the knowledge of the amplitude, frequency contents, temporal and spatial distribution of ground shaking and its causative relations with damage in buildings and triggering of other earthquake-related physical effects;
• To provide a data base for the uniform estimation of economic losses for development of appropriate rehabilitation programme and assistance in the reconstruction and future development of the affected region on the basis of improved seismic design regulations, codes and construction standards;
• To create a data base for the prediction of the consequences of future earthquakes in affected and other seismic regions;
• To extend the state-of-knowledge of seismic zoning, in order to push the limits of seismic microzoning to boundaries established by local and engineering scales;
• To provide data for the planning and organization of civil defense systems, elaboration of rescue operation plans, staff training, organization of emergency supplies, etc.;
• To record and classify damages for planning and performance of repair and strengthening of damaged buildings;
• To identify the principle elements of earthquake damage and to develop vulnerability relationships for different building categories indispensable for planning and performance of short-, intermediate- and long-term priority actions for the reduction of earthquake consequences and pre-earthquake assessments;
• To improve seismic design and construction codes and regulations, as well as design and construction practice;
• To improve the scientific basis for physical, urban and general planning for reduction of earthquake consequences and mitigation of seismic risk pertinent to seismically active regions;
• To improve the state-of-practice on land use, engineering design and construction; and,
• To initiate and activate new and revitalized programmes of research, mitigation, preparedness, response, and recovery, as well as to call for a change in public policy concerning earthquake hazards.

Post-earthquake damage evaluation and classification have to be organized by implementing a systematic methodology and rapid procedure in order to provide local and national decision-making authorities with essential information to allow them to undertake economically justified and technically consistent measures to reduce earthquake consequences in a uniform manner over the entire country.

Principal elements incorporated in the uniform methodology are condensed in the Earthquake Damage and Usability Inspection Form (see Form 1) and are based upon experiences gathered from earthquakes that took place during the last three decades in Yugoslavia and other countries located in seismically active regions in the world. The methodology and procedure for earthquake damage assessment, (originally proposed by IZIIS - Skopje and later accepted by other Balkan countries) will provide more reliable and transferable data for the practical elaboration of efficient pre-disaster risk mitigation and management or post-disaster reconstruction and revitalization programmes.

The Earthquake Damage and Usability Inspection Form as presented is prepared in a format suitable for easy and rapid field data collection, and the rapid transfer of data to computers, enabling a detailed analysis of relevant damage and usability classification parameters. It comprises the basic information pertinent to each individual building as follows:
• Identification Parameters (1-9);
• Structure and Quality of Construction Parameters (10-17);
• Damage and Usability Classification (18-24), based on detail description given in 18; and
• Emergency Measures and Human Losses (25, 27, 28) with appropriate recommendations to be given by the inspection teams.
Based on the collected earthquake damage data for each individual building, an overall presentation of the earthquake effects in the affected area could be performed. With a further detailed analysis, the direct economic and other types of losses could be determined.

Assessment of Economic Losses

From the completed earthquake damage evaluation and summary analysis of building damage, earthquake damage could be related directly with reference to structural types, usage categories of the buildings, and gross areas. For an estimation of economic losses, the first strategic decision should be made on the level to which earthquake damaged buildings should be repaired and strengthened. Two basic decisions are in general possible:

- buildings should be repaired and strengthened to be seismically structures with possible updated functioning, or
- buildings could be repaired to pre-earthquake conditions. (Many Balkan countries are implementing the first decision because large earthquakes occur often and the stock of non-seismic construction is large).

After the basic strategic approach is made, summary relationships on observed damage (empirical vulnerability functions with respect to ground shaking intensity) could be prepared for each structural type of building (Figures 1 to 3). Depending on the distribution of each structural type in the total gross area, a number of representative samples could be selected for detailed cost-estimate analysis of repair and strengthening of each category and for at least five levels of ground shaking. For each of the selected sample buildings, detailed analysis of design and detailing should be performed prior to the cost-estimate analysis. Based on the analysis of a sufficient number of selected samples, an estimation of the cost of repair and strengthening of the structural system, (nonstructural elements and installations, including improvement of the building function), could be developed similar to those shown in Figure 4. The cost of repair and strengthening could be presented as a percent of the total cost of new construction per unit area.

Once these preliminary tasks are accomplished, discovering the total direct economic losses will be rather simple. In addition to building losses, direct as well as indirect economic losses for local and regional infrastructures should be assessed by specialized inspection teams.

To illustrate the assessment of direct economic losses, two examples of the Skopje Montenegro earthquakes are given with a summary presentation of: earthquake damage classification of 16.478 residential buildings in three major categories and subcategories of damage (Table 1); and on 57.640 buildings summarized in 3 basic categories of damage in 6 coastal communes and 6 hinterland communes (Table 3) with discrete distribution given in Figure 6. Total direct economic loss assessment by major sections is given in Table 2 and Table 4 for Skopje and Montenegro respectively. Total direct economic loss is estimated to 15 and 10 percent of the gross national product of Yugoslavia for 1963 and 1979, which was a very serious blow to the economy of the country. A reconstruction programme is given in
Table 3, parallel with the direct economic loss assessed which is anticipated by the Federal Assembly Programme for reconstruction. Dynamic funds allocation and their realization for the ten year period of reconstruction is presented in Figure 5.

It is recognized that the efficient planning and implementation of reconstruction programmes could be organized only on the basis of a detailed earthquake damage classification of the affected entire stock of buildings and by implementing a consistent and uniform methodology and procedure. The data collected are of basic importance for: the development of the empirical vulnerability functions of traditional types of construction; the verification of the experimental and theoretical vulnerability functions of modern types of construction; as well as the verification and improvement of seismic design codes and regulations. Assessment of the earthquake losses in pre-earthquake and post-earthquake conditions, and planning of measures and activities for mitigation of seismic risk and reduction of earthquake consequences, will be much more realistic with the implementation of the methodology and procedure presented in this paper.
EARTHQUAKE DAMAGE AND USEABILITY INSPECTION FORM

1. Town (name - code):

2. Building Identification:
   2.1. Code of Town/Section of Settlement
   2.2. Working team code
   2.3. Number of the building

3. Principal Orientation of the Building:

4. Position of the Building in the Block:
   1. Corner, 2. Middle, 3. Free

5. Building Gross Area (m²):

6. Number of Stories:
   6.1. Basement: No/0/, Yes/1/
   6.2. Stories:
   6.3. Mezzanine: No/0/, Yes/1/
   6.4. Appendages: No/0/, Yes/1/

7. Usage (see description on back page):
   7.1. Building:
   7.2. Ground Floor:

8. Number of Apartments:

9. Construction period (To be defined by each country):
   1. 2. 3.

10. Type of Structure (see description on back page):
    11. Floors:
    14. Type of Load Carrying System (see description on back page):
        1. Bearing walls, 2. Frames, 3. Frames with infill walls
        4. Skeleton with infill walls, 5. Mixed, 6. Other (specify)
    15. Quality of Workmanship:
        1. Good, 2. Average, 3. Poor
    16. First Floor Stiffness Relative to Others:
        1. Larger, 2. About equal, 3. Smaller
    17. Repairs from Previous Earthquakes:
        1. No, 2. Yes, 3. Unknown

18. Damage of Structural Elements:
    (see description on back page)
    18.1. Bearing Walls:
    18.2. Columns:
    18.3. Beams:
    18.4. Frame Joints:
    18.5. Shear Walls:
    18.6. Stairs:
    18.7. Floors:
    18.8. Roof:

19. Damage of Nonstructural Elements and Installations:
    (see description in the manual)
    19.1. Interior Walls:
    19.2. Partitions:
    19.3. Exterior Walls (Facade):
    19.4. Electrical Installations:
    19.5. Plumbing:

20. Damage of Entire Building:

21. Damage due to Fire After the Earthquake:
    1. No/0/, Yes/1/

22. Elso-Soil Conditions:

23. Observed Soil Instabilities:
    1. None, 2. Slight settlements, 3. Intensive settlements,
    4. Liquefaction, 5. Landslide, 6. Rockfalls, 7. Faulting,
    8. Other (specify):

24. Usability Classification and Posting:
    Posted: 1. Green, 2. Yellow, 3. Red,
    Not posted: 4. To be posted after removal of local hazard,
    5. Soil and geological problems, reinspection,
    6. Unable to classify, reinspection, 7. Building inaccessible

25. Recommendations for Emergency Measures:
    1. None, 2. Remove local hazard, 3. Protect building from failure, 4. Protect streets or neighbouring buildings, 5. Urgent demolition

26. Photographs taken:
    No/0/, Yes/1/

27. Trapped in the Building:
    No/0/, Yes/1/
    (If yes stop inspection and inform authorities)

28. Human Losses:
    No deaths and injuries /0/, Possible deaths and injuries /1/
    (If information available, please indicate):
    Number of deaths:
    Number of injuries:

29. Date of Inspection: Month/Day

Names of Inspection Engineers: Signatures
1. 2. 3.
DESCRIPTION AND CODES OF USAGE CATEGORIES, TYPE OF STRUCTURE, LOAD CARRYING SYSTEM, STRUCTURAL DAMAGE CATEGORIES AND POSTING

BUILDING USAGE CATEGORIES:

10 Residential: 11 Family houses, 12 Apartment buildings
20 Office: 21 Entire building, 22 Partially
30 Economical: 31 Trade, 32 Finance, 33 Small industry, 34 Storage and ware houses, 35 Agricultural, 36 Fishing, 37 Forestry
40 Health and Social Welfare: 41 Hospitals and clinics, 42 Health services, 43 Social welfare (old people houses, invalids, day care centers)
50 Public Services: 51 Administrative—central or local government, 52 Police and Fire stations, 53 Transportation (buildings ground, rail, air, sea) 54 Communications (buildings, post, radio, TV)
60 Education and Culture: 61 Schools, 62 Universities, 63 Research centers, 64 Historical and religious, 65 Cultural and entertainment, 66 Sports (gymnasium, stadium)
70 Tourism and Catering: 71 Hotels, 72 Restaurants, 73 Coffee shops, pizzeria shops etc.
80 Industry and Energy: 81 Industrial, 82 Energy, 83 Public Services, 84 Administrative—central or local government, 85 Industry and Energy, 86 Other buildings (to be described)
90 Other Buildings (to be described)

TYPE OF STRUCTURE:

100 Masonry Buildings:
110 Adobe: 111 Adobe plain, 112 Adobe with timber belts
120 Solid brick: 121 With horizontal R.C. belts, 122 With horizontal and vertical R.C. belts
130 Hollow brick: 131 With horizontal R.C. belts, 132 With horizontal and vertical R.C. belts
140 Concrete blocks: 141 With horizontal R.C. belts, 142 With horizontal and vertical R.C. belts
150 Stone masonry: 151 Dry stone masonry, 152 Thin stone with low quality of mortar, 153 Plain stone with good quality of mortar, 154 Stone with timber belts, 155 Stone with horizontal and vertical R.C. belts
200 Reinforced concrete structures:
210 Cast in place frames: 211 With solid brick infill, 212 With hollow brick infill, 213 With light concrete blocks or panel infill, 214 With shear walls
220 Cast in place bearing walls: 221 With bearing walls in one direction, 222 With bearing walls in orthogonal directions
230 Prefabricated structures: 231 Frames with hollow brick infill, 232 Frames with light concrete or panel infill, 233 Frames combined with shear walls, 234 Large panel structures, 235 Small panel structures
240 Mixed structures: 241 R.C. frames with load bearing masonry walls, 242 Combination of steel frames with load bearing masonry walls
300 Steel structures:
310 Heavy industrial steel structures: 311 Without cranes, 312 With cranes
320 Light industrial steel structures: 321 Without cranes, 322 With cranes

330 Multi story steel structures: 331 Frames without bracing, 332 Frames with bracing, 333 Steel frames with R.C. cores, 334 Steel frames isolated with R.C.
400 Timber structures:
410 (Bamboo): 411 Bamboo with ground floors of stone masonry, 412 Bamboo only
420 Prefabricated: 421 Timber frames, 422 Timber system panel elements

14. TYPE OF LOAD CARRYING SYSTEM:
Vertical and lateral loads are carried by:
1. Walls, 2. Frames, 3. Frames with infill walls, 4. Skeleton, 5. Frame system, 6. Combination of walls, frames and/ or shear walls and infills, 7. Other systems (to be described)

15. DAMAGE OF STRUCTURAL ELEMENTS AND POSTING:

1. None — Posted Green: Without visible damage to the structural elements. Possible fine cracks in the wall and ceiling mortar, hardly visible nonstructural and structural damage
2. Slight — Posted Green: Cracks in the wall and ceiling mortar, falling of large patches of mortar from wall and ceiling surface, Considerable cracks, or partial failure of chimneys, arches and gable walls. Disturbance, partial sliding, sliding and falling down of roof covering, Cracks in structural members

Buildings classified in damage category 1 and 2 are without decreased seismic capacity and do not pose danger to human life, immediately usable or after removal of local hazard (cracked chimneys, arches or gable walls).

3. Moderate — Posted Yellow: Diagonal or other cracks in structural walls, walls between windows and similar elements of structural elements. Large cracks to reinforced concrete structural members, columns, beams, R.C. walls, Partially failed or failed chimneys, arches or gable walls. Disturbance, sliding and falling down of roof covering

4. Heavy — Posted Yellow: Large cracks or without detachment of walls with crushing of materials. Large cracks with crushed material of walls between windows and similar elements of structural walls. Large cracks with small detachment of R.C., structural elements: columns, beams and R.C. walls. Slight dislocation of structural elements and the whole building

Buildings classified in damage category 3 and 4 are with significant decreased seismic capacity. Limited entry is permitted, unusable before repair and strengthening. Need for supporting and protection of the building and its surroundings should be considered

5. Severe — Posted Red: Structural members and their connections are extremely damaged and dislocated. A large number of crushed structural elements. Considerable dislocations of the entire building and dislocating of roof structure. Partial or completely failed buildings

Buildings classified in damage category 4 are unsafe with possible sudden collapse. Entry is prohibited. Protection of streets and neighboring buildings or urgent demolition required. In case of isolated or stabilized buildings decision for demolition should be based on economical study for repair and strengthening.

- 91 -
Number of Buildings

Fig. 2. Site Dependent Vulnerability Functions for

D/nC-11 + III

D/nC-11

Percent Damage, DR (%)

PGA (in % of g)

R - Rock Site - Soil Condition
DA - Diurnal - Annual
STM - Strutted Masonry
BM - Brick Masonry
SM - Stone Masonry

Fig. 1. Generalized Physical (Function) Vulnerability Functions and Data Scatter plots for Stone Masonry (SM) Buildings

Percent Damage, DR (%)

PGA (in % of g)

0 10 20 30 40 50

Percent Damage, DR (%)

0 10 20 30 40 50

Number of Buildings

No. of Analyzed Digs

No. of Analyzed Digs

No. of Analyzed Digs

No. of Analyzed Digs
Fig. 3. Vulnerability Functions of Buildings by Structural Types

Fig. 4. Functions of Cost for Repair and Strengthening of Earthquake Damage Buildings
Fig. 5. Dynamic of Founds Allocation and their Realization in the Reconstruction of the City of Skopje in the Period 1963 – 1973

Fig. 6. Discrete Distribution of Damage in the Communes of SR Montenegro due to the Earthquake of April 15, 1979
Table 1: Summary of Building Damage Data Classification for Skopje

<table>
<thead>
<tr>
<th>Damage and Usability Classification</th>
<th>Commune</th>
<th></th>
<th></th>
<th></th>
<th>Total</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Idadija</td>
<td>Kale</td>
<td>Kisela Voda</td>
<td>Saat Kula</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nonispected</td>
<td>NB</td>
<td>ND</td>
<td>NB</td>
<td>ND</td>
<td>NB</td>
<td>ND</td>
<td>NB</td>
</tr>
<tr>
<td>Green (D&amp;U - C - I)</td>
<td>403</td>
<td>692</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>608</td>
</tr>
<tr>
<td>G - I</td>
<td>686</td>
<td>2,000</td>
<td>101</td>
<td>275</td>
<td>1,740</td>
<td>4,122</td>
<td>176</td>
</tr>
<tr>
<td>G - II</td>
<td>70</td>
<td>137</td>
<td>32</td>
<td>73</td>
<td>1</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>G - III</td>
<td>164</td>
<td>756</td>
<td>37</td>
<td>159</td>
<td>15</td>
<td>211</td>
<td>7</td>
</tr>
<tr>
<td>Yellow (D&amp;U - C - II)</td>
<td>452</td>
<td>1,107</td>
<td>32</td>
<td>43</td>
<td>1,724</td>
<td>3,910</td>
<td>169</td>
</tr>
<tr>
<td>Y - I</td>
<td>1,450</td>
<td>5,882</td>
<td>769</td>
<td>1,413</td>
<td>949</td>
<td>2,957</td>
<td>473</td>
</tr>
<tr>
<td>Y - II</td>
<td>84</td>
<td>552</td>
<td>85</td>
<td>98</td>
<td>36</td>
<td>177</td>
<td>13</td>
</tr>
<tr>
<td>Y - III</td>
<td>1,331</td>
<td>5,206</td>
<td>602</td>
<td>1,176</td>
<td>890</td>
<td>2,624</td>
<td>460</td>
</tr>
<tr>
<td>Red (D&amp;U - C - III)</td>
<td>35</td>
<td>124</td>
<td>81</td>
<td>139</td>
<td>14</td>
<td>156</td>
<td>-</td>
</tr>
<tr>
<td>R - I</td>
<td>1,554</td>
<td>4,110</td>
<td>3,867</td>
<td>5,547</td>
<td>1,174</td>
<td>1,829</td>
<td>2,537</td>
</tr>
<tr>
<td>R - II</td>
<td>1,212</td>
<td>2,954</td>
<td>2,892</td>
<td>4,305</td>
<td>956</td>
<td>1,458</td>
<td>2,210</td>
</tr>
<tr>
<td></td>
<td>342</td>
<td>1,156</td>
<td>975</td>
<td>1,242</td>
<td>218</td>
<td>371</td>
<td>327</td>
</tr>
<tr>
<td>TOTAL:</td>
<td>4,093</td>
<td>12,684</td>
<td>4,737</td>
<td>7,235</td>
<td>3,854</td>
<td>8,908</td>
<td>3,794</td>
</tr>
</tbody>
</table>

NB — Number of Damaged/Inspected Buildings
ND — Number of Damaged/Inspected Dwellings
Y - II Damage & Usability Classification /Yellow (Y) two (II) Strokes/
### Table 2: Funds Used for Reconstruction of Skopje in the Period 1963-1973

<table>
<thead>
<tr>
<th>Activities</th>
<th>Anticipated with Programme for 1963/73</th>
<th>Realized in the Period 1963/73</th>
<th>Realization in %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>000 DIN</td>
<td>000 DIN</td>
<td></td>
</tr>
<tr>
<td>TOTAL:</td>
<td>6,190,000</td>
<td>6,105,000</td>
<td>98.6</td>
</tr>
<tr>
<td>1. Facilities of the Yugoslav Army</td>
<td>330,000</td>
<td>5.5</td>
<td>330,000</td>
</tr>
<tr>
<td>2. Economy</td>
<td>1,837,000</td>
<td>28.6</td>
<td>1,755,134</td>
</tr>
<tr>
<td>3. Residence</td>
<td>1,009,000</td>
<td>16.3</td>
<td>1,003,879</td>
</tr>
<tr>
<td>4. Main Public Services</td>
<td>733,000</td>
<td>11.8</td>
<td>733,000</td>
</tr>
<tr>
<td>5. Public Buildings of SRM</td>
<td>415,000</td>
<td>6.7</td>
<td>415,000</td>
</tr>
<tr>
<td>6. Public Buildings of the City</td>
<td>1,309,600</td>
<td>-</td>
<td>1,308,498</td>
</tr>
<tr>
<td>6.1 Public Utilities</td>
<td>621,785</td>
<td>-</td>
<td>621,785</td>
</tr>
<tr>
<td>6.2 Education</td>
<td>435,746</td>
<td>-</td>
<td>429,912</td>
</tr>
<tr>
<td>6.3 Culture</td>
<td>118,814</td>
<td>-</td>
<td>120,510</td>
</tr>
<tr>
<td>6.4 Physical and Technical Culture</td>
<td>61,828</td>
<td>-</td>
<td>53,826</td>
</tr>
<tr>
<td>6.5 Health care and Social Welfare</td>
<td>69,145</td>
<td>-</td>
<td>69,145</td>
</tr>
<tr>
<td>6.6 State Bodies and Organizations</td>
<td>61,200</td>
<td>-</td>
<td>62,040</td>
</tr>
<tr>
<td>7. Preparation of Construction Sites</td>
<td>290,000</td>
<td>4.7</td>
<td>290,000</td>
</tr>
<tr>
<td>8. Compensation to Banks and Funds for Loans Canceled due to the Earthquake</td>
<td>145,000</td>
<td>2.3</td>
<td>145,000</td>
</tr>
<tr>
<td>9. Fund Expenses</td>
<td>34,000</td>
<td>0.5</td>
<td>32,489</td>
</tr>
<tr>
<td>10. Fund Reserves</td>
<td>23,000</td>
<td>0.4</td>
<td>23,000</td>
</tr>
<tr>
<td>10.1 Economy</td>
<td>7,000</td>
<td>-</td>
<td>7,000</td>
</tr>
<tr>
<td>10.2 Public Utilities</td>
<td>11,000</td>
<td>-</td>
<td>11,000</td>
</tr>
<tr>
<td>10.3 Education</td>
<td>5,000</td>
<td>-</td>
<td>5,000</td>
</tr>
</tbody>
</table>

1 US $ = 12.00 YU DIN (1963)

### Table 3: Classification of Damaged Buildings in Six Coastal and Hinterland Communes

<table>
<thead>
<tr>
<th>Commune</th>
<th>Non Damaged No. Bldg's</th>
<th>Non Damaged %</th>
<th>To B. Repaired No. Bldg's</th>
<th>To Be Repaired %</th>
<th>To Be Demolished No. Bldg's</th>
<th>To Be Demolished %</th>
<th>Total No. Bldg's</th>
<th>Total %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coastal</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ULCINJ</td>
<td>2,424</td>
<td>36</td>
<td>1,119</td>
<td>17</td>
<td>3,183</td>
<td>47</td>
<td>6,726</td>
<td>100</td>
</tr>
<tr>
<td>BAR</td>
<td>4,595</td>
<td>47</td>
<td>1,887</td>
<td>17</td>
<td>3,870</td>
<td>36</td>
<td>10,357</td>
<td>100</td>
</tr>
<tr>
<td>BUDVA</td>
<td>1,496</td>
<td>59</td>
<td>496</td>
<td>19</td>
<td>555</td>
<td>22</td>
<td>2,009</td>
<td>100</td>
</tr>
<tr>
<td>TIVAT</td>
<td>2,041</td>
<td>69</td>
<td>529</td>
<td>18</td>
<td>406</td>
<td>13</td>
<td>2,962</td>
<td>100</td>
</tr>
<tr>
<td>KOTOR</td>
<td>3,621</td>
<td>56</td>
<td>1,221</td>
<td>21</td>
<td>918</td>
<td>16</td>
<td>5,737</td>
<td>100</td>
</tr>
<tr>
<td>H. NOVI</td>
<td>3,601</td>
<td>63</td>
<td>1,221</td>
<td>21</td>
<td>918</td>
<td>16</td>
<td>5,737</td>
<td>100</td>
</tr>
<tr>
<td>Hinterland</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CETINJE</td>
<td>2,955</td>
<td>43</td>
<td>2,451</td>
<td>35</td>
<td>1,556</td>
<td>22</td>
<td>6,937</td>
<td>100</td>
</tr>
<tr>
<td>NIKŠIĆ</td>
<td>2,401</td>
<td>74</td>
<td>620</td>
<td>18</td>
<td>276</td>
<td>8</td>
<td>3,315</td>
<td>100</td>
</tr>
<tr>
<td>TITOGRAD</td>
<td>4,971</td>
<td>83</td>
<td>891</td>
<td>15</td>
<td>116</td>
<td>2</td>
<td>5,826</td>
<td>100</td>
</tr>
<tr>
<td>DANILOVGRAD</td>
<td>4,897</td>
<td>80</td>
<td>869</td>
<td>17</td>
<td>181</td>
<td>3</td>
<td>5,237</td>
<td>100</td>
</tr>
<tr>
<td>IVANGRAD</td>
<td>649</td>
<td>88</td>
<td>84</td>
<td>12</td>
<td>2</td>
<td>0</td>
<td>735</td>
<td>100</td>
</tr>
<tr>
<td>KOLASIN</td>
<td>498</td>
<td>69</td>
<td>184</td>
<td>26</td>
<td>35</td>
<td>5</td>
<td>717</td>
<td>100</td>
</tr>
<tr>
<td>TOTAL:</td>
<td>33,556</td>
<td>58</td>
<td>11,773</td>
<td>20</td>
<td>12,540</td>
<td>22</td>
<td>57,640</td>
<td>100</td>
</tr>
</tbody>
</table>
Table 4: total Losses Caused by April 15, 1979, Montenegro Earthquake

<table>
<thead>
<tr>
<th>Activities</th>
<th>Direct Losses 000</th>
<th>Indirect Losses 000</th>
<th>Total 000</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%</td>
<td>%</td>
<td>%</td>
</tr>
<tr>
<td>A Economy</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Industry</td>
<td>10,277,168</td>
<td>4,787,837</td>
<td>21,060,000</td>
</tr>
<tr>
<td>2. Transportation</td>
<td>3,352,654</td>
<td>1,049,926</td>
<td>4,402,580</td>
</tr>
<tr>
<td>3. Trade</td>
<td>4,359,412</td>
<td>924,794</td>
<td>5,284,206</td>
</tr>
<tr>
<td>4. Tourist Industry</td>
<td>1,389,547</td>
<td>514,876</td>
<td>1,904,423</td>
</tr>
<tr>
<td>5. Other Activities</td>
<td>4,714,369</td>
<td>1,918,110</td>
<td>6,632,479</td>
</tr>
<tr>
<td>B Public and Social Services</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Education,Science,Culture</td>
<td>2,491,176</td>
<td>380,131</td>
<td>2,871,307</td>
</tr>
<tr>
<td>2. Health and Social Welfare</td>
<td>10,948,886</td>
<td>132,653</td>
<td>11,081,539</td>
</tr>
<tr>
<td>3. Socio-Political Organizations</td>
<td>1,718,381</td>
<td>1,718,381</td>
<td></td>
</tr>
<tr>
<td>- Residential Stock, only</td>
<td>744,303</td>
<td>876,956</td>
<td></td>
</tr>
<tr>
<td>C Private Sector</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Residential Stock, only</td>
<td>21,958,946</td>
<td>21,958,946</td>
<td>31.07</td>
</tr>
<tr>
<td>D Expenses of Socio-Political Organizations</td>
<td>14,016,107</td>
<td>14,016,107</td>
<td></td>
</tr>
<tr>
<td>E Cultural and Historic Heritage</td>
<td>3,911,615</td>
<td>3,911,615</td>
<td>5.53</td>
</tr>
<tr>
<td>F Property of JNA</td>
<td>10,499,223</td>
<td>10,499,223</td>
<td>14.85</td>
</tr>
<tr>
<td>GRAND TOTAL</td>
<td>65,754,616</td>
<td>4,920,490</td>
<td>70,675,106</td>
</tr>
</tbody>
</table>

1 US $ = 17.83 YU DIN (1979)

Table 5: Spatial Distribution of Losses

<table>
<thead>
<tr>
<th>Zone</th>
<th>Damage Value and Percent x10^6 YU DIN</th>
<th>x10^6 US $</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>I Zone Montenegro Coast (6 Communes)</td>
<td>54,663</td>
<td>3,065.8</td>
<td>77.35</td>
</tr>
<tr>
<td>II Zone Central Part (4 Communes)</td>
<td>11,593</td>
<td>670.4</td>
<td>16.51</td>
</tr>
<tr>
<td>III Zone Northern Part (10 Communes)</td>
<td>1,744</td>
<td>97.8</td>
<td>2.47</td>
</tr>
<tr>
<td>IV Losses Of JNA</td>
<td>2,158</td>
<td>121.1</td>
<td>3.05</td>
</tr>
<tr>
<td>V Expenses of Socio-Political Organizations</td>
<td>155</td>
<td>8.7</td>
<td>0.22</td>
</tr>
<tr>
<td>TOTAL</td>
<td>70,675</td>
<td>3,963.8</td>
<td>100.00</td>
</tr>
</tbody>
</table>

1 US $ = 17.83 YU DIN (1979)
References

PRIORITIES IN EARTHQUAKE DAMAGE REDUCTION

Herbert Tiedemann
Engineering Consultant to the Swiss Reinsurance Group

INTRODUCTION

The potential gain from a reasonably precise risk optimization depends decisively on the elements constituting a building, their relative importance and their vulnerability which in turn often depends on many parameters. It also depends on "external" factors like foundation material and the subsoil in the general region and, of course, on the characteristics of the earthquake and of shaking at the site.

The most fundamental and universal rule governing gains to be expected from optimizing the architectural and structural design and the construction of buildings is

\[ G = f(V) \]

The gain \( G \) is a function of the vulnerability \( V \) of the particular element at risk. It does not only depend on damage to the building but on the indirect consequences of bad building performance, like harm to people, damage and loss of content, or business interruption. The correlation is, however, not a linear but an exponential one. If we consider the quality \( Q \) of an element at risk as the reciprocal of vulnerability, i.e.

\[ Q = \frac{1}{V} \]

we may use the quality of a structure instead of the vulnerability, or simplifying the base shear which is mostly used in design and earthquake building codes. Base shear is, however, only one of the many yardsticks of damage to be expected. Vulnerability and the probability distribution of damage depends on the combined action of many parameters. A pragmatic approach to earthquake damage optimization must therefore be based on as many actual data obtained from field inspections of earthquake loss and damage as possible and, if necessary, on models which must, however, be in agreement with such data.

