The Spitak, Armenia earthquake – Why so much destruction?

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ABSTRACT: For a moderate earthquake, the Spitak, Armenia, earthquake of December 7, 1988 caused unprecedented destruction. Unlike other destructive earthquakes, research to understand the circumstances and conditions that led to the extensive damage from this earthquake has not yet been adequately carried out. The paper pieces together the little research that has been accomplished to-date, to form a comprehensive view of why so much destruction occurred.

1 INTRODUCTION

Although there is much to be learned from quick reconnaissance surveys of damage immediately following an earthquake, initial reactions, especially by the popular media, usually tend to blame construction practices or the higher than expected ground motions as the main cause of the disaster, particularly if the earthquake is centered in a developing country. Most recently this occurred in relation to the Spitak earthquake of December 7, 1988.

The initial and often erroneous reactions are later modified following research that attempts to determine, at a more deliberate pace, what actually did occur. Unfortunately, a frontal investigation did not take place for the Spitak earthquake. And, as a consequence, many questions still remain unanswered.

For a moderate event, the Spitak earthquake caused unprecedented destruction. The objective of the paper is to piece together the little research that has been accomplished to-date, to form a comprehensive view of why so much destruction occurred.

2 EARTHQUAKE PARAMETERS AND DAMAGE STATISTICS

The source parameters given by the US National Earthquake Information Center for the Spitak earthquake are: time, 07h 41m 24.9s GMT (local time 10:41 am); location, 40.94N°, 44.29E°; depth, 10 km; $M_s = 6.8$. Surface breaks were traced over a distance of 13 km in the epicentral region near Spitak. They showed a reverse slip of about 1.6m with a considerable amount of strike-slip component (Kikuchi et al 1992).

Despite its moderate size, this earthquake caused the largest earthquake disaster since the 1976 Tangshan earthquake in China. Official estimates, which the locals claim are grossly underestimated, give the following disaster statistics:

Table 1. Disaster statistics

Dead	25,000
Injured	50,000
Evacuees	More than 100,000
Homeless	500,000

In the epicentral region only (EERI, 1989)

314	buildings collapsed
641	needed to be demolished
1264	needed repair or strengthening
712	habitable after the earthquake

Reconstruction 15 billion rubles (1988 value)

Given the extent of the disaster, it is distressing to note (see Table 2) that the emphasis by the Soviets at the Spitak-88 International Seminar held in Yerevan, May 23-26, 1989 was on seismology rather than on the understanding of the causes of the damage. On the other hand the non-Soviet delegates emphasized the engineering lessons learned. More papers were presented by the Soviets on earthquake prediction than on understanding the causes of the damage. It appears that understanding the reasons for this colossal destruction of modern engineered buildings had not become a prime objective of the Soviets.

Table 2. Approximate breakdown of papers - Spitak-88 International Seminar, May 23-26, 1989, Yerevan

Topic	Soviet Contributions		Other Contributions	
Geology, Seismology	15	(30%)	4	
Strong Motion Data	7		1	
Engineering Lessons, Damage Statistics	7	(14%)	12	(55%)
Societal Response	2		2	
Earthquake Prediction	10	(20%)	—	
Novel Solutions	7		2	
Retrofit of Damaged Structures	2		1	
Total	50	(100%)	22	(100%)

In the absence of definitive and complete research results, it is preferable to discuss the factors that contributed to the damage in terms of likelihoods. Thus, in the subsequent Sections, the several factors judged to have contributed to the damage will be characterized as likely, unlikely, or in between (50/50) candidates. The reader can refer to the summary of these evaluations listed in Table 4 of the Conclusions as each factor is discussed in the subsequent Sections.

3 SEISMIC HAZARD

According to the 1969 Soviet Building Code (SNiP II-A.12-69), on which most of the newer buildings in the felt area were constructed, the general basic design level for all of Armenia was Intensity 8 on the Soviet Intensity scale. Intensity 8 corresponds to horizontal ground acceleration of 0.2g. Since, for each change in intensity level the design acceleration is doubled, Intensity 9 corresponds to 0.4g and Intensity 7 to 0.1g. In the US Uniform Building Code (UBC) Intensity 8 corresponds roughly to design levels for most of the Western US except for California and parts of Utah. In the UBC seismic zonation, design accelerations have a 90% non-exceedance probability in 50 years, or approximately a return period of 500 years.

