Vol. XIX 72 Dicembre 1976

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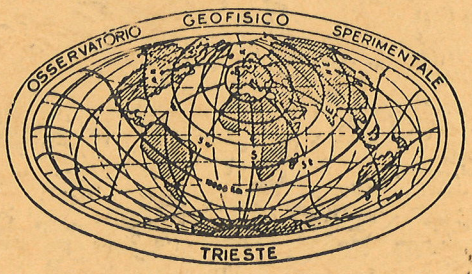

NUMERO SPECIALE:
ATTI DEL CONVEGNO INTERNAZIONALE
SUL TERREMOTO DEL FRIULI

Udine 4-5 Dicembre 1976

SEZIONE 2

INGEGNERIA SISMICA

SPECIAL ISSUE:
PROCEEDINGS OF THE INTERNATIONAL
MEETING ON THE FRIULI EARTHQUAKE

Udine, December 4-5, 1976

SECTION 2

SEISMIC ENGINEERING

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In memory of the 1000 victimes of the 1976 Friuli Earthquake.

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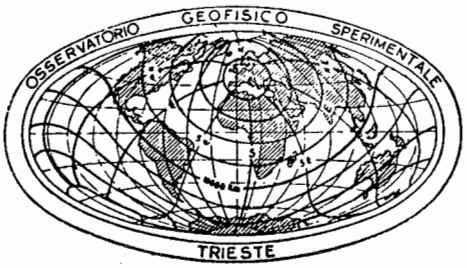

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PREFACE

Soon after the May 6, 1976 earthquake, many Italian and foreign scientists and seismic research groups organized study expeditions to Friuli. After the strong September aftershocks the interest in the seismicity of the Friuli region became even stronger. At present numerous different studies are still carried on, since the seismic phenomenon is not yet over.

Even if the results are obviously not definite, a vast quantity of observations have been collected. These observations will provide the basic material for further research- a valuable help for the future drawn from the painful experience of the past.

The publication of this information as articles scattered in the international scientific literature would however inevitably lead to dispersion and delay. It is for this reason that a number of Italian scientists decided to organize this Meeting. All those engaged in research in Friuli were invited to present the data they had collected, together with any further work that might have been completed so far. Thus, through the Proceedings of this Meeting, all those interested in seismic problems will have access to the most complete collection of information so far available on the 1976 Friuli earthquake.

The Meeting has been also organized for a second important reason: to give the scientists an opportunity to compare the collected data and discuss any conclusions that might have been reached. The Organizing Committee is aware of the fact that there may be overlapping and even contradictions in some of the information presented here. This is by no means strange, as it falls in the normal reserach dialectics.

The Proceedings, presented in Author's alphabetic order, are divided into the following two sections:

1. - SECTION 1: SEISMOLOGY, GEOPHYSICS AND GEOLOGY
2. - SECTION 2: SEISMIC ENGINEERING.

The Organizing Committee wishes to take this opportunity to thank all those who have supported this initiative, particularly CISM for its organizational help. A particular thank is due also to OGS (Osservatorio Geofisico Sperimentale) of Trieste for having dedicated the present issue of the Bollettino di Geofisica Teorica ed Applicata to the Proceedings and for the financial support granted. Our warmest thanks goes of course to all those scientists who have participated in this Meeting and who responded with such enthusiasm to the invitation to present their work united in this way.

Udine, December 5, 1976

THE ORGANIZING COMMITTEE

GENERAL INDEX

Section 1: SEISMOLOGY, GEOPHYSICS AND GEOLOGY

AMATO A., BARNABA P.F., FINETTI I., GROPPI G., MARTINIS B., MUZZIN A.: *Geodynamic Outline and Seismicity of Friuli-Venezia Giulia Region* . . .

AMBRASEYS N.N.: *Crustal Deformation after the Friuli Earthquake, 1976* . . .

ARIC K., GIESE P., MILLER H., MORELLI C., NICOLIC R.: *Crustal Structure and Seismicity of Northern Italy* . . . . . .

ARIC K., GUTDEUTSCH R.: *Remarks about the Structure of the Earth's Crust Near the Friuli Area* . . . . . .

ARSOVSKI M., MIHAILOV V., CVIJANOVIC D., KUK K.: *The Relation between Seismological and Neotectonic Characteristics of Friuli and Neighbouring Areas* .

BARTOLE R., BERNARDIS G., GIORGETTI F., RUSSI M., NIETO D.: *Earthquake Catalogue of Friuli-Venezia Giulia Region (from Roman Epoch to 1976)* . . .

BASILI M., BERARDI R., MURRI F., POLINARI S., TINELLI G., ZANETTI L.: *Strong Motion Records Relative to the Friuli Earthquake. Accelerogram Processing and Analysis* . . . . . .

BASILI M., IACCARINO E., TENAGLIA G.: *Macroseismic Aspects and Seismological and Statistical Considerations* . . . . . .

BEMMELEN (van) R.W.: *Note on the Seismicity of North Eastern Italy (Friuli Area)* .

BERNARDIS G., GIORGETTI F., NIETO D., RUSSI M., SUHADOLC P.: *Seismicity of Friuli-Venezia Giulia Region. Space Epicenter Distribution from Roman Epoch up to 1976. Maximum Observed intensities of the Last Century* . . . .

BONAFEDE M., BOSCHI E., MULARGIA F.: *Stress Corrosion Theory of Crack Propagation Applied to the Earthquake Mechanisms. A Possible Basis to Explain Two or More Large Seismic Shocks in a Geologically Short Time Interval* . . . .

CAPUTO M.: *The Area of the Fault, the Dislocation, the Stress Drop and the Seismic Moment of the Friuli Earthquake of May 6th, 1976* . . . . . .

CARDAMONE P., CAVALLIN A., MARINO C.M.: *Linears Study in Friuli by Means of Landsat 2 Images* . . . . . .

CHIARUTTINI C., ZADRO M.: *Horizontal Pendulum Observations at Trieste* . .

COLAUTTI D., FINETTI I., NIETO D., PUPIS C., RUSSI M., SLEJKO D., SUHADOLC P.: *Epicenter Distribution and Analysis of 1976 Earthquakes and Aftershocks of Friuli* . . . . . .

CONSOLE R.: *Focal Mechanisms of some Friuli Earthquakes, 1976* . . . .

EBBLIN C.: *Orientation of Stresses and Strains in the Piedmont Area of Eastern Friuli, NE Italy* . . . . . .

EMSC (European Mediterranean Seismological Centre) Working Group: *Revised Hypocenters and Magnitude Determinations of Major Friuli Shocks, 1976* . . .

FINETTI I., FRINZI U., SCOTTI A.: *Preliminary Spectral Analysis of Aftershocks of the 1976 Friuli Earthquakes* . . . . . .

FINETTI I., GIORGETTI F., HAESSLER H., HOANG T.P., SLEJKO D., WITTLINGER G.: *Time-Space Epicenter and Hypocenter Distribution and Focal Mechanism of 1976 Friuli Earthquakes* . . . . . .

GASPARINI G.: *Space and Time Distribution of 1976 Friuli Earthquakes* . . .

GEBRANDE H., MILLER H., SCHMEDES E.: *An Attempt of Focal Depth Determinations for Friuli Aftershocks by Means of Lateral Arrivals Using Realistic Models of Crustal Structure* . . . . . .

GIORGETTI E.: *Report on the Hydrogeological Survey carried out by the Government Special Delegation for Friuli* . . . . . .

GIORGETTI F.: *An Example of Microseismic Zoning in the Madonna di Buia Area* .

GIORGETTI F.: *Isoseismal Map of the May 6, 1976 Friuli Earthquake* . . . .

GRAF R., MAYER ROSA D.: *Macroseismic Investigations and Maps for the Friuli Earthquake of May 6, 1976* . . . . . .

HAKUMO MOTOHIKO: *Topographical Effect on the Distribution of Earthquake Damage* . . . . . .

KARNIK V. PROCHÁZKOVÁ D., RUPRECHTOVÁ L., SCHENKOVA Z., DUDEK A., DRIMMEL J., SCHMEDES E., LEYDECKER G., ROTH° J.P., GUTERCH B., LEWANDOWSKA H., MEYER ROSA D., CVIJANOVIC D.: *Macroseismic Effects of the Friuli Earthquake of May 6, 1976 in Austria, Czechoslovakia, F.R.G., France, Poland and Yugoslavia* . . . . . .

MAGRI G., MITTEMPERGHER M.: *Research on the Relationships between Surface Geology and Seismically Active Tectonic Structures* . . . . . .

MAISTRELLO B., MARCELLINI A., PETRINI P., RAMPOLDI R.: *Response Spectra Analysis for Different Sites in Friuli* . . . . . .

MARTINIS B.: *The Friuli Earthquake, May 6, 1976: Geology* . . . . . .

MAUGERI M.: *Geotechnical Aspects on the Recent Friuli Earthquake* . . . . .

MAYER ROSA D., PAVONI N., GRAF R.: *Aftershocks Statistics and Fault-Plane Solutions on Friuli Earthquakes 1975 and 1976* . . . . . .

MIHAILOV V.: *Strong-Motion Records of the May 6 and September 15, 1976 Friuli Earthquakes Obtained by the Yugoslav Strong-Motion Instrument Network* . . .

PAPASTAMATIOU D.: *May 6, 1976 Friuli Earthquake: Field Measurements* . . .

RIBARIC V.: *The Earthquake in Friuli, May 6, 1976 - A contribution to the Study of Seismic Fields in Slovenia* . . . . . .

RITSEMA A. REINIER: *Preliminary Analysis of Friuli Earthquake Records* . . .

ROULAND D., PETERSCHMITT E.: *On the Computation of Focal Mechanisms of Friuli Earthquakes* . . . . . .

SCARASCIA S., COLOMBI B., GUERRA I., LUONGO G., SLEJKO D.: *Preliminary Report on the Recording of Some Aftershocks along the Profile Gemona-Trieste* .

SIRO L.: *Liquefaction of Sands in Friuli during the May 6 and September 15, 1976 Earthquake* . . . . . .

SCHMEDES E., MILLER H., GEBRANDE H.: *vP/vs Ration and Implications for Hypocenter Determinations of Aftershocks of the Friuli Earthquake, May 6, 1976* .

TOKUYAMA A.: *Crustal Deformation after the Friuli Earthquake, 1976* . . .

VUKASHINOVITCH M.: *The Level of the Macroseismic Field for the Territory of Friuli*

VUKASHINOVITCH M.: *The Soil Acceleration Excited by the Earthquake of May 6* .

VUKASHINOVITCH M.: *Seismic Shakeability and Seismic Risk for The Territory of Udine* . . . . . .

VUKASHINOVITCH M., NEDELJKOVITCH S.: *The Duration of Oscillation as a Factor in the Earthquake Intensity Determination* . . . . . .

VUKASHINOVITCH M., NEDELJKOVITCH S.: *The Estimation of the Degree of Building's Damage According to the New Seismic MSK-64 Scale* . . . . .

BRAMBATI A., CORSI M., GIORGETTI F., ONOFRI R., STEFANINI S., ULCIGRAI F.: *Geological Emergency Interventions in the Earthquake Stricken Zones of Friuli: the Example of Osoppo Town* . . . . . .

Section 2 - SEISMIC ENGINEERING

BAYÜLKE N.: *Lateral Force Analysis of a Simple r.c. Structure Damaged in May 6, 1976 Friuli Earthquake* . . . . . .

BENEDETTI D., VITIELLO E.: *Ductility of r.c. Buildings during the Friuli Earthquake*

BENEDETTI D., VITIELLO E.: *Implicit Design Loads of Friuli r.c. Damaged Buildings*

BO G.M., DE STEFANO S., MANCUSO P.L., MANZONE G.: *Behaviour of Some Simple Structures during the Friuli Earthquake* . . . . . .

BOSTJANCIC J., SHEPPARD P., TERCELJ S., TURNSEK V.: *Use of a Modelling Approach in the Analysis of the Effects of Repairs to Earthquake-Damaged Stone-Masonry Buildings* . . . . . .

BRAGA F., PARDUCCI A.: *Elastic-Plastic Degrading Oscillators Behaviour under Earthquakes - Preliminary Results* . . . . . .

BRANCALEONI F., PETRANGELI M.P.: *Dynamic Investigation of the Effects of Non-structural Elements on a r.c. Building in Friuli* . . . . . .

BRISEGHELLA L.: *Damages to Bridges and Roads in the 1976 Friuli Earthquake* .

BRISEGHELLA L., CAPELLARI L., DALL'AGLIO B., D'EREDITÀ R., GORI R., SIMONI L., TURRINI G., ZAUPA F.: *Earthquake in Friuli (Italy), 1976. Damage to Historical Monuments and other Buildings of Artistic Interest* . . . .

CARTAPATI E., CHERUBINI A.: *Recently Constructed Masonry Buildings* . . .

FAORO G.F., FERRAZZA G.: *Report on Damages and Provisions Attaining to Earthquake Concerning the Electrical System* . . . . . .

GIUFFRÈ A., MENEGOTTO M., PINTO P.E.: *Collapse of a Non-seismic Resistant r.c. Building during the Friuli Earthquake: ASTRO Building in Maiano* . . .

GLAUSER E.C., HEIMGARTNER E., SÄGESSER R.: *Assesment and Interpretation of Building Damage* . . . . . .

HEIMGARTNER E.: *Behaviour of Damaged Buildings under Strong Earthquakes* .

IZUMI M.: *Damage in Friuli Earthquake 1976 - Buildings in Osoppo* . . . .

OLSZAK W.: *On the Inelastic Behaviour of Structures During Earthquakes as Exemplified by the Friuli catstrophe 1976* . . . . . .

PETRINI V., SETTI P., COLOMBO G., RAFFA C.: *On the Behaviour of a r.c. Building with Suspended Floors during the Friuli Earthquake in May, 1976* . . .

PETRINI V., VITIELLO E.: *Dynamic Analysis of Friuli r.c. Damaged Buildings* . .

PISTONE G., ROCCATI R.: *Notes on Old Masonry Buildings Structural Behaviour* .

LATERAL FORCE ANALYSIS OF A SIMPLE REINFORCED CONCRETE STRUCTURE DAMAGED IN MAY 6th 1976 FRIULI EARTHQUAKE

NEJAT BAYÜLKE

Chief of Structural Engineering Section(x)

Analysis of a simple reinforced concrete structure located approximately between GEMONA and OSOPPO has been carried out to determine the equivalent lateral static force required to cause the observed damage. It has been calculated that the lateral force required is in the order of 35 % g, with the fundamental period of vibration of the structure, a point on the response spectrum of the ground motion is obtained.

(x) Earthquake Research Institute, Ministry of Reconstruction and Resettlement, Yüksel Caddesi No.7/B Kızılay-Ankara/TURKEY

ANALYSIS

After the May 6th 1976 Friuli Earthquake a team of Turkish experts from the Ministry of Reconstruction and Resettlement has been sent to the disaster area and field investigations were carried out on the days of May 22-23,1976. During these investigations a damage reinforced concrete structure located approximately beetween the towns of Osoppo and Gemona were observed. Figure-1. This was a structure under construction. It had composed of two rows of eleven reinforced concrete columns supporting precast beams. Apparently this was going to be a warehouse with large span beams supported by these two rows of beams and columns in the longer direction of the structure. The longer direction of the structure was oriented in the North-South direction. During the earthquake all of the beams resting on the columns, but not connected as yet to the columns, had fallen down. Moreover many of the columns had bending failures. One of these columns was damaged as shown in Figure-2. The column shows typical bending failure. While on the compression side there is crushing of concrete, on the tension side there is tension crack at the base of the column. Tension failure of the column occured on the northern side of the column while the compression of concrete on the southern face.

Due to the simplicity of the structure and the nature of damage it was considered that the estimation of the lateral force required to cause the observed damage would be possible. The resulting lateral force/ weight of the structure ratio could be considered as an apparent equivalent static lateral force coefficient for this structure. Since this structure had a period of vibration, the resulting force coefficient and period of the structure provides a point on the approximate ground motion response spectra for this location.

The structure were composed of two rows of eleven cast in place reinforced concrete columns supporting precast beams. The spans of the beams were 7.50 meters and the columns were 4.00 meters high.At the time of the earthquake, the beams were resting on the columns. The columns. The beams had closed steel loops at their ends, and through which the longitudonal bars of the columns were passing. These column bars and the loops at the ends of the beams had not been welded and the concreting of the beam column joints had not been done at the time of the earthquake. The detailed pre-earthquake situation of the beam column joint was as shown in Figure-3. Each column was supporting two adjacent beams, at the time of the earthquake the supports was behaving like sliding supports. Since there was no connection between adjacent columns through beams, it is assumed that each column was acting alone while supporting the one half of the weight of the two beams.

It is assumed that each column was acting like an inverted cantilever beam with a concentrated mass at its free end. The cross-sectional detail of the column is as shown in Figure-3. The weight of the column and the load coming from the beams were calculated as 7.50 tons. The concrete compressive and steel yield strengtns were assumed as 200 kg/cm^2 (2850 psi) and 4000 kg/cm^2 (57000 psi) respectively and using the formula below

$$N_o = 0.85 . f'_c . A_C + A_s . f_y$$

the axial force capacity of the column was calculated as 450 tons. The axial force ratio of the column was found as 7.5/450-0.02 With such a low axial load, it can be assumed that the column was damaged in almost pure bending.

With given cross sectional and material properties and using ultimate strength methods, the bending moment capacity of the column is calculated as 1040 ton-cm. Assuming that the lateral earthquake force (P) was acting at the tip of the column, and under these conditions the moment (M) at the column base would be M= P.L, L being the height of the column. With known M and L values, P comes out as 2.6 tons. Thus the equivalent lateral static force coefficient would become C=P/N= 2.6/7.5= 0.35. Thus the lateral static equivalent acceleration coming to this structure was 35% g. This value is alower boundary, because after the formation

of the plastic hinge structure cannot take higher forces but can resist them by permanent deformations.

The period of vibration of the inverted cantilever was estimated using the formula $T = 2\pi\sqrt{m/k}$ where k is taken as $3\ E\ I/L^3$ and m=0.00765 ton-sec^2/cm and E= 200.000 kg/cm^2, I section modulus of inertia, L=height of the column. When these values are substituted, the period of the column comes out as 0.35 seconds.

INTERPRETATION

Within the margin of precision inherent in the assumptions made the lateral equivalent static force coming to a structure with a period of 0.35 seconds was 35% g. This form of damage was not observed in all of the columns. There was some columns without any cracking but the majority of them exhibited some form of bending damage. Initially it had been considered that the columns would have a much lower period of vibration so as to assume that the lateral equivalent static force could be taken as the true peak ground acceleration. Apparently this was not so.

Since there was no strong ground motion recorders within the epicentral area, the magnitude of the earthquake forces encountered during the earthquake is not known. This kind of analysis of damaged structures would provide valuable information on the magnitude of earthquake forces that could be acting on structures. The above values could be interpreted as one point on the response spectrum of the ground motion after assuming a damping factor.

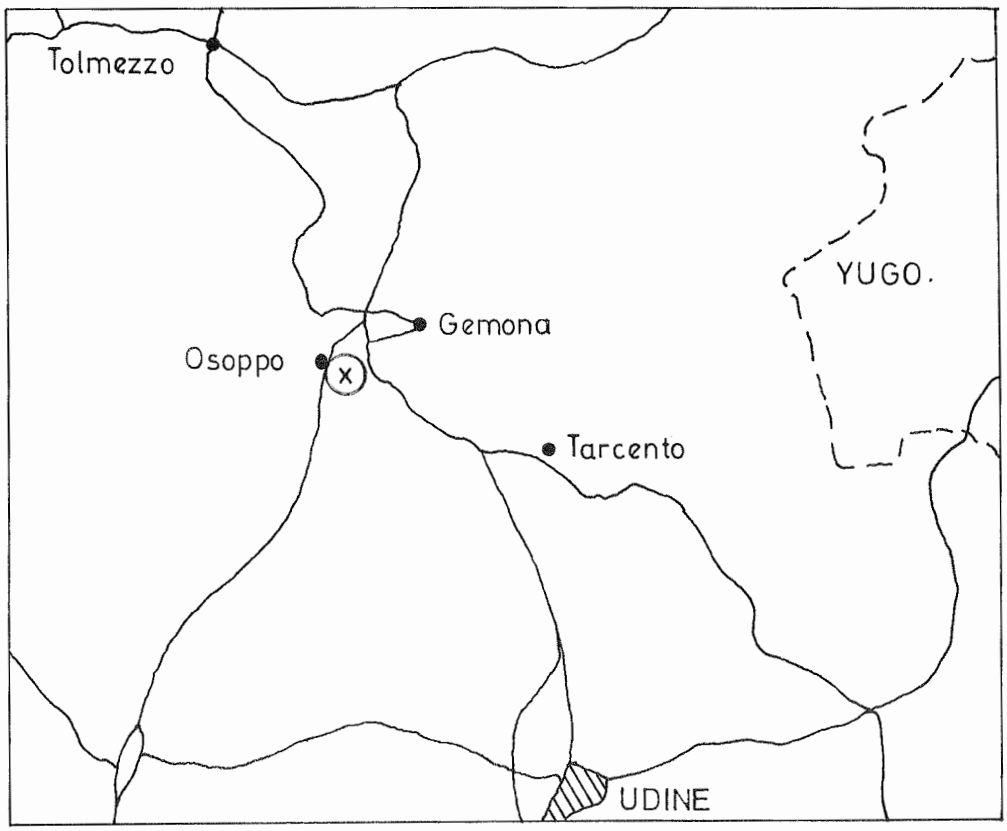


Figure-1 Location of the Analyzed Structure

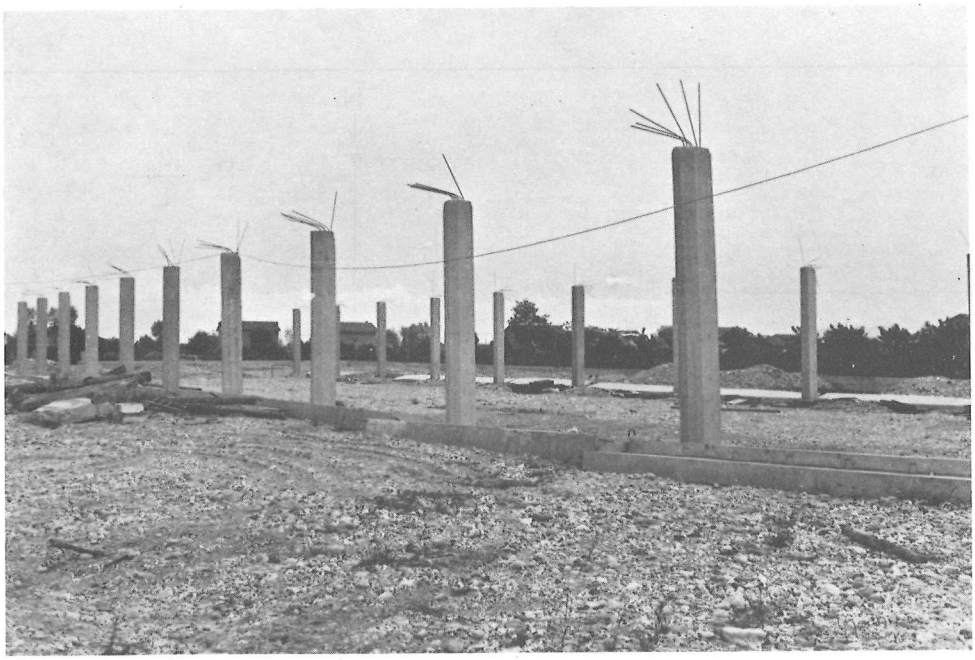


Figure-2 a General View of the Damaged Columns



Figure-2b Bending Damage in Analyzed Column

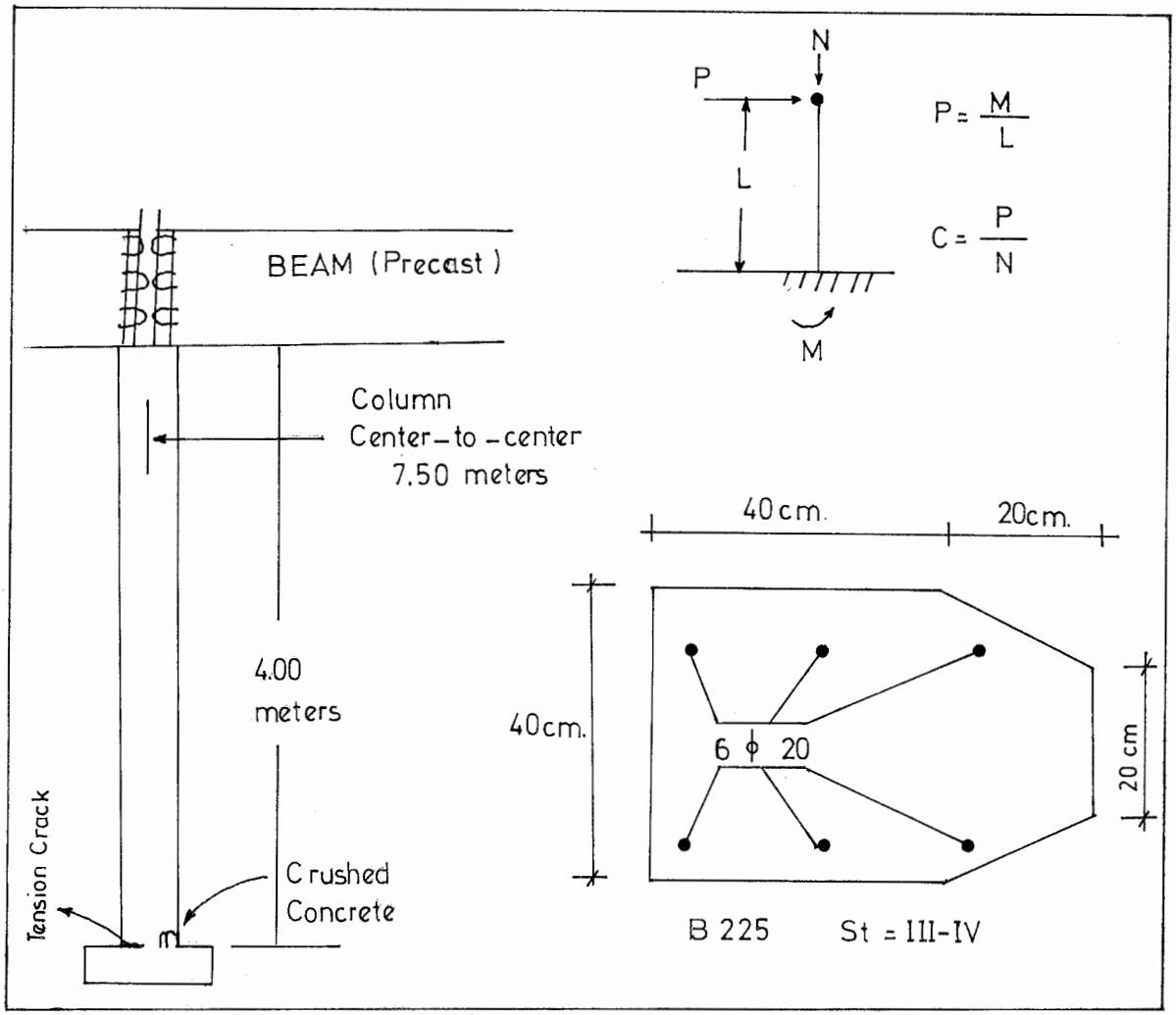


Figure-3 Sketch of the Analyzed Column

DUCTILITY OF R.C. BUILDINGS DURING FRIULI EARTHQUAKE

by D. Benedetti and E. Vitiello
Istituto di Scienza e Tecnica delle Costruzioni - Politecnico di Milano - Italy.

1. INTRODUCTION

Experience of past strong earthquakes, and again of Friuli earthquake, shows that structures leave the elastic range and go into the plastic one. In this stage the energy dissipated by the structural system increases, while displacements become significant. Moreover a change in the stiffness of cracked elements occours which produces a variable lateral force distribution among vertical stiffening elements.

The need of a post-linear analysis for structures to be erected in seismic areas is thus evident; however due to the complexity of non-linear dynamic analysis and to the difficulty of its implementation among practical engineers, simpliefied tecniques have been proposed to this aim. Basically they try to extend linear results to the non linear field. The simplest method of this type is called the ductility factor method [1] . Basically it consists in the following assumptions 1) design a structure to be within

Research carried out in the frame of CNR's Italian Geodynamics Project. Pubbl. n. 27.

the yield limit under an earthquake design spectrum ; 2) the same structure does not collapse up to the design intensity amplified by a coefficient depending on the ratio s^*_u/s^*_e (ductility factor) where s^*_e is the limit elastic and s^*_u the ultimate displacement. Note that this approach is consistent whith the response spectrum technique which seismic codes are based on. It turns out that the determination of single elements ductility is an important step: much research work has been carried out on this topic. Referring to reinforced concrete elements, literature allows an acceptable determination of ultimate sectional rotations as a function of normal load acting on the element, of reinforcement and of the cross sectional area [1,2,3] .In practical problems,where structures have many elements with different ductility, the problem of determing the total ductility factor of the structure arises. Many other factors must be considered in addition to the member ductility. Among them the most important seem to be the followings:

(a) torsion of the overall building;(b) P-Δ effect due to horizontal displacements experienced by the system, (c) variation of axial forces and therefore of ultimate column displacement due to overturning moment on the building,(d) changes in the element force distribution when the external loads increase due to fragile behaviour of some elements (brick walls).

In this paper four different structural systems which were heavily damaged during Friuli earthquake have been analized in order to make clear the influence of above parameters on the overall ductility of the systems. The four buildings have been described in more detail in ref. [4] .

The buildings taken into account can be considered to be in their ultimate conditions. The building behaviour under extreme loads has been reproduced by means of a numerical simulation in which flexural behaviour of vertical elements has been represented by a 2-stage curve (fig.1). A rigid slab was assumed connect all the vertical element and the load was applied in small incremental steps. Inertia forces are disregarded. In this way it became possible to follow single changes in stiffness of single members, due to

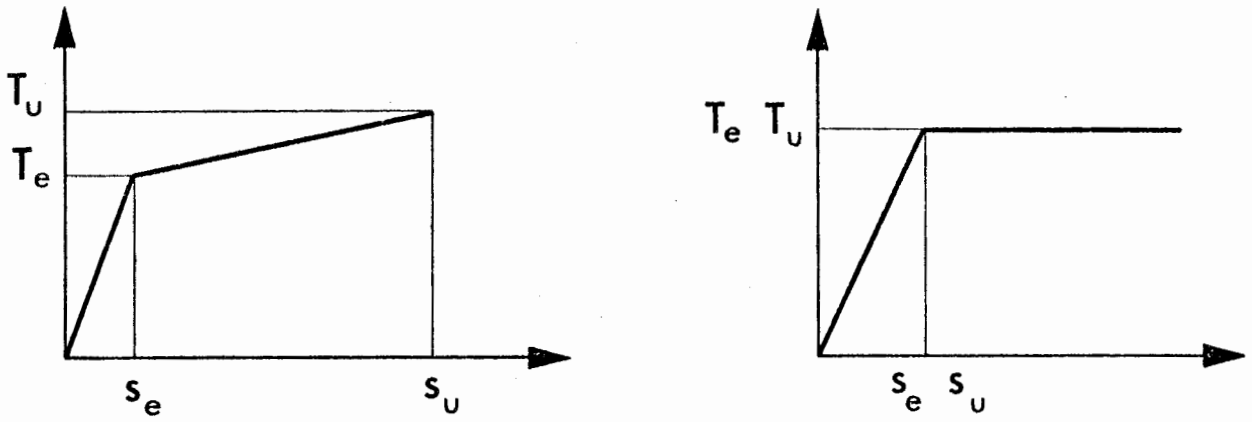


FIG. 1

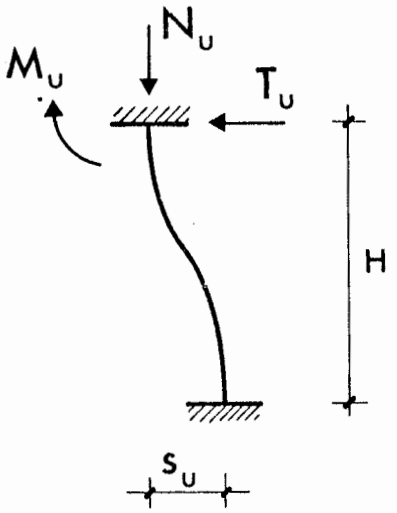


FIG. 2

The same theory leading to M_u allows the determination of both ε_b (concrete strain when M_u is attained) and K (percentage of the r.c. section that is compressed when M_u is attained). The ratio $\varphi_p = \varepsilon_b/(K.h)$ (where h is the total effective height of the cross section of the column) can be assumed as the relative rotation for unit distance of two plane cross sections. The column at limit state forms plastic hinges at top and bottom, the extension of which is suggested to be approximately h by ref. [1] . Therefore the value of

$$s_u \cong \varphi_p \,.\, h \,.\, H = (\varepsilon_b H)/K \qquad (5)$$

can be assumed with the boundaries of the above approximations.

4) T_u = column ultimate shear force. By calling N_u the axial load simultaneously acting on the column when the ultimate limit state is reached, the following equilibrium equation holds (fig.2):

$$T_u = \frac{2\,M_u - N.s_u}{H} \qquad (6)$$

Note that eq.(6) takes into account P- Δ effect.

The behaviour of brick wall panels has been represented by a two stage elastic plastic curve (fig.1).The maximum bearing capacity of the wall has been determined with reference to shear stresses equal to 6 Kg/cm^2 acting on the effective thickness of the wall. After having represented each column by the scheme just described it turns out that the whole system may be seen as consisting in a rigid slab horizontally connected to the ground by a number of non-linear springs (two for each column). Global structural behaviour has been analysed by means of an incremental procedure.External horizontal forces have been applied in steps; within each step the system was assumed to behave linearly,each

The mechanical behaviour of each column is represented by two curves of the type shown in fig.1, each of them refers to a principal direction of the column. The points characterizing each plots are the followings:

1) T_e = column limit elastic shear force. It must be in between the uncracked concrete shear force and the shear force producing yielding in concrete or steel. T_e is here defined as:

$$T_e = (2 M_e) / H \qquad (1)$$

where H is the column height and M_e is the moment producing limit allowable stress in the r.c. column section in bending. The determination of M_e is done by standard r.c. elastic theory (see for istance ref. [4] .Allowable stresses adopted here are referred to Italian r.c. codes and field data on the materials of the actual building in study).

2) s_e = relative top-bottom displacement of the column when the shear force T_e acts. Here it is calculated referring to moment of inertia I of the uncracked section adopting a low Young's modulus E=200000 Kg/cm^2

$$s_e = \frac{H^3}{12\ EI}\ T_e \qquad (2)$$

3) s_u = the ultimate column displacement. It is determined in the following way . In the ultimate condition top and bottom moments of the column attain the ultimate moment M_u. The definition of M_u, given the r.c. section and the standards of the materials, is done by the limit state theory of reinforced concrete. Ref. [4] summarizes the procedure. According to the Italian r.c. codes the limit strain are here assumed as:

$$\varepsilon_{au} = 0.01 \quad \text{for steel} \qquad (3)$$

$$\varepsilon_{bu} = 0.0035 \quad \text{for concrete} \qquad (4)$$

the reaching either of their yielding point or of their ultimate conditions. Redistribution of lateral loads could be automatically achieved and torsional and P-Δ effects fully represented.

The behaviour of brick walls has been included in the model by means of an elastic-plastic type curve: when cracks occour in the wall no incremental contribution to lateral resistance is given.

Results allow to obtain the yielding point of the overall system and the load level for which global ultimate conditions are attained, thus determining the overall ductility of the building under examination .

For each building the following parameters have been computed:

(a) The displacement field for which the structural system leaves the linear range: this will be characterized by the top displacement s^*_e of the column which first reaches the limit elastic shear force .

(b) The displacement field for which the system enters its limit state: this will be made in terms of the top ultimate displacement s^*_u of the column which first attains its ultimate shear force T_u .Note that s^*_u may refer to a column different from the one where s^*_e occurred.

The ratio s^*_u/s^*_e is assumed as the global ductility index of all the structure.

Together with the displacement ratio the analysis provides the horizontal load factor ratio P_u/P_e, where $P_{(.)}$ are the external horizontal loads producing the above mentioned displacements s^*_e and s^*_u.

2. STRUCTURAL ANALYSIS METHOD

All the buildings show the scheme of a rigid block (reinforced column frame with extensive brick walls and partitions) supported by a series of columns at ground floor.

member contributing with its tangent stiffness. The following features have been implemented into the numerical model: (a) tangent stiffness depends on the actual axial load on the colum through the dependence of s_u and T_u on N; (b) yielding in one direction has been assumed to produce simoultaneous yielding in the orthogonal direction too; this may allowe for the presence of common corner reinforcement.

3. RESULTS AND COMMENTS

Building 1: 4-storey building in Avilla. In fig.3 the plan of ground floor is represented. Torsion is prevented by the two lateral walls; in the same figure the values of s_u for each column are shown. It turns out that East-side columns first reach their ultimate state for a displacement $s_u^* = 1.54$ cm; this is due to the fact that they are the shortest columns in the building. The second series of elements reaching their ultimate state consists in the central row of columns ($s_u = 1.76$ cm); while the third row attains $s_u = 3.32$cm.

The difference between the values of s_u shown by the last two rows of columns (which have the same size and height) is due to the fact that the axial load acting on the central row of columns is much larger than the one acting on the lateral row. This feature has been discussed in [1] and [4]. The real behaviour of the building is in good agreement with previous considerations. Infact (fig.4) West-side row (close in the pircture) suffered minor damage; East-side row (bottom in the picture) suffered heavy damages, while one of the central columns was completely destroyed.

Building 2: 7-storeys apartment building in Maiano(fig. 5). Horizontal load was assumed to act in South-North direction. Column 1 is the first to go beyond the linear range ($s_e^* = 0.26$ cm) when the external load is $P_e = 66$ t (equivalent to 11.26 % of the vertical loads). Ultimate conditions are first attained by columns 1 and 5 ($s_u^* = 3.46$) .In this situation external load was $P_u = 106$ (18.1% of vertical loads). The global ductility of the system can assumed to be $s_u^*/s_e^* = 13.3$.

Building 3: 4-storey building in Artegna.Fig.6 shows the ground floor plan. Note the presence of brick walls (lateral walls and stair-box) which give rise to a marked torsional effect. Three analyses have been carried out, namely: (a) horizontal forces acting parallel to the longest side of the building; (b) h.f. acting in the orthogonal direction; (c) h.f. in the same direction as before but no walls considered. Results are summarized in the following table.

	case(a)	case(b)	case(c)
s^*_e (cm)	0.35	0.33	0.36
s' (cm)	0.21	0.22	/
s^*_u (cm)	4.32	4.11	4.23
s^*_u/s^*_e	12.3	12.45	11.75
P_e/W	0.083	0.105	0.085
P'/W	0.210	0.178	/
P_u/W	0.236	0.218	0.14
P_e/P_u	0.354	0.481	0.61

Being W the total vertical load, s' the wall ultimate top displacement and P' the corresponding horizontal load. Comparison between results of case (a) and (b) shows a slight difference in terms of displacements. This can be explained by the fact that in both cases a torsional effect is born which engages almost in the same way the farthest columns. Comparison between (b) and (c) results points out the influence of walls. In the case (c) (no walls) torsional effect becomes negligible and ultimate displacements are almost simultaneously reached by all the columns. The lack of walls results into a lower extimation of the ultimate bearing capacity. Note that in practice walls collapsed, so the latter analysis seems the most appropriate to investigate ultimate

behaviour.

Building 4: 3 storeys dwelling in Gemona. Fig.7 shows the ground floor plan.The presence of an infilled wall at only one side gives rise to strong torsional effects. The following values have been computed:

$s_e^* = 0.3$ cm (columns 16,17,18)

$s_u^* = 3.38$cm (columns 8,16,17,18,26)

$P_e/W = 0.10$

$P_u/W = 0.23$

4. CONCLUSIONS

Above numerical analysis indicates a global ductility factor rather constant for all the buildings taken into account ($s_u^*/ s_e^* \simeq 12$). The ratio between the ultimate horizontal force and the limit elastic force P_u/P_e ranges for 1.7 to 3. Torsional effects due to walls show to produce in the model an increase of the above ratio. $P-\Delta$ effect has a remarkable influence on both s_u^*/s_e^* and P_u/P_e, while it reduces the ultimate values.

Numerical analysis and field data show a reasonable agreement: measured permanent displacements range between 4 a 6 cm. In addition different actual damages can be explained by different theoretical values of member ductilities.

5. ACKNOWLEDGEMNTS

Thanks are due to Prof.G. Grandori for hints and to G. Nicolini for assistance in computations.

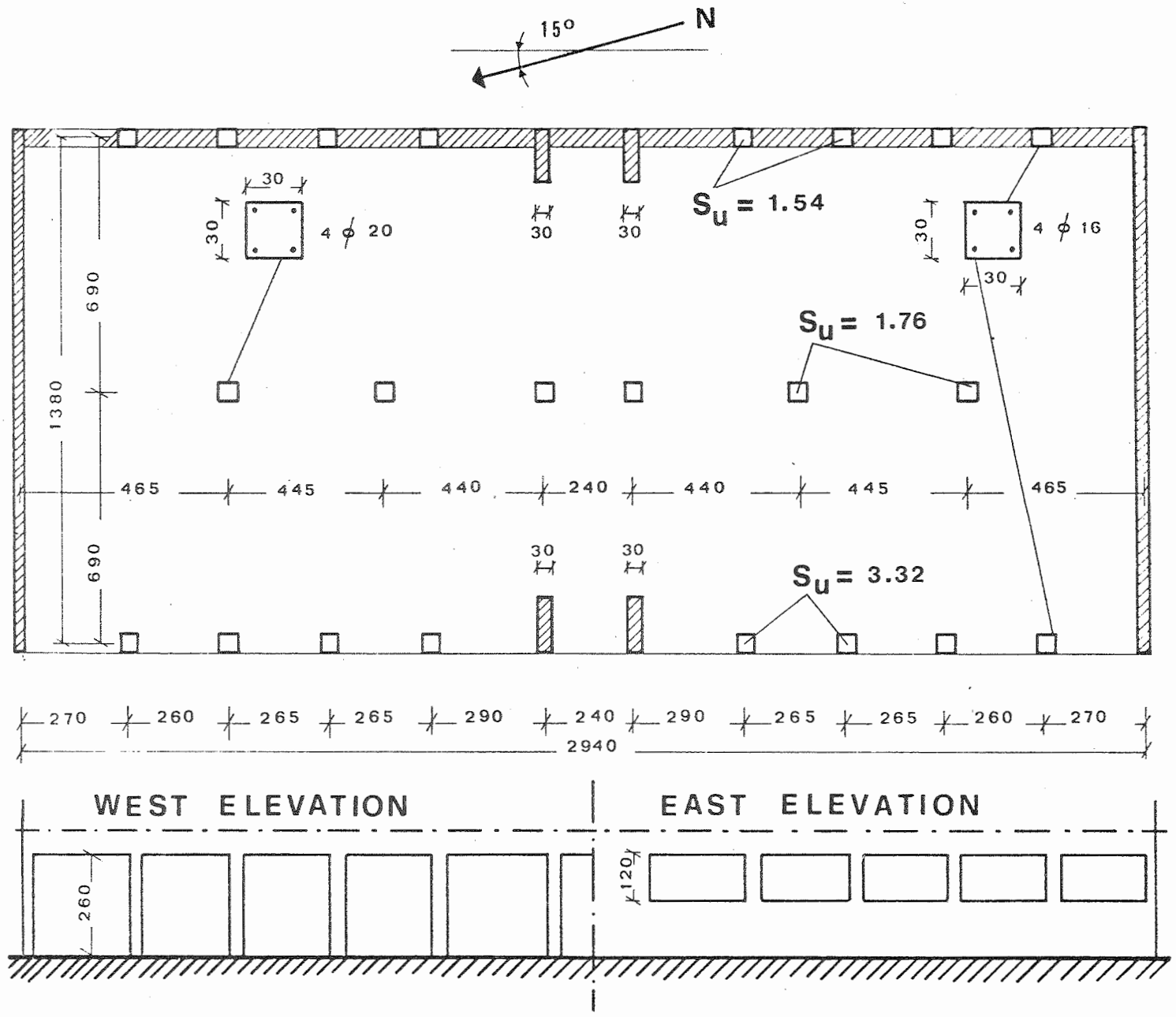

15°
N
30
30
4 ϕ 20
30
30
S_u = 1.54
30
4 ϕ 16
30
S_u = 1.76
690
1380
465
445
440
240
440
445
465
690
30
30
S_u = 3.32
270
260
265
265
290
240
290
265
265
260
270
2940
WEST ELEVATION
EAST ELEVATION
260
120

FIG. 3

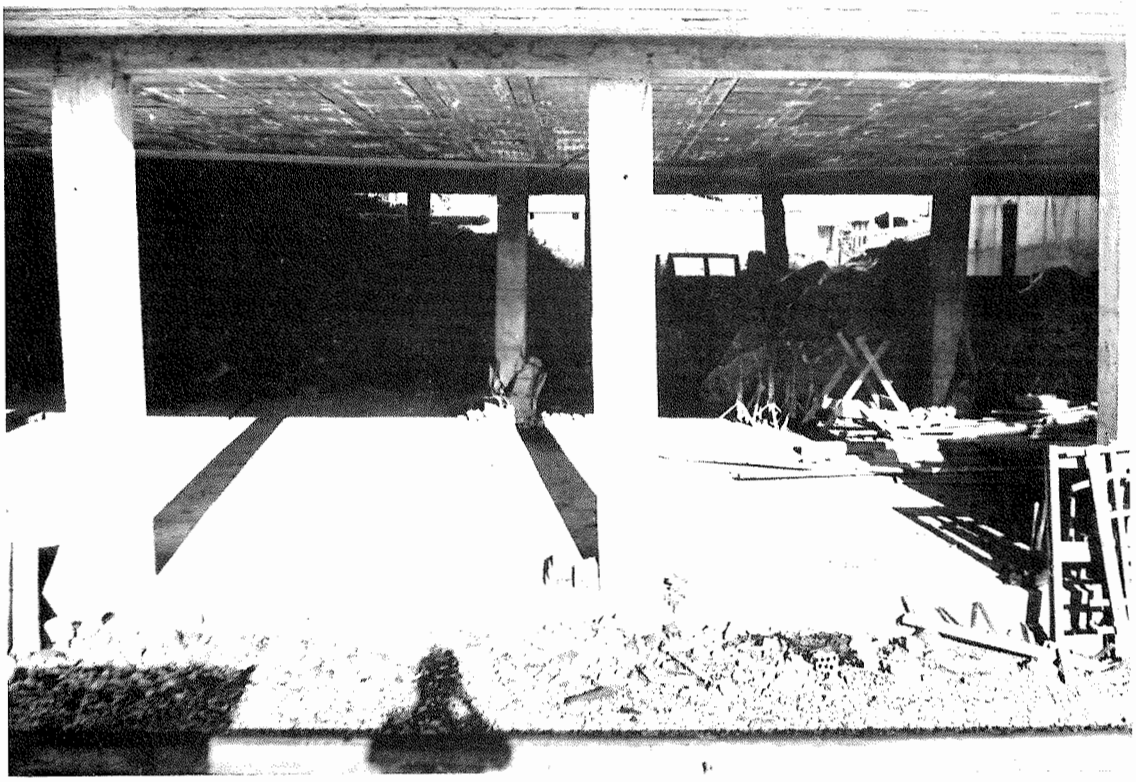
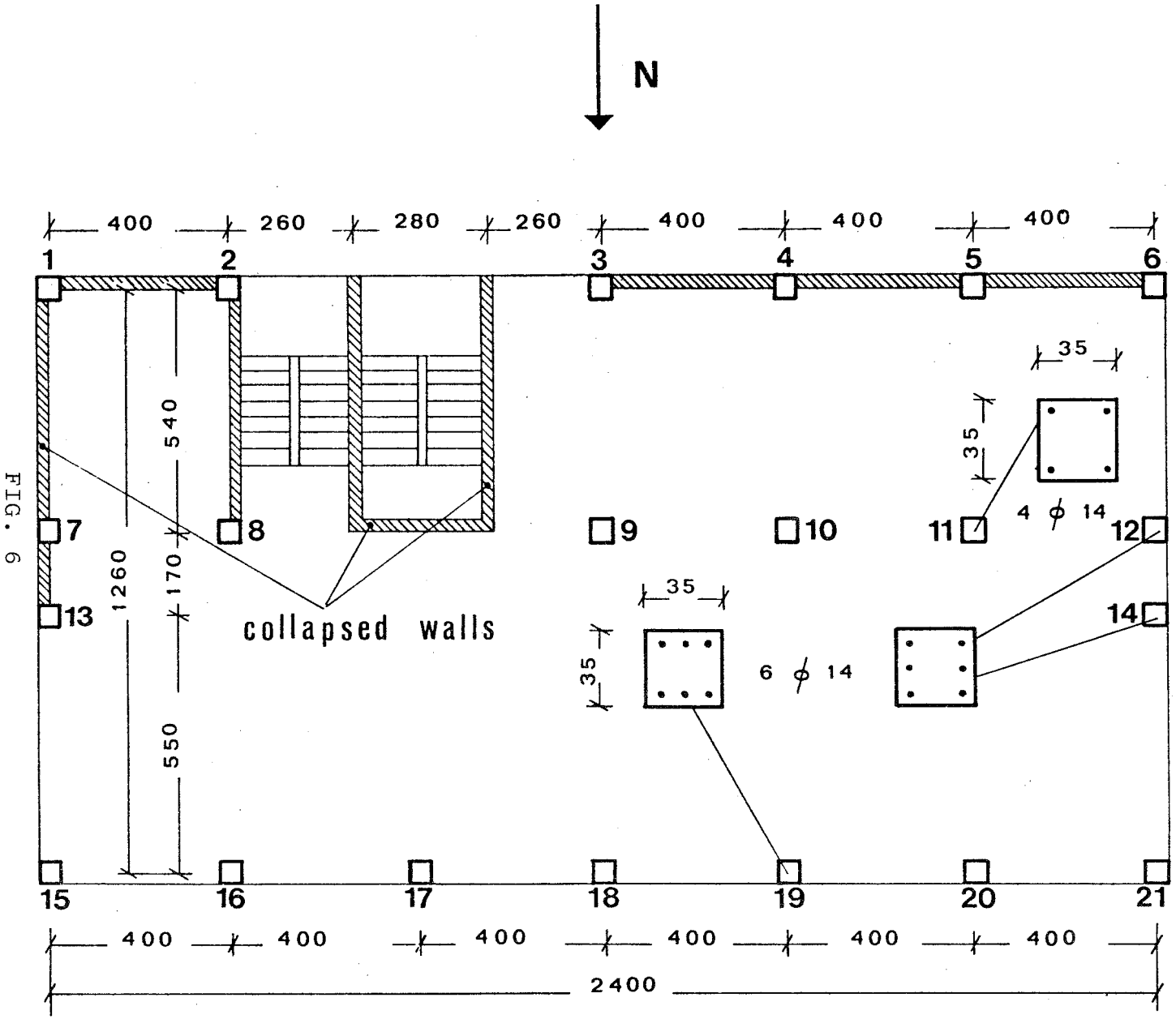


FIG. 4

N
20
21
22
23
24
25
joint
8 ϕ 16
35
35
13
14
15
16
17
18
19
290
590
1275
395
6
7
8
9
10
11
12
35
40
8 ϕ 16
1
2
3
4
5
300
300
300
260
345
265
1770

FIG. 5


N
400
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280
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5
6
35
35
540
4 ϕ 14
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8
9
10
11
12
1260
170
collapsed walls
13
35
35
6 ϕ 14
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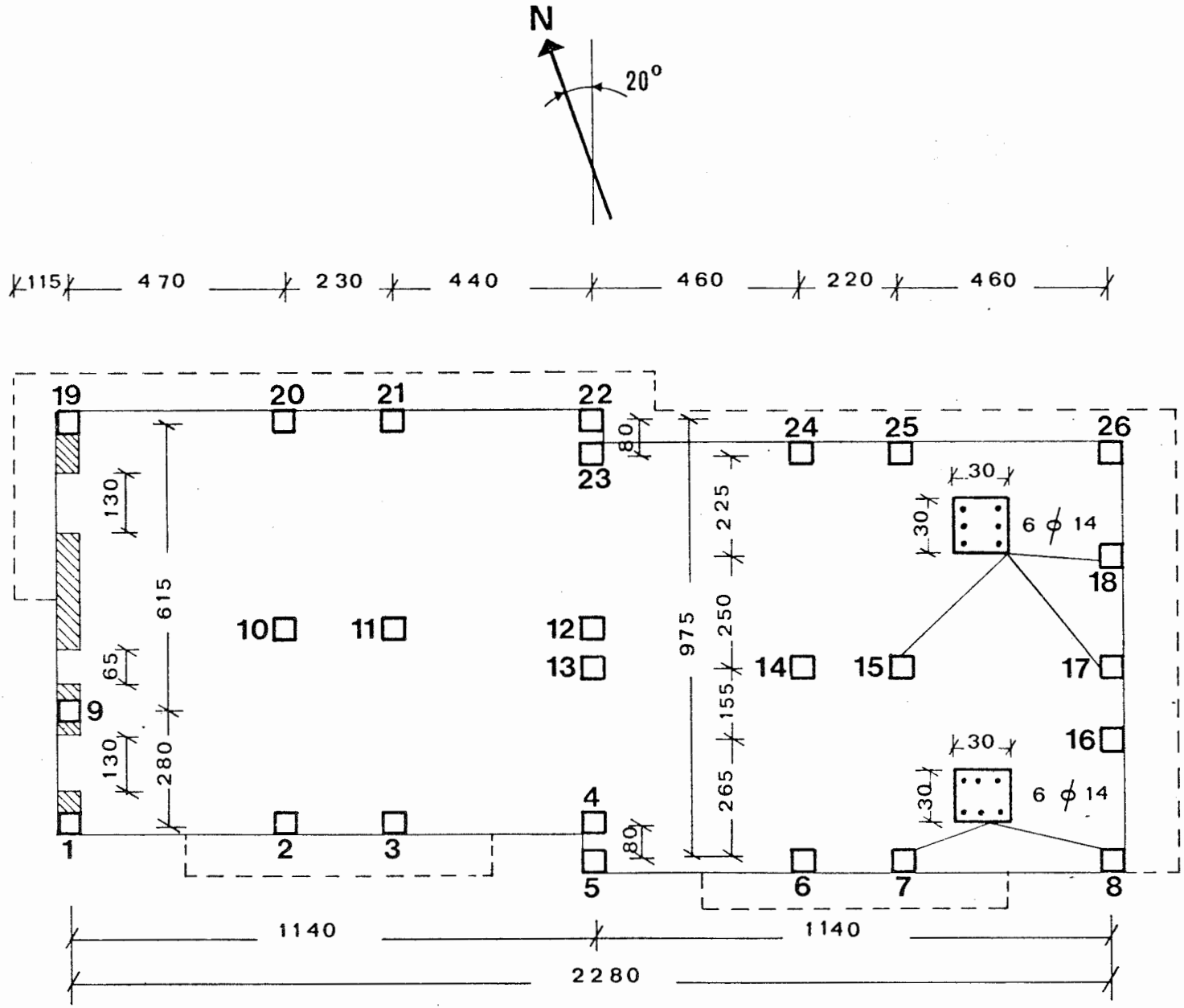
21
400
400
400
400
400
400
2400

FIG. 6


N
20°
115
470
230
440
460
220
460
19
20
21
22
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24
25
26
18
17
16
15
14
13
12
11
10
9
1
2
3
4
5
6
7
8
130
615
65
130
280
80
975
225
250
155
265
80
30
30
6 ϕ 14
30
30
6 ϕ 14
1140
1140
2280

FIG. 7

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4 D. Benedetti-E. Vitiello, Implicit design loads in Friuli r.c. damaged buildings, Proceedings. Int. Symposium on Friuli earthquake, Centre International des Sciences Mecaniques, Udine, 4-5 dec., 1976.

IMPLICIT DESIGN LOADS OF FRIULI R.C. DAMAGED BUILDINGS

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1. INTRODUCTION

The area hit by seismic activity during the Friuli earthquake of May 1976 is only partially included into the provision of the Italian seismic code. Therefore many, although recent, buildings have not been designed to withstand relevant horizontal forces. However field analysis showed that these buildings were able to withstand a certain amount of horizontal forces. This is due to the fact that buildings, particularly reinforced concrete ones, have a certain degree of lateral resistance, even if they were not explicitely designed in accordance to a seismic code. This means that if reference is made to a particular actual building its intrinsic lateral strenght is equivalent to the enforcement of a seismic code with a certain seismic coefficient, which evidently can be different from a building to another. In this paper the above feature is referenced as "implicit" seismic coefficient.

The present paper analyzes five reinforced concrete buildings as cases-history: they all are situated in Tagliamento valley (see fig.1) at almost the same epicentral distance (9 - 15 Km), and are based on similar ground (alluvium). The

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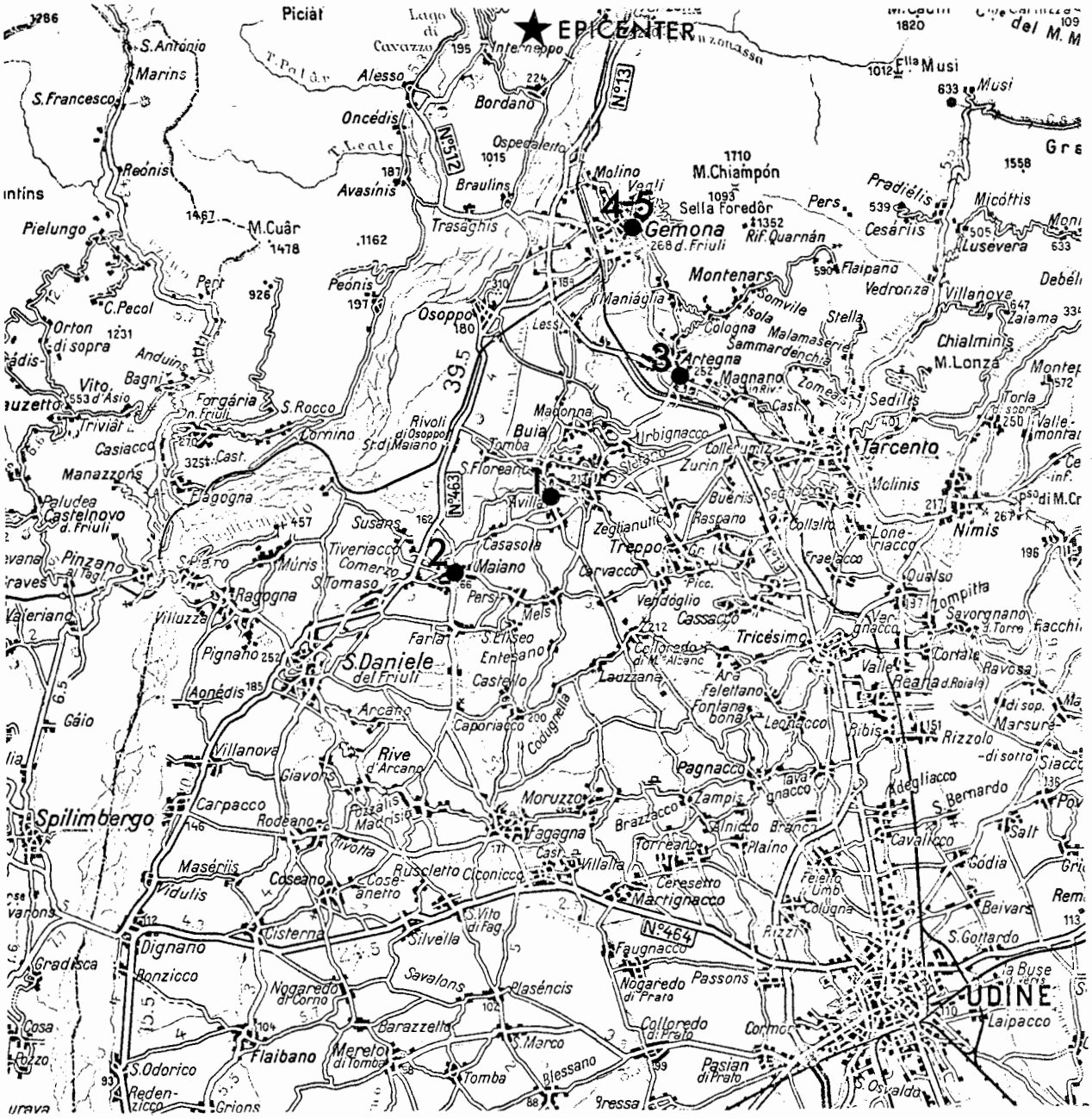
EPICENTER
1
2
3
4
5
S.Daniele del Friuli
Spilimbergo
Gemona
Tarcento
Nimis
Artegna
Osoppo
Buia
Maiano
Tricesimo
Pagnacco
Fagagna
Martignacco
UDINE
Pasian di Prato
Colloredo di Prato
Faugnacco
Passons
Nogaredo di Prato
Flaibano
Codroipo
Sedegliano
Rivignano
Spilimbergo
Dignano
Gradisca
Cisterna
Carpacco
Rodeano
Masérins
Coseano
Villanova
Rive d'Arcano
Arcano
Caporiacco
Codugnella
Pignano
Aonédis
Ragogna
Pinzano al Tagl.
Valeriano
Gaio
Forgaria nel Friuli
Vito d'Asio
Trivignano
Manazzons
Castelnovo del Friuli
Pagogna
Susans
Tiveriacco
Comeglians
Muris
S.Tomaso
Casasola
Treppo Grande
Cavacco
Verzegnis
Cassacco
Tricesimo
Lauzzana
Feletto Umberto
Tavagnacco
Adegliacco
S.Bernardo
Cavalicco
Godia
Colugna
Beivars
S.Gottardo
Laipacco
S.Osvaldo
Cormor
Basagliapenta
Moruzzo
Plaino
Torreano
Ciconicco
Villalta
Ceresetto
Silvella
S.Vito di Fag.
Savalons
Plaséncis
Nogaredo di Corno
Barazzetta
Merlo di Tomba
S.Marco
Tomba
Blessano
Redenzicco
Grions
S.Odorico
Bonzicco
Vidulis
Tivoria
Ruscletto
Cosa
Pozzo
Urbignacco
Madonna
Rivoli di Osoppo
Sr.di Maiano
S.Floreano
Avilla
Zegliannulis
Buerins
Raspano
Collalto
Loneriacco
Qualso
Ravosa
Savorgnano del Torre
Valle di Reana
Reana d.Roiale
Rizzolo
Marsure
Sibis
Lecnacco
Fontanabona
Ara
Zampis
Anicco
Brancia
Brazzacco
Pers
Mels
Farla
S.Liseo
Entesano
Castello
Picc.
Verdóglio
Zomeais
Sedilis
Torlano
Villanova
Lusevera
Micottis
Cesariis
Pradielis
Sella Foredôr
M.Chiampón
Molino
Venti
Maniaglia
Montenars
Somvile
Isola
Cologna
Malamaserie
Sammardenchia
Flaipano
Vedronza
Stella
Chialminis
M.Lonza
Zalama
Debeli
Monte
Valle-montrai
Torla
Ospedaletto
Bordano
Braulins
Trasaghis
Oncédis
Avasinis
Alesso
Cavazzo
Lago di Cavazzo
Interneppo
Peonis
Peonis
Pielungo
M.Cuâr
C.Pecol
Orton di sopra
Anduins
Bagni
Casiacco
Cornino
Forgaria n.Friuli
S.Rocco
Pers
Reonis
S.Antonio
Marins
S.Francesco
Piciat
Lago di Cavazzo
T.Leale
T.Palâr
Intins
Musi
Gre
1710
1558
1820
1012
633
1093
1352
1478
1467
1162
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926
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1197
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1572
1401
1217
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1212
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1136
1137
1110
1113
1170
1104
1102
1111
1146
1258
1212
1200
1199
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1193
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1252
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1025
1457
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1252
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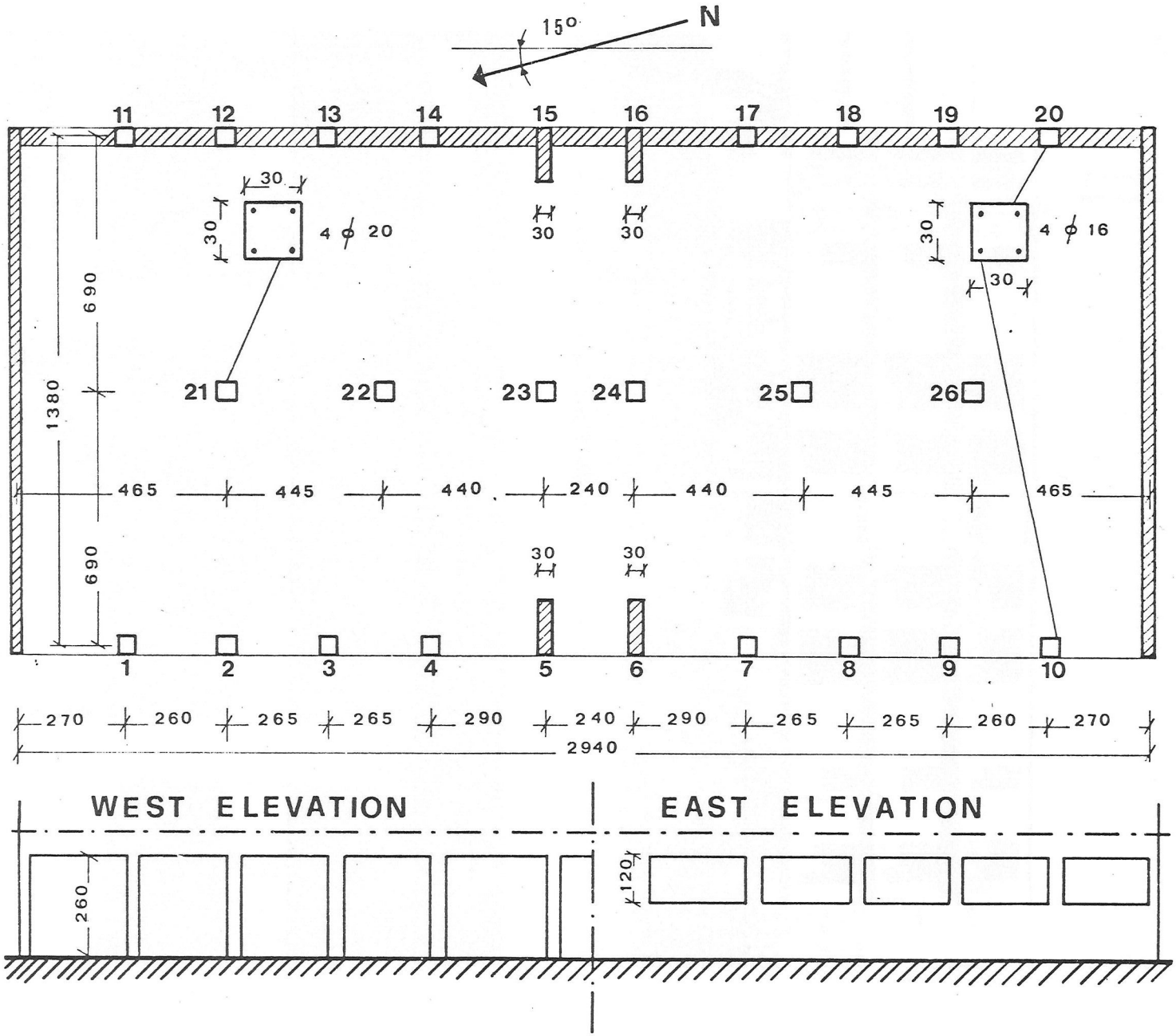
1188

structures of four of them (figs 2-5)were heavily damaged by the shock of May 6th such as it is possible to define the situation as to be at the point of collapse. The fifth(fig.6) of them collapsed. In all the cases taken into account the structural data were reliable: r.c. structures were clearly apparent and the actual reinforcing bars of the columns were in view in most of the istances (see figs.7-10). A field investigation has made possible the evaluation of actual masses and an extimation of the concrete qualities by percussion sclerometer tests.

The object of the paper is to correlate usual structural analyses, as they may be performed for the design of that type of buildings, and the events: "collapses of the considered structures". The paper wishes to contribute to a better definition of the relationships existing among a)reinforced concrete codes; b) current methods of design; c) design loads and criteria assumed by seismic codes and d)field data. Explicit reference is made to Italian codes [1],[2], whose specifications relevant to the present problem are common to other codes for istance [3],[4].

2. BUILDING SPECIFICATIONS

Building 1: 4-storey apartment building in Avilla,fig. 2. The structure is a r.c. frame. Slabs are made by reinforced concrete and brick-work. Girders have the same depth of the slabs. The building is adjacent to other two identical buildings. While it was still in construction the other two were completed. The studied building unlike the others did not have any internal brick-wall at ground floor.Minor structural damages were suffered by the adjacent buildings: only cracks into partitions and walls. The same is true at the apartment storeys (1st to 3rd) of the sudied building,while very heavy damages have been suffered by the r.c.structure at ground floor. The bearing structure at ground floor consists in a longitudinal central line of columns, which all exhibit clear plastic hinges at top and bottom. One of these columns is completely collapsed (see fig. 2). At one longitudinal side there is a similarly damaged line of columns of the same heigth while at the opposite side columns are shor-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465 445 440 240 440 445 465
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1 2 3 4 5 6 7 8 9 10
270 260 265 265 290 240 290 265 265 260 270
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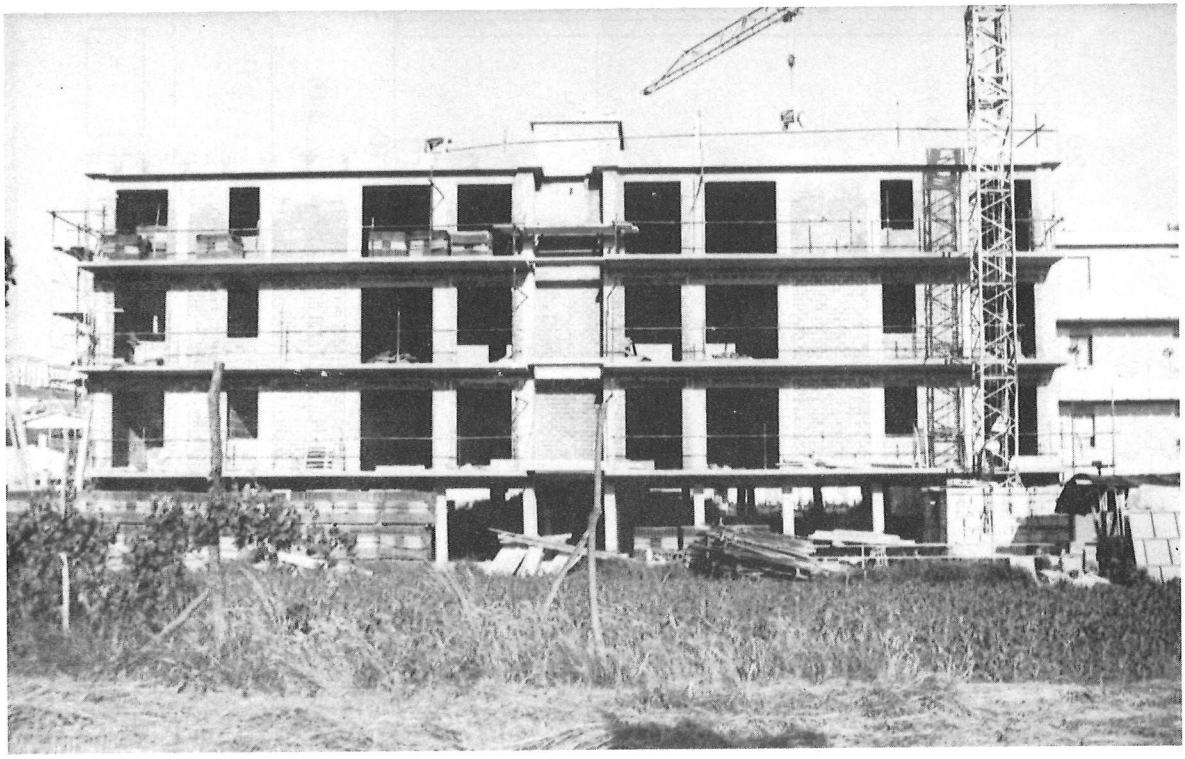
WEST ELEVATION
EAST ELEVATION
260
120

FIG. 2a



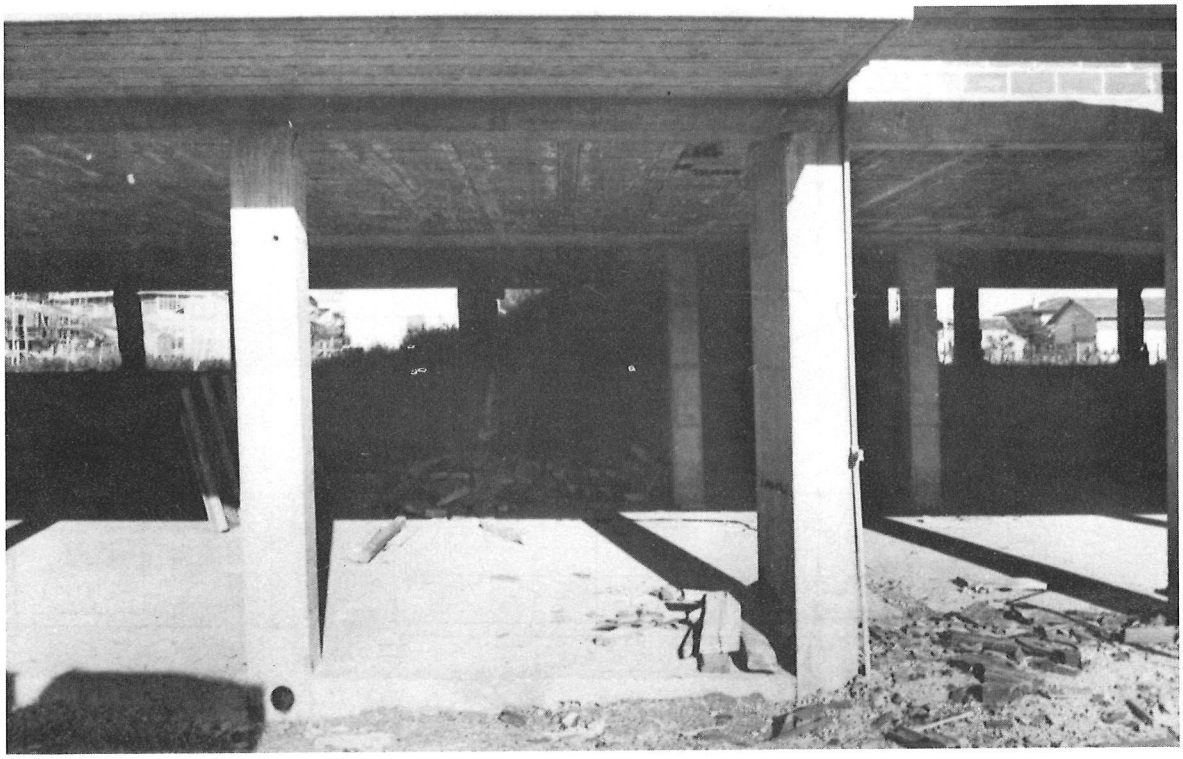


FIG. 2b

ter, supported by a r.c. wall.