One must, however, not only understand the parameters which contribute to the magnitude of damage but also the probability of its occurrence. Such knowledge correlating damage and its probability is indispensable because the impact, one may also say the importance \( I \) of an event is the product of its probability \( p \) and the loss expected \( LE \), i.e.

\[ I = p \times LE \]

In order to quantify the probability of earthquake magnitudes in a certain area, or of earthquake intensities or accelerations at a site, we have developed seismic index maps (SIM) more than a decade ago and refined them repeatedly.

DAMAGE PARAMETERS

A detailed account of the parameters contributing to earthquake damage of buildings has been provided elsewhere (1. & 3. - 16.) In short the most important factors are:
- Resonance between predominant frequencies of the foundation material and of the building or structure.
- Quality, i.e., predominantly hardness of the foundation material.
- Shear strength of the building resulting from the combined strength of structural and non-structural parts.
- Compatibility of behaviour of building materials and components under dynamic loads.
- Ease of repair.
- Regularity and symmetry as regards floor plans, elevations, shear strength, distribution of masses and damping.
- Design, quality, arrangement, and fastening of non-structural elements.
- Hammering between buildings
- Orientational sensitivity.
- Liquefaction

The general correlation, albeit for an already "decontaminated" sample, between MMI, building quality and mean damage ratio (MDR) is shown in Fig 1. It represents the situation for a moderately irregular and asymmetrical sample of buildings founded on alluvium of average hardness. The graphs show that if one decides to change code-requirements or design criteria from, for instance, 2.5% g shear strength to 6% g one would reduce MDR's by a factor of 3, 2.5 and 2 for intensities corresponding to MM VII, VIII, and IX.

Although not much extra money must be invested to render the building stronger by simply increasing the shear strength, about 3% of the value of the complete building in the case cited above, an even more economic approach can be employed because damage depends on other factors as well.

1. RESONANCE

Resonance which is in discussions often hidden in terms like 'site effects', confounding subsoil quality and resonance, is known to be responsible for spectacular damage to modern buildings, mostly of many storeys, although they were designed according to what are considered by many "good" earthquake building codes. Its importance has again been illustrated dramatically by the Mexican earthquake of 1985. Frail adobe buildings of few storeys suffered practically no damage whereas the MDR of modern engineered buildings in resonance with the nearly harmonic shaking of the ground was very high (13, 14, 16).

This lesson is certainly not new. After the Mexican earthquake of 1957 selective damage to high-rise buildings was seen (20). This happened again during the earthquakes of 1964 and 1979 (1, 12). This parameter was important during the earthquakes in Chile (1960) (21), Bucharest (1977) (22), Miyagi-Ken-Oki in 1978 (23), Campania-Basilicata, Italy in 1980, in the downtown area of El Asnam (1980) (1), Guatemala City (1976) (1, 10) and Caracas, Venezuela (1967). More cases could be added and there is little doubt that buildings "in resonance" with prominent frequencies of the subsoil suffer much higher MDR's than others which are clearly outside of such "frequency bands".

The difference in MDR depends on many other parameters and, in addition on the damage level, i.e. on saturation of damage. The MDR of frail adobe buildings of 1-2 storeys in the region of the extinct lake of Texcoco (Mexico DF) was below 1%. Bungalows on soft volcanic subsoil in San Salvador suffered very little damage. Modern engineered buildings, however, suffered MDR's of several dozen percent in both cases (13-16).

A further important lesson is that the MDR of buildings in Mexico City (1985) was as high as the one suffered by comparable buildings in San Salvador (1986) although the latter ones were exposed to roughly three times the acceleration. Many more examples from other earthquakes may be added to.
prove that is not acceleration which damages the buildings, at least in most cases, but the deflection or distortion of the members.

There are a number of theoretical models addressing subsoil characteristics but it is felt that a simpler method should be favoured, at least for the foreseeable future and in most seismic regions. The approach we suggest here is based on the depth of soft layers, and it is supported by theoretical reasoning. Fig. 2 shows two alternatives of soft deposits. We have found a good correlation between period of prominent shaking and depth to hard strata (11). A reasonably good approximation between depth \(d\) to hard strata in metres and period \(T\) in seconds is given by

\[
T = a \sqrt{d}
\]

in which \(a = 0.1\) (cf also Fig. 3)

Additional stiffness can be introduced not only by strengthening the structure. One may gain very much from a better mortar. If good-grade bricks are used the weak spot is usually the mortar and the bond between mortar and bricks if workmanship is poor. Bad workmanship should be considered a crime in seismic regions. Changing from ordinary lime mortar to good cement mortar makes a wall about two times stronger.

The use of strong bricks is important. Walls incorporating a thin-walled variety of bricks are shattered whereas others of strong bricks have a few cracks only or no damage at all.

What was mentioned above in connection with mortar holds for plaster as well. After the Campania-Basilicata, Italy, earthquake of 1980, for instance, we saw many buildings which had sustained less damage than others just because of good plaster.

Opening in walls do not only affect stiffness but regularity of the building, an aspect which is discussed later. Unless corners at openings are properly reinforced (1, 15) cracks will start from there because of a notch-effect. Some additional internal walls at critical places can stiffen the building considerably, in particular if external walls are few, asymmetrically arranged, or absent because large windows have been installed.

The chance of resonance can also be diminished by reducing weights and loads in upper storeys. One must understand that a soft ground floor does not only produce a building which is asymmetrical but in general also a top-heavy one. The catastrophic failure of many multi-storey factory buildings in Mexico City was only to some extent due to the overloading of upper floors by storing textiles, paper, etc. but because these considerable masses lengthened the period of these buildings so much that they were in resonance with the 2-3 sec. period of the ground shaking.

Most modern building activities take place on soft alluvium as sites of hard level ground have become scarce. This signifies a high chance of resonance (11) and of substantial damage and loss of life unless precautions are taken.

2. QUALITY OF THE FOUNDATION MATERIAL

The quality of the foundation material is basically its hardness and its content of water, the softer the material and the higher the groundwater level the greater the probability of damage and its extent.

Early observers, e.g. after earthquakes in Calabria, Italy of February 1783, noted that damage to buildings on soft ground was more severe than to others on firm ground. The San Francisco earthquake of 1906 confirmed this (24, 25). After the 1960 earthquake in Chile ground hardness to damage dependence was seen in Castro, Ancud, Concepción (26), Puerto Mont (27), and Valdivia. Soft
subsoil contributed to damage in Miyagi-Ken-Oki, Japan, (1978) (23, 29-30). Other examples are given in (12). A general correlation is shown in Fig. 3.

The problems due to soft foundation material can be generalized as follows. There is a chance of differential settlement of the building because safety margins applied in the design of the foundations were inadequate or because such material is compacted by the shaking. Moreover, deposits of alluvium are often not homogeneous. One must be particularly careful in regions where rivers may have been meandering, even if this happened aeons ago.

Soft subsoil will amplify the ground shaking. The amplification depends, however, on many parameters (1). In general one should be prepared for 3 - 5 times higher accelerations, velocities, and displacements of soft alluvium as compared to rock. Part of this is due to resonance between low frequency earthquake shaking and a low natural frequency of soft layers. The larger amplitudes on soft deposits induce larger deflections in buildings and therefore increase damage. The approximate impact of soft subsoil can be quantified in terms of MDR with the help of Figs. 1 & 4.

These characteristics do, however, also increase the probability of damaging ground shaking. As dangerous shaking of soft foundation material can happen at much larger distance from the earthquake source than on hard roc: its probability grows (8).

3. SHEAR STRENGTH OF BUILDINGS

Even fairly simple designs are often flawed. In normal, simple residential buildings of several floors damage is generally concentrated in the ground floor, sometimes extending to the second storey although each storey had been designed to resist earthquake forces. If such simple structures do not perform as one should expect one can easily visualize the problems in asymmetrical buildings.

The general stumbling block is not a complex shape of buildings, columns, spans, etc. different height or size, uneven distribution of masses, or earthquake waves which induce torsional loads or a tendency of the building to respond with torsional movements, but the lack of knowledge about the contribution of "fill-in walls" to the shear strength of a building.

How much such "non-structural fill-in walls" do in fact contribute to the strength of buildings will be discussed in more detail in chapter 9. That much may be stated now, the shear strength of a building can be improved dramatically by using strong fill-in walls(cf. e.g. 3, 9 & 12 and Fig. 7). Strong walls can contribute about as much strength as the structure proper. The factor of improvement can, however, amount to two orders of magnitude in the low MDR-range.

4. COMPATIBILITY OF BUILDING MATERIALS

The selection of building materials (cf. 8) should not only allow for their intrinsic strength but for the compatibility between interacting items. If one combines a structure tolerating comparatively large deflections without damage and reacting plastically if deformed even more with items which are shattered at much smaller amplitudes of deformation one has brought materials together which are not compatible. Cases in point are residential or commercial buildings with a skeleton of steel and walls of brick. Industrial buildings and structures or warehouses of steel which are covered by asbestos or other brittle sheets belong to this category.

5. EASE OF REPAIRS

The cost of repairs has developed into a very worrying issue as buildings became more and more sophisticated and wages rose persistently. Particulary those commercial and administrative buildings which are equipped with air conditioning and/or heating ducting, concealed wiring in difficult to
reach places, sophisticated panneling and suspended ceilings, and technical equipment, are far more costly to repair than their comparatively simple counterparts which were in use several decades ago.

It is sofar not customary in most countries to use wall elements which can be changed at ease if cracked by an earthquake. As walls carry numerous fixtures and as many things are built into them we cannot afford to look at walls in isolation but must consider the elements associated with them.

If pipes in walls are damaged, e.g., at a transition from a column, beam, floor, or diaphragm to the wall, or by the failure of the wall, repair results in much indirect damage. Wall to wall carpeting, floors, panelling, wall paper, etc. will be spoiled. It is not difficult to arrange such piping in easily accessible ducts. This holds for electrical wiring as well. If no ducts are used one should at least not install wiring or pipes where failures are most likely, e.g., where walls meet ceilings or beams.

6. REGULARITY AND SYMMETRY OF BUILDINGS

Regularity and symmetry contribute much to the performance of buildings during earthquakes. Most buildings are to some extent irregular or asymmetrical. This can be due to non-uniform foundation material or foundations, irregular floor plans or elevations, non-uniform column height, spans, stiffness, masses, damping of structural elements. Irregularity is often aggravated by the kind, size, shape, and distribution of non-structural elements.

Fig. 5 shows the effect of regularity on MDR's of buildings during the M 5.4 San Salvador earthquake of 1986 which is in line with our findings from other earthquakes. The MDR of buildings could be lowered by a factor of at least 4 by either avoiding irregular floor plans or by adequately separating individual wings of the building. A brief example at this place. According to Fig. 7 the general MDR of 3-4% g buildings is about 20% at MM VIII. Very irregular buildings of this resistance will, however, produce a MDR of about 80%. To bring their MDR down to about 20% and assuming that the 1:4 damage ratio holds at higher strength as well we have to follow the MM VIII MDR-graph upwards until it intersects with the 5% MDR level. This point corresponds to a base sheaf of about 13%.

7. NON-STRUCTURAL ELEMENTS

Non-structural elements contribute at least 80% to building damage (10). In spite of this practically all earthquake building codes fail to provide adequate guidance.

The single most important item, fill-in walls, has been mentioned earlier. In addition to selecting proper material and seeing to it that workmanship is impeccable, one may think of separating walls from columns, inserting some elastic material. This carries, however, a disadvantage as such walls do not contribute much to the stiffness of buildings which is particularly important for those founded on soft subsoil, i.e., the majority build today. The performance of walls also controls damage to items placed inside walls or fastened to them, like electrical wiring, piping, sanitary items, doors and windows.

The most important non-structural elements after walls are suspended ceilings, facading and windows. Whereas high MDR's must be expected for suspended ceilings fastened according to present methods a very substantial gain can be achieved by applying proper care. Damage to facading is controlled by the performance of walls and by the fastening of the elements. We shall not discuss windows in detail because, contrary to general belief, damage to windows is mostly not significant.
8. HAMMERING BETWEEN BUILDINGS

Hammering is considered in codes by stipulating a separation between buildings. Such distances are, however, in general inadequate if buildings are founded on deep alluvial material and if large earthquakes may cause shaking of long duration. In particular tall and soft buildings will experience large-amplitude oscillations. For such reasons cases of hammering were abnormally high during the Mexican earthquake of 1985. The first and foremost tenet should be not to erect tall soft buildings on deep alluvial, i.e. soft layers. This was already propounded by Charles F. Richter (32).

9. ORIENTATIONAL SENSITIVITY

If one analyses damage to oblong buildings which are otherwise symmetrical one persistently finds that MDR's depend on the orientation of the long axis of such buildings (1,3, 9, 12, & 15). Those having their short (weaker) axis about in the direction of predominant earthquake shaking suffer much more damage than identical buildings standing about perpendicular to them and therefore shaken mostly in the direction of their long (stronger) axis. The MDR's of oblong buildings could be reduced very much if structural engineers would compensate for the smaller stiffening from fill-in walls parallel to the short axis of the building by making the structure stronger in this direction. We shall hereunder give some figures to show how important the matter is.

Oblong buildings with their short axis in the direction of prominent strong shaking will in the medium MDR-range and at MM VIII experience damage which is 3-4 times higher than those predominantly shaken along their strong (long) axis. As shown in Fig. 6 the general MDR-level of 8 & 9 storey buildings in a very large housing colony in Mexico City was substantially below the one of 14 & 15 storey buildings. For the taller buildings which experienced a high MDR, the ratio of damage depending on the orientation of the long axis was 3.6 :1, but for the 8 & 9 storey buildings 100 : 1. Collapsed buildings were only found in those sub-samples of identical buildings in housing colonies which had their weak axis about parallel to the direction of prominent shaking.

These cases point at very serious flaws still existing in structural design: to neglect the strengthening effect of fill-in walls. Figs. 5-7 provide some guidance. We shall use a simplified example to illustrate what can be done (cf also Fig. 7 & (1)).

We assume that a not extremely oblong but otherwise regular building of a base shear of 4% g is to be strengthened in the short axis to reduce the damage to the general MDR of 4% g-buildings if exposed to MM VIII. We assume that the damage ratio (DR) depending on orientational sensitivity is 1:4. Intersecting 4%g with MM VIII in Fig. 7 and applying this DR we estimate a general MDR of about 18% of the new replacement value of such buildings. Buildings with their long axis in the direction of prominent shaking will suffer a MDR of about 9% those perpendicular to them one of 36%. If we wish that the MDR of the latter buildings is about 18% we have to see at which base shear the general MDR is 9%. This is so for 8%g-buildings. Therefore the structure should be designed for 8%g in the direction of the short axis.

10. LIQUEFACTION

Liquefaction has caused very serious damage. The best published case is the Niigata, Japan, earthquake of 1964. The precondition for liquefaction to happen is a granular soil and groundwater.

Liquefaction damage differs from normal earthquake damage to buildings. Like in the case of slides the earthquake leads to liquefaction which in turn affects the buildings. Moreover, damage will often follow the "all-or-nothing-law", which means that up to a certain amount of settlement the building can still be used and above it it must be considered a constructive total loss.
As improving subsoil is in general a costly enterprise which does not guarantee perfect safety, probably the best advise is to abandon dangerous sites if economic improvements are not possible.

SPECIAL MAPS

Most earthquake risk maps found today are not of great help, neither in estimating the probability of earthquake magnitudes, intensities, or accelerations nor when taking economic decisions. As discussed at the beginning, the impact of an earthquake can only be assessed if the probability and the loss expected can be quantified. We have developed such seismic index maps (SIM) (8, 17, 19).
Fig. 1. The graphs show the mean damage ratio (MDR) in percent of the new replacement value depending on the earthquake intensity (MMI) and the building quality. The intensity is given according to the Modified Mercalli scale (MM 31). The quality of buildings having a structure of reinforced concrete is stated as base shear. Such buildings are: \( C = 2 - 3\% \, g \), \( D = 3 - 4\% \, g \), \( E = 6\% \, g \), \( F = 12\% \, g \), and \( G = 20\% \, g \). Graph A represents buildings of adobe and frail rubble masonry, graph B is for unreinforced brick buildings. The graphs are valid for medium-hard alluvium, i.e. a bonus must be allowed for harder subsoil and a malus must be applied for softer one (cf. Fig. 4). Liquefaction requires special consideration as discussed in the main text. Moreover, the graphs are valid for buildings which are only moderately asymmetrical. If different buildings are to be evaluated, Fig. 5 and reference (12) give approximate information, details are to be found in (1). The graphs or those shown in Fig. 7 can also be used, for instance, for improving designs which are exposed to orientational sensitivity and for assessing damage/vulnerability relations of other structures and items. If, for instance, the MDR of an element at risk can be estimated with reasonable precision, e.g. 50\%, 10\%, or 1\% at MM VIII, one may use one of the curves shown in this graph or a new one fitting their general characteristics. This approach is possible because we have learned from the analysis of very large numbers of different elements at risk that the damage or loss distribution is very strongly determined by the MDR, i.e. irrespective of the kind of the element at risk and the hazard causing loss or damage, one will find that the distributions are comparable if MDR's are similar.
Fig. 2. Schematic and much simplified illustration of two different types of deposits of soft subsoil. HL stands for hard layers, e.g. bedrock, SD for soft deposits. The case shown in A is typical for extinct lakes which have been filled with sediments. It can also represent a cross-section of a river valley with alluvial layers overlying bedrock. The longitudinal section of such a valley would, however, be comparable to the case illustrated by B in which soft deposits are arranged more or less horizontally and are of about uniform thickness, or of a gradually changing one. If a "bowl" of soft material is agitated by an earthquake it will start oscillating. There will in general be further waves superimposed on the "natural frequency" of the soft deposits shown in this sketch. Large distant earthquakes which produce shaking of long duration and of a low frequency are prone to produce particularly strong oscillations in such deposits. The natural frequency depends primarily on the configuration of such fills in depressions in the hard strata and in particular on their depth. This depth to hard strata is the parameter which is predominantly responsible for the period of shaking of soft deposits depicted by B. Each column can be considered a beam which is more or less anchored at its lower end and deflected sideways. As elastic and plastic restoring forces, masses, etc., discussed in the main text are approximately identical for comparable deposits it is evident that the natural frequency is very much depending on the length of the deflected column. The excitation of a building depends very strongly on the ratio of the frequency of the force to the one of the structure which is exited. Therefore the knowledge of the predominant frequency of the force, which is the one of the subsoil is of paramount importance.
Fig. 3. The illustration shows some of our observations of predominant periods of ground shaking and the respective depth of soft layers above hard strata. As explained in the main text and in Fig. 2 one may expect that the most important parameter determining the frequency of the force which shakes the buildings is the depth of soft layers. Also the predominant period of the soil under Mexico City which has the configuration shown under A in Fig. 2 and the one in smaller similar depressions follows this general rule. This is not surprising because not only the frequency of a simple pendulum but of a bar clamped on one side (cf. Fig. 2) depends on its length. Each column of material underneath of a building can be considered such a bar. In order to avoid designing buildings which are in resonance with the subsoil or to change them properly during upgrading or repairs one should therefore assess the depth of soft layers. This is neither difficult nor costly with presently available technology. As more refined models become available one can correct for the influence of the physical properties of the soft layers. In this respect the parameter which should be considered first is hardness.
Fig. 4. Approximate correlation between the hardness of subsoil (VS - H) or shear wave velocity and increase in MM intensity or amplification factor of ground acceleration. General hardness indications range from hard (H), i.e. good rock, over medium hard (MH), represented about by soft rock and well-compacted, dry diluvial deposits to soft (S) layers (alluvial deposits with not much groundwater), and very soft (VS). The latter corresponds to ordinary fills (man-made land), soft alluvial deposits with the groundwater near to the surface. The values, in particular amplification which depends on many other factors, are to be taken as indications only. The envelopes indicate the approximate range.
Fig. 5. After the San Salvador earthquake of 1986 we analysed the performance of all larger modern buildings in the capital of El Salvador, which experienced about MM VIII or slightly more (15). This illustration is a simplified version of the data reproduced in the mentioned reference and it serves mainly the purpose to show the influence of regularity, irregularity and stiffness on the mean damage ratio (MDR). The strength of the buildings is represented by their base shear (BS) in percent of gravity. Graph A shows the BS:MDR-correlation for the complete sample, i.e. the general average. The graph I indicates how much higher the MDR was for irregular buildings. It must, however, be noted that these buildings in San Salvador were not particularly asymmetrical and irregular, i.e. one should be prepared for even higher MDR's if extravagant designs are hit by earthquakes. On the other hand, regular buildings (R) had much lower mean damage ratios and the MDR was particularly low if buildings were not only regular but stiff as well (R & S), for instance because of resistant fill-in walls. It should be noted that the valley is filled with deposits of volcanic eruptions and that the ground water level was low at the time of the earthquake. Soft and irregular buildings of about 3 - 4% g suffered about seven times the MDR of regular and stiff ones - irrespective of all engineering efforts. This lesson is not new to us, we noticed this already many years ago. A second lesson is that MDR-statements must be handled with care unless they are accompanied by an explanation of all parameters which influenced damage.
Fig. 6. This sketch shows on the left side the orientation of 8 & 9 storey buildings and on the right side of those of 14 & 15 floors which we analysed in a very large housing colony in Mexico City after the earthquake of 1985 (9). The floor plans are shown to scale. The percentages indicate the mean damage ratios (MDR) related to the new replacement value of the buildings. It is seen that those buildings which had their long axis about in the E-W direction suffered much less damage than those standing perpendicular to them. The MDR-ratio is about 100 : 1 for the 8 & 9 storey buildings and about 3.6 : 1 for the 14 & 15 storey buildings. The latter difference would, however, been larger, had those orientated about E-W been as wide as those perpendicular to them. This sample illustrates also the effect of damage saturation.

For completeness sake we add values of ground shaking recorded on the extinct lake of Texcoco. One should, however, not assume that these values represent the actual behaviour at all places of this large area.

<table>
<thead>
<tr>
<th>DIRECTION</th>
<th>ACCELERATION cm/sec²</th>
<th>VELOCITY cm/sec</th>
<th>DISPLACEMENT cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>NS</td>
<td>98</td>
<td>39</td>
<td>17</td>
</tr>
<tr>
<td>EW</td>
<td>168</td>
<td>61</td>
<td>21</td>
</tr>
</tbody>
</table>
Fig. 7. The graphs show the correlation between MDR of moderately irregular buildings founded on medium hard alluvium and building quality expressed as base shear (BS) and MM intensity. The example entered corresponds to the one discussed in the text. It is assumed that buildings of about 4% g base shear are to be optimized considering orientational sensitivity. The general MDR of buildings of such resistance is about 18%. We assume that the MDR's differ by 4 : 1 for buildings experiencing the maximum shaking in the direction of their short (weak) axis and buildings about perpendicular to them. For the buildings which are more exposed the MDR would therefore be about 36% and for the others 9%. A MDR of 36% implicitly means substantial non-structural damage and even damage to structural members. According to our damage probability matrix (19) about 28% of these buildings would suffer damage between 50 and 100%, and this would imply the possibility of partial or total collapse. We assume that in view of this one opts for strengthening the buildings along their weak axis to such an extent that the MDR is halved. Keeping the damage ratio at 4 : 1 the MM VIII graph tells us that the building's structure should be designed for 8% g for earthquake forces acting along the short axis. In a more detailed analysis one would have to study the project regarding all other MDR-determining parameters mentioned in the paper in order to see whether such buildings conform to those forming the basis of these graphs (moderate irregularity and medium-hard alluvium). The main text discusses an example of an asymmetrical building. A second investigation must address the most important intensity. Details are discussed in reference (1); it may suffice to state here that MM VIII contributes most to damage from all possible intensities for buildings of medium strength, configuration and the subsoil assumed here. The better conditions are (stronger buildings, harder foundation material, near perfect symmetry, compatible materials, etc.) the more important become the (less likely) higher intensities and vice versa. The pragmatic method shown here for correcting the adverse influence of different strengthening from fill-in walls, irregularity, etc. has the advantage of being based on data from a very large sample.
Fig. 8. Substantially simplified version of our seismic index map (SIM), showing the region of the Indian subcontinent. The seismic indices (SI), i.e., the numbers, are a quantifying indicator of the general seismicity of the region. The SI can be used to calculate the return periods of selected intensities, of an earthquake of a specified magnitude within a certain distance from a place or of accelerations (1, 17). The SI's, however, also indicate whether there may be a seismic gap in a region. There are, for instance, abnormally low SI's in the marked regions a, b, c, and also d for which the very low SI's have not been entered. As the SI's shown here are based on the number of M 7 - 7.9 earthquakes recorded within a period of 79 years within an elliptical counting area of 125,000 square kilometers one can estimate that in the northern part of a and c and in the central belt of b about 20 earthquakes of this magnitude or, considering magnitude/number distributions about two of M 8 and above are "missing." Herefrom the probability of experiencing such earthquakes, selected intensities or accelerations can be calculated when using data presented in (17) and particularly in (1). We refrain at this place from discussing special factors related to plate tectonics, faulting, historical seismicity, etc. which must be considered in any evaluation of gaps. As mentioned in the main text, reference (1) will contain a special SIM which permits a substantially more accurate assessment of such gaps than the general SIM which was used to prepare this illustration.
REFERENCES

32. Innenministerium Baden-Württemberg, Erdbebensicher Bauen.
LESSONS FROM THE MEXICAN EARTHQUAKE 1985:
QUANTITATIVE EVALUATION OF DAMAGE AND DAMAGE
PARAMETERS
by
H. Tiedemann
(Federal Republic of Germany)

Summary

In Mexico City, all buildings of more than 5 storeys were inspected in order to assess the relative importance of different damage parameters. The general mean damage (MDR) was 32.1%; ranging between 94.13% for 2% g buildings and 1.89% for 10% g buildings. Stiff buildings had much lower MDR's than soft ones. Irregularity and asymmetry increased MDR's significantly. The MDR's of oblong buildings strongly depended on their orientation. Most of the overall damage resulted from failure of non-structural items. The paper presents a detailed account of these findings and the salient lessons to be learned from them.

Introduction

The purpose of the quantitative analysis was to improve the information compiled after earlier earthquakes (ref.1). The most important damage parameters are shear strength, degree of softness or stiffness of buildings, regularity and symmetry, orientational sensitivity, hammering, quality of the foundation material, resonance between the subsoil and the building, and performance of non-structural parts.

Earthquake building codes address only some of the above parameters and, depending on the type and location of the building, not necessarily the most important ones. Moreover, essential aspects are often left out.

This paper endeavours to present the most salient lessons concerning damage and damage parameters learned from the inspection of practically all modern buildings of more than 5 storeys in the Fondo del Lago region of Mexico City (the extinct part of the Lago de Texcoco).
Method and sample

Following the method employed when investigating damage from earlier earthquakes, we tried to achieve a complete sample, i.e. we investigated each building that met the qualifications mentioned above. In addition to avoiding bias, only complete samples will, in general, be large enough to provide sub-samples of sufficient size per essential parameter. The total sample presented here comprises 491 buildings of 6 to 29 storeys. About 88% of the buildings had 6 to 15 storeys. The total volume of the investigated buildings, exclusive of basements, amounted to 15,278,989 cubic metres.

A detailed questionnaire was used to record the location, orientation, and type of each building, architectural and structural layout, design, structural and non-structural parts, etc., cause(s) of damage, as well as the damage to all components.

Results

Damaged buildings were nearly exclusively located in the region of the extinct Lake Texcoco and, because of the predominant near-harmonic low-frequency shaking of the ground, generally only buildings of 5 or more storeys were affected. The damaged buildings appear to be concentrated in pockets of damage; this phenomenon has been interpreted as a resonance effect of standing waves (ref.2). These pockets are, perhaps, more convincingly explained by the concentration of vulnerable buildings in such areas (ref.3).

The general exposure depended most prominently on the interaction of subsoil and buildings (‘resonance’). As a general rule, buildings of few storeys, even if old and frail, suffered little or no damage. The mean damage ratio (MDR) of tall buildings was primarily controlled by the base shear and their relative stiffness. This must, however, be taken with a pinch of salt because buildings in which natural frequency was a harmonic of the predominant ground frequency (abt. 0.5 Hz) show a higher MDR if the sample is normalized for base shear and if the effects of irregularity and asymmetry and of orientational sensitivity are removed. The correlation between base shear and MDR and the influence of other parameters is shown in Fig. 1 below.
Stiff buildings suffered less than soft ones (Table 1). The scatter in the MDR of stiff and soft buildings is caused by the other damage parameters mentioned above.

**TABLE I**

<table>
<thead>
<tr>
<th>MDR (%) of Stiff and Soft Buildings of Different Base Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Shear (% g)</td>
</tr>
<tr>
<td>Stiff Buildings</td>
</tr>
<tr>
<td>Soft Buildings</td>
</tr>
</tbody>
</table>

The stiffness of oblong buildings tends to be much greater in the direction of their long axis than in the direction of the short axis. In Mexico City, accelerographs measured much stronger ground movements in the EW-direction than perpendicular to it. It must be noted, however, that unless ground movements are analyzed vectorially it is difficult to state in which direction shaking was most prominent, particularly if one has to make proper allowance for frequency bands of interest. Buildings having their long, i.e. stronger, axis in the EW-direction suffered much less than identical buildings orientated perpendicular to them (ref.6). We shall return to this aspect when discussing orientational sensitivity, but it must be noted that this observation also underlines the importance of stiffness in reducing damage, in particular if tall buildings stand on deep soft ground.