The 1981 Soviet Building Code (SNiP II-7-81) introduces a refinement of the seismic hazard through the use of subscripts 1, 2 and 3, which refer, respectively, to design accelerations equal to 0.85, 1.0, and 1.15 that of the Intensity without a subscript.

Eisenberg (1991) states that the consequences of the earthquake in Spitak should not come as a surprise: the town was designed to withstand a maximum Intensity 7_1. The change of Intensity for Spitak (and Kirovakan) from 8 to 7_1 occurred in 1982. It is not clear which buildings that collapsed are of post-1982 design. The damage statistics of Table 1 to be meaningful must be modified to reflect pre- and post-1982 designs.

If all the buildings that collapsed can be traced to post-1982 designs then hazard estimation in the Soviet Republics must have been anything but scientific. A simple study of the seismic history of the region would have indicated that Intensity 7_1 is totally inadequate. Figure 1, reproduced from Cluff and Tobin (1989), shows a known major fault going through or very close by Spitak and Kirovakan. To blame seismic zonation (and poor construction) as Eisenberg does, does not square well with the fact that different types of buildings performed differently and certain types performed as expected.

Design accelerations, usually derived from magnitude and distance estimates, do not consider the type of faulting. For the same magnitude event, intensity of ground shaking is more widespread and, hence, less intense for strike-slip faulting; and less widespread and, hence, more intense for thrust faulting. Both the San Fernando earthquake of February 9, 1971 and the Spitak earthquake are examples of the latter. From a design perspective larger ground motions should be specified when seismic zones cannot be clearly identified solely with strike-slip faulting.

Assuming that most of the buildings in Spitak were designed according to the 1969 Soviet Building Code,

which specified Intensity 8 for all of Armenia, the seismic hazard does not seem to have been a significant factor contributing to the extensive damage in this earthquake. M_s = 6.8 is a moderate earthquake and a design ground acceleration of 0.2g is not an unrealistic design basis. However, the thrust faulting could have exacerbated an already bad (due to other factors) situation.

4 MICROZONATION

Despite the minute ($\pm 15\%$) changes in design accelerations for seismic zones with subscripts, microzonation according to the Soviet Building Code follows a rather crude procedure. Given a basic design level, say Intensity 8 (for example Armenia), depending on the type of foundation, the design level can be increased to Intensity 9 (for soft foundations) or reduced to Intensity 7 (for rocky ground of all types). Figure 2 shows the microzonation of Leninakan. Within a distance of 1-2 km, design accelerations vary from 0.1g to 0.4g. This range of variation covers the whole of the continental United States. Although the type of foundation should be considered in determining ground motion intensity, the method used by the Soviet Building Code is too crude a metric for microzonation.

More importantly, the determination of what constitutes rock or soft foundation is not uniformly defined. Even though most of Leninakan is situated on a broad alluvial plain, thin (4-5m) volcanic rock layers at shallow depths are considered to be rock foundations. The more important microzonation consideration in Leninakan should have been the fact that ground motion in the period range of 0.5-2.5 seconds is amplified (Borchardt et al, 1989).

Another peculiarity of Soviet practice relative to the effects of foundation materials on ground motion is discussed by Klyachko and Uzdin (1991): the Soviet Building Code allows the reduction of seismic design levels on high compressible soils by two intensity levels, which, as discussed earlier, is equivalent to a factor of 4 in ground motion accelerations, if a 5m deep artificial pad is constructed with a modulus of 60 MPa.

It is evident that microzonation due to local geology and site conditions is not based on modern geotechnical principles. It is based on a crude concept that is a very likely contributor to the damage during this earthquake.

5 DESIGN CRITERIA

The basic principles of construction in seismic regions as expounded in the Soviet Building Code are sound. Minimization of seismic loads, uniform distribution of rigidities, location of joints outside the zone of maximum forces, facilitating the development of plastic deformations, the selection of construction sites to avoid other soil and foundation related hazards and, finally, the detailed determination of spectral response are all provided for.

Figure 3 shows the response spectra for three different foundation conditions together with the modal response equations. It is to be noted that the type of foundation not only impacts the Intensity level but also the shape of the spectrum. In Table 3, the spectral amplifications for the basic zone of Intensity 8 is compared with the

corresponding UBC spectral amplifications for a 9-story building of 0.65-sec period.