Two concrete walls (with no reinforcement) terminate the building on the shorter sides. These walls seem to prevent any in plan torsion of the building, infact only longitudinal displacements took place. The direction of the epicenter is close to north, therefore the direction of the main inertia forces was likely coincident with the weakest building direction.

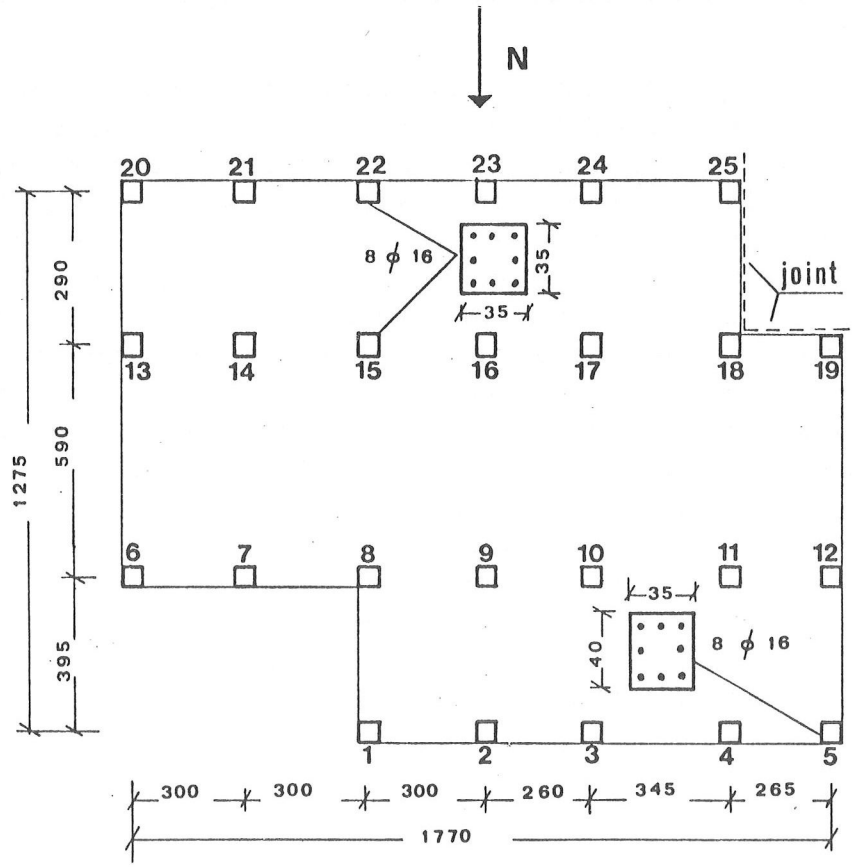
Building 2: 7-storeys apartment building in Maiano, fig. 3. This building is also described in ref. [5] .To the purpose of the present study the relevant data are the followings: the reinforced concrete frames include many partitions and brick walls at the upper storeys, while columns are free at ground floor (no shear wall extends down to ground). All the columns have been severely damaged and permanent displacements of all the upper part relative to the ground have been caused by the shock. The building will be studied as a rigid block resting on one-storey columns. Their strenght characterizes the global horizontal bearing capacity of the whole building.

Building 3: 4-storeys dwelling at Artegna, fig.4. This building is described also in ref. [5]. The ground floor was used as a warehouse therefore no internal partitions existed. External walls develop mainly alongone side of the building, while a number of brick walls were cast at the sides of two stair-boxes. The latter are very eccentric with respect the building causing an important in plan torsion of all the building during the earthquake. The wall around one of the stairs had also a bearing function with respect vertical loads.All the columns at ground floor developed plastic hinges at the bottom and at the top. The bearing walls around one of the stairs and some of the external walls collapsed during the shock. The upper storeys suffered minor damages, while they moved as a nearly rigid block and remained in a displaced and rotaded position after the earthquake. The epicenter lays northward and the main inertia forces were likely in the transverse direction of the building.

Building 4:3-storeys dwelling at Gemona, fig.5. The build



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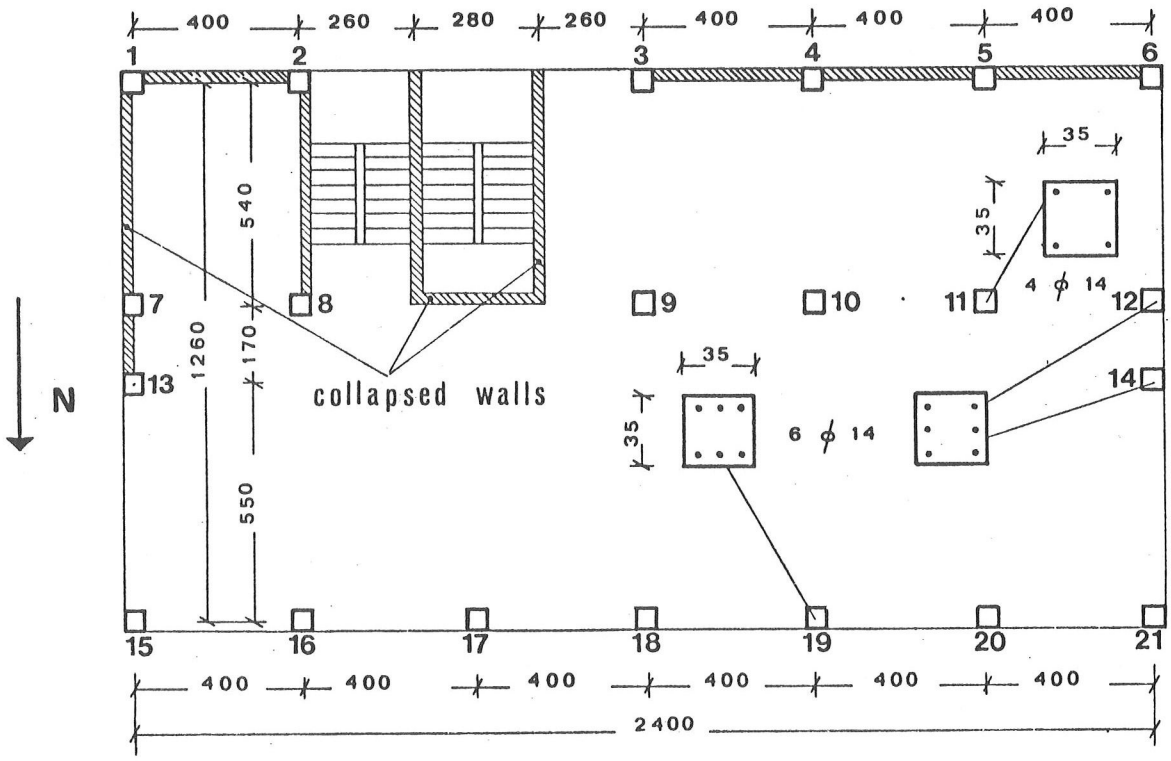
290
590
395
1275
6
7
8
9
10
11
12
35
40
8 ϕ 16
1
2
3
4
5
300
300
300
260
345
265
1770

FIG. 3



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4 ϕ 14
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170
13
collapsed walls
35
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6 ϕ 14
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550
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2400

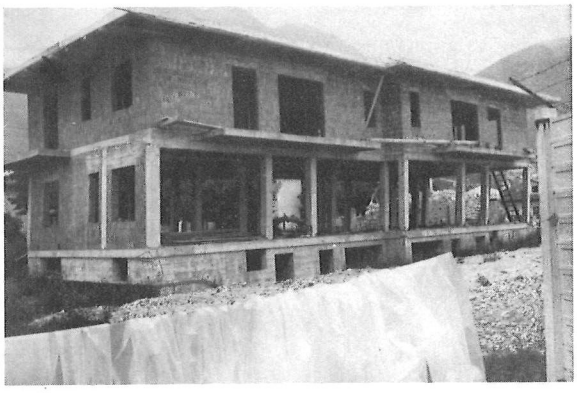
FIG. 4

ing was under construction at the time of the earthquake and the reinforced concrete structure (all completed) was clearly in view. It rests on a solid concrete basement made by concrete walls (25 cm thick) which suffered minor damages. The upper storey too, made only by brick walls and partitions had no damages. Very heavy damages are concentrated on the intermediate storey where only reinforced concrete columns were present. An infilled brick wall at one of the shorter sides were built at the intermediate storey and it caused a very apparent in plan rotation of the building. Plastic hinges are apparent in all the columns, some of them have even horizontal displacement of the bases.

Building 5: Elevated water tank of the railway station of Gemona. The structure is clean tubular reinforced structure supporting conic reservoir. The mass of the water at the time of the earthquake was 50 m^3. The water level was therefore at about 50 cm from the roof of the reservoir, hence some sloashing effect may have arised. At the bottom of the tubular column a door opening of about 80 x 180 centimeters existed around which a very strong additional reinforcement was placed (field observation shown 11 bars Ø20 at each side and top; vertical reinforcement of the tube was two rings of 25 Ø 14; horizontal rings Ø 8/25. The water tank completely collapsed during the earthquake. The direction at which it was lying was exactly opposite to the Monte S. Simeone, the epicenter. The base section of the tower was completely destroyed.

Common structural data. All the concrete of the observed structures seems to have been of good quality. Sclerometer tests showed strenght always higher than 350 Kg/cm^2. Therefore a characteristic resistence R'_{bk} =300 (in the sense of ref. [1], [3]) can be assumed as design specification. This implies a limit strenght of R'_{bk} = 165.6 Kg/cm^2 and an allowable stress of $\bar{\sigma}_c$=97.5 Kg/cm^2 according to [1]. Bars were of the bond type, and therefore Fe 44k can be a reliable guess for the design standard of steel (this implies a limit strenght of R'_a = 4400 Kg/cm^2 a design yielding stress R'_a = 3720 Kg/cm^2 and a allowable stress of σ_f=2200 Kg/cm^2, according to [1]).





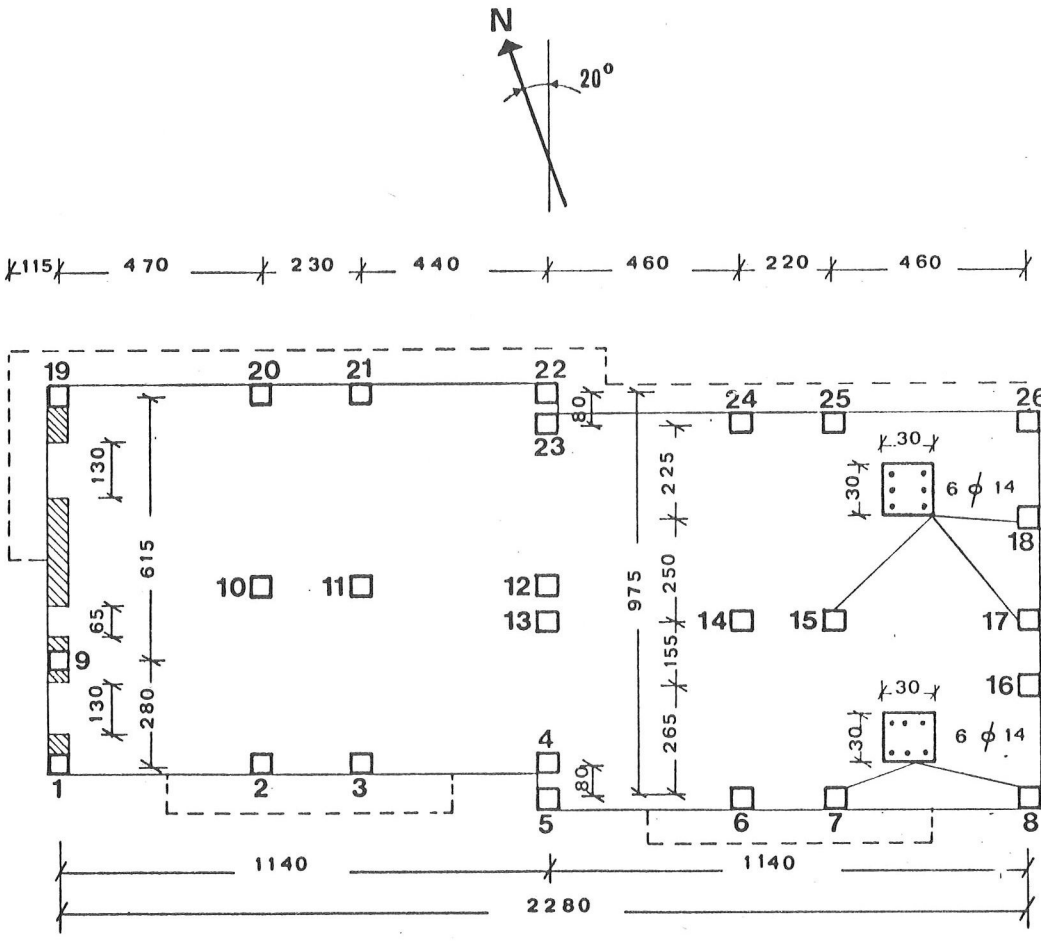

N
20°
115
470
230
440
460
220
460
19
20
21
22
23
24
25
26
18
17
16
15
14
13
12
11
10
9
1
2
3
4
5
6
7
8
130
615
65
130
280
80
225
250
155
265
975
80
30
30
6 ϕ 14
30
30
6 ϕ 14
1140
1140
2280

FIG. 5

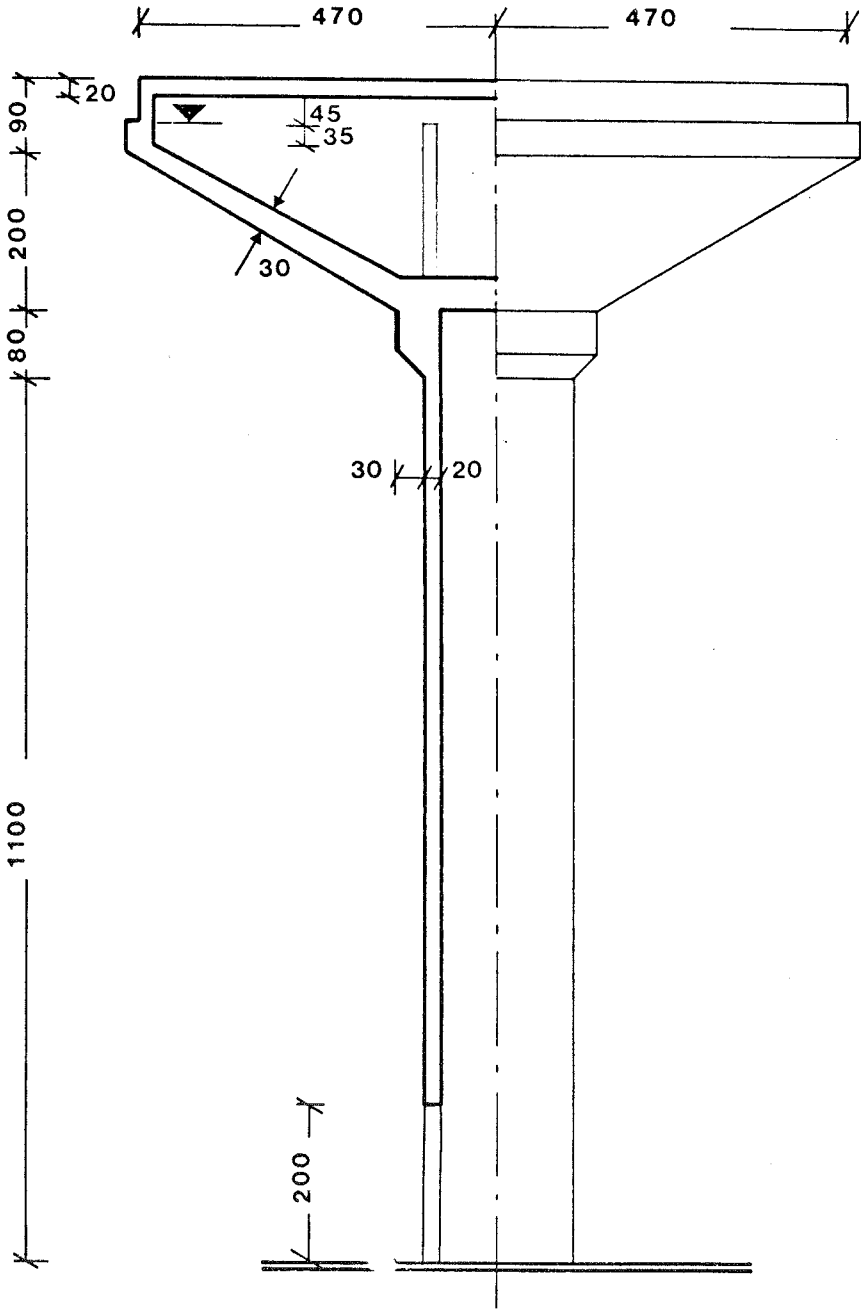
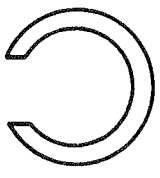


FIG. 6a





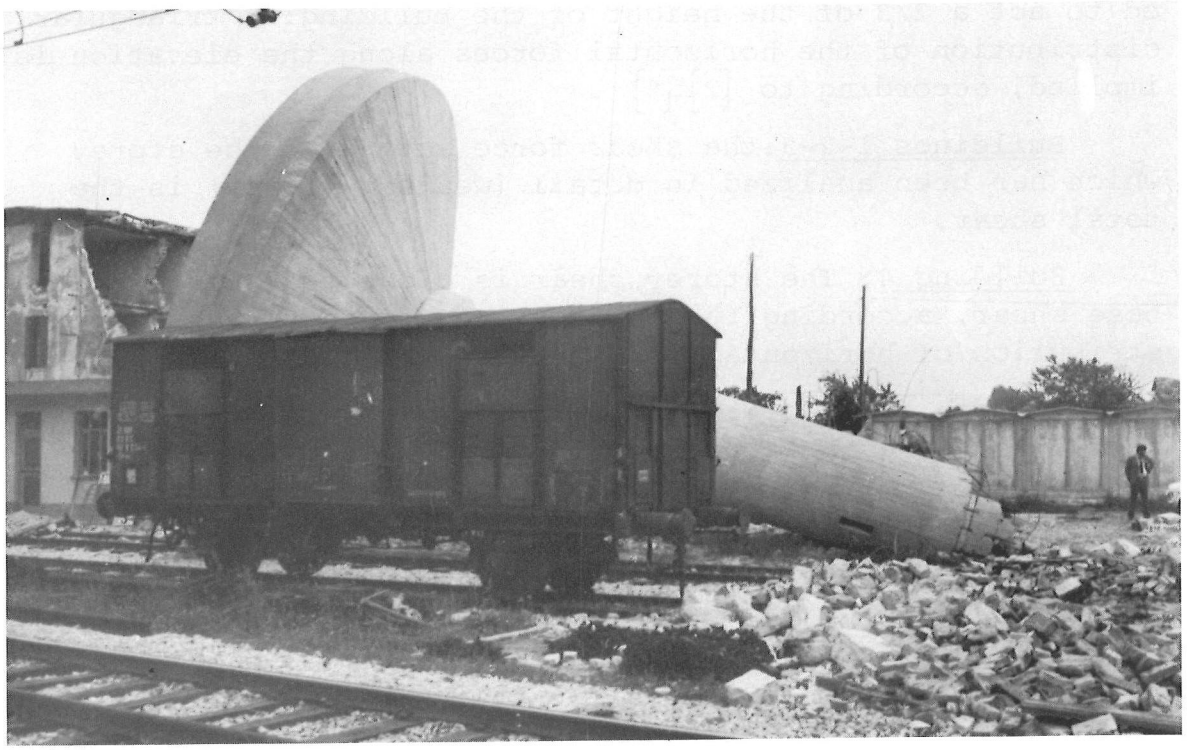


FIG. 6b

Walls were made by the vertical hole type bricks , with an hole ratio of about 50%. The inplan bearing section can be assumed as 1/4 of the total thickness of the wall, and therefore the effective thickness results in 6 centimeters. The limit shear in the brick wall was assumed to be 6 kg/cm^2,however this data is quite uncertain.

3. STRUCTURAL ANALYSIS CRITERIA

Buildings 1-2-3-4 show a common structural scheme: a rigid block supported by a number of columns and walls. The dead weight of each storey was assumed to be equal to 250 (slab weight) + 100 (equivalent weight of walls and partitions and finishing)= 350 Kg/m^2 for each square meter of storey. Design live loads were taken into account as a mass corresponding to 1/3 of the maximum live load (ref. [2]): therefore a weight of 100 Kg/m^2 is assumed.The in plan position of the horizontal forces resultant coincides with the center of gravity of the upper block. When overturining moment is considered in the analysis (causing additional axial forces into the side columns) the total horizontal force is supposed to act a 2/3 of the height of the building: a triangular distribution of the horizontal forces along the elevation is implied, according to [2],[4] .

Buildings 1-2-3:the shear force acting on the storey which has been analized in detail (weakest storey) is the total shear.

Building 4: The storey shear is close to 5/6 of total base shear, according to the above mentioned triangular distribution of horizontal forces along the elevation.

Buildings 1-2-3-4: The distribution of the total horizontal forces as shear forces in all the columns and walls is made by a standard technique that (for completeness sake) is summarized in appendix 1. Stiffness of columns is related to their uncracked cross section and free height. Young modulus of columns: 200000 Kg/cm^2. Shear modulus of brickwalls: 10000 Kg/cm^2. Top and bottom bending moments of the columns are supposed to be equal, according to the hypothesis of the upper rigid block. For the same reason, increments (or decre-

ments) of axial forces into the columns due to overturning (when considered) are obtained by the theory of a rigid body resting on a series of vertical springs (columns) whose stiffness is proportional to the column cross sectional area.

Building 5: The structural scheme is a cantilever built into the foundation. The weakest section is the base of the tubular columns. The relevant actions are the bending moment and the axial forces acting on this section.

All the buildings: The distribution of loads among the members is done by the theory of elasticity, as in current design. Therefore there is proportionality between loads and member forces. The vertical loads are fixed to certain values. The horizontal loads are proportional to a scale factor: the seismic coefficient C. If one refers to the N-M plane for each top (or bottom) section of the columns of a building in study, a variation of C will draw a segment. If, in the same plot, the allowable N-M domain of the considered reinforced concrete section is also drawn, one may detect the implicit seismic coefficient for that section of the building under examination. It is apparent that the minimum seismic coefficient which may thus be found can be assumed as the implicit seismic coefficient for the entire building. In the following two type of M-N domain will be plotted for each column: first, the allowable domain, i.e. the domain in which the stresses in the concrete and in the steel are less or equal to the allowable values ($\bar{\sigma}_c = 97.5$ Kg/cm^2, $\sigma_f = 2200$ Kg/cm^2); second the limit domain, i.e. the domain in which the concrete strain is less then the limit one (0,35% according to [1]) and the steel strain less then 1% [1]. The first domain can be used to investigate whether requirements of the actual Italian seismic code are met. Inspection of the second domain may enable to get some idea on the possibility of extending the philosophy of r.c. code [1], [3] to seismic loads. The former code allowes to make reference to the limit states, what is not admitted by the actual Italian seismic code where reference is made to the allowable stress. However reference to the limit states seems to be a trend in seismic engineering. The determination of the coefficient of reduction of the limit state (which recovers the same results given by the use of the seismic code in a given design) may throw some

light on the possibility of extension stated above

3.STRUCTURAL ANALYSIS RESULTS

Several analyses were performed on the buildings in study. In the sequel each analysis has a code number: A[(n) (m) (i)] where (n) is the number of the building (as defined in sec.2) and (m),(i) are numbers indicating the type of analysis carried out. (m),(i) have the following meaning:

m=1: overturning moment disregarded, brick walls assumed as having zero stiffness.

m=2: overturning moment disregarded, brick walls assumed with a shear stiffness (defined in sec.2)

m=3: overturning moment considered, brick walls as for m=2.

i=1: horizontal seismic loads acting parallel to the longer building plan extension (longitudinal)

i=2: horizontal seismic loads acting parallel to the shorter building direction (transverse).

The results of the analyses can be read in figs.11-12-13-14, where the plots of the limit domains of each column section type is reported, togheter with the segments indicating bending moments and axial forces due to horizontal loads of varying intensity. On the same segments the values of the seismic coefficients corresponding to any moment-axial force combination is also indicated. Each segment is referred to a particular analysis type by the above described code number, and is also referred to the column number P(.) indicated in figs.2-3-4-5. Only the more significant results have been plotted allowing the determination of the lowest value of the seismic coefficient for each building: what we have defined as the "implicit seismic coefficient".

The following table summarizes the results which have been obtained.

BUILDING	Code numbers		IMPLICIT SEISMIC COEFFICIENT		
	(m)	(n)	Allowable stress	Limit states	Min.among diff.analyses
A1	2	1	0.05	0.15	0.05
A2	2	2	0.12	0.19	
	3	2	0.10	0.15	0.10
A3	1	1	0.083	0.13	
	2	1	0.091	0.16	
	3	1	0.091	0.16	
	1	2	0.085	0.128	
	2	2	0.105	0.176	
	3	2	0.105	0.176	0.08
A4	1	2	0.12	0.17	
	2	2	0.10	0.14	0.10
A5		1	0.07		
		2	0.06		0.06

4. CONCLUDING REMARKS

The results of sec. 3 allow to make some remarks that may be helpful in correlating the methods of structural design to the behaviour of some real structure under a strong earthquake.

The authors will suggest the following considerations:

1) the implicit seismic coefficients of the studied buildings are reasonably high.Some of them can be considered as implicitely designed under the requirements of the Italian Seismic code for a 2.nd class zone (C=0.07).This may support the opinion that the enforcement of a seismic code

can have influence on the building costs only beyond a certain level of code severity. On the other hand, the fact that the studied building nearly collapsed, may support the opinion that proper lateral resting elements (like for istance shear walls) should be incorporated in to the body of seismic code, in addition to the specification of the seismic coefficient.

2) Different methods of structural analysis can lead to different and sometimes opposite conclusions as far as the safety of a building is concerned. (see next point).

3) The fact that brick walls are (or not) included into the analysis leads to different results, which may be very different in some instances (see e.g. analyses A 3-2-1 and A 3-3-1 or A 4-1-2 and A 4-2-2). Their contribution may lead to opposite effects: on the one side may increase the strenght of the system thus reducing the state of stress (see analyses A 3), on the other they may induce in plan torsion producing additional stresses on certain elements. A conclusion might be drawn that design should be in accordance with the most severe of the two alternatives, which have to be implemented in any particular case. Note that field data regarding the studied buildings show that some of the brick walls collapsed and therefore the real behaviour of the building during the earthquake is still different from the one predicted by linear analysis.

4) The overturning moment results into a considerable effect on axial forces of the side columns of ground floor. However if a design criterion based on allowable stresses is used, when columns are over-dimensioned with respect to the nominal vertical load, the inclusion in the analysis of the overturning moment may not lead to a design different from the one in which the moment is not accounted for. This is due to the fact that limit elastic moment is not very sensible to variation of axial forces in the usual range of column reinforcement (see fig.11-12-13-14). However if the columns are close to their maximum bearing capacity of axial forces (in the sense of the allowable stress criterion) the limit moment decreases very rapi-

dly together with an increase of the axial force, (and therefore of the overturning moment) associated with seismic loads. One may conclude that the overturning moment should be considered, when designing with the allowable stress criterion, by adding axial forces to the columns.

5) If the limit states criterion is used the design, one may observe from figs. 11-12-13-14 that the most severe restrictions hold for low axial forces. Therefore one may conclude that the overturning moment should taken into account in design by subtracting axial forces from the lateral columns.

6) The above mentioned divergent evaluations about the effect of axial forces connected with the use of two design methods call for caution about the possibility that one is replaced by the other only by means of a proper scaling of seismic coefficient. Experimental evidence of the buildings taken into account is yet contraddictory. In some istances (see column 26 of building 2) high axial forces together with seismic effects had a worse consequence than lower axial forces on columns of the same building which were similarly bent. In other istances (see columns in the corners of building 4) the lower the axial forces, the more relevant was the yielding. It is authors' opinion that a proper implementation of the limit state criterion for seismic design should not disregard the global building structural behaviour and not be only concerned with the element limit bearing capacity. This approach is attempted in ref. [8] .

5. ACKNOWLEDGEMENTS

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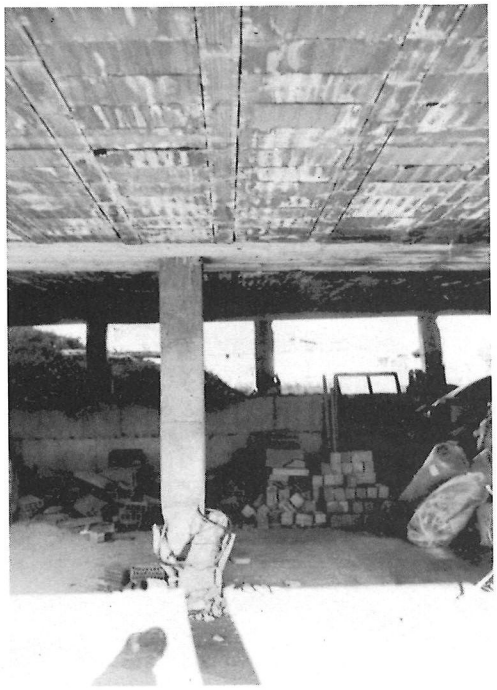

FIG. 7

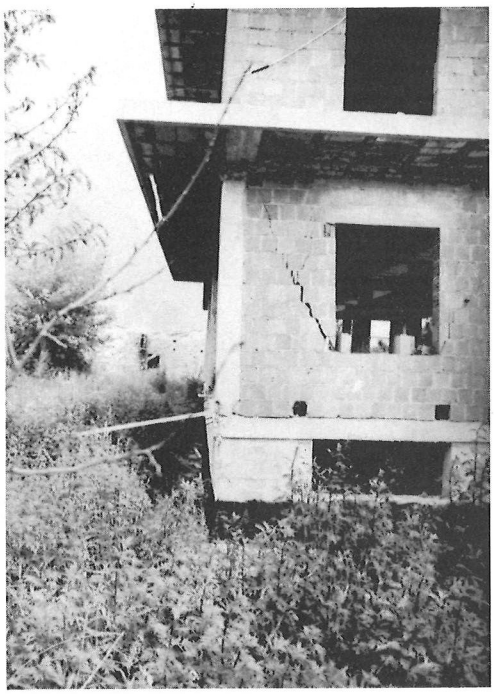

FIG. 8



FIG. 9

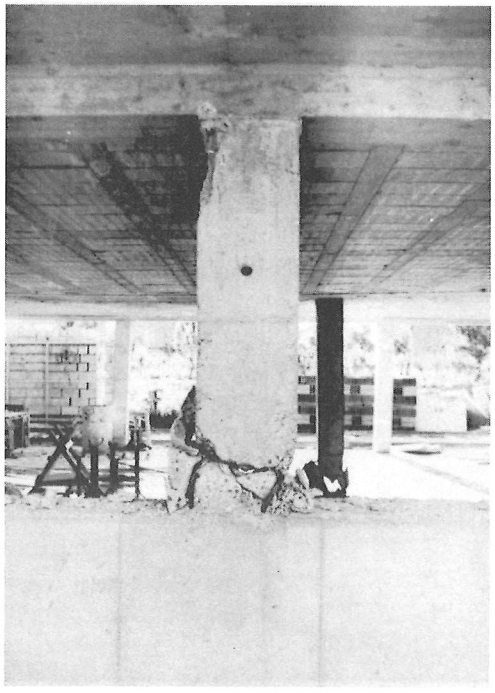

FIG. 10

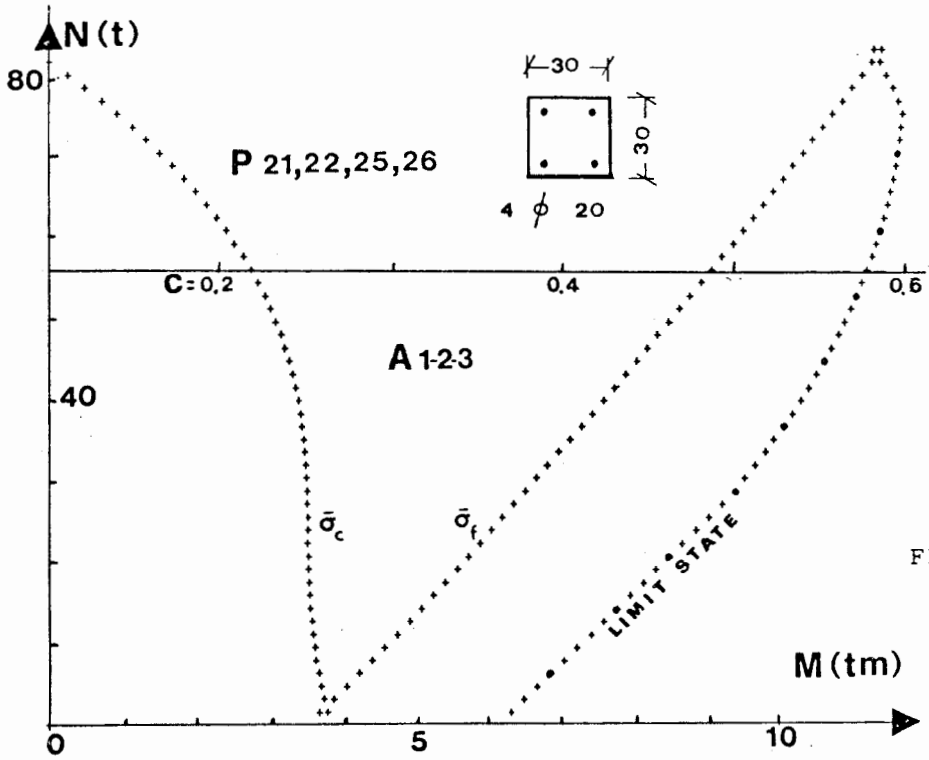
N (t)
80
40
P 21,22,25,26
30
30
4 ϕ 20
C=0.2
0.4
0.6
A 1-2-3
$\bar{\sigma}_c$
$\bar{\sigma}_f$
LIMIT STATE
M (tm)
0
5
10

FIG. 11

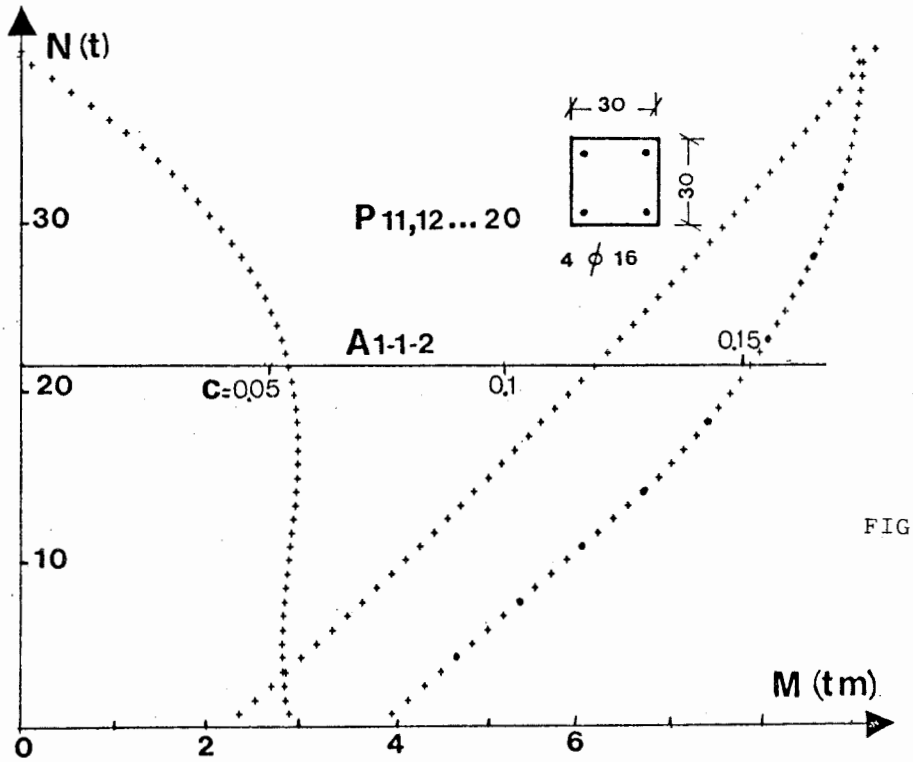
N (t)
30
20
10
P 11,12 ... 20
30
30
4 ϕ 16
A 1-1-2
C=0.05
0.1
0.15
M (tm)
0
2
4
6

FIG.12

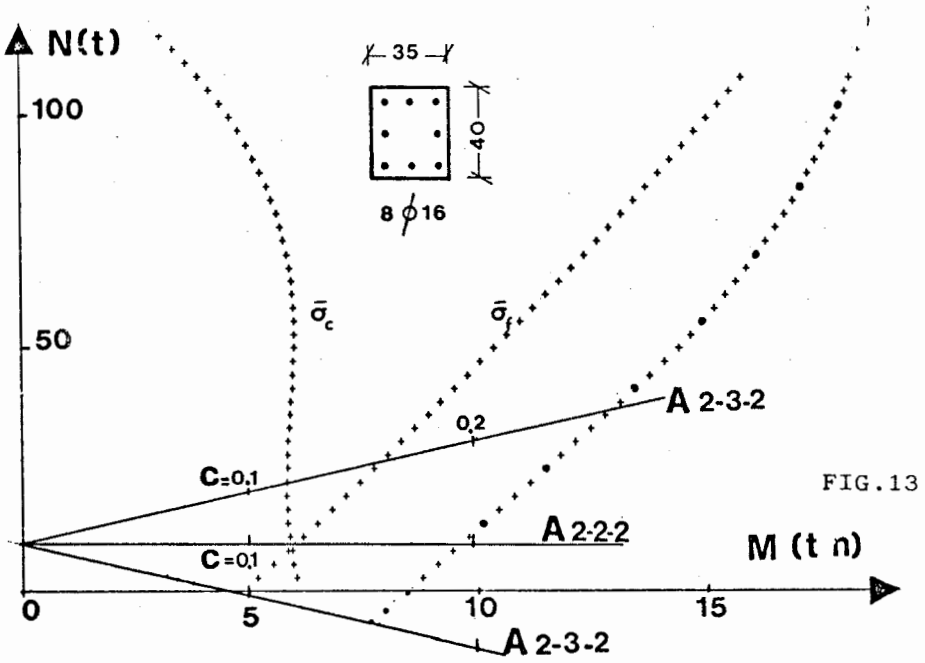
N(t)
100
50
35
40
8 ϕ16
$\bar{\sigma}_c$
$\bar{\sigma}_f$
A 2-3-2
0,2
C=0,1
A 2-2-2
C=0,1
M (t n)
0
5
10
15
A 2-3-2

FIG.13

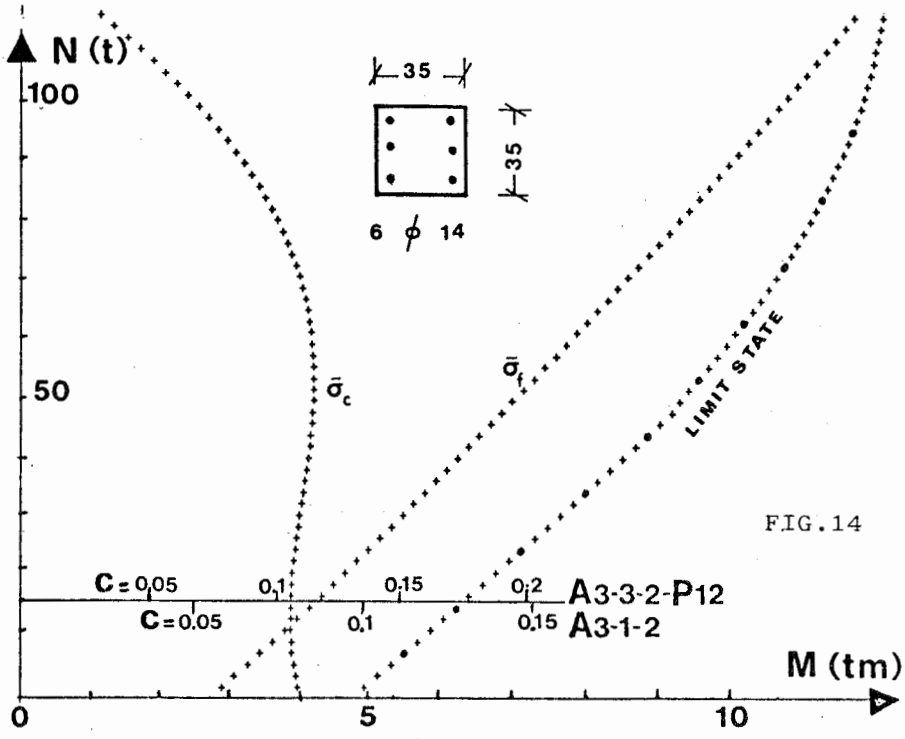
N (t)
100
50
35
35
6 ϕ 14
$\bar{\sigma}_c$
$\bar{\sigma}_f$
LIMIT STATE
C = 0,05
0,1
0,15
0,2
A3-3-2-P12
C = 0.05
0,1
0,15
A3-1-2
M (tm)
0
5
10

FIG.14

APPENDIX 1: DISTRIBUTION OF HORIZONTAL FORCES AMONG COLUMNS AND WALLS

The plan of building is referred to arbitrary x-y axes parallel to the column sides: they are assumed as principal directions of the building. Each column or wall has coordinates x_i, y_i. The stiffnesses of the i.th column with respect to displacements of the top in the x and y direction are:

$$s_{ix} = \frac{12\ E\ I_{ix}}{h_i^3} \qquad s_{iy} = \frac{12\ E\ I_{iy}}{h^3} \qquad (1.1)$$

where E: Young Modulus, I moment of inertia, h height of the column. The shear walls or brick (infilled) walls have stiffnesses only in their plane, therefore, for the i.th wall:

$$s_{ix} = \frac{1.2\ G\ A}{h_i} \quad \text{or} \quad s_{iy} = \frac{1.2\ G\ A}{h_i} \qquad (1.2)$$

according to its direction. G is the shear modulus and A the effective in plan area of the wall. The center of torsion for the storey has coordinates:

$$x_r = \frac{\sum_i s_{iy} \cdot x_i}{\sum_i s_{iy}} \qquad y_r = \frac{\sum_i s_{ix}\ y_i}{\sum_i s_{ix}} \qquad (1.3)$$

The coordinates of columns and walls with respect the center of torsion are:

$$X_i = x_i - x_r \qquad Y_i = y_i - y_r \qquad (1.4)$$

The polar moment of inertia of all the elements is:

$$I_p = \sum_i (s_{iy} X_i^2 + s_{ix} Y_i^2) \qquad (1.5)$$

The total shear forces F_x, F_y acting on the storey are applied to the center of gravity of the upper block having coordinates x_F, y_F. The in plan torsional moment is given by:

$$M_F = - F_x (y_F - y_r) + F_y (x_F - x_r) \qquad (1.6)$$

The shear forces at the top of each column and wall are given by:

$$V_{ix} = \frac{F_x \cdot s_{ix}}{\Sigma_i s_{ix}} - \frac{M_F \cdot Y_i \cdot s_{ix}}{I_p} \qquad (1.7)$$

$$V_{iy} = \frac{F_y \cdot s_{iy}}{\Sigma_i s_{iy}} + \frac{M_F \cdot X_i \cdot s_{iy}}{I_p} \qquad (1.8)$$

APPENDIX 2: ALLOWABLE STRESS DOMAINS FOR R.C. SECTIONS

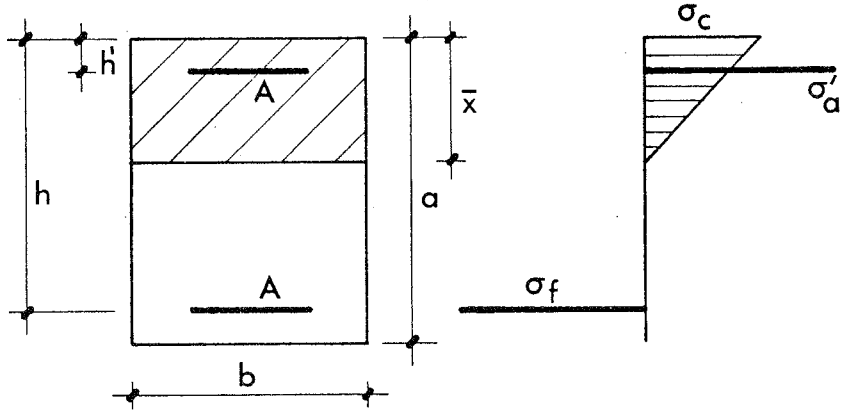

FIG. 15

Given:a,b, A (both tension and compression steel), the allowable maximum concrete stress $\bar{\sigma}_c$,the ratio $m=E_s/E_c=10$, the axial force N, the first problem is to find the limit value of $M_c=e.N$ such as the actual maximum stress in concrete

$\bar{\sigma}_c = \sigma$ .From standard r.c. theory [6] .

$$\sigma_c = N \cdot \left[\frac{bx}{2} + m A \left(2 - a/x \right) \right]^{-1} \qquad (2.1)$$

that is equivalent to:

$$x^2 \cdot \left(\frac{\sigma_c \cdot b}{2} \right) + x \cdot (2 \sigma_c m A - N) - \sigma_c m A a = 0 \qquad (2.2)$$

Calling:

$$p = 2 \cdot \frac{2\sigma_c m A - N}{\sigma_c b} \quad ; \quad q = - \frac{2 m A a}{b} \qquad (2.3)$$

the actual value of $x=\bar{x}$ is given by the non negative minimum root between:

$$x_{1,2} = - p/2 \pm \sqrt{p^2/4 - q} \qquad (2.4)$$

Because of equilibrium considerations between internal and external actions (see ref. [6]):

$$\bar{x}^3 + 3 \left(e - \frac{a}{2} \right) \bar{x}^2 + 12 m A e \bar{x} / b - 6 m A \left[e a + 0.5 \cdot (h - h')^2 \right] = 0 \qquad (2.5)$$

Rearranging the above relationship:

$$e = \frac{\bar{x}^3 + 3/2 \, a \, \bar{x}^2 + 6 m A . (h-h')^2 / (2 b)}{3 \bar{x}^2 + 12 m A \bar{x} / b - 6 m A a / b} \qquad (2.6)$$

Calling e_c the so-obtained value, the maximum allowable value of the moment is $M_c = e_c . N$.

The second problem is to find the limit value of the bending moment M such as the actual stress into the steel in tension $\sigma_a = \bar{\sigma}_a$ (maximum allowable stress for steel). From ref. [6]

$$\sigma_a = \left[m\ (h-x)\ N \right] . \left[x \left\{ b \frac{x}{2} + m\ A\ \left(2 - \frac{a}{x}\right) \right\} \right]^{-1} \qquad (2.7)$$

that is equivalent to:

$$x^2 \frac{\sigma_a . b}{2} + (2\ \sigma_a\ m\ A + N\ m) + (-\ \sigma_a m\ Aa - Nm\ h) = 0 \qquad (2.8)$$

Calling:

$$p = 2 . \frac{2\ Am\ \sigma_a + Nm}{\sigma_a .\ b}$$

$$q = -\ 2\ \frac{m\ A\ \sigma_a + N\ m\ h}{\sigma_a\ .\ b} \qquad (2.9)$$

The actual value of $x = \bar{x}$ is given by the non negative root between the root of eq. (2.4). The numerical procedure can now go on using the same eq. (2.6) and obtaining a new value of $e = e_a$ which defines the maximum allowable value for the bending moment $M_a = e_a . N$. In figs.11,12,13,14 both M_c and M_a are drawn: however it is evident that for any N the maximum allowable bending moment is defined only by the lowest value between M_c and M_a.

APPENDIX 3 - LIMIT STATE DOMAINS FOR R.C. SECTIONS

The treatment of this subject is derived from ref. [7] where specific reference is made to the ultimate strain of

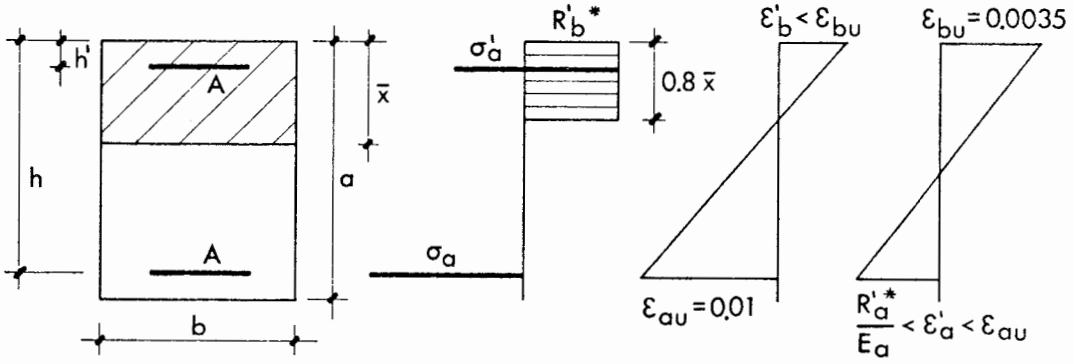

FIG. 16

steel and concrete suggested by the code [1] , see fig.16.

The limit state of the column can be reached either for limit plastic strain in tension steel (limit state 2) or for limit plastic strain in compressed concrete when the steel is beyond the yielding point and below its limit plastic strain. Other limit states can be disregarded due to the predominant values of the ultimate bending moment M with respect to N in the present case study.

Calling $K = \bar{y}/h$ the hypothesis of plane section implies that:

$$K = \frac{1}{1 + \varepsilon_a/\varepsilon'_b} \qquad (3.1)$$

The break-even value of K separating limit states 1 from l.s.2 is therefore:

$$K = \frac{1}{1 + 0.01/0.035} = 0.26 \qquad (3.2)$$

Calling R'^{*}_{b} the limit stress of the concrete

$$R'^{*}_{b} = \frac{R'_{bk} \cdot 0.85}{\gamma_c} \qquad (3.3)$$

according to [1] , where R'_{bk} is the characteristic cubic concrete strenght (see par.2 of the present paper) and $\gamma_c = 1.54$. The following ratios are defined:

$$\mu = A/(b.h) \qquad (3.4)$$

$$\alpha = \sigma_a / R'^{*}_{b} \quad (3.5) \; ; \; \alpha' = \sigma_a / R'^{*}_{b} \qquad (3.6)$$

$$\eta_a = (1 - \frac{h'}{h})/2 \quad (3.7); \; \eta_b = (1 + \frac{h'}{h})/2 - 0.4K \quad (3.8)$$

$$n^{*} = N/(b.h.R'^{*}_{b}) \quad (3.9)$$

$$m^{*} = M/(b.h^2.R'^{*}_{b}) \quad (3.10)$$

The domain M-N is given for the simmetrically reinforced section by the relationships (ref. [7]):

$$n^{*} = 0.8\,K + \mu(-\alpha - \alpha') \qquad (3.11)$$

$$m^{*} = \mu(\alpha + \alpha')\eta_a + 0.8\,K\eta_b \qquad (3.12)$$

Limit state 3 : the steel in tension is beyond the yield limit, therefore its stress is the yielding stress, assumed as:

$$\sigma_a = R^{*}_{a} = R'_{ak}/1.18 \qquad (3.13)$$

(for normally controlled steel) where R'_{ak} is the characteristic steel strenght (in the sense of ref. [1] , see also sec. 2 of the present paper).

The compression steel is also at the yield stress, due to large strain into the compressed concrete:

$$\sigma'_a = R'_{ak} / 1.18 \qquad (3.14)$$

Introducing (3.13)-(3.14) into (3.5)-(3.6) and hence into (3.11)-(3.12) one gets the M-N domain through eq. (3.9)-(3.10). The parameter K is the free variable for $0 < K \leq 0.26$.

Limit state 2. As in the previous case $\sigma_a = R_a$. The stress in the compression steel can be obtained by strain compatibility:

$$\alpha' = \frac{E_a}{R'^{*}_b} \frac{K - h'/h}{1-K} .10^{-2} \qquad (3.15)$$

K is free to vary up to the value corresponding to $\varepsilon_a = E_a / R^{*}_a$, i.e.

$$0,26 < K \leq 0,64 \qquad (3.16)$$

for $R'_{ak} = 4400$ Kg/cm^2. Introducing (3.13), (3.15) into (3.5), (3.6) and hence into (3.11)-(3.12) one gets again the M-N domain through eq. (3.9), (3.10) within the limit of (3.16).

BEHAVIOUR OF SOME SIMPLE STRUCTURES DURING THE FRIULY EARTHQUAKE

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SUMMARY

In this paper the elasto-plastic behaviour of two simple structures in reinforced concrete is examined by observing the effects produced by the earthquake of 6/5/1976, and the maximum velocities of response are derived.

From these maximum velocities, an indication of the values of the spectrum of velocities near the periods of oscillation considered can be derived.

Indications of the elasto-plastic behaviour of extremely large bent elements of reinforced concrete have also been found.

This paper deals with the behaviour of two relatively simple structures damaged during the earthquake on 6/5/1976.

The first structure examined is a caisson pier of the bridge, under construction, where the motorway crosses Lake Cavazzo: at the instant of the earthquake the pier was completed but the bridge girder on top was not in place.

The second structure is a portal frame formed of two round columns, with a transverse capitol connecting them, destined to support the beams of the bridge where the motorway crosses over the railway, between the stations Gemona and Osoppo, near the crossroads for Artegna and Buia on the S.S. 13 Udine-Tolmezzo road.

Figure one shows the first structure examined. The caisson pier after the seismic shock, showed a break in the concrete covering the steel rods, the whole length of one of the shorter sides, at the fixed joint between pier and foundation slab.

There was also clearly-visible damage at the fixed-joint-section on the entire perimeter of the pier.

The pier under examination was the only one to be damaged during the shock of 6/5/1976. It should be noted however that as a result of the succesive shocks of 15/9/1976, all the other piers, between this one and the lake, were damaged, where as the pier did not receive further damage.

The damage to the pier indicates that, due to the seismic shock, the bending moment, at the fixed-joint-section, reached a value between that at the yielding point and at the collapse of the outermost steel rods.

($\varepsilon_f = 10‰$)

The dynamic behaviour of the pier, with very close appro-

ximation may be assumed to be elastic; the plasticized zone being so small that it may be disregarded.

The period of the pier, fixed at its base, is found to be

$$T_2 = 0 \,.\, 26 \text{ sec.}$$

The natural period in the direction of least inertia is

$$T_{2\,\text{longitudinal}} = 0 \,.\, 46 \text{ sec}$$

The system pier foundation slab should in reality be represented at least as a system having 2 degrees of freedom. Since the ratio between the mass of the slab-foundation and the equivalent mass of the pier is very big ($\frac{m_1}{m_2} > 15$) , the periods of the complex system differ by very little from those of the two oscillating parts.

The period of the foundation, T_1, may be calculated supposing, as seems reasonable, a good resistance in the strata where the underground piles are fixed. It results a ratio

$$0.4 \leq T_1/T_2 \leq 0.7$$

decreasing as the supposed resistance of the ground increases. The oscillation of the system is the sum of the two ways of vibrating.

Let us suppose that the major part of the kinetic energy was absorbed by the oscillating structure vibrating in the fundamental way.

For ratios $\frac{T_1}{T_2}$ variable in the range stated above, the difference between the two coefficients of participation of the two masses, multiplied by the ratio of the masses given above, varies between the limits

$$1.1 \leq \Delta_{\beta}^{(1)} \leq 1.5$$

with an average value of 1.3.

If both the moment reached at the base of the cantilever

and the corrisponding elastic displacement-curve are known, the maximum displacement of the simple oscillator may be calculated according to the equation.

$$S_{d\ \substack{\text{simple oscillator}\\ \text{rigid foundation}}} = \frac{\sum m_r x_r^2}{\sum m_r x_r}$$

Considering also the effect of foundation, and simplifying, we may write

$$S_{d\ \substack{\text{simple oscillator}\\ \text{non-rigid foundation}}} = \frac{\sum m_r x_r^2}{\sum m_r x_r} \cdot \frac{1}{\Delta \gamma^{(1)}_{\text{average}}}$$

which gives us

$$S_{d\ \text{simple oscillator}} = 0.89\ \text{cm}$$

So we have also

$$S_{v1} = 21.51\ \text{cm sec}^{-1}$$

$$S_a = 5.19\ \text{cm sec}^{-2}$$

For the period value T=0.26 sec, ζ =0.05, and for semplicity the Housner average curves, we find a value for the coefficient of normalization of the earthquake of:

$$S_{v1}/S_{v\ \text{Housner}} = 2.86$$

The second structure examined underwent notable plastic deformation at its base (during the seismic shock).

This plastic deformation was evidenced by: extensive cracking of the concrete; breaking of the concrete covering of the steel rods; buckling of some of the reinforcing steel rods, and the breaking of one of these.

We may study the dynamic structural behaviour by assuming the uprights of the portal frame to be fixed at the base, and examining the transverse bending vibration, which is responsible for the damage.

Reducing the mass of the uprights at the top of the portal frame according to the coefficient $\frac{33}{140}$, (the natural period

of the oscillator being presumed elastic)

$$T_{\text{elastic I}} = 0.29 \text{ sec}$$

in the hypothesis of the uncracked section, and

$$T_{\text{elastic II}} = 0.48 \text{ sec}$$

in the hypothesis of the deformation intermediate between the entire section and the cracked section .

The bending moment of collapse is, in the hypothesis of deformation of the reinforcing steel rods to the point of collapse,

$$\varepsilon_{\text{steed}} = 10‰ \qquad M_{\text{final}} = 286 \text{ t.m.} \qquad (\varepsilon_c = 1.6‰)$$

As in the previous structure, we reach the point of collapse before reaching a value of deformation of 3.5‰ in the concrete. In fact we are here dealing with compressed beams with a relatively small percentage of reinforcement. The plastic deformation may be calculated by tracing the graph of the bending moment against the corresponding curvature, for the section; the graph of the bending moment against the abscissa on the center line of the column, and finally the graph of the curvature in relation to the abscissa of the column.

In calculating the graph bending moment against curvature we have taken the covering of the steel rods, at the base of the column, to be equivalent to a simple continuous reinforcement.

The plasticization of the column is such that, for the calculation, the section may be considered to have broken due to plasticization up to a height of 0.65 m, considering the rest of the column to have been elastically deformed.

The displacement of the mass at the summit is:

$$X_{elastic} = \frac{0.74+3.29}{2} = 2.01\ cm \qquad [E = 250.000\ kg/cm^{-2}]$$

intermediate between the hypothesis of uncracked section and that of the cracked section.

$$X_{plastic} = 4.03\ cm$$

From which it results the coefficient of ductility

$$\mu = 3.0$$

The maximum displacement of the equivalent elastic oscillator is, therefore:

$$S_{d\,elastic} = (X_{elastic} + X_{plastic}) \cdot \frac{\sqrt{2\mu-1}}{\mu} = 4.5\ cm$$

The average period of the equivalent elasto-plastic oscillator, is:

$$T_{average\ equivalent} = 1.22\ T_{elastic\ II} = 0.58\ sec$$

The histeretic damping quota may be calculated by assuming a histeretic cycle of area equal to half of that of the ideal elasto-plastic oscillator.

We obtain:

$$\zeta_{\substack{average \\ histeretic}} + \zeta_{viscous} \simeq 0.10 + 0.03 = 0.13$$

The spectral velocity of the equivalent elastic oscillator is:

$$S_{v\,elastic} = 4.5 \cdot \frac{2\pi}{0.58} = 48.7\ cm\ sec^{-1}$$

Developing the relationship with Housner's curves we get:

$$\frac{S_{v\ elastic}}{S_{v\ Housner,\ \zeta = 0.13}} \simeq 4.9$$

The results of the calculations give us a value for the maximum acceleration at the base of the structures examined which agrees well with the registrations made during the seismic shock in Tolmezzo.

The second structure examined lets us suppose an amplifi-

cation effect of the excitation at the base due to the natural vibration of the surface strata of the ground.

The good ductility of the second structure examined is due to the placement of the reinforcing steel rods at differing distances from the compressed edge of the section:

It seems, anyhow, to be unfavourable for safety; in particularly large piers, to assume a length of plasticized equivalent zone equal to the dimension of the section in the direction under consideration.

We found the concrete to have been damaged, though severely, by deformation certainly not much greater than 0.2 %.

Therefore, in the case where the concrete is not sufficiently held in a lateral sense, and if all the reinforcing steel rods have the same lever arm, when the force of compression is relevant, we are led to think that the behaviour of an element bent and compressed may approach elastic-fragile behaviour, so that we should consider as disputable a plastic redistribution of the moments in hyperstatic structures.

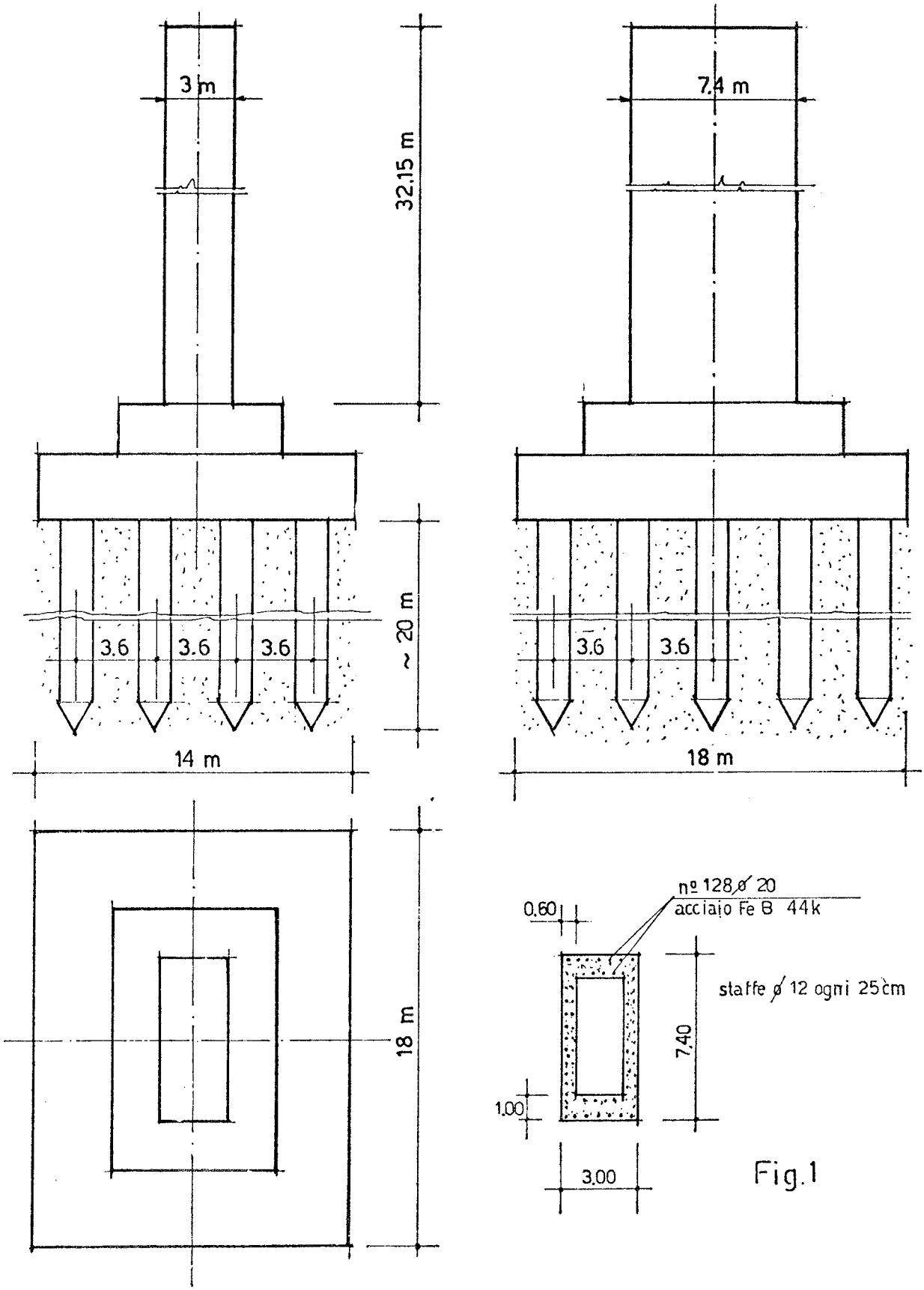
3 m
32.15 m
3.6
3.6
3.6
~ 20 m
14 m
18 m
7.4 m
3.6
3.6
18 m
nº 128 ø 20
acciaio Fe B 44k
0.60
staffe ø 12 ogni 25 cm
7.40
1.00
3.00

Fig.1

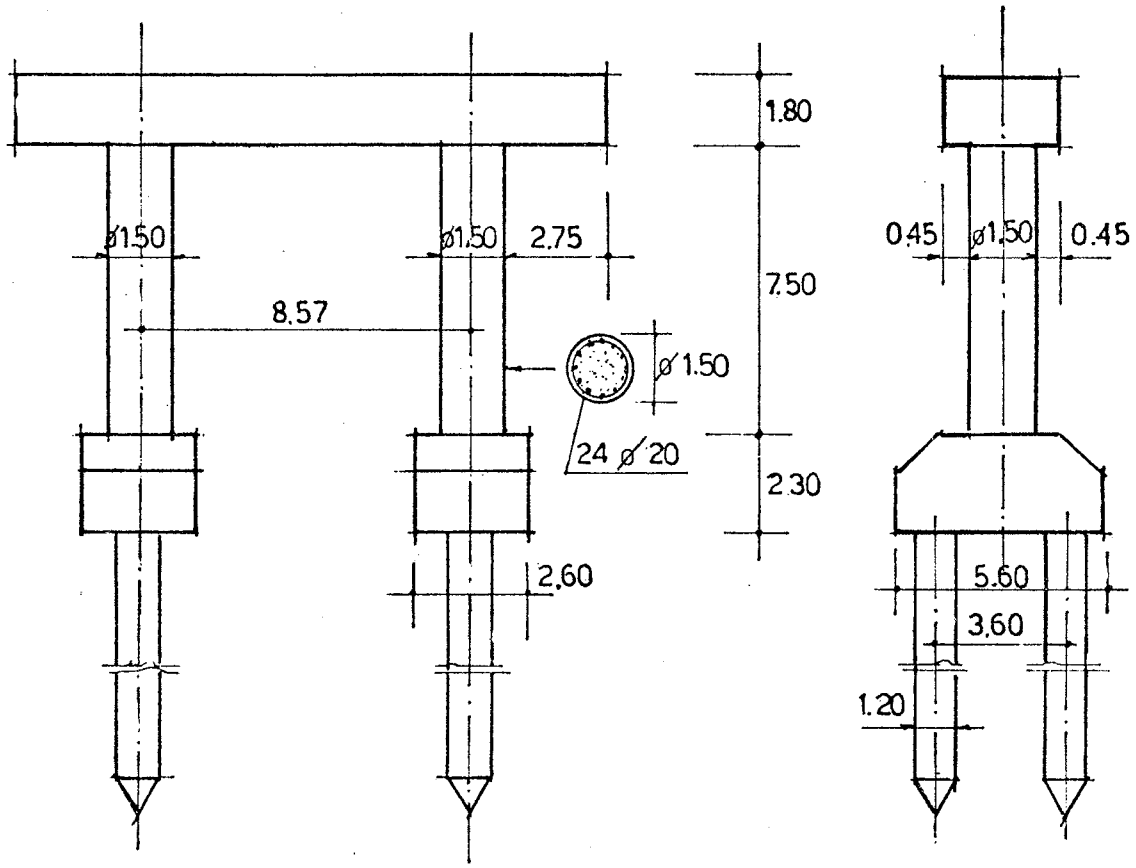
1.80
ø1.50
ø1.50
2.75
7.50
8.57
ø 1.50
24 ø 20
2.30
2.60
0.45
ø1.50
0.45
5.60
3.60
1.20

Fig. 2

USE OF A MODELLING APPROACH IN THE ANALYSIS OF THE EFFECTS OF REPAIRS TO EARTHQUAKE-DAMAGED STONE-MASONRY BUILDINGS

by

dipl.ing.

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1. INTRODUCTION

A visual inspection of damage caused to buildings by an earthquake provides an abundance of rough information about the mechanisms involved in such processes. The phenomena observed, however, are, in this case, as yet unquantified.

The strength and deformation properties of wall elements can be determined by means of laboratory tests, in which damage as near as possible identical to that caused by an earthquake in the natural environment is reproduced. The behaviour of walls elements as the constituent parts of a whole building can be investigated by means of model tests on the laboratory shaking-table. In this case the successfulness of methods-of-repair can be studied and quantified.

In this paper the fundamentals of modelling theory are treated and an account is given of a shaking-table test carried out on a model stone-masonry building. The test results have been treated in two different ways, assuming complete and general model similarity respectively, and thus the flexibility offered by modelling techniques is demonstrated.

2. DEFINITION OF THE CONDITIONS OF MODEL SIMILARITY

2.1 Dimensional analysis

Correct mechanical relationships between model and prototype are obtained when in the modelling of the mechanical properties of materials and of the loadings the conditions of model similarity are fulfilled. The latter depend upon those parameters which have a sufficiently important role in the process being considered. In other words, it is those parameters which, when suitably linked together, define the process being considered sufficiently well which must be included in the conditions of model similarity.

The most suitable method for defining the conditions of model similarity is dimensional analysis, which is based on the hypothesis that every physical problem in which a certain number of physical parameters are involved can be expressed by means of a homogeneous function. Here Buckingham's theorem is of importance. It states that every dimensionally homogeneous equation can be reduced to a relationship among a complete set of dimensionless products. Each dimensionless product obtained from a combination of the parameters involved in the problem defines a condition of model similarity. Within the infinite number of possible dimensionless products there exist only a certain number of independent dimensionless products, which we call a complete set. A set is complete when each dimensionless product in it is independent of the other dimensionless products and when any dimensionless product can be expressed as a power product of the independent dimensionless products in the set.

The importance of knowing the set of dimensionless products is that by means of it a complete set of conditions of model similarity can be defined.

It follows from a general knowledge of the principles of statics and dynamics that the following essential parameters are involved in statical and dynamical phenomena (the less essential parameters have been omitted in ~~the~~ order to simplify the treatment): $\sigma, \varepsilon, \nu, d, \Delta, m, u, p, l$, and t where the symbols have the following meanings:

σ ... stress

ε ... strain

ν ... Poisson's ratio

d ... logarithmic decrement of damping

Δ ... specific weight

m ... mass

u ... displacement

p ... force

l ... length and

t ... time

The parameters ε, ν and d can be eliminated at once from the above list as they are dimensionless, so the conditions of model similarity for them can be written directly:

$$K_{\varepsilon} = 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

$$K_{\nu} = 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

$$K_{d} = 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

In the above expressions K represents the ratio between the size of the physical quantity as found on the prototype and that on the model.

The remaining parameters are suitably rearranged and a dimensional matrix is formed from them:

	k_1 m	k_2 Δ	k_3 σ	k_4 u	k_5 p	k_6 l	k_7 t
Force	1	1	1	0	1	0	0
Length	-1	-3	-2	1	0	1	0
Time	2	0	0	0	0	0	1

The equations expressing the unknown arguments are therefore:

$$k_1 + k_2 + k_3 + k_5 = 0$$
$$-k_1 - 3k_2 - 2k_3 + k_4 + k_6 = 0$$
$$2k_1 + k_7 = 0$$

The number of dimensionless products in the set is defined by the equation:

$a = n - r$, where

a ... is the number of dimensionless products in the set,

n ... is the number of physical parameters in the dimensional matrix, and

r ... is the highest order of determinant in the dimensional matrix.

For the case under consideration: $a = 7-3 = 4$.