The MDR's of irregular and asymmetrical buildings were persistently higher than those of regular ones, even when considering the complete, i.e. heterogeneous, samples. In brief, irregular buildings are those which have a floor plan or elevations which are not a simple rectangle, e.g.: floor plans of the shape of an L, U, T, or H; or elevations like an L or inverted...
or buildings with a completely or partially soft ground floor; or of different resistance and
stiffness along the main axes, e.g. because of plate glass on one side and brick or concrete
walls on others. Due to the effect of fill-in walls, oblong rectangular buildings must, however,
be considered irregular.

For the entire sample of buildings with base shears of about 3% g and 4% g all regular
buildings had MDR's of about 62% and 26%, and irregular buildings had 77% and 42%,
respectively. Comparing soft regular with soft irregular buildings, the difference in MDR was
about 1:1.6.

As regards the beneficial influence of symmetry, for instance, all quadratic buildings of
3% g had an MDR of 15%, slightly higher than that of all stiff buildings, but much lower
than the MDR's of all buildings (64.7%), or of all irregular ones (76.8%). Such differences
should not be taken as hard and fast rules because of the contributing factors, but irregular
and asymmetrical buildings are in general much more exposed than regular and symmetrical
ones.

We turn now to the sub-samples of buildings arranged according to base shear and the
influence of different damage parameters.

Buildings of about 2% g had an MDR of 94.13%; they were soft and had their short
axis in the direction of maximum shaking. Also, all 2.5% buildings were soft. Those shaken
predominantly in the direction of their short axis had a MDR of 94%, those having their long
axis in the approximate direction of maximum shaking had an MDR of only 23%. The av­
erage MDR for the complete sample was 75.8%. This illustrates how misleading statements
may be which mention only average damage.

The sample representing 3% g buildings was contaminated by many parameters affect­
ing damage. For all soft buildings which were shaken more strongly in the approximate di­
rection of their width, the MDR was 65% for regular and 85% for irregular buildings. In spite
of the many other contributing parameters, all buildings shaken more strongly in the direction
of their length had a lower MDR than the others, viz. 59% compared with 68%. The MDR
of all buildings which were more or less regular was 62%, the MDR for irregular ones was
77%. The differences in MDR's generally decrease in the high-loss categories because of
damage saturation (ref.7).

The experience with 3.5% g buildings also points at lacunae in structural design. The
overall MDR was 48.1%. Soft buildings shaken mostly in the direction of their short axis had
an MDR of 46% if they were regular; irregular ones, however, had an MDR of 77.3%. Even
for buildings of heterogeneous stiffness the MDR's differed very much depending on their
orientation, viz. 20% (long axis) and 65% (short axis).

For 4% g buildings the overall MDR was 34.03%. The MDR of all rectangular
buildings was 26%, but 42.4% for irregular ones. If elevations were irregular as well, the
MDR reached 78.6%. The MDR's of buildings having their long or short axis in the ap­
proximate direction of maximum shaking were not too dissimilar (29% and 34% respectively);
this is explained by the other damage parameters concealing this effect. Irregular soft
buildings shaken more along their length had 39.7% MDR; for those shaken more in the direction of the short axis it was, however, 69.2%. The MDR of buildings of this strength which were asymmetrical (quadratic floor plan) and stiff, was only 1%. Oblong and stiff buildings with their long axis in the approximate direction of maximum shaking had an MDR of 2.83%.

The 5% g buildings had a global MDR of 19.67%. The MDR of those exposing their short axis was about 30%, those shaken more along their long axis had one of only 14%. The MDR of irregular buildings was 30.3% against 15.9% for regular ones. For 6% g buildings MDR-differences, except the one for stiffness/softness (cf. Table 1), were smaller because of other contributing factors. Nonetheless, MDR's for regular buildings, and those having their long axis in the approximate direction of shaking, were persistently lower.

For 8% g buildings the MDR-ratio for long to short-axis buildings was 1.73% to 7.63%. The MDR of stiff buildings experiencing shaking mostly in the direction of their long axis was only 0.81%. The 10% g buildings were all stiff, except for one which belonged to the mixed type. The global MDR was 1.89%.

As regards orientational sensitivity, some cautionary remarks are appropriate. In reality, the influence of the fill-in walls in oblong buildings is very much larger than indicated by the above figures. We have already published a special paper dealing with orientational sensitivity (ref.6). The MDR’s quoted here are for heterogeneous samples. First, the length to width-ratios differ considerably; for instance, a rectangular building measuring, e.g. 20 x 30 m is far less affected by this damage parameter than one measuring 12 x 60 m. Secondly, there is also scatter as regards orientation because the roads of México City, i.e. the orientation of buildings, do not form a regular rectangular grid. Thirdly, the direction of known average maximum and minimum shaking did not exactly coincide with the orientation of the oblong buildings; in addition attention is invited to our earlier comment on ground movement.

Conclusions and lessons

The conclusions and lessons from the investigation of earthquake damage to buildings in Mexico City confirm the results obtained from earlier earthquakes. The most important general lesson is that the shear strength of the structural parts of a building is just one parameter among many. More specifically, the following lessons emerge as regards important damage parameters:

(a) The current philosophy on which most earthquake ‘resistant’ building codes are based is the protection of people against partial or total collapse. The incidence of casualties is correlated nearly exclusively with the base shear of the structure. However, as soon as buildings are of reasonable strength, the chance of severe structural failure drops drastically, and the risk of casualties, and the contribution to damage, shifts from structural to non-structural items. The Mexican sample shows, for instance, that of the 16% buildings of about 4% g, 14 sustained damage rates (DR) of 75% or greater (8.33% of
the sample). Of the 6% g buildings (n = 62), however, only 2 had a DR of 75% or greater, i.e., 3.22% of the sample.

(b) When designing buildings, one should pay particular attention to columns. Of the 112 buildings equivalent to 6% g or better, 17 had damage to columns, and the MDR was 6.3%.

(c) The problem of ‘resonance’ between building and predominant frequency bands of the subsoil must be considered; and not only in Mexico City where an earlier chance to learn this lesson occurred in 1957 (refs.8,9). The author showed a significant dependence of predominant frequencies of soft layers on their depth (ref.10) permitting pragmatic corrective steps.

(d) The importance of stiffness of buildings, particularly when standing on soft foundation material cannot be overestimated. C.F. Richter has already drawn attention to this (ref.11), and our Mexican data, which are in line with our global sample (ref.1), support this view.

(e) The lack of knowledge about the contribution of ‘fill-in walls’ to the shear strength and/or stiffness of a building is appalling. The differentiation made between load bearing and non-load bearing members of a building is too parochial. This illogical attitude can lead to very severe damage and even to loss of life. In a building constructed entirely of brick, the respective walls are considered load-bearing. As soon as bricks fill the spaces between the columns, these walls are, however, thought of as non-structural elements and their contribution to strength (and damping) is neglected. A pragmatic approach is, however, possible (ref.7).

(f) Asymmetrical and irregular buildings are more exposed than symmetrical and regular ones. This factor must be considered in an attempt to reduce casualties and economic damage. Such approach has been published (ref.7).

(g) Orientational sensitivity is an important aspect, in particular for oblong buildings (ref.6). The present study underscores this; for instance, all buildings of 6% g or better which have collapsed experienced maximum shaking in the direction of their short axis.

(h) Damage is, in general, mostly found in the ground floor of buildings. However, in Mexico City, damage was scattered over all storeys and severe damage was often noted in the upper ones. A general conclusion is inescapable, viz. it still appears to be impossible to design buildings in such a manner that all storeys are of about similar strength.

(i) Hammering between neighbouring buildings was far more important in Mexico than in all other earthquakes we inspected. This is mainly due to the long duration of nearly harmonic shaking which amplified the deflection of tall buildings. Important damage from hammering occurred in 8.18% of all buildings with a base shear below 6% g, but only in 3.57% of the stronger ones.

(j) By and large, most of the direct and indirect damage resulted from non-structural items. This is no new lesson (ref.12,13). Much more attention must be paid to these items.
References


DESIGN STANDARDS
IMPROVEMENT OF DESIGN AND CONSTRUCTION STANDARDS

by

F. Aptikaev

(Moscow, USSR)

To reduce earthquake hazards to a minimum using economically viable actions it is necessary to conduct investigations of two kinds: "engineering seismology" and "earthquake engineering". These investigations must be regulated by the relevant codes. Engineering seismology consists in avoiding the hazard, i.e. in choosing the safest place in a region, as well as choosing sites with favourable ground conditions. The investigation of the seismic source zones is called "detailed seismic zoning" (DSZ); the investigation of local ground conditions is called "microzonation" (MZ). Engineering seismology also includes instrumental seismological observations. The weak and strong ground motion records obtained, as well as the results of investigation of seismic source zones and the medium where seismic waves propagate, allows strong ground motion to be predicted. All assessments must be given in probabilistic form. In the USSR at present only MZ work is included in the legal codes, but in practice the elements of DSZ are very often carried out during microzonation.

The main goal of DSZ is the assessment of seismic hazard for a particular construction site. This assessment should be based on the results of the following four studies:

• identification of zones of possible earthquake occurrence and the evaluation of their parameters;
• assessment of the earthquake recurrence and of the maximum magnitude;
• determination of possible strong ground motion;
• assessment of the hazard associated with slow crustal deformation: creep, vertical and horizontal displacements, tilt.

For critical structures, such as nuclear power plants, high dams, etc., DSZ is carried out within 40 km of the outer boundary of the construction site, whatever the supposed seismicity. By increasing the distance from 40 km to 100 km we can disregard seismogenic features capable of generating earthquakes with $M_{max} < 5.0$, and reduce the degree of detail for the map. The mapping scale for the first class structures is about 1:200,000, that for the second class structure is 1:500,000.

The results produced by DSZ and MZ must form the basis for estimating the following parameters of ground motion:

• level of vibration: peak acceleration, velocity, displacement (adding to these residual displacement when the relevant technique has been developed);
• impulse width, i.e., the time during which the level of trace envelope exceeds half the peak amplitude;
• spectral width.
Initially one must estimate the probabilities of the occurrence of various values of the parameters indicated and their combinations. According to the estimates obtained, one then selects an appropriate ensemble of strong motion records from the library of world data. These data are also used to generate synthetic accelerograms. The estimates obtained in engineering seismology form the basis for computations in earthquake engineering. The output of engineering seismology is the input of earthquake engineering.
DESIGN AND CONSTRUCTION STANDARDS IN
EARTHQUAKE-PRONE AREAS
by
V. Oizerman
(USSR)

Building codes in different countries are based, in the main, on similar theories. They take into account achievements of theory, experience in design, construction and exploitation of buildings in earthquake-prone areas, results of experimental studies, and the analyses of damage from earthquakes. The building code in the USSR, 'Construction in earthquake prone areas' was adopted in 1981. Four damaging earthquakes have occurred since that time in: Gazli (1984), Kairakum (1985), Moldavia (1986) and in Armenia (1988). The earthquakes turned out to be a test for building resistance and a check of the basic theories in the building code. In this process, certain illusions were given up, and new questions and problems appeared. For example, the Gazli earthquake drew experts' attention to the imperfection of walls carrying the load of frame buildings; in Kairakum the main damage was caused by falling floors; in Moldavia it was caused by mass damage of walls; and in Armenia many buildings collapsed from loss of stability.

The causes of mass damage to buildings during the Armenian earthquake will not be analyzed in detail, rather those aspects will be touched on which it is believed are decisive for understanding the behaviour of buildings under intensive seismic load. First, defects in construction and design caused the collapse of structures which did not correspond to the design, nor the design to the building code. Secondly, massive damage to joints of structural members took place. A building remained unchanged during the first, two or three vibration cycles only; then changed to such an extent that it appeared to be a building which was not earthquake resistant. Thirdly, the character of reinforced concrete frames manifested itself. One should admit that projections for the possibility of plastic deformations (at places of conventional plastic hinges) were only partly realized. Thus, initial grounds for certain theoretical coefficients turned out to be wrong in this case. Instead of gradual (from cycle to cycle) increases of damage, an instant 'failure' of the system occurred at a particular moment. Fourthly, errors in the assessment of seismic hazards, under-estimated in the design stage, led to structural changes.

The aspects mentioned above are not specific to the Spitak earthquake. To a certain degree they can manifest themselves during any intensive earthquake. This makes it necessary to take the above factors into account when revising building codes in earthquake-prone areas. Several suggestions on correcting the USSR building code are given below:
1. It is useful to distinguish three stages in the maintenance of a building: I: before an earthquake, II: during an earthquake and, III: after an earthquake. The stages differ first of all in types of excitation and intensity of load. Stage I involves maintenance loads (the main ones and their special combinations). In stage II it is necessary to analyze two design situations: a) seismic excitation corresponding to the design intensity (in case of the absence of the zoning maps); b) the maximum possible seismic excitations whose exceedance probability is very small. This design code allows one to take into account the possible overloading of structures causing assessment errors by the corresponding ‘overload coefficient’. In stage III the design intensity is to be taken into account.

The maintenance stages mentioned above differ both in the excitation and design schemes of buildings, and their limits. For example, in stage I structures should work in the elastic deformation stage; in stage II residual deformations and damage are permissible; moreover, in situation ‘a’ they should not prevent ordinary exploitation of the buildings for a long period, while in situation ‘b’ any damage is permissible if the general stability of the building and the safety of people is ensured. It is obvious that when calculating a building for aftershock (stage III) it is necessary to take into account the fact that the initial state of a structure depends upon the degree and level of damage which occurred in the building during stage II.

As the ultimate aim of calculations is to ensure the required safety level of structures with reference to limit states, then, in this case, it is worthwhile using the principle of continuous reliability to ensure the same reliability for all three exploitation stages. To put the above principles into practice, it is necessary to estimate the state of buildings after an earthquake.

2. Another acute problem is the permissible level of damage. An analysis of the consequences of earthquakes confirms a certain level of damage in buildings caused by design defects. In general, this be considered permissible when the damage of buildings does not exceed level 2 on the seismic scale. The USSR building codes and corresponding design coefficients have been worked out in accordance with a similar damage level. But the building codes do not give instructions as to where, and in which structural members, damages are permissible or the level to which they should be limited. Certain structures should be near the elastic stage, irrespective of the level of excitation; this means that damages in them are inadmissible and irreversible. The other structural members may be completely excluded from the considerations.

In a complex system, the failure of a member should not lead to the failure of the whole system. Structural members possess different levels of responsibility for the possible transformation of a system to the limit state and they can be designed with reliability coefficients adopted in connection with the characteristics of the members and their location.

3. A survey of buildings has revealed a somewhat higher vulnerability of structures along the outward contour. This phenomenon takes place even in those cases where the buildings are comparatively small in plan and the internal carrying members are symmetric about the main structural axes. One of the possible causes for torsion may be non-
simultaneous and different damages to structural members, which lead to transformation of the building during an earthquake, and to the occurrence of internal eccentricities. Thus, during the process of interaction between building and vibrating ground, the building which has been symmetric before the earthquake can behave as a building with asymmetric location of stiffness. It is useful to supplement the building codes with recommendations that design should meet the requirements of symmetry in the limit state.

4. Special attention should be paid to the recommendations on the design of joints of carrying members. To improve reliability of the joints, their design should take into account additional coefficients whose values are directly connected with the possibility of ensuring equal strength for joints and structural members.

5. Certain types of ground significantly change their physiomechanical characteristics under seismic excitations (especially high-frequency ones), which results in the subsidence of foundations, wells and lifelines, landslides and loss of stability of slopes, etc.

This paper has dealt, very briefly, with only the main problems which arise in any effort to improve the building codes for earthquake prone areas.
THE JAPANESE EARTHQUAKE OF 1978
by
Shigeji Suyehiro
(Japan)

Summary

In January 1978, an earthquake of magnitude 7.0 took place off the Pacific coast of Honshu, Japan about 110 km south-west-south of Tokyo, which caused some casualties and property damage. This earthquake was preceded by a large number of foreshocks and followed by many aftershocks. The earthquake itself was not very large as a natural event. However, it did provide a number of lessons on the importance of issuing timely information to the public in order to mitigate the impact of earthquake disasters.

Earthquake Data

The earthquake killed 26 persons and wounded 139. The focal depth was very shallow, and a slight tsunami was generated. The source elements as follows:

- Origin time: 12:24 hours 38.6 seconds on January 14, 1978
- Epicentre: 34 degrees 46 minutes north and 139 degrees 15 minutes east
- Depth: 0 km
- Magnitude: 7.0 (Ms)

In addition to various important scientific findings, the earthquake resulted in many lessons pertinent to the Japanese effort to mitigate earthquake disasters, particularly regarding their social impact.

Social Impact

(a) Perceptible shocks began to occur at about 8 o'clock in the morning. Accordingly, the Japan Meteorological Agency made an unprecedented special announcement: "The present earthquake is very large, and may be followed by even larger ones, which could cause some damage. Local people should be very careful and take every possible countermeasure." At that time, it was not certain that the earthquake would reach a magnitude of 7; however, within two hours, an earthquake of that size did occur. But the level of casualties and damage was mitigated considerably, in comparison to the possible level if
no warning had been made. Many individual houses in Japan were still vulnerable to fire; the earthquake occurred at lunch time, but, only one fire broke out, proving the importance and effectiveness of public preparedness in mitigating disasters.

(b) The Japan Meteorological Agency disseminated a series of information to the public but when one message was relayed by the local prefectural government, the "magnitude of the earthquake" and "seismic intensity" were confused. Namely, M 6 was wrongly reported to be Intensity VI (about 11 in the MM scale), which caused serious social unrest. Once again it was realized that the education of the population is extremely important.

(c) The possibility of a Tokai earthquake of M 8 class occurring in the near future has already attracted nation-wide concern in Japan, and triggered the promulgation of the "Countermeasures Act against Large-Scale Earthquakes".

Seismic Fault and Aftershocks

The seismic rupture of this earthquake was generated towards the west from the epicentre for a distance of about 50 km. The aftershock region also extended to the west along the seismic fault (see Figure 1). Within 30 minutes after the main shock, aftershocks occurred in the entire aftershock region. Past experience shows that there is a relation between the diameter of the aftershock region and the magnitude of the main shock:

\[ \log L = 0.5 \times M_m - 1.8 \]

L: diameter of aftershock region in kilometers
Mm: main shock magnitude.

<table>
<thead>
<tr>
<th>Mm</th>
<th>8.0</th>
<th>7.5</th>
<th>7.0</th>
<th>6.5</th>
<th>6.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>160</td>
<td>89</td>
<td>50</td>
<td>28</td>
<td>16</td>
</tr>
</tbody>
</table>

Together with the pattern of the decaying frequency of aftershock occurrence, this is one of the symptoms by which it can be diagnosed whether the earthquakes concerned were of the normal type, e.g., sequence of foreshocks, main shock, aftershocks, or that of main shock, aftershocks, and aftershocks which diminish with the elapse of time. Therefore, within a relatively short period of time, if a certain number of local seismic stations are available, the size and region of aftershocks should be established. When disastrous earthquakes have been followed by further perceptible earth tremors, the population in the region is usually attacked by the instinctive fear that another, equally large or even larger, earthquake might occur. This was the case in the Spitak Earthquake. Immediate dissemination of whether the frequency or size of impact of the aftershocks is normal or not will be important in calming the psychological fears of the public.
The Largest Aftershock

In addition to this kind of psychological and social unrest, there is the danger of partly damaged and unstable structures, falling rocks, etc. still being prone to further and larger aftershocks. Structures which have survived the main shock, because of their distance from it, may be damaged by larger aftershocks, occurring nearby which are smaller in magnitude, but stronger in local intensity.

This was indeed the case in the Japanese Earthquake of 1987. Roads on the western side of the Izu Peninsula were not damaged by the main shock but were seriously damaged by the largest aftershock, which occurred about 9 hours after the main shock on the opposite side of the aftershock region. Past experience of disastrous earthquakes in Japan indicates that:

(a) The mean magnitude difference between the main shock and the largest aftershock is slightly larger than 1.0;

(b) In many cases, the largest aftershock occurs near the opposite side in the aftershock region relative to the epicenter of the main shock;

(c) Often the largest aftershock occurs within 24 hours after the main shock, when the magnitude of that shock is larger than 7.

Even if the main shock cannot be predicted, it is important for possible disaster mitigation due to aftershocks, to establish the characteristics of aftershocks as quickly as possible, and disseminate this relevant information expeditiously. In the present state of seismology such actions alone would make a considerable contribution to the mitigation of earthquake disasters.

DISTRIBUTION OF FORESHOCKS AND AFTERSHOCKS OF NEAR IZU-OISHIMA EARTHQUAKE IN JAPAN, 1978

FIGURE 1
CHANGE OF FORESHOCKS AND AFTERSHOCKS WITH TIME NEAR IZU-OISHIMA EARTHQUAKE IN JAPAN, 1978
DATA ON NATURAL AND INDUCED SEISMICITY IN BRAZIL
by
J.A.V. Veloso
(Brazil)

Summary

Studies on the data collection and detailed analysis of Brazilian tremors are relatively recent and a better knowledge of our seismicity is only now being obtained.

Though Brazil has never been affected by a catastrophic earthquake, the growing size of large cities, and the number of new structures being built, demand that efforts be made to improve the monitoring of the territory.

This paper provides a general view of the distribution and the level of Brazilian seismicity.

Introduction

Almost all Brazilian terrains are represented by precambrian shield and paleozoic sedimentary rocks. Thus Brazil has had a relatively undisturbed seismic history, like most intraplate regions. The distribution of earthquakes have not occurred at random, and clusters of epicenters can be recognized. Although some tremors have occurred as large as 5.0 and with a maximum intensity VII MM, they have never produced casualties or heavy damage.

The first seismographic station in Brazil was installed, in the beginning of this century, in Rio de Janeiro. But only in the middle of 1970 were several further stations established in order to monitor large reservoirs following the construction of dams. This new situation promoted a development in seismological studies.

In the main, data from earthquakes in Brazil have been produced by Branner (refs.1,2) and by the recent catalogue of historical and instrumental data, covering the period from 1560 to 1980 (ref.3).
Brazilian Seismicity

Natural Activity

The epicenters of all shallow Brazilian events, with magnitudes above 3.5 and simplified geological features of the country, are represented in Fig.1. The present distribution of seismographic stations and seismological centers are shown in Fig.2. The scale of detectability available through this national network has a magnitude of 3.5 for the Southeastern and Northeastern regions and around 4.0 for the rest of the territory.

Comparatively-speaking, the Northeastern, Southeastern and Amazon regions are the most active seismotectonic provinces in Brazil. A particular activity area is located on the Peru border, but seismicity characterized by deep focus is related to the subduction of Nazca Plate.

No direct correlation between the main geologic provinces and the distribution of epicenters can be observed. One large shock recorded on January 31, 1955 ($M_a = 6.6; 12.42S, 57.30W$) occurred in the old craton area. Several events were located on the Amazon intracontinental basin, but few or none have appeared in the other two similar basins. The proterozoic fold belt areas revealed a similar pattern in the distribution of epicenters. In all probability, pre-existing planes of weakness and other tectonic boundaries inside large geological units, are more important than global differences in the geological provinces themselves.

Brazilian seismic history does indicate cases of tremor recurrence: Bom Sucesso-MG ($21.0S, 44.6W$) was shaken during the 1920's and 1930's by shocks which reached a magnitude 3.8 and a maximum intensity of VI MM. Another case occurred in Pereiro-CE ($5.9S, 38.6W$) where the main shock reached 4.6 and a maximum intensity of VII MM. But the most interesting sequence, which has been studied in detail (ref.4), affected the region of Joao Camara-RN ($5.5S, 33.7W$). The first tremors occurred in August 1986 and the area is still active. To date, almost 20,000 events have been recorded, several of them being felt by the local population. The main shock of this sequence reached 5.1, causing moderate damages e.g.: cracks in the walls of many houses, and partial collapse of walls located in rural and urban zones. Only one house, in the rural area close to the fault, totally collapsed. Preliminary results show that activity is concentrated along a well-defined fault, 30 km long but not detected on the surface. A composite fault plane solution indicates that the mechanism has, mainly, a strike-slip dextral motion (along a N40E) with a small normal component.

Some authors have indicated that the interior of some Brazilian areas show a general horizontal compression stress associated with the convergence of the Nazca and South America Plates (refs.5,6).
Induced Seismicity

The Seismological Observatory of the University of Brasilia was the first to study, with appropriate equipment, the phenomena of reservoir-induced seismicity (RIS) in Brazil (refs.7,8). Today, this institution coordinates the monitoring of 14 dams located in different regions of the country. The main seismic and physical parameters of the dams that have had RIS are shown in Table 1. None of these cases resulted in any type of damage to the dams. A summary of RIS information is given below:

- **CAJURU DAM** a small reservoir situated on precambrian rocks where the population only felt the first shocks some 16 years after the filling of the lake. No natural seismicity was observed a priori and the Brazilian seismic catalogue indicates occurrence around 100 km south of the reservoir. The main activity occurred in 1972, when several events were felt up to 15 km away, and the main tremor reached a value of 3.7. The focal mechanism suggests that the events were generated by a reverse fault (N20E-sub-vertical) that corresponds to the general fault trend in the area. At present the level of activity is low, though reactivations are observed from time to time.

- **PARAIBUNA-PARAITINGA DAMS** experienced the first shocks one year after the impoundment of the lake and seismicity reached its magnitude at 3.4. The hypocenters are situated less than one kilometer below the lake. Results of studies of focal mechanism were very important because they demonstrated that the activity was generated by stress on faults and secondary fractures, and could not reactivate the main structures of the area, which cross one of the dams and Paraibuna city.

- **EMBORCACAO DAM**, also located on precambrian rocks, was affected by micro-earthquakes recorded seven months after impoundment. Magnitude did not reach 2.0 and the epicenters were situated mainly in the lake area. The time distribution of seismicity is not uniform but shows periods of quiescence. The monitoring started before the construction of the dam and no natural seismicity was recorded in this area.

- **TUCURUI DAM** showed activity six months after the completion of the lake. Several events of the magnitude of 3.0 to 3.4 were recorded by a local network which was installed prior to the dam construction. The epicenters are located in the lake.

Final Remarks

It is probable that the main event triggered by reservoir impoundment in Brazil occurred on February 24, 1974 in the area around Porto Colombia (20.3S, 48.5W) and Volta Grande (19.9S, 48.2W) reservoirs. The date was subsequent to impoundment of the first lake and during the filling of the second. A m<sub>s</sub> 4.2 earthquake affected an area of 8.6 x 10<sup>3</sup> km<sup>2</sup> that is characterized by extremely low historical seismicity. After this occurrence, several Brazilian electric power companies initiated a programme of monitoring their large reservoirs.
Though not of a high magnitude, the level of seismic activity in Brazil, including cases of induced seismicity, is not negligible and should be taken into consideration in designing important structures.
FIG. 1 - EARTHQUAKES (FROM 1767 TO 1958) AND MAJOR GEOTECTONIC FEATURES OF BRAZIL

LEGEND

MAGNITUDE mb
- 3.5-4.4
- 4.5-5.4
- 5.5-6.6

DEEP EARTHQUAKES

CRATONIC AREAS

MOBILE BELTS OF THE BRASILIAN / PAN AFRICAN CYCLE.

PRECAMBRIAN AND PHANEROZOIC COVERS.

FIG. 2 - BRAZILIAN SEISMOGRAPHIC NETWORK

LEGEND

SEISMOLOGICAL CENTERS

LOCAL NETWORK

SINGLE STATION
<table>
<thead>
<tr>
<th>RESERVOIR</th>
<th>Dam height (m)</th>
<th>Capacity $\times 10^9$ m³</th>
<th>Year of impoundment</th>
<th>Parameters of main shock</th>
<th>PRE</th>
<th>POST</th>
</tr>
</thead>
<tbody>
<tr>
<td>CARMO DO CAJURU</td>
<td>20</td>
<td>0.19</td>
<td>1954</td>
<td>JAN/23/1972 3.7 VI</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>PARAIBUNA/</td>
<td>98</td>
<td>4.74</td>
<td>1976</td>
<td>NOV/16/1977 3.4 IV</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>PARAITINGA</td>
<td>104</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EMBORÇAÇÃO</td>
<td>158</td>
<td>17.50</td>
<td>1981</td>
<td>MAI/18/1984 1.7 -</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>TUCURUÍ</td>
<td>100</td>
<td>43.00</td>
<td>1984</td>
<td>FEV/06/1986 3.4 -</td>
<td></td>
<td>X</td>
</tr>
</tbody>
</table>

Table 1 - Reported cases of RIS studied by Seismological Observatory of University of Brasília
References

(a) Branner, J.C., 'Earthquakes in Brazil' Bull. Seism. Soc. Am., 2, 105-117, 1912
(b) Branner, J.C., 'Recent earthquakes in Brazil' Bull. Seism. Soc. Am., 10, 90-104
(g) Mendiguren, J.A., 'Estudos sobre sismicidade induzida desenvolvidos na Universidade de Brasilia.' Simp. Sismicidade Natural e Induzida. ABGE/CESP, Sao Paulo, 49-67, 1979
SEISMICITY AND SEISMIC RISK IN AND AROUND EGYPT
by
E.M. Ibrahim
(Egypt)

Summary

Earthquake data in Egypt and its vicinity were compiled for the period between 2200 B.C. and 1980 A.D. Data earlier than A.D. 1900 were adjusted to historical documents of earthquake disaster, and the others (Period: 1901 - 1980) were arranged by consulting seismological bulletins of many observatories in the world and the seismological report of Helwan Institute. It was recognized from the above data that active seismic areas in and near Egypt are: (1) the northern part of the Red Sea, (2) the central part of the Red Sea, (3) the Levant Region, and (4) the north east part of Egypt.

Seismic Risk maps for the area concerned, that is, regional distribution of maximum earthquake motion for some return periods were drawn up making use of the aforesaid data, attenuation models and the methods of extreme value fitting. The maps consist of the following two kinds; (1) The maximum particle velocity on the base rock, (2) The maximum acceleration on the ground.