Table 3. Comparison of Soviet and US code design accelerations for a 9-story building (T=0.65 sec) basic zone Intensity 8, 0.2g

Soviet SNiP

Ground Category	I	II	III
Microzonation	7	8	9
Amplification	1.5	1.7	2.0
Spectral Acceleration	0.15g	0.34g	0.80g

UBC (Zone 2B)

Soil Type	S_1	S_2	S_3
S Factor	1.0	1.2	1.5
Amplification	1.5	2.2	2.5
Spectral Acceleration	0.3g	0.53g	0.75g

For the basic case the spectral amplifications are comparable. For soft sites, the UBC accelerations *increase* in stages to a maximum of 0.75g. For the Soviet Code the increase to 0.8g is made in one step. Most significant is the drastic *reduction* of design accelerations for rock foundations. Since the UBC Code envisages only *increase* of spectral accelerations due to softer foundations, a more appropriate comparison with the Soviet Building Code would be Ground Categories of I, II, III with S_1, S_2, S_3, respectively, particularly when it is so easy to justify Category I foundations as discussed in Section 4.

K_1 in Fig. 3 is the seismic reduction factor due to ductile response. For nuclear power plants elastic response is enforced by setting K_1 to unity. For other structures, in which residual deformations could be tolerated, K_1 is set to 0.25. For single-story industrial and agricultural buildings that do not contain valuable equipment K_1 can be further reduced to 0.12.

K_2 is a factor that depends on the type of structural configuration. For example, for soft first stories $K_2 = 1.5$, the maximum value for any configuration. For high-rise structures of more than 5 stories, K_2 increases to its maximum value of 1.5. For skeleton-type buildings this limit is reached for 10-story structures. For large-panel buildings the limit of 1.5 is reached for 13-story buildings.

The Soviet Building Code also considers the importance of structures (Importance Factor) in design by changing the basic zone designation. Thus, key buildings and structures of union and republic significance are increased by one Intensity level, in effect doubling the design level. A similar concept, however, is used to *reduce* the design accelerations for certain other buildings. Thus, a single story factory building with no more than 50 workers and not containing highly valuable equipment can be reduced by one Intensity level, in effect halving the design accelerations. Like the microzonation based on foundation type the use of Intensity levels to consider the importance of structures is too crude a metric, particularly when forces are *reduced* as a result. In the UBC both foundation type and importance factors only *increase* design accelerations.

The rather ad hoc halving of design accelerations by reducing Intensities by one level categorizes the implementation of the Design Criteria as a likely candidate to have contributed to the damage caused by the Spitak earthquake.

6 STRUCTURAL ANALYSIS

Computer based structural analysis is common in the Soviet Republics. Classical frame analysis with rigid joints for lateral loads is well understood and practiced. On a joint project for a kindergarten complex (Hadjian et al) East and West teams produced very similar responses (frequencies, mode shapes, shears and moments). The issue is not the calculational capabilities. It is simply the modeling. Both East and West teams assumed classical rigid frame analysis for a precast concrete structure. But precast concrete structures cannot be made monolithic no matter who is attempting it. This is more true for the Soviet practice that considers diaphragms of planks placed side by side without a reinforced topping as rigid, flimsy beam-column connections as rigid joints and, finally, shear walls whose elements slide on each other as monolithic. Thus, the calculated results may have nothing to do with the actual response of the structure. Classical elastic frame analysis is not a realistic method to find load paths, force magnitudes and displacements for precast concrete structures.

An evaluation of the partially damaged buildings clearly shows that the collapse of long buildings started at the building ends. Although not analytically calculated, accidental torsion has been used in lateral force determination in US codes for a long time. It simply was a quick and easy fix to eliminate the observed damage at the ends of structures. The earlier Soviet codes did not have any such requirements. Later codes require a 2% accidental eccentricity only for very long structures.

The inadequate determination of load paths and seismic induced forces and deflections in precast structures is considered to have been a likely candidate that contributed to the damage caused by the Spitak earthquake.

7 DESIGN CONSIDERATIONS

7.1 Ductility

To achieve ductility US practice assures that reinforcing bars yield before concrete crushes. For this reason US practice does not allow the use of concrete less than 3000 psi and the steel yield is usually limited to 40,000 psi. The Soviet concrete design practice is just the opposite (Hadjian et al 1990). Concrete strength is set at 2000 psi and steel yield is set at about 53,000 psi. Under these circumstances ductile behavior cannot be expected.