The equations containing the unknown arguments can therefore be solved using four different combinations:

Combination:	Result:
$k_1 = 1,\ k_2 = 0,\ k_3 = 0,\ k_4 = 0$	$k_5 = -1,\ k_6 = 1,\ k_7 = -2$
$k_1 = 0,\ k_2 = 1,\ k_3 = 0,\ k_4 = 0$	$k_5 = -1,\ k_6 = 3,\ k_7 = 0$
$k_1 = 0,\ k_2 = 0,\ k_3 = 1,\ k_4 = 0$	$k_5 = -1,\ k_6 = 2,\ k_7 = 0$
$k_1 = 0,\ k_2 = 0,\ k_3 = 0,\ k_4 = 1$	$k_5 = 0,\ k_6 = -1,\ k_7 = 0$

The matrix of solutions has the form:

	k_1 m	k_2 Δ	k_3 σ	k_4 u	k_5 p	k_6 l	k_7 t
π_1	1	0	0	0	-1	1	-2
π_2	0	1	0	0	-1	3	0
π_3	0	0	1	0	-1	2	0
π_4	0	0	0	1	0	-1	0

The set of dimensionless products is therefore:

$$\pi_1 = \frac{ml}{pt^2} \ , \ \pi_2 = \frac{\Delta l^3}{p} \ , \ \pi_3 = \frac{\sigma l^2}{p} \quad \text{and} \quad \pi_4 = \frac{u}{l} \ .$$

The additional conditions of model similarity are therefore:

$$K_m K_l = K_p K_t^2 \qquad \ldots (4)$$

$$K_\Delta K_l^3 = K_p \qquad \ldots (5)$$

$$K_\sigma K_l^2 = K_p \qquad \ldots (6)$$

$$K_u = K_l \qquad \ldots (7)$$

2.2 Complete model similarity

If conditions (5) and (6) are combined the very important condition $K_\sigma = K_l K_\Delta$ is obtained. In the case of complete model similarity the latter condition is supplemented by the additional two conditions $K_\sigma = K_l$ and $K_\Delta = 1$. Then the physical parameters have to fulfill the following conditions:

$$K_\varepsilon = 1 \qquad K_\sigma = K_l$$

$$K_\nu = 1 \qquad K_u = K_l$$

$$K_d = 1 \qquad K_p = K_l^3$$

$K_m = K_l^3$ $K_l = \alpha$ α ... modelling scale

$K_\Delta = 1$ $K_t = K_l$

2.3 Simplified model similarity

In the case of simplified model similarity the additional condition $K_\sigma = 1$ is adopted. This makes it possible for the material of the prototype to be used for the construction of the model. However if this is done then the condition $K_\sigma = K_l K_\Delta$ is automatically unfulfilled. It can, however, be artificially corrected by increasing the forces due to the model's own weight S by S $(K_l - 1)$. When this condition is fulfilled the physical quantities must fulfill the following conditions:

$K_\varepsilon = 1$ $K_\sigma = 1$

$K_\nu = 1$ $K_u = K_l$

$K_d = 1$ $K_p = K_l^2$

$K_m = K_l^3$ $K_l = \alpha$

$K_\Delta = 1$ $K_t = K_l$

2.4 General model similarity

In practice we are sometimes forced, for various reasons, to depart from the modelling conditions which should ideally be fulfilled by the model material. This, however, does not mean to say that it is not possible to obtain useful results with the choice of a suitable testing method. When generalized, the coefficients of similarity which refer to the model material can be written as follows:

$K_\sigma = \beta$, $K_\Delta = \delta$, $K_\varepsilon = \gamma$, $K_\nu = \phi$ and $K_d = \psi$.

2.4

The condition $K_\sigma = K_1 K_\Delta$, which now becomes $\beta=\alpha\delta$, is, in general, not fulfilled. This error can be compensated for by increasing the forces due to the model's own weight S by S $(\alpha\delta/\beta-1)$. In this case the following conditions must be fulfilled by the physical quantities:

$K_\varepsilon = \gamma$ $\quad$ $K_\sigma = \beta$

$K_\nu = \phi$ $\quad$ $K_u = \alpha\gamma$

$K_d = \psi$ $\quad$ $K_p = \alpha^2\beta$

$K_m = \alpha^3\delta$ $\quad$ $K_1 = \alpha$

$K_\Delta = \delta$ $\quad$ $K_t = \alpha\sqrt{\gamma\delta/\beta}$

It must be pointed out that departure from the conditions $K_\nu = 1$ and $K_d = 1$ cannot be taken into consideration by making suitable corrections of the remaining coefficients and for this reason it is necessary to determine the effect of departure from these conditions on the phenomena being tested separately in each individual case.

2.5 General considerations

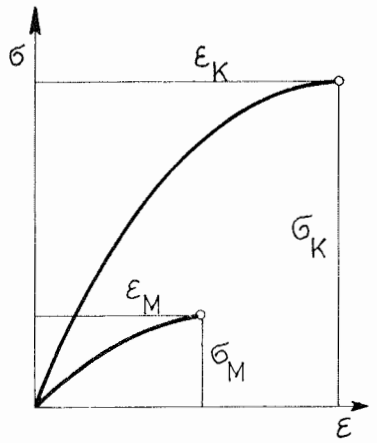
A more complete representation of the coefficients of model similarity, embracing a greater scope of physical phenomena and envisaging wider possibilities for modelling, is provided on pages 10, 11 and 12. In the preparation of these tables considerable reference has been made to Nazarov's publication[1].

PHYSICAL QUANTITY		GENERAL SIMILARITY $\beta \neq \alpha\delta$	SIMPLIFIED SIMILARITY $\delta=1$, $\beta=1$ $\beta \neq \alpha\delta$	COMPLETE SIMILARITY $\beta=\alpha$, $\delta=1$ $\Psi=1$, $\gamma=1$, $\varphi=1$
CASE:		1	2	3
Length l		α	α	α
Stress σ		β	1	α
Strain ε	should be	1	1	1
	is	γ	γ	1
Specific weight Δ		δ	1	1
Poisson's ratio ν	should be	1	1	1
	is	φ	φ	1
Log. decrement of damping d	should be	1	1	1
	is	Ψ	Ψ	1
Modulus of elasticity E		β/γ	$1/\gamma$	α
Mass m weight T		$\alpha^3\delta$	α^3	α^3
Stress gradient $\bar{G}$	should be	1	1	1
	is	β/α	$1/\alpha$	1
Force P		$\alpha^2\beta$	α^2	α^3
Stress due to own weight $\bar{\sigma}$	should be	β	1	α
	is	$\alpha\delta$	α	α
Increase vert. force on model by fac. Tm x		$\alpha\delta/\beta - 1$	$\alpha - 1$	0
Load/unit length q		$\alpha\beta$	α	α
Bending moment M		$\alpha^3\beta$	α^3	α^4
Time t		$\alpha\sqrt{\gamma\delta/\beta}$	$\alpha\sqrt{\gamma}$	$\sqrt{\alpha}$
Velocity v		$\sqrt{\beta\gamma/\delta}$	$\sqrt{\gamma}$	$\sqrt{\alpha}$
Acceleration w		$\beta/\alpha\delta$	$1/\alpha$	1
Displacement $\bar{u}$		$\alpha\gamma$	$\alpha\gamma$	α
Work u		$\alpha^3\beta\gamma$	$\alpha^3\gamma$	α^4
Spreading velocity of strains a		$\sqrt{\beta/\gamma\delta}$	$\sqrt{1/\gamma}$	$\sqrt{\alpha}$
Impulse S		$\alpha^3\sqrt{\beta\gamma\delta}$	$\alpha^3\sqrt{\gamma}$	$\sqrt{\alpha^7}$
Angle φ		γ	γ	1
Angular velocity ω		$1/\alpha\sqrt{\beta\gamma/\delta}$	$1/\alpha\sqrt{\gamma}$	$\sqrt{1/\alpha}$
Angular acceleration $\dot{\omega}$		$\beta/\alpha^2\delta$	$1/\alpha^2$	$1/\alpha$

PHYSICAL QUANTITY		SPECIAL CASES OF GENERAL SIMILARITY		
		$\delta = 1$ $\beta = \alpha$ $\beta = \alpha\delta$	$\delta = 1/\alpha$ $\beta = 1$ $\beta = \alpha\delta$	$\beta = \alpha\delta$
CASE:		4	5	6
Length l		α	α	α
Stress σ		α	1	$\alpha\delta$
Strain ε	should be	1	1	1
	is	γ	γ	γ
Specific weight Δ		1	$1/\alpha$	δ
Poisson's ratio ν	should be	1	1	1
	is	φ	φ	φ
Log. decrement of damping d	should be	1	1	1
	is	ψ	ψ	ψ
Modulus of elasticity E		α/γ	$1/\gamma$	$\alpha\delta/\gamma$
Mass m, weight T		α^3	α^2	$\alpha^3\delta$
Stress gradient $\bar{G}$	should be	1	1	1
	is	1	$1/\alpha$	δ
Force P		α^3	α^2	$\alpha^3\delta$
Stress due to own weight $\bar{\sigma}$	should be	α	1	$\alpha\delta$
	is	α	1	$\alpha\delta$
Increase vert. force on model by fac. Tm x		$\emptyset$	$\emptyset$	$\emptyset$
Load/unit length q		α^2	α	$\alpha^2\delta$
Bending moment M		α^4	α^3	$\alpha^4\delta$
Time t		$\sqrt{\alpha\gamma}$	$\sqrt{\alpha\gamma}$	$\sqrt{\alpha\gamma}$
Velocity v		$\sqrt{\alpha\gamma}$	$\sqrt{\alpha\gamma}$	$\sqrt{\alpha\gamma}$
Acceleration w		1	1	1
Displacement $\bar{u}$		$\alpha\gamma$	$\alpha\gamma$	$\alpha\gamma$
Work u		$\alpha^4\gamma$	$\alpha^3\gamma$	$\alpha^4\gamma\delta$
Spreading velocity of strains a		$\sqrt{\alpha/\gamma}$	$\sqrt{\alpha/\gamma}$	$\sqrt{\alpha/\gamma}$
Impulse S		$\sqrt{\alpha^7\gamma}$	$\sqrt{\alpha^5\gamma}$	$\sqrt{\alpha^7\gamma\delta^2}$
Angle φ		γ	γ	γ
Angular velocity ω		$\sqrt{\gamma/\alpha}$	$\sqrt{\gamma/\alpha}$	$\sqrt{\gamma/\alpha}$
Angular acceleration $\dot{\omega}$		$1/\alpha$	$1/\alpha$	$1/\alpha$

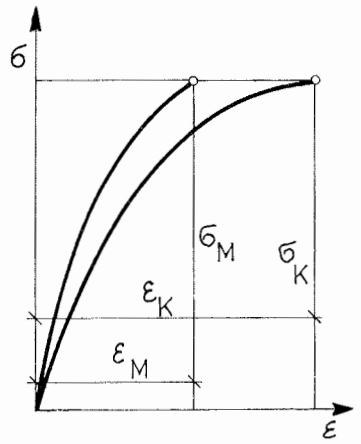
SHAPE OF $\sigma - \varepsilon$ DIAGRAMS FOR INDIVIDUAL CASES (ONLY COMPRESSIVE ZONE IS SHOWN)

CASE 1



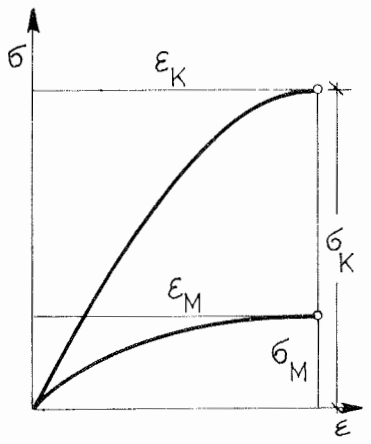
$$\frac{\sigma_K}{\sigma_M} = \beta \,, \quad \frac{\varepsilon_K}{\varepsilon_M} = \gamma$$

CASE 2



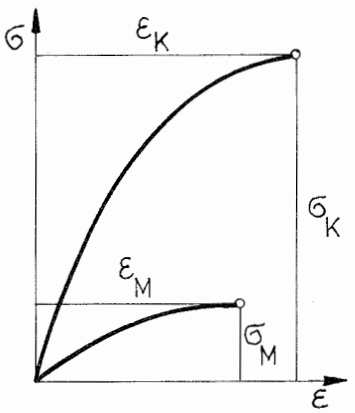
$$\frac{\sigma_K}{\sigma_M} = 1 \,, \quad \frac{\varepsilon_K}{\varepsilon_M} = \gamma$$

CASE 3



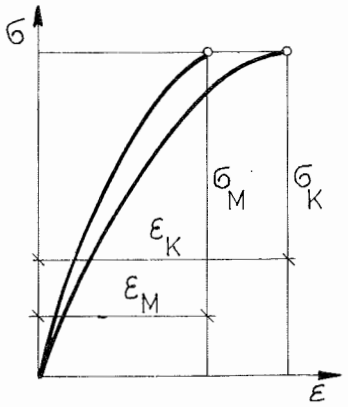
$$\frac{\sigma_K}{\sigma_M} = \alpha \,, \quad \frac{\varepsilon_K}{\varepsilon_M} = 1$$

CASE 4



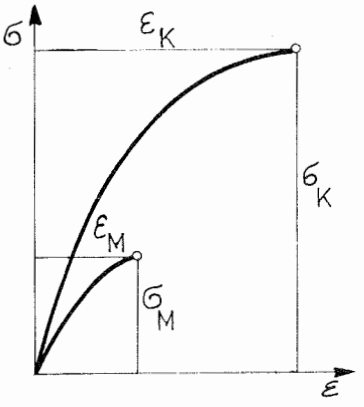
$$\frac{\sigma_K}{\sigma_M} = \alpha \,, \quad \frac{\varepsilon_K}{\varepsilon_M} = \gamma$$

CASE 5



$$\frac{\sigma_K}{\sigma_M} = 1 \,, \quad \frac{\varepsilon_K}{\varepsilon_M} = \gamma$$

CASE 6



$$\frac{\sigma_K}{\sigma_M} = \alpha\sigma \,, \quad \frac{\varepsilon_K}{\varepsilon_M} = \gamma$$

The various possibilities for modelling which are available have been demonstrated in the previous section. With the recent advances which have been made in computer handling techniques it is clear that modelling techniques can be used to their greatest advantage in the demanding field of dynamic tests of structures in the plastic region. In this case, modelling should be carried out, in principle, under conditions of complete model similarity. The basis for the latter statement is that under conditions of complete model similarity: firstly, there is no need to provide any artifical compensation which would otherwise be needed to make up for the model's insufficient own weight (in the majority of cases such compensation causes very great technical difficulties) and, secondly, the stress gradients on the model and the prototype are then the same (otherwise errors can arise in the results obtained).

Models test under conditions of complete model similarity can, however, only be carried out if advanced testing and measuring equipment is possessed and if specially-developed materials which make possible such modelling are available.

In the search for modelling materials which should fulfill the conditions of complete model similarity the question always arises as to whether an attempt should be made to fulfill a condition strictly - for this to succeed a comprehensive program of research work usually has to be carried out - or whether a certain degree of approximation to the condition should be tolerated, correspondingly less work then being needed.

Experience so far gained at ZRMK has shown us that it is much more economical to develop modelling materials which fulfill the conditions only as well as can be reasonably achieved, and then to treat the model according to the theory of general model similarity. In this case it is especially recommended

that the condition $\beta = \alpha\delta$ be strictly fulfilled, as then the awkward adding of compensating vertical loads is avoided in the case of general model similarity, too. Since the scale-factor α is a fundamental constant, special care must be paid to the relationship between strength β and specific weight δ.

If the problem being dealt with is well-understood then it is possible to depart a considerable amount from a particular condition without substantially spoiling the result. Thus it is advisable that the "weighting" assigned to the fulfilling of any particular condition be considered in advance, before the model is tested. In the following section an account is given of how the flexibility of modelling techniques was made use of in the treatment of a model test of a stone-masonry building, with which a successful attempt was made to quantify the effect of a particular method-of-repair for earthquake-damaged stone-masonry buildings.

3. THE TESTING OF A MODEL STONE-MASONRY BUILDING ON THE SHAKING-TABLE

3.1 Scope of the test

Considerable damage was caused to stone-masonry buildings in Slovenia by the 1974 Kozjansko earthquake and the 1976 earthquakes which hit the Soča Valley region. As a result our building enterprises were faced with the task of carrying out a lot of high-standard repairs to these buildings. A study of their design showed that they were mainly of either single -storey or two-storey construction, and that their walls were built of uncoursed stone rubble and poor-quality lime-sand mortar. Whereas the lengthwise standing walls were usually loaded, having to support the weight of timber floors and roof structure, the head (transverse) walls were not.

An analysis of the damage caused by the above-mentioned earthquakes showed that in the great majority of cases cracks had occurred at the junctions between the lengthwise-standing and head walls, that corners had broken away, and that in some cases collapse of the head-walls had taken place.

As an idea of what would be the most suitable way of repairing damaged stone-masonry buildings it was proposed that after any necessary rebuilding of parts of walls it would be necessary to tie the building at ceiling height with double steel tie-bars of diameter Ø 16 mm (one on either side of each wall) and to increase the fundamental shear strength of the walls by pressure-injecting them with cement grout. This idea was tested out in the laboratory on a model stone-masonry building and the successfulness of this method-of-repair was

thus quantitified. The scope of the tests involved was as follows:

a) Test walls ("Prototype wall I") were built from the actual materials used in the damaged buildings and tested under combined seismic load (cyclic horizontal loading at constant vertical load) until their maximum resistance was reached in order to determine their load-carrying capacity and deformation characteristics.

b) The same walls were repaired with the high-quality pressure injecting of cement grout into them ("Prototype wall II") and retested to failure under combined seismic load; load-carrying capacity, deformation characteristics and the success of the repair were determined.

c) A typical single-storey stone-masonry building from the earthquake-hit region was chosen, and a model of it was made at a scale of 1:5 (see Fig.1); this model was then subjected to dynamic loads on the shaking-table (see Fig. 2) until the typical kinds of damage expected were obtained: cracks at the joints between the lengthwise-standing walls* and the head-walls, the pulling-away of corners, and the collapse of one of the head-walls (see Fig.3).

d) The damaged model was repaired by rebuilding and pressure-injecting the head-walls and by providing model steel tie-bars, and then retested to failure, which this time occurred with the collapse of the lengthwise-standing walls, the head-walls remaining intact (see Fig.4).

e) An analysis was then carried out of the failure mechanisms obtained, the quantities measured on the model were recomputed for the prototype, and the effect of the method-of--repair used was thus determined.

* parallel to the axis of movement of the shaking-table

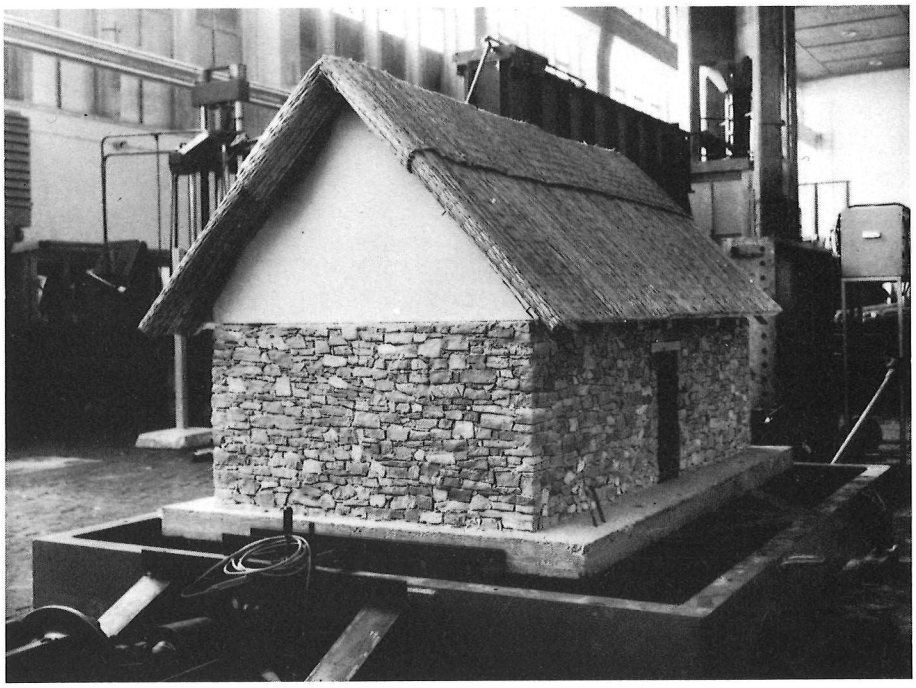


Fig. 1

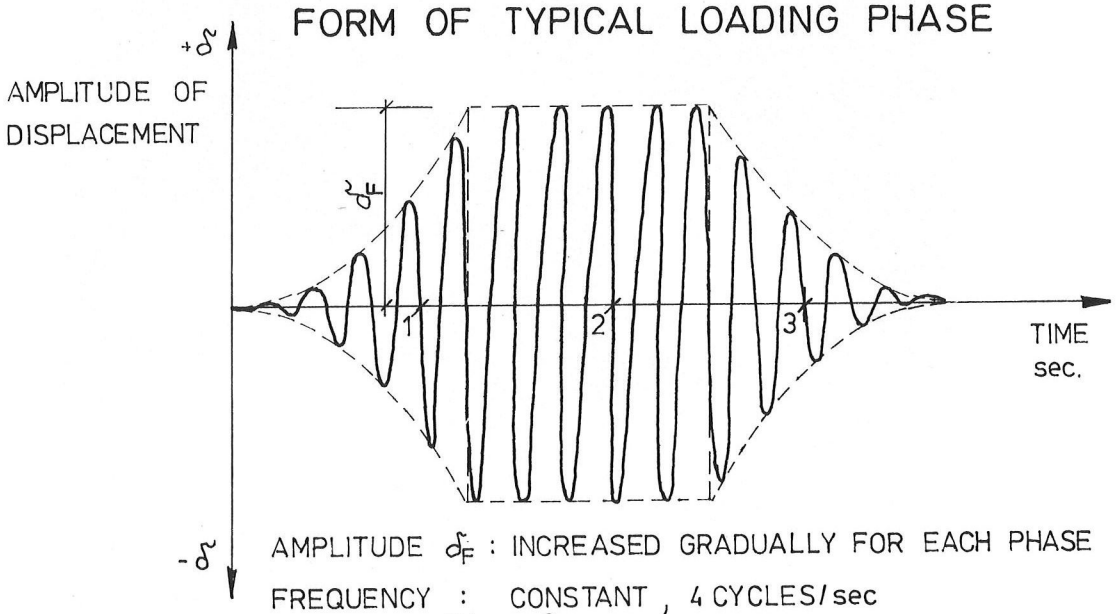

FORM OF TYPICAL LOADING PHASE
+δ
AMPLITUDE OF
DISPLACEMENT
δF
1
2
3
TIME
sec.
-δ
AMPLITUDE δF : INCREASED GRADUALLY FOR EACH PHASE
FREQUENCY : CONSTANT , 4 CYCLES/sec

Fig. 2

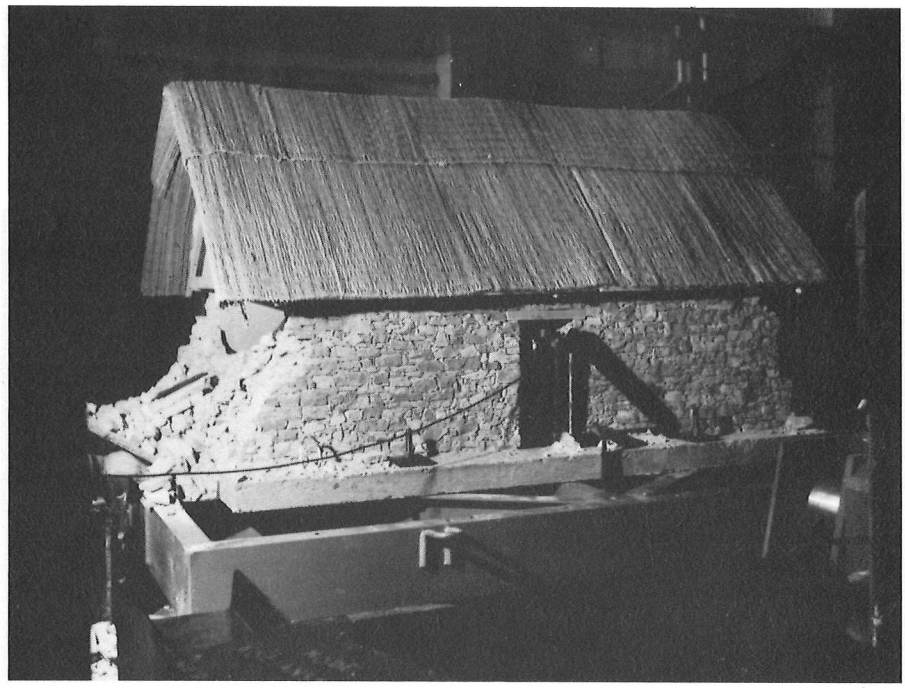


Fig. 3

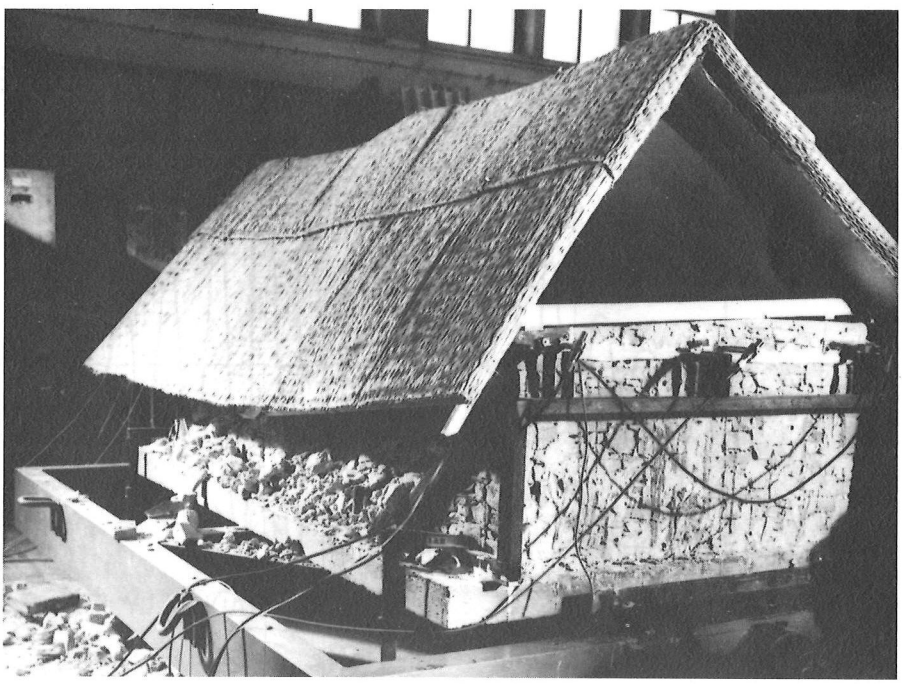


Fig. 4

3.2 Principles involved in the modelling of the stone-masonry building

In principle it was decided that the model should be built so that the conditions of complete model similarity be fulfilled. It was our aim to model the quality of the original prototype test walls ("Prototype wall I"). In advance it was realized that, in spite of all efforts, on account of the complicatedness of the prototype materials (low quality of mortar, irregular shape of stone, undefined quality of masonry-work) it could not be expected that modelling might be carried out in complete accordance with the necessary laws, and that it would be necessary to make use of the general model similarity approach, too, in the analysis of the results obtained.

As a basic test element for the determination of the quality of the model materials used a model wall, representing the prototype wall at a reduced scale of 1:5, was chosen. This model wall served for making strength comparisons between the model and the prototype wall and for determining the reference shear strength τ_k:

$$\tau_k = \frac{1}{1,5}\left[\sqrt{(1,5\ \tau_o)^2 + \left(\frac{\sigma_o}{2}\right)^2} - \frac{\sigma_o}{2}\right]$$

where $\tau_o = H/F$ ..... average shear stress in wall at failure

$\sigma_o = V/F$ ..... normal stress in wall due to vertical load

The following values of τ_k were obtained from the results of combined seismic loading tests on prototype and model walls:

prototype wall I ..... $\tau_k' = 1{,}44$ Mp/m^2

prototype wall II ..... $\tau_k'' = 8{,}00$ Mp/m^2

model wall ..... $\tau_k^M = 1{,}69$ Mp/m^2

On the basis of the ratios between the values given above the following coefficients β were obtained:

$$\beta_1 = \frac{\tau_k'}{\tau_k^M} = \frac{1{,}44}{1{,}69} = 0{,}852$$

$$\beta_2 = \frac{\tau_k''}{\tau_k^M} = \frac{8{,}00}{1{,}69} = 4{,}73 \simeq 5 = \alpha$$

Inspection of these values shows that the model building can be treated in two ways:

- either according to general model similarity, as "prototype building I " , with walls of shear strength τ_k = = 1,44 Mp/m^2 ,

- or, according to complete model similarity, as "prototype building II ", with walls of shear strength τ_k = 8,00 Mp/m^2.

The relationships between the test walls, the model, and the prototype buildings are shown schematically in Fig. 5, on the next page.

SCHEMATIC ANALYSIS OF MODEL USING TWO DIFFERENT THEORIES OF MODEL SIMILARITY

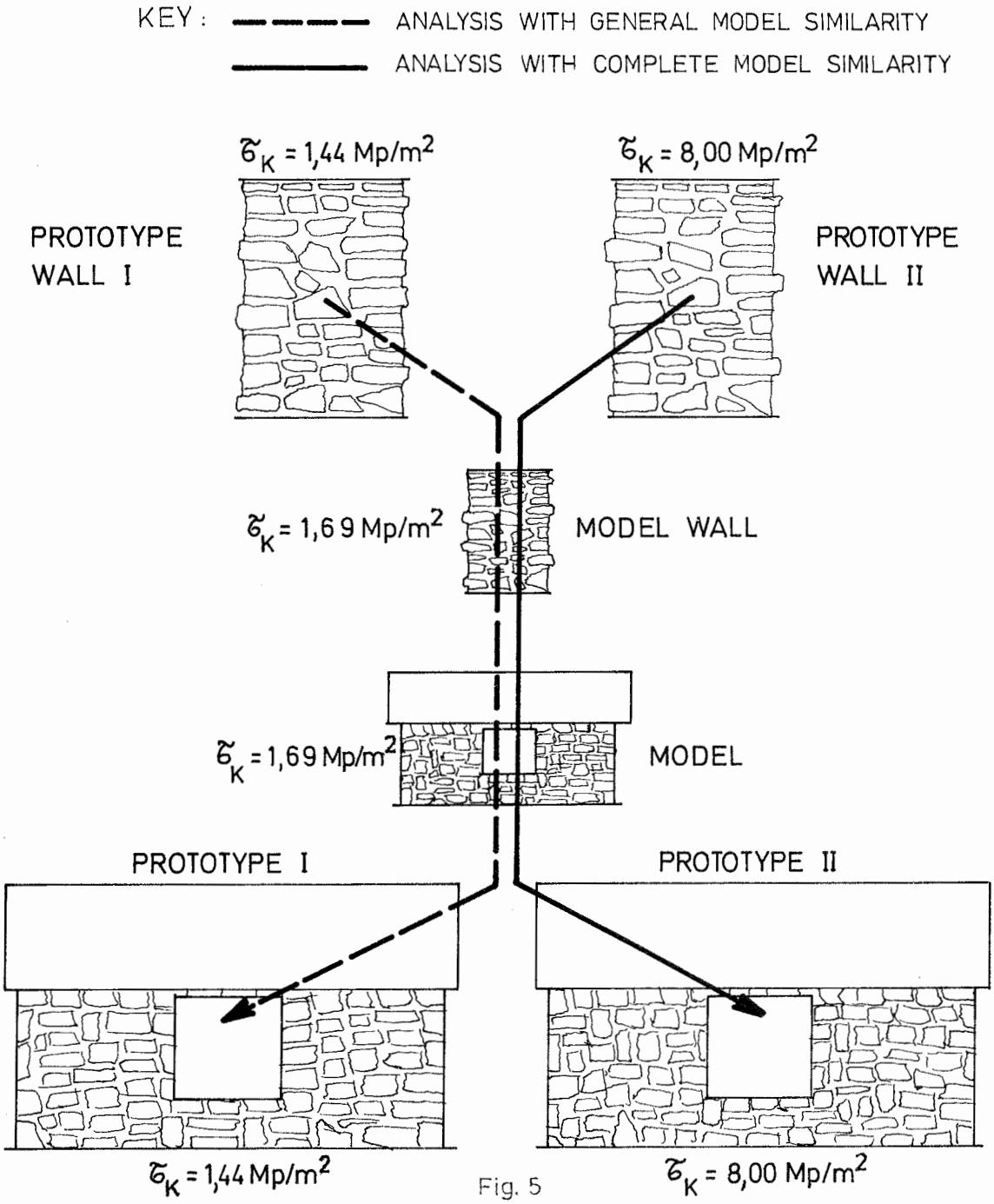


Fig. 5

3.3 "Prototype building I"

With respect to the fact that modelling was carried out using coefficients $\alpha = 5$, $\delta = 1$ and $\beta = 0{,}85$ the vertical forces acting on the model due to its own weight should have been increased by:

$$S' = S\ (\alpha\delta/\beta - 1) = S\ (5.1/0{,}85 - 1) = 4{,}88\ S.$$

As modelling had been carried out in principle according to complete model similarity these additional vertical forces had not been provided on the model, thus causing a reduction in the latter's shear-strength capacity. As a result, failure of the model occurred at a lower horizontal loading than would otherwise have been the case.

With respect to the above it is true to say that the state-of-stress in the model was not entirely the correct one, but, on the other hand, since the same kind of damage and failure mechanism was obtained as that caused in buildings by a real earthquake, we are justified in taking the difference in vertical loading into account mathematically and then suitably adjusting the coefficients of model similarity.

For the correction of the measured horizontal failure load, use is made of the equation: $\tau_o = \tau_k \cdot \sqrt{1 + \sigma_o/(1{,}5\tau_k)}$

$$k = \frac{\tau_o''}{\tau_o'} = \frac{\tau_k^M \sqrt{1+\sigma_o''/(1{,}5\tau_k^M)}}{\tau_k^M \sqrt{1+\sigma_o'/(1{,}5\tau_k^M)}} = \sqrt{\frac{1{,}5\tau_k^M + \sigma_o''}{1{,}5\tau_k^M + \sigma_o'}} =$$

$$= \sqrt{\frac{1{,}5.1{,}69+\sigma_o''}{1{,}5.1{,}69+\sigma_o'}} = \frac{H_2}{H_1}\ .$$

H_1 = actual force which was needed to cause failure of model wall under vertical loading σ_o';

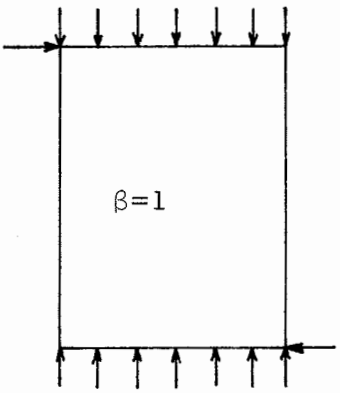
H_2 = force which would have been needed to cause failure of same wall if subjected to the increased vertical loading σ_o''.

The values of σ_o' and σ_o'' involved and the relationships between the horizontal forces acting on prototype and model walls (H^P and H^M) are shown in the following diagrams:

$\frac{\sigma^P}{1}$

$H^P = 25.H^M.1$

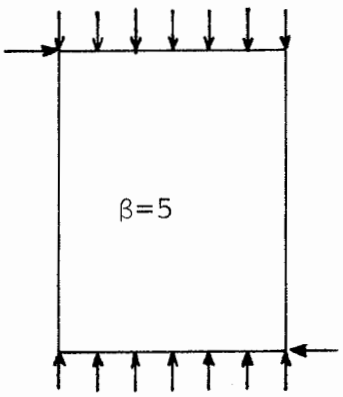
Simplified model similarity



$\sigma_o' = \frac{\sigma^P}{5}$

$H^P = 25.H^M.5$

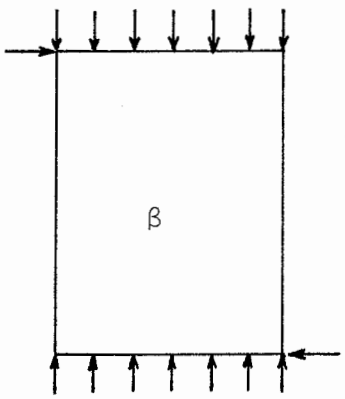
Complete model similarity



$\sigma_o'' = \frac{\sigma^P}{\beta}$

$H^P = 25.H^M.\beta$

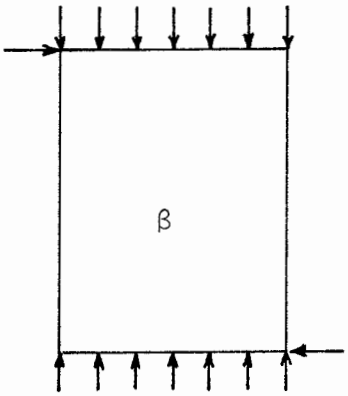
General model similarity



$\frac{\sigma^P}{5}$

$H^P = 25.H^M.k.\beta$

Actual conditions



The prototype walls were loaded vertically with a normal stress of $\sigma^p = 13,4\ Mp/m^2$. We therefore obtain:

$$\sigma_o' = \frac{\sigma^p}{5} = \frac{13,4}{5} = 2,67\ Mp/m^2 \text{ and}$$

$$\sigma_o'' = \frac{\sigma^p}{\beta} = \frac{13,4}{0,852} = 15,77\ Mp/m^2$$

$$\text{Thus } k = \sqrt{\frac{2,54+15,77}{2,54+2,67}} = 1,87.$$

The coefficient of model similarity which defines the relationship between the force measured on the model and the corresponding force on the prototype (K_p) is then given by the expression:

$$K_p = \frac{H^p}{H^M} = 25.k.\beta = 25.1,87.0,852 = 39.83.$$

On the basis of the values obtained for k and β and an analysis of the other physical parameters the expressions for and values of the remaining coefficients of model similarity were obtained. They are shown on the next page in the column marked "Model I" of the table provided.

The above considerations show that although not all the conditions of general model similarity were actually fulfilled, all the same, because the same damage mechanism was obtained as that caused by a real earthquake, this method can be justifiably used for the drawing of conclusions.

3.4"Prototype building II "

As has already been mentioned, the model stone-masonry building can also be treated by the theory of complete model similarity, using the coefficients $\alpha=5$, $\delta=1$ and $\beta=5$. In this case the model represents a building with systematically cement-grout injected walls ($\tau_k=8,00\ Mp/m^2$). The coefficients of similarity, given as expressions with their actual values as calculated, are shown in the table on the following page, in the column marked "Model II".

COEFFICIENTS OF MODEL SIMILARITY

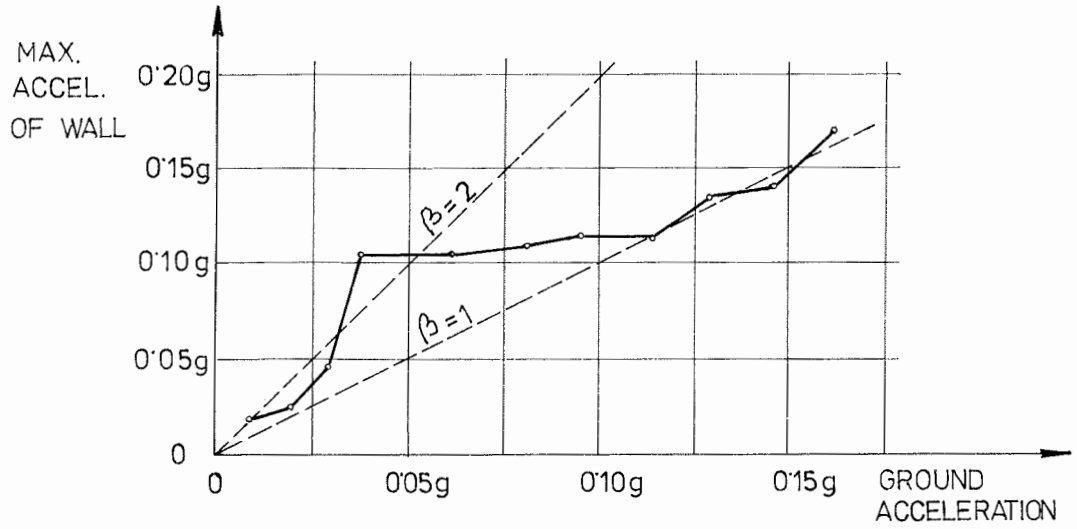
COEFFICIENT	PHYSICAL QUANTITY	MODEL I		MODEL II	
		GENERAL SIMILARITY		COMPLETE SIMILARITY	
K_l	Length	α	5,000	α	5,000
K_σ	Strength	β	0,852	$\alpha = \beta$	5,000
K_ε	Strain	$\gamma = 1$	1,000	$\gamma = 1$	1,000
K_Δ	Specific weight	1	1,000	1	1,000
K_ν	Poisson's ratio	1	1,000	1	1,000
K_d	Log. decrement of damping	1	1,000	1	1,000
K_u	Displacement	$k\alpha\gamma$	9,350	$\alpha\gamma$	5,000
K_P	Force	$k\alpha^2\beta$	39,831	$\alpha^2\beta$	125,000
K_t	Time	$\alpha\sqrt{\frac{\gamma\delta}{\beta}}$	5,417	$\alpha\sqrt{\frac{\gamma\delta}{\beta}}$	2,236
K_v	Velocity	$k\sqrt{\frac{\beta\gamma}{\delta}}$	1,726	$\sqrt{\frac{\beta\gamma}{\delta}}$	2,236
K_w	Acceleration	$k\frac{\beta}{\alpha\delta}$	0,319	$\frac{\beta}{\alpha\delta}$	1,000
K_f	Frequency	$\frac{1}{\alpha}\sqrt{\frac{\beta}{\gamma\delta}}$	0,185	$\frac{1}{\alpha}\sqrt{\frac{\beta}{\gamma\delta}}$	0,447

3.5 Practical conclusions

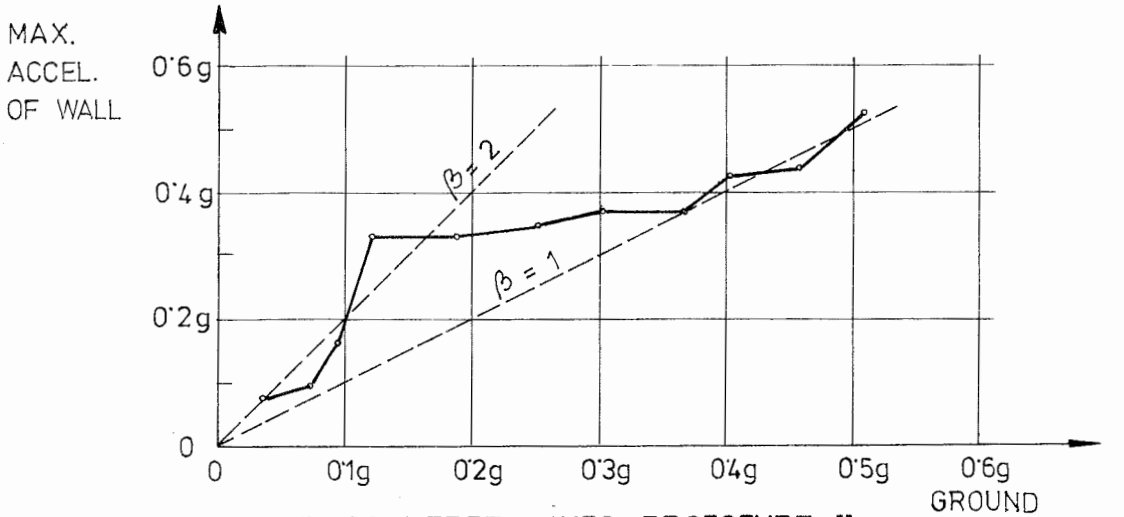
The main practical conclusions obtained with the model test are shown in the table given below.

BUILDING TYPE	NON-TIED BUILDING		BUILDING REPAIRED WITH STEEL TIE-BARS	
	Phenomenon	Ground acc.	Phenomenon	Ground acc.
PROTOTYPE BUILDING "I" $\tau_k = 1,44 \frac{Mp}{m^2}$	First crack at joint between walls	0,04g	First crack at joint between walls	0,05g
	Spreading of crack, pulling away of corner	0,06g	↓	
	Collapse of head-wall	0,17g ↔	Head-walls still undamaged	0,17g
			Collapse of lengthwise standing walls	0,42g
PROTOTYPE BUILDING "II" WITH SYSTEMATICALLY INJECTED WALLS $\tau_k = 8,00 \frac{Mp}{m^2}$	First crack at joint between walls	0,12g	First crack at joint between walls	0,16g
	Spreading of crack, pulling away of corner	0,19g	↓	
	Collapse of head-wall	0,53g ↔	Head-walls still undamaged	0,53g
			Collapse of lengthwise standing walls	1,30g

MAX. ACCELERATION OF WALL PARALLEL TO EARTHQUAKE LOAD



MODEL CONVERTED INTO PROTOTYPE I
BY GENERAL MODEL SIMILARITY



MODEL CONVERTED INTO PROTOTYPE II
BY COMPLETE MODEL SIMILARITY

Fig. 6

Apart from the above table, in Fig. 6 the acceleration of the longitudinal walls of both prototype stone-masonry buildings I and II has been plotted against ground acceleration.

On the basis of the results shown it can be concluded that with the tying together of the walls with steel tie-bars the collapse of head-walls is prevented. Failure of the building does not take place until a substantially much greater loading is reached, causing,this time, the collapse of the lengthwise-standing walls.

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ELASTIC PLASTIC DEGRADING OSCILLATORS BEHAVIOUR UNDER EARTHQUAKES - PRELIMINARY RESULTS

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1. General Background

In studying antiseismic structures the responses given by simple oscillators are usually utilized. The oscillators are meant to simulate the dynamic behaviour of structures, at a first approximation level.

The constitutive link adopted for these oscillators has been gradually refined by the various authors who have considered successively :

- a linear elastic law which takes into consideration the dissipating effects by itroducing a viscosity which is also linear;
- a perfectly plastic elastic law in which the dissipating effects are linked to the plastic deformations;
- a perfectly plastic elastic law having a progressive degradation of mechanical characteristics.

The latter law is particularly suitable for describing the behaviour of reinforced concrete structural elements since, for some types of compound stress these elements are particularly apt to undergo an increasing degradation when subjected to alternating high intensity loads, as occurs during violent earthquakes. The problem then arises of examining the possible plastic engagement of an oscillator of the third type in the response to assigned accelerograms as the threshold of the initial plasticity varies, by considering also the evolution of the degradation phenomena.

2. Definition of the Model

The behaviour of reinforced concrete structural elements subjected to alternating high intensity loads which are repeated a limited number of times has been examined by numerous authors [2,8,9]; however, to the present, only in a few cases the importance of the degradation of the mechanical characteristics as the very load cycles are repeatd, has been pointed out [4,10].

Referring especially to elements which are subject to a constant axial force N and to an alternating bending moment M, a progressive reduction of the resistance characteristics and of the flexural rigidity, may be observed in the following cases:

- as the number of cycles increases (see fig. 1) and
- as the value of the axial force increases (see fig. 2)

In more detail, figure 1 reproduces a typical diagram of moments M as a function of imposed curvatures χ; figure 2 represents the trend of the maximum value of M as a function of the number of n cycles.

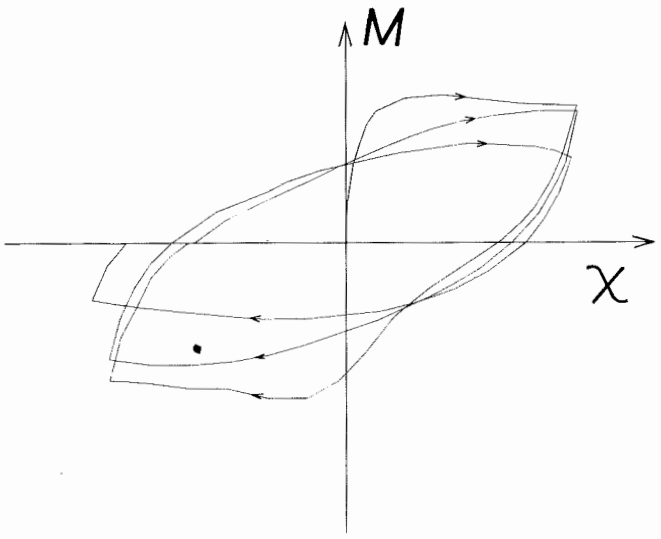

fig.1

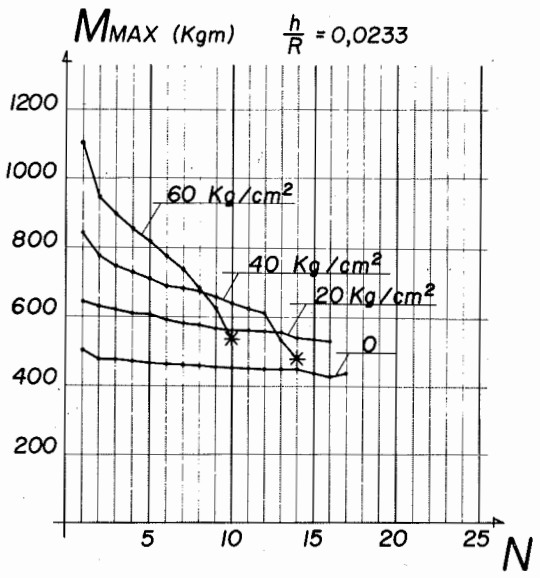

fig.2

For the definition of a calculation model in this study it is hypothesized that the degradation depends exclusively from the sum of the absolute value of the paths in the plastic field (*); by then indicating with C a multiplying coefficient of the mechanical characteristics defined "degradation coefficient", one sets up the following :

$$C = C(s) \qquad \text{being} \qquad C \leq 1$$

By examining the experimental results of the available test [9,10], it seems suitable to link the form of law $C(s)$ to the constant value of the axial force N, this setting up

$$C = C(s, N)$$

Figure 3 illustrate a general hypothesis of behaviour in which coefficient C, having an initial value of 1, reaches the limit value C_d

(*) This criterion is equivalent to linking the degradation to the energy dissipated during the plastic phases, holding this to be the principal cause of modification of the natural characteristics of the structure.

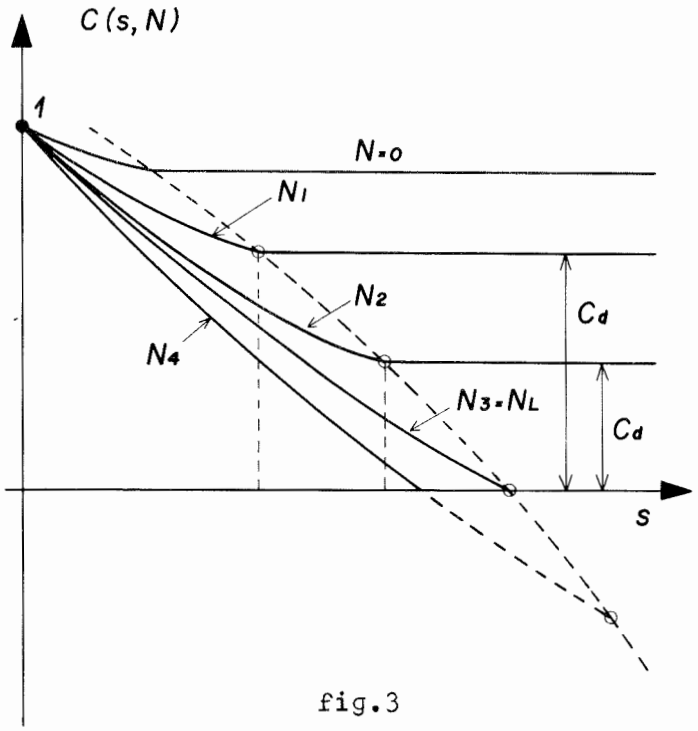

fig.3

depending on $\mathcal{N}$; besides this value, each subsequent increase of s does not produce further reductions of the mechanical characteristics.

The described behaviour may be traced back to the mechanism illustrated in figure 4 for a section having a double symmetric reinforcement (column).

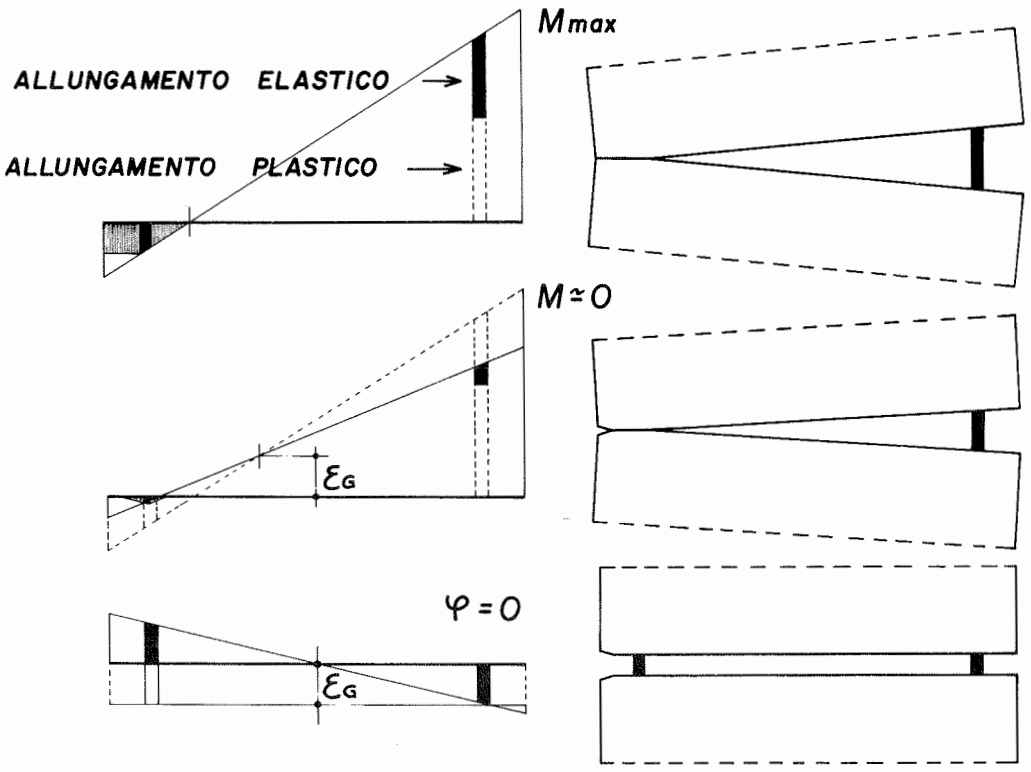

fig. 4

Due to the yelding of the steel and to the lack of traction resistance of the concrete, the cracks which form in the latter tend to become progressively larger, as bending alternations are repeated, until, there being no axial force, cooperation of the concrete may be totally excluded [9,10].

Then going back to fig. 3, curve $N=O$, which reproduces the situation described for the case of simple bending moment the first part of the curve from $s=O$ to $s=s_1$, shows the effect of the progressive enlarging of the cracks. The subsequent horizontal section, on the other hand, indicates the reaching of the residual moment capable of being borne by the reinforcements alone. The mechanism described is also influenced by the value of the axial force N, which reduces the rextant entity of the residual moment (for example curve N_2). For the axial force, however, a limit value N_L exists. It corresponds to the compressionyielding of all the reinforcements, beyond which it is no longer possible to obtain that cooperation of the compressed concrete is absent. In this case, as the cycles proceed, the resisting moment, decreases to very low values due to the great damage undergone by the concrete near the edges of the section. These values, have been approximated to zero in fig. 3. In fact, the experimental tests mentioned above have pointed out that for $N<N_L$ definite cracking of the section may never be reached, while for $N>N_L$ cracking always occurs at a number of cycles which is so much the lower as the value of the axial force N is higher, as can also be deduced by examining fig. 2 again.

Finally, fig. 5 shows the width of the above mentioned cracks in correspondence of the barycentric fibre of the loaded beams measured for each cycle in condition $\chi=O$. In curves a) and b)

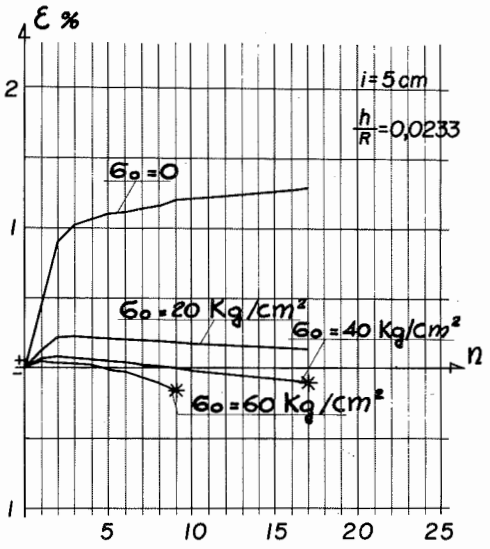

fig. 5

corresponding to $N<N_L$, the knee point indicates the reaching of the residual moment; curves c) and d) correspond instead to $N>N_L$.

To come to the point, a first analysis of the already mentioned experimental results has led to a simplification of the diagrams of figure 3, according to the outline shown in fig. 6 . Altough this outline should in any case be considered as a first approach to the problem, it may be maintained to be valid in the field $0 \leq N \leq N_L$ (*). In the proposed law it was established that the descending section of curve $C(s,N)$ be independent from N, until the value of s corresponding to the already mentioned limit value C_d of the degradation coefficient were reached. The numerical analysis of experiments conducted for different values of N, allows one to hypothesize a value equal to 100 for the parameter α of fig. 6.

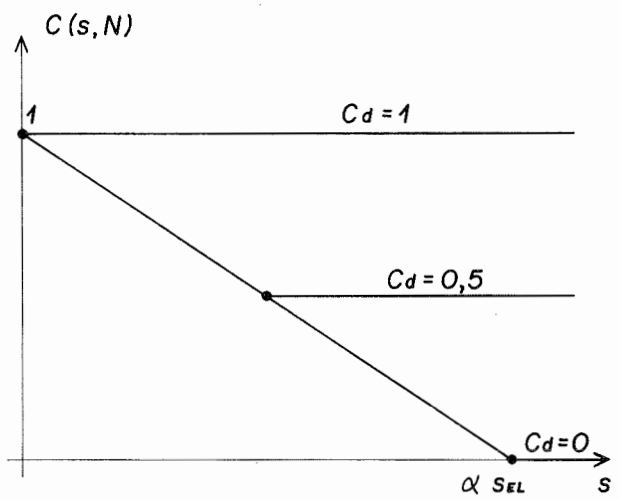

fig. 6

(*) The field of axial force being analysed corresponds closely enough to actual situations occurring in seismic zones. For example, for sections having reinforcement percentages equal to 1,5% and a steel yelding limit equal to 4,400 Kg/cm^2 , the value of the average pressure on the concrete corresponding to N_L is 60 Kg/cm^2.

3. Elastic Plastic Degrading Oscillator

Fig. 7 shows a common elementary type elastic plastic oscillator having linear damping and the equations of its movement both during the elastic and plastic phases.

By adopting the degradation fig.7 criterion described in paragraph 2 and summarized in fig. 6, a progressive reduction of the plastic threshold is obtained according to the law $\lambda_c = \lambda C$.

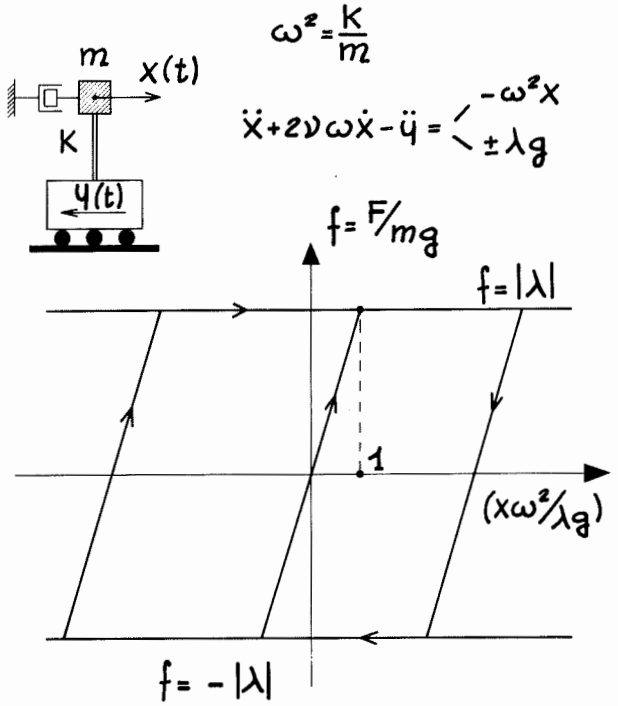

fig. 7

A similar reduction was also worked out on the elastic rigidity K of the oscillator, according to law $K_c = KC$.

At each instant the equation of the movement of the oscillator is then modified as follows :

$$\ddot{x} + 2\nu\omega\dot{x} - \ddot{y} = \begin{cases} -\omega^2 C x \\ \pm\lambda g C \end{cases} \qquad 1)$$

Consequently paths of the type show in fig. 8 are obtained.

Equation 1) has been discretized using the finite differences method and integrated step by step with an appropriate program, pro-

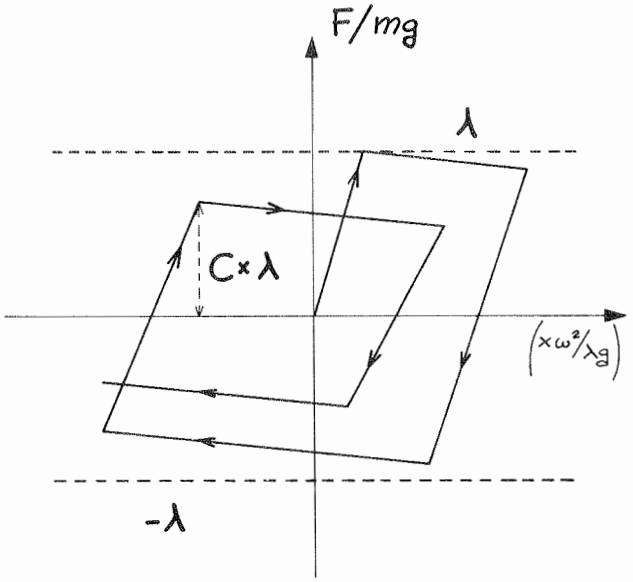

fig. 8

viding a genering digitalized accelerogram for $\ddot{y}$ (*). In particular for the numerical applications the results of which are shown in the next paragraph, the component NS of the accelerogram registered at Tolmezzo on May 6th, 1976 at 9.01 P.M. was used [12] .

The typical oscillator parameters utilized are the following :

- the oscillators own T_o period in an elastic phase variable from 0.2 sec. to 1.6 sec. at intervals of 0.025 sec.;
- the elastic threshold variating from 0.05 to 0.3 at intervals of 0.05 (besides, the indefinitely elastic solution was also considered);
- the degradation limit coefficient C_d equal to 1, 0.5, 0.

Regarding the values chosen for the limit coefficient C_d, it was established that the value 1 be representative of the alternate flexion condition in the absence of axial force ($N = 0$), for which the residual moment actually generally asummes value which are very close to the maximum value of initial plasticizing.

In this first phase of the study a damping coefficient $\nu = 0$

(*) The numerical calculations were conducted at the SO.GE.NE computer center in Rome, kindly made available to the authors who would like to express their thanks especially to Dr. Bacigalupi for his valuable cooperation.

was assumed.

4. Numerical Results

The numerical results obtained have been directly represented with the aid of a "plotter", this obtaining the graphs that follow. Figures 9, 10, 11, 12, shown as examples, illustrate some typical paths of the oscillator characterized by T_o = 0,4 sec. C_d = 0,5 for λ = 0.30, 0.25, 0.20, 0.15. The great increase of plastic engagement to which the oscillator is subject as λ diminisher appear evident. It can be observed that longer elastic path correspond to the higher plastic threshold values. For this reason a study which might also consider a certain damping in the elastic phase, might point out the differences obtained in plastic engagement even more.

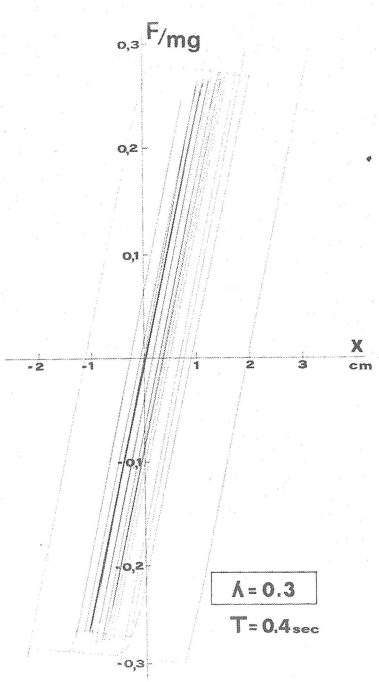

fig. 9

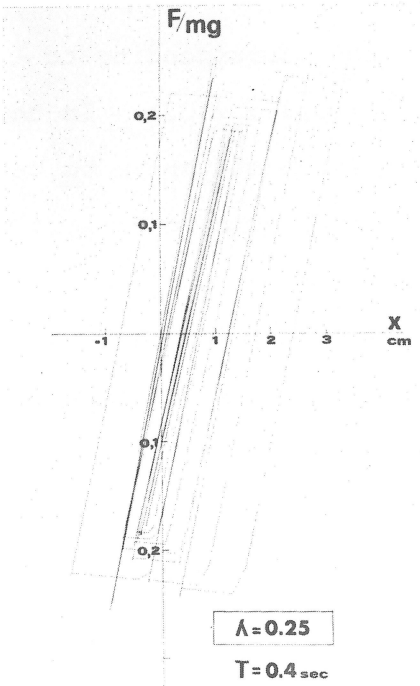

fig. 10

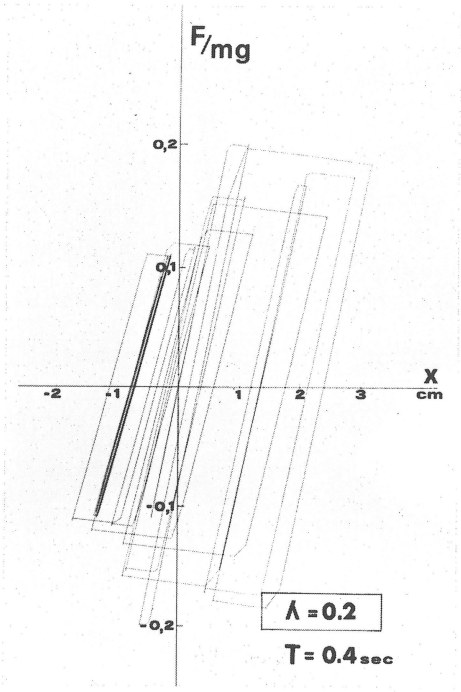

fig. 11

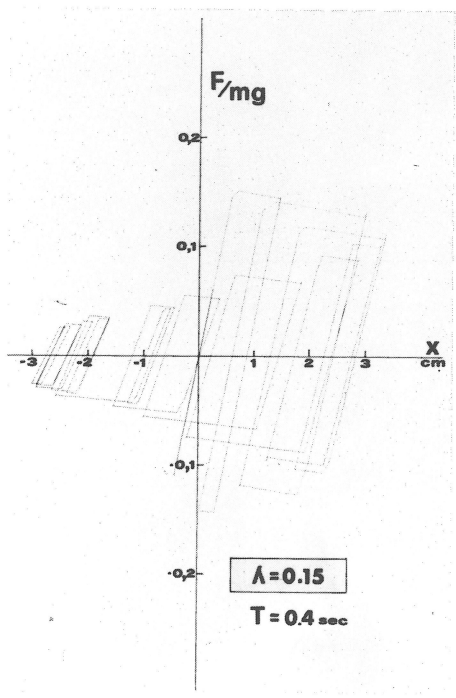

fig. 12

Figures 13, 14, 15 show the spectra of the displacements as a function of their own period T_o, for the three different values of C_d being considered and corresponding to $\lambda = \infty$ (indefinitely elastic behaviour), $\lambda = 0,3$, $\lambda = 0,2$ $\lambda = 0,1$. In the absence of degradation ($C_d = 1$) the maximum displacements prove to be noticeably lessened in changing from an elastic oscillator to elastic plastic ones, especially for own periods of less than 1 sec.

It is however important to notice that for own periods of less than a 0.6 sec., this tendency decreases and finally is decidedly reversed to the reduction of C_d.

Although the results obtained refer only to one accelerogram it is in any case still necessary to examine the degradation effects in order to obtain a correct evaluation of the seismic response of the structures.

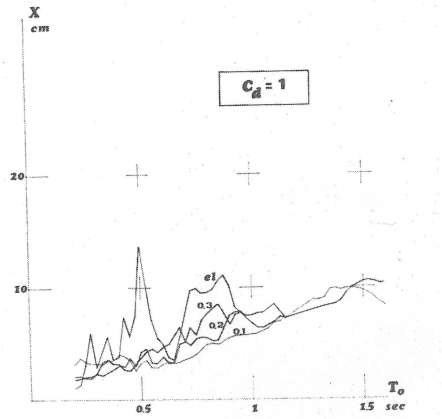

fig. 13

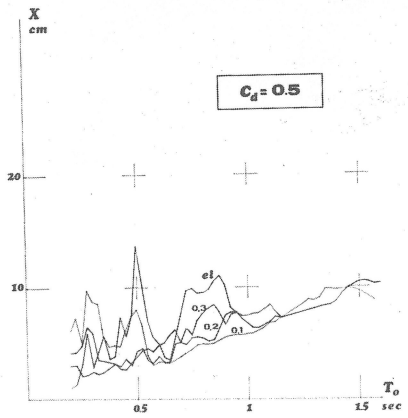

fig. 14

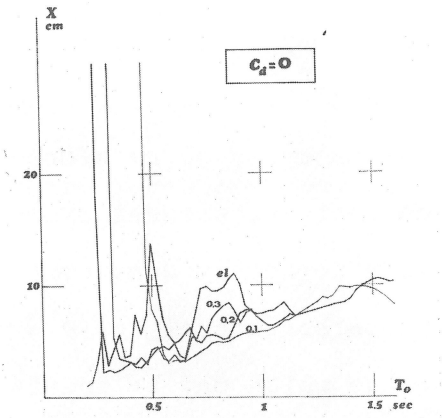

fig. 15

In order to point out the amount of the plastic engagement as the plastic threshold varies λ , the ductility coefficient D was utilized. It is defined as the relationship between the maximum displacement reached and the maximum elastic displacement $\lambda g/\omega^2$. In fig. 16, 17, 18 D is represented as a function of T_o by means of curves corresponding to the six values of λ called (λ = 0,05 ÷ 0,30).

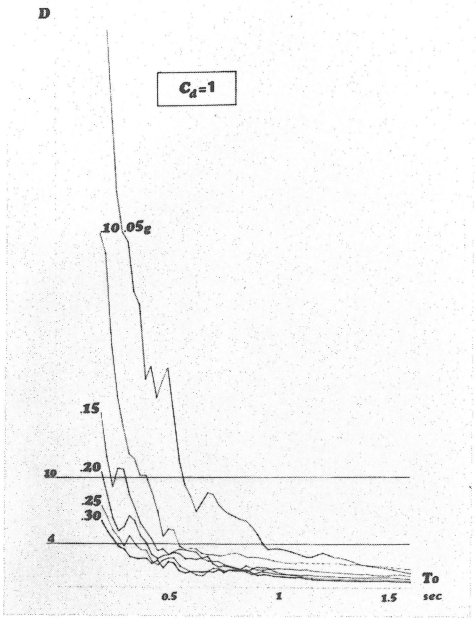

fig. 16

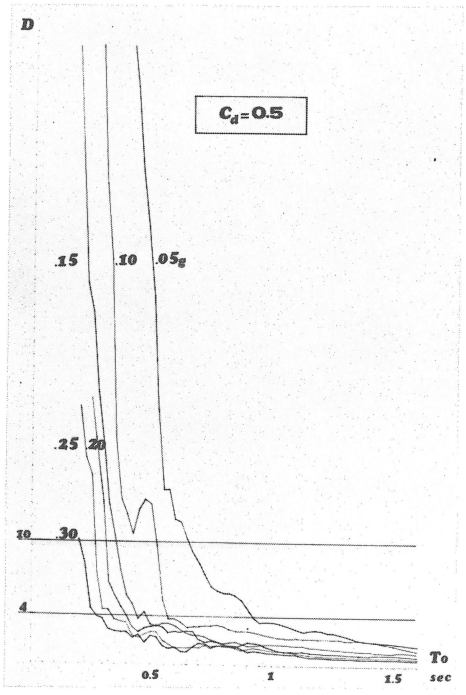

fig. 17

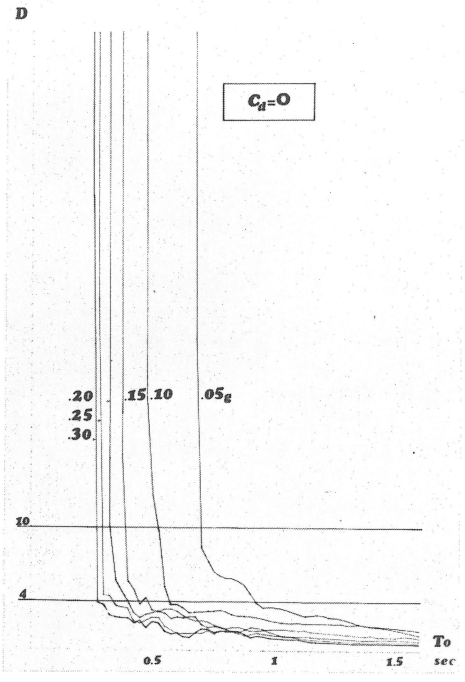

fig. 18

The three figures refer to the three values of C_d which have already been indicated; each figure shows the straight lines horizontal to D = 4 which may be considered to represent a ductility common to the majority of reinforced concrete structures which have been correctly constructed and those horizontal to D = 10, a value which may be considered difficultly reachable by such structures, unless particular

provisions are taken in construction.