The return periods of these maps are 300 and 600 years, respectively. One the basis of these results, Egypt and its vicinity can be regarded as areas of moderate seismicity.

Introduction

The seismicity of a region can be defined as the spatial and temporal distribution of earthquakes. For Egypt and its vicinity, seismicity investigations have taken place since 1960 or earlier by Madwar & Madwar and Ismail (1960) (ref.8). In 1968, Gergawi and El-Khashab could find two local seismic belts, where earthquakes of magnitudes between 0 and 4 may occur (ref.4). Ibrahim (1976) could catalogue the Egyptian earthquakes since 2200 B.C. till A.D. 1972 (ref.6).

According to Maamoun and Ibrahim (1978) (ref.9) the earthquake activity in the country was emphasized and related to the tectonic frame work of the region. Allam et al. (1978) (ref.1) could give more details about the geology and tectonics of the Egyptian earthquakes. Maamoun et al. (1980) (ref.11) gave a fault plane solution of Dec. 9, 1978 earthquake (Fig.1.). The northern Red Sea and its two gulfs of Aqaba and Suez are the most seismic active regions.

Historical and instrumental earthquake data should be combined to construct seismotectonic and seismic zoning maps. Two such maps for Egypt and the around were
compiled by Ibrahim in 1976. A further seismic zoning map for Egypt was compiled by Maamound et al. in 1980 (Fig.2). Different macroseismic maps for individual historical and recent earthquakes were compiled by Allam et al. (1978), Maamoun & El-Khashab (1978). Macroseismic maps for the major Egyptian seismic events were compiled by Maamoun (1979) (Fig.3,4, and 5) (ref.10).

Hattori and Ibrahim tried to make seismic macrozoning maps which mean regional distribution of expectancies of maximum earthquake motions (acceleration, particle velocity) for some return periods (1981) (ref.5) (Fig.6 and 7). The results were in coincidence with a seismic risk study made by Bath (1981) (ref.3) in spite of the different statistical approach used for the last.

Regarding all of these investigations, we can conclude that Egypt and its vicinity are regions of moderate seismicity. However, an isoseismal map of the Nov. 14, 1981, Aswan Lake earthquake and the focal mechanism study of the event done by Kebeasy, Maamound and Ibrahim (1981) can simply prove our conclusion (Fig.8) (ref.14).

**Geology and Tectonics of the Seismic Active Regions in and Around Egypt**

As earthquake occurrence is a leap in tectonic activity, the comparison of the geological and/or tectonic maps and the maps of the geographical distribution of epicenters of the earthquakes, which took place during a period more than 1000 years may give an idea about the future seismic risk in any region.

Geologically, Egypt can be divided into: the northern part (North to 28° latitude), which is tectonically unstable; the southern part (South to 28° latitude), which is known as the stable shelf; the Red Sea rift system, with its two gulfs; and the Arabo-Nubian (African) massive. These main four tectonic regions are surrounded in the north by the south east Mediterranean basin, and all are penetrated by the Nile river (Said 1962 and 1981).

Two local seismic belts discovered earlier by Gergawi and El-Khashab (1986) have two directions. The first, running almost parallel to the Red Sea, starts far in the south in the Abu-Dabbab and Wadi El Gemal area. The second local belt crosses the Nile delta in a direction parallel to the Mediterranean Egypt coast. The epicentres of historical and recent earthquakes are located parallel to these tectonic belts, where normal and horizontal faults were generated either in the Red Sea coastal area or in the north of the country.

South-west of Cairo and Giza, the two principal earthquakes of the Fayoum province took place in 1303 and 1847, and, in southwest of the country, the three recent earthquakes occurred in 1920, 1926 and 1978.

Tectonic and focal mechanism investigations of these earthquakes indicates the generation, or reactivation, of the old normal and horizontal faults. In 1982 Neev et al. (1982) (ref.13) considered these fractured zones as a part of the Pellusium megashear system, which
extends from the border zone of Anatolia, along the eastern Mediterranean and across Africa from the Nile delta to the delta of the Niger in the gulf of Guinea (Ibrahim (1976) and Maamoun and Ibrahim (1978) (ref.13).

In the Lake Nasser (Aswan/or High Dam) area, the presence of such a huge water reservoir could create very localized stresses especially at old fault systems. The present distribution of microearthquake epicentres in the above mentioned area make the above assumption is acceptable.

In Levant region, the more geologically young rift systems were confirmed by the occurrence of the historical and the present century earthquakes.

The southeast part of the Mediterranean Sea, where the Nile delta cone and the different sea ridges and trenches are present, marks of deep faulting in the two directions of the geographical latitudes and longitudes were found. These areas of the sea have a big accumulation of recent sediments. However, the rare events took place in the south of both Crete and Cyprus islands does not mean that these areas are not tectonically active.

Towards the north of the East Mediterranean the colliding of Africa and south Europe take place and many big earthquakes were reported historically or at present.

Making use of all the geologic, tectonic, and earthquake data of this area (between 21°E - 40°E and 15°N - 36°N) and the regional distribution of maximum earthquake motions for return periods 300 and 600 years, (Fig. 6 and 7), a seismic zoning map for the area is suggested (Fig. 9).

This macroseismic zoning map is a general standard map for the use of earthquake engineers. For sites of special engineering interests, informations on subsurface conditions and vibratory characteristics of the sites by means of study of dynamic characters and microtremors are also necessary. These are the matter of study nowadays in Egypt.

Acknowledgements

The author wishes to convey his gratitude to JICA and ISEE authorities for giving him the opportunity to attend the earthquake engineering seminar for 1983.

Gratitude and thanks are also due to Dr. Makoto Watabe, director of ISEE, and members of the Institute for their review of this manuscript and giving valuable comments.
References
1. Allam et al., "Geological and Seismology Studies for three sites, El-Dana, Zafarana
2. Allam, A.M., "Seismological Criteria for site selection in evaluating a design
76, (1968).
(1979).
9. Maamoun, E. & Ibrahim, E., "Tectonic Activity in Egypt as indicated by Earthquakes,
Vol. 18, (1980).
12. Megahed, M., et al., "Earthquake Activity and Earthquake Risk around Alexandria,
GISSAR EARTHQUAKE

Facts and Figures

Date: 23 January 1989
Local Time: 05:11
Magnitude: 5.5
Epicentre: 25-30 km southwest from Dushanbe (USSR).

The earthquake triggered liquefaction and landslides which buried the villages of Sharora, Okuly-Bolo and Okuly-Poen.

Persons Perished: 274
Injured: 73
Homeless: 30,000

<table>
<thead>
<tr>
<th>Intensity</th>
<th>MSK-64 scale</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epicenter</td>
<td>VII</td>
</tr>
<tr>
<td>Gissar</td>
<td>V-VI</td>
</tr>
<tr>
<td>Dushanbe</td>
<td>V</td>
</tr>
<tr>
<td>Tursunzade</td>
<td>V</td>
</tr>
<tr>
<td>Nurek</td>
<td>III-IV</td>
</tr>
</tbody>
</table>
Five shallow foreshocks \( (K = 5.7) \) had occurred in December 1988. From January 23 to February 26, there were 348 aftershocks with \( K < 9 \) (\( M \leq 3 \)), and one \( K > 10.6 \) (\( M \approx 3.7 \)). Then the daily number of aftershocks dropped. The earthquake occurred in the seismic zone of the Iliak deep fault within the junction zone between two large mountain formations, South Tien Shan and the Tajik Depression. The state of stress for the Iliak fault generally corresponds to that of the entire area of Tajikistan. The main axis of compression has a EW direction and the axis of extension is northeast.

Both axes are almost horizontal. The planes of maximum shear are almost vertical, one of these coinciding with the dip plane of the Iliak fault which is nearly vertical. The structure of this area involves three blocks separated by tectonic breaks: the Gissar-Alay uplift (the southern part of South Tien Shan), the Sub-Gissar basin, and the Tajik Depression. The Gissar-Alay consists of slopes composed of primarily Paleozoic formations among which granitoids are widely developed. Folded structures in the Gissar Valley are less complicated than those in the Gissar-Alay region. The Paleozoic foundation of this basin also occupies an intermediate hypsometric position. The structure of the Gissar Valley was primarily formed during the neotectonic stage of development. To the south of the sub-Gissar basin lies the Tajik Depression. Practically the whole of the area is covered by Mesozoic-Cenozoic sediments 10-15 km thick.

The northwestern area of the Tajik Depression includes the edges of the ridges of Babatag, Aktau, Karshitau, and Abdulaaka. The common feature of almost all large ridge anticlines are longitudinal faults in the base of the slopes. The analysis of general data on the geologic structure of the three identified blocks in the region under discussion leaves no doubt as to their condition. The deep fault in the Gissar-Kokshal (South Gissar) sediment fold structure of the Gissar-Alai should be noted.

The main earthquake generating zone of Tajikistan is associated with the Gissar-Kokshal fault, where earthquakes with \( M = 7.5-8.0 \) can occur. Along the southern slope of the Sub-Gissar Depression, can be traced the Iliak fault, which is the western branch of a large infringement structure extending through the Surhob-Vahsh river valleys. The fault is characterized by a steep southward dipping plane with a small ‘cap peak’. The displacement amplitude along the basement surface is as large as 2.5-3.0 km in the eastern part of the Gissar Valley. Geophysical and seismological data show the significant depth of the Iliak fault pen-
Eruption is about 25-30 km. There are areas of less active faults in the region, which were mainly formed during neotectonic time. Reverse faults are dominant between the faults of the Gissar-Alay region and the pre-Gissar basin.

In the epicenter zone of the Gissar earthquake, the Bobatag overthrust is contiguous in the SSW with the Iliax fault. It is worth mentioning that the sub-Gissar basin has been experiencing a considerable compression in the north/south direction. The lower depths (30-35 km) of the Moho are confined to the central part of the Tajik Depression. In the peripheral parts there is a deepening of the boundary down to 45-55 km. The character of release of energy E accumulated in the crust is determined by the Benioff graph of the Iliax fault for the period 1960-1989 for earthquakes with K ≤ 7 and K ≤ 9. Each point on the graph represents the total strain energy released during a 30 day period.

In the period January 1988 - January 1989, an anomaly of local geomagnetic field variations preceding the Gissar earthquake was registered by magnetometric stations. The field variations were found by subtraction of synchronous data for lines Komsomolabad, Sultanabad, Jerino, Chujiangaron, and Shaartuz. The measurements were carried out with an accuracy of 0.5 nT. Some 'bay-like' anomalies were registered in the geomagnetic field variations. According to seismic station data from Sultanabad, Chujiangaron, Jerino, Gissar the anomalies in the local geomagnetic field variations appeared as early as March 1988. The largest anomaly values were observed at the seismic station Sultanabad (8nT), Chujiangaron (4nT), Jerino (2.5nT). The anomaly values recorded at the stations Sultanabad and Chujiangaron were nearly double the largest anomaly values registered since the stations began operation.

For technical reasons the Chujiangaron station functioned only until October 1988. A linear extrapolation shows that variations in the level recovery to the background values could have occurred by the time of the Gissar main shock. The long period duration T registered at the Sultanabad, Chujiangaron, Gissar, and Jerino stations is about 300 days; i.e., together with the empirical relationship dependence of the area under discussion, \( \lg T = 0.73 - 1.54 \) is characteristic for the tectonic magnetic effect of a local earthquake having the magnitude of \( M = 5.5 \). According to this, the radius can be assessed as \( \lg R = 0.35 \text{ m} + 0.04 R = 90 \text{ km} \).

From the end of December at the Jerino station (30 km north of the epicentre), there began an anomalous extension of rocks in the north-south and east-west directions. From 29 to 31 December the increment of extension was: N-S direction \( 5 \times 10^7 \), E-W direction \( 1.5 \times 10^7 \) direction. The compression continued up to January 20 and was equal to \( 3 \times 10^7 \) (N-S direction) and \( 4 \times 10^8 \) (E-W direction). After January 20 a tendency to a reversal of extension sign appeared. On January 23 at 23.15 (GMT), there was a main shock with \( M = 5.5 \). After the main shock deformation stabilized at a new level.

The anomalous extension began on January 5 and ended on January 23. The anomaly amplitude at Shaartuz station was less along the N-S direction and twice as small along E-W direction than those at Jerino. At the Karasu station (15 km west of the epicentre), the de-
formation anomaly manifested itself as an extension phase from January 5 until the main shock occurred, after which it then stabilized at a new level. Residual extension was $2 \times 10^8$ At the Langar station (50 km south of the epicentre) the deformation of compression was recorded in both components which was $4.5 \times 10^7$ in the N-S component and $6.5 \times 10^7$ in the E-W direction.

In January 1989, at the Dushanbe station (20 km northeast of the epicentre), a deformation of tension in the N-S and E-W components was observed. A decrease in the deformation rate in N-S was observed on January 17, and in E-W on January 19. From January 31 the rate decreased in both components to the minimum value. At the Gissar station (nearest to the epicentre), the anomaly and its duration was equal to those recorded at Jerino. Tiltmeter data displayed the same character of processes as the deformation. It should be pointed out that at the five nearest stations (up to 120 km), short-term anomalies of horizontal deformations and earth tilt preceded the earthquake. Those anomalies were not important in the prediction of the earthquake.

Numerous cases of anomalies in the course of crustal tilt and deformation were observed before the occurrence of local earthquakes of $M = 5$, as well as before some of the large deep focus Pamir-Hindu Kush earthquakes of $M = 7$, occurring at a distance of about 250 km. Thus, short term anomalies are not always synchronous at different stations, and do not necessarily correlate with the epicentral distances.

Before the earthquake in Tajikistan, an anomaly in the equilibrium state in the atmosphere temperature and pressure functions was often observed, while under general conditions this state is subject to the Klaiperon-Mendeleyev law. Before the occurrence of large earthquakes, the pressure level becomes lower, and the stronger the coming event the longer the period of anomalies. Before an earthquake of $M = 6-7$ the anomaly duration can be as long as 6-8 months. Anomalies propagate at a high rate within large areas (up to 300 km across), which is why their manifestation can be registered at all the nearest weather stations almost simultaneously.

Nevertheless, due to the high dynamics of atmosphere, and some uncertainties in functions (about 5 mBar), it is difficult to identify the sequence of anomalous beginnings at different stations. As we had previously assumed, the drop in temperature and pressure appears to be the result of atmosphere ionization in the area caused by some physical field of unknown origin. The Gissar astrophysical observatory, located 3 km away from the Gissar earthquake, registered the atmosphere ionization level some days before the first shock.

In the area of the Dushanbe geophysical test site, (mainly in the north at stations Shaambary, Almasy, Khodja-Obi-Garm, Yavroz, Zajron, Obi-Garm), observations of the content of (He, H) and some other components in wells were carried out. The most significant response to the Gissar earthquake preparation was from the nearest geochemical stations such as Almasy, Zajron, and Yavroz. At some stations - Yavroz, Almasy, Shaambary, Zajron - bay-like anomalies of the elemental content were observed.
Similar results for several, but not all, stations were also obtained for the content of Co, He, radon, etc. The earthquake resulted in significant material damage to some settlements located in the epicentral area. Maximum damage occurred in Sharora, Okuli-Bolo, Okuli-Poyon, Karapuchak, and Pervomaj, where the intensity reached VII and a half (on the 12-grade MSK scale). The destructive effect of the shock was aggravated by two mudflows originating from the Urtaboz ridge slopes. The flow moved to the Sharora settlement, and the villages Okuli-Pojon and Okuli-Bolo. Part of Sharora and Okuli-Bolo, and the whole village of Okuli-Pojon, were buried under debris 15 meter thick. The thickness of the mud reached 20-30 m at some places. The length of the area covered by the mudflows was 5.2 km in W-E direction, while the width was 1.2 km.

In the settlement Sharora, 67 houses were fully destroyed and buried under the mudflow, and 207 people perished. The adobe houses suffered maximum damage; in most of them, portions of walls and roofs collapsed. In many brick buildings, gaping fractures appeared and chimneys were broken. Four-storey residential buildings, as well as the three-storey building of the Agricultural Institute, were severely damaged. The horizontal fractures covering these buildings at ground level and first floors, along with cross-like fractures in the carrying walls, testify to their proximity to the earthquake focus. But it should be noted that these buildings were constructed without any antiseismic measures and the building standards had been violated.

In the village of Okuli-Bolo, 108 residential houses collapsed and 67 people perished. The remaining buildings in the village were severely damaged. In the epicentral zone, the electricity and water supplies, and telephone installations broke down; bridges and 40 km of motor routes were damaged. According to witnesses, the earthquake was preceded by a strong underground rumble. It was succeeded by a sharp vertical shock, which was followed by horizontal east-west oscillations.

The earthquake caused landslides and soil liquefaction, adding to the number of lives lost. It was the first time such a phenomenon, due to a relatively small earthquake, had occurred in Tajikistan. Landslides had not been observed before the earthquake. The soil here is light loess-like loams 60-100 m thick. The loams are subsidence-prone, belonging to the second type of subsidence. The physical-mechanical parameters are the following: natural humidity 6-12%; density 2.68 T/m³; volume mass 1.67 t/m³; porosity 45-54%; the porosity coefficient 0.9-1.2; the internal friction angle 25-27°; adhesion 1.4-1.8 MPa. Such was the situation up to 1968. Thereafter, irrigation channels were constructed to irrigate fields where mainly cotton was cultivated.

The investigation carried out after the earthquake indicated that the natural humidity of the soil increased by 20-27%. According to the drilling data, the moisture distribution within the sequence of loess-like loams is as follows:

- Four to six meters from the surface, the soil is practically dry with a humidity of 6-17%;
- Up to 30 m further on, the soil is water-saturated with a viscous-plastic consistency which under mechanical effect (e.g. drilling) turns to a liquefied, mud-like state;
Tight-plastic loams are found at a depth of more than 30 m, which gradually turn to practically a dry state at a depth of 40-50 m. Such conditions can be observed all over the area. The drilling data proves that water saturation at the bottom is absent.

The cause of landslides during the earthquake was the long-term irrigation of the territory and the ploughing of slopes which produced a local water saturated horizon of very low strength parameters which triggered the sequence of loess-like loams. The earthquake acted as a trigger for these phenomena, which can be called 'earthquake-generated'.

Conclusion

The analysis of the Iliak fault seismicity and some geophysical fields has shown that before the Gissar earthquake of January 23, 1989, a long term prediction (10 years ahead) and a medium term prediction (1 year ahead) had been made, which is why it was not unexpected. Nevertheless, the possibility of soil liquefaction in the epicenter of this earthquake, and the mudflows which caused most of the deaths and damage, was not predicted. There were some short term precursors (up to 1.5 month), such as deformation and tilt of the ground surface, drastic atmospheric and pressure changes, atmosphere ionization, and changes in some geochemical parameters. At present, short-term anomalies are only relatively reliable because they are not intensive and convincing enough. In addition, short-term earthquake prediction methods, based on several types of data, along with their criteria and uncertainties, are still imperfect. To increase the reliability of short term and immediate earthquake prediction, it is necessary to organize a stable continuous collection of data and processing of different geophysical data.
GISSAR EARTHQUAKE, 1989: SEARCH AND RESCUE OPERATIONS AND EMERGENCY MANAGEMENT

by

F. Niyazov

(USSR)

The Gissar earthquake took place at 5.11 am (local time) January 23, 1989. From the point of view of the time factor above, the earthquake was unfavourable: it was early morning and the population was asleep.

During the first few hours after the earthquake in the Gissar region, the republican authorities formalized the following objectives to mitigate losses from the earthquake:

(a) Assess the nature and scale of the damage;
(b) Organize search and rescue operations;
(c) Render medical aid to injured and evacuate them into Republican hospitals;
(d) Carry-out recovery operations on municipal and power supply networks;
(e) Save and protect public and private property, preserve law and order in the disaster area;
(f) Provide population with shelter, food, and water;
(g) Monitor the epidemiological situation in the earthquake area;
(h) Organize traffic in the disaster area;
(i) Co-ordinate the work of management bodies.

Among all the objectives of the recovery process the most important were search and rescue operations and resource control. Following the decision of the Chairman of the Council of Ministers of the Republic (Chief of the Republican Civil Defense), about 2000 persons and 500 units of equipment of different functions were involved in rescue and recovery operations. From the point of view of specialization, there were teams for rescue, for mechanical operations, for reconstruction of roads and bridges, for repair of power supply networks and communication lines, for transportation and logistics, for distribution of material supply, for medical aid groups, and others.

In the disaster area there was a need for considerable relief forces from republican and local bodies of the Ministry of Internal Affairs to preserve law and order, and to control traffic in the area. People crushed by badly damaged buildings were killed instantly. Search and extrication of these dead from under the debris was at first impossible, and later was simply inexpedient. In places where great loss of life occurred, memorial cemeteries were organized following the decision of the Republican Government and the consent of the local population.

In the first days after the earthquake, personnel worked day and night in 2 to 3 shifts. During the period of search, rescue and recovery operations (about 14 days) rescue teams were
provided with three free hot meals a day at the expense of the Council of Ministers of the Republic financed by funds for assistance to the injured. Military units of the Soviet Army from local garrisons were sparingly used for carrying out search, rescue and recovery operations, since the situation in the disaster area did not require enlisting extensive military personnel.

The management bodies in charge of organization and control of population rescue and recovery were:

- The Commission of the Bureau of Central Committee of the Communist Party of Tajikistan (Governmental Commission) headed by the Chairman of the Council of Ministers of the Tajik Soviet Socialist Republic;
- The Republican Permanent Emergency Commission (RPEC) on the elimination of consequences of large industrial accidents, catastrophes and natural disasters; the Council of Ministers of the Tajik SSR headed by the First Deputy of the Chairman of the Council of Ministers of the Tajik SSR;
- The operation group of the Civil Defense Board of the military district;
- The Permanent Emergency Commission of the Gissar Region headed by the Chairman of the Regional Executive Committee of the Soviet People's Deputies (regional earthquake recovery headquarters);
- Recovery Headquarters in three different locations of the disaster area under the leadership of members of the RPEC and Chairmen of Rural Soviets;
- Headquarters of Civil Defense Departments of Republican Ministries and government departments of the republic under the leadership of Chiefs of Departments (members of the RPEC).

The Civil Defense Headquarters of the Republic operated as a working body of the Republican Permanent Emergency Commission and co-ordinated the activities of all management bodies. The Governmental Commission and Recovery Headquarters were established in the first hours after the earthquake in the disaster area. All the other management bodies were quickly established along the lines of existing plans on eliminating the consequences of industrial accidents, catastrophes and natural disasters, and began their activities immediately after being notified of the earthquake.

The Governmental Commission conducted the co-ordination of operations at the republican level. The republican PEC was the main executive body for planning and organizing. The main purpose of the RPEC was to organize operations to mitigate earthquake consequences and to provide assistance to Republican Ministries and Departments, the PEC of the Gissar region, local headquarters and others, ensuring the readiness of management bodies and forces and controlling activities.

The RPEC as an executive body carried out the following tasks:

- Ensured the readiness of subordinate and enlisted management bodies, forces and resources, and the control of their activities;
• Assessed the situation and scale of the earthquake, the extent of damage, the forecast of consequences, etc.;
• Controlled evacuation operations of population to safe areas;
• Ensured life support measures;
• Controlled operations to save cattle and reduce rural damage to ecological consequences of earthquake;
• Established rules for entering and remaining in the disaster area, organized general measures to preserve law and order;
• Co-ordinated with military authorities;
• Provided information about the situation and reported results of recovery operations to the Bureau of the Central Committee of the Communist Party of Tajikistan, the Governmental Commission, military authorities, organizations concerned, and the population;
• Organized work of mass media, informing the population, etc.

The operational group of the Civil Defense Board of the military district provided co-ordination with military authorities and enlisted military units with the RPEC and the Republican Civil Defense Headquarters. In the first place Republican Ministries, Departments and working bodies should operate under the direct leadership of their ministers who are members of the RPEC. Their tasks were specific and corresponded to their normal activity. The permanent Emergency Commission of the Gissar region operated under the direct leadership of the RPEC solving problems similar to tasks of the RPEC within its competence.

The main objective of activities at regional headquarters was to collect and assess information on practically all the questions which arose during operations (assessment of the scale and consequences of the disaster, information on operations of the forces involved, problems in conducting rescue operations, evacuation of the population and injured, providing medical aid, life support systems, logistics, etc.) in order to make the best decisions for conducting operations and to make rational use of available resources. It should be noted that this information was also received from other institutions (ministries and departments, military authorities, etc.). In the interests of recovery activities, three services were created: a political information centre; a dispatch service; and a logistics service.

The implementation of decisions by higher management bodies was carried out by recovery headquarters at 3 levels: Kishlak (a village in central Asia), the Soviets of People's Deputies and representatives of the Republican Permanent Emergency Commission. The organization of search and rescue operations and the most effective forms of operating management bodies cannot be the same for all emergencies. It is necessary to use a creative approach in finding ways to solve problems for each emergency situation.
GISSAR EARTHQUAKE, 1989 - ECONOMIC ANALYSIS
by
S.V. Kozharinov, F.G. Garvrikov
(USSR)

The catastrophic consequences of the relatively small (M = 5.5) January 23, 1989 Gissar earthquake can largely be explained by the peculiarities of its manifestation. The epicenter was located near the village of Sharora, 15 km from Dushanbe, the capital of the Tajik SSR. About 30,000 inhabitants became homeless due to the collapse of, or severe damage to their houses. Many school buildings, hospitals, kindergartens, industrial and agricultural structures were severely affected, and 120 km of electric power lines, 21 km of communications lines, 17 bridges and 54 km of roads were damaged or destroyed.

The severest damage occurred in the villages of Sharora, Okulibolo and Okulipoyon due to landslides in the vicinity of these settlements. In Sharora, the earthquake caused a landslide 2.45 km long and 1 km wide. Under soil mass 22-25 km thick, 67 houses were buried, and practically all the inhabitants perished. Search and rescue of victims started as early as the first hour after the earthquake, resulting only in the rescue of a few persons at the front border of the landslide. Very often one could not even find debris of previously-existing structures, for they had been moved from one place to another by the landslide.

At the border of the mudflow were some buildings made of baked bricks, which were seriously damaged due to lateral ground pressure although some of them withstood the shock. Many one-storey brick buildings in Sharora received fractures in the walls; chimneys fell down and joints between reinforced concrete floor plates opened. Damage to buildings was significantly less further away from the landslide zone. On inspection, damage of degree 2 to 3 was found in three 4-storey buildings of the 1-401 TTZh type, designed to withstand an intensity of VII-VIII, located approximately 300 m away from the front border of the landslide. Within the buildings, partitions partly fell down, joints in floor plates opened and in some transverse walls, perpendicular to the landslide, inclined fractures appeared. Damage was significantly higher in one of the buildings which was in an emergency state due to the particularities of its foundation soil. On the first floor, concrete framing of windows and door apertures situated in the longitudinal external walls were cut away, bricks were broken in separating walls, and large parts of plaster fell down everywhere. Inspection revealed a poor quality of construction work in all three buildings.

Inspection of the 3-storey administrative building of the Research Institute of Agriculture, built of baked brick and having no antiseismic strengthening (which should have been made according to the Codes now in force), revealed horizontal fractures along all the building perimeter at the level of the ground floor and the first floor, together with damaged piers of the carrying walls by cross-like fractures.
Another landslide of the mudflow type formed on the southern slope of the Urtabaz ridge and developed over an area with comparatively gentle slopes, 5-7° its length was 3.8 km, its width 1.2 km and it involved 10 million m³ of soil.

In the village of Okulibolo the landslide mass covered and destroyed some of the buildings situated near the foot of the Urtabaz ridge slope. The ground seismic oscillation caused the most severe damage to residential houses and various ancillary buildings. 108 residential houses were destroyed and 67 persons perished. Some houses which withstood the earthquake were no longer habitable. The most severely damaged houses were older structures in which full or partial collapse of walls and roofs could be observed. In residential houses of recent construction (1-2 years old), many vertical and inclined fractures and partial collapse of building corners, were found.

Thirty-four minutes after its origin the mudflow of the Okuli landslide reached Okulipoyon, located 5-5.5 km from the upper border of the stripping of the earth mass. The liquid, plastic mixture of loess and water gradually overflowed the majority of buildings in the settlement of Okulipoyon. Fortunately, the inhabitants were awakened by the noise of the moving mudflow, and left their houses. All the inhabitants on the upland slopes of the valley saved themselves. At the settlement of Okulipoyon, 68 residential houses were buried under the mudflow. In buildings outside the mudflow, no structures collapsed. In some walls of residential adobe houses, slight damage was observed, such as a few cracks 1-2 mm wide. It should be noted that Okulipoyon is located 5-6 km away from the earthquake epicenter.

In addition to the damage to residential houses and traditional buildings, within the zone of the landslide body electric power lines, irrigation systems, roads and other objects were destroyed. The landslide generated by the Gissar earthquake was largely the result of human activity: the form of the upper edge of the Sharora landslide practically mimics the path of the irrigation canal which was laid upon the upper part of the hill near the settlement of Sharora. Furthermore, the foot of the slope was cut away during road construction. Within the territory of the Okuli landslide, a dense irrigation network was to be found and intensive irrigation of cotton fields had been carried out. Favourable conditions for the Okuli landslide formation were partly created by the existence of a tight plastic clay aquifer found at a depth of several tens of meters when an engineering geological study was carried out in the area after the Gissar earthquake.