Incidentally, the observations as to the rather poor quality of concrete in the damaged structures may not stem from pilferage or bad workmanship but is obtained by design. 2000 psi concrete simply should not be used in seismic design.

7.2 Seismic detailing

The K_1 factor (Section 5) used in estimating lateral loads is set to 0.25 for most buildings. The code, therefore, presumes ductile ·response. Ductile response can be

obtained if special seismic details are incorporated. There is a wealth of knowledge on how to achieve ductility in monolithic reinforced concrete construction. For precast construction achieving ductility is a much more demanding task. This area of seismic detailing is not much researched. However, for precast high-rise construction, the design of the connection should be a primary consideration: not only how to achieve a ductile connection but where such a connection should be made.

Despite the stated purposes of the Soviet Building Code (e.g. location of joints to be outside the zone of maximum forces), the elements of precast structures are usually connected where both shear and moment during earthquakes are maximum. Even with the best of workmanship, these connections are the weakest link in the lateral load carrying system. Once yielding begins at these connections the deformation tends to grow. When the connection fails due to excessive deformation the structure would simply collapse on being loaded laterally. Investigating the innumerable failed structures it is inescapable to wonder if *any* lateral load resistance was ever designed into these structures. Connection details of floor planks to beams, beams to columns and columns to foundations seem to have considered only vertical loads. If K_1 is to be less than unity, proper seismic ductile details must be incorporated, in particular for precast construction.

7.3 *Factor of safety*

In US practice, the factor of safety is introduced by multiplying the loads with Load Factors, α_i, and reducing the resistances by Strength Reduction Factors, ϕ_j. Thus, for the adequate design of a member it is required that

$$\Sigma \, \alpha_i \, L_1 \leq \phi R$$

Assuming that for critical sections the seismic moments would be about 80% of the total, an overall load factor in US practice would be about 1.3.

For non-special structures ϕ has the following values. Lower values are specified for special ductile moment resistant structures.

Axial tension with flexure	$\phi = 0.9$
Axial compression for tied columns	$\phi = 0.7$

In Soviet practice, the load factors are relatively small and, for ease of comparison, an overall load factor of 1.0 will be assumed. No explicit strength reduction factor is used (Hadjian et al, 1990). Thus, the ratio of US safety factor to Soviet safety factor for columns is $\frac{1.3}{1.0} \times \frac{1.0}{0.7} = 1.8$, and for beams $\frac{1.3}{1.0} \times \frac{1.0}{0.9} = 1.4$.

These are not insignificant ratios. Considering the above three design related shortcomings of the Soviet practice, it is concluded that design considerations were highly likely contributors to the damage during this earthquake.

8 QUALITY OF CONSTRUCTION

Much has been said, in the popular media, about the deficiencies in the materials of construction in the structures that collapsed in this earthquake. This may be true but is relatively insignificant because precast construction elements are lifted into position. If strength of concrete and/or amount of reinforcing bars were so deficient as to be the major cause of the destruction, then they should have failed during the construction process. The fact that they survived lifting and construction loads and that they served their purpose for vertical loads until the earthquake, leads one to conclude that this situation could not have been that serious.

Despite the above assertion plenty of evidence exists to characterize concrete technology as a likely contributor to the damage. As discussed earlier concrete strength by design is 2000 psi. The situation is worsened by the lack of aggregate gradation, inclusion of granite chunks, and significant amounts of fines in the concrete mixes. There is no need to invoke pilferage to explain the poor quality of concrete.

In one plant for prestressed floor planks it was observed that the ends of the planks, where shear would be a maximum, were systematically of poor quality. This was not due to workmanship, but rather to the method of fabrication. Each plank was fabricated individually and it was very hard to achieve adequate compaction at the ends. Thus, the ends ended up being porous and vulnerable. In US practice, floor planks are produced in long beds and are cut to size as required. The two end pieces are cut off and discarded for the same above reason, that of the difficulty to achieve adequate compaction.

Welding of bars from the viewpoint of strength does not seem to have been a problem. The issue with welding of bars has to do whether butt welds were substituted for lapped welds thus creating hard points and detracting from ductile performance.