The second limit is exceeded for periods which are usually inferior to 0,6 - 0,7 sec., greatly depending on the value of C_d and λ . In particular shifting from fig. 16 to 18, it appears evident that degradation accentuates this phenomenon. In order to note de dependence of D on λ more clearly, figures 19, 20, 21 should be examined. These figures are

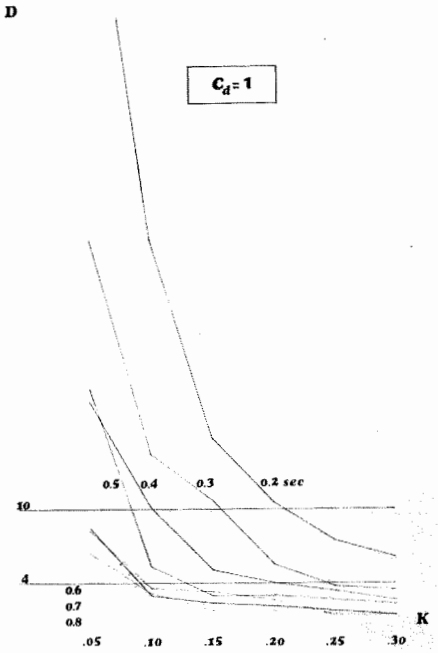

fig. 19

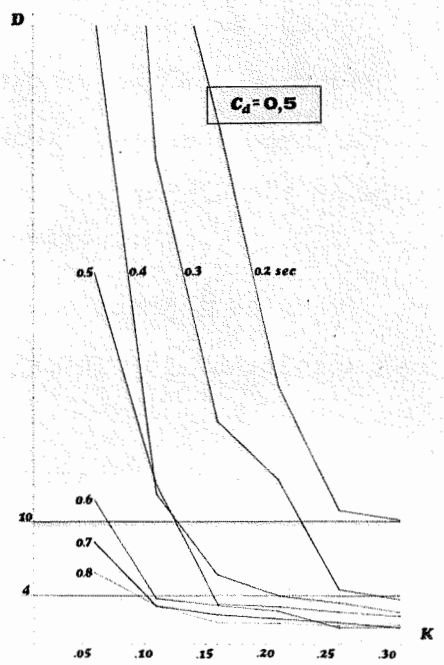

fig. 20

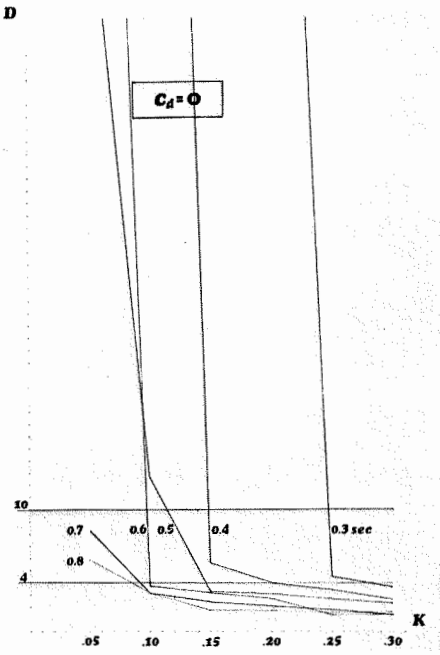

fig. 21

similar to the preceding ones but have λ for abscissa; the various curves correspond to T_o varying from 0,2 to 0,8 sec. for intervals of 0,1 sec.

The most interesting result, at least from the point of view of planning antiseismic structures, is that, even considering only the Tolmezzo accelerogram it appears necessary that these be a value λ exceeding a certain limit in order to maintain D at a lesser value, for example to 10, and that this limit should increase as T_o decreases.

This requirement was particularly made evident by the fact that structural degradation was considered.

5. Conclusions

As has been stated, the study precedent constitutes an initial approach to the problem: in order to generalize the resulting considerations, the investigation must be extended by utilizing numerous other recorded accelerograms so as to be able to disregard the particular "form" and "intensity" of the earthquake. Extending the study will then have to involve considering the importance of the linear damping during the elastic phase which, as has been see, may prove to be a determining factor in correspondance of the higher values of λ.

Finally, it must be observed that the effect of $P-\delta$ not considered in this study, may noticeably influence the situations to which the greater displacements correspond.

The principal result obtained, however, is that of having pointed out the importance of degradation, although the latter was evaluated in a simplified form, by defining the seismic response of a reinforced concrete structure. Obviously, the structures having lower plasti-

cizing thresholds proved to be more stressed; in particular, degradation pointed out the plastic engagement of structures having a smaller own period.

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DYNAMIC INVESTIGATION OF THE EFFECTS OF NON-STRUCTURAL ELEMENTS ON A REINFORCED CONCRETE BUILDING IN FRIULI

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M.P.Petrangeli,Assoc.Professor,University of Rome.

Abstract

A dynamic analysis in the elastic range for a reinforced concrete building is presented. The building was located in the epicentral area of the May 1976 Friuli earthquake.The Authors consider the perimetral brick walls as plane stress finite elements with different elasticity modulus.

The resulting structural response is valued.

Both the modal and the step by step analysis prove that the influence of the walls is in the examined case considerable, also if bad mechanical characteristics of the masonry are assumed.

1) Preliminary remarks

The visits carried out in Friuli immediately after the earthquake of may 6th 1976 showed the great influence of the perimetral brick walls on the behaviour of the r.c. buildings, this influence being in some case of primary importance for the forming of a collapse mechanism. This happened, for instance, when a non symmetrical disposition of the walls in plan caused twisting effects in symmetrical structures.

Taking into account also the fact that the present Italian building Code does not assign a specific resistant function to the brickwork, it seemed useful to get some more knowledge in this field.

The Authors developed therefore the dynamic analysis of one among the most typical surveyed buildings.

The analysis was carried out in the elastic range. The limits of this kind of approach, when the wall-structure interaction has to be taken into account or when the stresses go beyond the elastic level, are well known.

The results however give useful information about the stresses in the various structural elements during low intensity earthquakes, as well as in the initial phase of strong motions, before the first yielding. This information, moreover, found a good corrispondence with the on site observations.

2) Description and schematization of the studied building.

The building was located along the National Road N.13 near Artegna, and, at the time of the first quakes, was nearing the completion of the construction. It was not designed as earthquake resistant and the strong September aftershocks caused later its complete collapse.

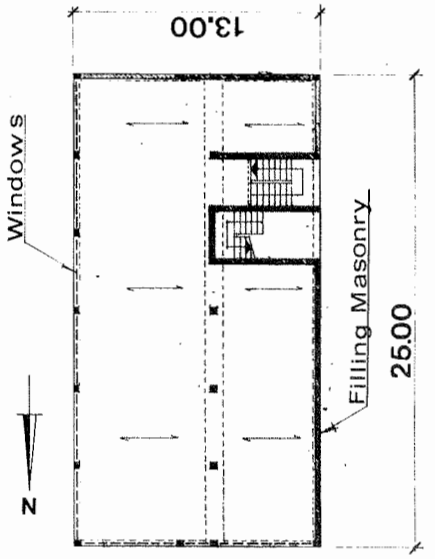


FIG. 1

The building had an underground floor with r.c. perimetral walls and three overground floors, plus a mansard roof. The foundations were continuous along the perimeter, while the central columns had simple footings.

The perimetral hollow brick masonry, everywhere present over the first floor, was lacking at the ground floor along two consecutive sides, which were used as shop windows. The ensuing dissymmetry was magnified by the stair case, which was made with load carryng hollow brick masonry too (see fig. 1).

The building observation after the quakes allowed the following schematizations:

a) The underground floor acted monolitically with the soil. The columns were therefore considered as fixed at the ground level.

b) The South side wall was not considered, since it immediately ruined out of its plane, because of the N-S main direction of the motion.

c) The masses of the mansard roof were concentrated in the upper floor joints. Fig. 2 shows the building after the first violent shocks in May.



3) The analysis

The Authors carried out preliminarily a parametric analysis in order to investigate the influence of the masonry, according to its assigned elastic properties.

To this aim the modal analysis of the building was developed; the structure was then considered as composed by beams, walls and floor slabs; the latter two structural elements were schematized as plane stress finite elements, both isotropic and orthotropic (walls and floor slabs), by means of the SAP IV Program.

The geometry of the structural elements was derived from the original design drowings.

The assumed Young's modulus are:

a) beams and columns $E_b = 300\,000$ kg/cm^2

b) floor slabs: along the floor beams $E_{sl} = 160\,000$ "

b') floor slabs: across the floor beams	E_{s2} =	110 000	kg/cm^2
c) masonry	E_m =	1 000	"
or	E_m =	20 000	"
"	E_m =	60 000	"
"	E_m =	100 000	"
"	E_m =	150 000	"

The value E_m = 1 000 kg/cm^2 corresponds to a not significant masonry influence.

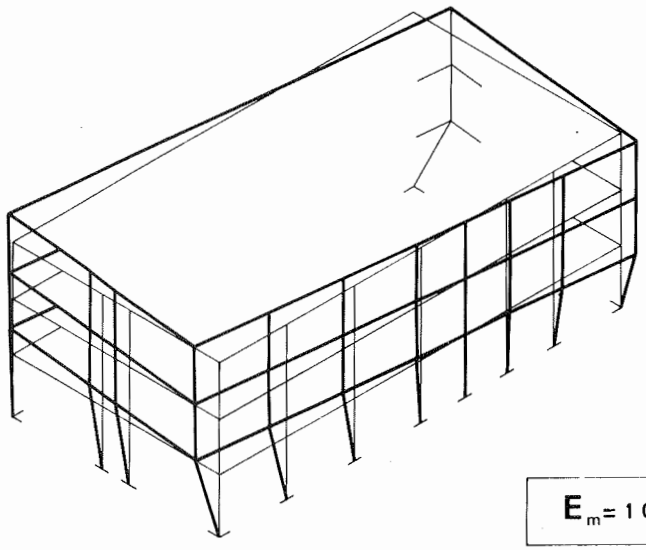
The eigenvalues and the displacements for every value of E_m were computed; fig.3 shows an assonometric view of the second mode for E_m = 100 000 kg/cm^2. From this figure one can see that the upper floors act as a rigid block; the only significant relative displacements are therefore those between the ground, and the first floor. This happened for every value of E_m, but E_m = 1 000 kg/cm^2. That is why the fig.4,5, 6,7,8 represent only the displacement of the first floor slab.

In the aforesaid figures the sign of the vertical displacements for the upper joint of the four corner columns are also indicated, so as to help the understanding of the corresponding mode type.

The results show that for values of E_m immediately over 20 000 kg/cm^2 the first mode is a twisting one, and the corresponding value of the natural period does not vary appreciably with E_m.

The average value of 0,37 sec agrees quite well with the approximate one, which was evaluated by means of the empirical formula suggested by the Italian building Code ($T_o = 0{,}1\ H/\sqrt{B} = 0{,}36$ sec, with H = height of the building and B its minimum dimension in plan).

MODE I° T = 0,375 sec.



$E_m = 100.000$ kg/cm²

MODE II° T = 0,144 sec.

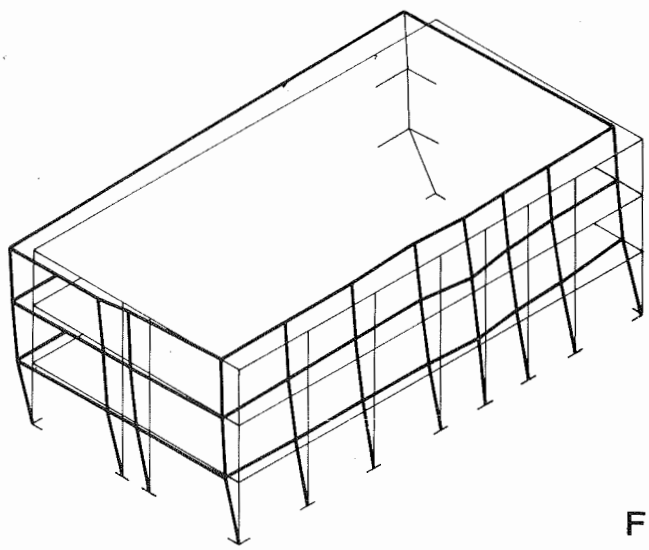


FIG. 3

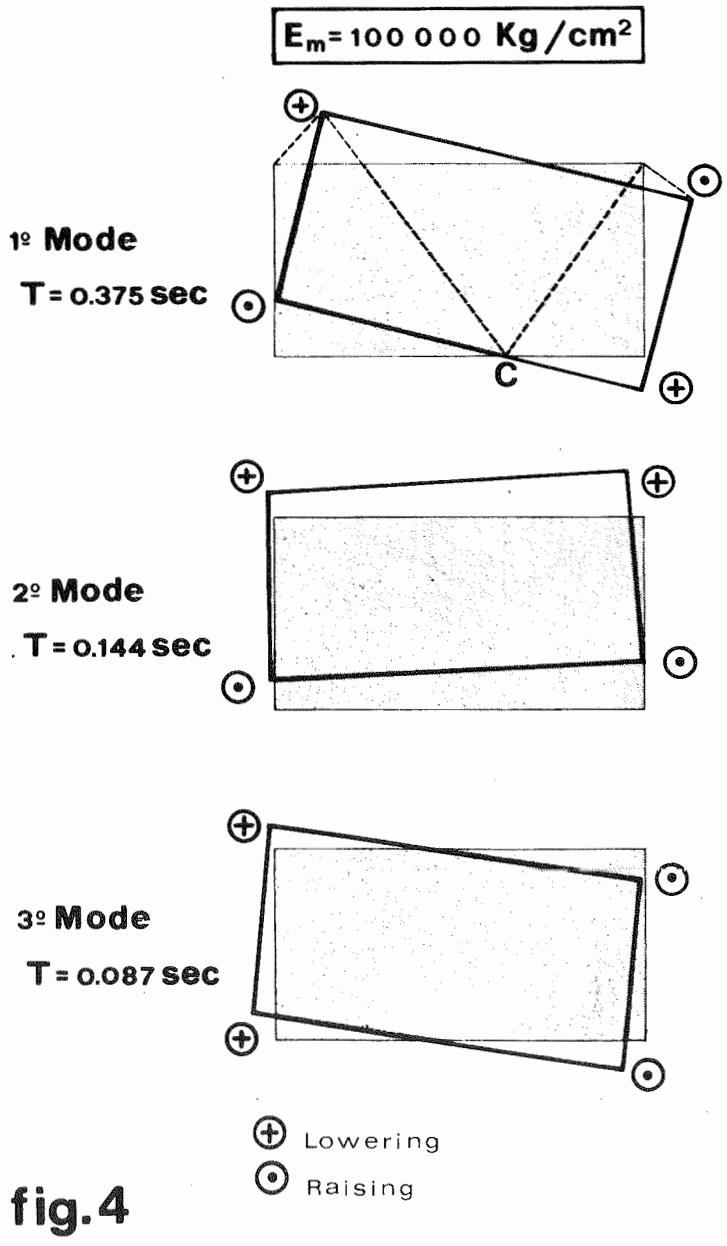

E_m= 100 000 Kg/cm²
1º Mode
T = 0.375 sec
C
2º Mode
T = 0.144 sec
3º Mode
T = 0.087 sec
Lowering
Raising

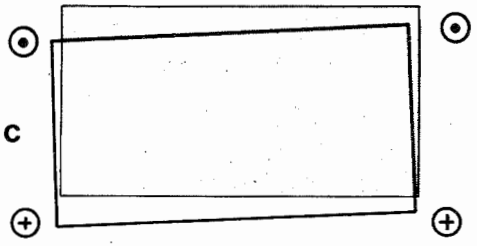
fig. 4

fig. 5

$E_m = 1000$ Kg / cm²

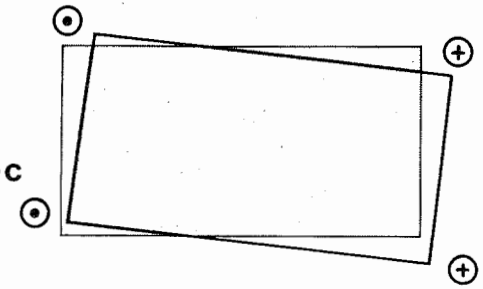
1º Mode

T = 0.601 sec



2º Mode

T = 0.587 sec



3º Mode

T = 0.485 sec

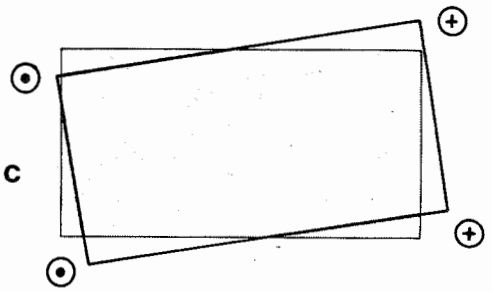
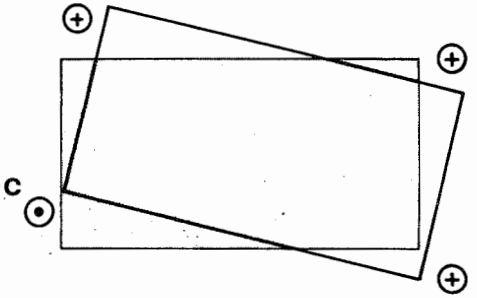


fig. 6

$E_m = 20000$ Kg / cm²

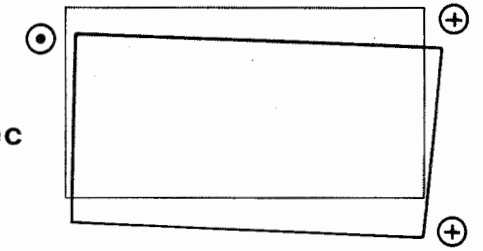
1º Mode

T = 0.438 sec



2º Mode

T = 0.257 sec



3º Mode

T = 0.168 sec

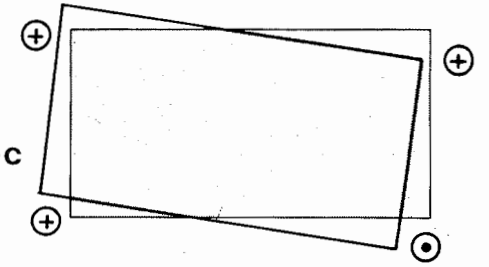


fig.7

$E_m = 60'000\ Kg/cm^2$

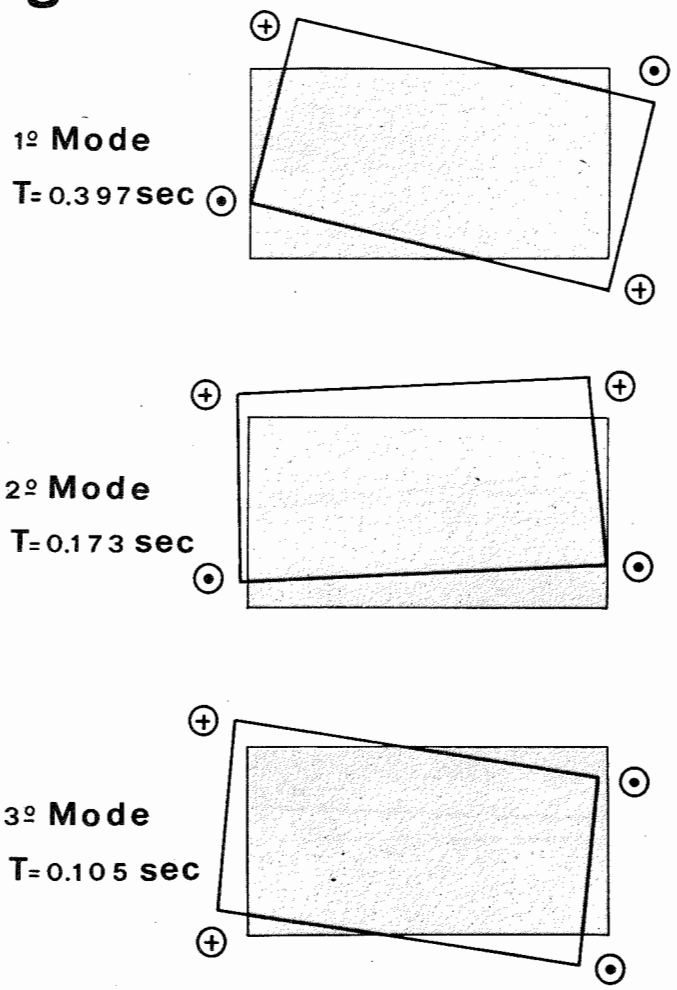
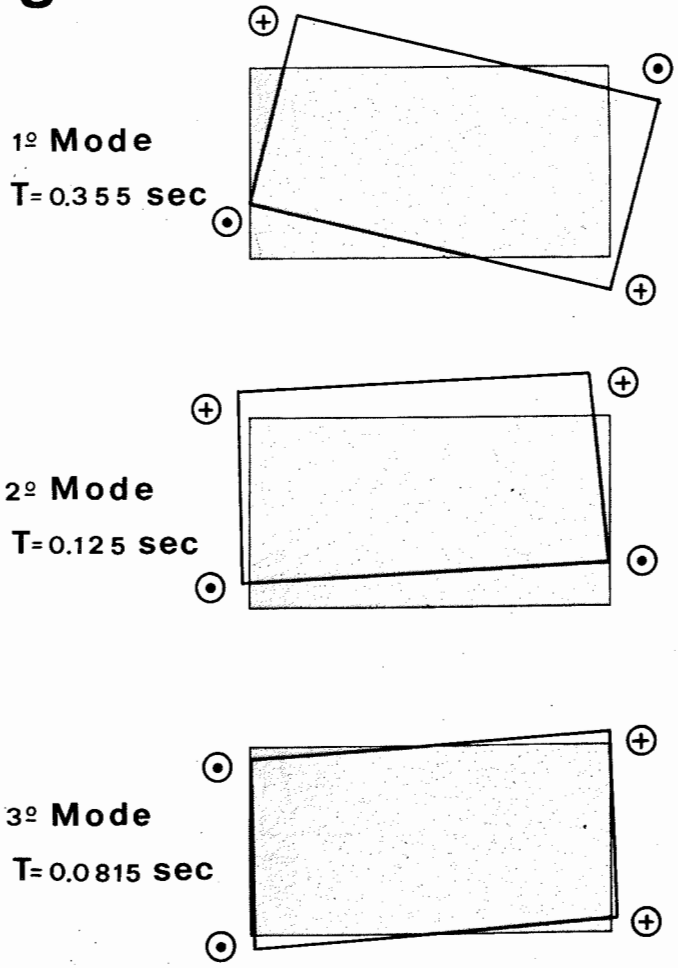


fig.8

$E_m = 150'000\ Kg/cm^2$



E_m [Kg/cm²]	1º MODE		2ºMODE		3º MODE	
1'000	0.601	T_x	0.587	RT_y	0.485	RT_y
20'000	0.438	R	0.257	T_x	0.168	RT_y
60'000	0.397	R	0.173	T_x	0.105	RT_y
100'000	0.375	R	0.144	T_x	0.087	RT_y
150'000	0.355	R	0.125	T_x	0.081	RT_y

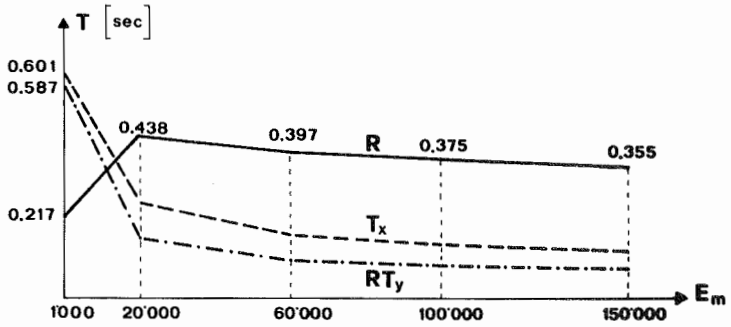


fig.9

After the little influence of the walls elasticy modulus has been pointed out, the analysis with $E_m = 1\ 000$ kg/cm^2 and $E_m = 100\ 000$ kg/cm^2(*) has been examined in detail by means of two types of computation.

Firstly the stress components (as square root of the sum of the squares of the modal stresses) have been computed for the conventional response spectrum of the italian building code ($C_o = 0.1$ for $To \leq 0.8$sec $C = 0.0862/To^{2/3}$ for $To > 0.8$ sec).

The earthquake has been assumed to act in the N-S direction, according to the observations on site.

(*) This is the most likely value of E_m for the bricks currently used in Friuli. These bricks were lightened with holes for about 40 % and showed a compression strength(computed on the gross area) of 120/150 kg/cm^2. They were placed in situ with a good quality mortar too.

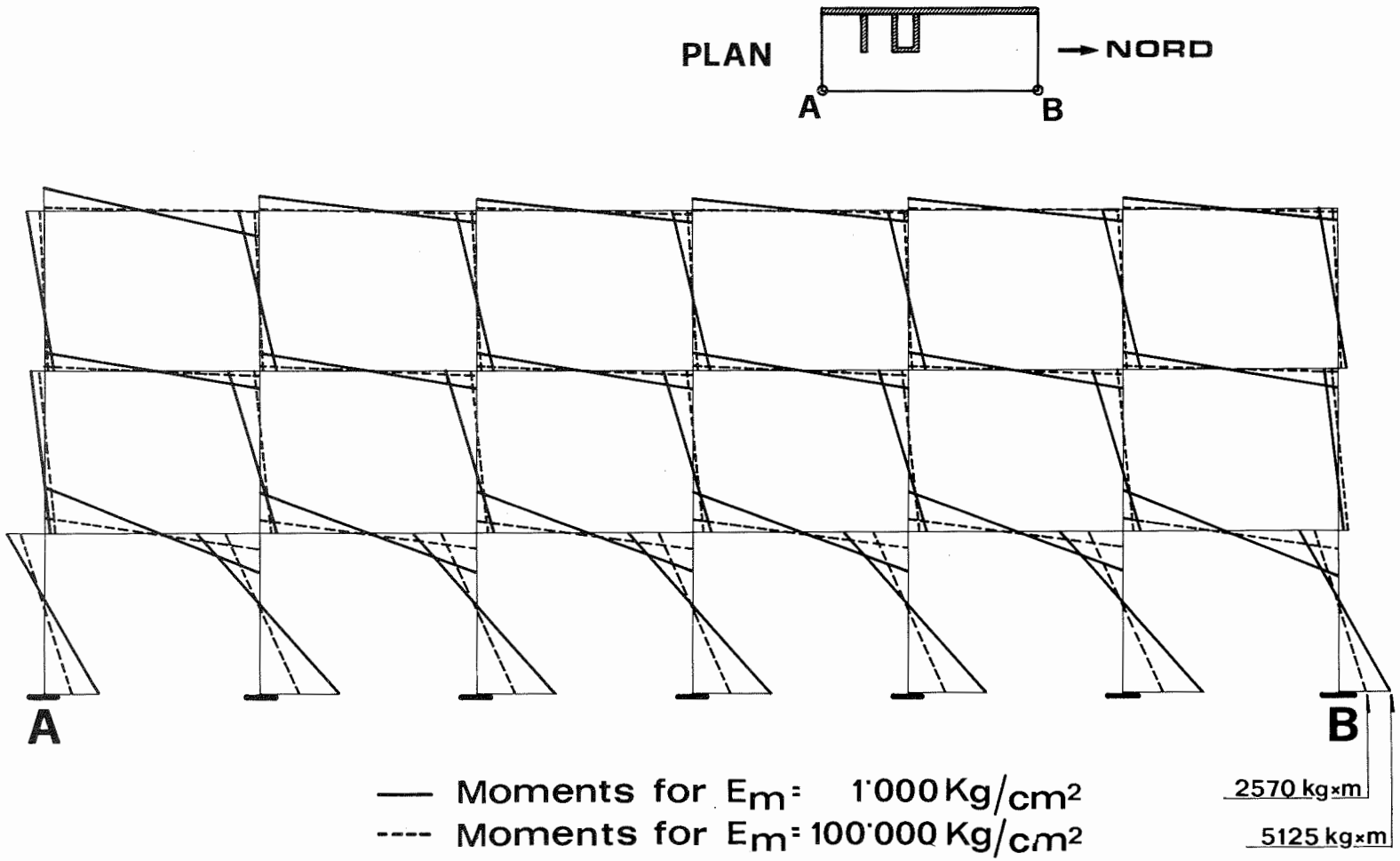

PLAN
NORD
A
B
A
B
Moments for E_m = 1'000 Kg/cm²
Moments for E_m = 100'000 Kg/cm²
2570 kg×m
5125 kg×m

Fig. 10

Fig. 10 shows the moments' diagrams in the East side frame for the two values considered for E_m. This frame was parallel to the assumed earthquake direction and opposite to the entirely walled façade.

The remarkable increase of the stresses if walls are ignored, is quite evident, this increase being up to 100 %.

If the dimensions and the actual reinforcement are considered, the collapse load for the most severly loaded column can be evaluated.

Assuming a cylindrical concrete strength equal to 200 kg/cm^2 and with the hipotesis indicated in the ACI (American Concrete Institute)Pubblication SP 7 the ultimate moment should have been approximatly equal to 8 000 kg.m. That means a safety factor with regard to the Italian Code Spectrum of 1.56 if brick walls are ignored or 3.11 if they are considered with E_m = 100 000 kg/cm^2.

Since the maximum soil acceleration has been certainly greater than that foreseen by the Italian Code, and since the studied building did not completly ruine, it is evident that the behaviour was that corresponding to the presence of walls.

Finally a time history for E_m = 100 000 kg/cm^2 was carried out with a damping factor of 0.05.

The assumed ground motion was that recorded in Majano during the aftershocks of May 10th 1976 (fig. 11).

Fig. 13 shows the displacements history of the upper joint for the B column. The oscillation period in the 2-4 sec interval (maximum response) is 0,38 i. e. very close to the calculated first natural period.

The moments history at the base of the same column is also illustrated in fig. 12.

The maximum moment is equal to 1615 kg.m.

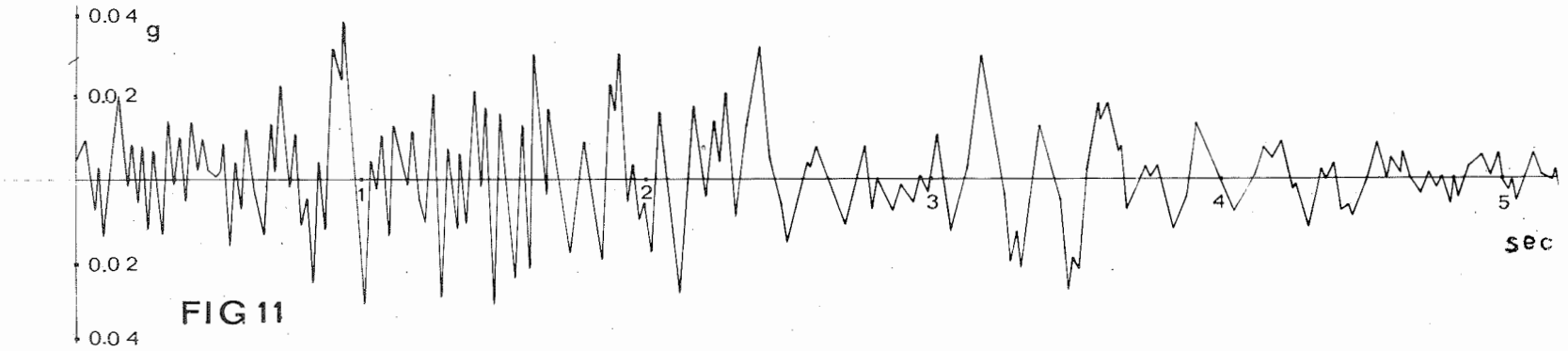


FIG 11

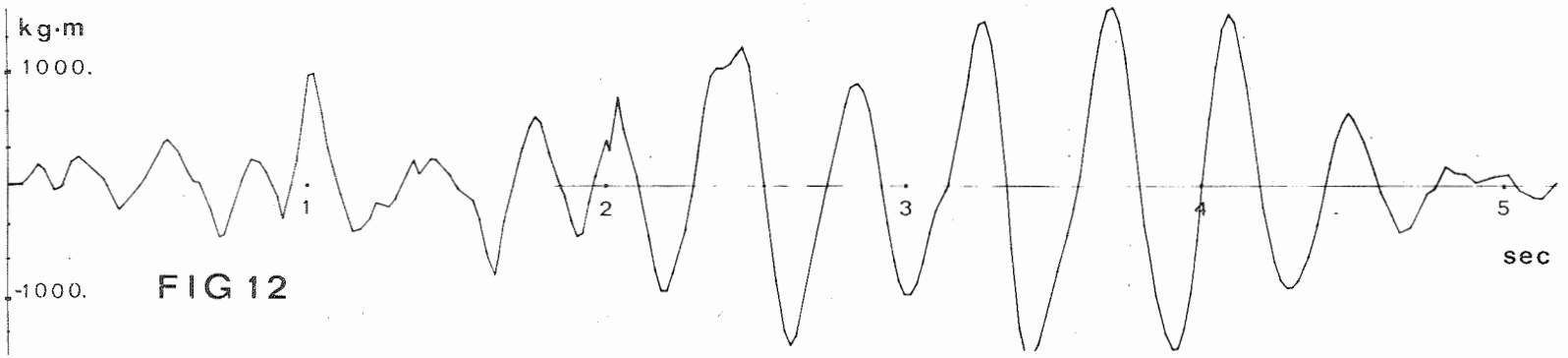


FIG 12

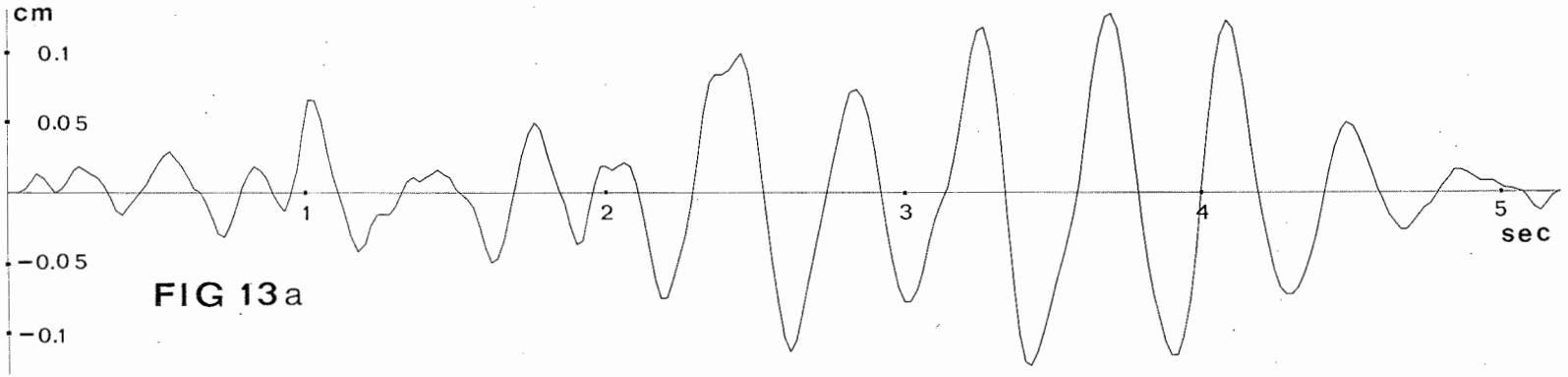


FIG 13a

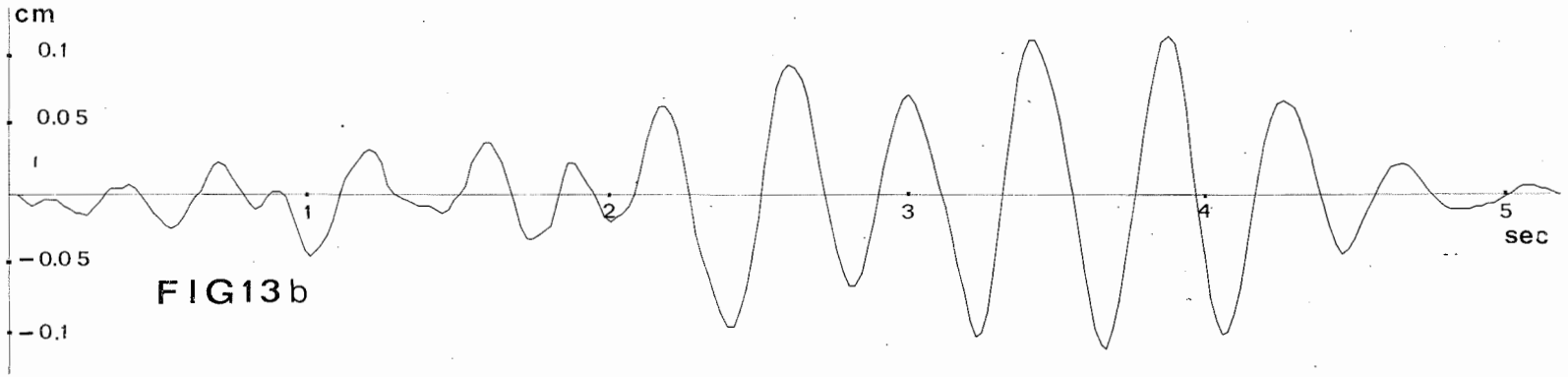


FIG13b

To obtain the maximum value (in the same point) as that evaluated by modal analysis, with the code spectrum, the soil accelerogram should be amplified with its maximum value up to 0,06 g.

Conclusion

The obtained numerical results confirm the great importance of the hollow brick walls in the behaviour of the investigated building.
This non structural elements (also if considered of bad quality) changed the way of vibrating and therefore the mechanism of collapse.
The computed stresses resulted always smaller than those corresponding to the total absence of walls at ground floor. We can therefore suppose that in the last case the building should have collapsed during the first shocks.
Viceversa a regular distribution of perimetral walls should have probably avoided the structure to suffer important damages and, at least,to collapse during the last aftershocks.

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DAMAGE TO BRIDGES AND ROADS IN THE 1976 FRIULI (ITALY) EARTHQUAKE

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1. Introduction

The area of Friuli most severely hit by the earthquake (1) is crossed by numerous roads, with many large bridges, important both as regards size and static behaviour. Moreover, a highway is being constructed in the area, crossing the epicentre with bridges which, at the moment of the earthquake, were in various phases of construction. It should also be noted that, although the seismicity of the area is quite well known, only one part of the territory was normally considered seismic. The structures illustrated here are divided according to construction techniques, and the type of material used in the following categories: stone bridges, bridges made of steel, or of both steel and concrete, bridges in reinforced prestressed concrete, and highway bridges.

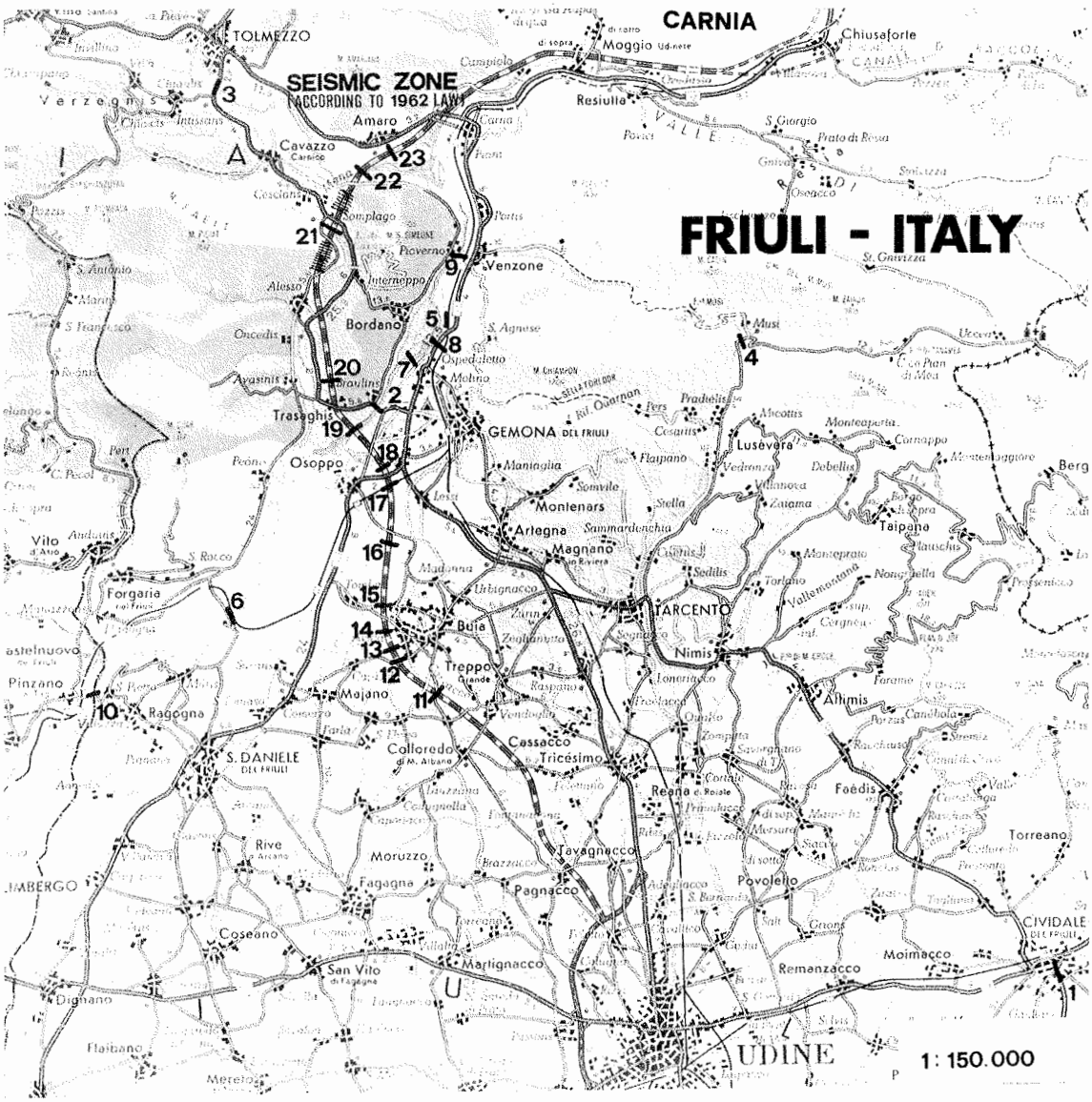


Fig. 1

1 - Cividale Bridge; 2 - Braulins Bridge; 3 - Tolmezzo Bridge; 4 - Vedronanza; 5 - Rivoli Bianchi railway bridge; 6 - Cornino Bridge; 7 - Pipeline; 8 - Rivoli Bianchi Bridge; 9 - Pioverno-Venzone; 10 - Pinzano Bridge; 11 - Osovanna I; 12 - Juliense; 13 - Roggia Alta; 14 - Fiume Ledra; 15 - Osovanna II; 16 - Rio Gelato; 17 - Gemona-Casarsa; 18 - Ledra-Tagliamento; 19 - Tagliamento I; 20 - ENEL; 21 - Somplago; 22 - Tagliamento II; 23 - Rio Maggiore.

Lastly, we mention damage suffered by roads, taking into account that part of the area in question is mountainous, with some tracts of road involving considerable engineering commitment.

2. Stone bridges

These are generally arched bridges of squared-off blocks of stone, all made before 1950. Among these the most important historical one is the CIVIDALE bridge (2, 3), called "Il Diavolo" ('The Devil') dating back to 1452. Over the centuries, it has been restored several times. Finally partially demolished by the retrating Italian troups during the First World War, it was then rebuilt by the Austrian troups in 1918. It is 20.60 m high, with two asymmetrical spans, measuring 24.50 m and 18.40 m each. It underwent only slight damage to the parapets and side walls of the main structure.

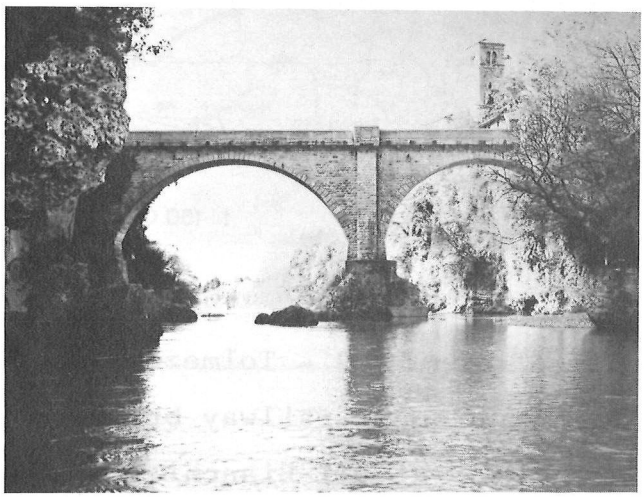


Photo 2: Bridge called "Al Diavolo" at Cividale. General view.

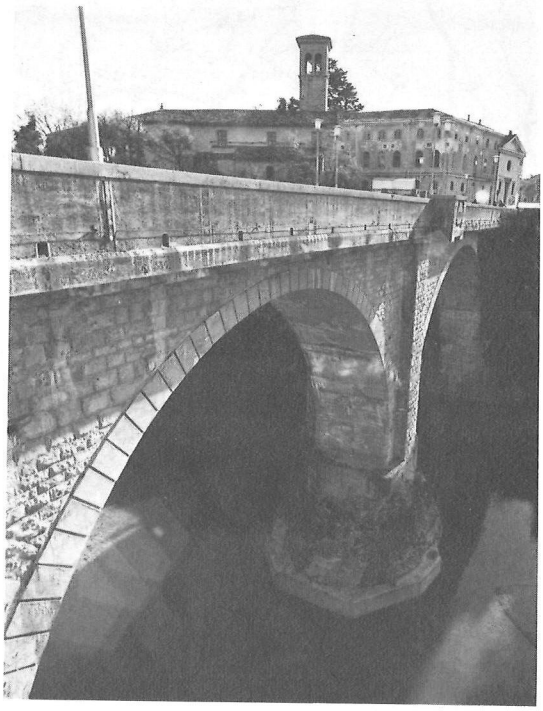


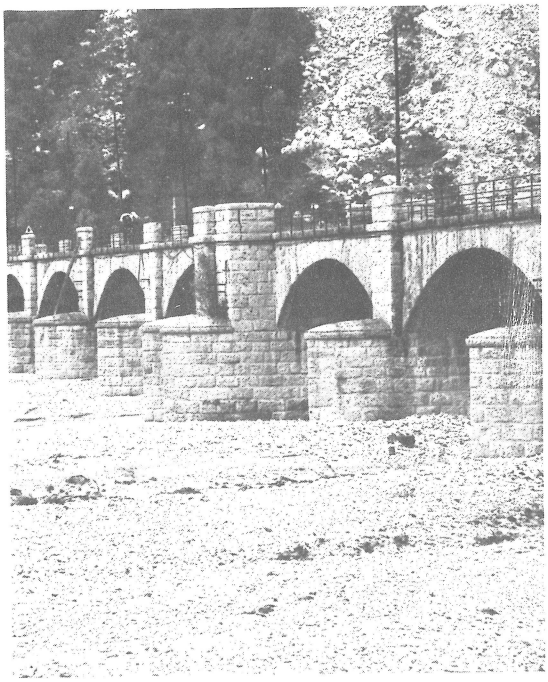
Photo 3: Side view.

We should also mention here the bridge over the Tagliamento River at Braulins (4) which underwent damage to its parapets and transversal lesions of the road surface at each pier (5, 6). We also mention the bridge over the Tagliamento at Tolmezzo (7) which showed slight displacement. Of less static importance, but much more seriously damaged (September 15 1976) was the bridge over the Torre River at VEDRONANZA (8) on the Tarcento-Uccea road, whose parapets are tilted and cracked, and whose containing wall of the left abutment has collapsed. Another important bridge crosses the railway on the left bank of the Tagliamento at Rivoli Bianchi (9, 10), near Venzone. This appears to be in a good state of preservation.

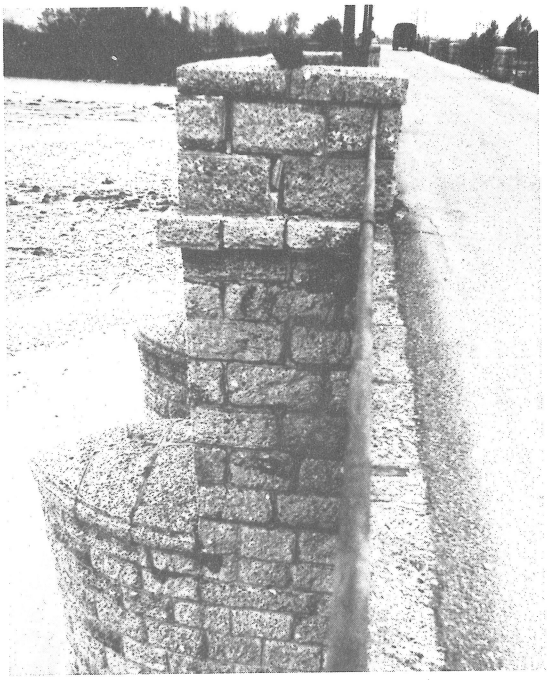
It should be noted how all the lesions on the external stone facings are difficult to notice if not evident. The gravity functioning of the structures themselves tends to bring the structures back to the situation before the earthquake. Notwithstanding these survey difficulties, the behaviour of the structures was good, since they were evidently able to tolerate large displacements and accelerations (for example, the kilometre-long railways bridge under Mt. S. Simeone, at the epicentre of the May earthquake.



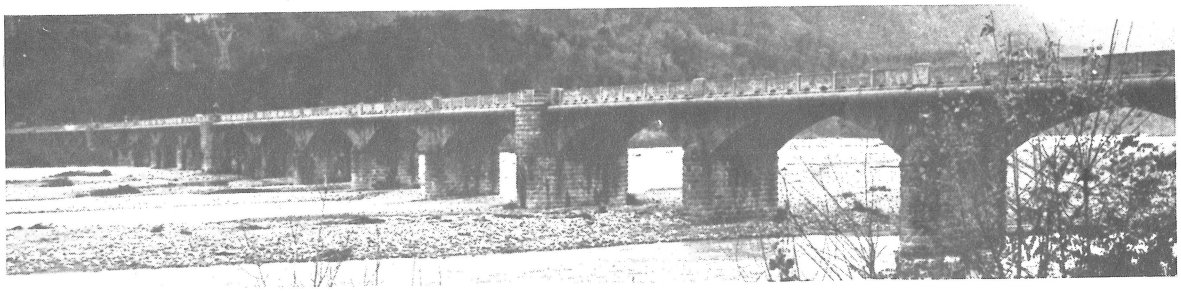
4: Bridge at Braulins. To the left, the landslip which overcame the village.



5: Bridge at Braulins. Piles.



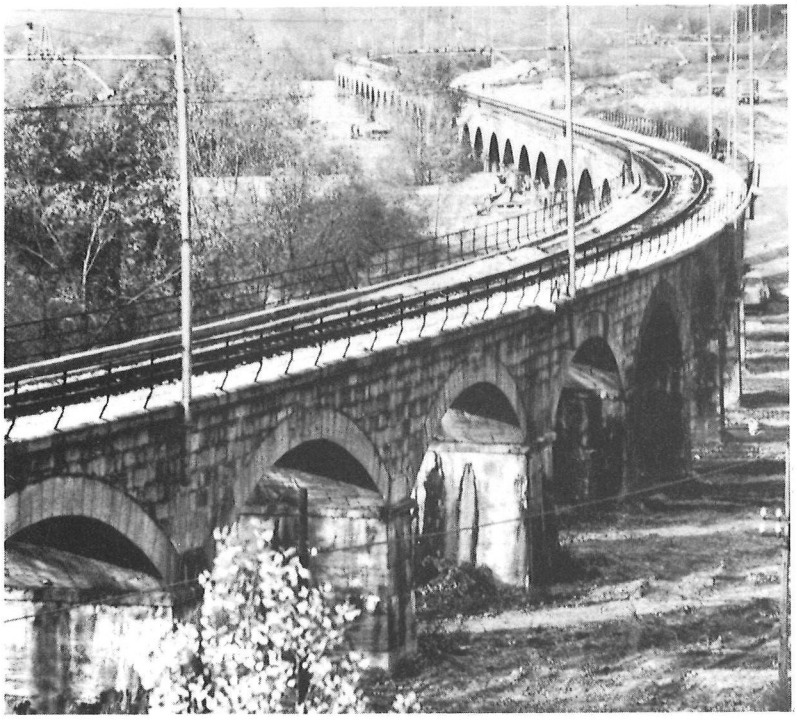
6: Details of piles.



7: Bridge over the Tagliamento River at Tolmezzo.



8: Bridge over the Torre River at Vedronanza.



9: Railway bridge at Rivoli Bianchi.



10: Railway bridge at Rivoli Bianchi: detail.

3. Bridges made of steel and steel-concrete composite structure

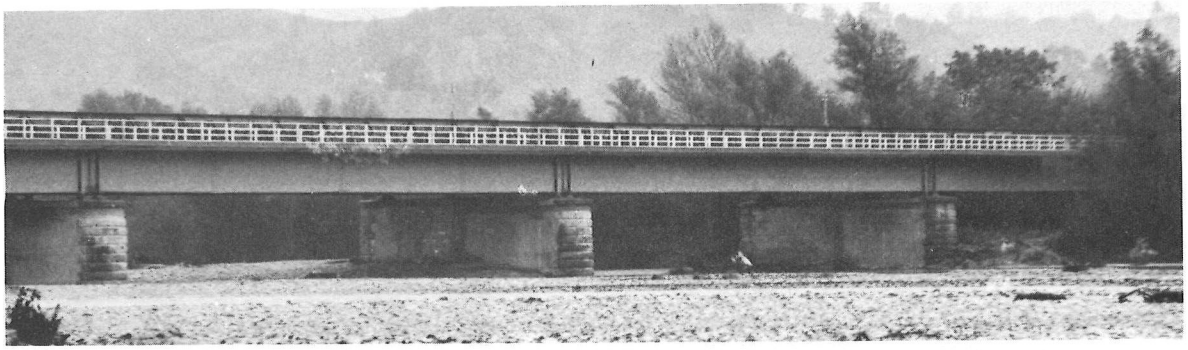
We must immediately quote here the bridge over the Tagliamento at Cimano, carrying both the road and the railway. It is an old railway bridge composed of piers at 44.5 m from each other, recently widened to hold a road as well. The railway part is composed of a steel reticular structure (11), with U-shaped transversal sections, continuous along all piers, with metal supports fixed on a central pier and running longitudinally on the other piers. The railway structure of the bridge behaved well under stress; the joints are riveted, and showed great ductility; the mobile bearings behaved well too, although the fixed bearing requires maintenance work at its base-plate. The road deck (12), 8.40 m wide and 2.65 m high including reinforced-concrete slab, behaved well except for the bearings, both fixed and mobile. As regards the fixed bearings, the anchoring reinforcement of the lower plate made of 4 Ø 22 was sheared (with longitudinal and transversal displacement of about 6 cm.

The mobile bearings were sometimes broken owing to the detachment or folding of the transversal block of the mobile roller, and sometimes dovetailing against the upper and lower plates of the transversal block which, going beyond the maxi-permissible displacement, transformed the mobile bearing into a fixed one. In this case, damage to the opposite fixed bearing was greater (13-20).

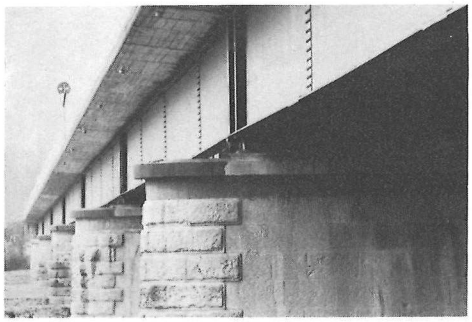
We also add that near Gemona the Tagliamento is crossed by the Italy-Central Europe methane-gas pipeline (21). This was unhooked from its supporting rollers (22) mounted on iso-



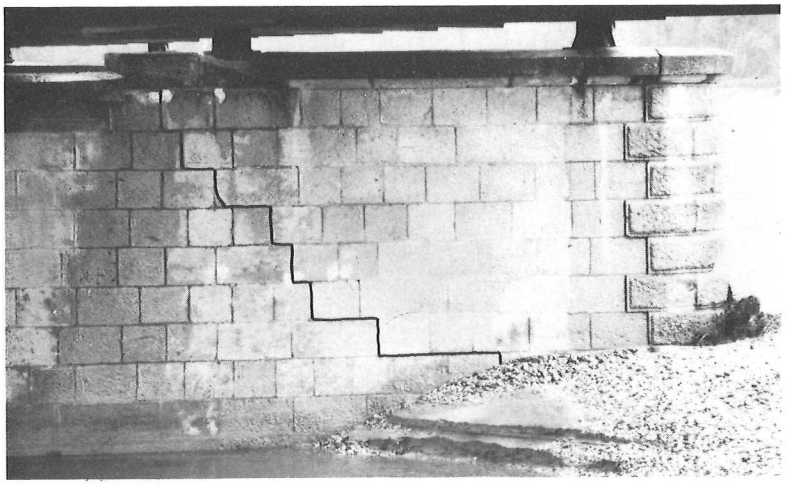
11: Railway bridge at Cimano-Cornino.



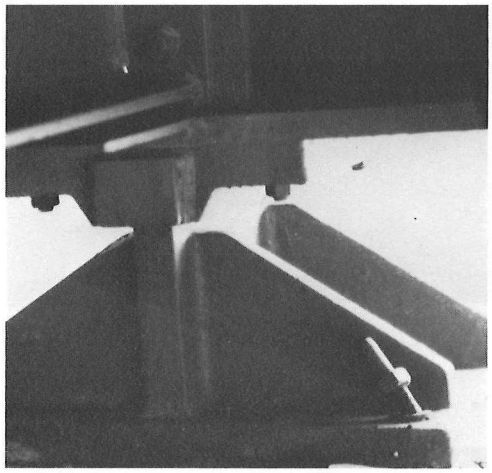
12: Road bridge at Cimano-Cornino.



14: Cimano-Cornino bridge: detail.



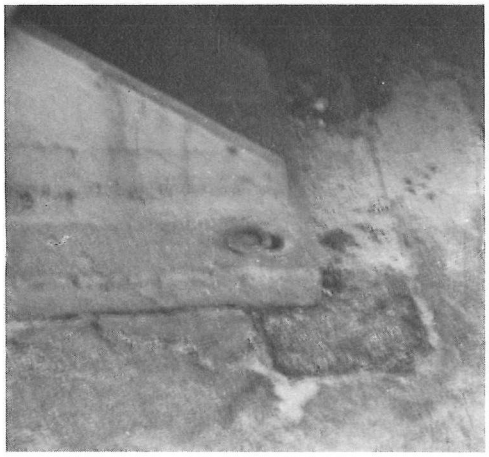
13: Cimano-Cornino bridge: lesions to the piles.



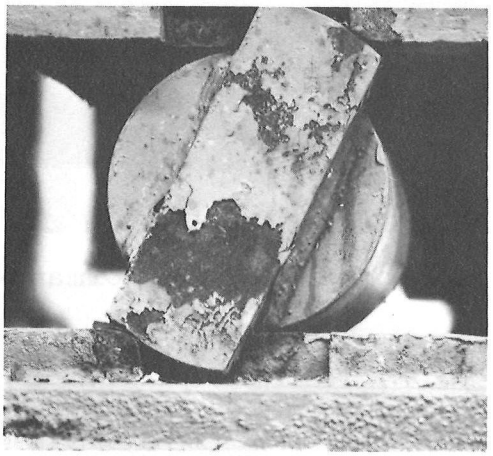
15: Fixed bearing.



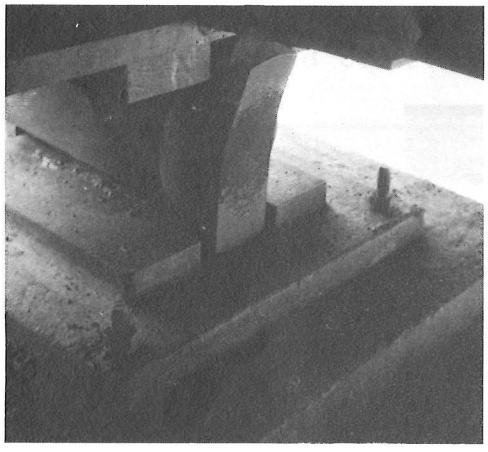
16: Fixed bearing.



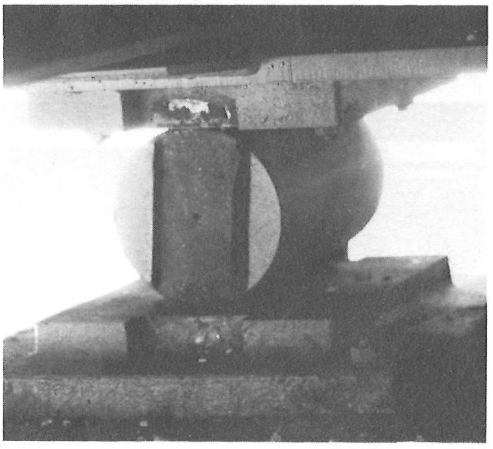
17: Fixed bearing: displacement in two directions.



18: Mobile bearing, blocked.



19: Damaged mobile bearing.

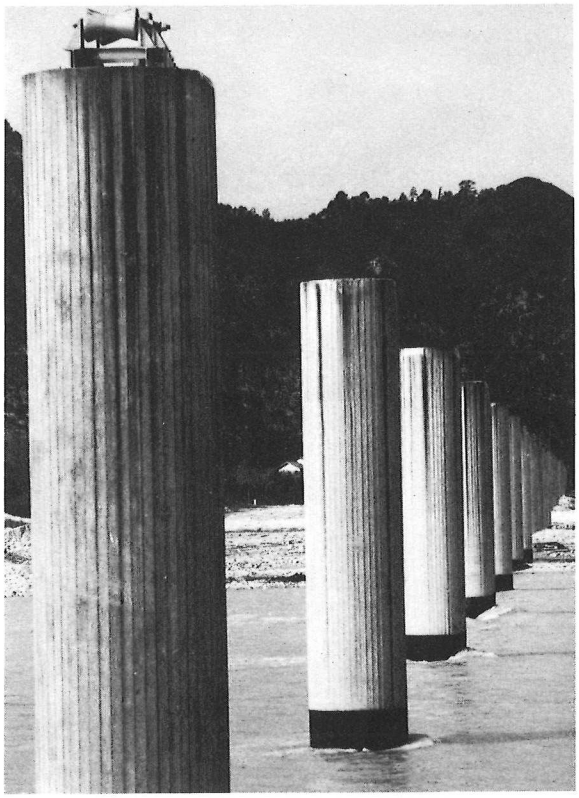


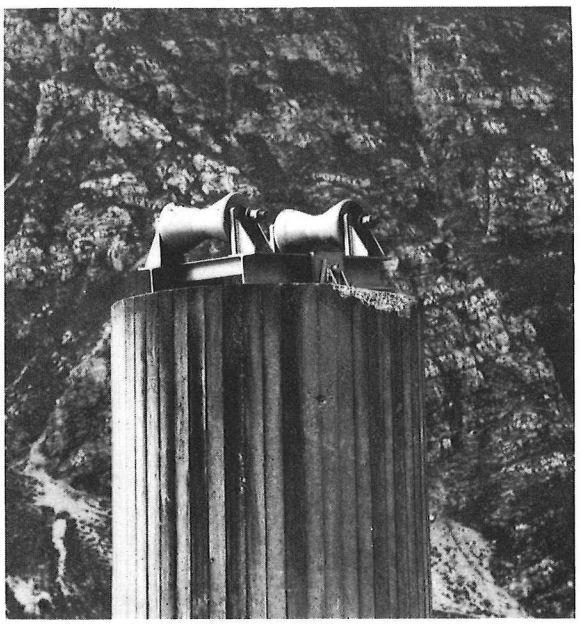
20: Damaged mobile bearing.

lated piers, which were not able to prevent side displacements.

The metal structures behaved quite well, with the exception of the bearings and linking equipment, which did not appear sufficient to stand up to the kind of stresses encountered in earthquakes. Particular attention should be given to a study of the anchoring of the metal bearing plates to the reinforced concrete surfaces.

21: Pipeline. The pipeline has fallen into the river.





22: Detail.

4. Bridges in reinforced concrete

These bridges have girders simply supported on their piers. The road bridge at Rivoli Bianchi (23) between Gemona and Venzone, made of girders simply supported on lead plates, behaved well. The foot-bridge over the Tagliamento between Venzone and Pioverno (24) at the foot of Mt. S. Simeone, made of two prefabbricated beams linked by the slab and simply supported on neoprene plates, also behaved quite well. In fact, we noted only one instance of loss of alignment, indicating side displacements of 10 cm maximum.

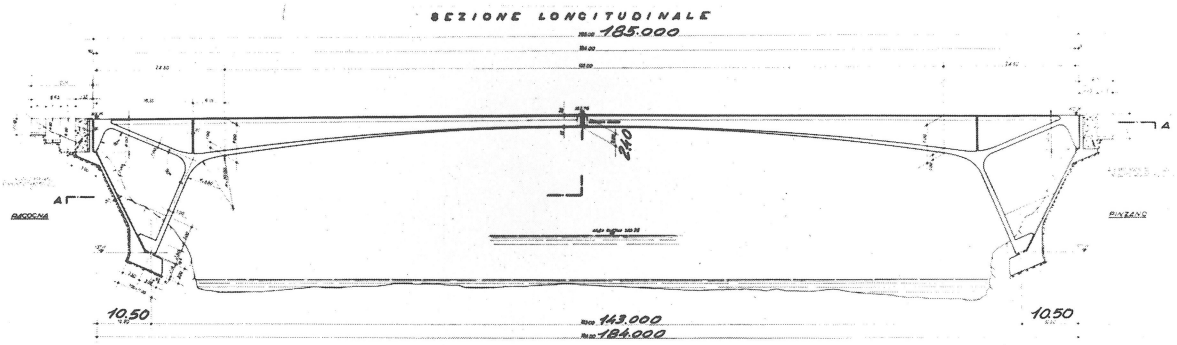


23: Road bridge at Rivoli Bianchi.

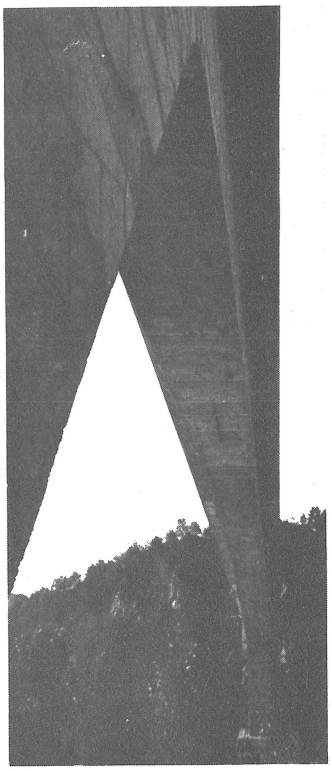


24: Footbridge. In the background: Mt. S. Simeone.

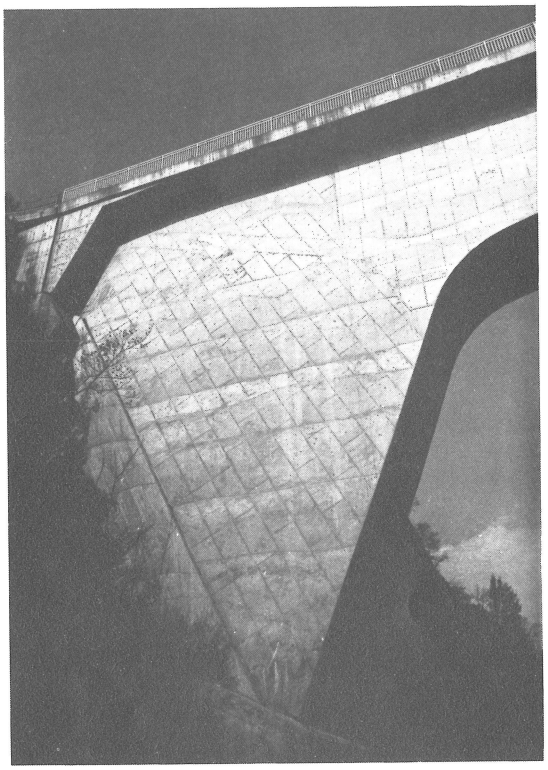
Of particular interest, both for construction problems and static studies, is the Pinzano bridge over the Tagliamento (25-29). This is an arch bridge, with three cylindrical hinges with box-sections of variable depth, made using the cantilever method of construction with the aid of steel provisional anchorings in the rock which blocked the cantilever responding to the deck about 180 m long. The cylindrical hinges are made of metal reinforcements suitably crossed and a corresponding great reduction of the reinforced concrete section. The bridge shows lesions corresponding to the base cylindrical hinges and on both abutments, particularly the right one (30-35). These lesions are composed of fissures running horizontal/inclined on the side facings, and vertically on the counter-slab. They run to a height of about 2.50 m. They occurred on May 6 and spread on September 15. Their maximum amplitude is about 2 mm. The road surface was reinflated (locally) corresponding to the end joints.



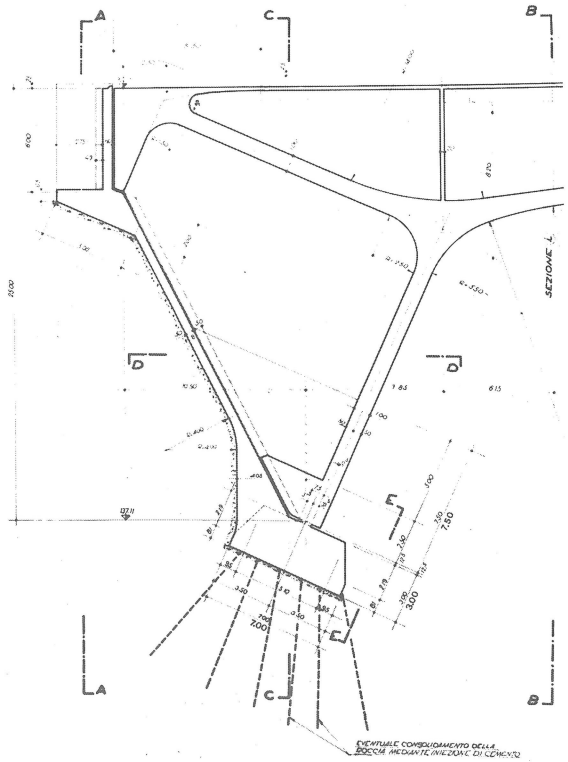
25: Pinzano Bridge: section.



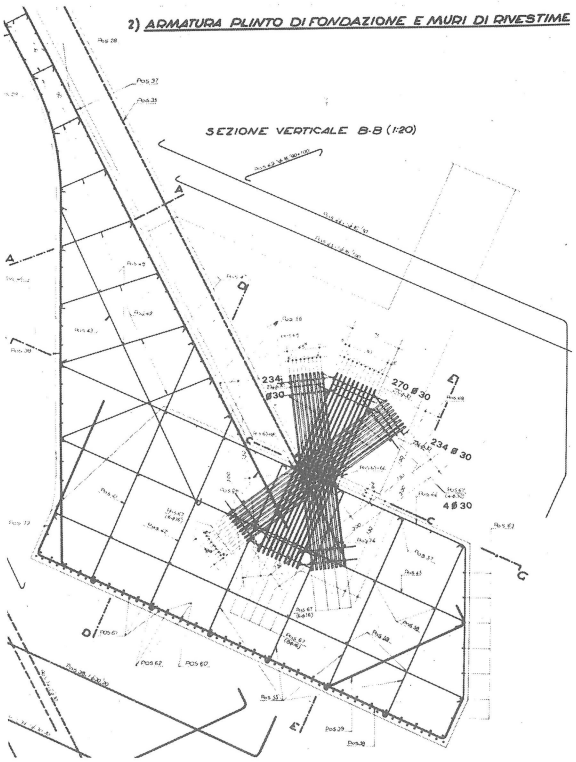
26: Pinzano Bridge.



27: Right abutment.



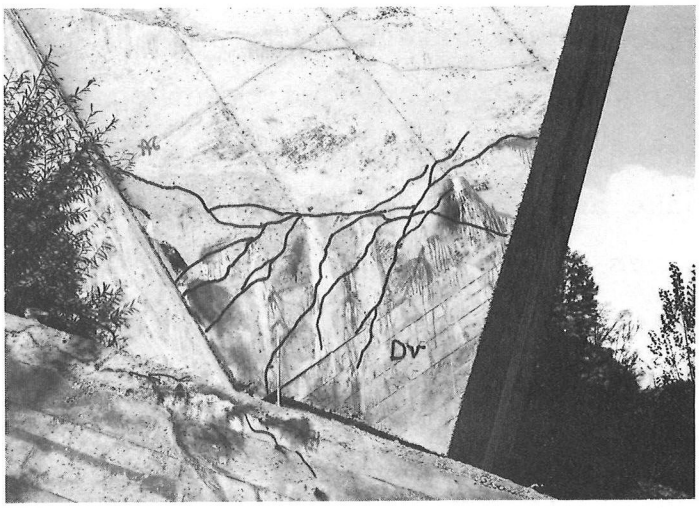
28: Abutment detail.



29: Hinge detail.



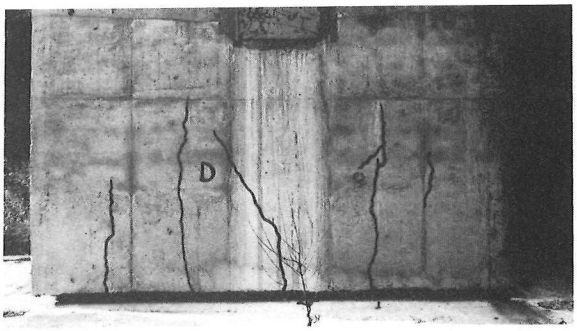
30: Right abutment; upstream side.