The main destruction of objects due to the Gissar earthquake were observed in the epicentral area of the intensity VII of about 25 km². Within the intensity VI area of about 100 km², mainly adobe houses were damaged, while within the intensity V zone (2000 km²), similar houses received only slight damage (fracturing). During the earthquake, a ground acceleration of 0.125g was recorded at a point located 2-3 km away from the epicentre by the Strong Motion Service of the Institute. In the Dushanbe area observational points operating under different engineering geological conditions registered acceleration changes from 0.025g to 0.1g.
An analysis of results of the 1989 Gissur earthquake allows us to draw the following conclusions:

(a) The earthquake caused mass destruction of adobe buildings and significant damage of 1-3 storey brick buildings having no antiseismic strengthening within the epicentral zone. Buildings with carrying brick walls strengthened with reinforced concrete and designed to withstand a seismic excitation of intensity VII-VIII had damage of degree 3. These buildings can be restored for further utilization. Buildings having non-uniform foundations were seriously damaged.

(b) The intensity within the epicentral zone, determined from the character of the degree of damage, corresponds to VII on the seismic intensity scale MSK-64. The magnification of the seismic effect in some parts of the epicentral zone can be explained by unfavourable geological conditions and is related to the development of landslide phenomena in water-saturated soil.

(c) The catastrophic consequences of the earthquake point to the necessity of correcting the actual practice of economic development on landslide-prone slopes in seismic areas. Territories for future construction should be chosen taking into account the stability in the course of economic development.

(d) Within earthquake-prone areas, the control of Building Requirements for buildings made of local materials should be improved. The construction of buildings with walls of clay having no strengthening frames, braces, rings, etc. should be discouraged.
SPITAK EARTHQUAKE

Facts and Figures

Date: 7 December 1988
Local Time: 11:41
Magnitude: 7.0 Richter scale

- Three consecutive shocks (M 7.0) during 30 sec., another shock (M 5.8-6.5) 4 min 20 sec. later.
- The main epicenter coordinates: 40°58 N 44°28 E.
- The depth of the epicenter: 8-10 km.
- The maximum vertical displacement: 180 cm.
- The maximum horizontal displacement: 50 cm.
- The Intensity (MSK scale) felt in cities:

<table>
<thead>
<tr>
<th>City</th>
<th>Population</th>
<th>MSK</th>
<th>Distribution Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spitak</td>
<td>25,000</td>
<td>IX-X</td>
<td>100%</td>
</tr>
<tr>
<td>Leninakan</td>
<td>220,000</td>
<td>VIII-IX</td>
<td>75%</td>
</tr>
<tr>
<td>Kirovakan</td>
<td>171,000</td>
<td>VII-VIII</td>
<td>25%</td>
</tr>
<tr>
<td>Stepanavan</td>
<td>21,000</td>
<td>VII-VIII</td>
<td>67%</td>
</tr>
</tbody>
</table>
In estimating the heavy consequences of the Spitak earthquake, one should take into account numerous factors which influenced the behaviour of the buildings: characteristics of excitation, quality of design and construction, state of the foundation, main design features, and their location with reference to the direction of seismic waves. We should concentrate on the first three factors mentioned above since they were the most important.

The first group of buildings may be classified as "B" category in the MSK scale. Eighteen residential blocks, containing 642 single and two storey stone buildings of a similar type were inspected in Leninakan, in areas with different engineering geological conditions. In each residential block, the amount of average damage was used to estimate intensity values as follows: VII for 2 blocks (40 objects), VIII for 7 blocks (302 objects), and IX for 7 blocks (300 objects). Thus, the earthquake had different effects on buildings of a similar type according to their engineering geological conditions.

Most of the buildings of the second type collapsed, in particular those in which the continuity of carrying walls in plan was interrupted. Furthermore the following typical damage occurred: partial collapse of floors and walls, fractures of reinforced concrete belts and vertical structural members followed by buckling of reinforcement, segregation and buckling of masonry in between wall joints of carrying walls. Classification of the damage and an engineering analysis of the data has shown that 53% of the buildings inspected had a damage level of 4, the level was 3 in 37% of the buildings, and 2 in only 10% of the structures.

The tragic consequences of the Spitak earthquake were mostly connected with the mass collapse of modern buildings, mainly nine-storey buildings of the III series. Reinforced concrete frames of this series have the following significant drawback: a built-up frame is made of linear structures, connection is made during construction, with welded joints being located in zones of peak force values. This is aggravated by the design, that is, the buildings have different stiffnesses in lateral and longitudinal directions; in the lateral direction there are stiffness diaphragms in the buildings. Fifty percent of the buildings in Leninakan collapsed, the other 50% were damaged at levels 3 and 4. According to the results of an engineering analysis of the buildings, design drawbacks played a part in the tragedy but the main causes were exceedance of the expected excitation and the poor quality of construction.
Lack of space prevents a detailed engineering analysis of earthquake effects for other structural systems, but a brief review of some results of the analysis for frame panel systems is given below.

To estimate the behaviour of frame panel buildings (series III) during the earthquake, calculations were made on the basis of theoretical elastic ductile models for movement of the foundation, based on the accelerogram of the Spitak earthquake recorded at the station in the town of Gukusyan. A time-frequency analysis of seismic waves revealed changes in the spectral content of the earthquake. Then corrections of intensity records were made in order to make it possible to analyse the whole range of intensity values from the one recorded to intensity IX. The calculations were made by the investigation program "RUPS". A building was regarded as a multi-mass non-stationary inelastic system which changes its characteristics with the accumulation of damage. The model allows one to analyse physical processes in the behaviour of reinforced concrete frame buildings under intensive seismic excitations.

According to the integral estimate, the earthquake intensity exceeded the expected values in the map of seismic zoning of the USSR: at Spitak by 2, at Stepanavan and Kirovakan by 1, at Leninakan by 0.5-1.

Since no instrumental records were made in the epicentral zone, therefore a macroseismic investigation was the only method allowing one to estimate intensity. Practically all settlements in the epicentral zone were destroyed or heavily damaged. Heavy damage was inflicted on the village of Nalband: monuments, railway road and fences, telegraph posts and wires, significant displacements (up to 21 cm) of railways, and fractures of gas pipelines were recorded. An analysis of the total macroseismic effect on the surface in the epicentral zone, reveals the seismic intensity as IX or more. Fifty percent of the buildings in the stricken area were buildings with bearing stone walls. The dominance of stone buildings in Armenia is due to the availability of the building material, tuff.

All the stone buildings inspected can be classified into two groups: old (1 to 3 storey) buildings and modern (4 to 5 storey) buildings with reinforced concrete elements or reinforced concrete frames and precast reinforced concrete frames. Parameters of the buildings and estimates of their vulnerability during the earthquake were collected.

The results of this theoretical analysis are as follows: in Spitak and most of the Leninakan territory earthquake resistance of 9 storey frame panel buildings, designed for intensity VIII, in areas with intensity IX had no extra resistance strength. Furthermore, earthquake resistance was not completely ensured in areas where the intensity was VIII or more (the design intensity for the systems), and only those buildings which were favourably oriented North/South withstood the shocks. This was characteristic of certain areas in the town of Leninakan. Lastly, in areas with intensity VII and VII and a half, the earthquake resistance was ensured in buildings of series III (built without glaring violations). This situation is typical for most of the territory of northern Armenia.
The main causes of heavy consequences of the Spitak earthquake were:

- Exceedance of expected seismic excitation;
- Underestimation of the seismic hazard;
- Poor quality of construction;
- The use of designs which do not meet the requirements of building codes in earthquake-prone areas.
The Spitak earthquake of December 7, 1988 attracted the attention of scientists and experts because of its extraordinary consequences. Its main characteristic was its comparatively high intensity. Earthquakes with magnitude 7 have not been registered in that area during the period of instrumental observation. However, besides the high intensity, the earthquake had a number of other noteworthy features as follows:

**Intensity of the Earthquake**

Unfortunately, there is no data on ground vibration at the epicentre. Therefore, it is necessary to estimate the intensity of the earthquake at the epicentre by its magnitude and source depth. Several experts have suggested formulas to estimate the intensity which vary significantly: N.V. Shebalin: 9.4, L. Esteva: 11.3, and a team of Caucasian scientists: 10. The data proves that the intensity of the earthquake in the epicentral zone was at least X. By analogy, estimates can be made for peak values of accelerations, velocities and ground displacement, and even values of the response spectrum. Thus, for the peak ground accelerations in the epicentral zone with $M = 7$ are 0.74 g, according to the Esteva and Villaword’s formula, and 0.8 g according to the Davenport’s formula. It can be stated that the peak acceleration in the epicentral zone exceeded 0.6 g. Similar estimates for the town of Leninakan report an intensity of IX and peak acceleration of more than 0.4 - 0.5 g. Reliability of the intensity level in Leninakan is verified by direct instrumental recordings made with a seismometer with 8% damping of the peak value and with $T = 0.25$ sec period. The records were obtained by the experts of the Institute of Geophysics and Engineering Seismology, and the Armenian Academy of Sciences. Pendulum amplitudes at three stations in Leninakan were 18 mm, 15 mm, and 13 mm, which corresponds to XIII, X, and X intensity according to the MSK-64 scale. In the Gukasyan settlement, ground accelerations recorded by an accelerograph reached 0.2 g. Peak accelerations in Erevan (80 km from the epicentre) reached 61 cm/sec$^2$ and ground displacement was 3.5 mm.
Duration of the Earthquake

The main characteristic of the Spitak earthquake was the length of its duration. It is known that 4 min 20 sec after the main shock another shock with $M = 5.6 - 6.5$ occurred. In reality, as was shown by acceleration records made in Erevan and by seismograms from Grafschaber (FRG), there were at least two further shocks between these two main shocks. Thus the Spitak earthquake lasted more than 4-5 min. This is a very rare anomaly in the history of seismology.

Effects on Buildings

The main cause of the mass destruction of buildings during the Spitak earthquake was its higher intensity compared with its design (before the earthquake the intensity at Spitak was considered to be VII, at Kirovakan VII, at Stepanavan VII and at Leninakan VIII). According to the testimony of eye-witnesses, it was the aftershocks which destroyed the majority of the buildings. A similar phenomena also occurred during the last earthquake in San Francisco in October 17, 1989, when some buildings which seemed to have successfully withstood the main shock began collapsing during the following aftershocks. In addition, certain buildings suffered from unfavourable combinations of ground conditions, the dynamic characteristics of the buildings, and poor quality of construction. Nine-storey frame-panel buildings in Leninakan suffered the most damage from the earthquake. As well as poor design and construction, the low strength of the columns (section 40x40cm), of buildings were also subjected to a seismic effect which increased significantly because of the coincidence of the periods of free vibration of buildings with ground vibration periods. The numerous experimental measurements of the vibration periods of many of the buildings, showed that the values lie between 0.55 - 0.65 sec. Experimental measurements of ground tremors made by Japanese experts after the earthquake, showed that prevailing vibration periods of ground lay between 0.50 - 0.65 sec at Leninakan, between 0.25 - 0.30 sec at Kirovakan, and between 0.25 -0.40 sec at Spitak. Thus, taking into account the long duration of the earthquake, one may suppose that the 9-storey frame-panel buildings in Leninakan suffered a significant increase of seismic effect due to resonance. Records show that at Leninakan (by pendulums of the Nazarov seismometer with 0.6 - 0.8 sec period) there were 60-80 cycles with peak amplitude during the earthquake. This fact also suggests that a resonance phenomenon affecting 9-storey frame-panel buildings leads to their destruction due to the so-called 'short-cycle fatigue'.

Lessons from the Spitak Earthquake

The general public, and even experts believe that the problem of protecting the population against earthquakes mainly concerns the prediction of place, intensity, and time of the coming
earthquake. Undoubtedly, a successful time prediction of destructive earthquakes will reduce the number of victims and the level of material loss. But the most precise prediction cannot prevent the destruction of poorly designed and constructed buildings. If only some portion of global investment and effort spent on earthquake prediction were invested in the improvement of earthquake-resistant design, the negative consequences could be minimized. Therefore, reliable earthquake-resistant design is the only method of successful protection against an earthquake. Attention should be paid to the behaviour-predictions of buildings during earthquakes. This will make it possible to improve the reliability of buildings at a minimum cost.

Analysis of the consequences of a large number of destructive earthquakes and world experience of earthquake-resistant design, show that earthquake-resistance of multi-storey flexible buildings is higher when they are constructed on the hardest rocks; low-storey stiff stone and brick buildings are more earthquake-resistant when constructed on comparatively porous foundation. A general principle of design, supported by the majority of experts, is as follows: a building should be stiff in plan but flexible. But low-storey buildings are preferable since they provide better possibilities for the rapid evacuation of people, and are safer. However, business institutions can be located in multi-storey buildings with higher earthquake-resistance, if metal frames are used. Limited land resources should be taken into account in seismic areas. It should be emphasized that concrete structures, especially prefabricated ones, are at a disadvantage in relation to metal frames. Underground buildings are quite safe during earthquakes; shops, canteens, cafes, service centers can be located in them. But, hospitals, schools, kindergartens, trade centers, and other buildings where people concentrate, should have a higher grade of earthquake-resistance. Most of the current standards of earthquake-resistance, should be revised in particular the values of dynamic coefficient, levels of torsional and vertical seismic effects, category of ground, storey index, and anchoring of partitions. The value of permissible damage coefficient needs significant correction.

The Spitak earthquake showed very clearly that it is necessary to stick to the generally known principles of earthquake-resistance, which include symmetric design schemes, uniform distribution of stiffness and masses, monolithic jointing of floors to walls, close location of columns, continuity of walls from top to ground, openings limited in number and size. Much of the structural damage during the Spitak earthquake was caused exactly by the violation of these simple principles; mainly by the poor joining of carrying members and the complete absence of monolithic elements in between-storey floors; and by torsional and vertical effects which were not taken into account in the design.
EMERGENCY MANAGEMENT AND RELIEF EXPERIENCE FOLLOWING
THE SPITAK EARTHQUAKE (1988)
by
B. Chernichko
(USSR)

The nuclear threat still exists in today's world. In addition, there is the increasing pressure of global problems engendered by the negative results of social, scientific and technological progress. One of these unresolved problems is the so called ‘ecological imperative need’, associated with the challenge for mankind to survive in a deteriorating environment. There is another imperative need no less important and urgent, i.e. the safety of human life in the face of multiplying man-made and natural catastrophes. Experience gained in this field shows that it is possible to significantly reduce losses due to natural disasters by organizing effective emergency management. Many countries have recognized this imperative need to protect human life and reduce the damage inflicted by natural and industrial disasters, and have organized coordinated nation-wide response-and-management systems to be operated during peace time. Such a system is also being created in the USSR under the auspices of the State Commission on Emergency Situations of the USSR Cabinet of Ministers.

The experience gained in mitigation of the Spitak earthquake, as well as many other similar events, convincingly points to the need for developing management methods in cases of emergency, thus forming a new sphere of state and public management. An emergency should be understood as a set of conditions, circumstances and factors resulting from a catastrophic man-made or natural phenomenon that has produced loss of human life and economic damage, and, it should be stressed, has disrupted the way of life in the zone of emergency. From this, it follows that a complex emergency management system should: involve many spheres of human activity; have a complex hierarchical structure; be based on functional principles; and finally, that its effective implementation is only feasible through central and local executive agencies.

The question of the purpose of management system is an important one. Management in emergencies during peace time should envisage the achievement of the following strategic aims:

- the maximum possible, and practically realizable, reduction of all, or any, risks;
- the reduction of disaster impact (hazard to human life and health, material losses, damage to the economy, natural resources, the environment, etc.);
- emergency response.

Emergency management implies planning the activities of appropriate agencies to create preparedness for measures which can be broken down into four groups:
1. Measures designed to prevent emergency situations as such and to reduce potential damage to the population and the economy;
2. Measures for the pre-disaster preparation of management systems, task forces, finances and the population for actions in emergency situations;
3. Measures for the organization of rescue operations;
4. Measures designed to mitigate the consequences of emergency situations.

1. The preventive measures of the first group are diverse and not easily summarized. Nevertheless, the most important of these measures is a scientifically-sound determination of likely hazard sources and risk areas. Areas prone to disastrous earthquakes need stable urban systems that are capable of preservation or rapid recovery of function following a natural disaster. It is impossible to achieve this degree of survival capability without special socio-economic and technical measures. It follows that the building codes governing the construction of residential and public buildings in earthquake-prone areas should aim to ensure safety in disaster emergencies.

2. The pre-disaster planning measures of the second group, can be divided into three sub-groups:
   1. preparation of management systems;
   2. preparation of forces and resources;
   3. population training.

   The first sub-group includes:
   • control of the national economy in order to be able to act in emergency situations;
   • training of existing management personnel or the possible rapid creation of a new one which would be able to exercise control;
   • development of specific plans of action in emergency situations;
   • training of managers of various ranks to act in emergency situations;

   The second sub-group includes:
   • a substantive, rational structure for special forces to conduct efficient search, rescue, and recovery work;
   • creation of centralized reserves of special machines, search-and-rescue and repair aids, medicine reserves, survival aides, etc.;
   • systematic check on preparedness to conduct rescue, repair and recovery operations;
   • creation of rapid-response forces and organization of their co-ordination with other forces and services.

   The third sub-group includes:
   • training of the population;
   • training of personnel for action in an emergency;
   • early creation of information and warning systems and systems to control population behavior in emergency situations.
It has been established that efficient control in critical situations requires:

- first, a clear knowledge of, and the ability to use, patterns in the behaviour of different population strata in emergency situations;
- secondly, the ability to forecast and use the socio-demographic and socio-psychological consequences of emergency situations when rehabilitating the injured population;
- thirdly, education of the population (through special social services in hazard-prone areas). Prior to this, the population should be made aware of the need for self-reliance in the case of disasters.

3. The third and fourth groups of measures are associated with the organization of the management of rescue bodies and forces in emergency situations as well as with the mitigation of the impact of disasters, and the recovery of the social and economic potential of damaged areas.

These measures form a basis for the everyday control of emergency situations, in several successive stages, i.e.:

- Stage 1: emergency action;
- Stage 2: everyday planning;
- Stage 3: rescue operations;
- Stage 4: elimination of the consequences of the emergency.

In stage 1, the report of the time, location and character of a natural disaster is disseminated. The members of the emergency control body provide a preliminary assessment of the situation and take the necessary measures to protect the population and aid the injured. This preliminary assessment provides the likely direction and rate of development of the catastrophe: their effect on the activities of the population, the expected number of injured, the forces and machines necessary to locate the disaster areas, the character and assessment of damage and impact, and any peculiarities in rescue operations. This is followed by specifying emergency measures to protect the population, locate the boundaries of the stricken area, and render first aid to the population.

Stage 2, everyday planning, is performed after a detailed reconnaissance, collection and examination of data has provided a comprehensive assessment of the situation, which includes:

- an assessment of the impact of the disaster;
- an assessment of the available forces and resources and their capabilities and preparedness to deal with unavoidable tasks;
- an assessment of the terrain and routes for bringing forces into the stricken area;
- an assessment and forecast of hydrological and meteorological conditions.

This detailed assessment of the situation is used to update the mitigation plans and to make decisions on the organization of necessary operations and their maintenance, as well as the organization of local control and co-ordination.
As rescue operations begin immediately the second and third stages frequently coincide. Control during rescue, urgent and provision operations is to ensure the organization and efficient carrying out of:

- rescue operations (location and extraction of the injured, evacuation of the population from the stricken area and medical assistance to the surviving population);
- urgent operations (to extinguish fires, to strengthen or destroy collapsing structures, to rehabilitate lifeline systems, to provide survival aids for the population and rescue units, to remove or protect valuable material);
- provision operations (reconnaissance and observation, provision for roads, transportation, chemical provision, materials and equipment, engineering and technology).

Stage 4 involves the organization of control, during the elimination of consequences and search-and-rescue operations in the stricken area, over two periods.

The first period involves urgent rehabilitation operations, which aim at creating the necessary conditions for life in the stricken area. These operations involve chemical decontamination, or disinfection, repair or erection of temporary housing facilities, recovery of life-support systems (partial or complete), etc. When this period is over, the emergency situation is considered to have been overcome, because the threat to life and health has been eliminated and the necessary conditions for life are available.

The second period begins with the less urgent rehabilitation operations which aim at the recovery of economic potential and the rehabilitation of social and economic functions in the stricken area. These operations include the construction of new (or reconstruction of collapsed) industrial units; repair or replacement of equipment; reconstruction or construction of new facilities in line with the former infrastructure. It should be noted in conclusion, that any analysis of emergency situation highlights new issues in the field of protection at a time when there is an increasing threat of tragic consequences caused by accidents, catastrophes and natural disasters. Without active international co-operation in this field it will be difficult to resolve such problems and guarantee everyone the right to live in safety.
SPITAK EARTHQUAKE, 1988: ORGANIZATION OF RESCUE AND RESOURCE MANAGEMENT

by

A. Kapochkin
(USSR)

The Spitak Earthquake in Armenia involved 40 percent of its area and 960,000 people. The greatest damage was sustained by 4 towns and 17 districts within the republic. Twenty-five thousand people were killed by collapsing buildings, and 514,000 people were left homeless. Three hundred and sixty-five villages were damaged, 58 being completely destroyed. Over 200 large enterprises were in the disaster area, 157 of these stopped functioning.

The disaster area was extensive, buildings, transportation, communications and lifelines were severely damaged, and many people were killed. All this involved many diverse management bodies and forces which combined in an attempt to alleviate the consequences of the Spitak earthquake. To understand these processes better, we shall consider the situation in the republic in more detail. Practically all buildings and structures were damaged in the town of Spitak, 80 and 33 percent of those in Leninakan and Kirovakan respectively. In Leninakan, the largest of these towns, ninety percent of high rise buildings, 10 percent of five-storey structures, 26 schoolhouses and 11 hospitals totally collapsed. Various degrees of damage were sustained by 120 works, including 38 industrial ones (thirteen of which totally collapsed). The total loss of industrial facilities amounted to over 400 million roubles.

The severest blow fell on the district centre of Spitak. Every other inhabitant was killed and every tenth severely injured. Thirty percent of more than 450 buildings and structures were damaged and the rest collapsed. The town ceased to exist. In the countryside, damage and total collapse affected about 45,000 houses, 83 schoolhouses, 88 kindergartens, 84 medical facilities, 2260 shops, and public service facilities. The loss to agriculture amounted to 2,000 million roubles.

The power system of the Armenian SSR was disconnected from the unified Transcaucasian power system as a result of earthquake damage, although the power supply system in Leninakan and Kirovakan was back in operation three days after the earthquake. The resulting damage completely paralyzed the Kirovakan-Leninakan railway line.

Automobile transportation in this area was paralyzed; as a result practically all businesses in Leninakan and Spitak stopped functioning. There were 12 chemically hazardous facilities in the stricken area; all work at these facilities stopped immediately after the earthquake. Overall, both the radiation and the chemical situation in the areas remained within health standards.

The elimination of earthquake consequences took place in two stages:

- Relief and rescue operations (from December 7 to 31, 1988);
• Reconstruction (from January 1, 1989).

The relief and rescue operations will be examined in more detail. An analysis of these emergency operations shows four main periods which can be identified as follows:

• Initial period;
• Main period;
• Terminating period;
• Transition period (in relation to the reconstruction period).

The initial period was characterized by the absence of a single co-ordinating centre which could organize rescue operations. Operations were carried out by local residents, military units stationed near the settlement in question, fire brigades and local police. Engineering rescue machinery was collected; for example, 13 cranes, 4 bulldozers and 6 excavators had been collected in Leninakan by 18:00 hrs on December 7, 1988. The unsanctioned arrival of great masses of people in private cars trying to get to their relatives created traffic jams and paralyzed the movement of relief equipment and personnel. Overall, the initial period was characterized by inadequate provision of equipment, the absence of objective information on the size of the disaster, the amount and degree of damage, and number of casualties. This period lasted 9 hours in Leninakan, 8.5 hours in Spitak, 5.5 hours in Kirovakan, and 18.5 hours in Stepanavan. About 1400 persons were extracted alive from the debris in these towns, together with about 1700 dead. Approximately, 6,000 m$^3$ of debris was cleared, and about 1,000 m$^2$ of roads were cleared; more than 50 cranes, 20 bulldozers and 30 excavators took part in the operations.

During the main period, the authorities were engaged in organizing a control system for local rescue operations. Forces were collected in the disaster area from the Civil Defense, the Soviet Army, fire brigades, local police, civil volunteers stationed near the disaster area, and rescue workers from abroad. Operations were conducted continuously in shifts. Access routes to settlements, approach roads and passages to locations of rescue work were cleared. The emergency work mainly concentrated on schools, kindergartens, hospitals, educational establishments, businesses, etc. Because large load dumpers were insufficient in number, construction debris was carried to free areas nearby, regardless of its designed use. Night work at some operations stopped because lighting was not available. Unfortunately, the main mechanized equipment for engineering rescue operations, and other machinery available in the republic, was brought to the disaster area at an inadequate rate. For example, of the 1620 cranes available at the time of the earthquake, only 108 cranes were actually in the disaster area on December 7; 186 on December 8; 507 on December 14; and 584 on December 18. Special units from the Ministry of Internal Affairs organized the registration of the dead with detailed descriptions of physical characteristics, discovery location and time. An information service and a service to produce death certificates were organized. It was during that period that the most important economic units were put under guard.

The main period, beginning on December 8, lasted 6 days, in Leninakan, 5 days in Spitak, 4 days in Kirovakan, and 1.5 days in Stepanavan. During these periods, the work done in the towns involved the extraction of 13,489 people alive and 13,566 dead, the removal of 49,200 m$^3$ of debris,
and the clearance of 4,400 m of roads. This work was carried out by 37,100 personnel using 357 cranes, 80 bulldozers, 109 excavators, and about 2000 automobiles. The work was conducted day and night, in three shifts.

The terminating period was characterized by an already routine control system for rescue operations. The Leninakan area was divided into four sectors (five from December 20), subdivided into areas where the control of rescue operations was carried out. In all, 14 areas were created, each including several objectives; altogether there were 400 objectives in the town, each including several buildings and structures. The Spitak area was divided into five sectors where rescue operations were carried out, while Kirovakan contained ten sectors, in accordance with the number of existing residential areas. Military units and formations of ministries and departments from other regions took part in operations. For example, by December 19, there were 19,500 people from 500 organizations (coming from the Russian, Ukrainian, Belorussian and other republics) taking part in this work in Leninakan. The formation of ministries and departments arrived completely equipped, having the personnel and all that was necessary for day and night autonomous operations. However, further excavators, lift trucks, and large load dumpers were needed.

During this period, buildings threatening to collapse or hamper rescue operations were brought down without the use of explosives. Some lifeline facilities were once more made operational. A rigorous permit system and patrol service were organized in the disaster area, and important facilities were put under guard. The dead, as well as those evacuated, were registered. The terminating period lasted 12 days (December 14 to 25) in Leninakan, 10 days (December 13 to 22) in Kirovakan, and 13 days (December 9 to 22) in Stepanavan. The work done during this time involved the extraction of 448 persons alive (the last was saved in Spitak on December 12, 1988) and 4723 dead, the removal of 184,800 m$^3$ of debris, and the clearance of 59,700 m of roads. The operations were carried out by 52,800 personnel using 689 cranes, 286 bulldozers, 161 excavators, and about 3000 various automobiles. The complete removal of thoroughly investigated debris took place.

Rescue operations were virtually over by the beginning of the transition period. This period ended for all areas at about the same time, December 31, 1988. The amount of work done during that period included the extraction of 150 dead bodies, the removal of 124,300 m$^3$ of debris, and the clearance of 29,400 m of roads. Operations involved 47,100 personnel using 690 cranes, 266 bulldozers, 158 excavators, and about 3200 cars. By the seventh day after the earthquake, 90-100% of the living had been evacuated and 65-95% of the dead located under the debris had been removed. Nevertheless, rescue operations would have been much more effective if the bulk of the work had been done during the first four days after the earthquake. The success of such a task depends on two factors: the availability of required equipment and the availability of appropriate technology for rescue operations.

The most effective techniques for performing various technological operations (primarily extracting the injured from under the debris) were first used as late as the third or fourth day after the earthquake. An extensive use of these only began on the fifth to eighth day, which considerably diminished the overall efficiency of rescue operations and increased the number of dead trapped in the debris. The earthquake created a very complex situation in all fields of life in the
area, including management: many leading officials who were responsible for important mitigation work were killed; the central warning system, telephone and radio communications were put out of order; the state communication system was seriously damaged. Under these conditions, the management system had to be recreated.

A Commission from the Politburo (headed by a Member of the Politburo, Prime Minister N.I. Ryzhkov) was sent to the disaster area to speed up search-and-rescue operations. Groups from the USSR Ministry of Internal Affairs and Civil Defense arrived at about the same time. Direct management for eliminating the consequences of the earthquake was assumed by the Government Commission of the Armenian Republic; its main working body was the Civil Defense Headquarters of the Republic. That Commission worked under the Politburo Commission in close co-operation with the Union level groups mentioned above. There were three well-defined disaster areas in Armenia: the towns of Leninakan, Kirovakan, and Spitak with their neighbouring rural areas. This determined the structure of control systems at local levels. Three headquarters were created to manage search-and-rescue operations in these zones. The centres worked under the Armenian Government Commission. Additional management groups were created in Leninakan and Spitak, controlled by the Head of the USSR Civil Defense.

The following conclusions can be drawn from the above:

1. A management system for emergency situations should be created beforehand and maintained in a state of readiness.

2. Special groups sent to the disaster area should be fully self-dependent until lifeline facilities have been restored.

3. Management bodies should be duplicated, particularly at the local level.

4. The communications and warning system should be as resistant to damage as possible in emergency situations, and have reserves to ensure rapid reconstruction.