9 CITY PLANNING

If the concentration of buildings of the same design in one location is avoided, the likelihood of most all of these buildings failing together, as was the case in this earthquake, would be much less. This is so because spatial variation of ground motion and subtle foundation differences would tend to modify the response of these same-design buildings throughout a city. Obviously this could work in reverse also, in that, if collapse of all buildings in one location would have been avoided because of site specific motions and foundation conditions, locating these buildings throughout a city would cause some of them to collapse. However, comparing the experiences of the Mexico (1985) and Armenia (1988) earthquakes, distributing of same-design buildings throughout a city is a prudent seismic decision. It is concluded that the cookie-cutter city planning was a likely contributor to the damage in this earthquake.

10 UNUSUAL GROUND MOTION

This section is not considered with Section 3 because of its significance and implications.

10.1 *Eyewitness accounts*

Talking with survivors of the calamity, it became very clear that 1) the earthquake was of long duration, 2) buildings collapsed quite late in the shaking, 3) and occupants did not have the opportunity to leave the buildings before their collapse. Goenjian (1989), during his many visits on psychiatric missions to the survivors agreed to explore the possibility of two events in succession with the survivors. He writes "people who described to me their activities then makes me think that the span was approximately 40 to 50 seconds between the two events". "Apparently many buildings collapsed after the second quake. One would think that people would have had plenty of time to flee from the buildings in five minutes..."

10.2 *Seismological evidence*

A recent paper (Kikuchi et al, 1992) provides the seismological evidence for the two earthquakes in succession. The paper investigates the source complexity of the Spitak earthquake. The authors show that the Spitak earthquake was a series of four subevents. The last subevent is a small contributor to the total seismic moment and is ignored. As shown in Fig. 4, Subevents No. 1 and 2 form the first group of strike-slip events. The effective rupture duration is $19 - 4 = 15$ sec. The third event initiating at about 30 sec after the initial break is a dip-slip event. The effective rupture duration is $59 - 32 = 27$ sec, somewhat longer than that of the initial strike-slip faulting. The scaler seismic moment of the third subevent is 5.6×10^{25} dyne-cm, nearly 40% of the total. The magnitude of the third subevent is 6.4 (Kanamori, 1990). The important feature of this rupture sequence is the late north-south compression thrust subevent which occurred about *40 sec after* the beginning of the sequence.

The authors conclude that the mechanism of the afterslip event is consistent with the present-day deformation pattern in the Caucasus, where north-south compression is predominant. Mechanism changes appear to be relatively common and important aspects of large earthquakes.

10.3 *Misinterpretation of the Ghoukassian records*

Strong ground motion was recorded in the town of Ghoukassian which is located 27 km north of Leninakan and 33 km northwest of the epicenter. The accelerograph seems to have special peculiarities that must be considered before making conclusions about what the records really represent. The record starts with a 25-sec long straight line — the equipment ran but did not record. Then comes a 3-sec gap, a characteristic of the recording system, possibly due to a film advance mechanism. This follows by the first recording of 25-sec duration. Then a second 3-sec gap occurs. This is followed by a 25-sec long line similar to the starting segment, after which the third 3-sec gap occurs again. And finally, a second ground motion recording. A simple arithmetic shows that the first record starts at 28 sec and the second record at 84 sec after the instrument was activated. Thus, the difference in the starting times of the two records is 56 sec, or 50 sec if we ignore the 3-sec gaps. There is nothing in the Ghoukassian record to

indicate that the second record represents, as reported in the EERI (1989) reconnaissance report, an aftershock of 5.8 magnitude that actually did occurr 4 min 21 sec after the main event. In fact, time separation of the two records, the seismological evidence of a third subevent, and the eyewitness accounts, all, very strongly, suggest that the second Ghoukassian record is nothing but the third subevent as determined by Kikuchi et al (1992). The magnitude of the third subevent is 6.4 (Kanamori, 1990), which is more in line with the recorded accelerogram than a 5.8 magnitude aftershock.

10.4 *Implications for design*

Despite the claim by Kikuchi et al (1992) that fault mechanism changes appear to be relatively common, the resulting double earthquakes that occurred in northern Armenia, both of significant magnitudes and following one another within about 30-40 sec is not commonly considered, if at all, by the design profession. Since both groups of events were of similar intensity, the first one degraded the buildings (foreseen by building codes) for the second event to simply demolish them (not foreseen by building codes) before the occupants could have evacuated the buildings. Had the third subevent been an aftershock 4-5 min later, then the number of deaths and injuries would have been significantly less even though the total destruction would have been the same.