31: Right abutment; downstream side.



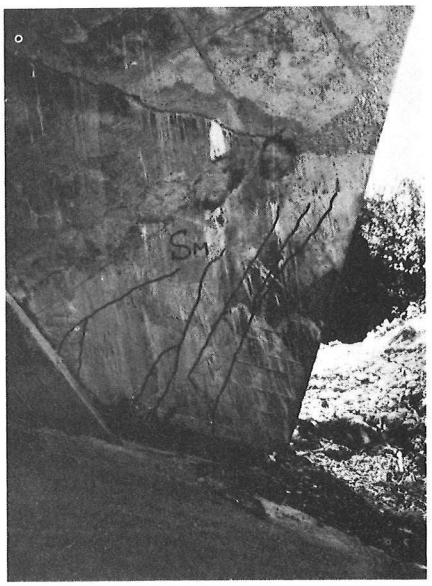
32: Left abutment; counter-deck.



33: Right abutment; counter-deck.



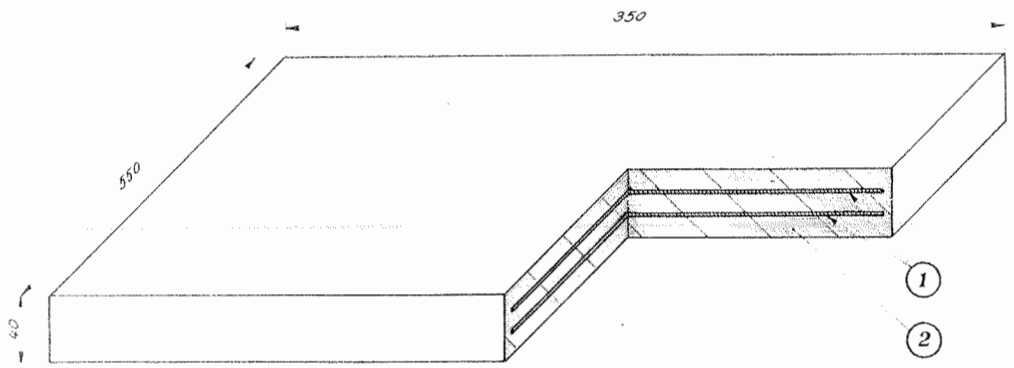
34: Left abutment; downstream side.



35: Left abutment; upstream side.

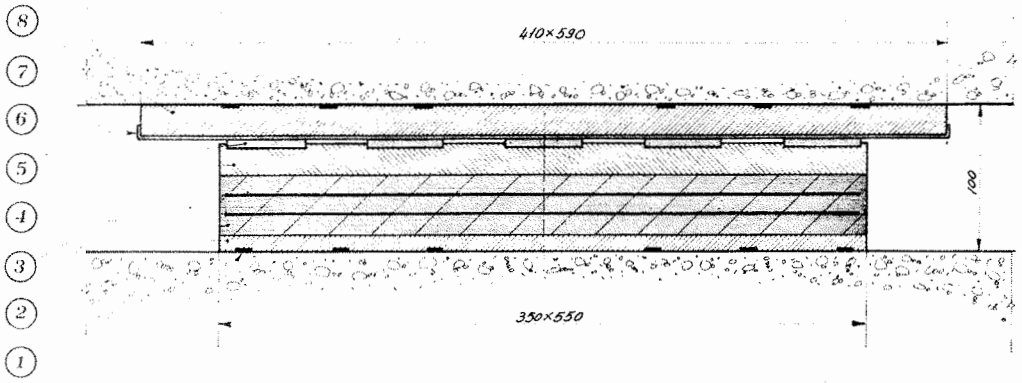
5. Highway bridges

These are included together with highway works since the highway was under construction at the time of the first earthquake and information on homogeneous structures in various phases of construction is therefore available. It is also possible to obtain detailed documentation on the characteristics of materials and foundation soils. Moreover, design and construction methods are technologically very advanced. The bridges will be listed as they are found along the highway running from Udine towards the Carnia area. In all the following works, commercial support devices were used, referred to as follows:
Type A: Steel-reinforced rubber bearing consisting of two or more rubber slabs bonded to metal plates so as to form a sandwich arrangement. It has a certain vertical and also horizontal stiffness, the latter being such that the bearing is considered fixed for moving loads and mobile as regards creep, shrinkage, and thermic variations.



Type A: fixed bearing:
1 Metal plates
2 Rubber slabs bonded to metal plates

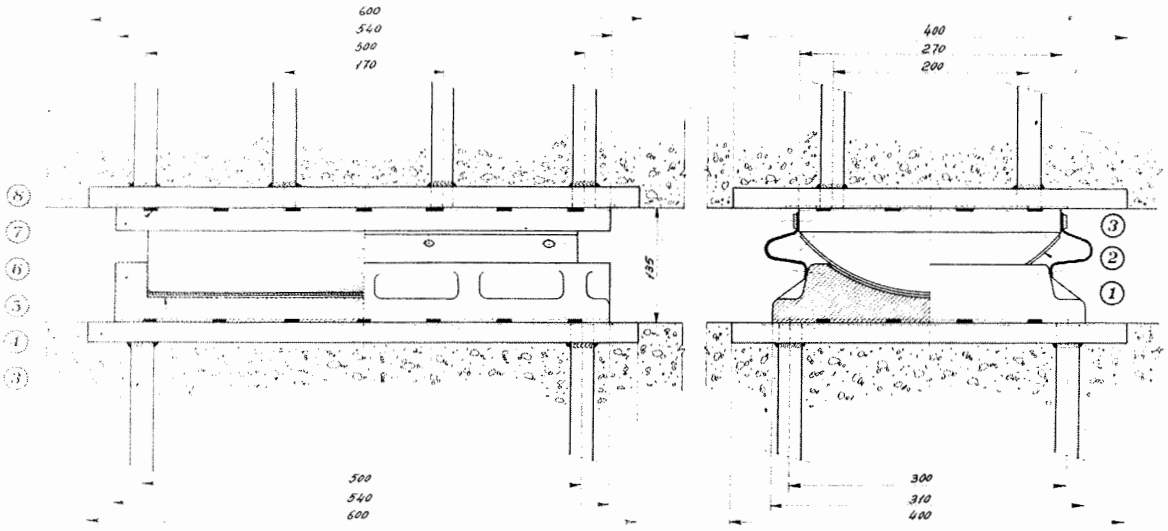
Type B: Single-direction bearing with hinge. Permits rotations around one horizontal axis and translations in a single horizontal direction.



Type B: Mobile bearing:

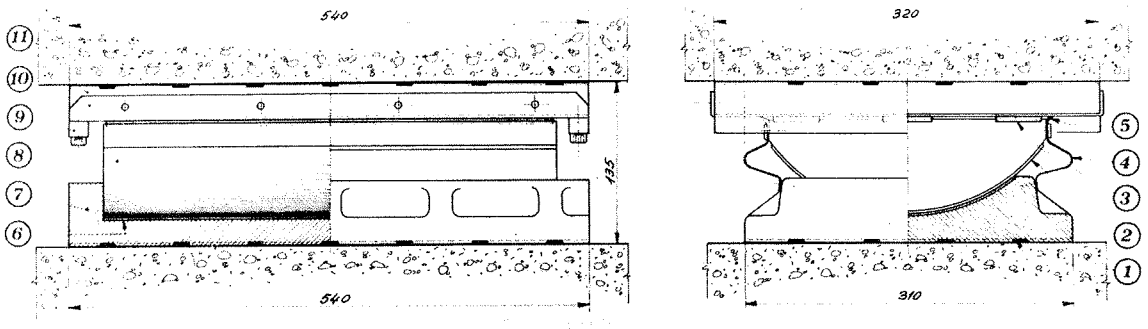
1 - Epoxy mortar housing; 2 - Base steel element; 3 - Rubber slabs bonded to metal plates; 4 - Metal plates; 5 - Intermediate steel element; 6 - PTFE strips; 7 - Upper element stainless steel sheet; 8 - Upper steel element.

Type C: Fixed bearing with hinge. Permits rotations around one horizontal axis and prevents any translation movement.



Type C: Hinge fixed bearing.

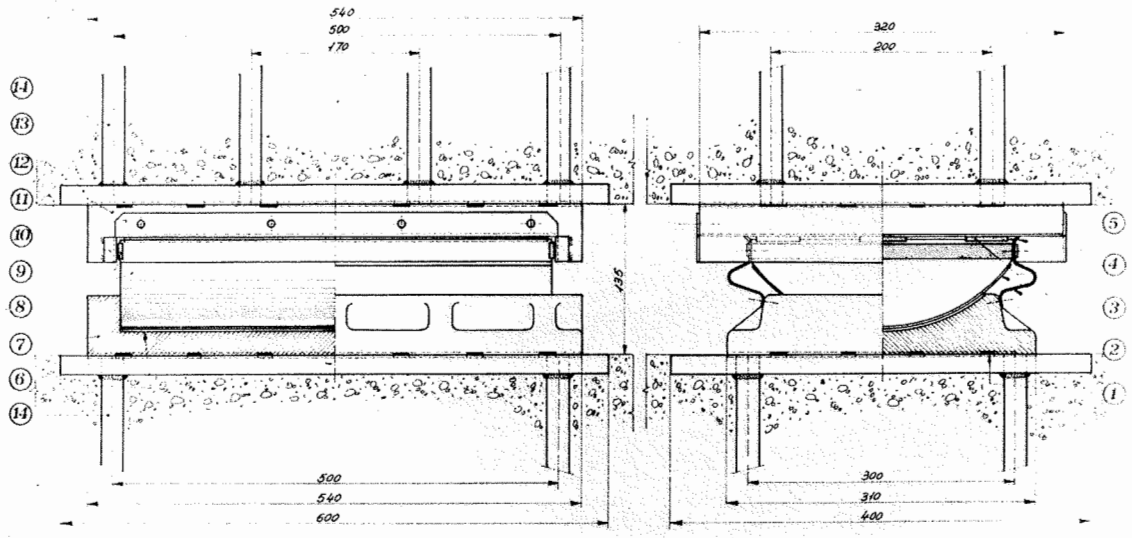
1 - Epoxy mortar housing; 2 - Intermediate element cylindrical sector stainless steel surface; 3 - PTFE strips; 4 - Antidust and rust protection; 5 - Dust scraper; 6 - Base element cylindrical sector PTFE seat; 7 - Base steel element; 8 - Intermediate steel element; 9 - Side rods; 10 - Upper element stainless steel sheet; 11 - Upper steel sliding element.



Type D: Single-direction mobile bearing:

1 - Epoxy mortar housing; 2 - Antidust and rust protection; 3 - Intermediate element cylindrical sector stainless surface; 4 - PTFE strips; 5 - Dust scraper; 6 - Base element cylindrical sector PTFE seat; 7 - Base steel element; 8 - Intermediate steel element; 9 - Side PTFE strips; 10 - Upper element s.s. sheet; 11 - Upper element s.s. sheet; 12 - Upper steel sliding element; 13 - Plate with anchor bolts to be let into the concrete; 14 - Anchor bolts.

Type E: Multidirectional bearing with hinge. Permits rotation around one horizontal axis and translations in any horizontal direction.



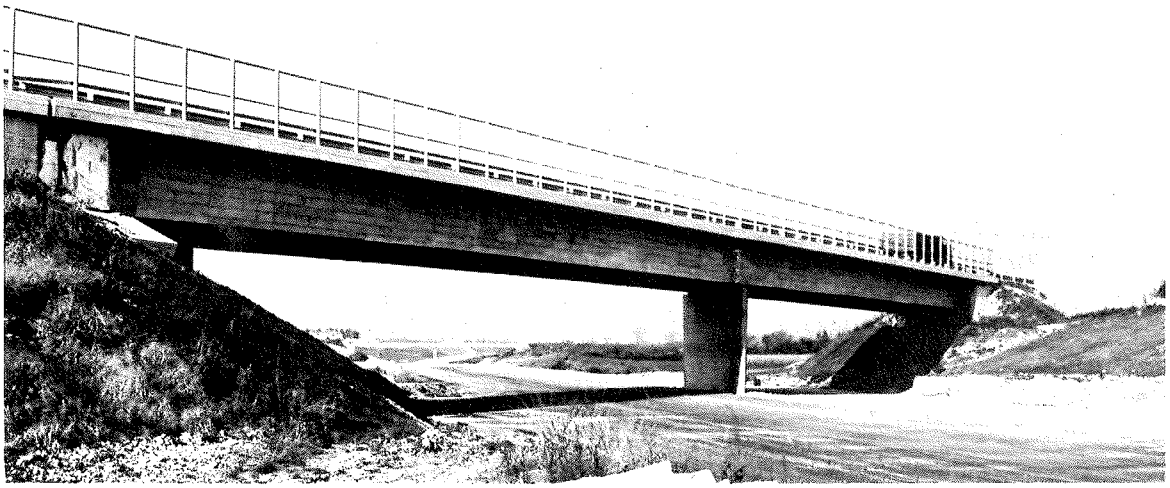
Type E: Multidirectional mobile bearing:

1 - Upper element cylindrical sector stainless steel surface; 2 - Antidust and rust protection; 3 - Anchor bolts; 4 - Plate with anchor bolts to be let into the concrete; 5 - Base element cylindrical sector PTFE seat; 6 - Base steel element; 7 - Upper steel element; 8 - Epoxy mortar housing.

Considering the enormous importance of bearings, we indicate the position of these bearings in any bridge:

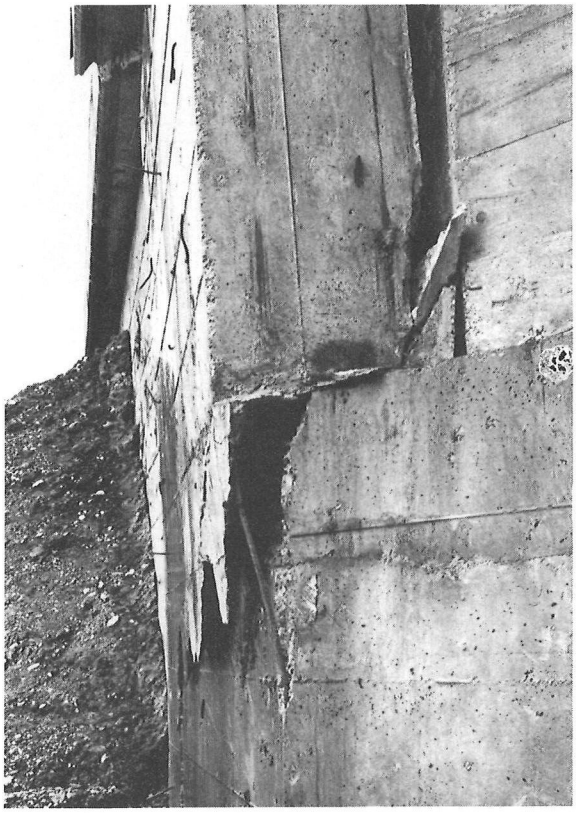
Bridge no. 11, viaduct over the OSOVANNA PRIMA road, km 10.+754. This is a lightened slab (36) with two spans of 45 m each. The slab is rotated by $0^{c}1540$ allowing maximum displacements of 11 cm abutment on the Buia side. The mobile bearings on the bridge abutments moved on an average of 10 cm; and the fixed ones on the pier are deformed. The right-hand headpiece crossbeam is damaged (37-42).

36: Osovanna I road, general view.





37: Right abutment, detail.



38: Right abutment, detail.



39: Deck, detail.

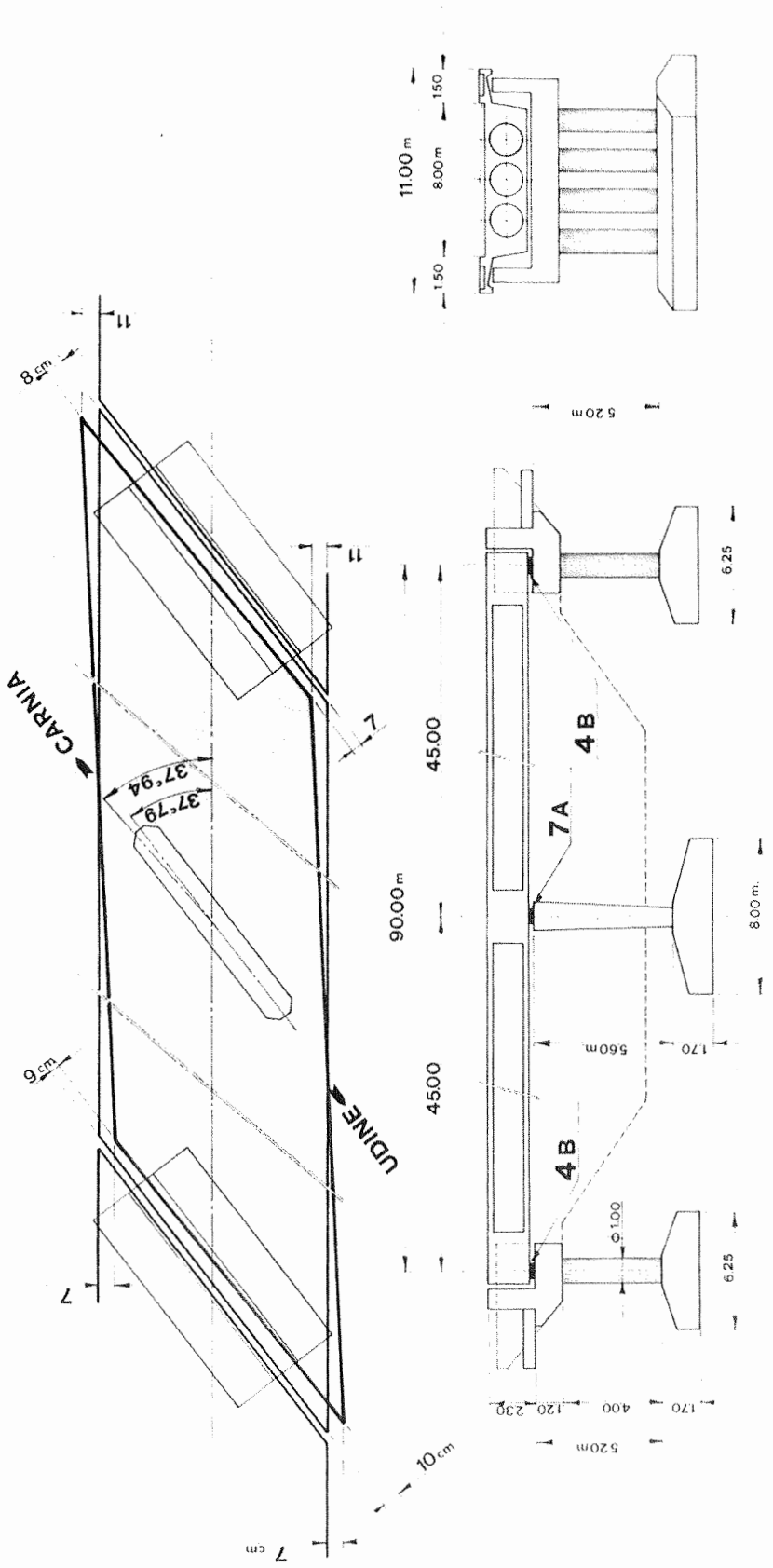


40: Left abutment, detail.

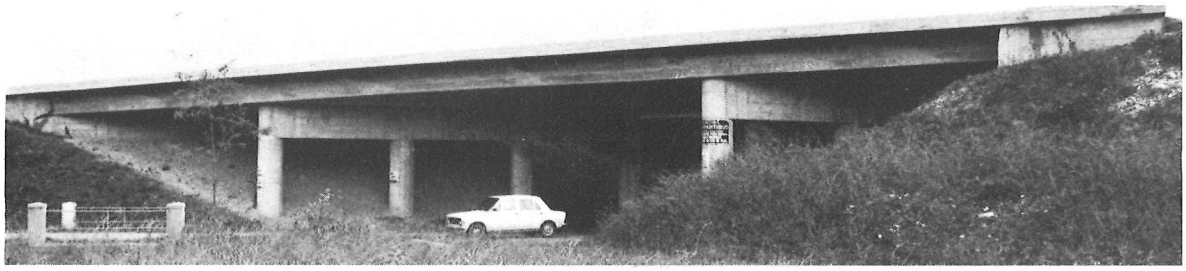


41: Left abutment, bearings.

42: Osovanna I road.



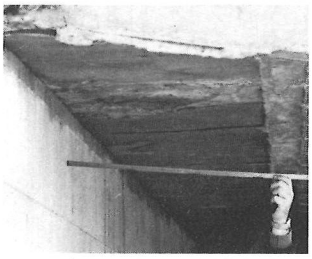
Bridge no. 12, viaduct over the Juliense road, km 12.+ 813. This is a lightened slab (43) with two spans of 11.00-18.41-11.00 m respectively. The deck is rotated through $1^{c}300$ with a maximum side displacement of 70 cm, and longitudinal displacement of 69 cm corresponding to the Udine abutment to the right. The fixed and mobile bearings are completely out of position and some of them have fallen. The piers show maximum displacements of up to 3 cm at a height of 3.50 m from their bases (44-52).



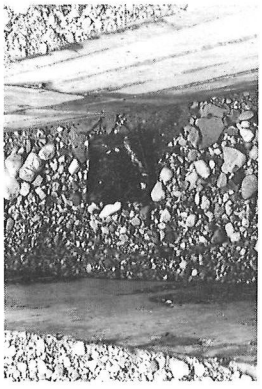
43: Viaduct over the Juliense road, km. 12.+813. General view.



44: Abutment.



45: Bearing displacement.



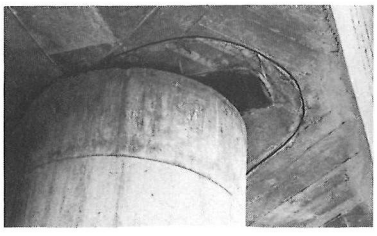
46: Bearing exit.



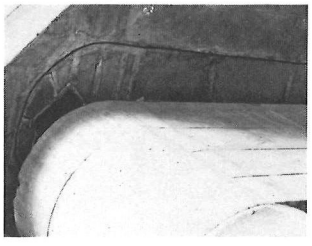
47: Bearing exit.



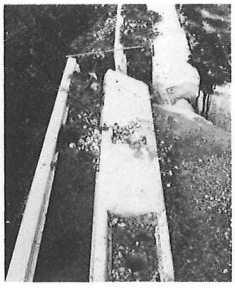
48: Detail.



49: Slab displacement.

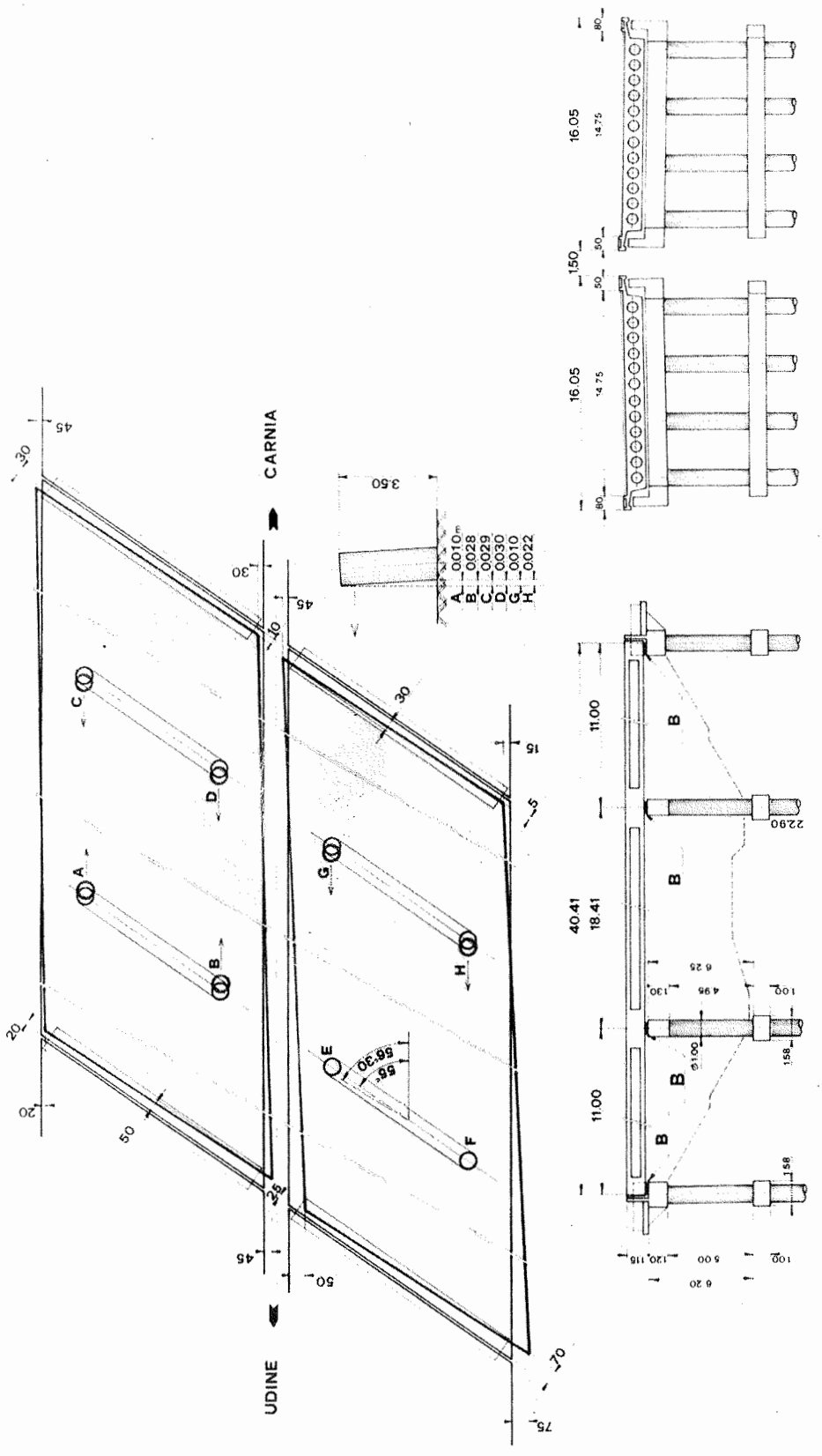


50: Slab displacement.



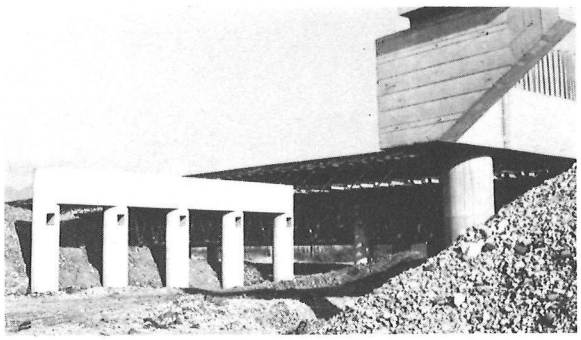
51: Displacement.

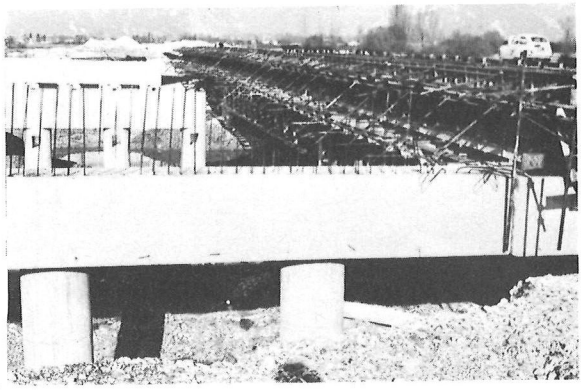
52:
Juliense
road, km.
12.+813.



Bridge no. 13, over the Roggia Alta River, km 13. + 954. At the moment of the earthquake, the deck (53, 54) was under-construction, and only half of the right-hand road was built. The bridge underwent a rotating movement to the right with maximum side displacement of 45 cm on the Carnia side. The piers are damaged, both those supporting the already constructed part and the unfinished part, with maximum displacements of approx. 1 cm towards Udine (55-57).

53: Bridge over the Roggia Alta, general view.

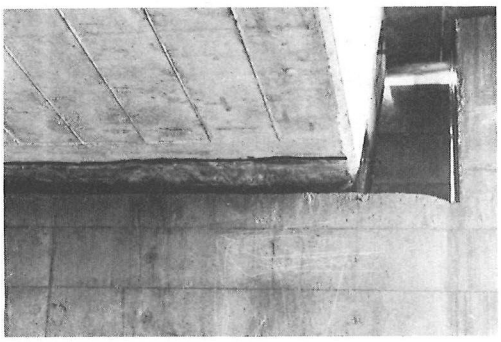




54: General view.

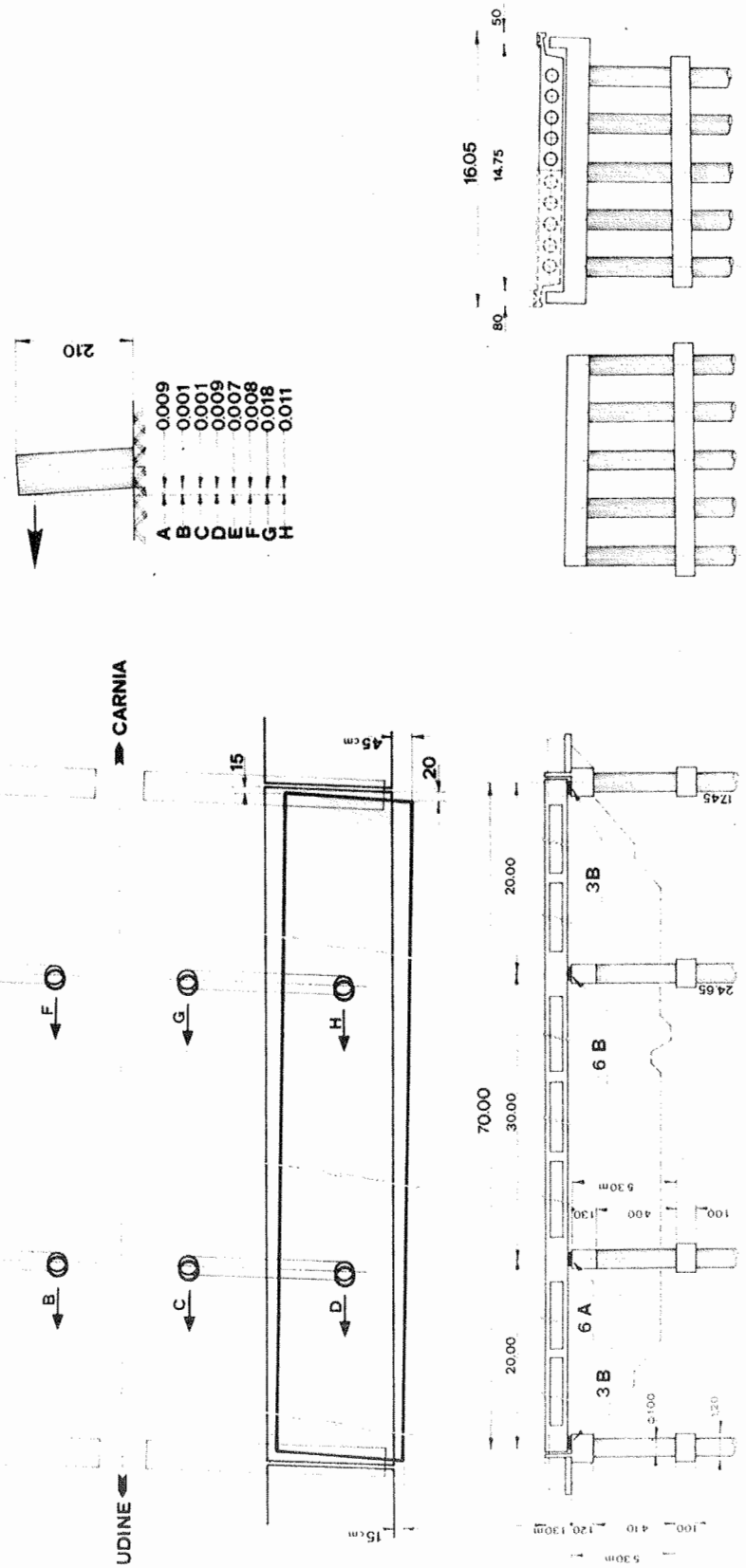


55: Slab displacement.



56: Carnia side abutment, bearing.

57:
Roggia Alta
road, km.
13.+954.

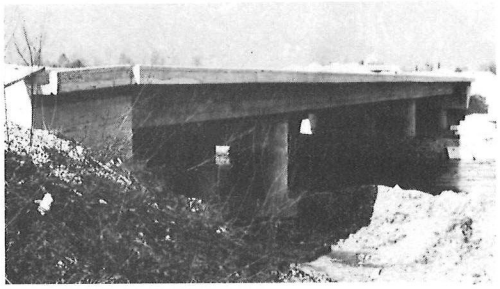


Bridge no. 14 over the Ledra River, km. 14.+ 169. This consists of a lightened slab (58, 59) with three spans, measuring 18.20-34.00-18.20 m each. The deck is rotated, maximum displacement being 19 cm to the right on the Carnia abutment. The 3.50-m high piers moved up to distance of 8 cm (60-63).

58: Bridge over the Ledra River, general view.

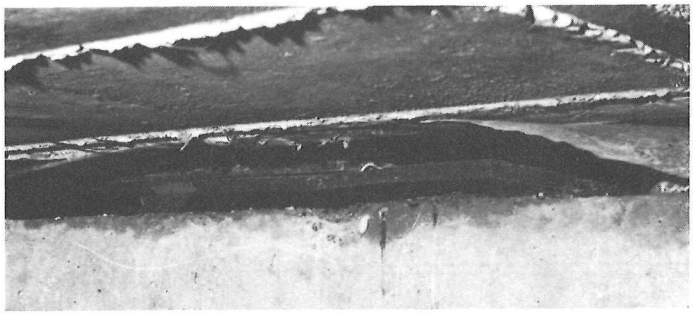


59: General view.

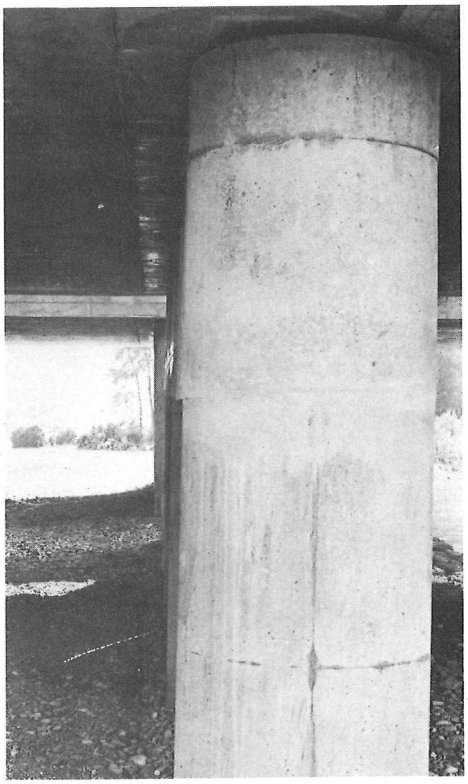




60: Bearings, detail.

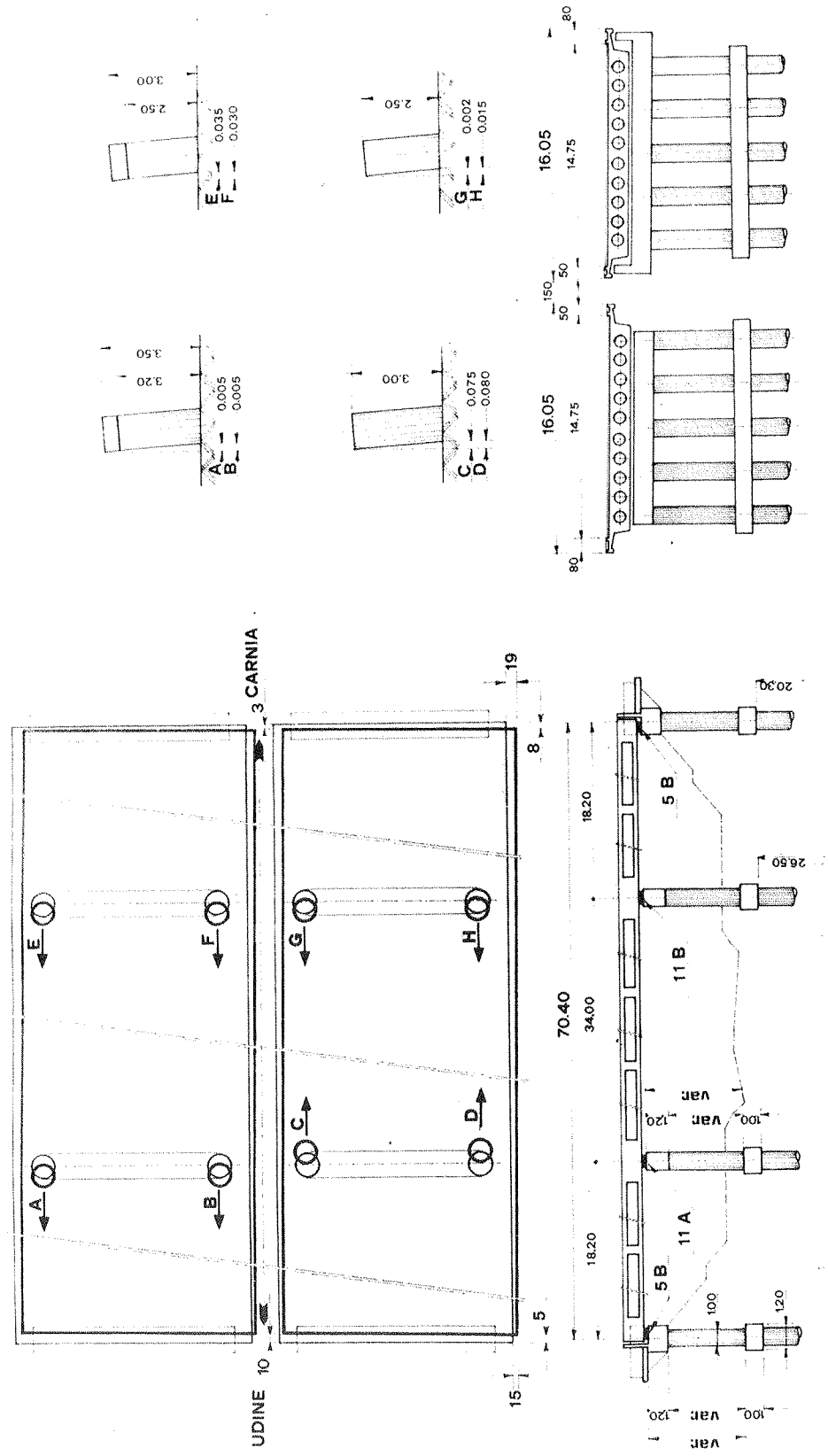


61: Udine side abutment, right lane, detail of bearing.

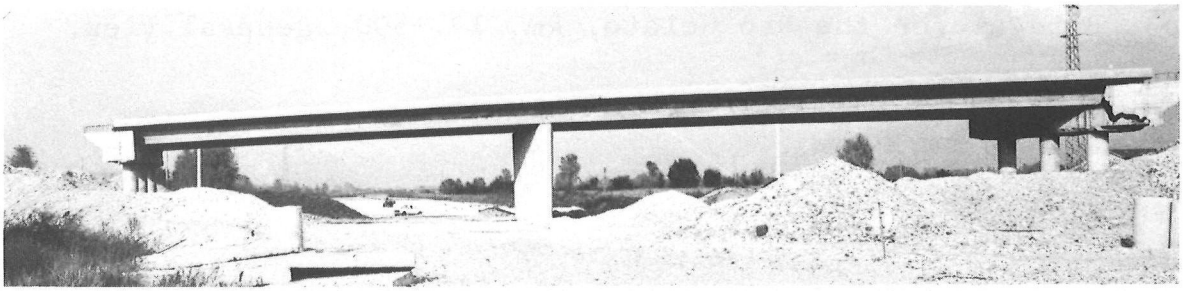


62: Pier displacement.

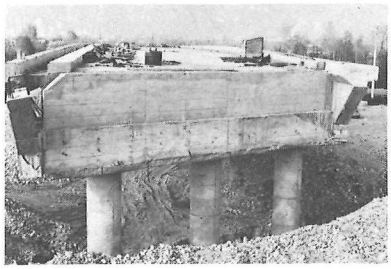
63:
Ledra River
road, km.
14.+169.



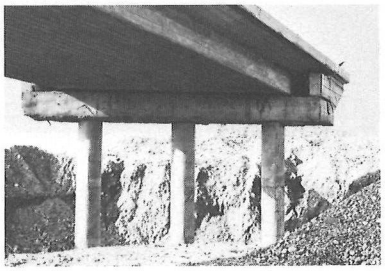
Bridge no. 15 over the OSOVANNA SECONDA road, km. 15.+ 011. This consists of a lightened slab (64) with two spans of 30.00 m each. The deck has been translated to the right by 11 cm. on the Carnia abutment. The bearings have moved out of their position (65-68).



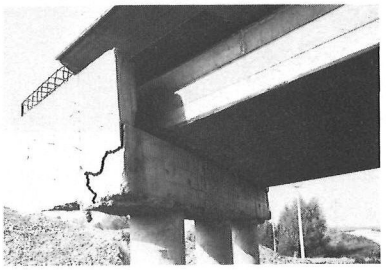
64: Osovanna II bridge, general view.



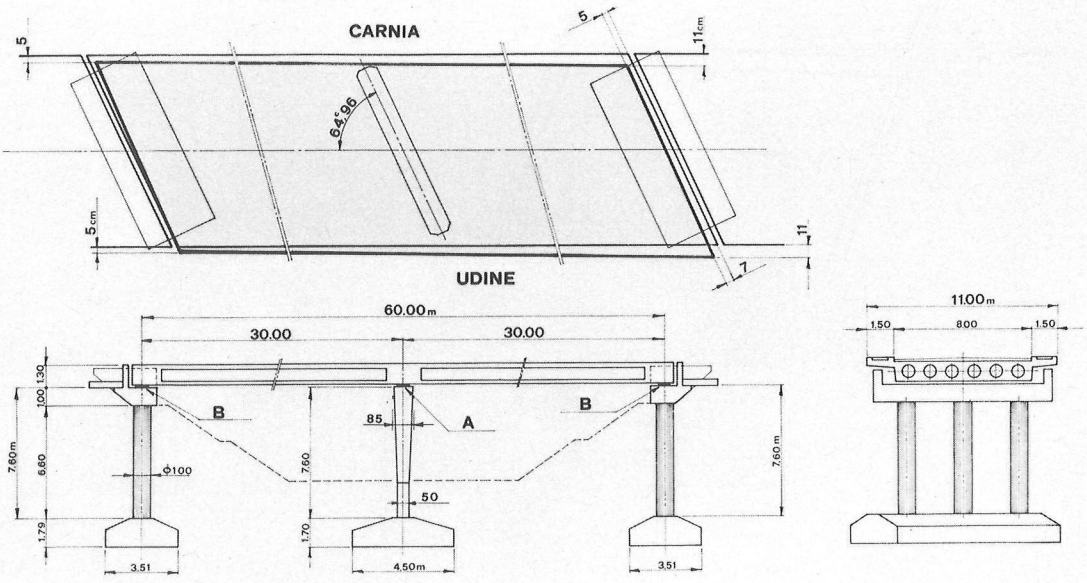
65: Abutment detail.



66: Detail.



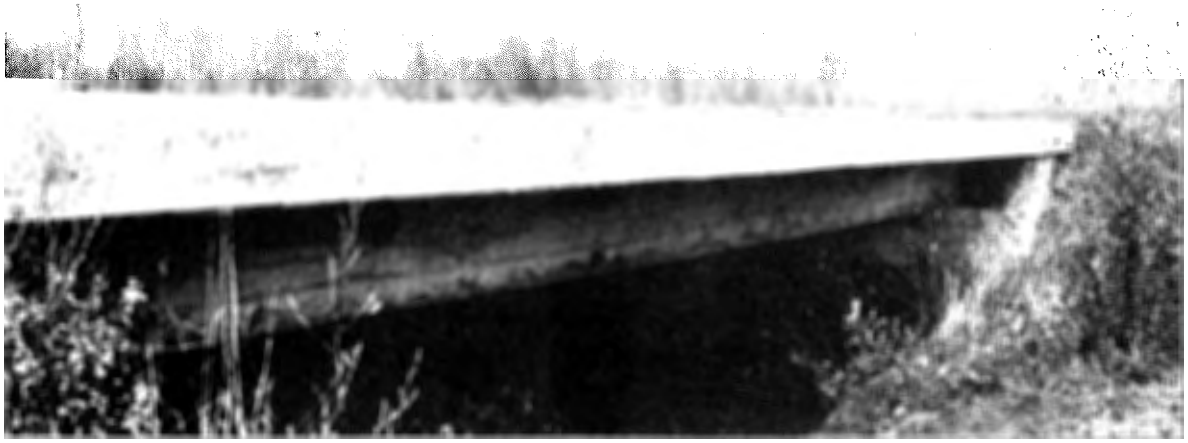
67: Detail.

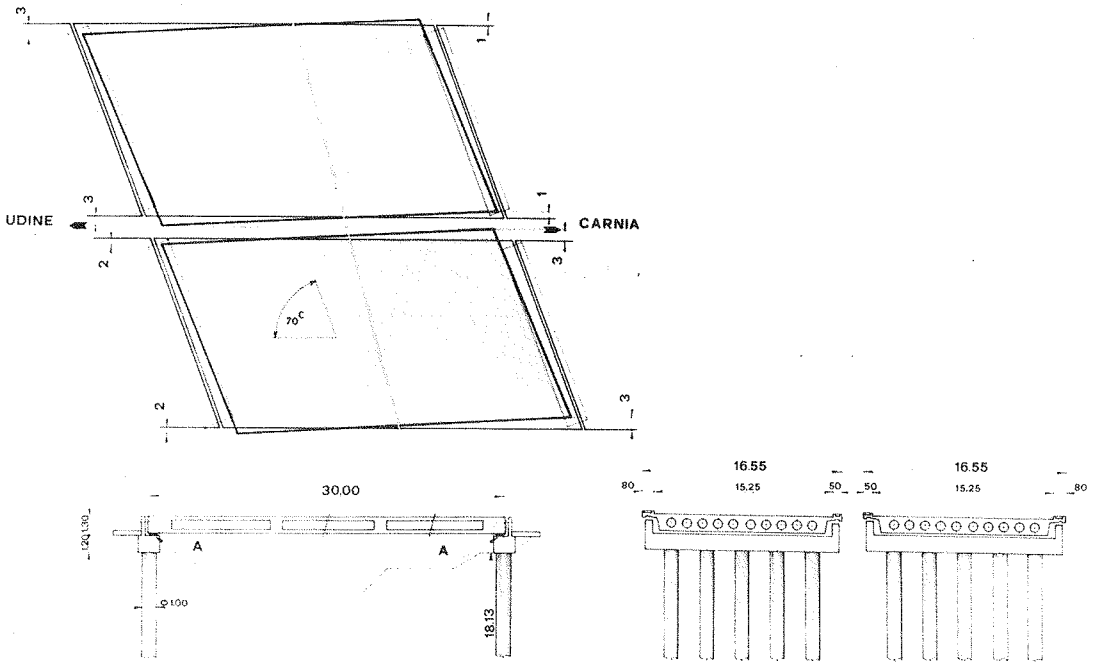


68: Osovanna II road, km.15.+011.

Bridge no. 16 over the Rio Gelato River, km 15.+ 890. This is a lightened slab (69) with a single span of 30.00 m. The neoprene bearings with three layers of steel behaved well. However, the deck struck the retaining walls, damaging them (70).

69: Bridge over the Rio Gelato, km. 15.+890, general view.





70: Bridge over the Rio Gelato, km. 15.+890.

Bridge no. 17 over the Gemona-Casarsa railway line, km 19.+ 851. This was under construction (71). The foundations and the left column of pier no. 1 were completed. Pier no. 2 was com-complete, as was the Carnia abutment with the exception of the dosseret. This work deserves special attention owing to the behaviour of the piers (72).

The left single column (Ø 150 cm) shows displacement at 10.45 m and is damaged horizontally all round the circumference at about 3 m from the base. Pier no. 2, made of two column frames (Ø 150 cm) and a crossbeam, was complete. The left frame suffered much damage, the upper crossbeam being rotated and translated and the columns evidently damaged at their bases, with shearing of the metal reinforcement. The column lesions go to a considerable height. The right frame appears even more badly damaged, with particularly serious lesions in the right column. This is damaged horizontally all round its circumference, at various heights to about 2 m from its base (78-82). The behaviour of the piles under the base plinth is also interesting (74-77). In the area of foundation pile/plinth attachment, the reinforced concrete of the piles is badly crushed, and can even be removed by hand, for about 30 cm over a truncated cone surface, inclined 45°. This phenomenon was also found in the foundation piles of other structures, together with frequent shearing of the metal reinforcements.

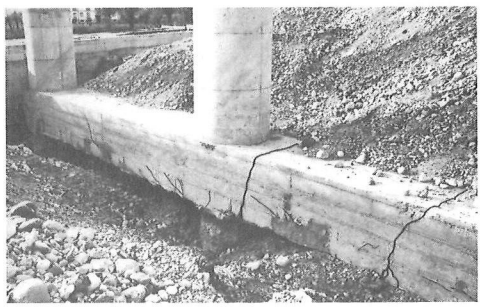
The base crossbeam of the Carnia abutment (73) was damaged, piers being damaged at the point of attachment of the crossbeam. The bridge yard contained 12 unprestressed beams of which 7 fell, producing serious damage (88).

71: Gemona-Casarsa railway bridge.



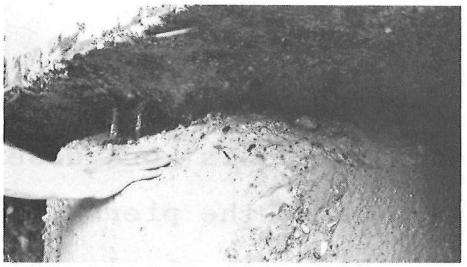
72. Detail of pier.



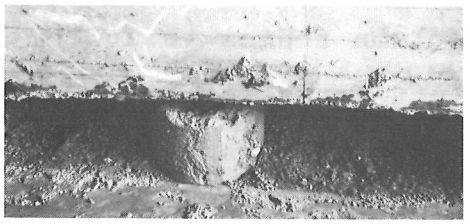


73: Detail of headpiece.

74. Detail of foundation pier.



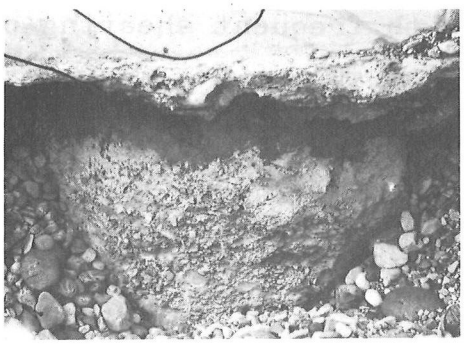
75: Broken foundation pile head.

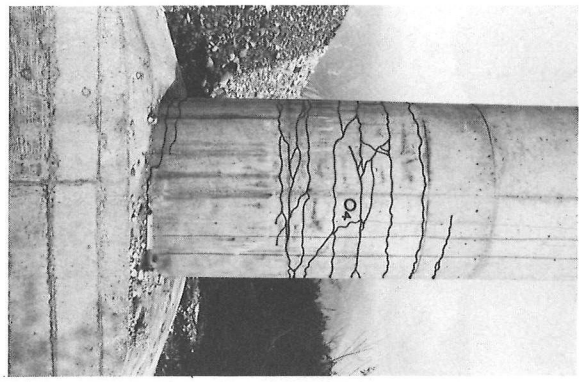


76: Plinth/pier attachment breakage.

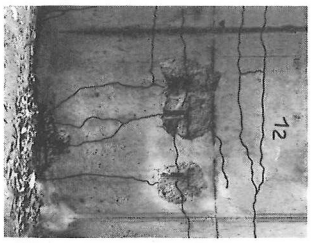


77: Pier breakage.

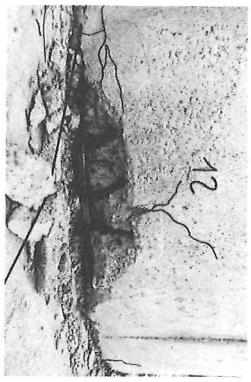




81.



85.



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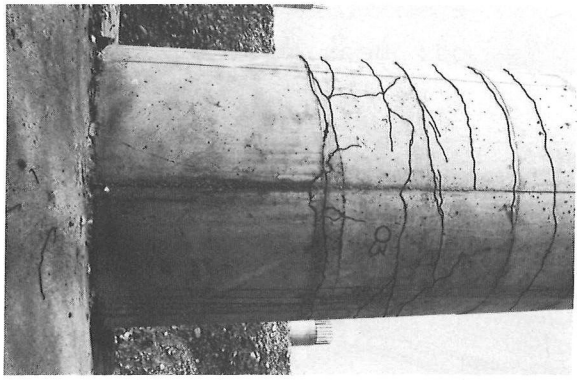
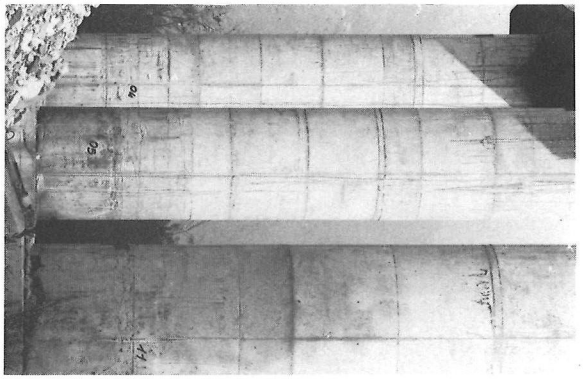
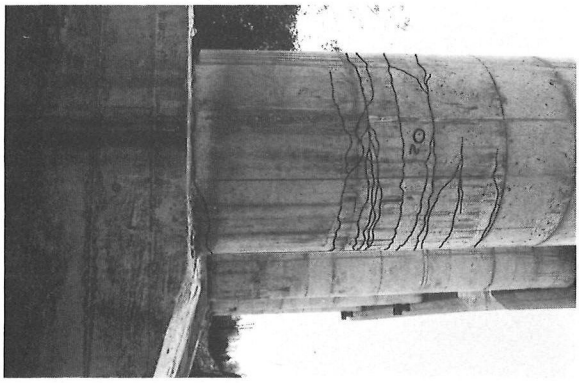


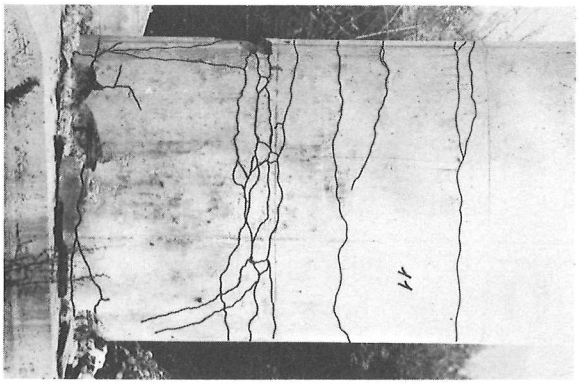
80.



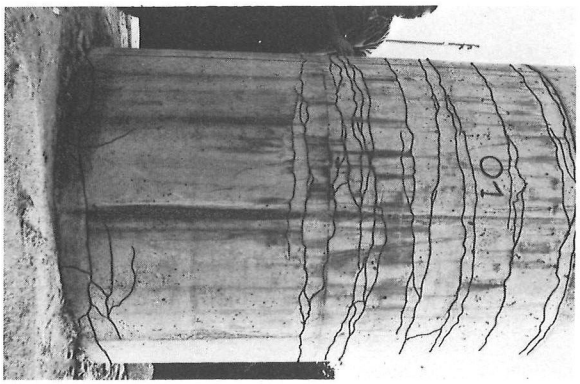
84.



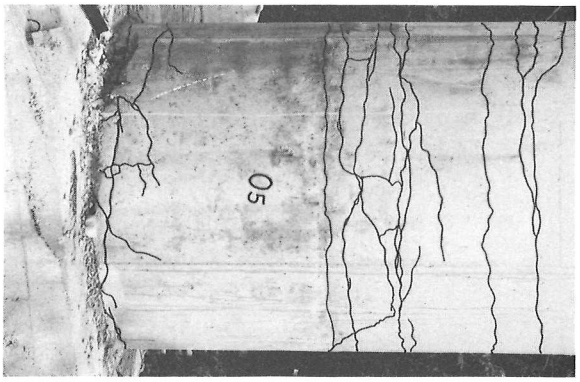
79.



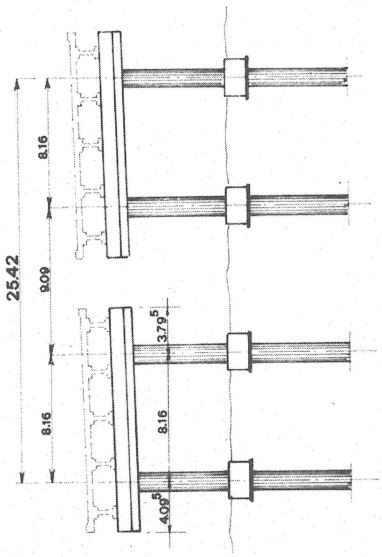
83.



78.



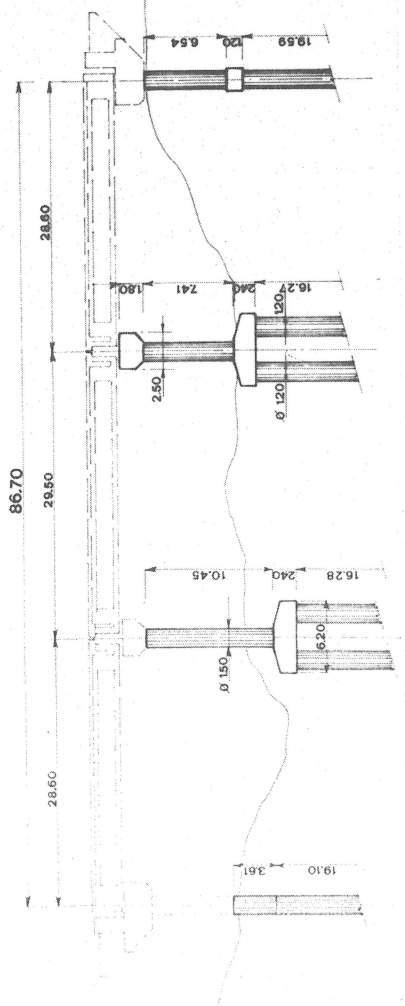
82.



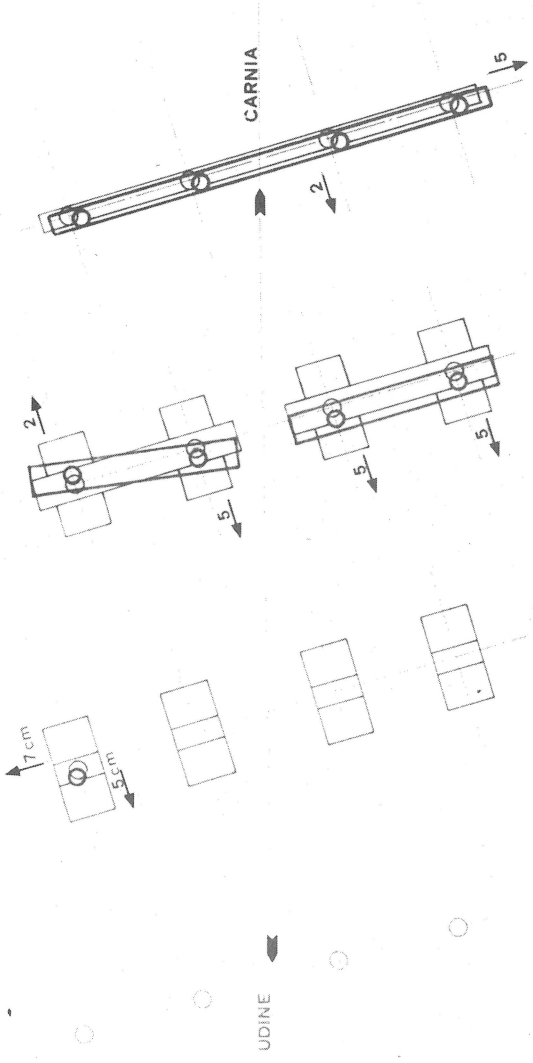
Gemona-
Casarsa rail-
way bridge,
km. 19.+85.1.



88: Beam yard.



87.

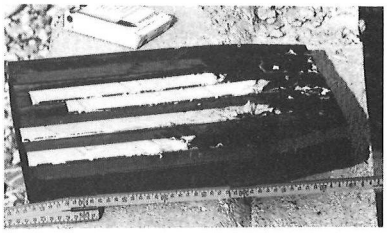


87a.

Bridge no. 18 over the Ledra-Tagliamento canal, km 20.+ 304. This was under construction. The slabs of the side-spans were completed (89). The bearings were made of strips of neoprene reinforced with steel and Teflon (90). The two slabs on the Carnia side fell in May; of those on the Udine side the right one fell in September; the other is strongly displaced from its original position, with longitudinal translations of 60 cm and transversal translations of 100 cm (91-95). The piers are damaged at various heights, also as a result of the fall of the slabs (96).



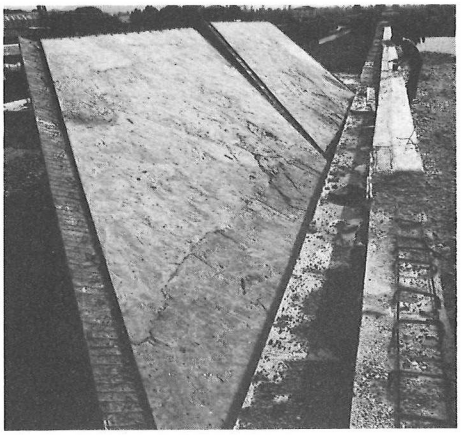
89: Bridge over the Ledra-Tagliamento canal, general view.



90: Bearing, detail.



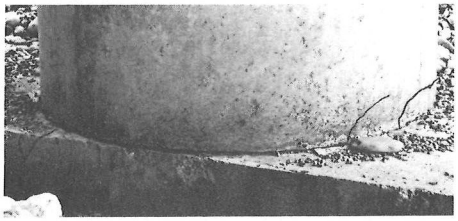
92: Pier detail.



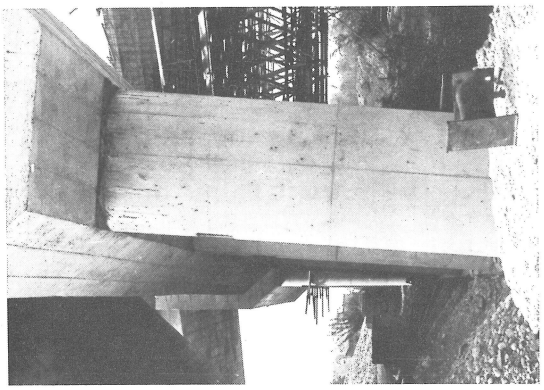
91: View, Carnia abutment.



94: Slab collapse.

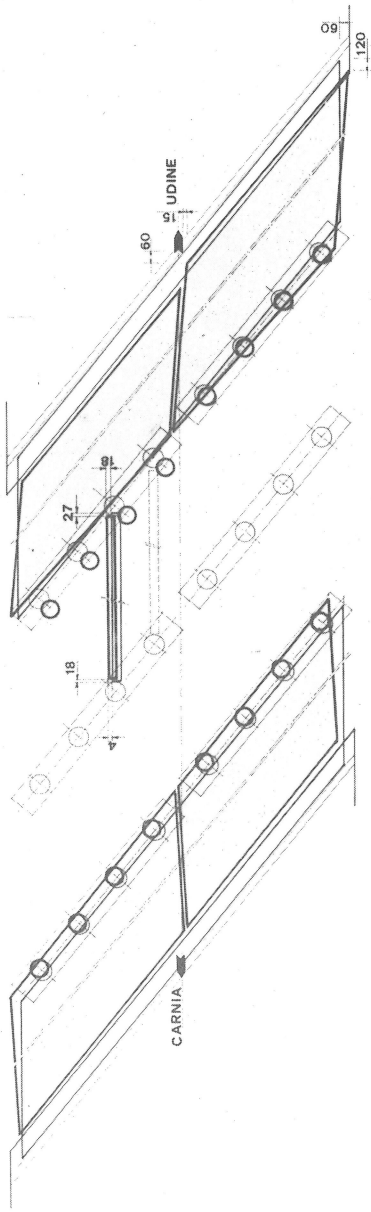


93: Pier, detail.

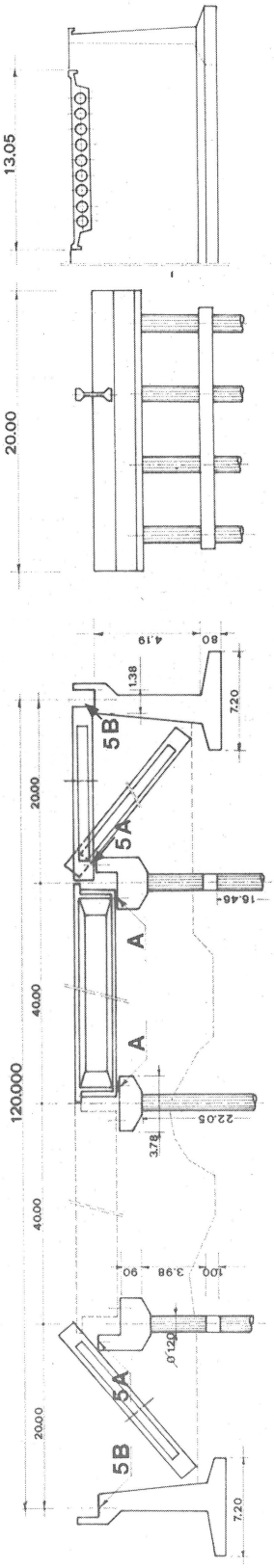


Bridge over Ledra-Tagliamento.

95: Pier tilting.



96a.



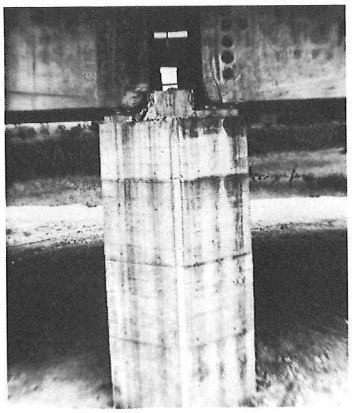
96.

Bridge no. 19, Tagliamento I; km 22.+ 216. This structure (97-99) was designed according to legislation for seismic areas. It is a bridge of 28 spans of 43.00 m each, fully completed at the time of the earthquake. Special blocking teeth were supposed to serve as a last safeguard against collapse (100-110). It should be noted how, transversally, the girder was safeguarded by similar blocking teeth which behaved satisfactorily. Longitudinally, instead, in both fixed and mobile bearings, anchoring plates were found out of position. These plates had been glued to the beams and piers with epoxy resins. There was also some impact between the headpieces of the beams (not yet completed) and the blocking teeth, with very serious damage to both (111).

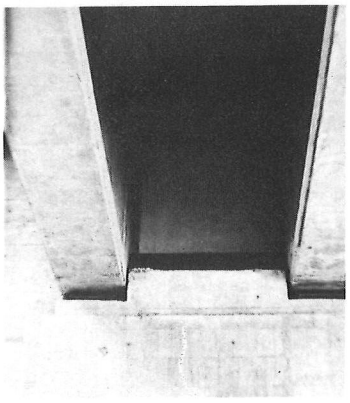
97: Bridge over the Tagliamento River, general view.



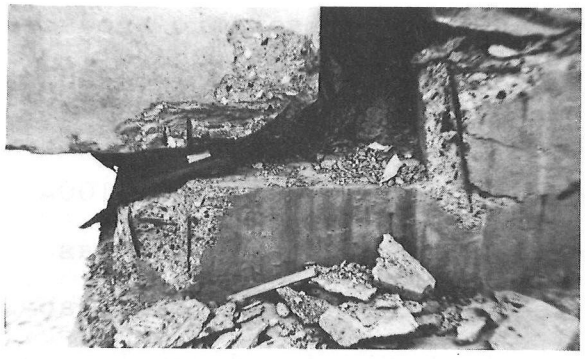
98: General view.



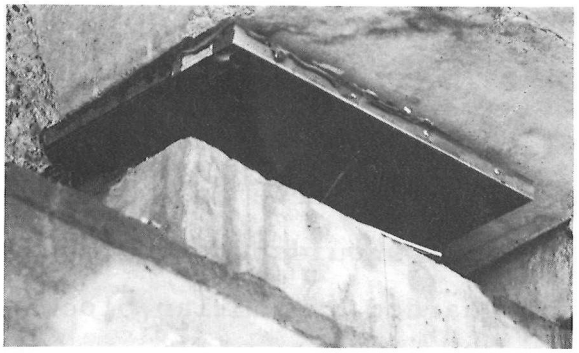
99: Bearing.



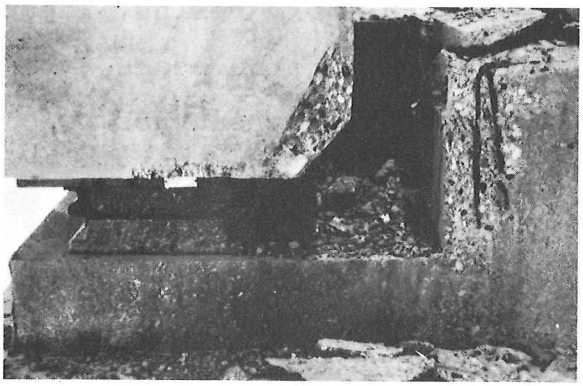
100: Blocking teeth.



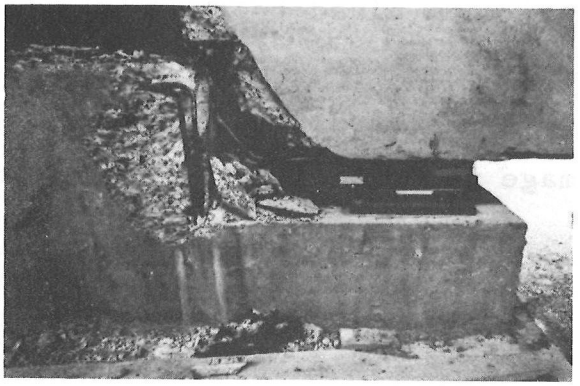
101: Breakage of bearing.



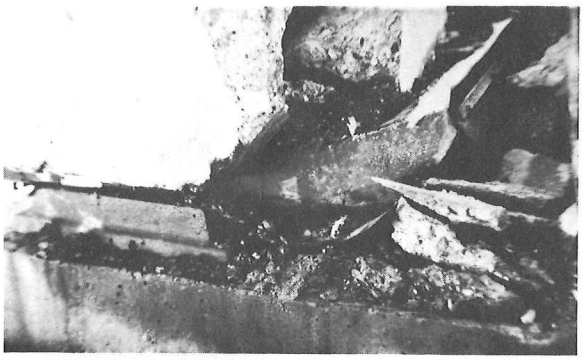
102: Beam head lesions.



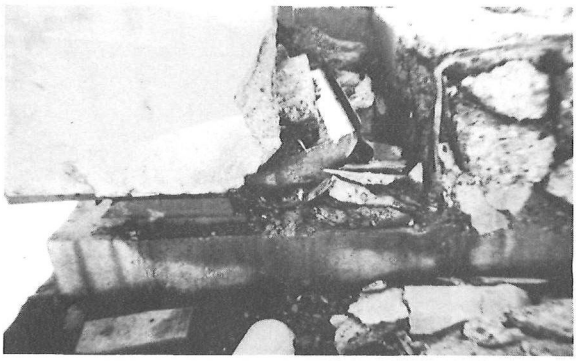
103: Beam head lesions.

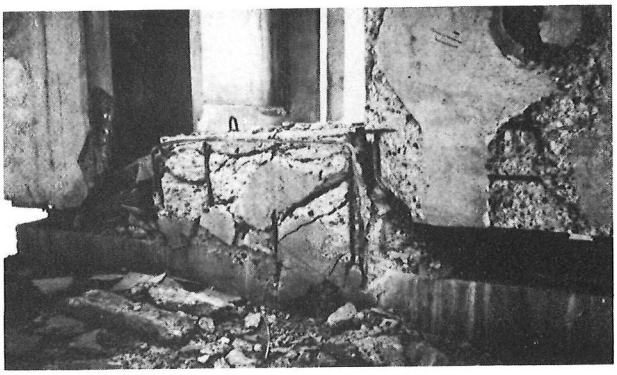


104: Breakage of bearing equipment.



105: Detail of bearing.





106: Detail of bearing.



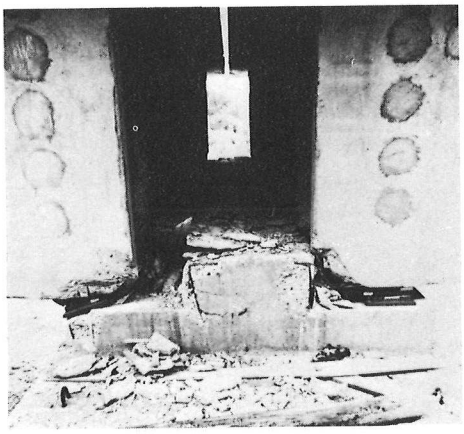
107: Detail of bearing.