The most important result from the experience in eliminating the consequences of the Spitak earthquake is the creation of the State Emergency Commission, USSR Council of Ministers. The lesson learned in Armenia, and the conclusions drawn from them, will be used in establishing the State Emergency Commission and the relevant commissions in the union republics and regions, as well as in everyday activities for the prediction and mitigation of accidents, catastrophes and natural disasters.
The earthquake took place in an active tectonic area which has a set of faults branching away from the main Anatolian fault around the town of Erijan. A.A. Nikonov notes the existing undated traces of old earthquakes of similar intensity in the epicentral area. The Spitak area is a marginal zone of the Javakhet upland where earthquakes of energy $K = 12-13$ typically occur (energy being given in $K = \log E$ in joules). The Spitak earthquake was one of the largest ones in the Transcaucasian region where major shocks occur infrequently but entail serious consequences wherever they do occur. There is historical evidence of previous large earthquakes. The largest took place on February 13, 863 A.D. and December 24, 893 A.D. These both completely destroyed the capital Dvin, which was then moved to Ani (near contemporary Leninakan) which was also destroyed by earthquake in 1320. We know of other large earthquakes: one in 1679 at Garni, near Erevan, one at Leninakan in 1926 and a smaller one in 1935 at Digor. The area of Armenia was also struck by Turkish earthquakes which took place in Erijan, Erzerum, Kars. The last event in Kars occurred on October 30, 1983.

We shall use some considerations from plate tectonics as a working hypothesis in our discussion of seismicity in the Caucasus and Turkey. The Caucasus is pushed by the Arabian plate from the south. This motion is insufficiently known, but can be estimated from Very Long Baseline Interferometry. The motion of the Arabian plate to a great extent affects the Anatolian fault in Turkey and the entire Caucasian area. It can be supposed that this small, but always present pressure causes stress increase, first in Armenia, which is then released by earthquakes. This should be regarded as a working hypothesis, and does not claim to be universal.

The aftershocks of the 1988 Spitak earthquake were studied by both regional and local network stations. The Epicentral Expedition at the Institute of Earth Physics, Moscow soon began operating a high sensitivity station and six strong motion instruments. Later the Strassburg telemetered network (9 stations) was deployed with additional 10 MEQ-800 stations having a velocity response curve and smoked paper recording. These observations were conducted until late February 1989 and provided high accuracy location of epicenters and depth of focus, the magnitudes of small shocks also being based on this data. An American network of digital stations began operating on December 23, three stations being deployed in the south and the remaining 9 stations in the epicentral area. These observations of the Geological Service were supplemented by 10 MEQ-800 stations operating in the epicentral area. All materials were presented in the account. Six digital stations continued operating until late March. Apart from these networks, some work was done using a few (2 or 3) stations by Seismological Institutes from the Turkmenia, Kazakh, Kirgiz, Georgian, Moldavian Republics, etc. Some of the recordings were used in the exercise.
It should be admitted that co-ordination was inadequate with sometimes too many stations operating in the epicentral zone, and sometimes too few.

An objective-enough characteristic of the aftershock sequence is provided by regional seismograph stations which were in continuous operation and supplied homogeneous material, although for energy class $K=9$ only. Although the magnitude of the main shock was 7.0, the damage was high. The town of Spitak collapsed; every second inhabitant was either killed or injured. The epicentral intensity was X. The town of Leninakan was struck with intensity VIII to IX and many were killed. Modern buildings proved to be particularly vulnerable. The towns of Kirovakan and Stepanavan were less affected, the intensity there being estimated as VII. A number of villages were destroyed. The total number of lives lost is 25,000, according to official reports.

The macroseismic consequences were studied by many teams and individual specialists (American, British, Japanese, and other researchers). Much has been done by the Institute of Seismology, Turkmenian SSR Academy of Sciences, as well as by Institutes from Kazakhstan, Kirgizia and a number of institutions of the USSR State Committee on Construction. Macroseismic investigations in Armenia were conducted by the all-union Institute 'Atom-energy-project' under the scientific guidance of F.O. Arakelyan, and a team from the Institute of Geophysics and Engineering Seismology, Academy of Sciences of the Armenian SSR headed by T. Babayan. A special study was carried out by a team headed by I.V. Anan'in, Institute of Earth Physics. The results are not always consistent. It should be said that nearly all researchers were unanimous as to the poor construction quality; also, the unfortunate choice of house type was noted. Violations of building codes were significant and, consequently, resulted in a great number of people being killed. This shows once more that the best protection from damage in earthquake prone areas is quality construction, as was proven by the San Francisco earthquake of 1989.

The Spitak earthquake occurred in the zone of expected intensity VII-VIII, according to the 1978 map of seismic zoning for the USSR. It should be noted that buildings ought to withstand an earthquake of an intensity one degree higher than the design intensity for the area. The only location likely to have been severely damaged by such an earthquake was Spitak. The construction drawbacks were clearly stated in the conclusion of the Government Commission which investigated the situation in the epicentral area.

The earthquake was followed by extensive international aid amounting to 300 million dollars in material aid and 18 million dollars in cash. Aid continues to come in. It should be noted that according to the studies by Sarkisyan at the IGES, the intensity which occurred in Leninakan was generally a unit greater than that of other areas, for instance, Soviet Central Asia. This effect is due to the geologic sub-structure. Lacustrine deposits, a few hundred meters thick, are widely found throughout the Leninakan area, especially in the north. They overlie basalts. This low velocity sequence is overlain by tuff layers sometimes as thick as ten meters. This low velocity structure can vibrate at about 1 sec in the vertical component and about 2 sec in the horizontal. These resonant vibrations cause two effects: the 'lengthening' of vibration and increased amplitude. This can be clearly seen in the records of American seismographs. This magnification of the seismic effect sheds new light on the problems of seismic microzoning. Knowledge of the upper 10-20
m sequence is not adequate. Vibrations are controlled by the entire low velocity section. These features also determined the effects of great damage, as in the 1985 Mexico earthquake. This emphasizes that the characterization of seismic processes due to large earthquakes would be inadequate when a factor of acceleration alone is predicted. Knowledge of other factors are necessary, such as: velocity, displacement, duration and spectral content. For example, knowledge of the geology, for Armenia indicates that at many sites in the area, one can expect increased seismic intensity due to sub-structures like that in Leninakan. This possible effect may be foreseen even for small earthquakes.

Another manifestation of this earthquake deserves notice, i.e. the relevant premonitory effects, and the conclusions to be drawn from them. First of all, we note that, in spite of the fact that many geophysical observation sites were available throughout Armenia (in addition to the sites where seismograph stations were located), observations differed from site to site. For example, it was only at the Garni Observatory, 93 km from the epicenter, that strain was observed using tiltmeters and strainmeters. Electrical observations were conducted at similar distances (Garni and Paraker near Erevan), and magnetometric recordings were made in different areas within the Republic. Routine measurements of radon content were conducted in Dilizhan and Leninakan. Measurements of water level in wells were numerous. This work was done by the USSR Ministry of Geology. Seismic observations were carried out at 14 seismograph stations.

Geophysical observations were not conducted in a systematic manner. The most effective were strain observations which started a month before the main shock. Electric changes were well defined. Radon content changed in Jermuk spiring as late as two or three hours before the earthquake. In addition, strain observations were of interest during the large aftershocks. Changes in strain occurred in the vector strain diagram before the shocks. However, even though the changes in the fields are large, it is extremely difficult to use them for definite conclusions about the time and the location of future earthquakes. Such an indefinite prediction could hardly be taken into account and could only produce panic.

It would be also useful to have some information on changes in soil temperature in the epicentral area during a period of time prior to the earthquake. There is no reliable evidence of such changes. There are some reports from local residents of increased temperature of spring water; this information could also be used in short-term prediction. The Spitak earthquake has led us to several important conclusions.

1. The prediction of large earthquakes requires extensive observations, fast data-processing, and a more homogeneous system of observations;
2. It is necessary to ensure reliable supervision of construction quality and technology in earthquake-prone areas. Local municipal bodies should include a team of seismologists who would supervise all earthquake resistant measures;
3. It is necessary to have a competent rescue team readily available;
4. It is necessary, following an earthquake, to provide better co-ordination of work, both scientifically and with regard to Civil Defense. Special attention should be paid to communications;
5. Greater attention should be paid to current work on seismic microzoning with a view to refining the methods involved, and ensuring adjustments to local environments.
CAUSES OF THE CATASTROPHIC CONSEQUENCES OF THE SPITAK EARTHQUAKE

by

V. Udaltsov
(USSR)

One of the more important lines of research in coping with this kind of natural disaster is a thorough analysis of the damage and the formulation of practical measures which should be taken to mitigate consequences. Identification of the real factors that cause heavy consequences is particularly important for those earthquake prone areas where apparently all precautions were taken to rule out a catastrophe of this size. One example is certainly northern Armenia where the Spitak earthquake occurred. It was for this reason that the USSR Council of Ministers created a special government commission, to identify the causes of such heavy consequences. Its tasks included, an analysis of the parameters of the seismic loads involved; the quality of seismic zoning and microzoning in the region, and design and construction quality, i.e. practically the whole range of possible factors.

The commission called on many organizations, scientists and specialists to participate, among them this author who was directly involved in analyzing various factors and formulating conclusions and suggestions. For this reason the present report is largely based on materials from the commission. Below we discuss the analysis and assessment of factors that caused such disastrous consequences. The Spitak earthquake developed as a sequence of shocks.

An analysis of instrumental records, computations based on theoretical relationships derived from world earthquake data, and the results of macro-seismic investigations of damaged buildings and residual soil deformation (according to GOST - state standards - scale), provide mean intensity due to the main shock.

The intensity depends on soil conditions; the mean intensity is for average soils, i.e. category II according to the building code in force. The intensity of the second shock was 1.0-1.5 below that of the main shock. The expected intensity as given by the seismic zoning map for the USSR, was VII for Spitak, Kirovakan, and Stepanavan; and VIII for Leninakan.

Thus, the intensity of the main shock was two to three units above the building and structure design of units in Spitak; and as much as one unit in Leninakan, Kirovakan, and Stepanavan. The intensity of the second shock in Spitak was also above the design value. We shall briefly discuss the purpose of expected or design intensity for the region at hand.

The adopted design intensity for Leninakan was VIII, this recurring in all seismic zoning maps since 1940. In 1972, a microzoning map for the town area was made where zones of intensity VII and IX were identified, depending on soil conditions. The design intensity for the Spitak-
Kirovakan-Stepanavan area was VII until 1968. The new seismic zoning map compiled for the USSR in 1968 gave the value VIII. However, the authors of the 1978 seismic zoning map incorporated into the 1981 Building Code (SNiP II-7-81) returned the design to intensity VII. This decision, the reduction of design intensity from VIII to VII for the Spitak-Kirovakan-Stepanavan area, was an error.

What were the specific features of the Spitak earthquake which had such an effect on its consequences? Two earthquakes occurred within a short space of time (about 4 minutes), the intensity of the first exceeded the design at Spitak by two to three intensity units; by as much as one unit at Kirovakan and Stepanavan; and by a half to one unit in Leninakan. The second earthquake involved intensities close to the design values. The combination of two large earthquakes is an extremely unfavourable factor for the behaviour of buildings and structures. In addition to horizontal motion, there were fairly large vertical oscillations. The ratio of maximum vertical to horizontal acceleration reached 0.7. Such a ratio is not infrequent, as can be gathered from world earthquake statistics, but this too should be regarded as an unfavourable factor affecting the behaviour of buildings and structures. The net seismic effect was magnified in Leninakan by a number of factors: the area includes large zones of extremely unfavourable seismogeologic environments due to faults and seismically poor soils as can be seen from the new seismic microzoning map of Leninakan. In addition, many areas have been found to be water saturated (mostly owing to anthropogenic factors). An examination of seismic excitation over the Leninakan area suggests the influence of long period ground oscillations. In the absence of direct instrumental recordings of such oscillations, relevant indications can be found in available aftershock records. According to these records, ground displacements in Leninakan are 4-8 times larger than those of hard rock. Spectral ratios demonstrate enhancement within the period range 0.6 to 3 sec. The aftershock records thus suggest that long period ground motion generated by thick layers of sedimentary rock and tuff could have been a factor causing greater damage in Leninakan than in Kirovakan.

The assessment of design quality included residential, public and various other structures such as: frame panel, stone masonry, large block, large panel, and lift slabs. The estimation of each design was carried out by independent experts whose conclusions were then analyzed to formulate final conclusions for each design. The results of this expert assessment are as follows: 73% of the designs were found to be completely inadequate because they involved significant departures from building codes. Two percent of the designs required refinement or correction. The design of frame panel houses and buildings most severely damaged revealed a lack of design for the joint work of diaphragms and frames; staircase and elements; etc.

Because the junctions of members of the carrying frame are located in zones of maximum excitation, the earthquake resistance of these buildings depends on the quality of the junctions. The design of stone masonry and large block buildings had the following drawbacks: mixed designs using heterogeneous materials (reinforced concrete frame filled with tuff masonry, etc.) which violated the code as to design and size; stiffnesses not distributed symmetrically in plan, normal adhesion of stone and mortar not stated in the design, and no connection made between the anti-seismic belt of the upper storey and the underlying masonry, etc. These drawbacks have led to the significant reduction of earthquake resistance.
The design of 9 storey houses requires serious study due to the great departure from building codes. No discrepancies were found between building codes for design intensity VII and the design of the 10 storey residential lift slab building in Leninakan, as well as the 16 storey residential lift slab buildings, permitted to be constructed in Leninakan. These buildings, however, should be strengthened. Overall, the most typical serious departures from building codes were the unfortunate choice of structural large panel design; and the absence or lack of structural measures to ensure the construction of a building as a single spatial system creating the necessary resistance to earthquakes. Among the unfavourable factors were poor quality structural designs of joints which produced unpredictable changes in the theoretical structural scheme of buildings. All these factors contributed significantly to the reduction of resistance of frame panel and stone masonry buildings to earthquakes.

We wish to comment on the quality of engineering studies conducted by design institutes at the Armenian Gosstroj during the years 1969-1985. The quality was in many respects not equal to the demands of the all-union building codes. The exploration wells were actually drilled shallower than intended (by 20-30%). The physical and mechanical properties of the underlying soils for construction were determined without a proper study and the design intensity at some construction sites was reduced by a unit for no apparent reason.

The assessment of the construction quality was conducted along the following lines. First of all, 66 objects were selected for careful investigation by building type. The investigations were conducted using ‘damage-less control’ equipment on samples of materials and structural members. The technical team of physical control determined properties of construction materials in 1057 structural members. In-situ-strength was determined for 651 reinforced concrete members, including 268 precast 327 solid and 62 junction members. Seventy-two places of stone masonry were investigated and the quality of 178 welded joints were assessed.

We also checked 7 main quarries which supplied pebble and sand for construction, 5 mortar making facilities and 4 reinforced concrete factories. These investigations revealed, in all buildings inspected, serious departures from design and building codes, and poor quality joints and structural members, necessary to ensure earthquake resistance of buildings. Concerning construction quality, the most typical defects in residential stone masonry buildings are as follows:

- Practically everywhere (in 92% of the houses inspected), design and building code measures to ensure horizontal stiffness in prefabricated floors were not followed;
- The anti-seismic belts were weakened in 83% of the houses inspected due to a smaller cross section, and there were gaps in the reinforcement or concrete filling. A typical defect was the non-connection of anti-seismic belts to columns and beams of frame buildings, as well as to self-bearing house walls;
- Longitudinal frames in multi-sectional residential buildings which provide longitudinal stiffness had lower strength and stiffness. This defect was found in 83% of the houses inspected and is due to the replacement of soil collar beams by prefabricated ones without ensuring uniform strength and continuity of reinforcement;
- Poor quality of workmanship i.e.; gaps, reduction of design concrete strength by factors of 1.5 to 3, especially in reinforced concrete inclusions with masonry;
• Serious departures from design and building code requirements were detected in 83% of the residential buildings inspected;
• The strength and solidity of tuff masonry was reduced in all houses checked for these parameters. The adhesion strength in stone masonry is extremely low everywhere, amounting to less than 0.1 kg/cm², 6 times lower than the lowest limit of adhesion strength. Laboratory testing has also shown that the strength of the mortar was 1.2 to 6 times lower than the design value. About 60% of the stone masonry 'midis' type in outward walls of most houses was made of stones shorter than the 30 cm (14-28 cm) required by republican technical standards. No proper longitudinal and transverse bonds were available;
• Lack of required number of reinforcement nets in many places where the masonry was joined to reinforced concrete members in 67% of the houses inspected;
• Weak connections between prefabricated staircase flights to staircase landings in 42% of the houses inspected;
• Weak connections between large wall blocks and adjacent structural members;
• Lack of concrete of partitions everywhere (violates requirements of building codes SNiP 11 - A.12 - 69).

Many cases have been detected in which the resistance was reduced by exploitation or maintenance services, and by the inhabitants. In addition, soil was moistened at the base of the foundation which had a negative effect on earthquake intensity and resistance.

Frame Panel Residential Buildings (Series III)

All houses of this series involved significant departures from building codes and design requirements, thus reducing their resistance. Defects were found in the assembly of frames which should have normally ensured resistance to earthquakes. For example, welded joints had strengths 22-33% below the standard in junctions of working reinforcement of columns due to departures from the design scheme and production technology. The axes of the rods to be welded were displaced relative to one another by 10-25 mm instead of the permissible 1.5 mm. Mortar strength was lower than required by 35-60% in places where joints should have been solid in 73% of the buildings inspected. In the construction of beam column joints in the frame ensuring longitudinal stiffness, the poor quality of bath welding in the reinforcement and 3-4-fold reduction in the strength of concrete used for concrete welding led to the junction of members of the frame being virtually a hing instead of a stiff connection. This resulted in departures from the working model of the building as a whole and produced a significant reduction of its design earthquake resistance, in addition to influencing other defects.

When transverse stiffness diaphragms were being assembled, they were fastened to frame columns with weakened joints because of smaller pad plated (by 25-40%) and weld seams (by 30-50%). Some diaphragms were traversed by floor plates, the result being that they were not connected between themselves in a single structure. In making the connecting seam between diaphragms soils, the connecting rods were frequently bent outwards. Faulty work in stiffness diaphragms is very extensive. Plates in floors were not earthquake resistant in 36% of the
buildings inspected. The seams between the plates either had not been soiled or were filled with weak mortar instead of brand 150 mortar. Facing plates were not always welded to collar beams and water chutes, and have insufficient rest depth (2-3 cm instead of 9 cm).

The staircase is the major means of evacuating a building, but here the staircases had significant defects. In 55% of the buildings, most of the flights were welded to landings at 2-3 joints instead of 4, or else were not welded at all. When connecting built-in details in flights and landings, unlawful weakened pads were welded with thinner weld seams. Built-in details in the flights and landings inspected were fastened in concrete with anchors whose strength was less than the design by factors of 2 or 3. Flights collapsed in 55% of the houses inspected. In all the houses, the outward wall panels were fastened to the frame with bent anchors which allowed the panels to be displaced outwards.

Partitions made of small size stones, contrary to the SNiP II-7-81, were not reinforced and not fastened to walls, columns and floors. Forty-seven percent of the factory made reinforced concrete members were of a lower grade, 200-250 instead of 300. A 16 storey lift slab building contained defects which reduced strength and earthquake resistance. Dense concrete (a large of 40 cm, grade 200 instead of the design 300 value) was insufficient. The core failed along these layers. The junctions of vertical working rods in the core were made by electric weldings, these junctions being easily destroyed by the earthquake. The vertical rods were connected transversely with a reinforcement of 1200 x 600 mm instead of 600 x 200 mm. The concrete in solid floors was not adequately compressed. It was porous at the bottom of the ground floor and the working reinforcement was visible over areas of as much as 3 m². Partitions of small sized stones had no reinforcement and were not fastened to walls and floors. Most partitions collapsed.

The quality of reinforced concrete structural members, concrete mortar, fillings, reinforcement, built-in members, and tuff bricks was unsatisfactory due to the low level of technology, the ineffective use of available equipment, the use of inadequate materials, as well as unsatisfactory organization and quality control of the technological processes. The poor quality of inert filling materials (especially sand), was not up to State standards and contributed to the poor quality concrete and construction mortar. The construction work was in glaring violation of design requirements and building codes which caused a considerable reduction in the design earthquake resistance.

Overall, the catastrophic consequences of the Spitak earthquake were caused by an unfavourable combination of the following factors:

- Earthquake excitation exceeded design and had certain peculiarities (a second strong shock suggests that actually two large earthquakes occurred; long period oscillations; large vertical oscillations reaching 0.7 of the horizontal amplitude). The catastrophic consequences in Spitak were due, in the first place, to the actual earthquake intensity significantly exceeding the design value (by two units). Buildings whose resistance was sharply reduced due to poor design and construction work resulted in catastrophes in most cases;
- Poor construction quality;
- Departures from design codes.
It was poor construction work combined with departures from design codes which were responsible for the disastrous consequences which took place in Kirovakan and Stepanavan, as well as in Leninakan where an additional factor, soil conditions and water saturation, magnified the earthquake intensity by a unit and caused long period ground oscillations dangerous for high rise buildings. The above examination of causes of the catastrophic consequences of the Spitak earthquake suggests a number of important conclusions concerning design and construction in earthquake prone areas. Design and construction organizations should remember that earthquake resistant design will always, or at least for a very long time, have to deal with basic seismological data of some uncertainty.

For this reason one should design buildings that not only would have design resistance but would also not collapse under excitations that exceed the design value by one or two, possible three units. This can be achieved without incurring significant additional expenses. Buildings under earthquake excitation should be able to withstand considerable damage, without the loss of lives. The damage due to large deformations must not affect the spatial stability of the building, that is, a collapse must not occur.

This can be achieved in the first place by the proper choice of design scheme involving the so called 'natural' earthquake resistance. At the same time, the adopted structural design should not be sensitive to the quality of construction work done at the construction site. Such structural schemes include large panel houses, buildings involving steel frames, volume block and solid reinforced concrete buildings. Refinement of these structural schemes and the development of new ones is an important task facing construction science in the field of earthquake resistant construction. This theoretical and experimental study based on deterministic and stochastic methods should be extended. The focus of these studies should the comprehensive dynamic field testing of buildings and structural members. Testing should aim at deriving reliable data on limiting states under earthquake excitations of various intensities. This testing will provide the necessary information and basic data for the design and construction of buildings and structures of high earthquake resistance, as well as the survival of inhabitants in conditions were the actual intensity of earthquakes exceeds the design value.
PUBLIC INFORMATION AND EDUCATION
PUBLIC AWARENESS AND INFORMATION SYSTEMS

by
Shigeji Suyehiro
(Japan)

Summary

One of the most difficult aspects of how to prevent and mitigate earthquake disasters, especially from the viewpoint of social preparedness, is that although disastrous earthquakes do not occur frequently, once they do, they can claim an enormous number of lives and even jeopardize the national economy. Nevertheless, even in Japan, where seismicity is the highest in the populated regions of the world, it is not easy to keep a constant high level of public awareness.

Introduction

More than two billion people live in the world’s seismic zones. The most deadly earthquakes of this century occurred in Algeria, Chile, China, Guatemala, Greece, Italy, Japan, Mexico, Morocco, Nicaragua, Pakistan, Peru, Romania, Turkey, U.S.A., U.S.S.R. and Yugoslavia; they claimed more than one million human lives and caused a countless amount of property damage.

The structure of human society, particularly in large cities, has become highly mechanized, depending to a large extent on life-line facilities and computerized systems without taking sufficient countermeasures against earthquakes. Furthermore, the population in large cities has been ever increasing. Should the next large earthquake occur near a country’s capital or a large city, unprecedented damage could be caused, and the consequences would not be local, but nation-wide.

Since the occurrence of earthquakes cannot be controlled, the only way to mitigate casualties and damage is to make our society more resistant against them. It is also false to assume that earthquakes never recur in the same place. T. Terada, father of geophysics in Japan, said “natural disasters occur when bitter experiences of the previous one have been forgotten”.

So it is, that in Japan, "Natural Disaster Prevention Day" is commemorated annually on 1st September in memory of the Great Kanto Earthquake of 1923, which claimed 143,000 lives. Every effort to educate and train the population on a nation-wide scale is implemented around this time each year in which the mass media play a very important role. If such an effort were not made annually, the general public would easily forget the seriousness of earthquake damage. Figure 1 shows the complex nature of earthquake disasters.
Progress of Earthquake Disaster Countermeasures

In order to be well prepared against potential earthquake disasters or natural disasters in general, it is absolutely necessary to install relevant legislation and ensure such legal measures against disasters persist from one generation to the next. Until 1960, the national policy of the Japanese government towards earthquakes was mainly concentrated on remedial actions ex post facto, such as providing financial assistance for relief and recovery activities after the damage inflicted by disasters.

With the enactment of "The Disaster Countermeasures Basic Act" in 1961, the national policy gradually integrated various countermeasures (emergency, prevention, restoration, and others) so that various actions could be coordinated into a comprehensive, and objective, executive plan with the ultimate purpose of disaster relief.

The National Land Agency, inaugurated in 1974, took over the ministerial functions for such countermeasures, and in 1984 the Disaster Prevention Bureau was established, a function of which is to develop disaster countermeasures, through comprehensive coordination with various ministries and agencies, to meet the complex nature of earthquake disasters.

Figure 2 shows national coordination and assignments at different levels. Figure 3 shows the framework of earthquake disaster countermeasures.

Information and Education

These subjects have already been touched upon in the previous sections. So it will not be necessary to explain the importance of providing the public expeditiously with correct information at the time of an emergency. Both the Disaster Prevention Related Telecommunication Network and the mass media would be used for this purpose. In particular, the NHK (Japan Broadcasting Corporation), a designated public corporation, has already completed its preparations for uninterrupted broadcasts under the worst possible conditions and a scenario in which not only bad news but good news will be be broadcast. In 1987, on Disaster Prevention Day, the NHK made a continuous radio broadcast from 7 am until 5 pm. This special programme consisted of many on-site reports of the annual drill and relevant lectures by seismologists and disaster-prevention specialists. Local authorities and other mass media are also making efforts to educate the people throughout the year.

So, because disastrous earthquakes seldom occur, it is not easy to continue to maintain the nation's preparedness. Fortunately, in Japan the National Land Agency exists, whose main task and responsibility is to protect the nation from natural disasters. This appears to be the only way to retain the determination and will to act against such disasters.
DISASTER MANAGEMENT AND THE ROLE OF CIVIL DEFENSE
MITIGATING NATURAL DISASTERS
by
E. Lohman
(Netherlands)

Introduction

Floods and storm surges in Bangladesh, cyclones in the Philippines, earthquakes in Turkey, Mexico, Armenia (USSR) and California (USA), volcanoes in Columbia and landslides in Indonesia: all these recent major disasters indicate a trend of rising severity of disasters in both developing and developed countries. The changing dimensions of these disasters have been population growth, urbanization and industrialization. While every effort must be made to deconcentrate development (and the population) in disaster-prone regions, the fact remains that such a process is complex and slow to establish. The vulnerability/disaster/relief spiral must, and can, be brought under control by means of disaster mitigation.

Aims and Objectives

Disaster mitigation aims at an optimal risk reduction based on existing limitations, making the effects of disasters less severe, violent or painful. Within the given socio-economic, physical and technical situation, disaster mitigation emphasizes planning measures such as: modifying hazard through protective measures or improving the sites; reducing structural vulnerability through the strengthening of buildings of infrastructure elements; or changing functional characteristics such as the regulation of landuse (zoning or resettlement) or the expansion of the infrastructure system.

Policy Framework

The effective mitigation of disaster does not just happen, it is created. Moreover, it is created by hard work within government and non-governmental organisations striving to reduce loss of life and property caused by natural disasters. The policy framework within which this occurs has three aspects: risk assessment, which defines the type and magnitude of disasters that may occur and their effects on settlements; planning and decision-making, which organises a response to these risks; and implementation, which translates plans and decisions into actions at field level.
But these three important activities cannot operate in a vacuum, rather they require the fourth aspect - the policy framework: that of government administration which provides opportunities and constraints in planning for the mitigation of disasters. Many strategies arise from this framework which are elaborated in the various sections of the UNDRO Manual. Both the policy framework and the strategies stress the fact that the three phases involved in the effective mitigation of disasters lie within the sphere of government administration which, in turn, affects the efficiency and nature of all other activities.

Initiating Disaster Mitigation

There is no one perfect way to begin. There have been many points of departure. For example, risk reduction measures against natural hazards may be routinely practised by public works engineers and river basin authorities; local communities may practise measures to lessen risk of loss of homes. On the other hand, none of this may be happening.

Then disaster strikes. A government emergency response is made. It is recognised as being less than effective. There is a call to ensure a more efficient response and for efforts to be made to reduce risks. Thus the government decides to intervene in national life to achieve risk reduction and to ensure disaster preparedness.

Interventions by government in disaster management activities can be thought of as constituting a disaster "cycle". This is followed by another cycle of rehabilitation, reconstruction and of improved preparedness and relief organisation. In this second cycle, opportunities arise to apply carefully thought-out risk-reducing or mitigating measures.

Preparedness and risk reduction - or mitigation - can be thought of as being two sides of the same coin. To the extent that mitigation is not practised, so preparedness needs to increase in scale. Humanitarian issues dominate the relief phase and penetrate that of preparedness. Economic issues tend to dominate mitigation procedures and practice. The role of government is always central, however, and its administration must reflect this role.