11 CONCLUSIONS

Based on the available data to the author several factors are categorized as to their likelihood of having contributed to the enormous damage in this earthquake. These factors and their associated likelihoods are listed in Table 4.

Table 4. Likelihood of factors contributing to damage during the Spitak earthquake of December 7, 1988.

Sect.	Factor	Very Unlikely	Unlikely	50/50	Likely	Very Likely
3	Seismic Hazard			├──────┤		
4	Microzonation			├───────┤		
5	Design Criteria	├─────────────────┤				
	- Principles	├───────┤				
	- Implementation		├──────────┤			
6	Structural Analysis		├──────────┤			
7	Design Considerations		├─────────────┤			
8	Quality of Construction	├─────────────────┤				
	- Workmanship		├──────┤			
	- Concrete Technology		├──────────┤			
	- Welding of Rebars	├─────────┤				
9	City Planning		├──────────┤			
10	Unusual Ground Motion					

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Figure 2. Microzonation map of Leninakan.

$$S_{Ik} = K_I K_2 S_{oIk}, \qquad S_{oIk} = Q_k A \beta_I K_{\psi} \eta_{Ik},$$
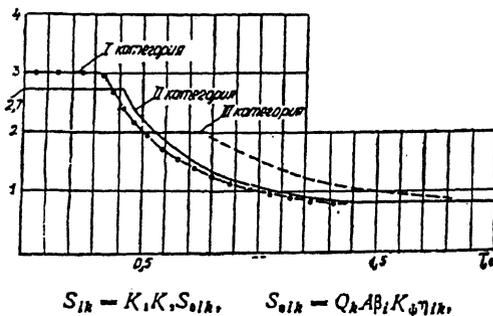
Figure 3. Design response spectra and modal response equations (from SNiP II-7-81).
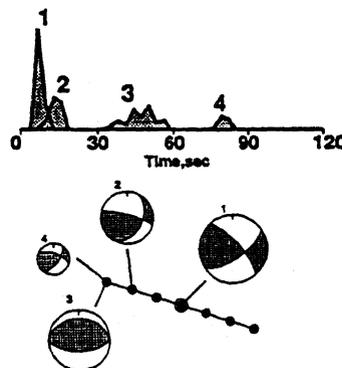
Figure 4. Final solution of the inversion and a base system of double-couple elements. The radius of the mechanism diagrams is proportional to the seismic moment (from Kikuchi et at, 1992).
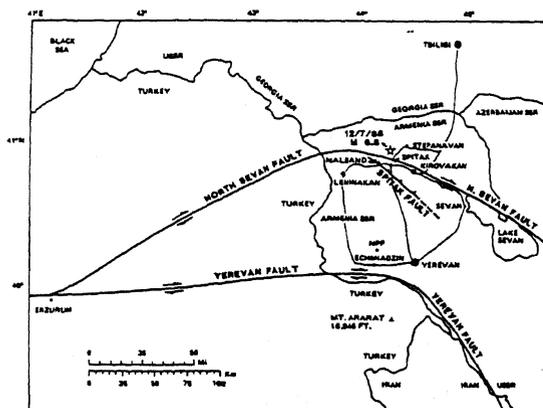

Figure 1. Regional map of Armenia and surrounding area showing epicenter of Dec. 7, 1988 earthquake. Regional tectonics and faults are from Armenian Geological Institute maps (from Cluff et al, 1989).
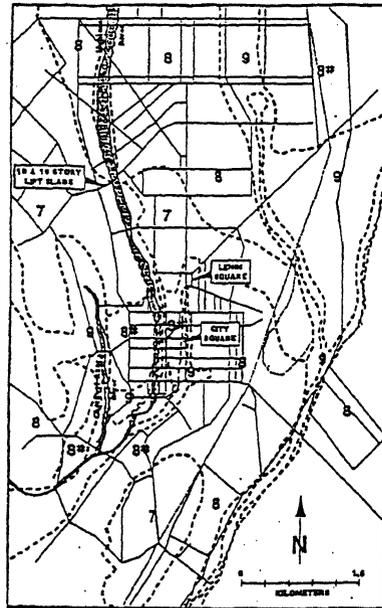
10