108: Detail of bearing

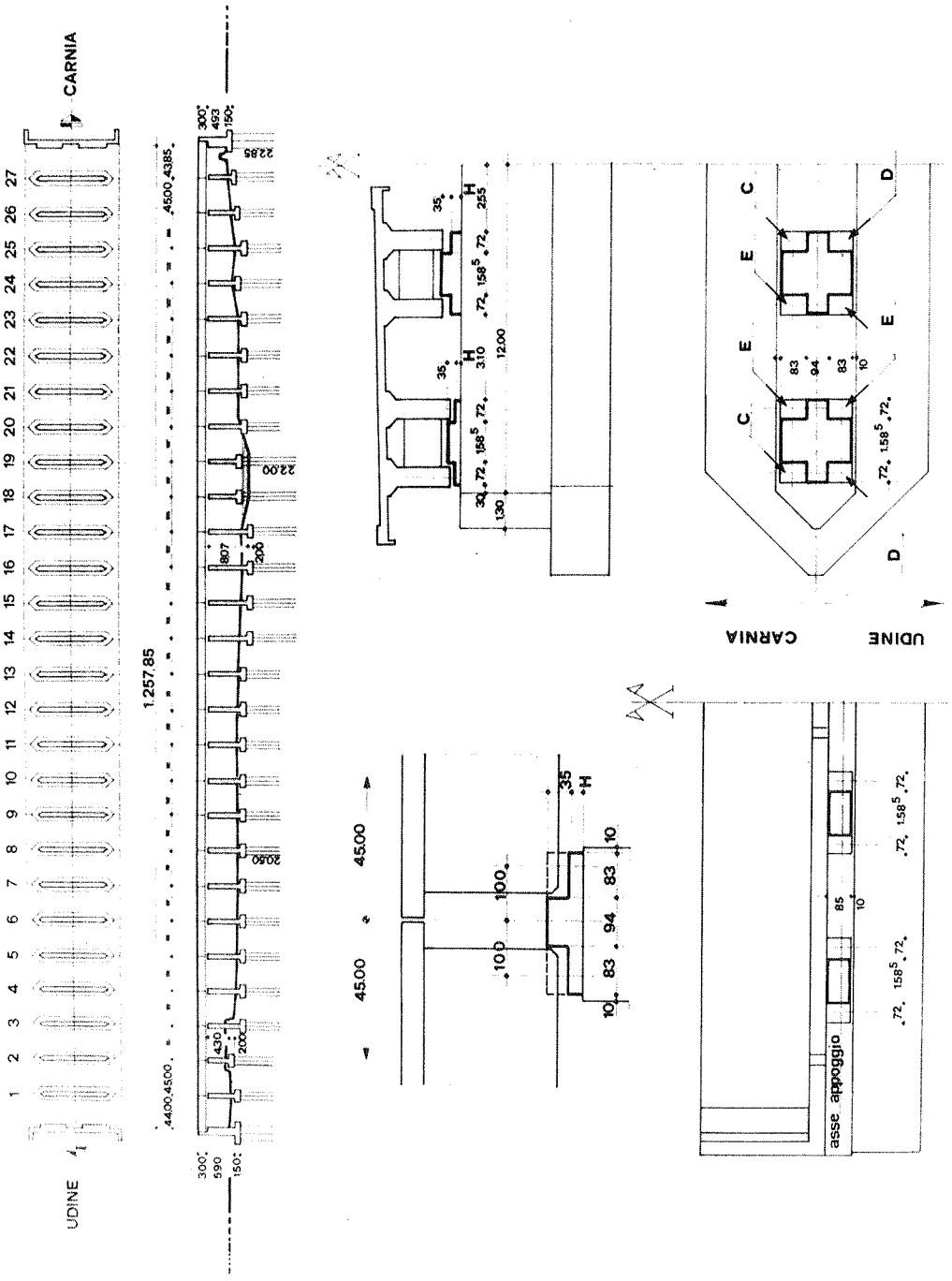


109: Detail of abutment.



110: Detail of bearing.

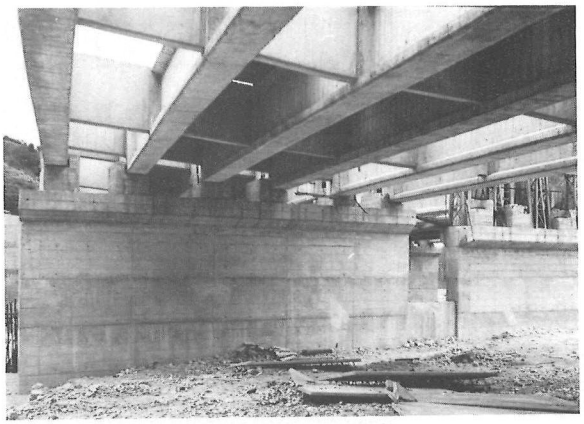
111: Tagliamento I bridge, km. 22.+216.



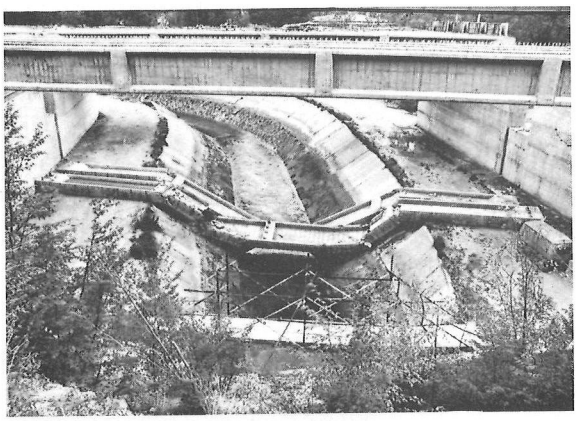
Bridge no. 20 over the ENEL canal, km 24.+ 770. The foundations, abutments and piers had been completed (112). The prefabricated beams had been placed on the piers, and the deck was being formed, with prefabricated slabs and crossbeams. The Udine abutment was translated towards the first pier by about 30 cm. Nine of the already-placed beams fell, since they were not sufficiently connected to each other. The bearings of all the beams were damaged (113-116).

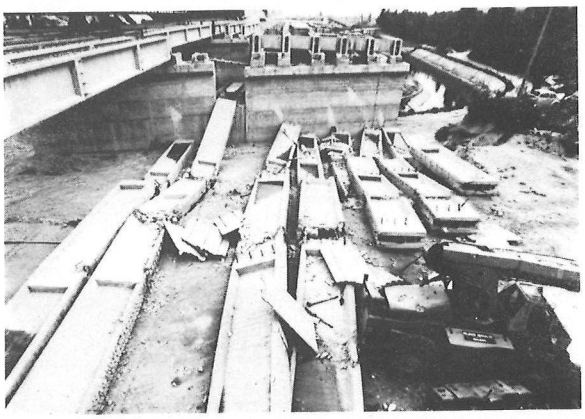
In the road yard along the highway were 16 precast beams set on reinforced concrete blocks. Fifteen of these were damaged by falling in May; the last fell in September (117,118).

112: Bridge over ENEL canal, km. 24.+770.



114: Fallen beams.

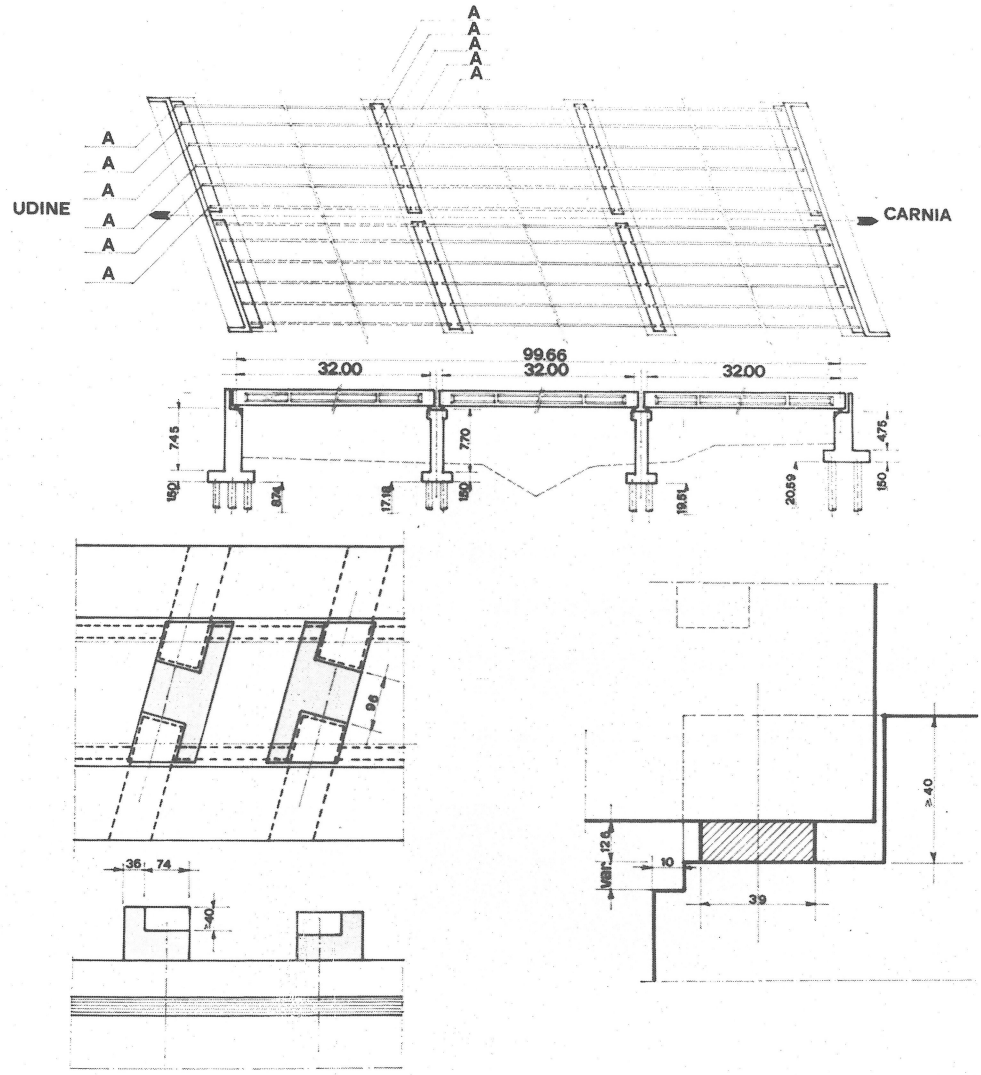




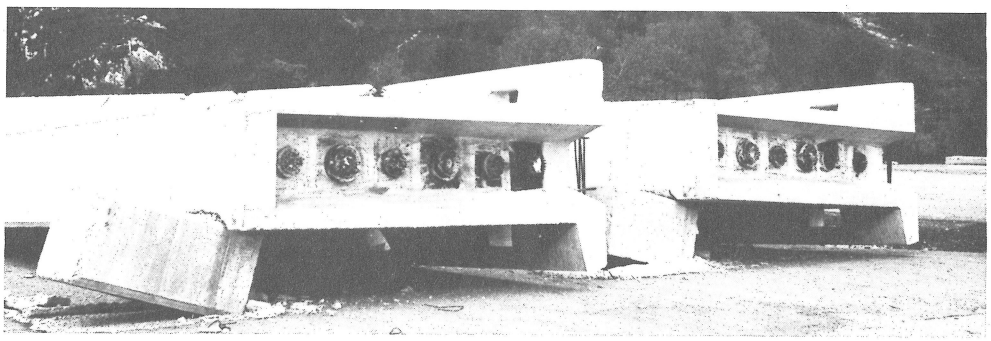
113: Fallen beams.



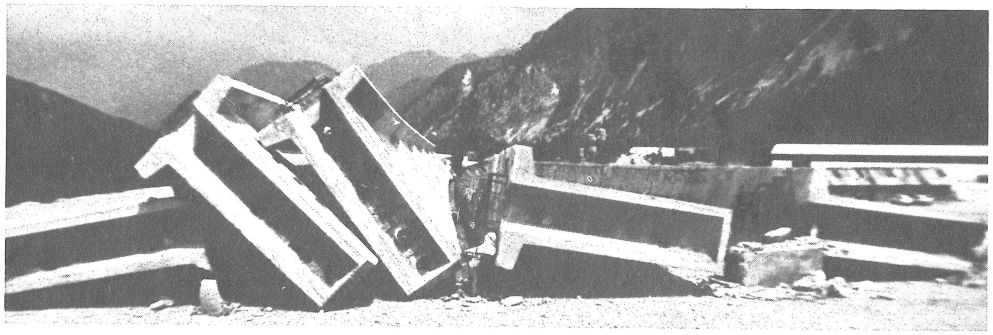
115: Fallen beams.



116: Bridge over ENEL canal, km. 24.+770.



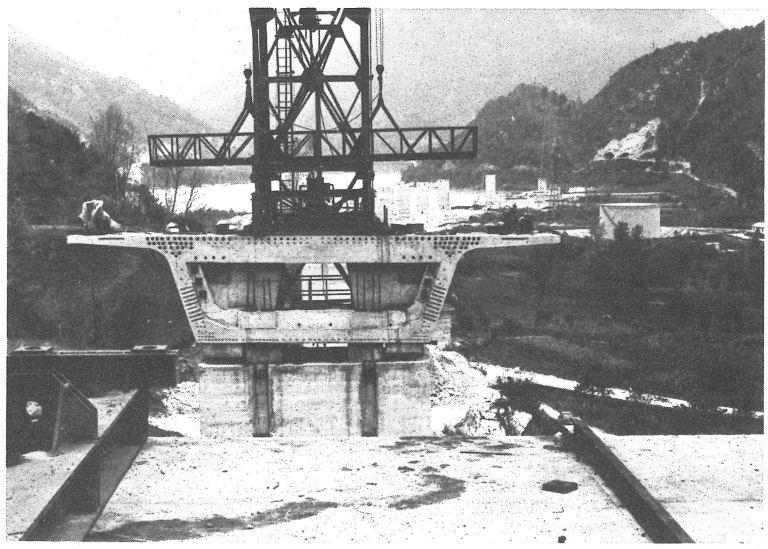
117: Trasaghis. Close-up of beams in yard.

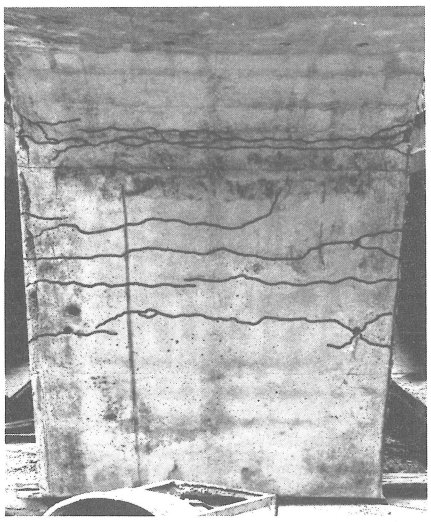


118: Fallen beams.

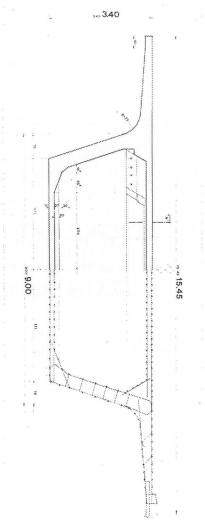
Bridge no. 21, at Somplago, km 31.+179. This structure was designed according to legislation for seismic areas. It is a very important static structure (119), composed of a deck made up of precast elements, prestressed in situ. The piers of the right-hand lane were already constructed; the others were in various phases of construction. There are horizontal capillary lesions in the piers from 10 to 16 cm. The September shocks caused further damage to other base piers. The prefabrication yard held several precast elements stacked in two planes. These show very evident lesions on both side walls (120-125). The upper and lower slabs of the precast elements are damaged too, together with the transversal elements for the stiffening and anchoring of cables. The transversal reinforcements do not appear to be correctly designed at the corners. It should be noted, however, that the lesions are substantially distributed in all areas even far from the corners. On the other hand, it should also be noted how the yard is situated precisely on the slopes of Mt. S. Simeone, the epicentre of many shocks (126-132).

119: Bridge at Somplago.

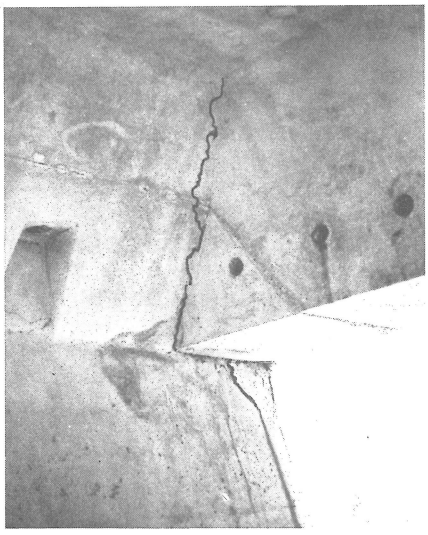




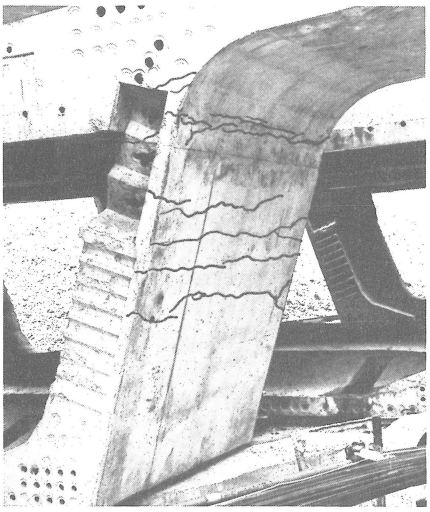
120: Prefabricated element.



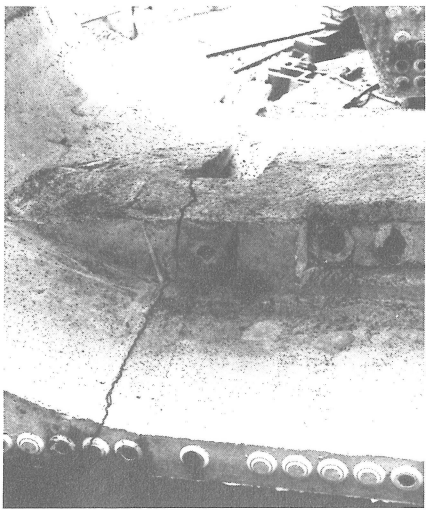
123: Section.



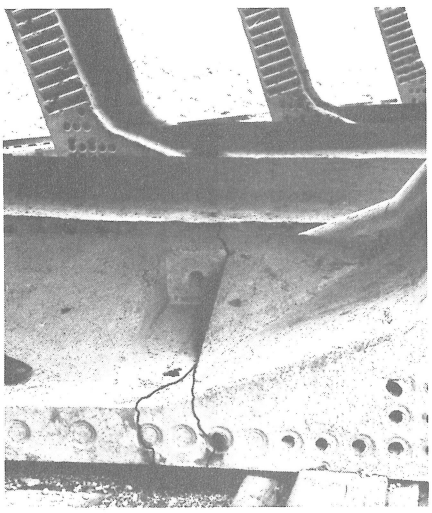
121: Detail of slab.



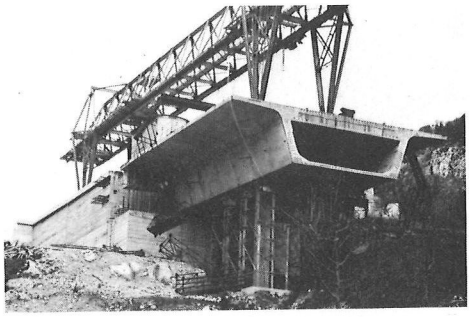
124: Lesions of core.



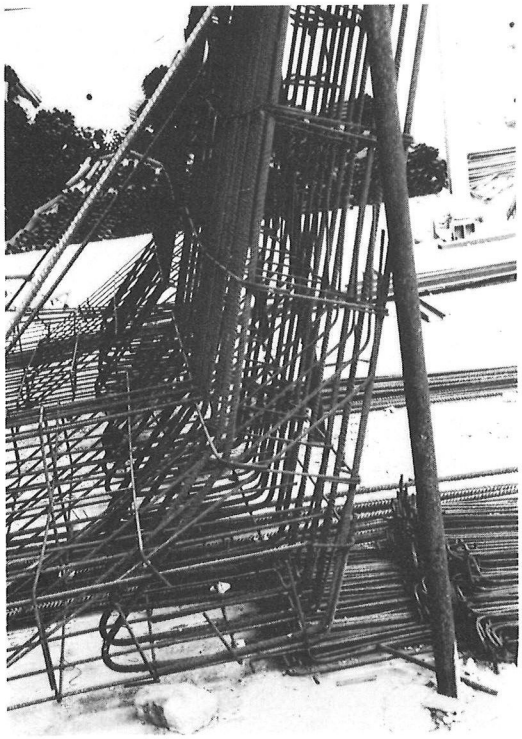
122: Lower slab lesions.



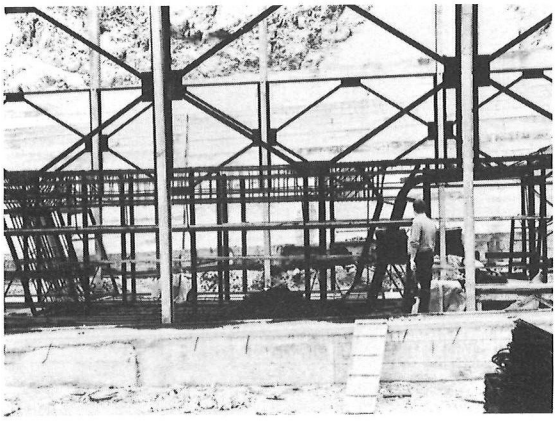
125: Lower slab lesions.



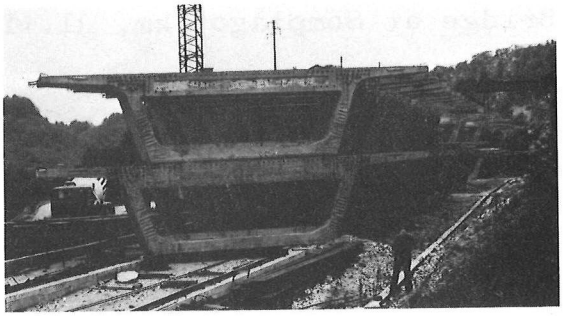
126:Carnia abutment.



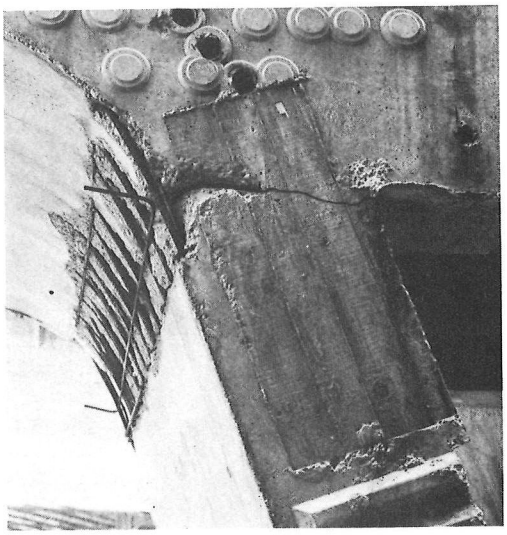
127: Scaffolding, detail.



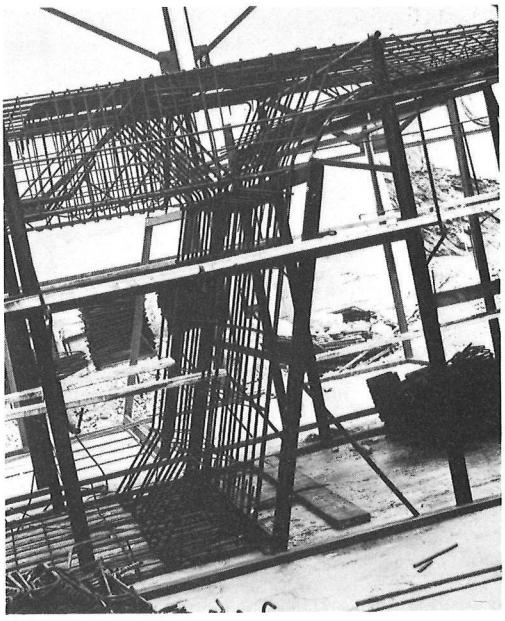
128: Scaffolding, detail.



129: Prefabricated elements.

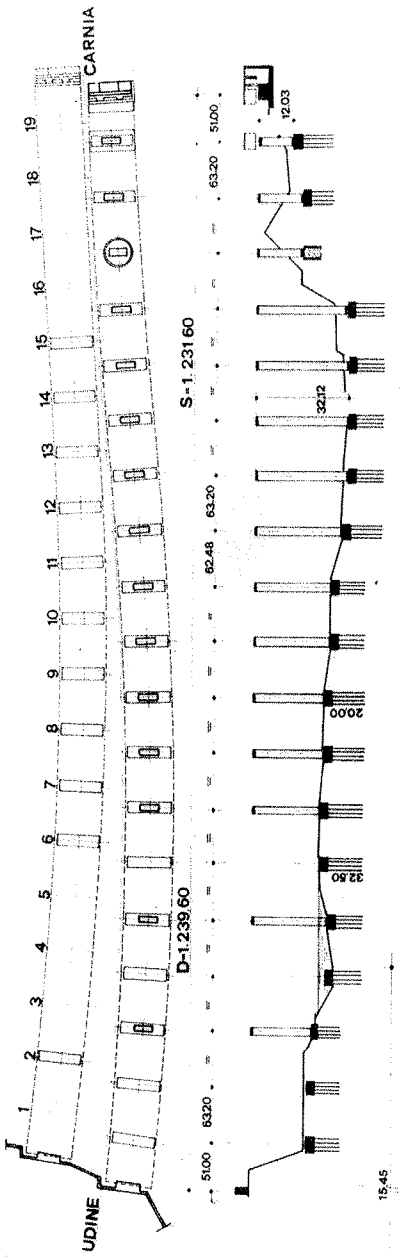


130: Destruction test.

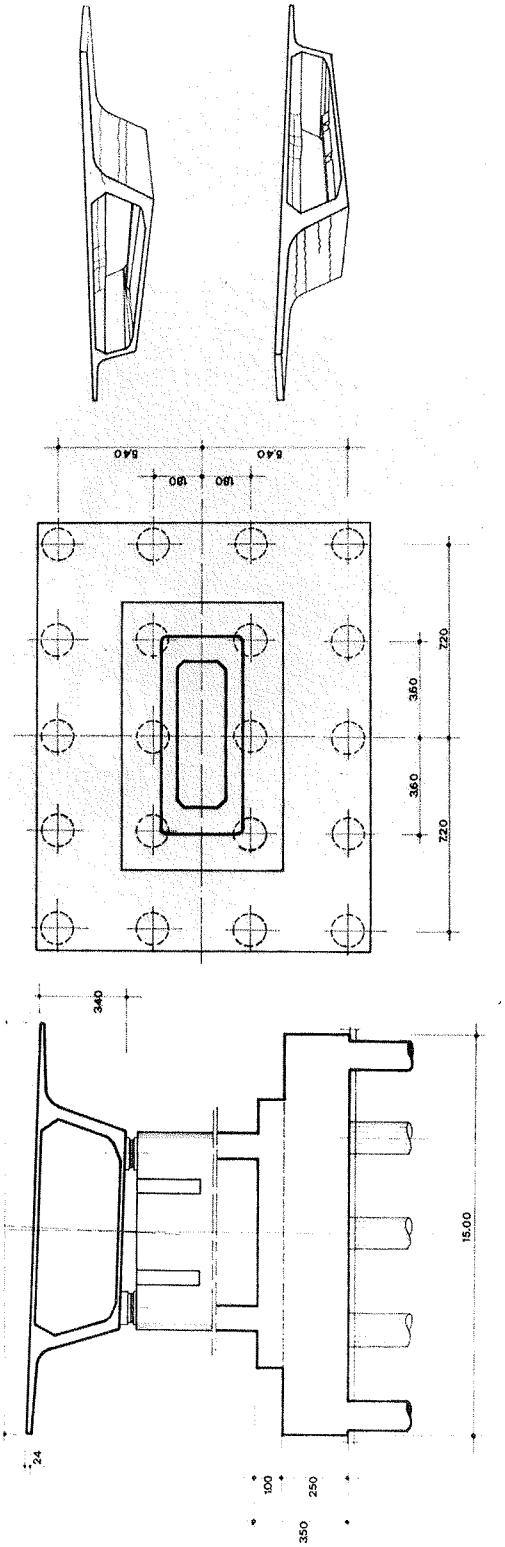


131: Scaffolding, detail.

Bridge at Somplago, km. 31.+179.

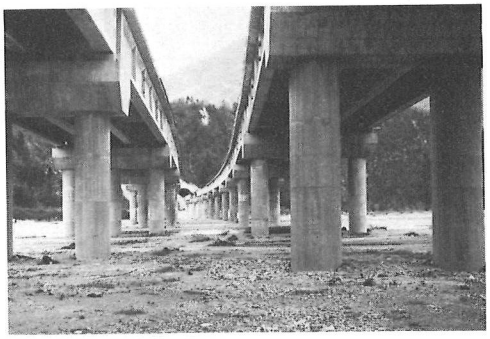


132.

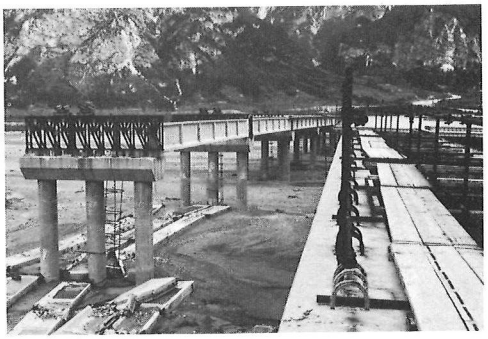


132a.

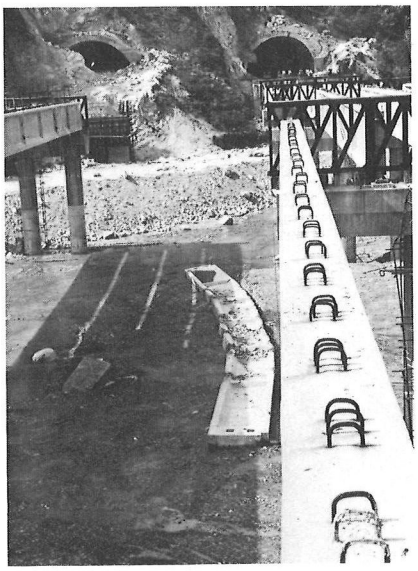
Bridge no. 22, Tagliamento II, km 33,214. This bridge was designed according to legislation for seismic areas. It was complete (133), with the exception of the right Udine abutment. Precast beams were lacking on three spans together with the crossbeams on 6 spans, and the slabs on 7 spans (134-139). Several piers were tilted out of alignment. In this structure too there was considerable detachment between base-plates anchored in the dosseret and bearings, and breaking of beam base plates. Some beams of deck nos. 3 and 4 to the left fell; the beams linked by crossbeams and slabs moved, showing that their bearings were not sufficiently strong.



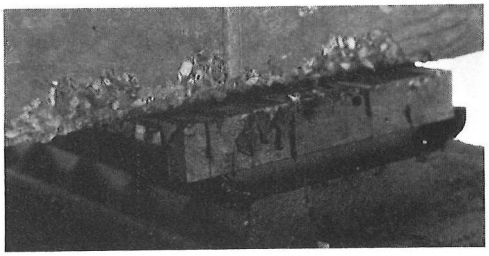
133: Bridge over the Tagliamento.



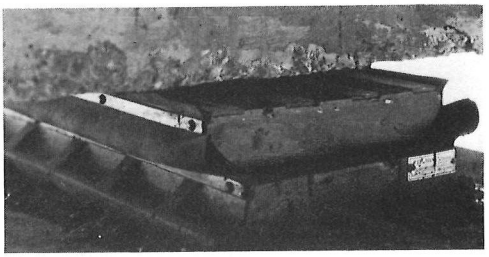
135: Fallen beams.



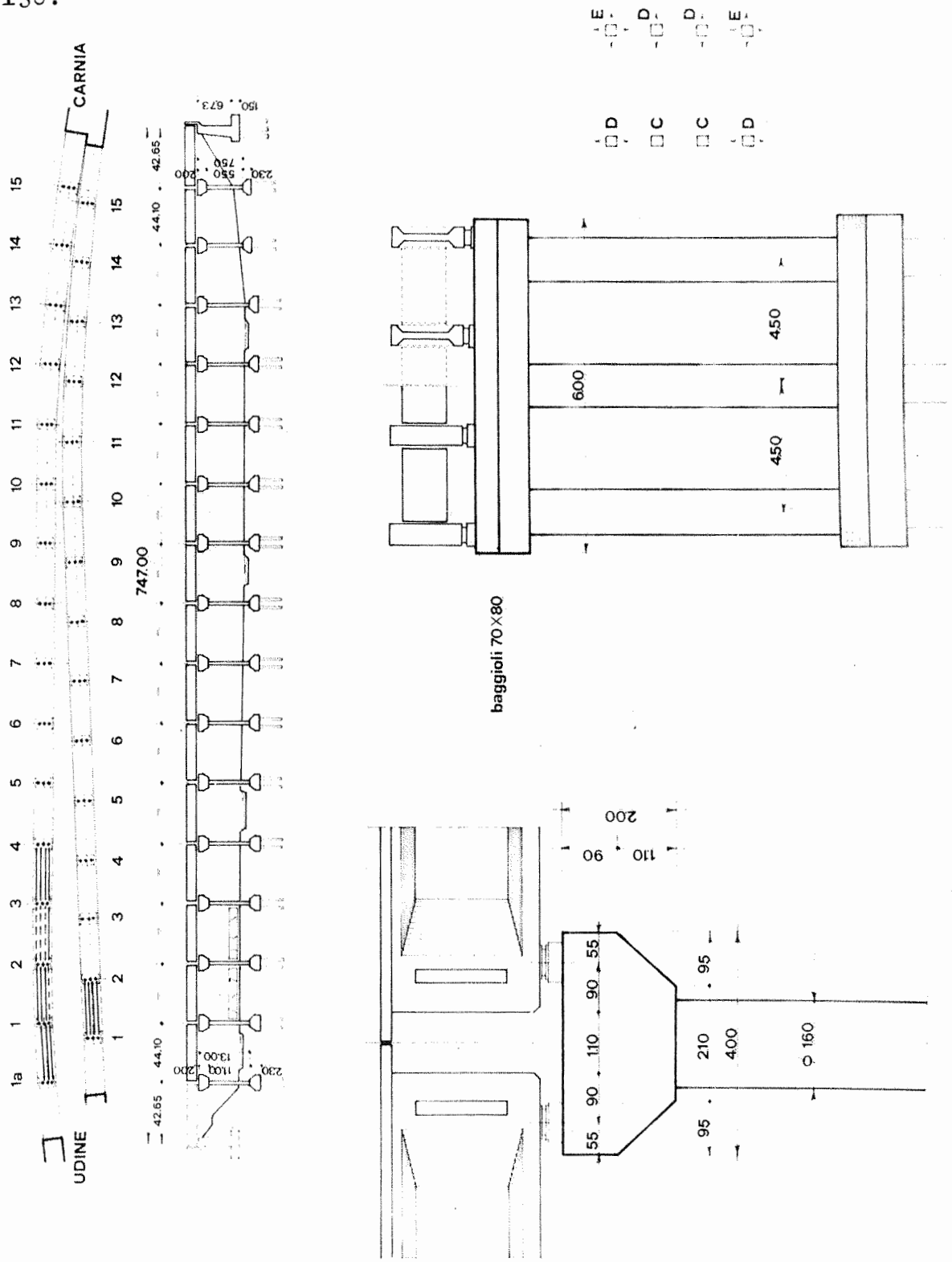
134: General view.



136: Detail of bearing.



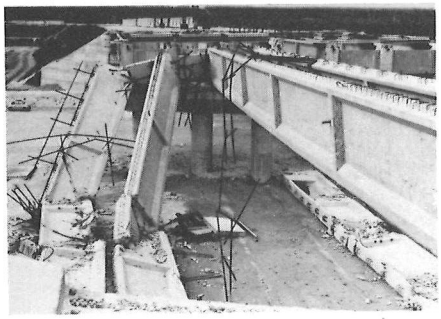
137: Detail of bearing.



Tagliamento II bridge, km. 33,214.

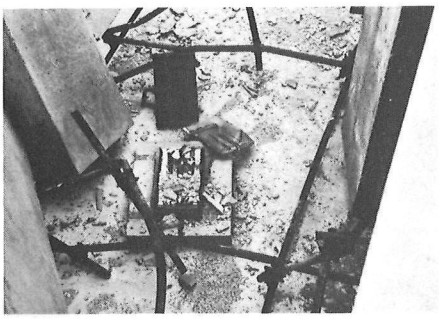
Bridge no. 23 over the Rio Maggiore River, km 34+292,0 This bridge was nearly finished (140): in the first span, beams and crossbeams were complete; in the second span, crossbeams were being assembled. In the span on the Udine side, the bearings underwent irregular displacements. In the Carnia span, 5 beams fell, 4 moved out of position (141-148).

140: Rio Maggiore bridge.

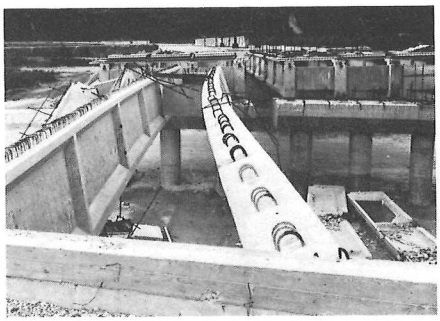


141: General view.





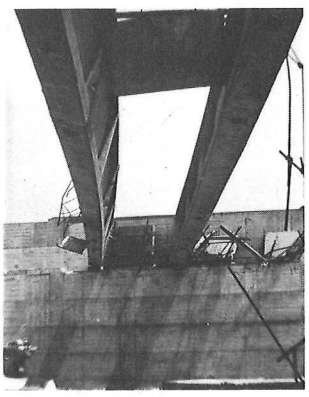
142: Detail of bearing.



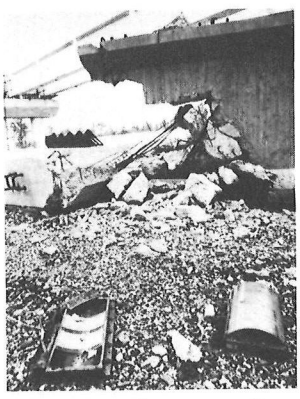
143: General view.



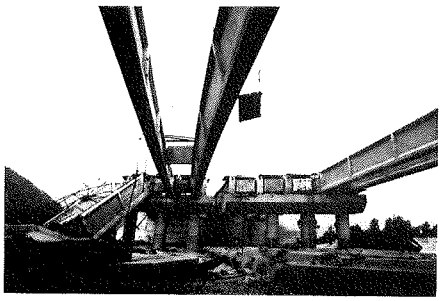
144: Detail of bearing.



145.

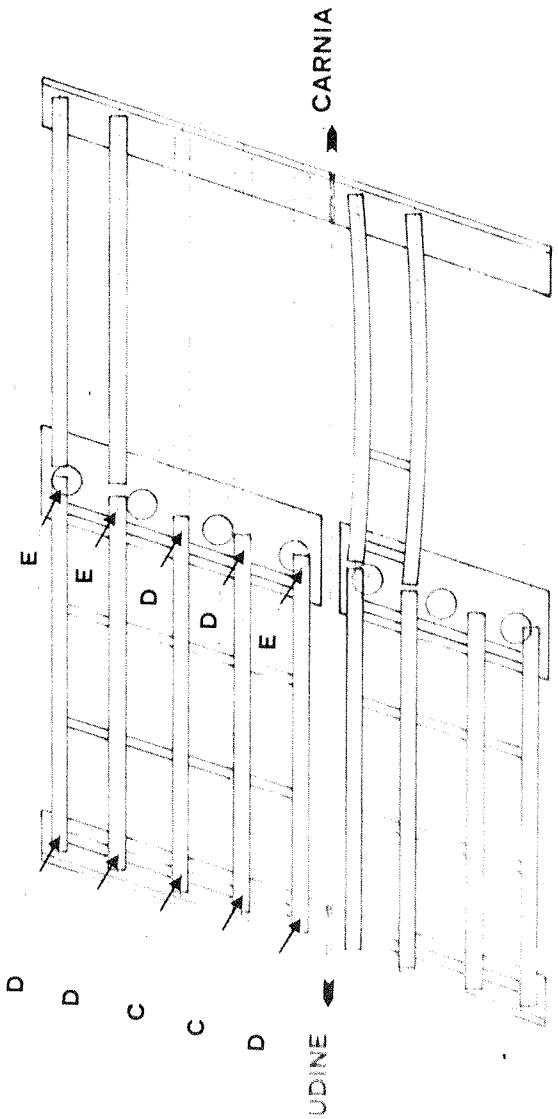


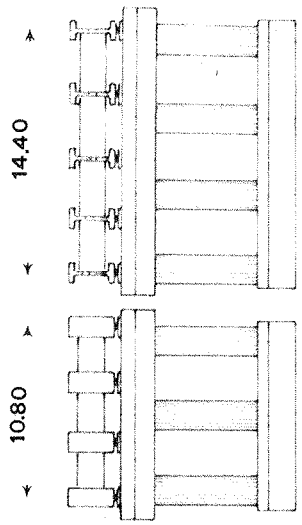
146: Bearing det.

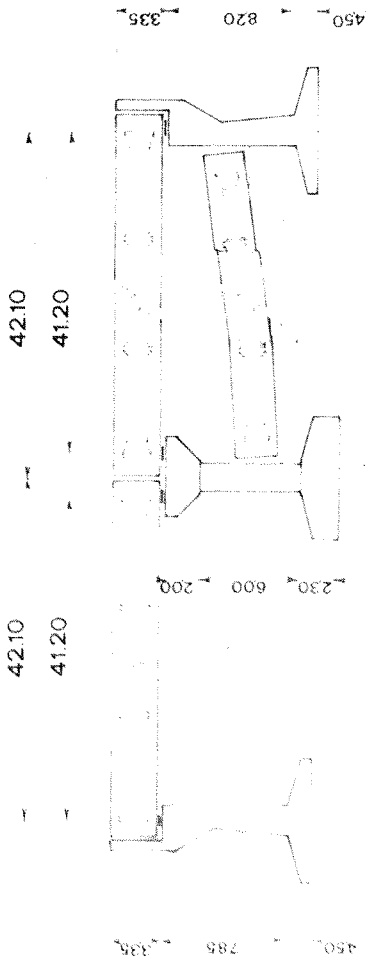


147: Beams, Carnia side.

148: Bridge over the Rio Maggiore River, km. 34 + 292,0







148a.

In conclusion, we can list the following types of damage typical of construction phases or earthquake damage:

a) damage to prefabricated beams not yet used.

b) damage to beams put into position, but without crossbeams or deck.

c) damage to foundation piles.

d) damage to isolated piers or to the frame without girders.

e) damage to continuous slabs on several bearings.

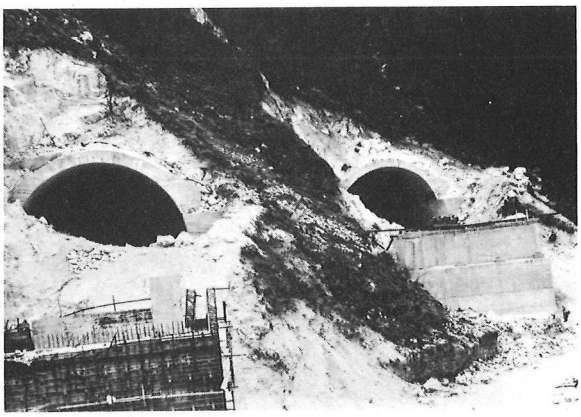
f) damage to simply supported slabs.

g) damage to complete structures, built according to anti-earthquake regulations. In effect, the behaviour of such bridges does not appear satisfactory, both as regards the bearings, and the supporting structures.

150: Tunnel exit, Carnia side.



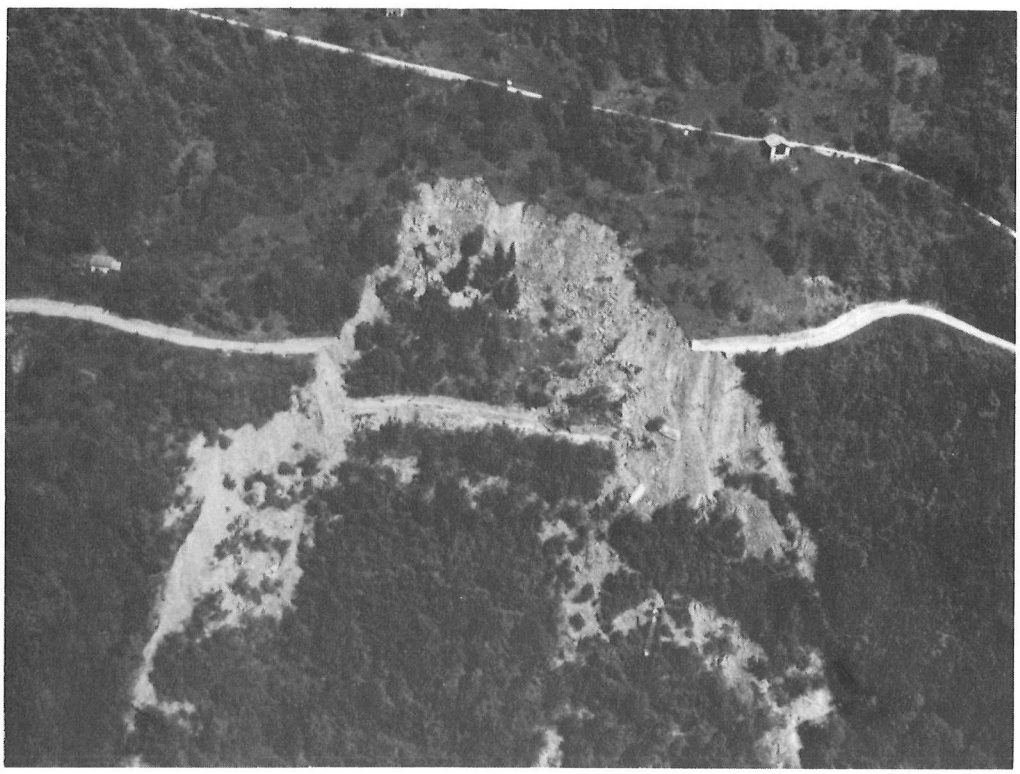
149: Tunnel exit, towards the Tagliamento II.



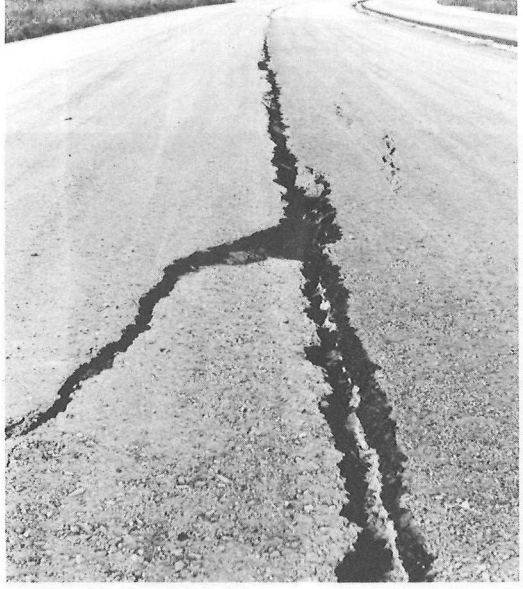
6. Roads and supporting masonry

The roads behaved well. In fact, the greatest damage is to be attributed to landslides (151) and falling blocks on to the road surface (157-160), damaging it. The falling blocks, sometimes enormous, often interrupted traffic. However, we should also note the formation of large lesions in even recently constructed roads (152-155). The supporting masonry, made in reinforced concrete, behaved well; those parts in stone without mortar sometimes showed large-scale settling (156). However, most of these are very old and often in bad condition. Tunnels behaved well (139-140), if lined with reinforced concrete. The unlined ones often let large blocks fall on to the road surface.

Padova, 15.11.1976.



151: Provincial road towards Paularo.



152: Lesions to road surface.



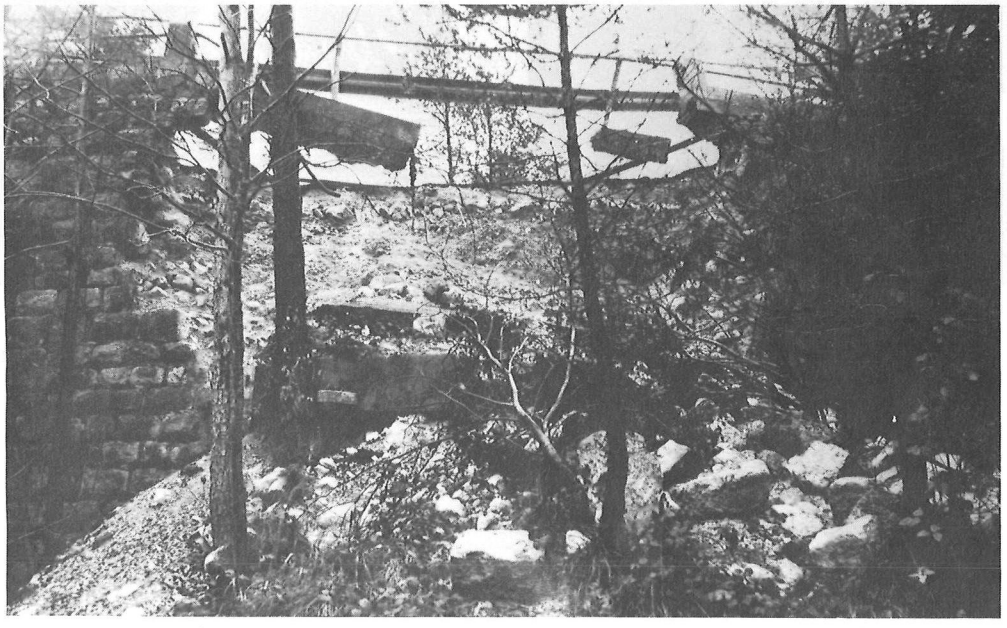
153.



154.



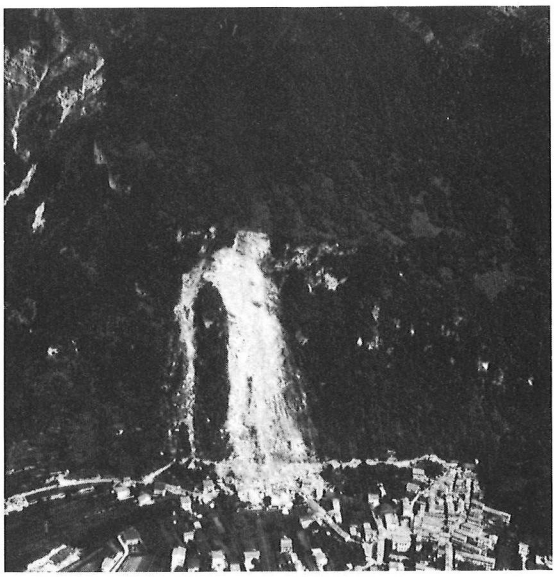
155.



156.

157: Landslide at Braulins.

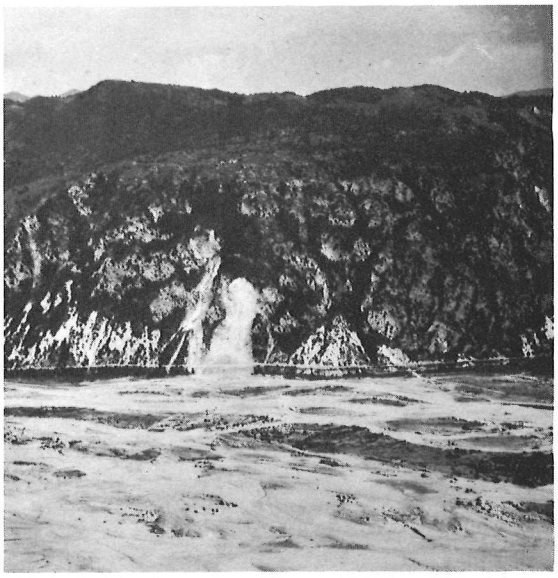




158: Landslide at Braulins.



159: Road to Paularo.



160: Bordano-Trasaghis road, under Mt. S. Simeone.

EARTHQUAKE IN FRIULI (ITALY) - 1976
DAMAGE TO HISTORICAL MONUMENTS AND OTHER BUILDINGS OF ARTISTIC INTEREST

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1. PREFACE

The purpose of this work is to provide the reader with a wide and commented collection of photographs taken in the area hit by earthquakes from May 6 1976 (date of the first shock) till now. This rich documentation aims at providing a synthesis of the phenomena which determined the behaviour of different buildings under the influence of the earthquake. The collection also makes it possible to show the seismic weakness of the damaged buildings, underlining the importance of the problem of prevention or, better, of preventive reduction of the so-called "seismic risk". It is necessary to note how this problem does not concern only any artistic patrimony but also involves whole buildings. In fact, we consider here the need to apply certain criteria, resulting from this calamity as from any other grave event, to the execution of any restoration or rebuilding, wherever it occurs.

2. OLD TOWN CENTRES

2.1. Introduction

The four most violent shocks of the Friuli earthquakes[1] and the innumerable earth tremors of lesser violence which occurred after the first shock and which are by no means yet exhausted[2], have caused serious damage to a large part of Northern Italy, old in history and the seat of flourishing past civilizations. Together with many important architectural monuments, an enormous urban and environmental heritage, directly utilized and modelled by the peoples of the region over the centuries, had been handed down to us. It was composed of small towns and the many rural townships surrounding them, still almost unchanged in identity and still exercising a vital function over the whole territory, but extremely vulnerable in physical structure. This "minor" building and the most important architectural monuments, of which the former was essentially the supporting urban structure, composed together an integral unitary common good, of enormous cultural value and harmonious functionality.

As well as the great loss of human lives (officially 996, of which 988 on May 6, and 8 on September 15) and the enormous economic damage (unofficial estimates, only partially updated, speak in terms of 6,000 billion lire), caused by the earth-

(1) May 6 1976: first shock, degree 9 on the Mercalli-Richter scale; September 11, second shock, degree 8; September 15, third shock, 5.15 a.m., degree 9; fourth shock, 11.20 a.m. degree 9.

(2) The Osservatorio Geofisico Sperimentale di Trieste, till

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quakes, there is also the bitter and probably irremediable loss of a great historical and artistic heritage of Italian culture.

In this chapter we present a preliminary overall panorama of the damage suffered by the historical town centres of Friuli. Before passing to a description of the earthquake damage, we give some general information on the region.

2.2. Brief geographic and geological data of the territory containing the worst hit towns and villages

The region of Italy hit by the earthquakes is centred in the valley of the Tagliamento River, running from its source in the pre-Alpine chain (Carnian Alps to the north-west, and Julian Alps to the north-east), to the morainal amphitheatre in the south (approximately on a line through the towns of Sequals, Pinzano, Buia and Tarcento). It is an enormous glacial basin, excavated during the pre-Alpine dolomitic-calcareous formations of the Triassic period, and later filled in with alluvial deposits.

From the geotectonic point of view, the territory is composed of different geological units (Carnian Alps, Julia Alps and the Friuli plain), intersected by a system of faults and shear planes (among which the Periadriatic fault line) crossing it from east to west, and separating the mountainous area

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today (30/11/76) has registered a total of 359 shocks of intensity equal to or greater than degree 4 on the Mercalli-Richter scale.



2.1. The historical town centre of San Daniele, cultural and linguistic centre of Friuli, after the May 6 shock.

of older rocks (Carnian-period dolomies) from the foothill area of more recent formation (younger dolomies, limestones, sandy limestones, and marly limestones).

The attached map shows briefly the litho-petrographic features of the region. It also shows the area of the municipalities declared "disaster areas"[3], and the position of the larger towns. Mt. S. Simeone, the presumed epicentral zone of the earthquakes, is also indicated.

2.3. Notes on the history and art of Friuli

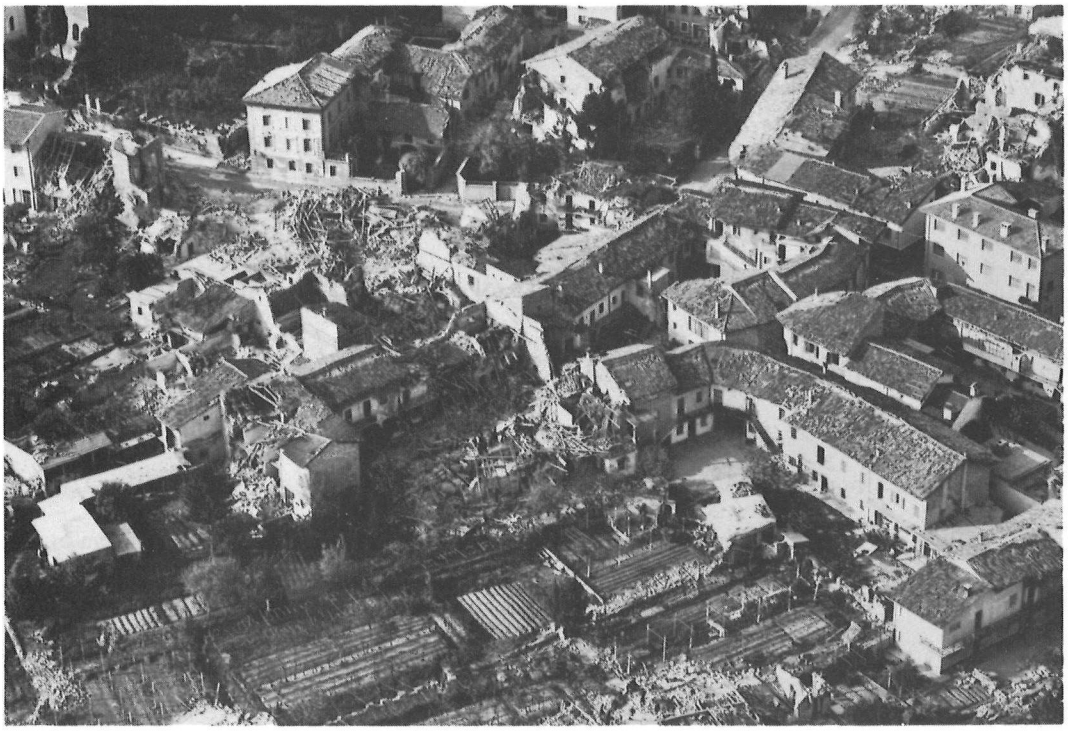
The origin of many Friuli settlements goes back to pre-Roman times, populated by peoples of probable Celtic or Celtic-Carnian stock. The Romans occupied the region around 50-40 B.C. (150 years after the foundation of the large colony of Aquileia in 181 B.C.), building the first roads across the Alps and establishing vanguard colonies.

The consular road of Julia-Augusta linking Aquileia to the north through the present Monte Croce Carnico pass, and the foundation of the Forum Julium Carnicum (Zuglio), are the most important features of Roman civilization: many remains have come down to us: epigraphs, reliefs, funeral urns, and traces of road paving in Gemona, Ospedaletto and Artegna; ruins of the forum, villas and temples at Zuglio.

With the fall of the Roman Empire, the Friuli region, situated as it was between east and west, found itself directly

(3) Regional Decree of May 20 1976: regional delimitation of: a) devastated municipalities; b) seriously damaged municipalities; c) damaged municipalities.





2.2. 2.3. Heaps of crumbled rubble and splinters of wood in two old villages, showing the great fragility of such buildings to earthquakes.

exposed to barbarian invasions, and some communities, such as Clauzetto and Anduins on the slopes of the Carnian foothills, came into being as refuses from invasion.

During the Longobard domination (568-780 A.D.) the townships of Gemona, Osoppo, Artegna, Nimis, Ragogna and Forgaria[4] grew up. Important artistic remains of this time are still preserved at Cividale del Friuli, capital of the Longobard duchy.

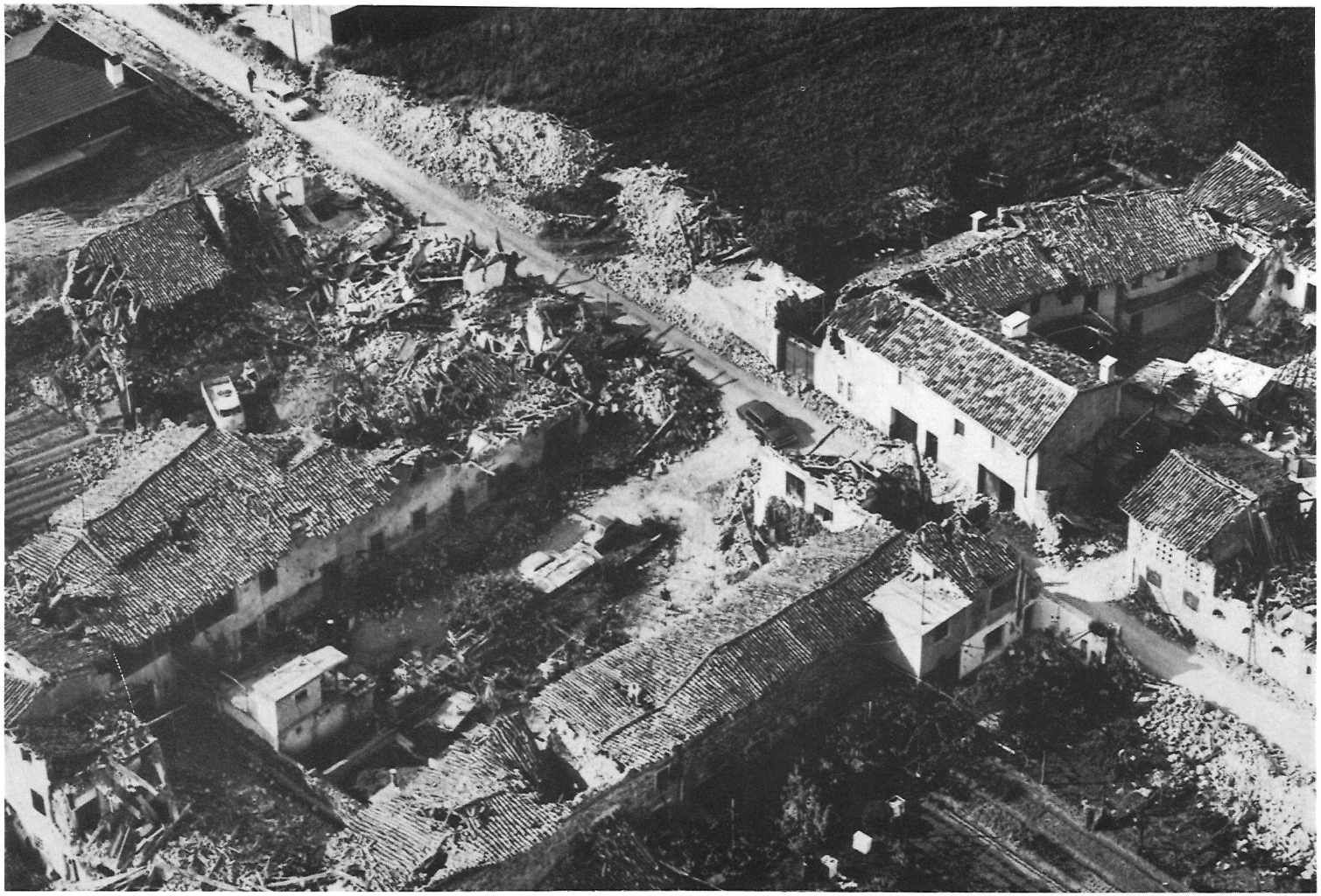
During the Carolingian domination, the region developed a feudal system, and its links with the Germanic world grew (10th-century Otto Empire). It was during this period that building began on the most important castles of the region - widespread defensive strongholds for control over the strategic passes over the pre-Alpine mountains and over the morainal plateaus joined to the northern Adriatic plain.

Economic and political development for the Friuli peoples began under the ecclesiastical domination of the Patriachs of Aquileia (1077-1418).

The larger towns became bigger and richer, principally due to commerce and Alpine pass traffic, reaching a type of communal government which allowed them a certain degree of administrative autonomy.

The new town of Venzone, situated in the narrow Tagliamento valley at the point where the river descends from the

(4) Quoted in "Storia dei Longobardi" by Paolo Diacono, a historian-poet who lived in the second half of the 8th century, or by popular tradition.



2.4. Destruction of an old rural township.

mountains, was in such a position that all north-south traffic passed through it. Its urban and economic development was so rapid that it soon began to contend with the largest town in the area, Gemona.

Art flourished, particularly in Gemona and Venzone: the Cathedrals of Gemona and Venzone, the Walls of Venzone, and its Town Hall, and the large palazzi in Gemona are (or were) of particular architectural and artistic interest, since they illustrated the efforts of humble local artists - architects, painters and sculptors - to create their own expressive language, freed from old models and at the same time not yet part of the new trends.

Further architectural and urban development of the larger towns (although less important than during the preceding three centuries) and the poverty and regression of the rural villages, characterized the period of Friuli government by the Republic of Venice (1418-1797).

However, as regards the artistic heritage, the whole area benefitted by Renaissance art and Venetian baroque. Many Friuli and Venetian artists left numerous important works in a great number of towns, villages and boroughs. The innumerable little votive churches, both in towns and in the country, little masterpieces of architecture, enriched with important frescoes and paintings, so characteristic of the Friuli environment, date back to this period.





2.5. 2.6. Massive damage to an isolated hill community. May 6 earthquake.

2.4. Type and state of preservation of the urban building of the historical town centres

The Friuli historical settlements are distributed over the territory in many small centres, and are self-sufficient units from the viewpoint of economic sustenance. They gradually came into being, by means of a process of aggregations around already-existing nuclei (even older traces of habitation, main lines of communication, public squares and churches).

Until the last century, dependence on the geography and climate of the area, difficulties in communication, the poverty of the ordinary population except in the few larger towns set along main roads, all contributed towards the evolution of "minor" towns, with a slow rhythm of life and harmony with the surrounding natural environment.

The elements making up the environment of the town centres were single buildings of one or two storeys and attics and whole blocks. Especially in the rural towns and in smaller boroughs, isolated dwellings hardly existed. On the contrary, dwellings were predominantly aggregated round communal open spaces, with buildings joined together round a courtyard. In this sense, many of the villages destroyed by the recent earthquakes were interesting urban structures, closely dependent on the natural configuration of the ground on which they were built. (See photographs 2.5 and 2.6; also 2.47 and 2.49).

Monoaxial, linear or echelon structures were common in villages such as Forgaria, Artegna and Pinzano, arranged along a generatrix composed of the main street passing through the entire borough, with terraced houses on either side, (2.64,



2.7. Difference in behaviour between old houses and those of recent constructive technology.

2.69, and 2.72). Radial urban structures which developed around a single point of aggregation can also be seen in the old boroughs of Buia, such as Tomba, Avilla and Urbignacco. Spiral sequences can be seen in small hill-towns, such as the nucleus of houses making up the little borough of Monte di Buia (2.47, 2.51).

Lastly, terraced structures were typical of the towns established on hill-sides, such as Gemona and Vito d'Asio in which the principal roads, parallel to the contour lines, were linked to a close network of footpaths along the steepest parts (2.25 and 2.74).

It is interesting to see how in some cases it was this very structural continuity in a group of buildings which allowed everything to resist the stress of the earthquakes better, cases of bad collapse being rare.

As regards construction techniques, these were very simple, using local materials easily found in a natural state nearby.

External and internal walls were composed of stone masonry. In the northern part of Friuli and in the mountainous areas, more or less well-dressed limestone blocks were used. The southern part of the region, with its abundant morainal and more recent alluvial deposits, used smooth rounded pebblestones. Mortars were normally poor in lime (in some cases it was totally lacking), and made up of clayey sands.

The floors and roofs (generally with two pitches) consisted of a monodimensional roof frame of wooden beams fixed into or resting on the masonry walls, without any particular

anchoring system, strengthening of the joints or topmost strengthening of the walls or reinforcing ties between walls and floors[5].

For the more important buildings - the houses of prominent citizens, public and religious buildings - construction techniques were better, particularly as regards the use of dressed stone walls and mortar richer in lime, the insertion of large corner-stones of harder material, and the use of monolithic columns, arches and vaults, and metal ties. But this was not enough to make these buildings safe against earthquakes. In general, therefore, owing to its very constitution, the whole urban structure was abominably ill-suited to offer any kind of resistance to horizontal forces (2.2 and 2.3).

Finally, if we consider the state of preservation of these Friuli buildings at the time of the recent series of earthquakes, we find another serious weakness. The houses were built between 150 and 300 years ago, partly in substitution of preceding buildings which had been destroyed by earthquakes, fire, war, or simply old age, and partly as the result of new expansion. Statistics are not available on these, but the state of semi-dilapidation of the oldest houses, above all in the smaller villages and those in the hill areas, was already evident to all. The cause of this abandon lies chiefly in two factors: emigration from the area towards the larger towns, and urbanization.

(5) In some buildings of the last two centuries, we can find brick floors with fan-structures and beams in wood and iron, whose local behaviour towards the earthquakes appeared generally good.

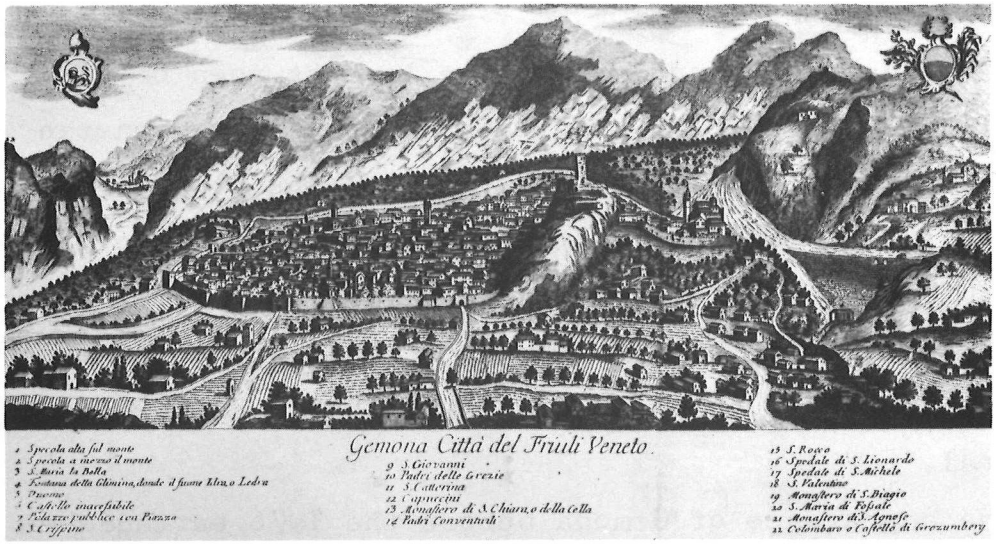
This situation gave rise to a widespread state of structural precariousness in the remaining buildings characterized by: leaking roofs; lack of connection and wear and tear of the principal and secondary roof and floor frames, principally at their junction with the walls; broken or fissured walls attacked by damp; crumbling external plastering.

On the other hand, various restorations and structural strengthening, sometimes extended to the whole building and often very expensive, have recently been made to national monuments or to the dwelling-places of the more important town centres not yet included in the area declared seismic as, for example, Gemona, Osoppo, Artegna and Venzone. However, it must be noted that these works hardly ever followed restructuring planning aimed at giving the buildings in question small but overall resistance against the dangerous horizontal actions involved in an earthquake.

2.5. Description of general damage suffered by the old town centres

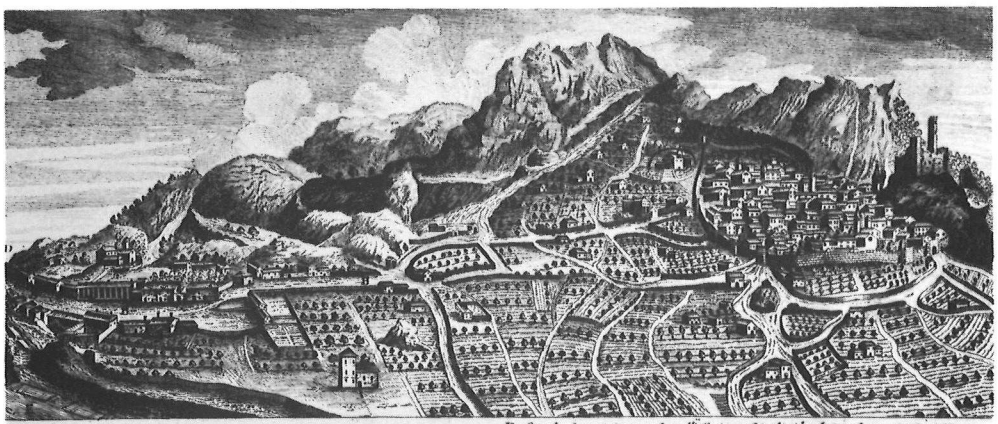
We examine here the disastrous effects of the earthquakes on the urban structures of the old town centres of Friuli, using aerial photographs and codified location maps.

We take into consideration here only a certain number of towns and villages which were among those most badly hit by the earthquakes.



Gemona Città del Friuli Veneto.