Methodology

Part Two of the UNDRO Manual brings together the various technical disciplines needed to produce effective mitigation of disaster. With its focus on structural and non-structural aspects (for example buildings and landuse), it will serve as a cross-reference system for those concerned with the planning and implementation of mitigation measures. The framework within which effective mitigation of disaster occurs has three aspects: risk assessment, planning and decision-making and implementation.
The design of mitigation measures involves close study of four parameters: a) hazard, b) vulnerability, c) elements at risk, and d) specific risk. These parameters are necessary in order to produce an estimate of risk - that is, the total expected losses in lives and property for a given area due to a given type and magnitude of natural hazard. It is by using such an estimate that well-founded disaster mitigation measures can be designed and put into effect.

a) Hazard means the probability of occurrence, within a specific period of time in a given area, of a potentially damaging natural phenomenon. It is a probabilistic function of magnitude - or intensity, according to the hazard type - over time. Hazard functions can be derived for different sites if there are sufficient records going back over a significant period of time. While some countries have gathered records of natural disasters over several centuries, a majority of countries have only had reliable disaster statistics for the past century, especially as regards scientifically measured records. Therefore, it is common to extrapolate hazard from limited data. While the reliability of hazard assessment under such conditions may be open to question, the planner needs clear orientation on how to make optimal use of this data for mitigation action in disaster-prone areas. Two groups of hazards to which the planner should respond are considered in the manual: hydrological hazards (floods and cyclones) and geological hazards (earthquakes, volcanoes and landslides).

b) Vulnerability constitutes the link between hazard and risk assessment. It is a function relating expected degree of damage - expressed as a percentage of total loss of value - to the corresponding magnitude or intensity of hazard. Vulnerability, therefore, cannot be mapped as it clearly describes an intrinsic structural quality irrespective of its location. There are different methods of assessing vulnerability, as well as varying attitudes as to what vulnerability should cover.

c) Elements at risk are the population, buildings and civil engineering works, economic activities, public services, utilities and infrastructure etc., at risk in a given area.

d) Specific risk is the expected degree of loss due to a particular natural phenomenon and as a function of both natural hazard and vulnerability. While risk is the estimate of the total expected losses for a given area, specific risk is the expected degree of loss to a given category of elements at risk. Risk is thus the product of specific risk and elements at risk for all categories of elements at risk combined. With this information, proper measures to reduce risks at national, regional and local level can be developed in the form of prevention, mitigation, preparedness or relief actions.

Risk Reduction

Risk reduction includes using many means to apply specific safety measures, which fall into two categories: those that are structural (i.e. flood barriers) and those that are non-structural (i.e. hazard and disaster warning systems). The UNDRO Manual considers various options for struc-
tural risk reduction, namely those of: a) modifying the hazard, b) reducing the structural vulnerability, or c) changing the functional characteristics of settlements.

a) Modifying the hazard. One of the options to mitigating disaster is to reduce the hazard proneness of sites:

- Protective measures. Such measures aim to reduce the impact of hazard, and include the construction of embankments or the regulation of debris flow caused by landslides. The protection of sites is often a costly option; therefore this option should be related to capital investments in the area as a whole, and should be compared carefully with other options.

- Improvement of sites. Site improvement mitigates disaster by changing the physical characteristics of the site itself. The objective of this option is to prevent the triggering of the hazard (e.g. landslides) or to regulate its impact by ground improvements, or drainage and slope modification. This option is, of course, governed by the type of hazard involved.

b) Reducing the structural vulnerability. Vulnerability is reduced by improved engineering design, by construction methods, and by strengthening construction. The latter, however, does not contribute to risk reduction when the vulnerability is 100%. This is the case for most landslides and for the direct effects of volcanic eruptions.

- Reducing the vulnerability of buildings. For the reduction of earthquake risks, strengthening of construction is the main option. Resistance against groundshaking can be increased, which leads to significant risk reduction. For flood and cyclone hazards, the strengthening option can be applied to all structural types.

- Reducing the vulnerability of infrastructure elements. The physical strength of infrastructure systems is improved in order to assure adequate functioning of day-to-day facilities and services during, and after, a natural disaster. This option may comprise, for example, the strengthening of bridges against lateral slide in an earthquake or against flood-induced forces. Infrastructure systems require high public investments. Therefore, it is important that the vulnerability of such elements be low. These extra investments should be compared with the costs of repair after a disaster which may require even higher investments.

c) Changing the functional characteristics of settlements. Changing such functional characteristics can have a direct bearing on risk reduction. Examples include the changing of population density, the layout of a neighbourhood or the relative importance of infrastructure systems. The UNDRO Manual distinguishes between physical structures (vulnerability) and the functions they serve. The expansion of infrastructural systems will lead to a reduction of risk to parts of these systems through in-built redundancy.
Mitigation Planning

On the basis of the methodology for risk assessment and risk reduction measures, as developed in the UNDRO Manual, a sequence of activities will enable planners to carry out hazard and risk assessments, together with various specialists, and to propose risk reduction measures as a basis for disaster mitigation projects. The sequence of activities for all hazards is as follows:

a) Hazard assessment
b) Structural vulnerability analysis
c) Risk assessment
d) Risk reduction measures
e) Preparing mitigation plans
f) Implementation of mitigation plans

This sequence of activities focuses on curing the effects of disasters on settlements at the local level and their direct environment, rather than on causes at the regional level. The causes of the disasters are often too complex for most settlement planners, who are not trained in geoscientific assessments over large areas. The UNDRO Manual helps planners to translate risk assessments into risk reduction measures and mitigation plans at local level.

Targets for Mitigation Implementation

The preceding sections have covered risk assessment, risk reduction measures and mitigation planning. In turning to the subject of implementation, it is essential to recognise that protection has two objectives:

(a) the reduction of deaths and injuries;
(b) the reduction of property losses of buildings and economic assets.

These losses could be direct (i.e. immediate damage as a result of the disaster impact) or they could be indirect (i.e. longer term damage to livelihoods as a result of a factory being out of production for a long period of time). Indirect losses are likely to be less tangible but can be of a greater, far-reaching social and economic impact than the highly-visible direct losses.

Different measures are needed to select targets and address these situations, and the decision-making process already identified has described a systematic way to determine suitable areas requiring protection. In identifying targets for mitigation, it is important to emphasize that they are all moving targets - none are static. As patterns of risk rapidly change - due to such pressures as urbanization, environmental degradation and population growth, assessment techniques, implementation strategies and mitigation actions will also need to adapt in order to relate to this dynamic context.
Information Needs

The rapid growth of settlements in hazard-prone areas calls for careful studies of environment, configuration of terrain, and sub-soil conditions - particularly in suburban zones. It is therefore essential to indicate clearly the variables which describe the different aspects of environment, terrain and subsoil; the type of data needed, the form in which they should be presented, and how this information should be communicated to the various professions involved in its use.

The complex and interdisciplinary character of mitigation planning requires clear concepts. Among the key questions are: which maps, data, and decision-making tools are required for the various phases in order to implement projects at regional or local level within time-frames and budgets fixed in development plans? In addition, different types of geo-scientific data, different forms of land use, socio-economic and demographic data will also be required at various stages of decision-making and implementation. The type of information (including the level of accuracy, speed of data collection required and its scale) needs to be in line with the requirements of each phase.

Training

Significant risk assessment requires specialist multi-disciplinary task forces which should include geo-scientists, engineers, planners, environmentalists, economists and sociologists/anthropologists. They need to identify hazardous land in and around centres of population and to make a detailed analysis of these hazard-prone zones.

1. The task force should initially be organized, say for one year, on an ad hoc basis, with its own funding, temporary staff and equipment. Full use of available maps, aerial photographs, satellite imagery and statistical data of all settled land should be guaranteed without restriction.¹

2. In the medium term, the ad hoc task force should be converted into a more established form of co-operation between the settlement planning teams and the organizations responsible for geo-scientific and environmental assessments. The regional administration should play an important role in drafting regionally-applicable policy guidelines within a national policy on disaster mitigation.²

¹ Training: Special “in-service” training programmes need to be designed to create such task force groups and to facilitate initiation of their work.
² Training: All professionals involved in settlement planning in disaster-prone regions should be encouraged to apply for “mid-career refreshment” university training of some weeks in multi-disciplinary mitigation planning.
3. In the longer term, a more regular form of co-operation between planning teams and institutes responsible for geo-scientific and environmental studies should be considered as a fully integrated part of settlement planning.\(^3\)

\(^3\) Training: Universities may play a significant role in the development of research activities and training programmes for disaster mitigation. Training curricula of relevant professions, especially in disaster-prone regions, should include multi-disciplinary training for disaster mitigation. The following departments may be of particular interest:
- department of geology, including geomorphology, hydrology, geography, remote sensing and image processing;
- department of regional and city planning;
- department of sanitary and environmental engineering.
REORGANIZATION OF A CIVIL DEFENSE SYSTEM

by

F. C. Cuny

Dallas, Texas, U.S.A.

SUMMARY

This paper sets out a step-by-step procedure under which a typical civil defense system might be reorganized and decentralized to form a more community-based emergency response organization. This procedure is based on an analysis of changes in the organization and mission of civil defense agencies in large, industrialized countries where the trend has been to convert organizations set up for civil control following war, to civilian agencies with a broader mandate: namely, assisting state and local government to prepare for and respond to a wide range of natural and man-made hazards.

DEFINING THE CONCEPT OF CIVIL PROTECTION

The first and most important step for most agencies is to determine whether or not civil defense should remain a part of a defense ministry or be separated and re-formed as a civilian emergency preparedness and management agency. Ultimately, this decision will determine how the agency is configured, the role it will play in various types of emergencies, and the extent to which it becomes involved in various phases of disasters and emergency operations.

Worldwide, the trend has been to separate civil emergency operations from military control and supervision. While the military establishment still plays a vital role, especially in such critical areas as emergency assessment, logistics, medical support, etc., these are functions which can be carried out more effectively if they are subordinated to civilian authority. In the western industrialized countries, the trend is to devolve emergency authority from the central government to states and local governments (or in Soviet terms, from the center to the republics). This approach permits much greater flexibility and places responsibility for immediate response at the scene. The role of national organizations is supportive, with national resources and assets brought in to supplement the local, on-site resources and personnel; national authorities take over only when there are gaps in the services which occur as a result of casualties or excessive damage.

While there may be some advantages to leaving civil defense in a ministry of defense, the advantages of being a civilian agency are far greater. Among the immediate benefits are:

1. Emergency management can be configured more readily to existing administration, thereby building on local capacities;
2. Emergency plans can be more closely tailored and configured to civilian needs;
3. The workings of the preparedness agency are open to far more people and organizations; and
4. A larger number of international contacts are possible, thereby opening new areas of technical exchange, training and information-sharing.
A. Determining the Method of Operation

In general, there are three alternative roles for a national emergency management agency. They are:

1. To take overall charge and responsibility for the emergency from the center;
2. To support on-site leaders; and
3. To complement local community response capabilities by augmenting their resources as necessary.

In the first, the emergency management agency is required to have several large, mobile response teams fully equipped and able to respond anywhere in the nation on short notice. This type of organization is relatively large in size and requires a highly centralized organizational system. It must rely on sophisticated command and control systems and must be able to mobilize hundreds of people and move them quickly in all weather conditions to remote areas.

In practice, this model is very expensive and the performance of agencies configured this way in all parts of the world is very poor -- no matter what state of readiness the agency has achieved, it is still forty-eight to seventy-two hours before its personnel can be fully deployed and operational.

Conversely, emergency organizations that are designed to support local officials by responding to their needs and coordinating assistance from outside the area are much smaller and can be easily decentralized. As a rule, they are much cheaper to operate.

One major drawback is that if the local leaders sustain high casualties during the disaster, relief operations may be delayed and critical functions may not be carried out swiftly and thoroughly. For this reason, the third model has often been proposed.

An agency designed to complement or augment local officials requires a slightly larger team but is still relatively small compared to the first model. This type of organization can also be decentralized. It differs from the second model in that small, self-sufficient and highly mobile teams familiar with specific emergency needs or operations are formed which can be deployed to the disaster area to provide essential services or operate critical facilities and systems under the authority of state or municipal authorities. The teams usually consist of technicians, repair crews and emergency management advisers. This model is far less common than the first two but, in the countries where it is used, it has proven to be cost-effective and efficient.

B. Determining the Extent of the Agency’s Involvement in Various Phases of an Emergency

Each disaster type, and the preparations for it, can be seen to progress through distinct, recognizable phases. Pre-disaster actions usually include disaster mitigation and emergency preparedness; post-disaster phases include emergency response, rehabilitation and reconstruction. As new concepts of civil defense are analyzed, the leadership must decide which phases of a disaster will be addressed by the organization and the extent of involvement in each. Most agencies have tended to focus only on preparedness and response, but most specialists today recommend that emergency agencies be involved in all phases. Disaster mitigation should be viewed as an opportunity to reduce the potential workload of the agency, and both national and local authorities and economic planners should recognize that integrating civil defense measures into their planning process will not only save lives but, in many cases, increase the scope and
effectiveness of their planning efforts. Taking a limited view of emergency management is to ignore a useful asset and input into long-range development plans.

If broad-based disaster preparedness planning is undertaken, the advantages of working in all phases of disasters will soon become apparent. For example, disaster preparedness is not simply the stockpiling of goods and equipment for an emergency; the primary task is to move decision-making forward — in other words, to examine what the emergency needs are likely to be and the decisions that need to be made. By analyzing them long before an emergency strikes, all the various options can be identified, viewed and tested long before an emergency manager is required to confront the issue. In this process, the sequential nature of decision-making will also become apparent and, as the agency reviews its preparedness measures, it will soon identify many areas where mitigation measures can be taken that would greatly reduce both the scope of disasters and emergency response requirements. In broad-based preparedness planning, it will soon become apparent that it is almost impossible to take any decision out of a context which includes the entire spectrum of disasters.

Finally, if the organization takes a broad, all-phase approach, the development of emergency response doctrines and training for decision-making will be facilitated.

ASSESSMENT OF REQUIREMENTS

The next step most governments take in reorganizing civil defense is to conduct a hard evaluation of past performance and identify the real needs and requirements of communities threatened under various disaster scenarios. This evaluation must be thorough and critical. The entire success of the reorganization will be based on these studies.

Performance must be analyzed with an extremely critical eye. In particular, the performance of the organization and of state and local officials must be scrutinized. At the same time, the needs and requirements of both survivors and institutions must be identified. It is important that real needs be separated from perceived needs; much of what passes for disaster relief today is not only unnecessary, but often counterproductive.

At this stage it is also important to analyze how people on the scene respond to various emergencies. Many of the standard assumptions about the behavior of people has proven to be incorrect; many of the traditional relief measures employed fail to utilize and build upon the collective response and convergence of people at the scene and, in many cases, have created relief systems that confuse and delay emergency operations. Disaster response should be based on supporting the popular mass responses. This is done by enhancing local capabilities and helping the survivors and others in the affected zones to cope with emergency needs themselves as the first step in the overall response.

Perhaps one of the best examples is in the case of search and rescue. In the immediate aftermath of an earthquake, thousands of people are rescued by friends, relatives or other survivors as well as by people converging on the site within the first 24 hours. In contrast, specialized rescue teams usually arrive the second day, or later, and only extract a few dozen (or less). The implication is clear: if survivors can be provided simple tools and equipment -- such as hammers, crowbars, ropes and tackle, flashlights and other hand tools -- hundreds more lives can possibly be saved. Conversely, if resources are channeled into small, highly-specialized rescue teams that must be flown in from outside the area, the number of lives saved is likely to remain low.
In carrying out an assessment of past performance, it is important to evaluate performance according to various sectors. This is because responsibilities ultimately will be borne by sectoral, or line, ministries. For example, medical and public health activities are the responsibility of public health officials. Housing and shelter are likely to fall under a housing ministry (or in the USSR, the State Committee for Construction).

Among the more important sectors are:

1. Medical and public health;
2. Housing and shelter;
3. Agriculture;
4. Economic enterprise (divided into small, medium and large enterprises and industrial facilities);
5. Lifelines and infrastructure (e.g., water, sewage, electricity, etc.);
6. Critical governmental facilities;
7. Personal needs of survivors; and
8. Emergency operations (search and rescue, communications, logistics, etc.).

It is important that every aspect of past emergency responses be closely analyzed to determine what went right as well as what went wrong. Wherever possible, the decisions that were made should be identified and the officials that made the decisions should be queried about the assumptions on which the decision was taken.

Finally, foreign aid received should also be closely examined. Unfortunately, many countries have witnessed the mass arrival of much useless, unsorted and counterproductive aid. Many relief teams arrive too late to be of any consequence. Most are configured improperly for their role (especially in winter earthquakes). Few have the proper resources or transportation necessary to carry out their mission effectively, and the majority of the tons of supplies that are sent are of little practical value to disaster victims. Foreign contributions can be of value, but only if they are properly controlled and if emergency management officials have predetermined the services and goods that could be useful and have developed specifications that can be transmitted to potential donors. A close examination of the assistance received can yield information upon which guidelines for donors can be prepared; this will facilitate the coordination and distribution of international aid.

DEVELOPMENT OF DOCTRINES

The development of emergency management doctrines is the principal outcome of an evaluation of past performance. Doctrines embody, and are an extension of, the process of setting policies.

Most importantly, doctrines shape the various responses and provide a basis for planning. They define the objectives, the policies and the operational approaches for the critical segments of emergency operations.

To be effective, doctrines must consider:

A. The Climate and the Range of Weather Conditions Likely to be Encountered

For example, doctrines for response to a winter earthquake are much different from those needed for responding to a summer earthquake. In a winter earthquake some elements of the population will need to be evacuated, while in the summer survivors may remain close by. If a
doctrine mandates evacuation, logistics and shelter needs will be greatly different from a situation where the majority of people remain on site.

B. Long-Term Objectives

The process of establishing doctrines permits planners to examine the relationship between immediate responses and long-term reconstruction objectives. For example, in rural areas, where reconstruction of single-family dwellings will be more prevalent than in urban areas, emergency shelter strategies can be adjusted so that building materials that can be used for emergency shelters, then later to rebuild permanent housing, can be provided rather than tents. By establishing a shelter-to-housing response doctrine, economies of scale and effort can be made.

C. The Interrelationship of Various Emergency Actions

For example, doctrines regarding evacuation will determine what types of emergency shelter responses are required. If the doctrine calls for minimal evacuation, attention must be placed on providing emergency shelter to the victims on-site. Conversely, if the doctrine calls for evacuation on a large scale, the means for rapidly identifying long-term, off-site lodging must be developed along with transportation, systems to record and trace those who have been temporarily relocated, and systems to provide evacuees support while they are in their temporary setting.

D. The Scale of the Disaster

Doctrines must define the potential scale and the types of resources that will be committed by each level of government. (These become the basis for preparing enabling legislation.)

At a minimum, doctrines should be developed for:

1. Search and rescue;
2. Evacuation and provision of emergency shelter;
3. Utilization of foreign resources;
4. Triage and first aid procedures;
5. Comprehensive medical assistance (from rescue through post-operative care);
6. Tracing and family reunification;
7. Operation of municipal systems (especially lifelines);
8. Security operations and the use of military resources;
9. Public assistance (food, blankets, etc.); and
10. Coordination and communications.

DETERMINING STRATEGIES AND APPROACHES

The next step usually taken is the development of strategies and approaches. This process too is based on the detailed assessment of performance and needs. Strategies describe the overall plan and how to tackle specific problems; they should define which organizations and resources are to be committed and how, when and where. Approaches define the methods to be used, the quantities of personnel, equipment or goods that will be provided under different scenarios, and the programs that will be set up to administer relief and other assistance.

The question of how many supplies or other resources should be deployed in a specific emergency is always a major concern. There is a need to respond rapidly without under- or over-supplying. One of the ways some disaster preparedness organizations meet this dilemma is to plan a system according to "blocks" of population. In this method, ratios of needs to specified numbers
of families under different disaster scenarios are established and in an emergency, supplies are committed on the basis of these ratios. For example, disasters may be classified as small-, medium- and large-scale. In a small-scale disaster, blocks of 50 families might be chosen as the basic planning unit; in a medium-scale disaster, 500 families; and in a large-scale disaster, 5,000. If a small disaster were to occur -- for example, a chemical leak -- prepackaged supplies and equipment based on the estimated needs of 50 families could be supplied incrementally. If the disaster were larger -- for example a flood -- supplies could be provided in increments for the needs of a population of 500. By using this method, it is possible to develop a modular system for supplying most relief needs. There would be some over-supply, but the approach permits relief authorities to commit resources immediately without a risk of major overcommitment or of failing to supply enough resources in the immediate aftermath.

Selection of one strategy or approach should not preclude the adoption of others if the agency's resources allow. It is especially important that approaches be balanced and complementary.

PREPARATION OF PLANS

At this point, it is possible to begin the preparation of detailed plans. Again, the plans will be shaped according to the doctrines, strategies and approaches chosen earlier. Plans are needed for each type of natural hazard or threat and should provide guidance for both pre-disaster and post-disaster actions.

The types of plans that are needed are:

A. Mitigation Plans

Plans to mitigate disasters are the most important element of pre-disaster planning. The primary role of emergency management agencies should be to work with other ministries to integrate disaster mitigation planning into normal economic and urban development plans. For example, cities should be planned in such a way that buildings are properly spaced so that if they collapse they will not damage surrounding structures; roads should be sufficiently wide that emergency vehicles can gain access in the immediate aftermath of a disaster; and safe areas and potential refuges should be planned as a part of all large communities.

B. Preparedness Plans

Preparedness plans serve two purposes: first, they define the resources that are to be committed during an emergency and the method in which they will be employed. Second, and more important, in preparing the plans emergency management specialists have an opportunity to review the options for each emergency action and provide guidance to leaders on the proper decisions to make in various situations they are likely to face.

Preparedness plans should address the following:

1. Warning;
2. Evacuation;
3. Temporary shelter; and
4. Support.
C. Emergency Response Plan

Emergency response plans should generally be set out in the sequence that actions will be taken in the aftermath of a disaster. Plans should include, but not necessarily be limited to:

1. Damage and needs assessment;
2. Casualty evacuation;
3. Medical and public health needs;
4. Evacuation and shelter;
5. Public assistance (food, water and personal supplies);
6. Tracing and family reunification;
7. Disposal of the dead; and
8. Control of secondary threats.

The development of plans should be in accordance with the doctrines and strategies elaborated earlier. As each response is developed it is important to define coordination and communications requirements. Resources to be employed should be integrated sequentially, according to when they are likely to become available.

The major problem with many plans is that they are unrealistic about when resources can actually be committed. As a general rule, planners should only plan to use resources already in the community during the first 12-24 hours. Resources stationed nearby could reach the site within a 12-48 hour period; national resources after approximately 36 hours; and international resources within a 48-72 hour period.

It is important to emphasize the plans should be designed to support popular responses, especially in the areas of search and rescue and shelter. Priorities should be given to those actions that save the most lives.

D. Transition Plans

One of the most often overlooked requirements in emergency planning is developing guidelines to help officials transition from emergency to longer-term, rehabilitation and reconstruction activities. It is important to define when emergency operations should be terminated or scaled back. Transition plans should identify what resources should be left in the community, the type of supplies to be provided and what changes in organizational structures are necessary to facilitate longer-term actions.

DESIGNING THE EMERGENCY RESPONSE ORGANIZATION

Once the plans have been developed, the process of structuring the emergency organization begins. No one organizational structure can meet all emergency requirements; different emergencies require different configurations and different scales of organization. As a general rule, it has been observed that "pyramidal" organizational structures have proven to be far less effective than "matrix" style organizations. Pyramidal organizations promote compartmentalization and concentrate too much authority at the headquarters rather than in the field, thereby increasing the number of decisions that are made by leaders outside the community and impeding the flow of information up and down the chain of command.

If a pyramidal organization is required, some of its shortcomings can be overcome through the use of task forces and the deployment of small, semi-autonomous technical and operational teams.
Whatever type of organizational structure is chosen, it is very important to remember that it should be kept as simple as possible and it should permit small unit leaders to operate independently.

LEGISLATION

Once emergency plans are complete, it may be necessary to revise the legislation that provides emergency managers and the agency the authority to perform their tasks. Legislation that activates emergency powers and provides extraordinary authority to officials is known as "enabling" legislation.

Enabling legislation should:

1. Define the conditions under which civil defense rules will go into effect;
2. Define who in the government may declare that the rules are activated (i.e., declarations of emergency);
3. Define what resources may be utilized without prior approval to meet emergency needs; and
4. Define what changes in governmental structures are required in order to successfully manage an emergency (for example, certain departments may become subordinate to emergency management authorities under a declaration of emergency).

Most countries develop legislation for three levels of government: national, state and municipal levels. Since in a decentralized system the prime responsibility for emergency response rests with local communities, the process of developing legislation usually begins at the level of municipalities, then for states and finally for the national system, incorporating the plans of the state and local systems.

TRAINING

One of the most important elements of disaster preparedness is training. Training has two purposes: improving the skills of the emergency team and verification of plans. Of the two, the latter is more important. Training exercises, drills, simulations, etc., can point to major weaknesses in the conceptualization of plans and indicate problem areas, potential bottlenecks or operational constraints. For this reason, training should be viewed as a continuing part of the emergency planning process. Any time a plan is changed, drills or exercises should be carried out to verify that the new procedures, equipment, etc., perform as planned.

Training should be based on the various scenarios that are anticipated. Realistic simulations and exercises can be designed which can give the participants a feel for the conditions they are likely to encounter and familiarize them with the tasks that they must carry out.

In designing training programs, it should be remembered that the most important thing to test is the capability of team leaders to make the best decisions under the circumstances. Emergency decision-making (EDM) is an emerging science that can be applied during training. EDM utilizes two approaches: analysis of "decision chains", i.e., the cause-and-effect relationships of decisions and how one decision sets the stage for subsequent decisions; and "situational awareness", i.e., identification of typical situations that occur after emergencies and the points at which decisions must be made in order to influence the outcome of events.
EDM stresses the sequential nature of decisions and how to identify and correct bad
decisions.

A major issue of disagreement among western civil defense experts is the scope of training
and who should receive the training. Most agree that key staff, municipal leaders, and specialized
teams can never receive too much training, but there is considerable disagreement about whether
training for volunteers, volunteer brigades, etc., is cost-effective. In one Western country, over
100,000 volunteers receive annual emergency management training at a cost of over 50 million
dollars. Yet studies have shown that less than one percent of these volunteers have been called
into action in the last decade. Even when they are called up, studies have shown that most arrive
at the scene long after the need for their skills has passed.

This low use rate has led several emergency management specialists to propose that training
be redirected and emphasis given to providing post-emergency training on site. For example,
instead of organizing and training standby search and rescue teams, it is proposed that methods be
developed to rapidly disseminate information to the spontaneous search and rescue groups that
converge on disaster sites to teach them, on the spot, where to look for survivors, how to pinpoint
them and how to make simple rescues. In this approach, emphasis is placed on training trainers
and developing a vast array of self-teaching methods and media and means for rapidly deploying
both trainers and instructional materials.

Proponents of this approach argue that it is much more cost-effective and will yield better
results because it builds on popular responses to an emergency.

PUBLIC AWARENESS

A major function of an emergency management agency is the creation of general public
awareness about hazards, what to do should an emergency strike, and what behavior is expected of
people in the aftermath. The public should be made aware of where to go for assistance, the role
each agency will have in providing specific services, and how aid will be organized and
distributed.

Public awareness plans should not be too grandiose nor should messages be too alarmist.
Emergency managers should decide on three or four principal messages that the public should
remember and focus on these. Elaborate public service messages that require memory of, and
discrimination between, multiple items have not proven successful. For example, in the United
States few people remember what the different number of blasts on warning sirens or horns
indicate; few can distinguish between a tornado warning and an alert for nuclear attack. Even the
wording of messages must be closely analyzed. Recent studies have shown that few people in the
United States can differentiate between severe storm watches and severe storm warnings.

Nonetheless, public awareness is a major task of emergency management agencies and
should be seen not only as a means of disseminating information, but also as a means of gaining
public support for emergency preparedness and disaster mitigation efforts. A well-planned public
awareness effort can build a strong constituency for the agency and will help ensure that disaster
plans are prepared and followed by community leaders.

EVALUATION

Evaluation is the final element of emergency preparedness planning and, in many ways, is
the most important action taken after the plans are established. Evaluation must be a continuing
process. Training programs can be used to assess plans prior to an emergency, but detailed
evaluations must be carried out in the aftermath of disasters. It is crucial that every element of the disaster plan and the actual emergency response be closely examined so that improvements can be made in future plans.

When evaluating plans, it is important to evaluate them according to the objectives set out during the planning process. Were the objectives reached and, if not, what revisions are necessary?

Evaluation can be carried out in two ways: First, self-evaluations conducted by staff deployed during the emergency. Self-evaluations, if properly structured, can be a most effective means of determining what went right and what went wrong.

To assist in the evaluation process, it is important to establish a permanent evaluation office under the leadership of an Inspector General. The purpose of this office is to provide assistance to state and local staff to help them plan and carry out self-evaluations. The office also conducts research and investigations on the overall emergency response. The IG staff should see themselves in the role of "devil's advocate", i.e., they should approach every operation with a critical eye and dig deep to identify areas that need improvement. The IG staff should begin with an assumption that nothing worked correctly until proven otherwise. Only by taking a hard look at all phases of emergency response can the overall emergency system be improved.
RESTORATION AND STRENGTHENING OF DAMAGED BUILDINGS

by

A.I. Martemianov, S.G. Shaginian, T.G. Markarian
(USSR)

In countries of high seismicity greater consideration has been given recently to the restoration and strengthening of buildings. When looking into the maintenance of buildings and evaluating the extent of damage, three main groups of buildings are to be distinguished:

(a) Buildings damaged by an earthquake with the intensity corresponding to the design;
(b) Buildings damaged by seismic waves with higher intensity as compared with the design;
(c) Buildings having no anti-seismic structures and damaged by the earthquake.

The proposed classification permits one to reveal the main causes of damage and to formulate the necessary steps for restoration and strengthening. Following an analysis of the consequences of the December 7, 1988 Spitak earthquake, we can state that the most damaged buildings were residential, cultural and social. These mainly included stone houses of series IA-450, 1-451, and their modifications: frame panel houses of series 111, public buildings and large panel residential buildings of series IA-451 KP, and two frame panel lift slab ones. Returning to the adopted classification, it should be mentioned that the most severely damaged were buildings of the second and third groups, i.e. in Kirovakan and Stepanavan where the intensity was slightly higher than VII, and in Leninakan IX and Spitak X which did not coincide with the design values of standard codes of seismic regions VII, VIII and VII, respectively.