1 Spedale alla sud monte
2 Spedale a mezzo il monte
3 S. Maria la Bella
4 Fontana della Glimina, donde il fiume Ulla, o Ledra
5 Duomo
6 Castello inaccessibile
7 Palazzo pubblico con Piazza
8 S. Osspitio
9 S. Giovanni
10 Padri della Grazie
11 S. Catterina
12 Capuccini
13 Monastero di S. Chiara, o della Cella
14 Padri Conventuali
15 S. Rocco
16 Spedale di S. Lionardo
17 Spedale di S. Michele
18 S. Valentino
19 Monastero di S. Biagio
20 S. Maria di Fosale
21 Monastero di S. Agnese
22 Colombaro e Castello di Grezumberg



A. *Gemona.*
B. *Ospitaletto di lei Borgo, dove sono disotterrate rovine, e le tapide Romane.*
C. *Priorato di S. Spirito, o S. Maria de via Carentana.*
D. *Strada che quivi entrando nell'Alpi e poscia dividendosi in due va in Carintia, o Norico da un lato, e dall'altro per la Carnia, e Cadore va nella Toglia, Tirole, Svizzeri &.*
E. *Fiume Tagamento con sue Rotte, o Ripari.*

2.8. 2.9. Gemona in the 18th century.

2.5.1. Gemona

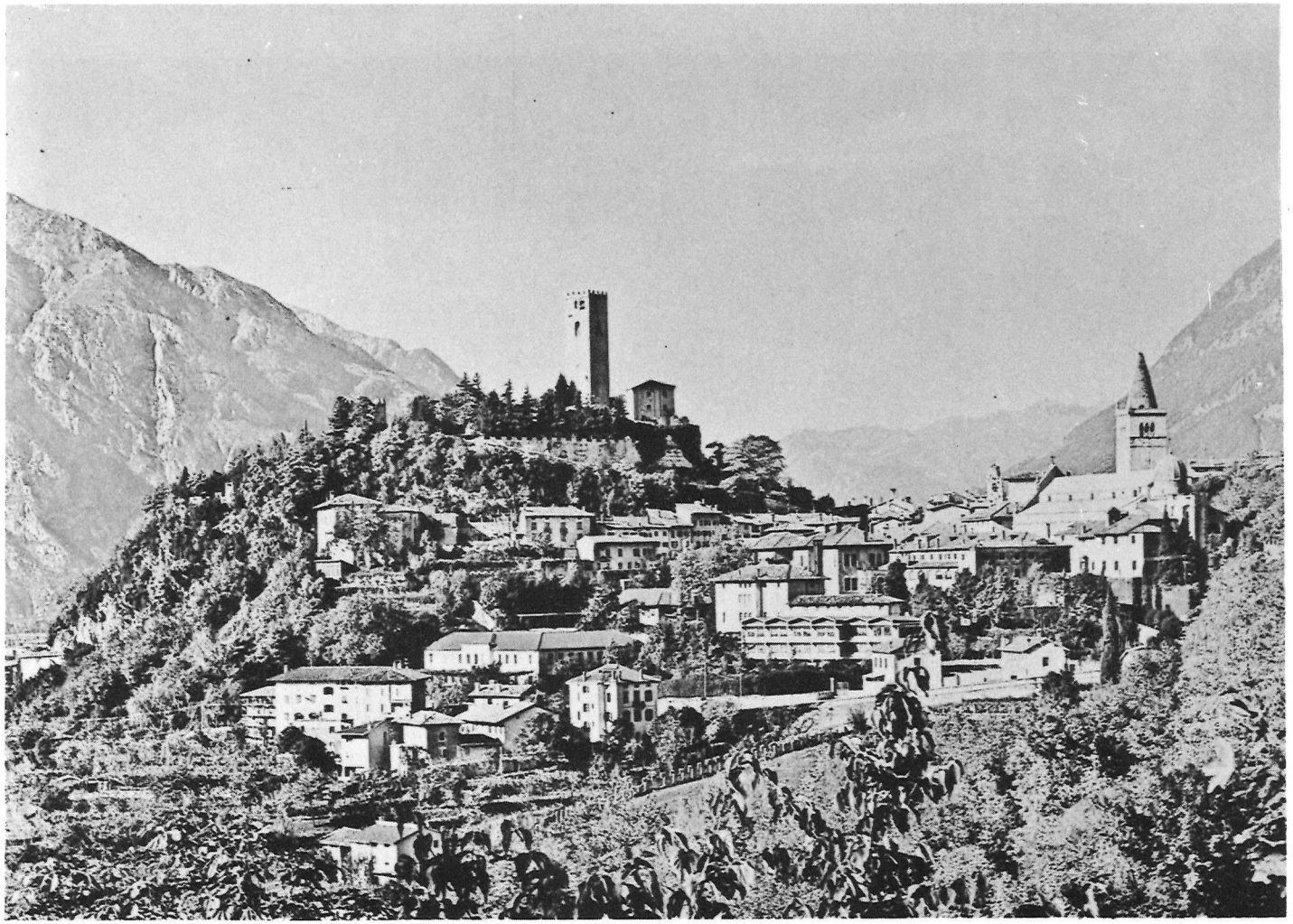
The oldest part of Gemona, the largest town of the central-northern part of Friuli, grew up half-way on the deposit cone of the Vegliato torrent, at the foot of Mt. Chiampon and Mt. Glémina, dominating the Tagliamento valley.

2.8 and 2.9 show two 18th century prints of the town as it then was, encircled by walls with its castle on the rocky spur in the centre, and the complex of buildings around the Cathedral.

The appearance of Gemona before the 1976 earthquakes is seen in 2.10, in a view of the southern borough of the town, under the castle and the cathedral.

Its appearance immediately after the May 6 shock can be seen in 2.11, taken from the south-west. Immediately obvious are: the collapse of the castle (one spur of the tower still remains), the collapse of the bell-tower and that of the south nave of the cathedral. The foreground shows extensive damage to recently built areas; and a closer examination gives an idea of the terrible wound inflicted on the old town centre.

2.12 to 2.17 show a closer overall view of the general condition of the town after the first shock of May 6 when work was immediately begun on going through the ruins to find those buried alive and to bring out the many dead. It is worth noting that the opening of paths for heavy rescue vehicles in the higher parts of Gemona was particularly difficult both owing to the general obstruction caused by the already narrow streets of the town, and because of the quantity of rubble encountered. Many old buildings collapsed completely, becoming formless



2.10. View of Gemona before the May 6 earthquake.

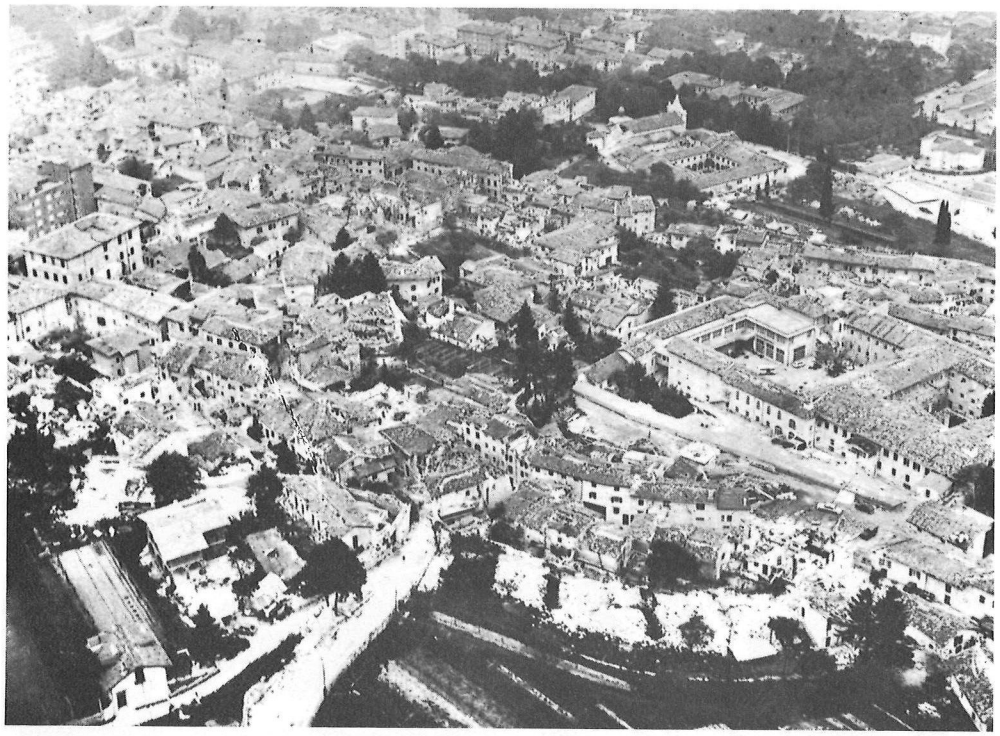


2.11. Gemona after May 6.





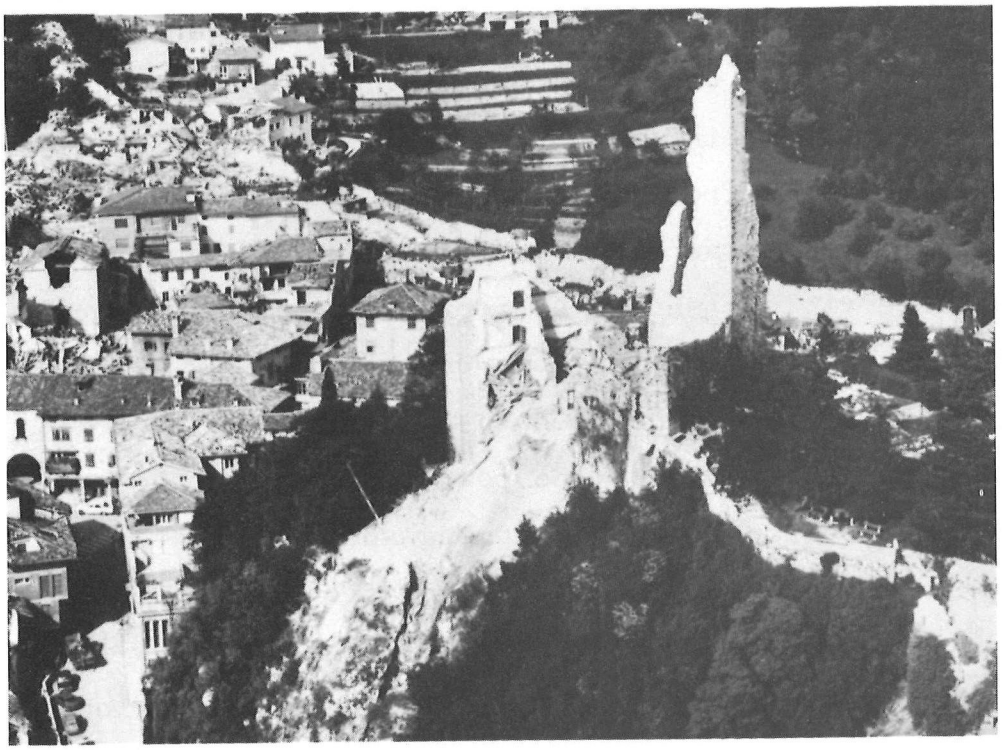
2.12. 2.13. Gemona after May 6.





2.16. 2.17. Gemona after May 6.





2.18. 2.19. The Castle of Gemona, respectively before and after May 6.

heaps of stones and broken beams. Displacement of roofs with the local detachment of the wooden load-bearing frame was a general phenomenon. It is, however, impossible to understand the serious lesions to those buildings "still standing" from these general photographs.

2.18 and 2.19 show the castle of Gemona before and after May 6, respectively from the south-east and the south-west. The degree of crumbling of the rubble of the old fortress is exceptional, and is due to the extremely poor quality of the mortar used in the walls. The destruction of the higher part of Gemona is visible in the background.

Taken near the railway station, 2.20 shows the different degree of resistance to the earthquake offered by the old houses built of masonry and wooden beams, and by the more recent ones constructed of brick masonry and floors with a composite structure of reinforced concrete and bricks. The problem of freeing the streets from obstructions, to allow the passage of rescue and service vehicles, is also noticeable.

The three violent shocks of September completed the almost total destruction of the town centre, already mortally wounded on May 6, as can be seen in 2.21 to 2.24.

2.21 shows the whole of the town centre from the north-east, partially covered by the morning shadow of Mt. Glémina. The exceptional level of general devastation is evident, and details can be seen in 2.22, 2.23 and 2.24. They show that part of the town near the castle and the cathedral, along Via Bini. This street was in a slightly better state after the shocks of September 11 and 15, probably due to the pro-



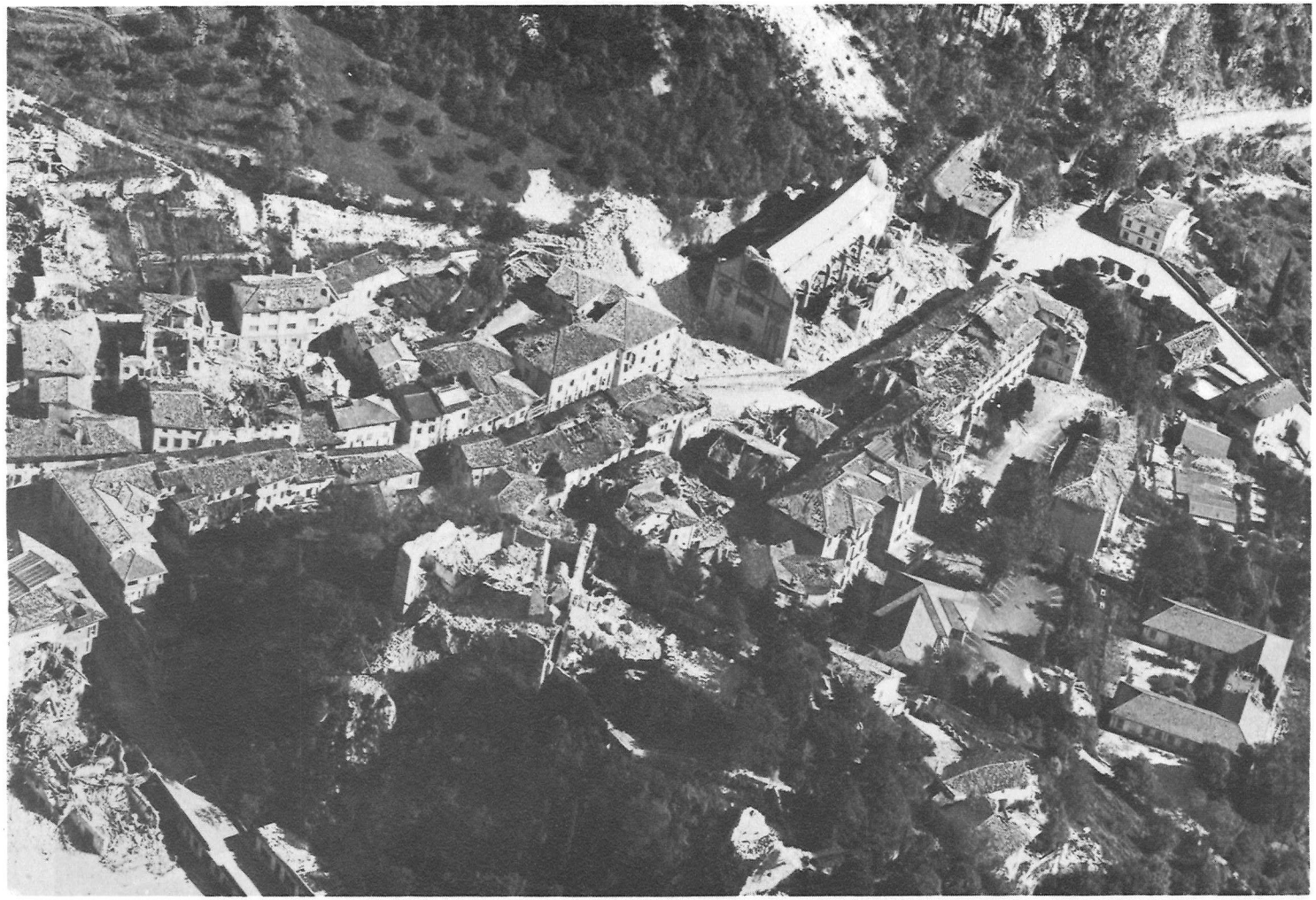
2.20. Gemona, near the railway station, after May 6.



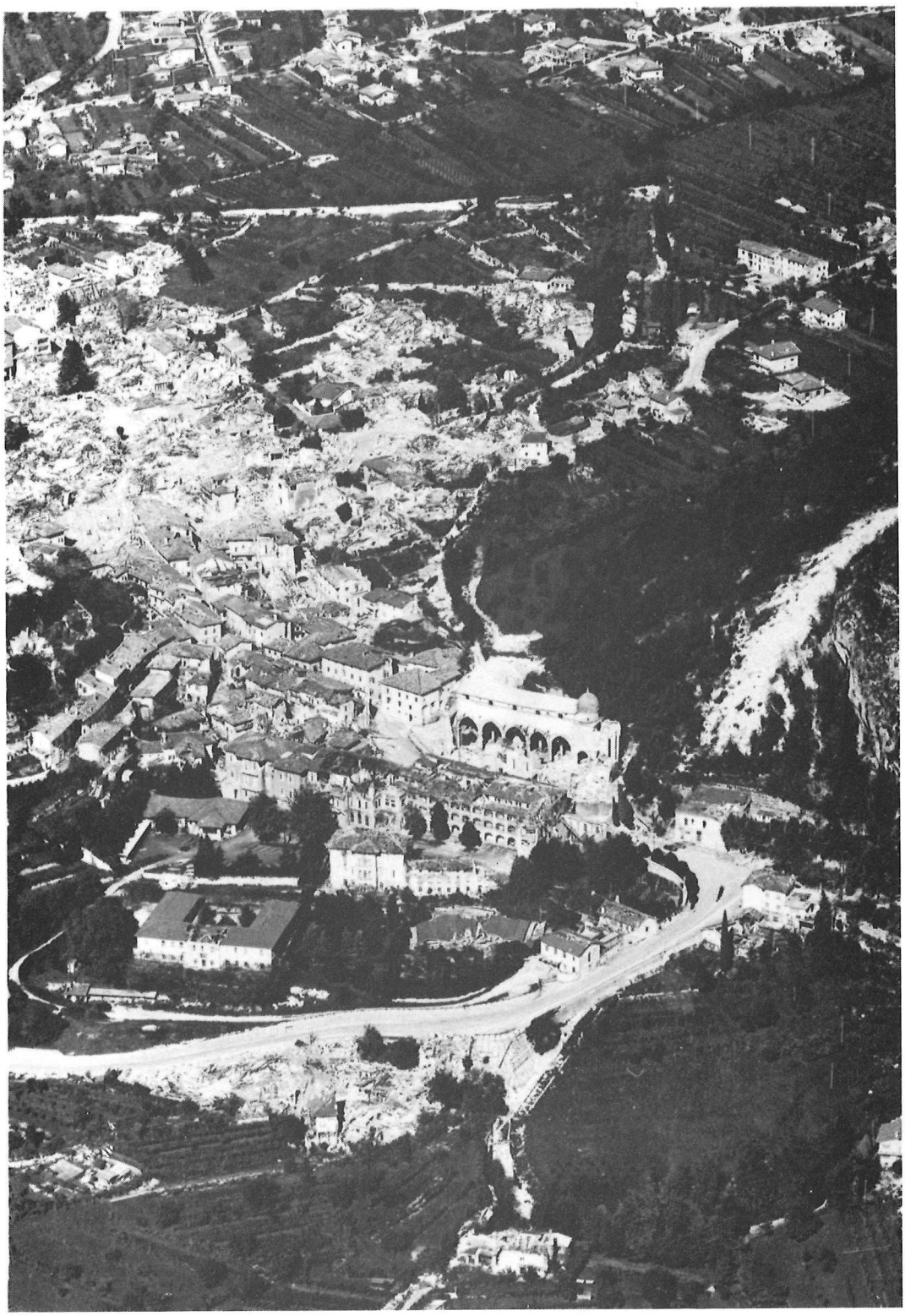
2.21. Gemona, after the September shocks.



2.22. Gemona, after the September shocks.



2.23. Gemona, after the September shocks.



2.24. Gemona, after the September shocks.



2.25. Map of Gemona before the Earthquake.



2.26. Map of Gemona - State of buildings after May 6.



2.27. Map of Gemona - State of buildings after September 15.

tection afforded by the massive rubble-clearing and shoring operations carried out after May 6. The worsened state of the cathedral and the town hall can be seen. Above all, however, 2.22 and 2.24 show the very serious and total loss of identity of Gemona's town centre. The detailed location maps of this area are shown in 2.25 before the earthquakes, in 2.26 after the May 6 shock and, lastly, in 2.27 after the September shocks.

2.5.2. Venzone

The 13th century walled city of Venzone, in the narrow Tagliamento valley between Mt. S. Simeone to the west and Mt. Ledis to the east, was the most complete historical town centre of the area. The original town plan, with its wheel-shaped structure, was admirably preserved, together with many of the oldest buildings including the Medieval cathedral and the irregular hexagon of magnificent city walls with twelve towers and three gates made of pale dressed stone.

2.28 shows a panorama of the walled town; 2.29 a view of the walls, with one of the towers, the moat and outer bank, and part of the other buildings of enormous architectural value.

This unique historical and architectural heritage was seriously damaged on May 6, as 2.30 (from the west) and 2.31 (from the north) show. Many houses have been destroyed; in other points the rubble has already been removed; the walls have collapsed in several places. In spite of all this, the city still maintains its character and identity.



2.28. Venzone, before May 6 1976.



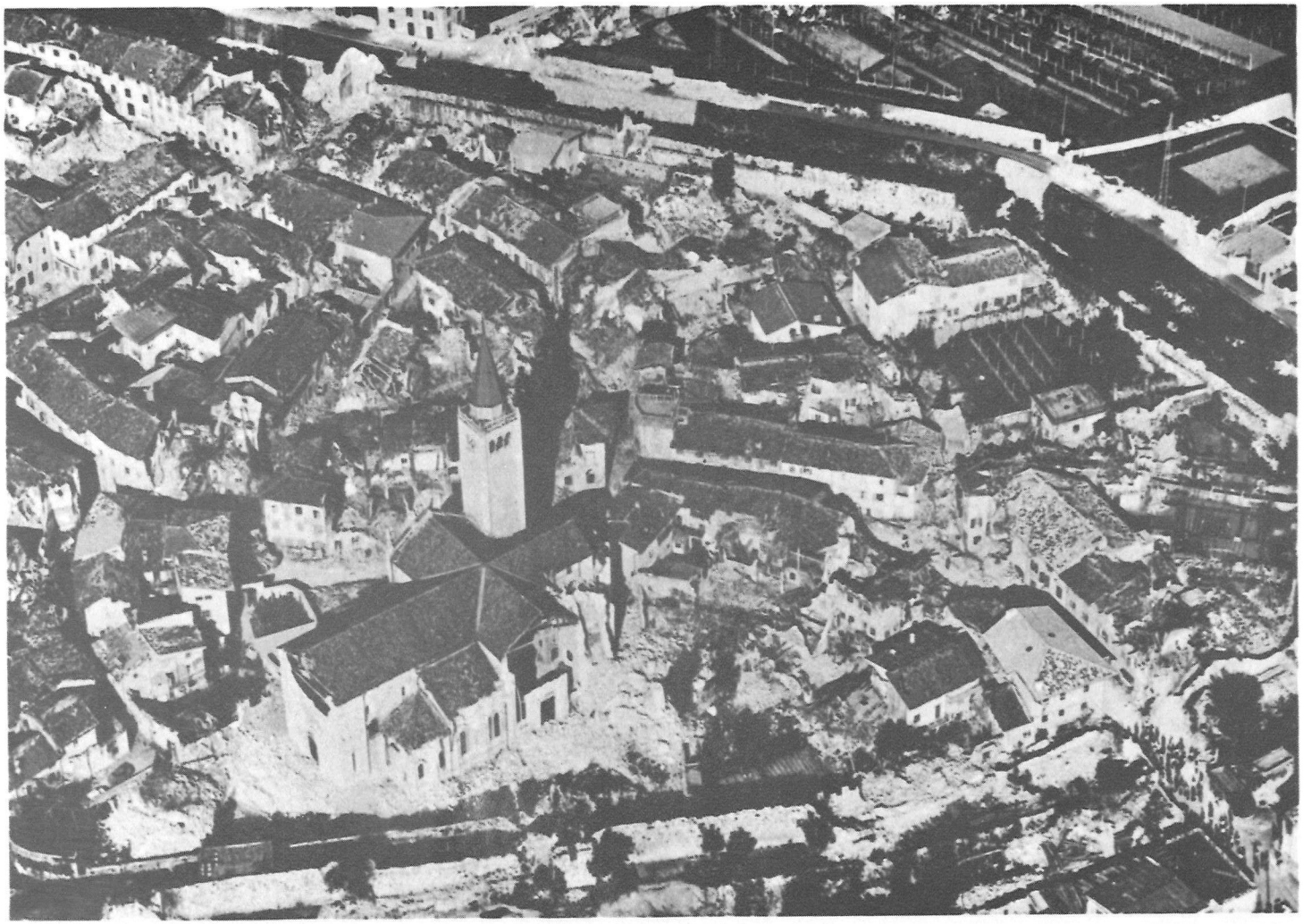
2.29. Venzone. View of the walls and houses before May 6.



2.30. Venzone after May 6.



2.31. Venzone after May 6.



2.32. Venzone and its cathedral after May 6.



2.33. Venzone and its cathedral after September 15.



2.34. Venzone after the September shocks.



2.35. A street in Venzone after the September shocks.

In September, Venzone was almost literally razed to the ground, as can be seen in 2.33, 2.34 and 2.35.

Of the old Medieval and Renaissance houses, only heaps of white rubble remain, almost as if the stones of the buildings had suddenly become separated from each other. 2.32 (taken immediately after the May shock, and 2.33 (after September 15), taken from the same point, allow a direct comparison between the effects of the May shock and the September ones. The latter, however, acted on structures whose global mechanical resistance was already almost exhausted by the first shock. In the foreground, the few remains of the impressive and austere volume of the cathedral emerge from a heap of stones. The partially destroyed town hall can also be seen (on the left in 2.33; in the centre in 2.34). Its little tower also appears in the background of 2.35. The state of the rubble in this last photograph gives a tragic idea of the structural weakness of this type of construction, which, while it was standing, was of exceptional beauty.

The loss of Venzone can be seen in the location maps of 2.36, 2.37 and 2.38.

2.5.3. Osoppo

Also the historical town centre of Osoppo, of great architectural and artistic value (although in a worse state of preservation than the centres of Gemona and Venzone) was violently destroyed on May 6.

The old town lay at the foot of an isolated Eocene hill in the centre of the alluvial plain of the Tagliamento River.



2.36. Map of Venzone before the Earthquake.



2.37. Map of Venzone - State of buildings after May 6.



2.38. Map of Venzone - State of buildings after September 15.



2.39. Osoppo after May 6.



2.40. Osoppo. An advanced phase in the clearing operations for removal of rubble and destroyed houses after May 6.



2.41. Osoppo after May 6.



2.42. Osoppo after May 6.



2.43. Osoppo after May 6.



2.44 Map of Osoppo before the Earthquake.



2.45. Map of Osoppo - State of buildings after May 6.

A famous military fort, transformed long ago into a Medieval Longobard castle, rose on the top of the hill in an extremely advantageous strategic position.

2.39 shows the state of the historical town centre of Osoppo on May 6, immediately after the earthquake. In all, it appears perhaps in a worse state than Gemona or Venzone, owing to the enormous number of cases of total collapse. This situation led the authorities to evacuate that area of the town and demolish all the seriously damaged buildings which remained standing. This operation was already nearing completion (2.40, taken from a point very near the previous photograph), when the September shocks occurred. Their effects were thus not able to cause very much more destruction to the few remaining and unrecognizable buildings.

2.41, 2.42 and 2.43 show in further detail the quality and entity of damage to the fragile structure of the town.

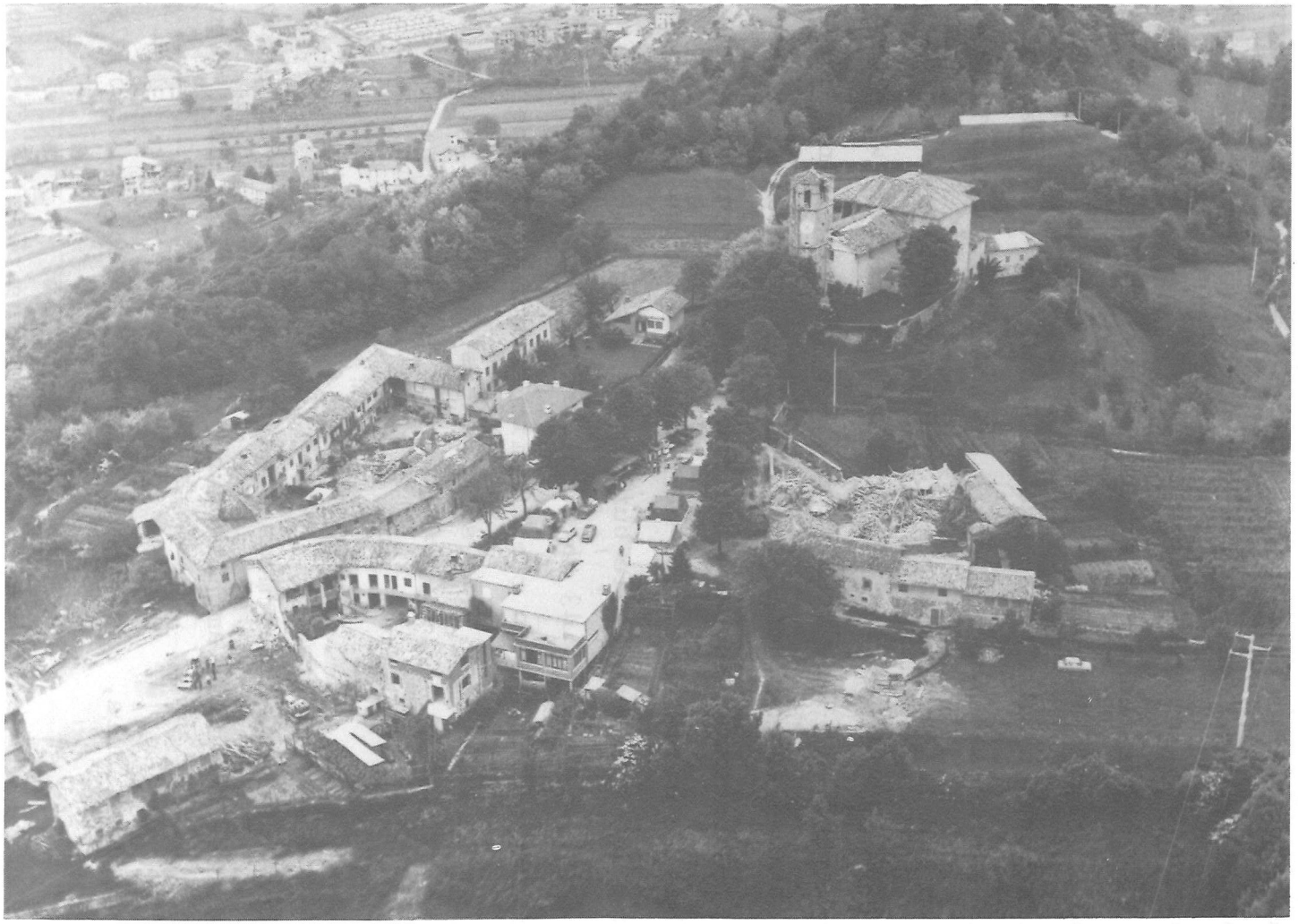
The location map 2.45 shows the state of buildings after May 6.

2.5.4. Buia

Buia covers an enormous area on the northern flank of the hill of the morainal amphitheatre, and contains about twenty boroughs.

Monte di Buia, one of the oldest, is seen in 2.46, 2.47 and 2.48.

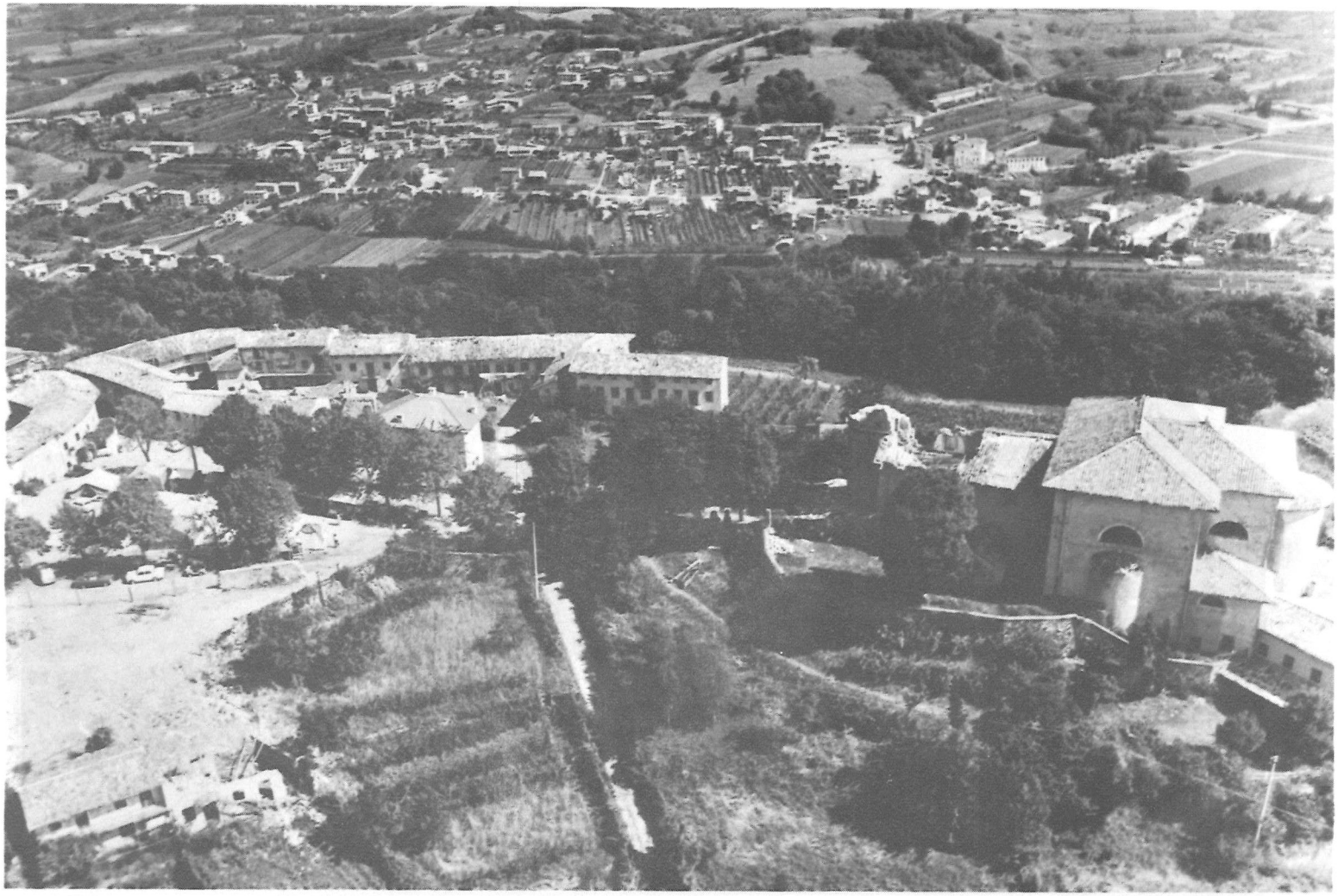
Structural continuity and the winding plan of the two buildings at the edge of the road are interesting elements. It cannot be excluded that this type of structuring gave these



2.46. Monte di Buia after May 6.



2.47. Monte di Buia: detail after May 6.



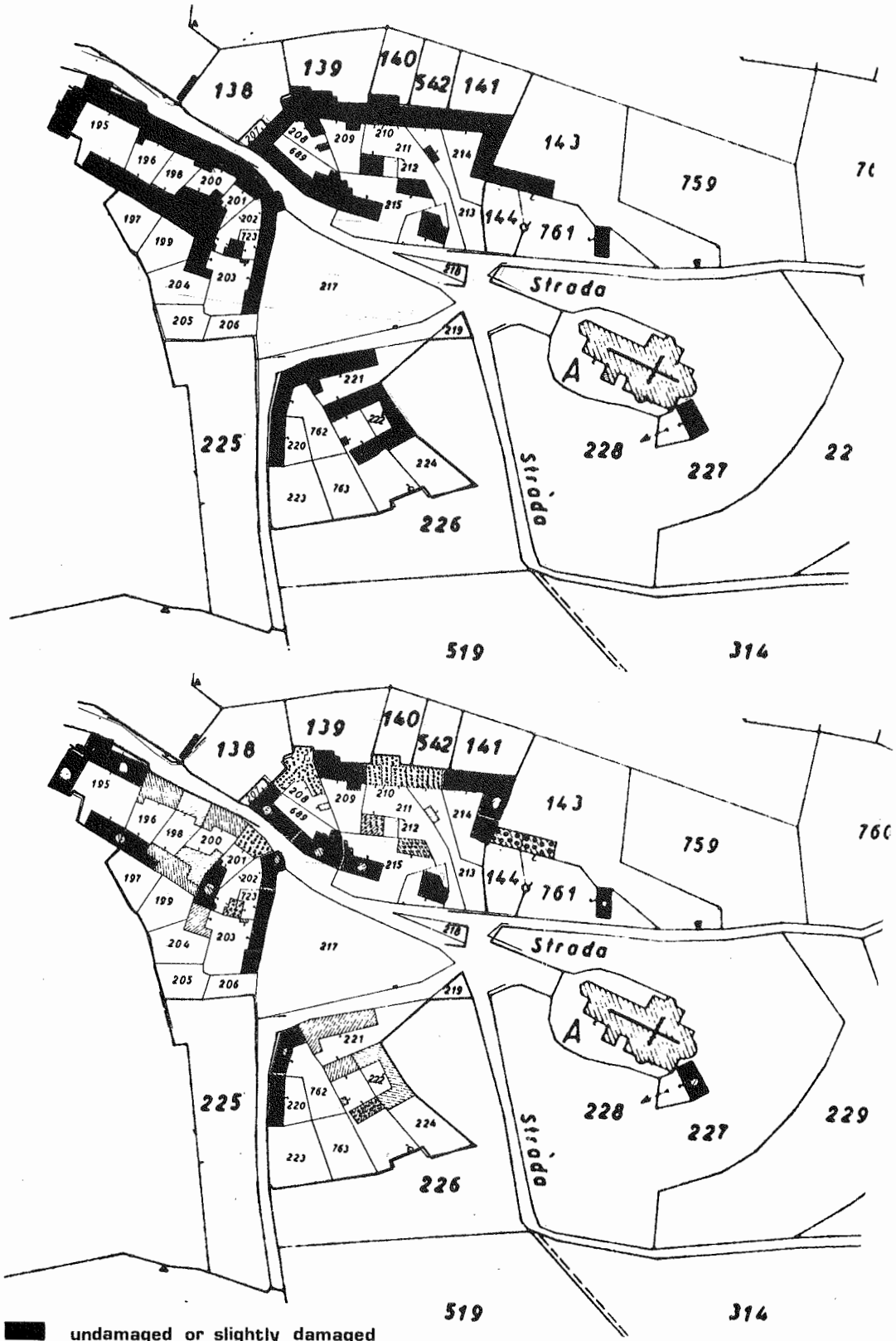
2.48. Monte di Buia: the bell-tower of S. Lorenzo collapsed after May 6.



2.49. One of the boroughs of Buia after May 6.



2.50. The same borough after May 6.



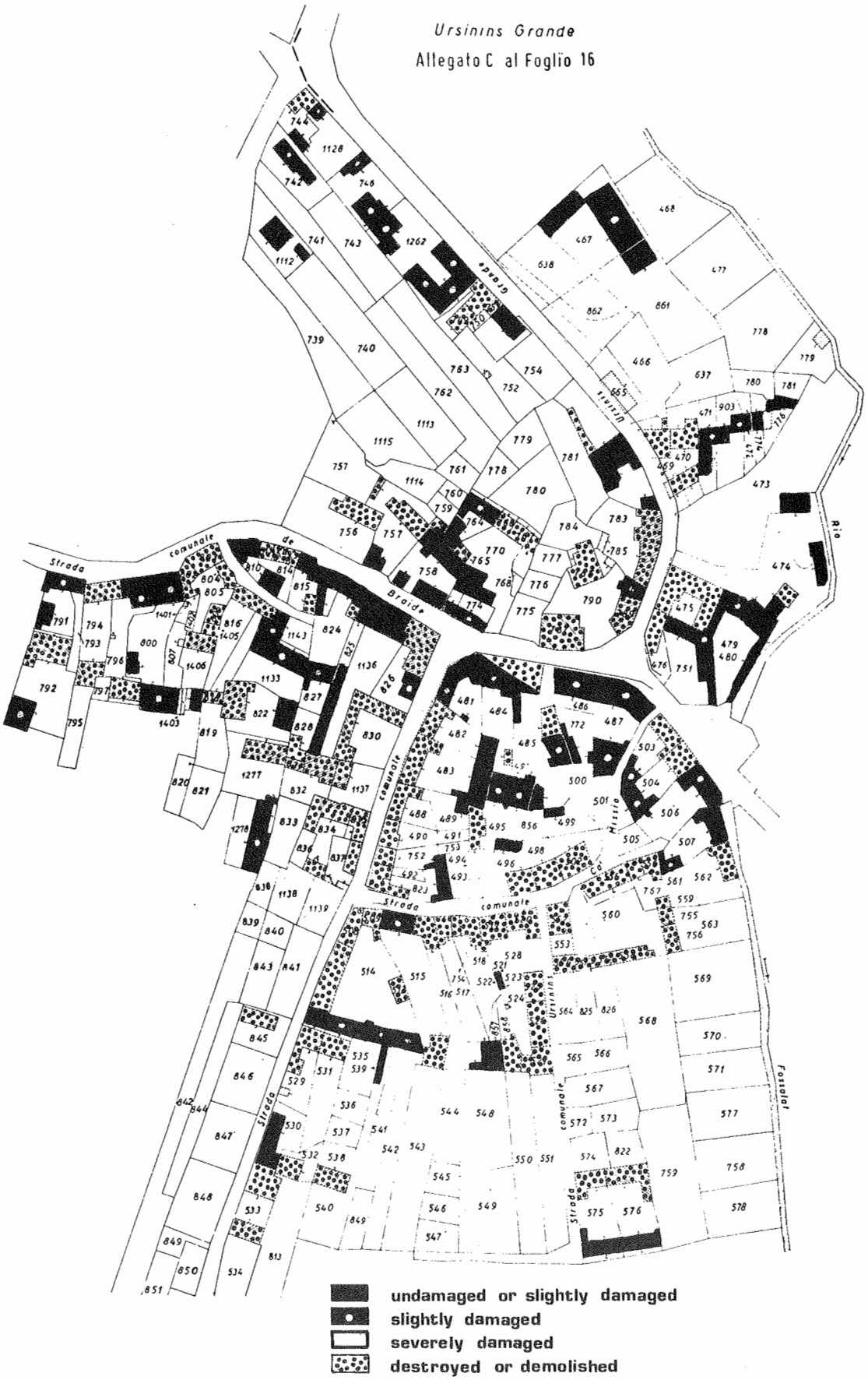
2.51. Map of the borough of Monte di Buia - State of buildings before an after May 6.

undamaged or slightly damaged
slightly damaged
severely damaged
destroyed or demolished

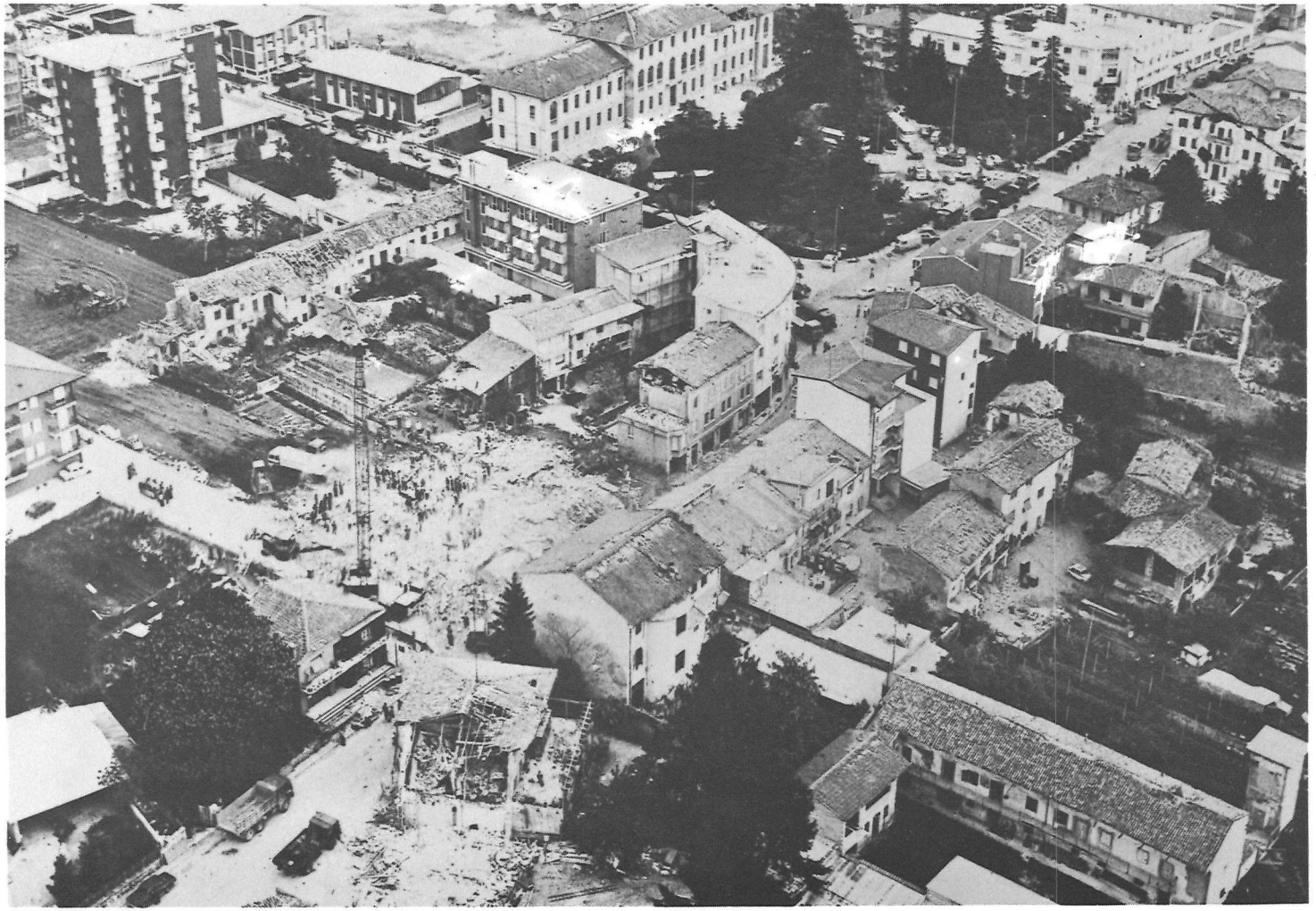
2.52. Map of the Borough of Sopramonte - State of buildings before and after May 6.



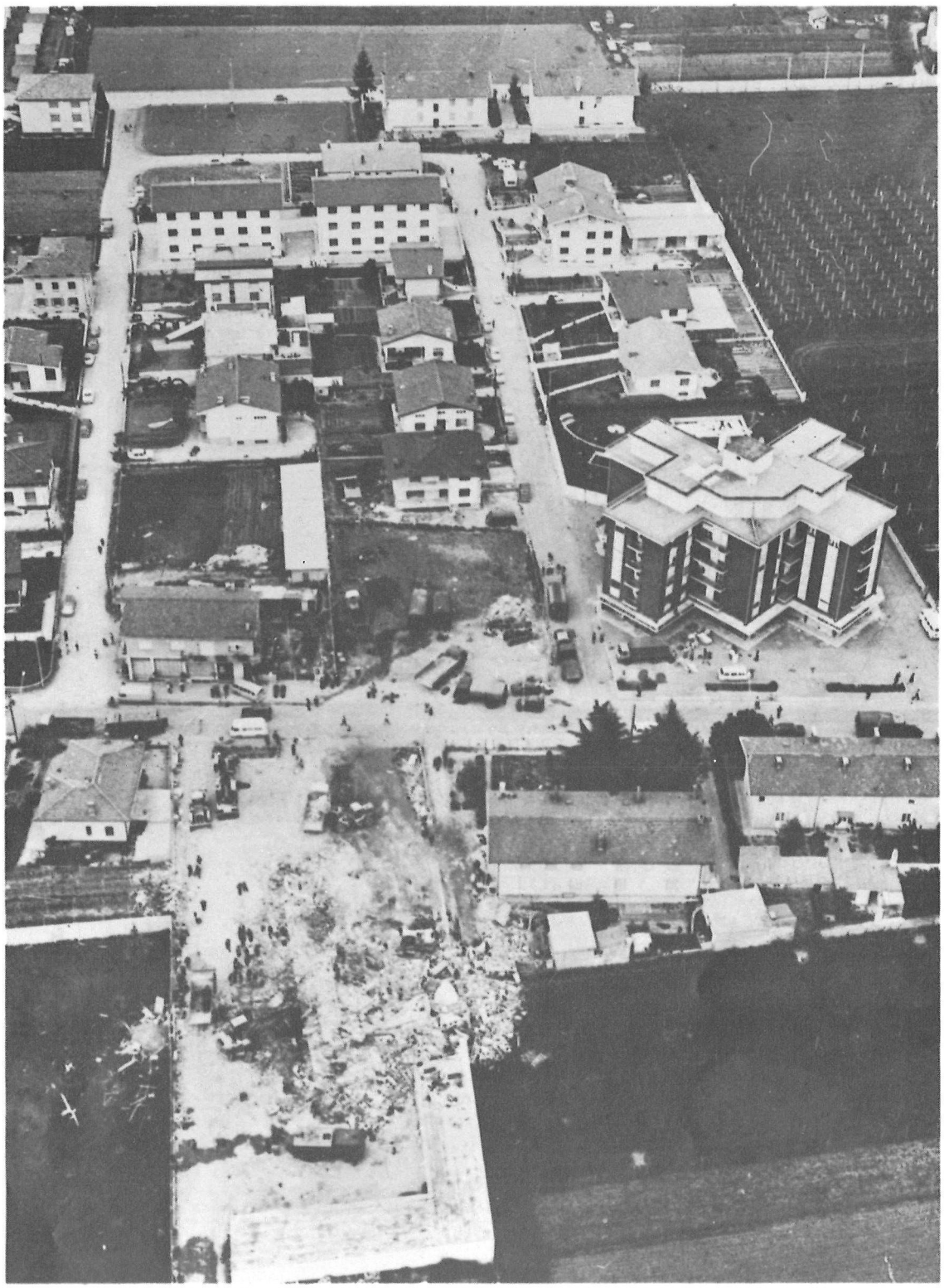
2.53. Map of the borough of Ursinins Grande before the Earthquake.



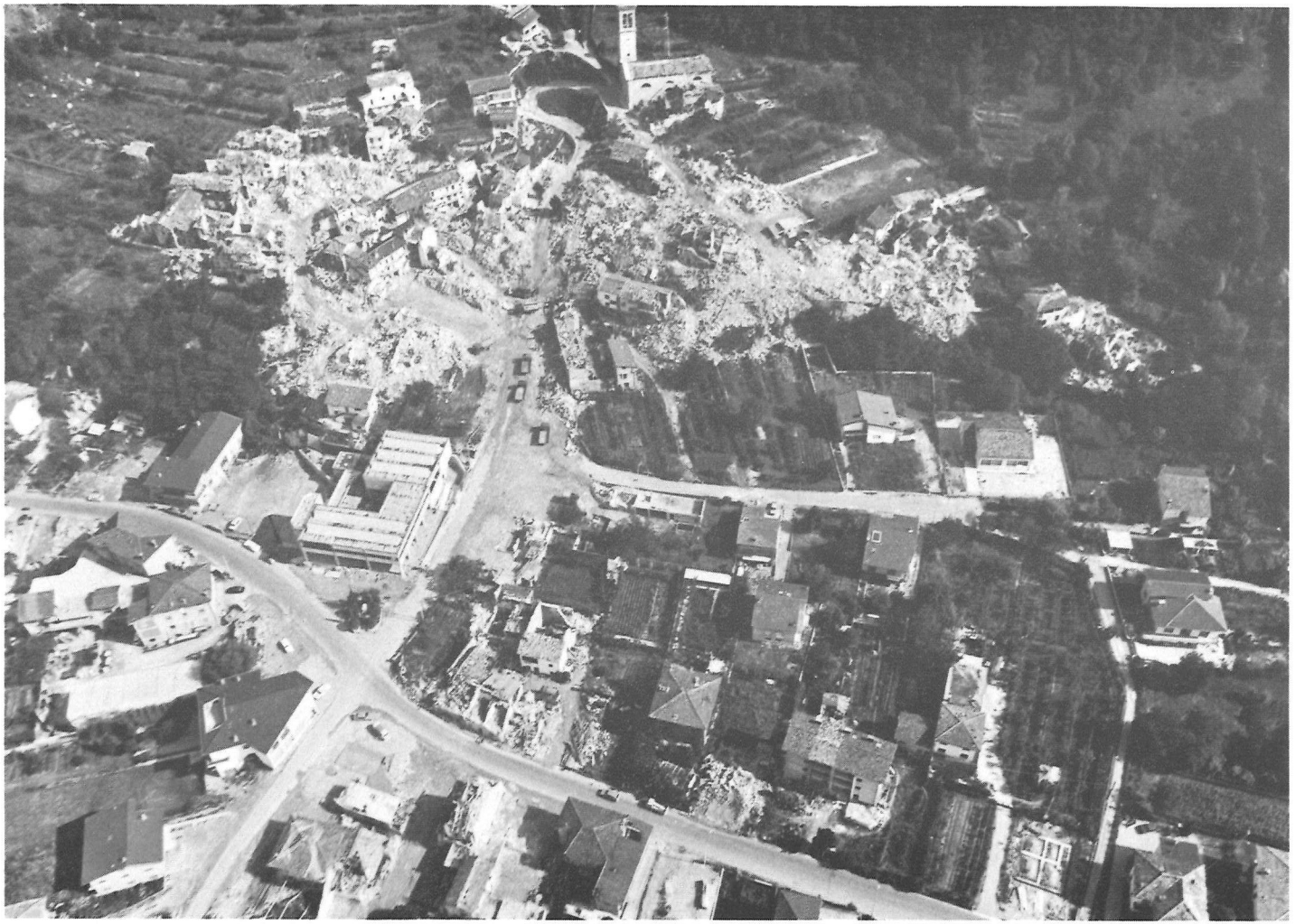
2.54. Map of the borough of Ursinins Grande - State of Buildings after May 6.



2.55. Majano after May 6.



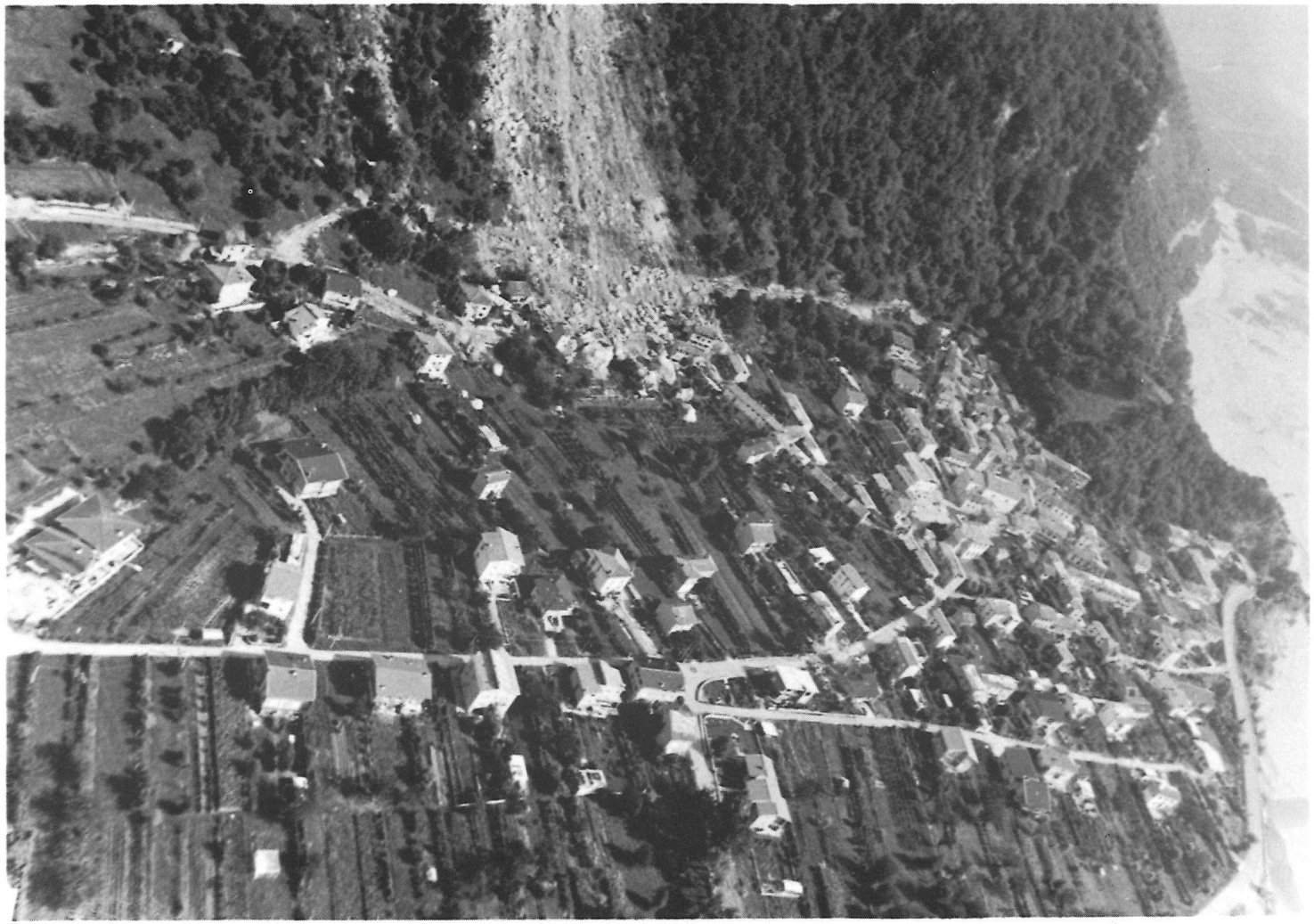
2.56. Majano after May 6.



2.57. Trasaghis after May 6.



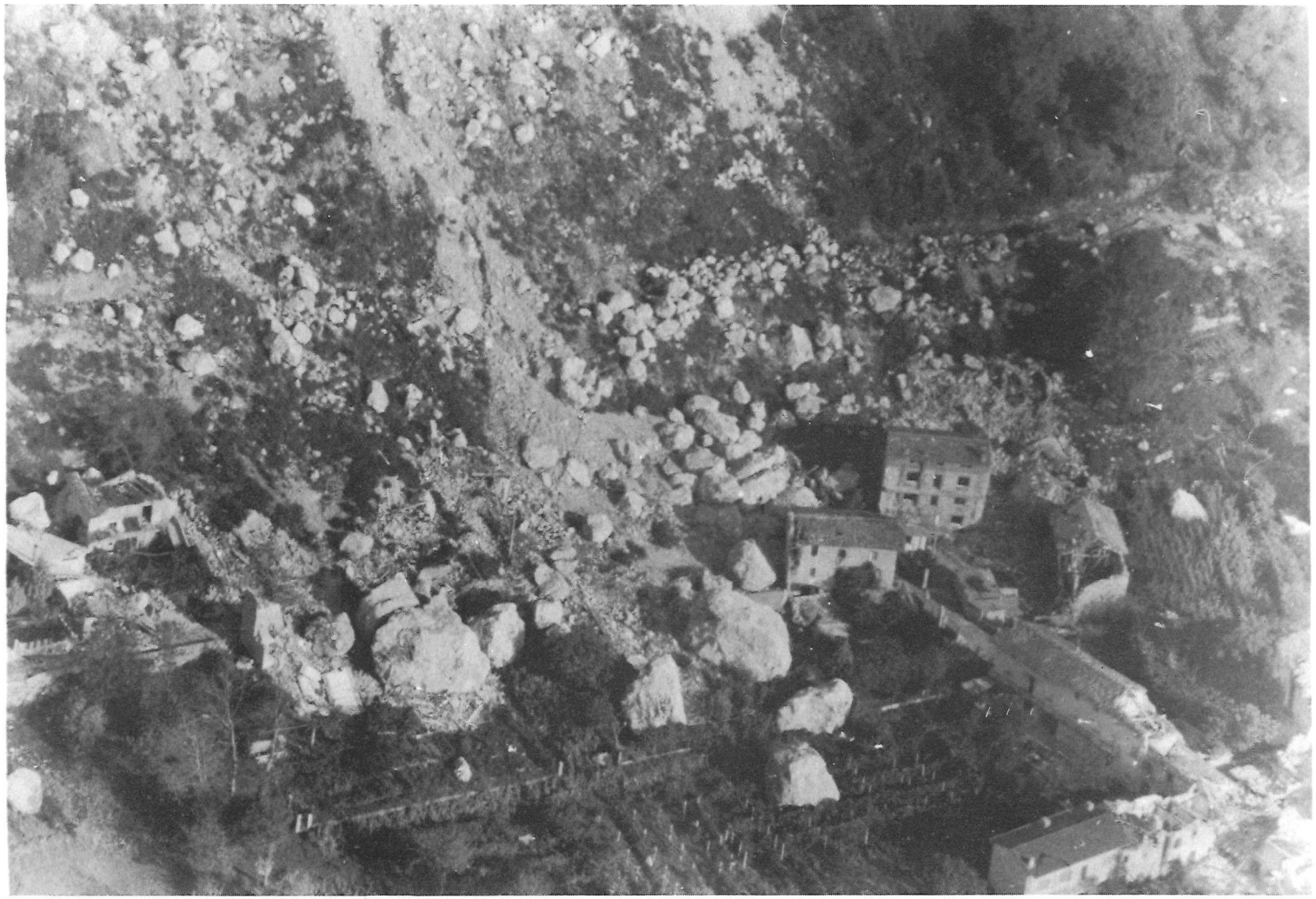
2.58. Braulins and the Mt. Brancot landslip after May 6.



2.59. The landslip on to Braulins after May 6.



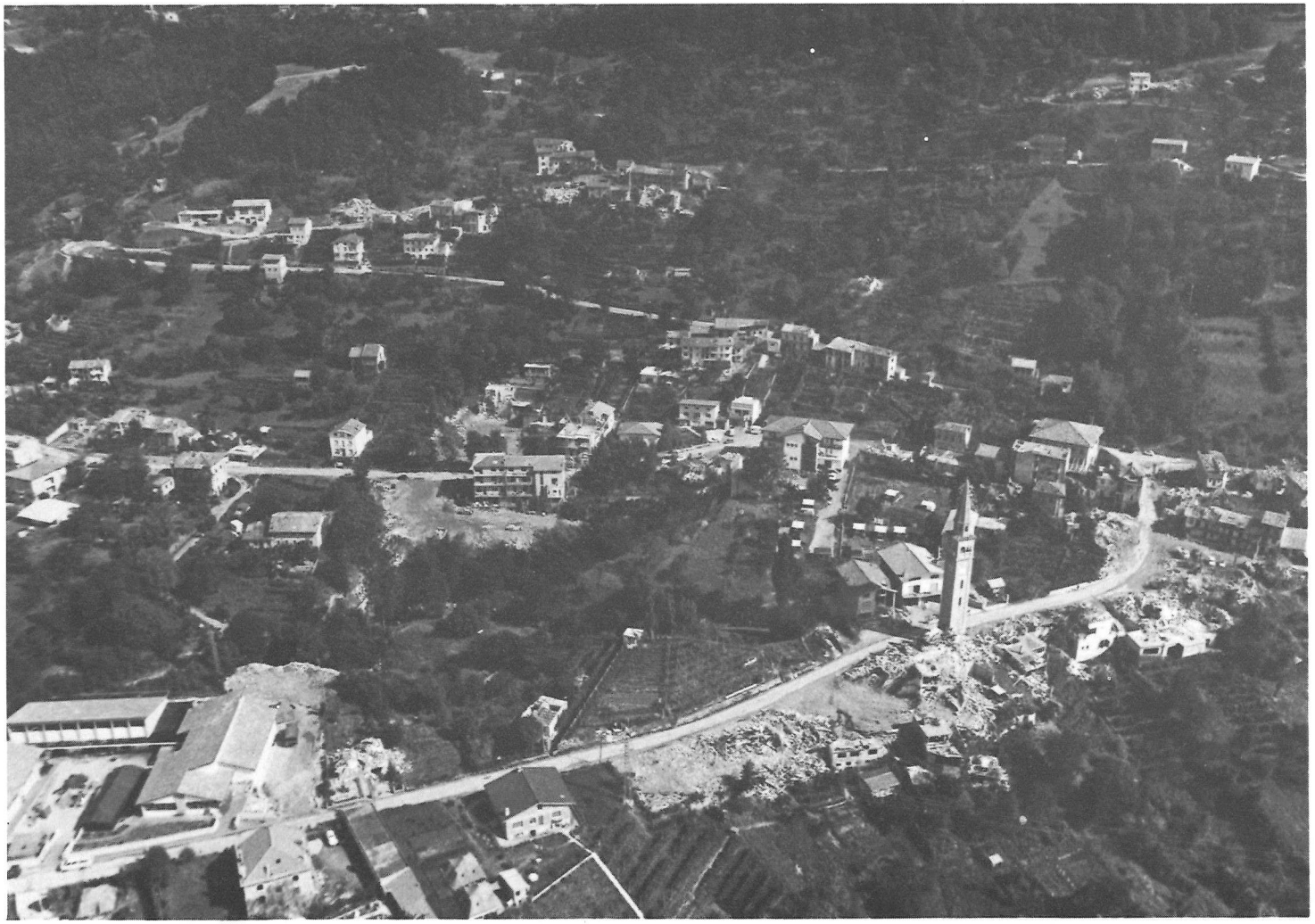
2.60. The landslip on to the village of Braulins.



2.61. Boulders from the landslip on the houses of Braulins.



2.62. Forgaria after May 6.



2.63. Forgaria after May 6.

buildings better global resistance to the May 6 shock.

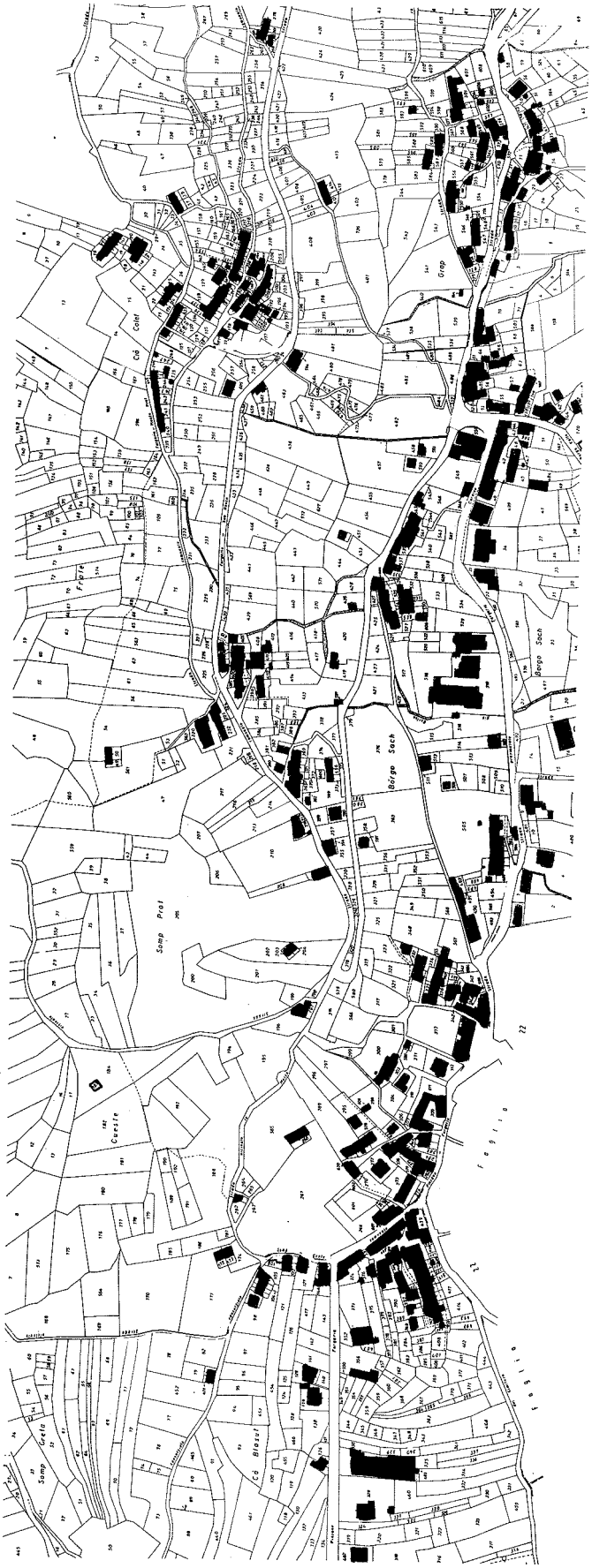
To the right in 2.46 and 2.48 is the church of S. Lorenzo, respectively as it was before the earthquake, and after the collapse of the bell-tower, the old pentagon of a Medieval castle. The background of 2.48 shows the main square of Madonna di Buia, already cleared of the rubble of many ruined houses, and the apse of the church, under demolition.

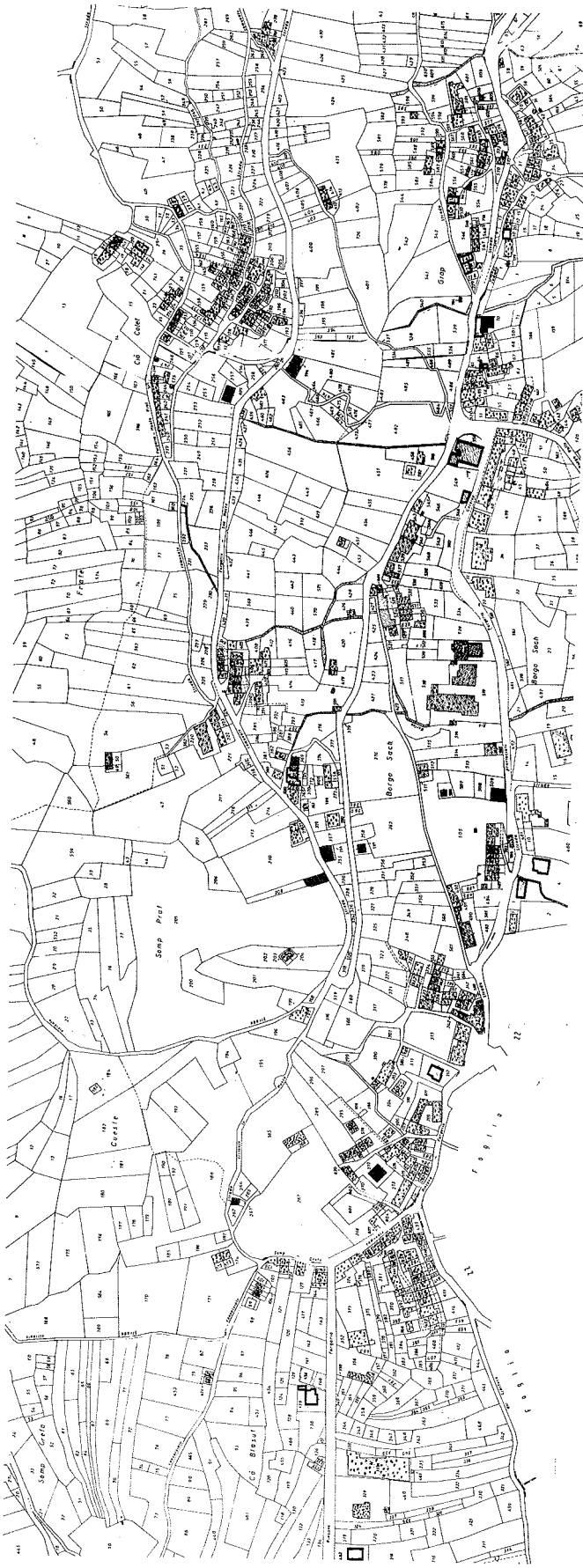
2.49 and 2.50 show the complete destruction of the entire roofs of the continuous structural complex of another borough of Buia. The roofs were placed transversally to the front line of the buildings. All the façade walls up to the first storey, on which the roof beams were resting, crumbled. The thrust of the beams and the weak scarfing in the front and side walls were two probable causes of collapse.

2.51, 2.52, 2.53 and 2.54 show the location maps of various parts of Buia, before and after May 6.