Three main types of damage to residential buildings by the Spitak earthquake can be distinguished. In 5 storey stone buildings with anti-seismic strengthening the following occurred:

- Stability loss of end walls due to unreliable joints with longitudinal walls;
- Fall out of walls due to lack of floor anchorage in carrying walls and bad workmanship of anti-seismic belts;
- Collapse of end sections due to insufficient cross rigidity, horizontal displacement of slabs and disturbance of floor joint plates and carrying walls;
- Weakening of carrying walls by activities which were not projected;
- Separation of wall masonry in a longitudinal direction in some cases due to inadequate joint design and construction.

The failure mechanism of these buildings could be represented in the following order: fall-out of end walls followed by failure of end sections; failure of middle sections due to stability; loss of longitudinal walls having no reliable links with floors. In buildings with a 'flexible' ground floor, the comparatively rigid walls of stairways failed following the failure of columns and exterior walls under a horizontal displacement.
In 9-storey frame panel buildings with anti-seismic strengthening the following damage occurred:

- Failure of stiffening diaphragms due to weak or bad workmanship, bringing about the disturbance of shear wall joints, the building frame, and the total collapse of buildings;
- Failure of exterior suspended wall panels which in turn increased the inertia load;
- Failure of frame joints and prefabricated reinforced concrete elements due to bad workmanship.

The failure mechanism of these buildings can be represented in the following order: failure of stiffening diaphragms brought about horizontal displacements. This was followed by the cracking of concrete in frame joints and columns, insufficient quantity of reinforcement, and poor welding quality of the reinforcing bars. It should be taken into account that the large mass (500 kg/m²) of exterior suspended walls and interior bearing partitions, which exceed the design value, created additional loading on bearing structures. Though the above mentioned is not a decisive factor in the failure of frame panel buildings it is always necessary to use light walls and partitions in the construction of these buildings.

In 9-storey large panel buildings with anti-seismic strengthening no significant damage was observed. Some deformations in the joints of large panel buildings were of an unusual nature. Insignificant damage justify the spatial layout of buildings with closely spaced arrangement of bearing walls, reliable flexibility of joint connections, and apparently reliable and easily controlled quality of anti-seismic strengthening.

In 10-and 16-storey frame panel lift slab buildings with anti-seismic strengthening the following damage occurred:

- Crushing of the stiffening core with walls weakened by piers;
- Displacement of floor with torsion in its plane;
- Displacement of building from the vertical position by as much as 6 degrees.

In low rise stone buildings without anti-seismic strengthening the following damage occurred:

- Separation of external bearing walls followed by collapse of floor parts;
- Separation of angles or vertical cracks in places of wall continuity;
- Damage of bearing walls and partitions in the form of diagonal, cross shaped and horizontal cracks in window sills;
- Separation of masonry due to its poor quality owing to the weak link between the mortar and the stone.

Buildings in Leninakan, Kirovakan, and Stepanavan could serve as a visual illustration of the above. Constructive decisions provide geometrical stability to buildings even for an earthquake with an intensity exceeding the design seismicity of buildings. Constructive anti-seismic measures include reinforcement of masonry, anti-seismic belts, restriction of building size, number of floors, etc. The total cost to strengthen a damaged building is determined by the cost of restoring the building before the earthquake (X₀) and the cost of additional seismic resistance to buildings (X₁). An economic criterion for restoration can be assumed as follows:
XI > Xo: The building should be restored to the state before the earthquake.

XI ≤ Xo: The building should be restored to the stage fully satisfying the requirements of seismic resistance.

Of course, other evaluation methods are quite acceptable but the economic expediency of the choice to restore should be based on the comparison of the cost to restore or strengthen verses the cost of erecting a new building. According to recent assessments by seismologists, intensity at these cities and settlements increased, whereas the erection of these buildings was realized based on the design seismicity adopted beforehand. Cities erected in this way are Yerevan, Tashkent, Alma-Ata, and a few others. So, we are facing a very serious problem of reinforcement of erected buildings in several cities.

For the restoration of buildings damaged by the Spitak earthquake the following recommendations are given:

For Stone Buildings
- When necessary, erect additional longitudinal and cross walls instead of partitions and transverse walls including 'flexible' ground floors;
- Place fine grained concrete of class B12, 5 40-50 mm on a welded mesh for spatial stiffness of floors;
- Spray concrete over the welded metallic screen of damaged bearing walls and partitions on both sides or inject mortar;
- Strengthen narrow partitions up to 100 cm in width and stone columns by metallic pilar casings;
- Arrange joints between exterior and interior walls by stressing metallic belts on floors;
- Replace heavy small sized elements by lightweight large sized elements or strengthen partitions made of small sized elements of plaster over the metallic welded screen;
- Remove and replace separated parts of masonry, etc.

For Frame Panel Buildings
- Arrange additional metallic braces or reinforce concrete stiffening diaphragms in both directions;
- Strengthen reinforced concrete columns and cross bars by metallic casings;
- Strengthen damaged staircase walls by spraying concrete over the metallic grid or inject polymer mortar in cracks as well as arranging additional braces for fixing staircase elements;
- Arrange additional braces to fix exterior wall panels;
- Replace some elements in case of necessity.
For Large Panel Buildings

Arrange connections reinforced by polymer mortar in vertical and horizontal joints. However, along with general recommendations, the main focus in evaluating building maintenance should be on the detailed investigation of each system, and in case of necessity to study the physical mechanical properties of bearing structures, materials, etc. The final design for restoring and strengthening buildings should take into account the above mentioned causes of damage and failure of buildings.

An acceptable level of restoring and strengthening a building should be determined taking into account all the above mentioned facts along with an evaluation of appropriate building materials, adaptability of strengthening methods to manufacture, and, finally, the professional skills of workers and available facilities. A catalogue of the most favourable structural decisions for restoring buildings damaged by an earthquake and recommendations for the evaluation of restoration work should be carried out during the restoration.
REINFORCEMENT OF EXISTING BUILDINGS
by
T. Chachava
(USSR)

Earthquakes and other natural disasters bring about a heavy toll of human lives. Society suffers great material losses and difficult social problems arise. Destructive earthquakes may affect enormous regions and their consequences become a national calamity.

An example of such a calamity is the Spitak earthquake (Armenia, 1988) - which caused the loss of thousands of human lives, left hundreds of thousands of people homeless, and created enormous material damage amounting to millions of roubles. The entire Soviet Union is taking part in the restoration of northern Armenia; and important help is being rendered by the world community. After every destructive earthquake, the question is asked whether it was possible to avoid, or at least to considerably lessen, its tragic effects.

It is said that people are killed not by earthquakes but by buildings, erected to provide protection from the environment and to create comfortable living conditions. For it is the destruction of buildings that lead to the loss of human lives and economic damage. Attempts to find an answer to the above question should be aimed at substantially increasing the efficiency of earthquake engineering through developing methods of seismic prediction as well as those of building design. To this end, normative documents which regulate design and construction in seismic regions need to be updated and improved.

However, in order to considerably lessen the negative effects of earthquakes, attention must also be given not only to planned new buildings but also to existing buildings. The fact is that most towns situated in seismic regions have a number of buildings that do not satisfy present-day requirements of seismic stability. In particular, such buildings include:

(a) Old buildings, constructed with a disregard for the seismic code, frequently have no earthquake resistant properties at all; and their design, in some cases, even contradicts the requirements of seismic stability;
(b) Buildings constructed in compliance with the rules but whose design seismicity is below the norm that is currently accepted for a given locality and therefore demand reinforcement;
(c) Buildings and structures which, though constructed by designs satisfying the seismic code, have, for a number of reasons, deteriorated and hence do not at present satisfy the requirements of seismic stability (among these buildings are those which depart from the seismic code, by being poorly built, from off-grade materials, and which suffer from damage caused by underestimating soil conditions, etc.)
All such buildings demand reinforcement.

An analysis of the technical state of earthquake nonresistant buildings allows one to believe that a small amount of work can save much larger material investments. The determination of the efficiency of the reinforcement of buildings is one of the major tasks discussed in this report. General methods and specific solutions to design problems have been developed for buildings of various types.

Problems of this type should be solved in the context of work on urban reconstruction and planning, based on the principles of economic efficiency. It is necessary to determine the strategy and tactics of the reinforcement of existing buildings and to establish priorities. Because of the scale and complexity of the work, this is best achieved by the use of a computer system. This implies that a package of programs with the required seismic codes must be created in order to solve the above problems using the database obtained as a result of processing the overall screening of constructed buildings.

Brief mention is also made below of other aspects of research connected with the elaboration of a program on the reinforcement of existing buildings. It is important to carry out a detailed visual screening of all existing buildings to ensure a sufficiently simple identification of any inappropriate object (from the viewpoint of its seismo-resistance) within the framework of the existing classification. Such classification should include an assessment of structures based on their quality, the availability of antiseismic measures, etc. Methods of work concerning reinforcement of buildings should take into consideration the following basic points:

(a) Reinforcement of insufficiently reliable structures/foundations;
(b) Reinforcement of insufficiently reliable joints between structures;
(c) Alteration of structural schemes by installing additional links, decreasing the number of storeys, etc., so as to decrease stresses produced by operational load and seismic waves.

These methods of reinforcement should be developed for all buildings included in the classification, and as far as possible they should be included in the initial computerized database. When executing this part of the work, use should be made of the most effective design solution available. Alternative design solutions are possible. The computerized system should include a program package allowing one to calculate the seismic resistance of any of the classified buildings and to determine stresses in the structures.

Thus, to access the seismic resistance of any building one must compare the results of numerical analysis (the repeated numerical analysis necessary to take into account changes in the building design model), and the design solutions, with the seismic requirements for buildings of the type being considered.

The classification and the methods of reinforcement should be carried out by highly qualified experts. Certification data and reinforcement methods should be detailed and formalized so that the computerized system for designing the reinforcement of existing buildings can function in the interactive mode. Using simple 'yes' and 'no' answers, a specialist of av-
average qualifications could identify the structures to be reinforced, and their seismic drawbacks, and choose reinforcement measures from the 'menus' available for buildings of this type.

This approach will guarantee a complete solution for an individual local structure and the preparation of required documentation (containing the specific design solutions with all technical and economic indexes). Finally, by using a computerized system to solve the problem of reinforcement of an individual structure, one can consider problems such as optimizing capital investments in the reinforcement of existing buildings, within the framework of general problems of urban reconstruction and development.

The considerations presented here can be used to elaborate computer programs to reinforce existing buildings in seismic regions in order to reduce the possible heavy losses from future earthquakes. Investigations of this kind are carried out by many researchers in the world (in particular, much progress has been achieved in the USA). However, computerized systems for the reinforcement of existing buildings in seismic regions are very cumbersome and demand thorough analysis and adjustment to specific local conditions. The coordination of the activities of large groups of experts is necessary, and it is suggested that an organization such as UNDRO should undertake this task.
ANNEXES
ANNEX A

USSR/UNDP/UNDRO: TRAINING SEMINAR ON ENGINEERING ASPECTS OF EARTHQUAKE RISK MITIGATION

17 - 28 October 1988

Location: Institute of Seismo-Resistance Construction and Seismology, Tajik Academy of Science, Dushanbe, Tajik, USSR.

Preliminary Programme

17 Oct. Mon. am
Opening - Welcome addresses by representatives of Tajik SSR, USSR Ministry of External Economic Relations, UNDP/UNDRO.

Break

Lecture - 1. Seismic hazard assessments and estimation of largest possible earthquakes, (V. Karnik, UNDRO).

pm

18 Oct. Tue. am
Lecture - 5. Spectra and attenuation of strong ground motion under different ground conditions and at different magnitude levels, (The Tajik Institute for Seismology and Seismic Engineering, Dushanbe, USSR).

Note: Timetable: AM: 9:00-11:00, coffee break, 11:15-12:15; PM: 14:00-16:00, coffee break, 16:15-17:15; Evening Sessions in Hotel Tajikistan - projection of films, demonstration of maps, books, records, etc., round-table discussions - optional.
Lecture - 6. USSR methods of seismic microzoning (O.V. Pavlov, Institute of the Earth Crust, USSR).

Discussion

19 Oct. Wed.  am-pm

Lecture - 9. Practical demonstrations of strong motion records processing, data storage, use of results in earthquake resistant design, strong motion network in Tajikistan (description, calibration of instruments), near-field and far-field observations, (The Tajik Institute for Seismology and Seismic Engineering, Dushanbe, USSR).

Discussion

20 Oct. Thu.  am


Discussion

pm


21 Oct. Fri.  am


Discussion
pm Lectures - 15, 16. Special aspects of earthquake resistant design of different types of constructions (masonry, panel, steel frame, reinforce, concrete, monolith), (The USSR State Committee Ministry for Construction; The Tajik Institute for Seismology and Seismic Engineering, Dushanbe, USSR).

Discussion

22 Oct. Sat. am-pm Lecture - 17. Demonstration of full-scale testing of structures at the Lyaur site, visit to the Geophysical Observatory at Gissar, (The Tajik Institute for Seismology and Seismic Engineering, Dushanbe, USSR).

Discussion


Lecture - 20. Vulnerability of different types of buildings, (J. Petrovski, UNDRO)

Discussion

pm Lecture - 21. Reconstruction of buildings after earthquakes, (J. Petrovski, UNDRO)

Discussion

25 Oct. Tue. am-pm Lecture - 23. Excursion to the Nurek Dam - Lecture on the design of the dam located in a high seismicity zone; examples of monitoring local and induced seismicity of dam structure and behaviour, (The Tajik Institute for Seismology and Seismic Engineering, Dushanbe, USSR; USSR Ministry of Power).
Lecture - 25. Land use and urban planning seismic zones, (UNDRO/USSR).
Discussion

pm Lecture - 26. Excursion in Dushanbe - examples of urban planning, (The Tajik Institute for Seismology and Seismic Engineering, Dushanbe, USSR).
Discussion

Discussion

pm Lectures - 29, 30. Extended lecture on earthquake preparedness including public information and education, earthquake scenarios, social aspects, (S. Suyehiro, UNDRO).
Discussion

Lecture - 33. Presentations by participants on various topics of the seminar, (UNDRO).

pm Lecture - 34. Final discussions, evaluation of the seminar, (UNDRO).
Closing Session
# ANNEX B

LIST OF PARTICIPANTS

UNDRO Training Seminar on Engineering Aspects of Earthquake Risk Mitigation

Dushanbe, 17 - 28 October 1988

<table>
<thead>
<tr>
<th>Name of Participant</th>
<th>Country</th>
<th>Organization</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Dr. Vit Karnik</td>
<td>Czechoslovakia</td>
<td>Geophysical Institute</td>
</tr>
<tr>
<td>2. Mohammad Naim Aslami</td>
<td>Afghanistan</td>
<td>Seismological Institute</td>
</tr>
<tr>
<td>3. Djillali Benouar</td>
<td>Algeria</td>
<td>Centre de Recherche Astronomique</td>
</tr>
<tr>
<td>4. Juan Carlos Rosas</td>
<td>Argentina</td>
<td>INPRES</td>
</tr>
<tr>
<td>5. Jose Alberto Vivas Veloso</td>
<td>Brazil</td>
<td>Seismological Observatory</td>
</tr>
<tr>
<td>6. Peter Sotirov</td>
<td>Bulgaria</td>
<td>University of Architecture</td>
</tr>
<tr>
<td>7. Chen Dasheng</td>
<td>China</td>
<td>Institute of Engineering Mechanics</td>
</tr>
<tr>
<td>8. Mario Mejia</td>
<td>Colombia</td>
<td>Ministerio Minas y Energia</td>
</tr>
<tr>
<td>9. Osvaldo Bebelagua</td>
<td>Cuba</td>
<td>Academy of Sciences</td>
</tr>
<tr>
<td>10. Hugo Yepes</td>
<td>Ecuador</td>
<td>Instituto Geofisico Escuela Politecnica National</td>
</tr>
<tr>
<td>11. Constantinos Ioannidis</td>
<td>Greece</td>
<td>Earthquake Planning &amp; Protection Organization</td>
</tr>
<tr>
<td>12. Murdiati Munandar</td>
<td>Indonesia</td>
<td>Institute of Human Settlement</td>
</tr>
<tr>
<td>13. Najdat A. Mohammad Ali</td>
<td>Iraq</td>
<td>National Center for Engineering</td>
</tr>
<tr>
<td>14. Hassan Asharif Sulaiman</td>
<td>Libya</td>
<td>Public Work Organization</td>
</tr>
<tr>
<td>15. Hamouda Farid</td>
<td>Morocco</td>
<td>Ministère de l’Energie</td>
</tr>
<tr>
<td>16. Alejandro Rivas-Vida</td>
<td>Mexico</td>
<td>Public Works Department of Mexico City</td>
</tr>
<tr>
<td>17. Jchinorovyn Baljinniam</td>
<td>Mongolia</td>
<td>Academy of Science of Mongolia</td>
</tr>
<tr>
<td>18. Enrico Mangao</td>
<td>Philippines</td>
<td>Institute of Volcanology and Seismology</td>
</tr>
<tr>
<td>19. Gheorghe Marmureanu</td>
<td>Romania</td>
<td>Center of Earth Physics and Seismology</td>
</tr>
<tr>
<td>20. Ruchan Yilmaz</td>
<td>Turkey</td>
<td>Earthquake Research Institute</td>
</tr>
<tr>
<td>21. Nguyen Hong Phuong</td>
<td>Vietnam</td>
<td>Institute of Geophysics</td>
</tr>
<tr>
<td>22. Abdul Majid Mohammed Abdul</td>
<td>Yemen PDR</td>
<td>Department of Geology &amp; Mineral Exploration</td>
</tr>
<tr>
<td>23. Abdul Samad Alsa Nabani</td>
<td>AR Yemen</td>
<td>The Supreme Council for Reconstruction</td>
</tr>
<tr>
<td>No.</td>
<td>Name</td>
<td>Country</td>
</tr>
<tr>
<td>-----</td>
<td>-------------------------------</td>
<td>-----------</td>
</tr>
<tr>
<td>24</td>
<td>Petrovic Miodrag</td>
<td>Yugoslavia</td>
</tr>
<tr>
<td>25</td>
<td>Herbert Tiedemann</td>
<td>Germany</td>
</tr>
<tr>
<td>26</td>
<td>Shigeji Suyehiro</td>
<td>Japan</td>
</tr>
<tr>
<td>27</td>
<td>Ludovic Van-Issche</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>Julio Cesar Greico</td>
<td></td>
</tr>
<tr>
<td>29</td>
<td>Lakshman Saran Srivastava</td>
<td>India</td>
</tr>
<tr>
<td>30</td>
<td>Daniel Alonso Domnibvez</td>
<td>Cuba</td>
</tr>
<tr>
<td>31</td>
<td>Fauret Manuel Ismael</td>
<td>Cuba</td>
</tr>
</tbody>
</table>
ANNEX C

USSR/UNDP/UNDRO TRAINING SEMINAR ON LESSONS FROM MANAGEMENT OF RECENT EARTHQUAKES, AND CONSEQUENTIAL MUDFLOWS AND LANDSLIDES

23 October - 3 November 1989

Location: Joint Council on Seismology and Earthquake Engineering,
USSR Academy of Sciences, Moscow, USSR;
Institute of Seismo-Resistance Construction and Seismology, Dushanbe;
and Tadjik Academy of Sciences, Tadjik, USSR

Moscow Programme

23 Oct. Mon. am
Opening - Welcome addresses by the Academy of Sciences of the USSR, USSR Ministry of External Economic Relations, Civil Defense, UNDP/UNDRO, Joint Council on Seismology and Earthquake Engineering, Tadjik Institute of Seismo-Resistance Construction and Seismology.

Coffee Break

1. General introduction to the topics of the seminar (H. Tiedemann, UNDRO).
2. A short introduction to seismicity:
   - Cause of earthquakes;
   - Seismic history, records and uncertainties;
   - The main parameters of earthquakes (magnitude, duration, acceleration, velocity, displacement, intensity). (V. Karnik, UNDRO; F.F. Aptikaev, Institute of Physics of Earth, Moscow, USSR.)

Note: Time-table: AM: 9:30-13:00, PM: 15:00-18:00
3. Differences between earthquakes in different tectonic settings Circum-Pacific belt - Kamchatka, Eurasian earthquake zone, California: magnitude, duration, area and probability aspects (N.V. Shebalin, G.I. Reisner, T.G. Rautian, Institute of Physics of Earth, Moscow, USSR; L.A. Kogan, The Tadjik Institute for Seismology and Seismic Engineering, Dushanbe, USSR.)

4. Earthquake risk and microzoning maps - magnitude, intensity probabilities, site effects, liquefaction, landslides (O.V. Pavlov, Institute of Physics of Earth, Moscow, USSR; M. Trifunac, UNDRO.)

Coffee Break

Discussion
Film about Spitak Earthquake 7 Dec 1988

24 Oct. Tue. am

5. Case studies of various earthquakes:
5.1. Managua 1972 and Friuli 1976 earthquakes (H. Tiedemann)

Coffee Break

5.2. Spitak earthquake 1988:
- Seismotectonics and seismology (I.L. Nersesov, IFZ).
- Engineering consequences (E.G. Khachian, Armenian Institute of Civil Engineering)

Lunch

- The causes of engineering consequences (V.S. Udaltsov, Civil Defense).
- Film about Spitak earthquake
- Rescue management (V.M. Kozhakhteev, Civil Defense)
- Film about Spitak earthquake

Coffee Break

- 242 -
Questions and Discussion

1FZ film on case histories

25 Oct. Wed. am

5.3. Gissar earthquake 1989:
- Seismotectonics, seismology and engineering geology
  (K.M. Mirzoev, The Tadjik Institute for Seismology and Seismic Engineering, Dushanbe, USSR.)
- Film about Gissar earthquake
- Engineering consequences (S.V. Kozharinov, TISSS)
- Rescue management (F.N. Niyazov, Tadjik Civil Defense)

Coffee Break

Questions and Discussion

5.4. Guatemala earthquake 1976
5.5. Montenegro earthquake 1978 (J. Petrovski, UNDRO)
5.6. El-Asnam earthquake 1980

Lunch

26 Oct. Thu. am

6. Emergency management and rescue operations
   (Debris and debris removal, location of victims, methods, tools, equipment, medical facilities): (B.I. Chernichko, Civil Defense USSR)

Coffee Break

6.1. Vulnerability of pre-fabricated concrete structures
   (F. Krimgold, UNDRO)

General discussion

Lunch

pm

7. Main parameters controlling loss and damage:
7.1. Direct damage to elements at risk (magnitude,
duration, time history of shaking epicentral distance, subsoil quality, asymmetry/irregularity, strength of structure, vulnerability of non-structural parts, quality of material, quality of workmanship)
(J. Petrovski, UNDRO)

Coffee Break

Continuation of presentation by J. Petrovski
Questions and discussion

27 Oct. Fri. am 7.2. Indirect loss and damage:
- Consequential damage because of failing structures, buildings, etc: contents of buildings, machinery, goods, etc.
- Landslides, liquefaction, tsunami
- Fires, explosion
- Failure of power, water, telephone service

7.3. Casualties and socio-economic effects
(Main causative factors and consequences)
(H. Tiedemann, UNDRO)

Coffee Break

Continuation of presentation by H. Tiedemann
Questions and discussion

27 Oct. Fri. pm 8. Risk management and optimization
8.1. Inventory (M. Klyachko) (A. Paramzin)
Questions and discussion

Coffee Break

8.2. Predisaster improvement of existing buildings/structures (S. Kozharinov, TISSS)
8.3. Improvement of design and construction standards (H. Tiedemann, UNDRO; F.F. Aptikaev, IFZ).
8.4. Catastrophe/emergency planning
(Lohman, UNDRO)
- UNDRO emergency planning procedure (N. Solomatine, UNDRO)

Coffee Break

8.5. Emergency Management Planning (F. Cuny, UNDRO)

Lunch

pm

9. Training of engineers, contractors, workmen, rescue teams, population (H. Tiedeman, UNDRO)
9.1. Training of population (S. Suyehiro, UNDRO)
10. Conclusions, lessons and suggestions (H. Tiedemann, UNDRO)
General discussion
Closing Session of Moscow part of the Seminar
The object of practical on-site training was to familiarize the participants with the analysis of buildings as regards their vulnerability and the economic possibilities of strengthening the buildings before earthquakes.

This practical part of the Training Seminar was of great importance as it gave the participants a chance to learn on the spot why certain buildings are vulnerable, the type of damage they are likely to suffer, and whether the people in such buildings are exposed. Moreover, on-site training conveyed the various problems resulting from debris, debris removal and rescuing of people from the respective structures. This forms the basis for developing the capacities needed in case of disasters.

30 Oct. Mon. 10:00-11:00 11. General introduction to the problem and purpose of practical on-site training (emphasis of training: risk assessment and improvement)

11:00-13:00 12. Inspection of modern buildings: re-frame, lift-slab, different characteristics - soft, stiff, regular, asymmetrical, analysis of the exposure, economical improvements

Lunch

15:00-17:00 13. Inspection of factories. Analysis of exposure, evaluation of economical improvements.
17:00-18:00 Discussion

31 Oct. Tue 10:00-13:00 Continuation of inspection of modern buildings

Lunch

15:00-18:00 14. Inspection of valuable brick buildings.
18:00-19:00 Discussion
Analysis and improvements.

Lunch

15:00-18:00  15. Inspection of epicentral and slide-region of recent earthquakes; inspection of rural buildings
18:00-19:00  Discussion

2 Nov. Thu  10:00-13:00  Continuation of inspection

Lunch

15:00-17:00  16. Conclusions, lessons and recommendations
Evaluation of the Seminar

Closing Session
ANNEX D

LIST OF PARTICIPANTS

UNDRO Training Seminar on Lessons from Management of Recent Earthquakes including consequential mudflows and landslides

Moscow, 23 October - 23 November 1989

<table>
<thead>
<tr>
<th>Name of Participant</th>
<th>Country</th>
<th>Organization</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atifa Yusufzai</td>
<td>Afghanistan</td>
<td>Council of Ministers</td>
</tr>
<tr>
<td>Joaquim Mendes Ferreira</td>
<td>Brazil</td>
<td>University Natal</td>
</tr>
<tr>
<td>Shi Zhenliang</td>
<td>China</td>
<td>State Seismological Bureau</td>
</tr>
<tr>
<td>Dario Omar Cardona</td>
<td>Colombia</td>
<td>National Office of Disaster Assistance</td>
</tr>
<tr>
<td>Luis Barreras Canizo</td>
<td>Cuba</td>
<td>Civil Defence</td>
</tr>
<tr>
<td>Ezzeldin Mohamed Ibrahim</td>
<td>Egypt</td>
<td>Institute of Astronomy and Geophysics</td>
</tr>
<tr>
<td>Konstantinos Ioannidis</td>
<td>Greece</td>
<td>Earthquake Planning Protection Organization</td>
</tr>
<tr>
<td>Imadal-Din Hussain Al-Shibib</td>
<td>Iraq</td>
<td>Civil Defence</td>
</tr>
<tr>
<td>Keisuke Ataka</td>
<td>Japan</td>
<td>Ministry of Home Affairs</td>
</tr>
<tr>
<td>Shigeji Suyehiro</td>
<td>Japan</td>
<td>UNDRO Consultant</td>
</tr>
<tr>
<td>Bayar Gangaadorjiin</td>
<td>Mongolia</td>
<td>Academy of Sciences</td>
</tr>
<tr>
<td>Raman Rewati Pokharel</td>
<td>Nepal</td>
<td>Home Ministry</td>
</tr>
<tr>
<td>U Thin Than</td>
<td>Myanmar</td>
<td>Department of Meteo/Hydro-Meteorology</td>
</tr>
<tr>
<td>Mohammad Asghar-Khan</td>
<td>Pakistan</td>
<td>Emergency Relief Cell</td>
</tr>
<tr>
<td>Francia Iberica Cedeno</td>
<td>Panama</td>
<td>Ministry of Public Works</td>
</tr>
<tr>
<td>Abdul-Haq Hussain Kassim</td>
<td>PDRY</td>
<td>Geology Department</td>
</tr>
<tr>
<td>Arguedas Madrid Cesar</td>
<td>Peru</td>
<td>Instituto National Defensa Civil</td>
</tr>
<tr>
<td>Ton-Adrian Mihalcea</td>
<td>Romania</td>
<td>Hydrolic Research Institute</td>
</tr>
<tr>
<td>Ali Ali</td>
<td>Syria</td>
<td>Civil Defence</td>
</tr>
<tr>
<td>Ozcan Umit</td>
<td>Turkey</td>
<td>Ministry of Public Works</td>
</tr>
<tr>
<td>No.</td>
<td>Name</td>
<td>Country</td>
</tr>
<tr>
<td>-----</td>
<td>-----------------------</td>
<td>----------</td>
</tr>
<tr>
<td>21.</td>
<td>Fred Cuny</td>
<td>USA</td>
</tr>
<tr>
<td>22.</td>
<td>Frederic Krimgold</td>
<td>USA</td>
</tr>
<tr>
<td>23.</td>
<td>Mihailo D. Trifunac</td>
<td>USA</td>
</tr>
<tr>
<td>24.</td>
<td>Vlado Kuk</td>
<td>Yugoslavia</td>
</tr>
<tr>
<td>25.</td>
<td>Jakim Petrovski</td>
<td>Yugoslavia</td>
</tr>
<tr>
<td>26.</td>
<td>Zoran Milutinovic</td>
<td>Yugoslavia</td>
</tr>
<tr>
<td>27.</td>
<td>Herbert Tiedemann</td>
<td>Germany</td>
</tr>
</tbody>
</table>