2.5.5. Other towns

Majano is a town in a state of great urban development, near the industrial zone of Rivoli-Osoppo. The situation in the town centre after the first shock is shown in 2.55 and 2.56. The massive damage suffered by the older and poorer buildings can be seen, together with the ruins of two recently built multi-storey apartment blocks,made of reinforced concrete conglomerate, which collapsed on the evening of May 6. The other two multi-storey buildings "still standing" and also visible in the two photographs, received severe damage to their ground-floor structures. After the September shocks





undamaged or slightly damaged
slightly damaged
severely damaged
destroyed or demolished

2.64. Map of the borough of Forgaria before the Earthquake.
2.65. State of buildings after May 6.

2.66. Map of the borough of Somp Cornino before the Earthquake.
2.67. State of buildings after May 6.



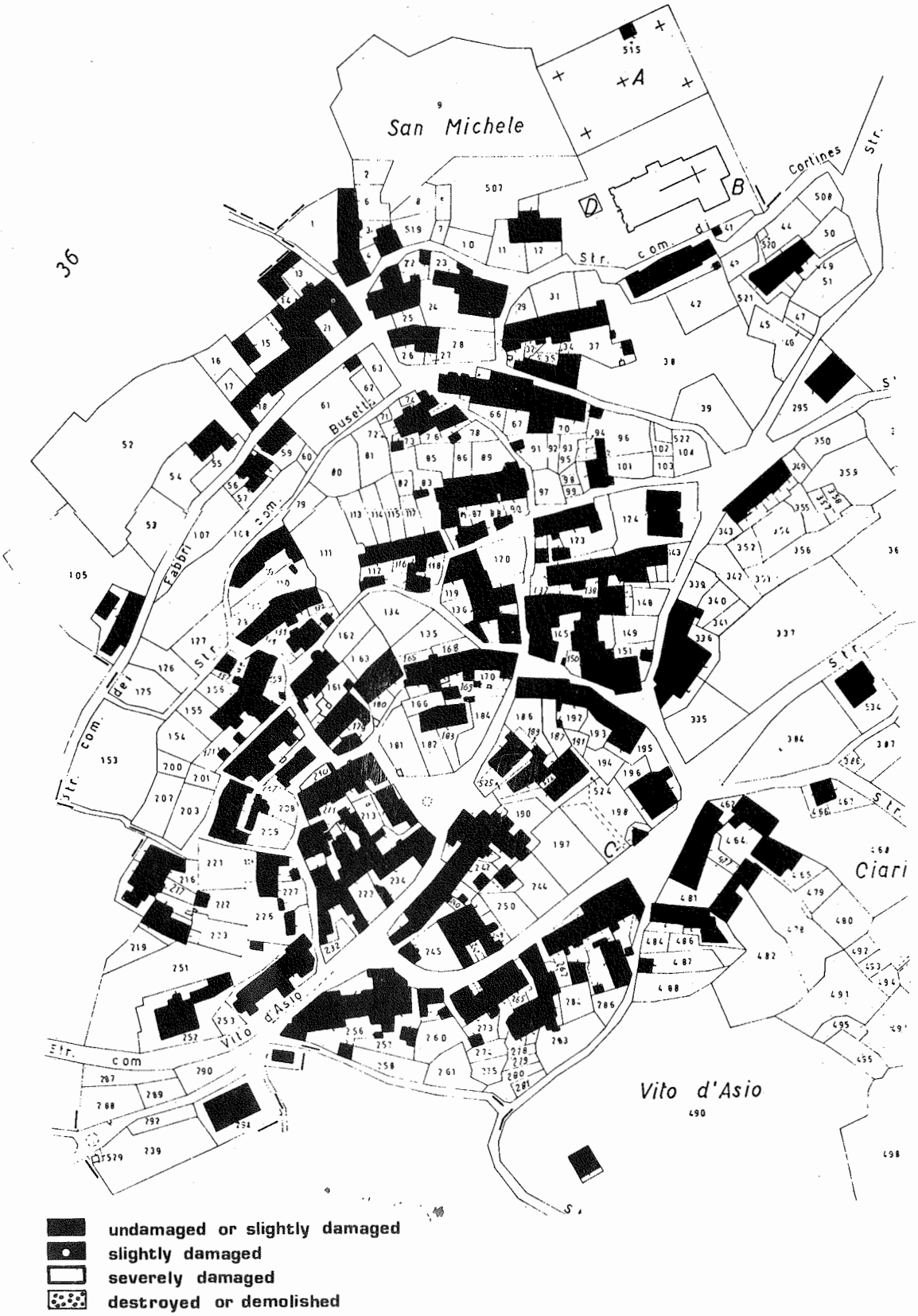
2.68. Massive destruction of the May 6 earthquake at Montenars.



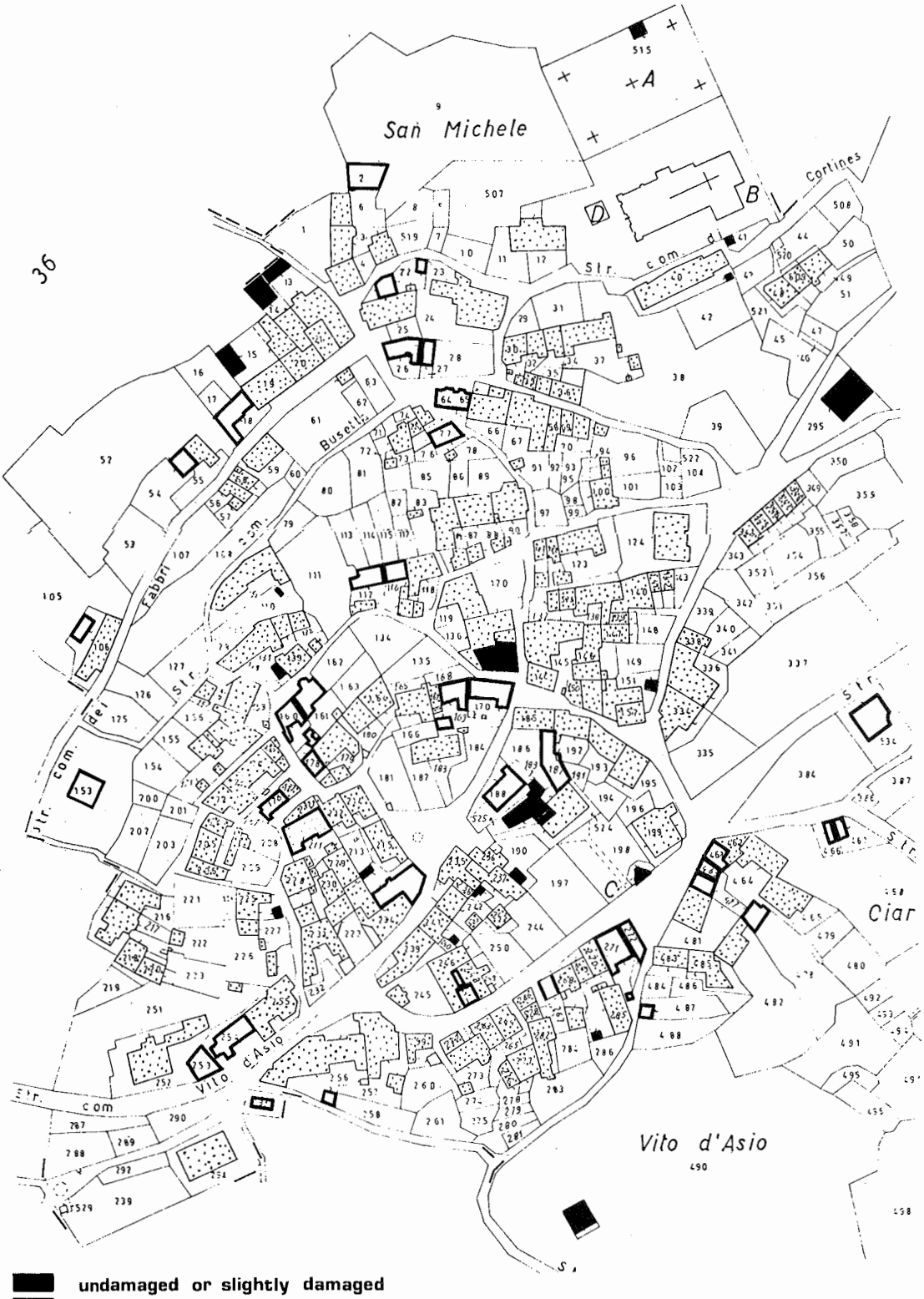
2.69. Map of Artegna before the Earthquake.
2.70. State of buildings after May 6.



2.72. Map of Pinzano al Tagliamento before the Earthquake.
2.73. State of buildings after May 6.



2.74. Map of the borough of Vito d'Asio before the Earthquake.



2.75. Map of the borough of Vito d'Asio - State of buildings after May 6.



2.76. Map of the borough of Anduins before the Earthquake.



2.77. Map of the borough of Anduins - State of buildings after May 6.

one wing of the first collapsed, and the second just collapsed entirely.

The massive and general destruction of the town centre of Trasaghis is shown in 2.57. After May 6, only the church and a very few houses were still recognizable. The general behaviour of the more recent buildings, of different construction technology, was very different.

In Braulins, on the right bank of the Tagliamento, a large landslip descending from Mt. Brancot on May 6 overwhelmed part of the town, crushing some houses (2.58 and 2.59). The size of the boulders and the houses destroyed can be seen in detail in 2.60 and 2.61.

Forgaria, comprising some boroughs spread out along the Carnia hills in the alluvial valley of the Tagliamento, underwent enormous damage, particularly in the boroughs of Forgaria itself, San Rocco and Cornino. 2.62 shows the main borough of Forgaria with many completely ruined houses and several others which suffered irreparable damage. The parish church is in the foreground; further back is the bell-tower of the church of S. Nicolò. This church and some of the houses of the borough were destroyed (2.63). Once again, the better resistance, in statistical terms, of the more recent buildings is evident.

The state of structural consistency of the buildings on May 6 is shown in the location maps of 2.65 and 2.67, for the centre of Forgaria and Somp Cornino.

For the town centre of Artegna, the situation of the buildings after the first shock

TABLE ON THE PRINCIPAL STATISTICAL DATA ON SOME OF THE WORST HIT MUNICIPALITIES - FRIULI EARTHQUAKE, MAY 1976

	No. inhabitants before earthquake.	No. dwellings before earthquake.	No. dwellings destroyed or demolished.	No. dwellings severely damaged but not demolished.	No. dwellings slightly damaged, and no. Dwellings undamaged.	No. of victims
VENZONE Town centre	565	165	30	129	6	16
VENZONE Whole Municipality	2780	1301	840	447	15	47
ARTEGNA W. Mun.	2850	1200	600	540	60	33
MONTENARS Whole Municipality	741	244	137	63	44	32
OSOPPO Town Centre	1200	739	671	68		110
OSOPPO Whole Municipality	2580	1147	759	388		110
TARCENTO Whole Municipality	9300	3450	606	660	2184	8
LUSEVERA Whole Municipality	1124	438	132	267	39	9
BORDANO Whole Municipality	1075	340	113	222	6	
CLAUZETTO Whole Municipality	823	317	30	235	52	11
VITO D'ASIO Whole Municipality	1486	504	192	312		14
FORGARIA Whole Municipality	2095	1335	797	511	27	72
CAVAZZO CARNICO Whole Municipality	1540	1177	62	200	915	
RESIA Whole Municipality	1663	940	530	400	10	5
BUIA Whole Municipality	6846	2519	813	1644	62	47
MAIANO Whole Municipality	5502	1782	537	941	304	131
S. DANIELE DEL FRIULI Whole Municipalyty	6845	2064	151	1040	873	6

can be seen (2.70). 2.72 and 2.73 relate to Pinzano al Tagliamento, while 2.75 and 2.77 show the state of the buildings after May 6 in Anduins and Vito d'Asio, the worst hit of the boroughs of the municipality.

Table 2.78 gathers together, finally, some of the most significant statistical data (not official) regarding the damage wreaked by the Friuli earthquakes, relative to a representative group of municipalities in the region.

3. BUILDINGS OF PARTICULAR ARTISTIC INTEREST

3.1. From the informational point of view, we present here some of the consequences suffered by the national monuments and artistic heritage of Friuli, as a result of the recent earthquake and particularly of the stronger shocks (first: 9 p.m., May 6; second: 6 p.m., September 11; third: 5 a.m., September 15; fourth: 11 a.m., September 15). We refer particularly to the most important monuments totally or almost totally destroyed by the earthquakes, in a sequence which we will try to indicate by means of photographs taken during the various phases.

It is obvious that reference to the most important monuments cannot be taken as a guide for the whole region, as "importance" is a parameter which cannot be defined. We will deal, rather, with a simple exemplication devoted to some buildings

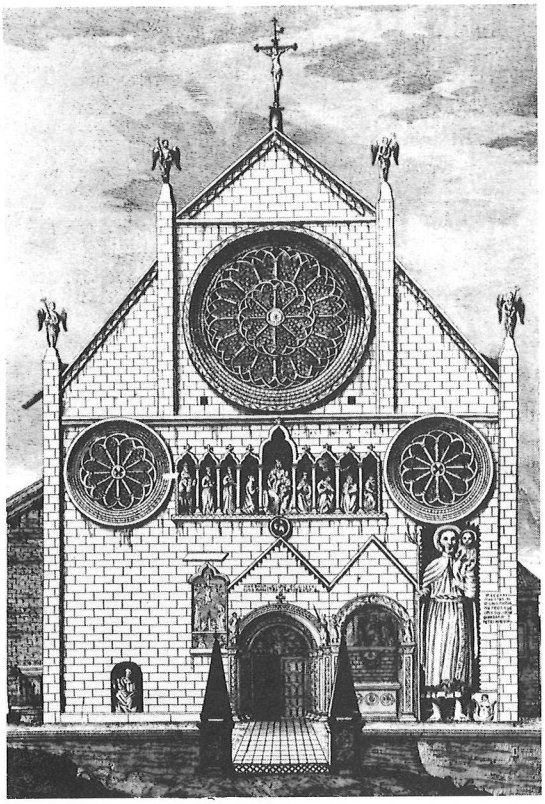
of particular interest, making up the artistic and traditional heritage of this part of Italy, and of special interest as the symbolic centre of the public life of a population.

We will therefore consider the historical masterpieces of the towns of Gemona and Venzone, in order to examine their most important public buildings: the Cathedral and the Town Hall.

3.2. The Cathedral of Gemona

3.2.1. It is difficult to gather together the most important historical information on this monument, owing to its numerous and continual refacings, transformations and adaptations over centuries.

The Cathedral was consacrated in 1337 (almost the same year as the consacration of the Cathedral of Venzone). The church was originally rectangular in shape with three naves divided by two series of seven piers with a rectangular presbitery. The continual refacings and reworkings mentioned earlier led to the pre-earthquake state of the building, with the complete reconstruction of the façade between 1825 and 1828. This work was suggested by the precarious state of the façade due to the thrust of the arches dividing the naves, whose spans had been increased during previous transformations, and to the effect of earthquakes. The reconstruction was carried out maintaining the decorative features of the cathedral, but architectural changes were made, consisting above all in the subdivision of the façade itself into three compartments with the insertion of four pilaster strips. The original façade is shown in a print dating from 1771 (see Photograph no. 1). The present façade is seen in 2.



3.1.

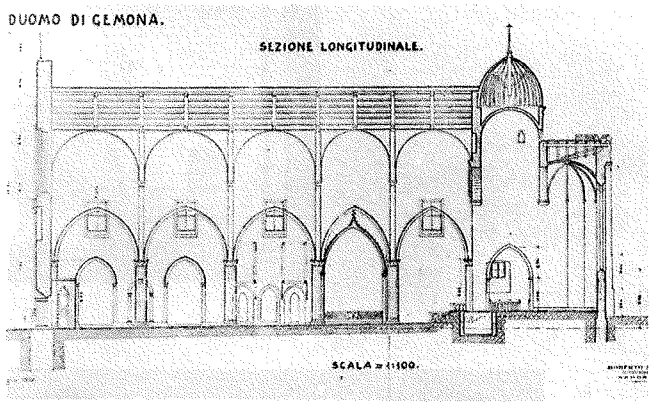


3.2.

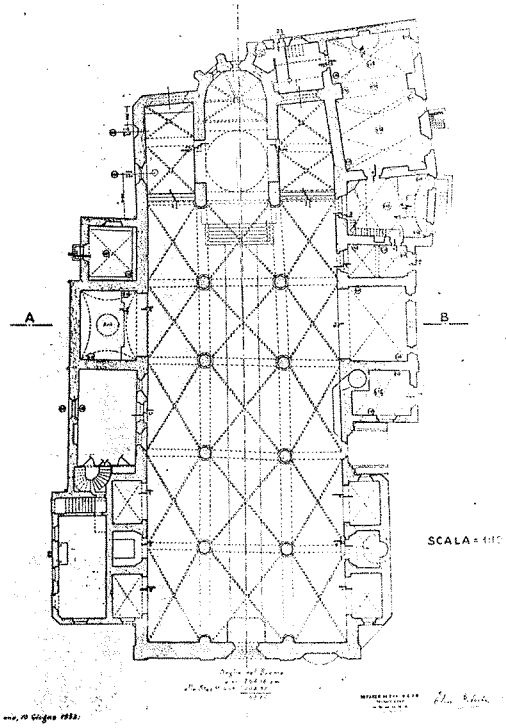
The three-naved church is 49 metres long and 16.60 m wide, with the central nave of 6.40 m. 3, 4, 5 and 6 (photographs by Roberto Elia) show respectively the plan, the longitudinal section, the cross-section and the right side.

7 shows the inside of the church: as can be seen, the space inside is enclosed with a semicircular apse formed by four slender ribs with high windows at intervals.

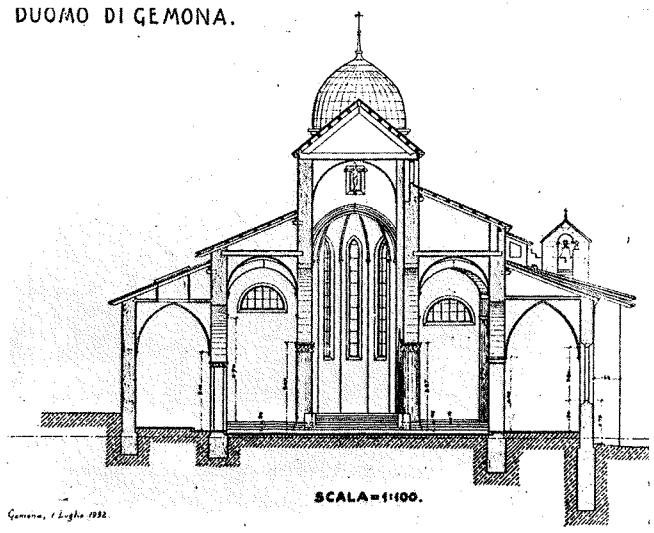
The church is built on an earth bank, supported on its right side by an old massive face wall made of stone, from which the "Porta Udine" develops.



3.4.



3.3.



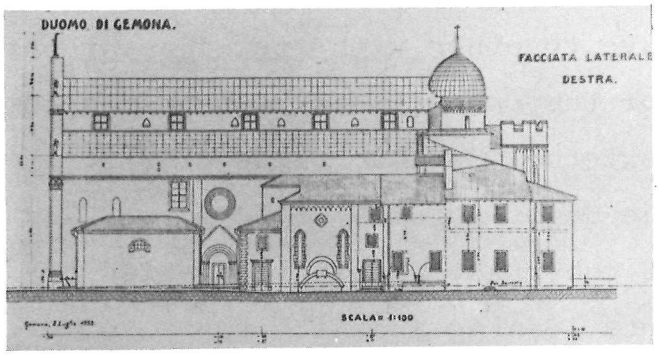
3.5.

The bell-tower, built between 1341 and 1369, was detached from the actual building of the cathedral, and rose to the left of the façade to a height of 50 m including the spire. The massive structure of the tower can be seen in 8.

Both the church and the bell-tower are composed of external facings in stone, filled in with pebbles or scabbling and mortar. The thickness of the walls ranges from about 80 to 100 cm.

3.2.2. Taking into account the effects of the earthquake and considering the façade of the building, we can see how, with respect to the first shock, the façade stood up to the strain very well, probably due to its recent reconstruction (9). Features to be noted are: the fall of the angel placed at the top of the pilaster strip on the left; the shifting of the angel to the left of the central gable (10), and the movement of the gable itself, some stones being moved out of alignment with the façade (11).

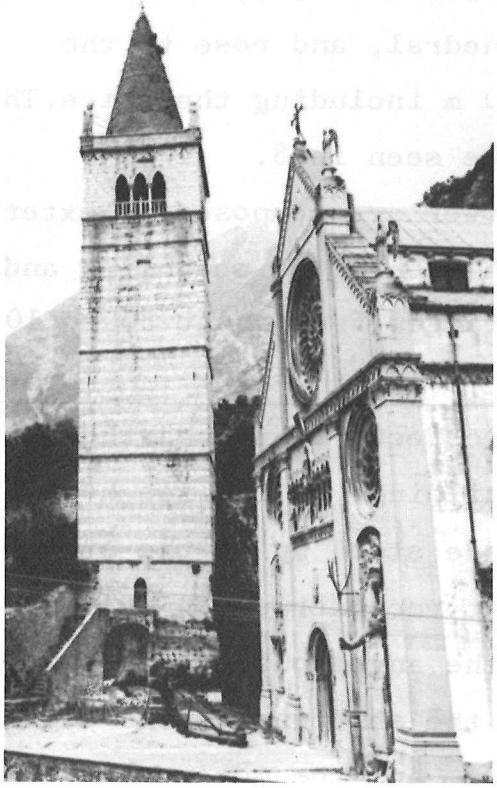
The second shock caused the fall of the angel to the extreme left and that to the right of the



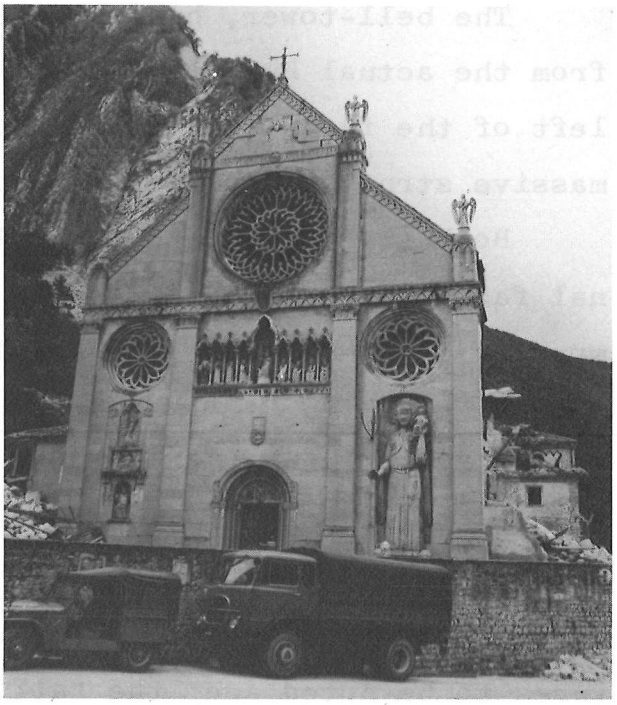
3. 6.



3. 7.



3.8.



3.9.

gable. The gable itself was even further moved out of alignment (12) (the angel to the left of the gable had been previously dismantled).

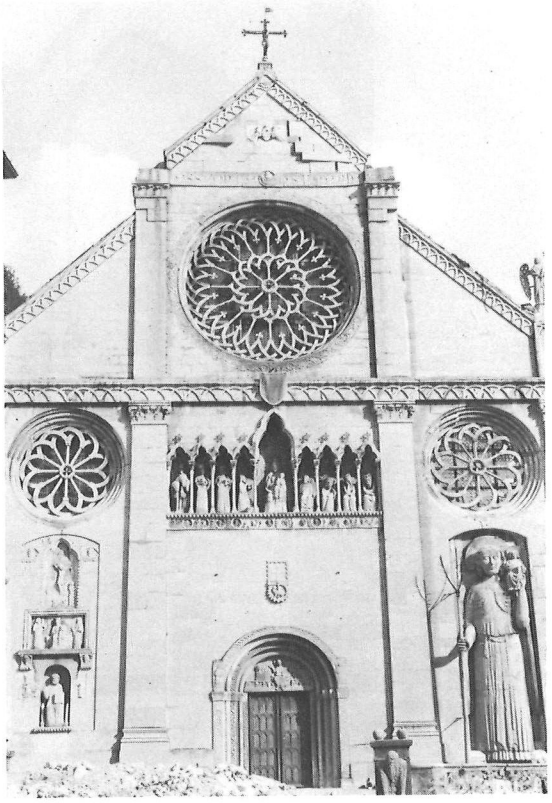
The most recent shocks caused the entire collapse of the gable (13).

As regards the body of the church, the first shock caused the collapse of the right nave together with its adjacent parts, except for the cross-vault of the compartment between the third and fourth columns and the external body corresponding to it (Chapel of the "Beata Vergine degli Angeli") (14, 15, 16 and 17). The collapse of most of the right wall is connected to weak wind-bracing by the parts leaning against it.

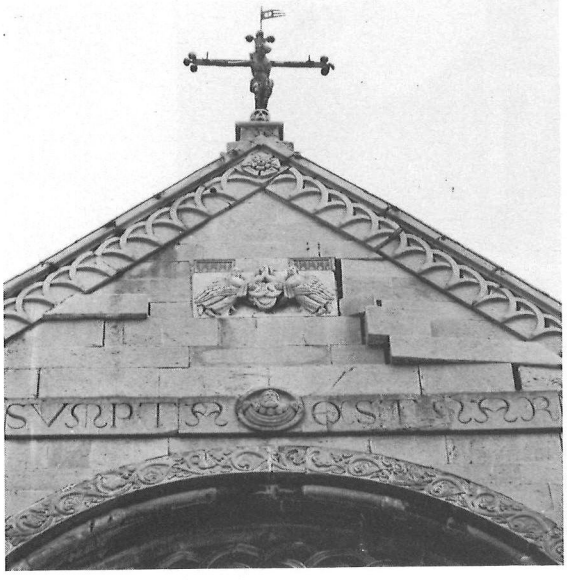
At this point we must mention the settling influence of the face wall; this settling, perhaps already under way, is



3. 10.



3. 12.



3. 11.



3. 13.



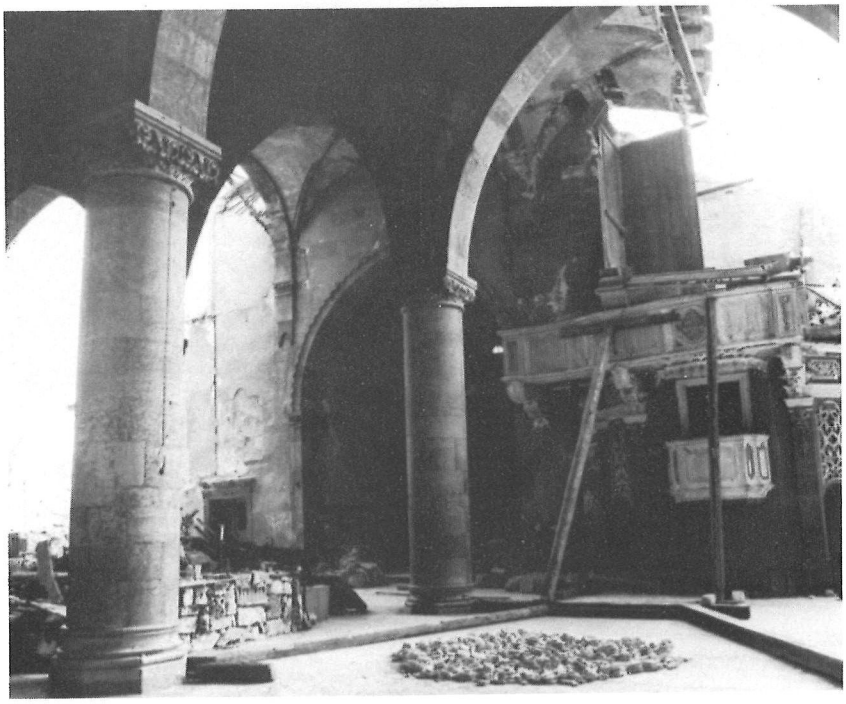
3.14.



3.15.



3.16.



3.17.

very obvious in the front part of the wall in question (18).

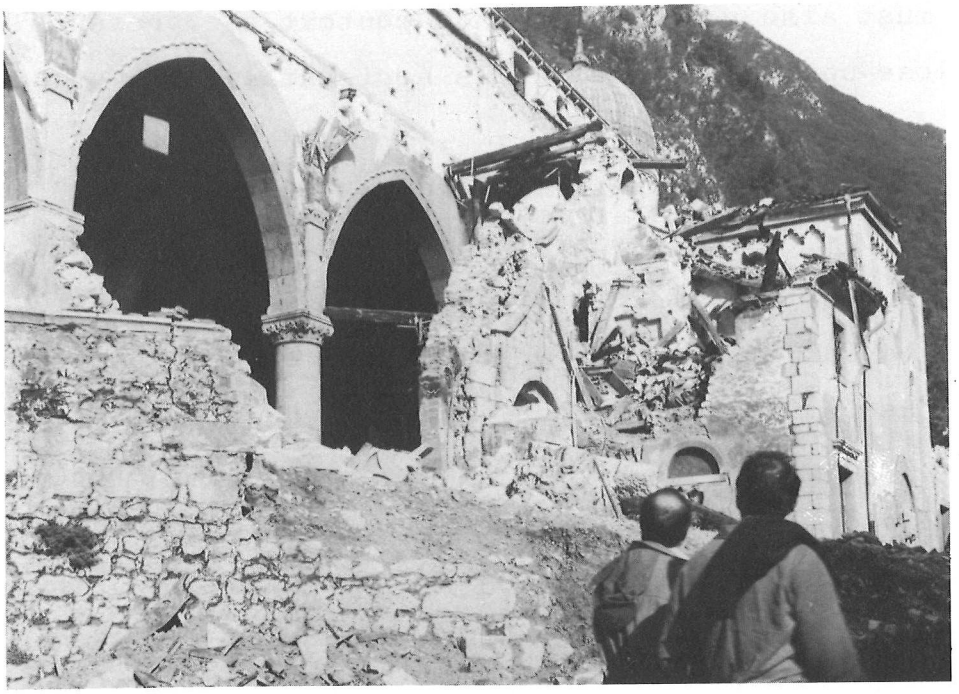
The second shock brought about the complete collapse of the right nave and the partial collapse of the remaining portions leaning against it (19, 20).

The later shocks completed the destruction of the right nave, and also caused the total collapse of the apse roof and serious settling of the arches supporting the drum (21, 22, 23). Lesions and further settling appeared at the base of the columns (24, 25).

It can therefore be seen that the building resisted several severe shocks quite well, leaving out of consideration the right-hand area, already subject to several structural defects which weakened it.



3.18.



3.19.



3.20.

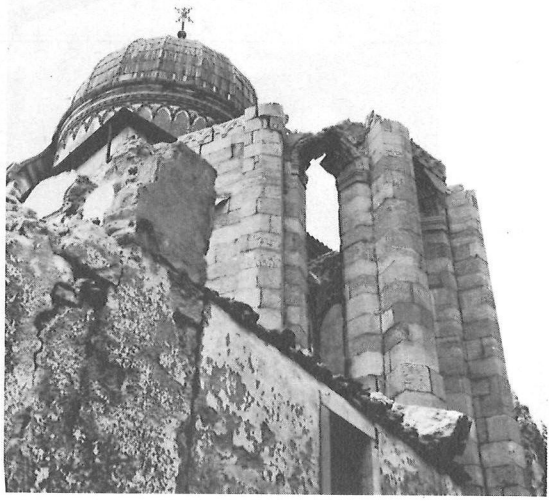
We must also remember in this context the presence of several close-up longitudinal walls horizontally linked at roof level.

3.2.3. Of extreme interest are photographs 25, 27 and 28 taken at 2 p.m. on Wednesday, September 15, three hours after the last shock. The back portions of the old face wall, outside "Porta Udine" (visible on the left) can be seen in these photographs immediately before, during and after collapse.

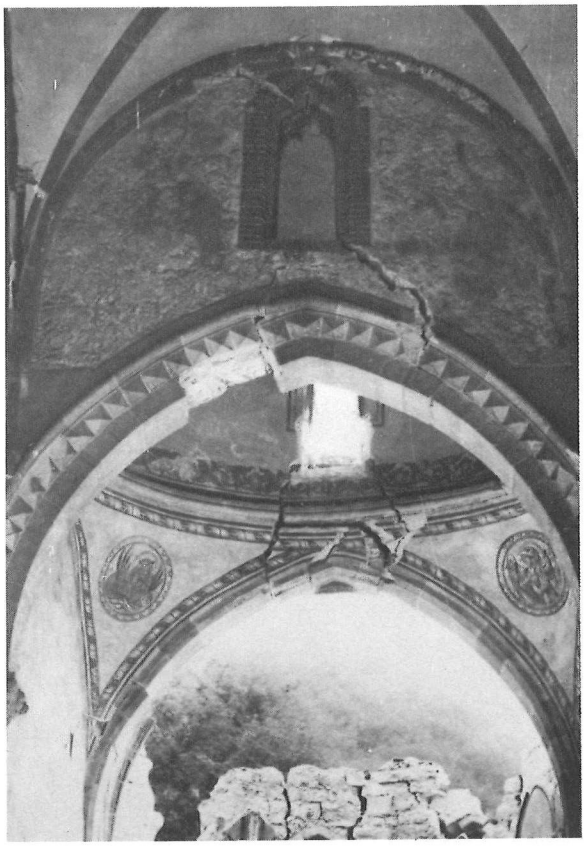
3.2.4. As a result of the first shock, the bell-tower had practically collapsed in its entirety. The tower itself was fractured into four portions lengthways, down the midline of each face. It must be presumed that even before the earthquake the bell-tower walls had lesions or cracks corresponding to these midlines (we know that an earth-tremor took place during its construction). 29 shows the pieces which remained after the collapse; these were later demolished. 30 shows the beginning of one of these lesions above the entrance.



3.21.



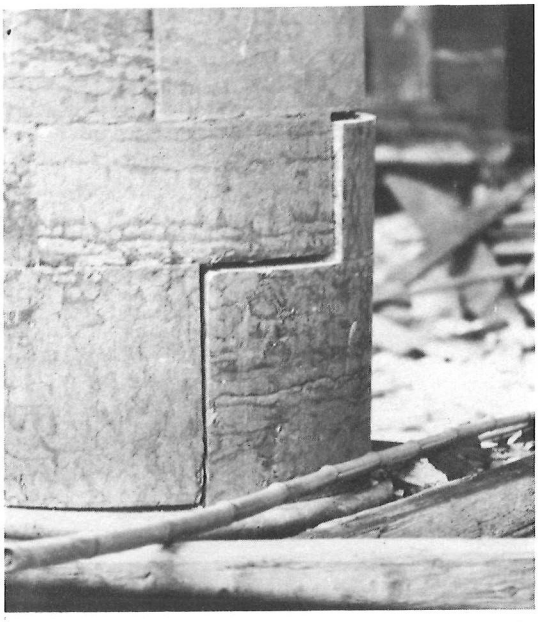
3.22.



3.23.



3.24.



3.25.

3.3. The Town Hall of Gemona

3.3.1. The Renaissance Town Hall of Gemona was built at the beginning of the 16th century in the same form that, with the exception of a few details, it still has today (see 31: 1771 print). It is composed of a lower loggia facing outwards with three openings consisting of Roman arches and an upper hall (main hall) supported and covered by wooden floors.



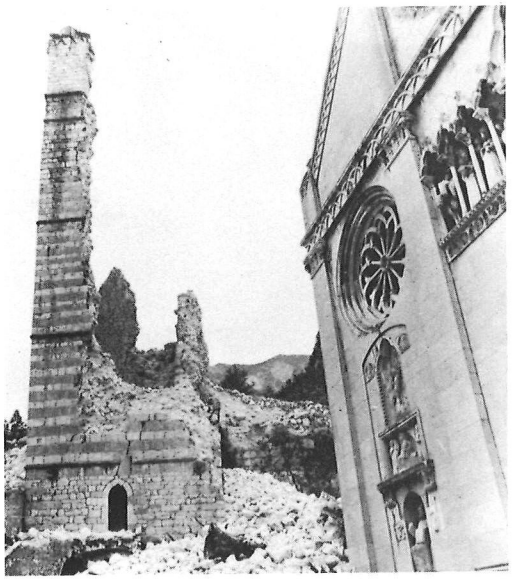
3.26.



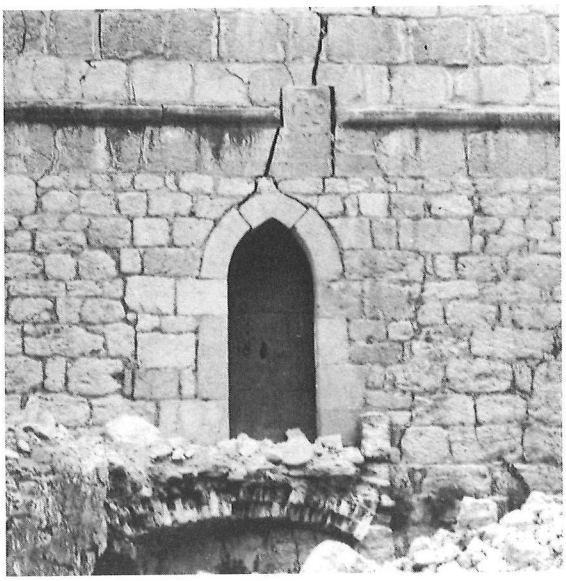
3.27.



3. 28.



3.29.



3.30.

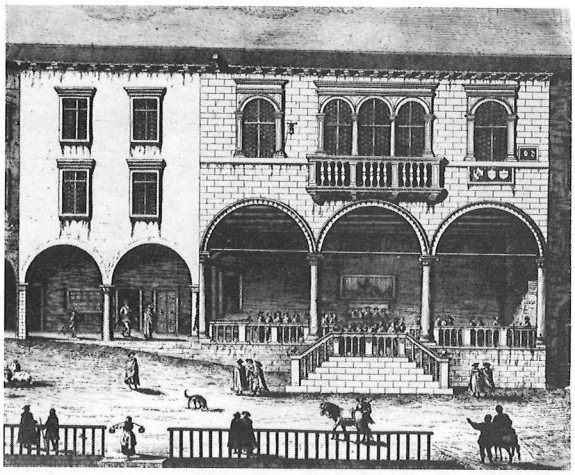
3.3.2. The building, wind-braced in the plane of the façade by the parts which flank it, survived the first shock relatively well. 32 shows some disjointing in the façade facing and in the arches, and some damage to the decorative elements.

The second shock worsened the situation of the façade. 33 shows the collapse of some elements of the external facing with evidence of swelling towards the outside of the left portion of the façade, not sufficiently well joined to the wall behind.

The last shock brought about the collapse of this portion of this façade caused also by the similar collapse of the adjacent parts (34, 35).

3.4. The Cathedral of Venzone

3.4.1. This monument, finished in the 14th century, was consacrated in 1338 by the Patriach Bertrando. It was built by a certain Magister Johannes as an amplification and development



3.31.



3.32.



3.33.



3.34.

of a previous building belonging to the first half of the 13th century, further reworked in the second half of that century. (The raising carried out during this reworking is shown by a trace existing on the façade relative to the previous position of the pitch of the roof, which was very sloping (40).

The Cathedral was in the form of a Latin cross positioned over a surface os 27 x 48 m. The nave, 12 m wide, goes on for

23 m to meet the transept, again 12 m wide. The rear part contained a presbitery divided into three compartments closed by three semicircular apses illuminated by high windows. Two chapels, the Gonfalone Chapel and the Rosary Chapel, were later built at the sides of the nave and close to the transept, the former dating from 1394 is on the right, and the latter dating from 1647 is on the left. The sacristy, adjacent to the Gonfalone Chapel, was on the right. 36 shows this.

The roofing consisted of wooden trusses at a height of approx. 15 m. The presbitery and the apses were covered with cross-vaults.

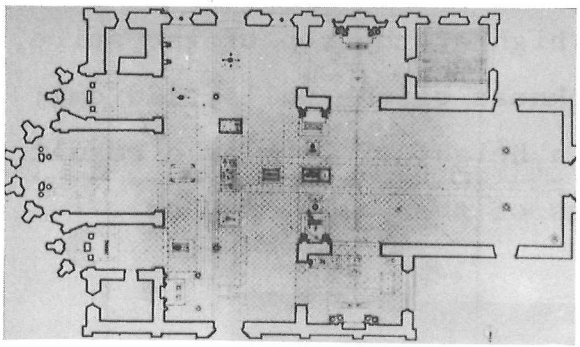
On the sides of the presbitery and very close to the extreme wings of the transept there are two bell-towers: the northern one was complete, 50 m high at the top of the spire, and 5.70 x 7 m at the base (thickness of the walls, 120 cm); the southern one was cut off at a height of 26 m as a result of the 1348 earthquake (thickness of the walls, 160 cm).



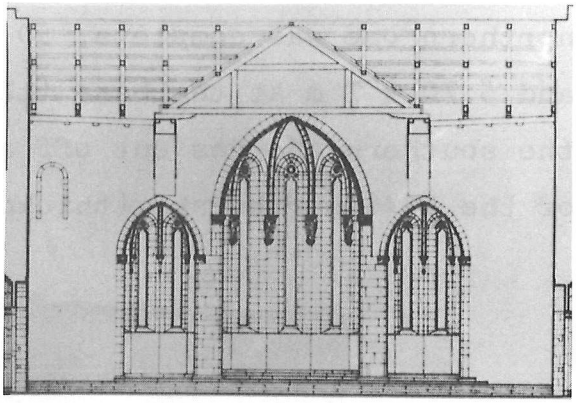
335.



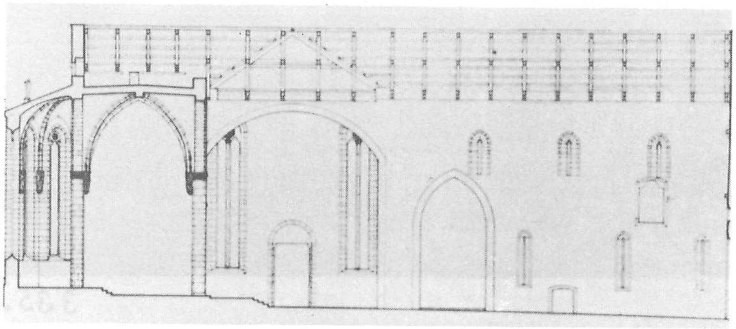
3. 36.



3. 37.



3. 38.



3. 39.



3.40.



3.41.

The walls of the building, approx. 100 cm thick, were mostly composed of two unplastered facings in stone blocks, with pebble and mortar fillings.

The drawings of 37, 38 and 39 by Prof.Guido Confero refer to the plan and sections of the transept and nave, and illustrate the above.

3.4.2. In order to follow the phases which led to the complete destruction of this monument, we consider first of all the main façade (40) to see the effects of the first shock (41). Most of the gable, lacking orthogonal links, collapsed, while the remaining part leans heavily outwards. The remaining portions are those scarfed to the lengthways walls. The masonry of the portal erected close to the façade, without scarfing, has collapsed.

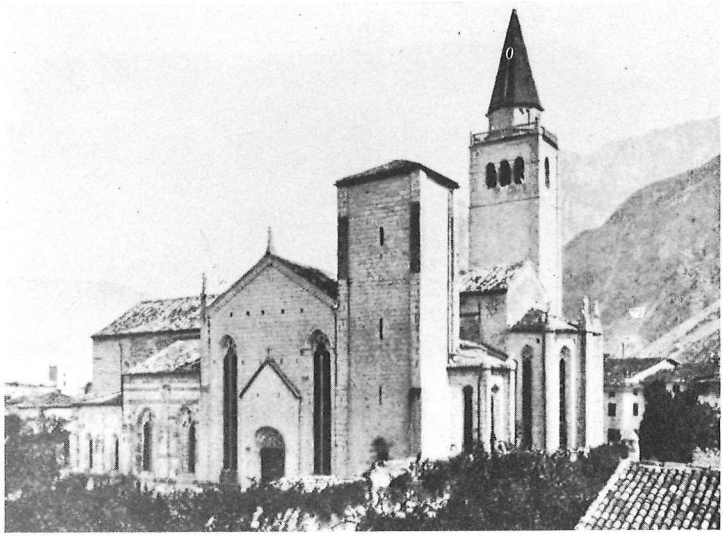
42 shows the same façade after the third shock. Collapse is almost complete and extends to the front part of the roof.



3.42.



3.43.



3.44.



3.45.

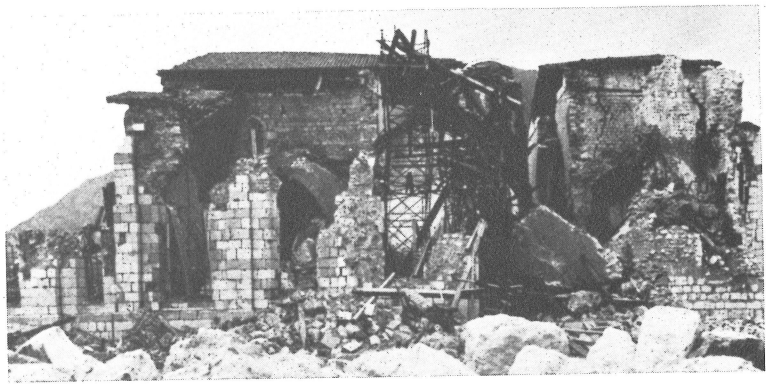


3.46.

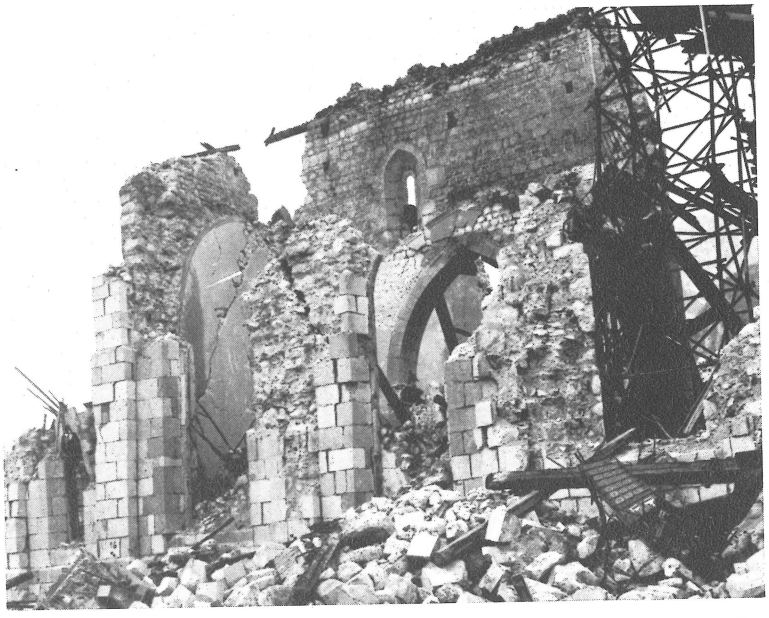
The remaining portions of the walls flanking the nave are braced by the side chapels.

43 shows the situation after the last shock. The bas-relief lunette on the portal fell, while the roof collapsed completely.

44 shows an interesting view of the southern flank of the church, illustrating the structure of the building itself,



3.47.



3.48.

from the principal façade to the apse area. 45 and 46 show the effects of the first shock (the foreground area was cleared of rubble during initial clearing operations). The façade of the transept, weakened by two high openings to the sides of the portal, entirely collapsed. The Gonfalone Chapel and the sacristy, although seriously damaged, did not collapse, presumably owing to the presence of short, orthogonal walls. The low bell-tower completely collapsed.

47 and 48 show respectively the southern flank after the



3.49.



3.50.

third shock, and the Gonfalone Chapel after the last shock. The building is completely ruined.

49 and 50 show respectively the area of the apses and the northern façade of the transept. 51 shows the area of the apses after the first shock. Their structures stood up to the strain quite well, owing to the cylindrical form of the walls, while the collapse of the cross-vault over the main apse is a consequence of the lack of horizontal linking to the top of the ribs. 52 shows the same area after the second shock. The ribs, already free above, became disconnected, and one of them fell. The last shock (53) caused their complete collapse and the ruin of other ribs.

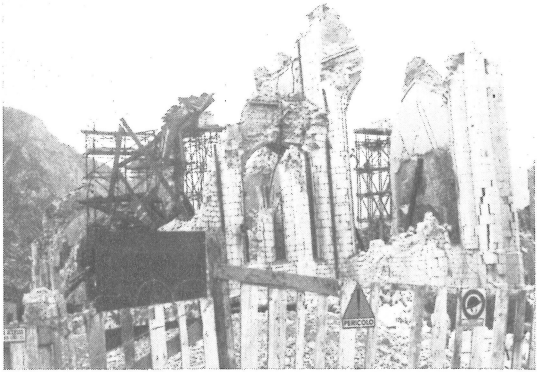
As regards the bell-tower and the northern façade of the transept, we can observe how, after the first shock, the bell-tower showed damage to the belfry itself (54) and a serious V-shaped lesion in the east wall adjacent to the apses (55). This last, aggravated by the presence of an opening, showed



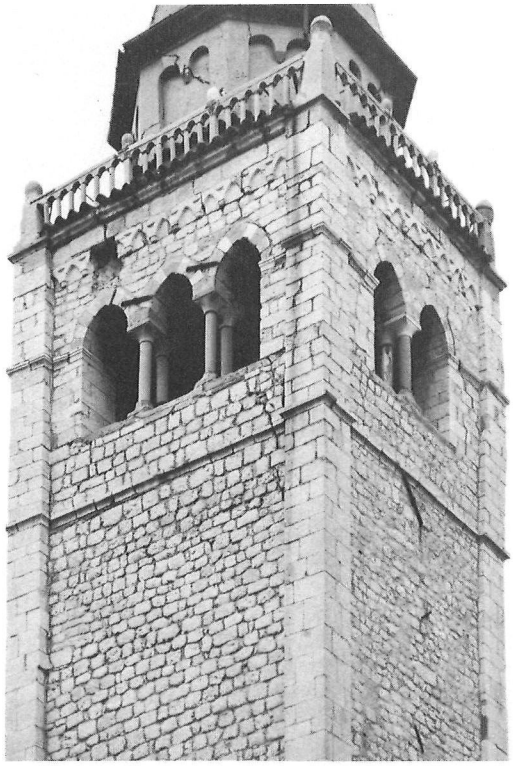
3.51.



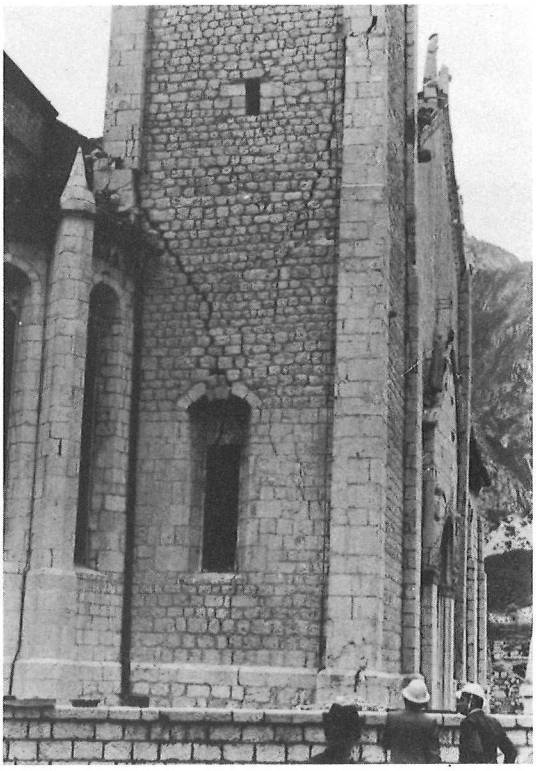
3.52.



3.53.



3.54.

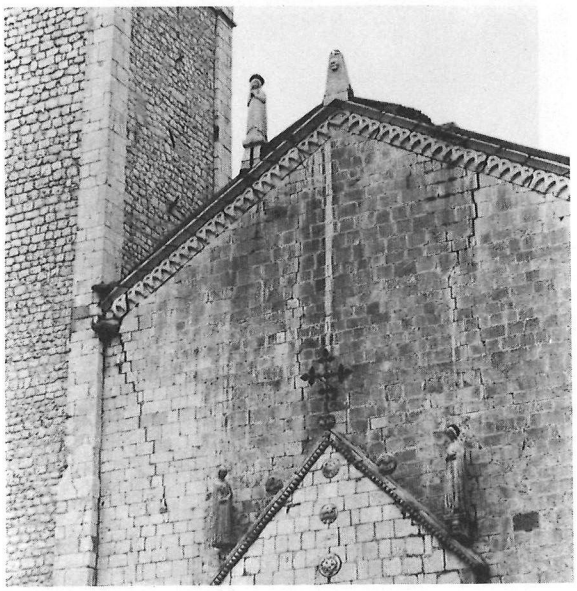


3.55.

the same characteristic on the opposite façade, and is to be related to the solidity of the tower with respect to the structure of the presbitery.

The façade of the transept scarfed to the bell-tower and, moreover, not sufficiently connected with the structures behind, was moved out of alignment (56), shifting outwards increasingly towards the opposite flank to the bell-tower, where the same non-alignment reached 50-60 cm. The consequent wide crack in the roof and on the side of the transept is clearly visible in 41 (on the left).

The situation became worse after the second shock. The portal of the transept, not scarfed in the masonry lying behind (57), collapsed, while evident signs of shifting in the corner rib near the façade of the transept appeared in the



3.56.



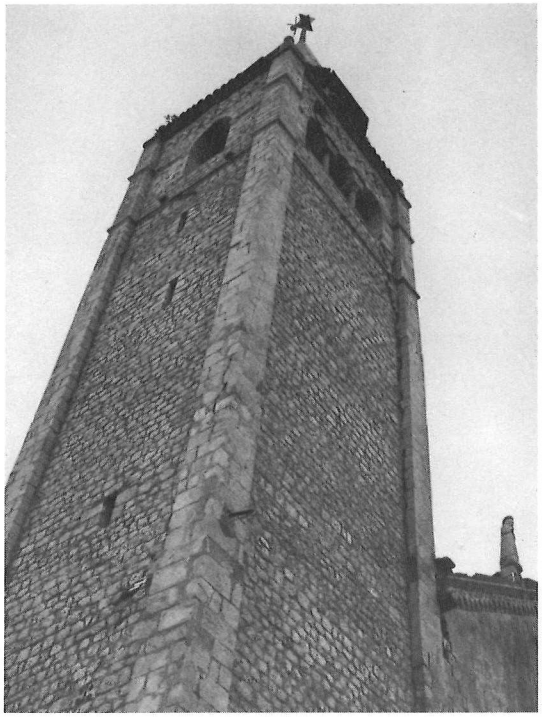
3.57.

bell-tower (58). The V-shaped lesion, carefully examined, did not appear in an obviously worsened state.

The third shock brought about the collapse of the façade of the transept and the bell-tower, which fell in upon itself owing to the collapse of the walls at the V-shaped lesion (59). 60 shows the final situation.

3.5. The Town Hall of Venzone

3.5.1. Built at the beginning of the 15th century, this monument is composed of a lower loggia, open to the outside with eight Roman arches, and a corresponding upper hall supported by a wooden floor. Access to the hall is by means of an outside staircase, built at the end of the 15th century, together with the corner tower with its clock.



3.58.

This is delineated by two massive walls belonging to the façades of the building, and by two other lighter walls overhanging the hall.

The building, which was not further altered after the Renaissance restoration aimed at repairing the damage done by a fire in 1571, was completely demolished at the beginning of 1945 by bombing. The almost complete recovery of the stones and masonry of the façades, including the worked stone pieces, allowed the monument to be faithfully reconstructed (61).

3.5.2. Considering the effects of the earthquake, it should be observed how the building survived the first shock without sustaining serious damage (limited to the falling of some decorative elements, and to some disjointing in the intersecting areas between floor and walls), (62, 63). This is



3.59.



3.60.

evidently to be ascribed to the good quality of the recently reconstructed masonry, and to the efficiency of the horizontal links; moreover, the strengthening effect of the outside staircase should not be forgotten.

Instead, grave damage was done by the last shock. Apart from damage to the decoration and the lesions in the walls,



3.61.



3.62.

the roof of the west part of the building collapsed, together with the end wall and the wall parallel to the façade giving on to the square (64).

Finally, very serious shifting can be noticed in the clock-tower: the two corner walls, split at their joint with the walls of the façade, were strongly moved towards the outside, together with the whole of the clock-tower itself (65, 66).

3.6. Conclusive considerations

Considering the very serious damage caused by the earthquake to an artistic heritage of inestimable and almost irreparable value, it is opportune here to recall the fundamental importance of the problem of preventive protection aimed at reducing the so-called "seismic risk". This observation, by no means original since many other scientists have proposed it and worked on it for a long time, should nevertheless be re-



3.63.



3.64.

peated, since it concerns the only possible efficient way of safeguarding monuments, even though it involves very large objective and financial commitments.

The seismic weakness of any artistic heritage means that even relatively minor earthquakes cause damage which is only theoretically reparable, as there are great difficulties involved, and above all great risks to workers. In other words, present technology offers very wide possibilities in the field of static restoration, but its application must be carried out by man on structures of a certain size whose static conditions can hardly ever be clearly defined. Therefore, the static works carried out preliminary to general restoration present those who undertake them with conditions of great and unknown danger. Moreover, as there can be no doubt about the impossibility of comparing the importance of a monument to that of a single human life, the result is that artistic monuments damaged by a serious earthquake are mostly lost to posterity.



3.65.



3.66.

We insist on this point, as it is impossible to forget the experience of Venzone, where works just begun on immediate static intervention on the Cathedral were interrupted by the September shocks.

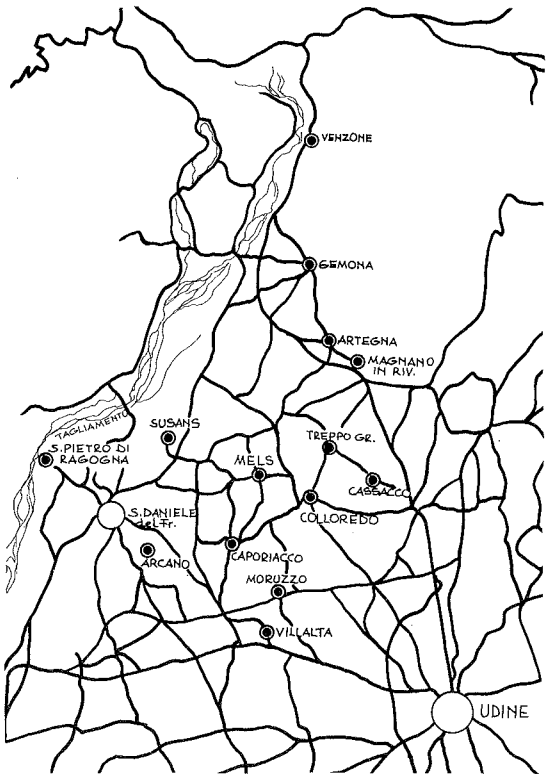
Even not taking into account these destructive shocks, which by pure good fortune did not claim any human victims, the grave risks run by the work teams during the recommencement of the interrupted work must not be forgotten.

Lastly, on the basis of this observation, it would be as well to reconsider the severity of certain opinions expressed towards those who, immediately after May 6, carried out initial demolition work on buildings of undoubted artistic interest. The buildings concerned were of great artistic merit and, theoretically, could have been salvaged but, from the above point of view, they represented serious hazards.

4. CASTLES

Several castles, recalling the past history of the Friuli region of Italy, still exist - feudal fortresses erected both to protect economic interests, linked to commercial traffic, and as refuges for the population during the repeated barbarian invasions which took place over the whole area.

With the aim of providing information on the damage suffered by this historical heritage during the recent earthquakes, we have prepared here photographic documentation on some of these castles. Obviously, this is not a complete survey: the castles in question are those encountered in the itinerary marked on the map as far as Venzone, with concluding observations on the walls and towers which surrounded the old centre of that town.



4.1.

4.1. Castle of S. Pietro di Ragogna

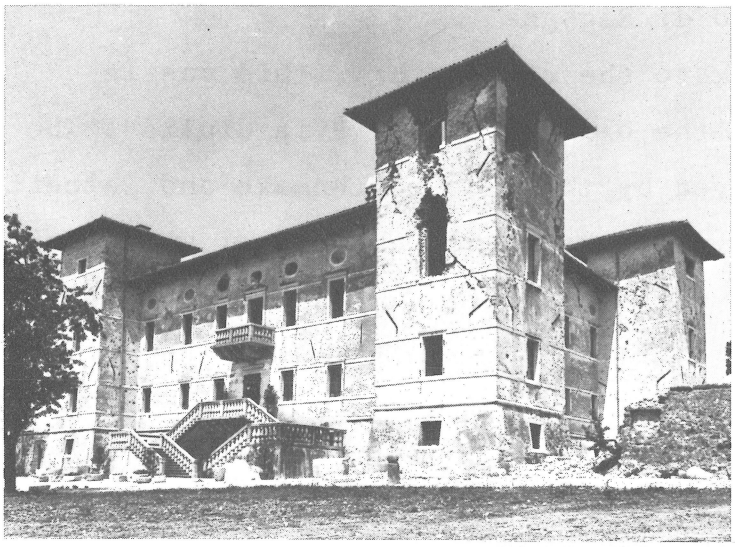
Perhaps dating back to the 6th century, this castle was probably erected on the old Roman road "Via Giulia". The castle was first destroyed by the 1348 earthquake and, albeit in a state of dilapidation, some ruins remained of which only portions of the outside walls are left.



4.2.
The remains of the ruins of the Castle of S. Pietro di Ragogna.

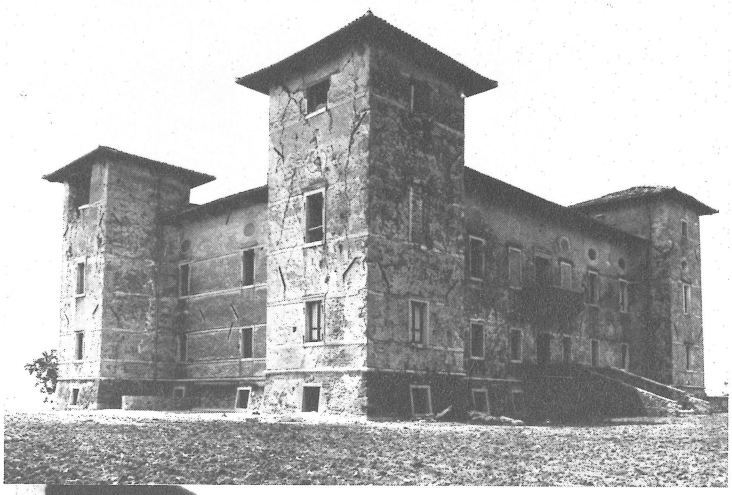
4.2. Castle of Susans (Majano)

The castle, in its present Florentine form, was built in 1636 by order of Fabrizio di Colloredo. Before the earthquake, it was being restored: of the four corner towers, two had been strengthened with concrete slabs; the other two were still hollow. The earthquake showed up the different behaviour of the four towers: the reinforced ones were hardly damaged at all, while the others underwent serious damage.



The Castle of
Susans

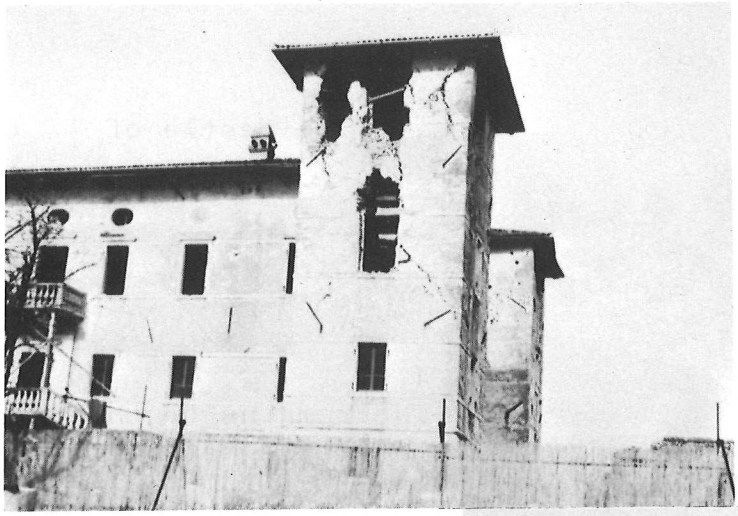
4.3. After the May shock.



4.4.
After the May shock.



4.5.
After the September
shock.

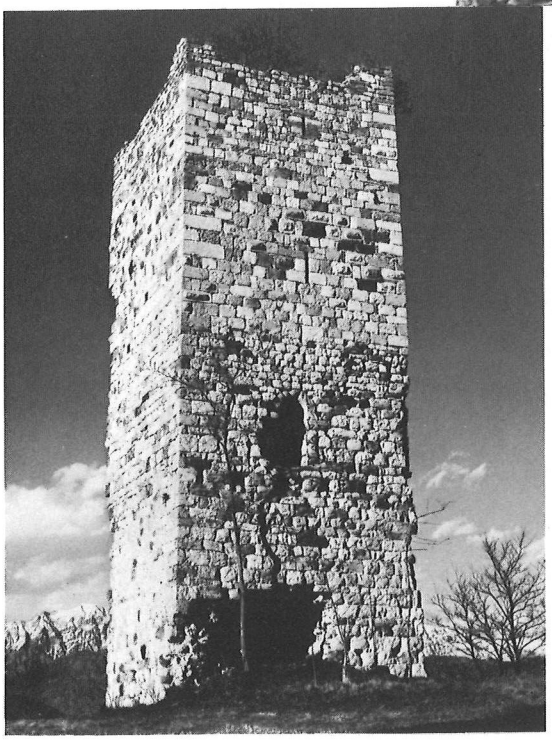


The Castle of
Susans

4.6.
After the September
shock.

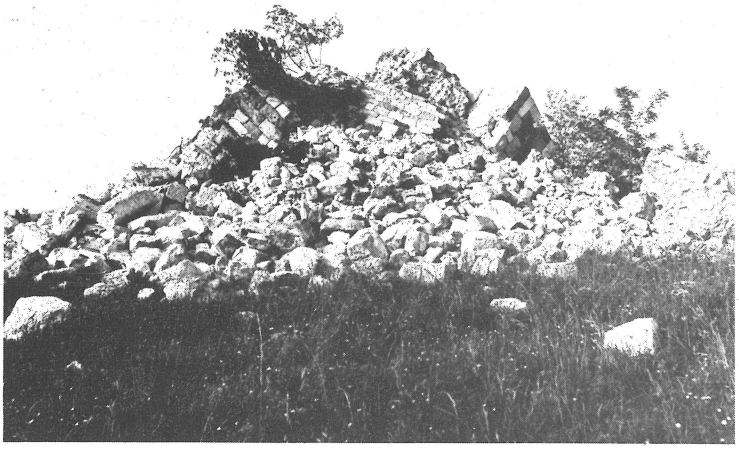


4.7.
After the September
shock.



The Castle of
Mels

4.8.
The tower before the
earthquake.



The Castle of Mels

4.9. The remains of the tower after the May earthquake.

4.3. Castle of Mels

This was the dwelling of the Viscounts of Mels, descendants of the Waldsee Swabians. Of the old building, one tower remained almost intact.

4.4. Castle of Colloredo

This castle, planned by Guglielmo dei Visconti di Mels, was begun in the 11th century and it seems that the fortress, in its present form, was already complete in 1310. It was severely damaged in the 1511 earthquake but was immediately restored and made larger. Its rooms contain frescoes and other

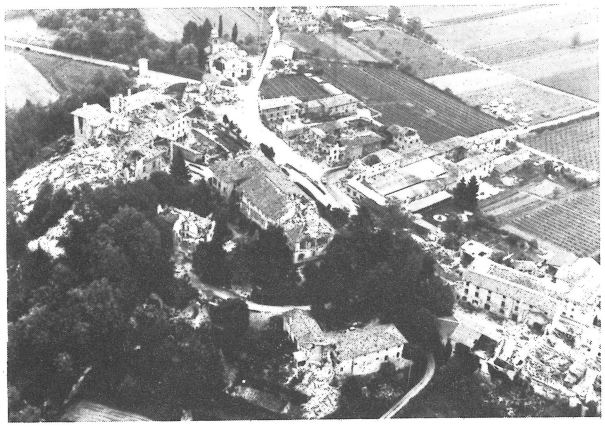


4.10. The Castle of Colloredo, before the earthquake.



4.11.
After the May shock.

The Castle of
Colloredo



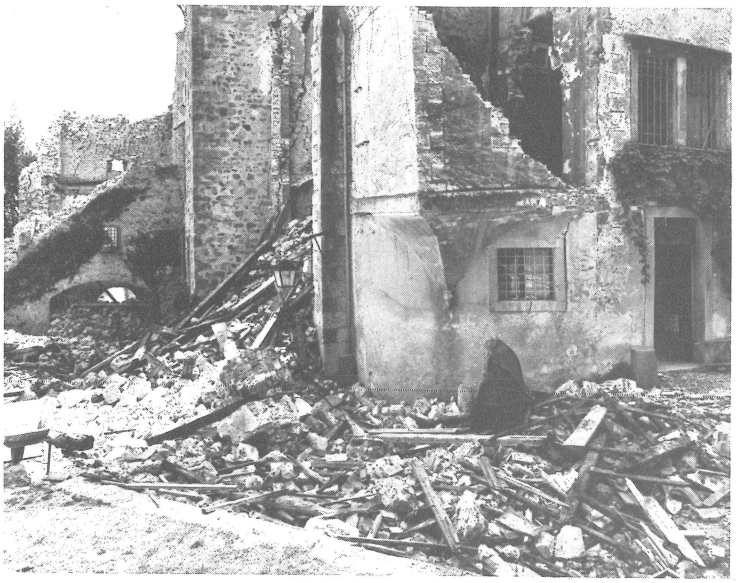
4.12.
Aerial view of the ruins of the castle.



4.13. Ruins of the main tower - exterior.



4.14. Ruins of the main tower - exterior.

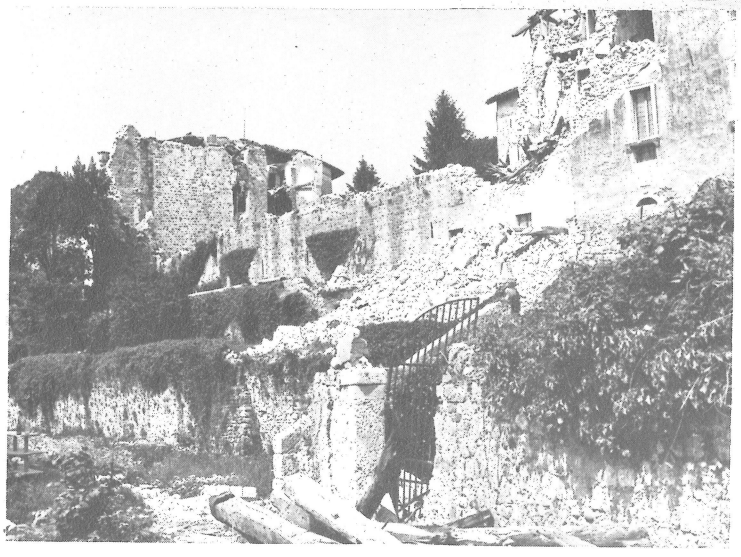


The Castle of
Colloredo

4.15.
Ruins of the main tower -
interior.



4.16.
Ruins of the castle -
detail of the façade.



4.17.
Ruins of the castle -
detail of the façade.



The Castle of Colloredo

4.18.
Interior view.

important artistic works. During the last century, it was occupied by the Italian writer Ippolito Nievo, who wrote there most of his masterpiece "Le Confessioni di un Italiano" (The Confessions of an Italian).

4.5. Castle of Cassacco

Erected before 1300, the physical state of this castle



The Castle of Cassacco

4.19.
After the May shock.

was already not good, and the recent series of earthquakes worsened its deterioration. The last restoration, although it was restored several times over the centuries, seems to date back to 1480.

4.6. Castle of Zegliacco(Treppo Grande)

The tower which collapsed during the earthquake was the only original structure of the old castle built before 1300. The present construction dates



4.20. Before the shock - ext.



4.21. After the May shock - exterior and interior.



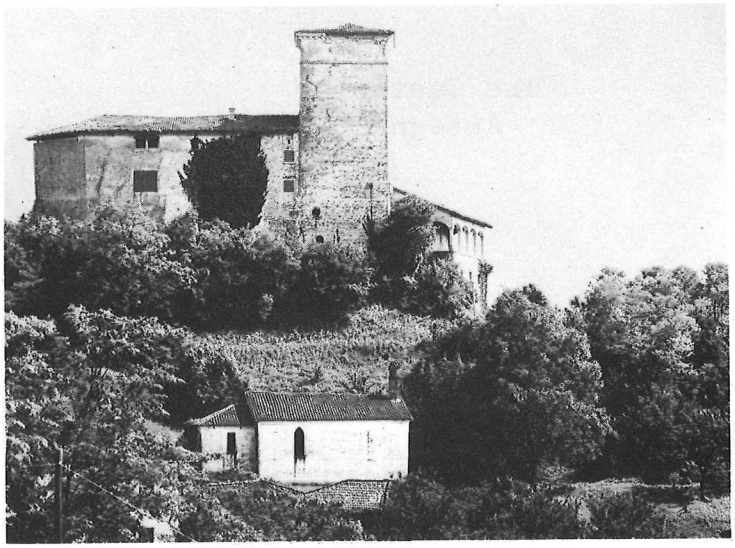
4.22.

to the 1600's. At the moment of the earthquake, it was in a very bad state of preservation.

4.7. Castle Di Prampero (Magnano in Riviera)

This castle was erected during the 12th century by the Seigneurs of Gemona, and then passed to the Di Prampero family, of German origin. It was destroyed in the 1511 earthquake and restored almost immediately afterwards. After a period of abandon, the castle and the small loggia added during the Renais-

The Castle of Prampero



4.23. Before the earthquake.



4.24. The ruins of the castle.

sance had recently been restored. The castle, already severely damaged by the 1976 earthquakes, was demolished for safety reasons.

4.8. Castle of Artegna

Artegna was originally built as a fortress by the Longobards, and belonged to various lords during the period of Friuli feudalism. The castle, which had been preserved carefully

The Castle of Artegna



4.25. Before the May shock.



4.26. After the May shock.





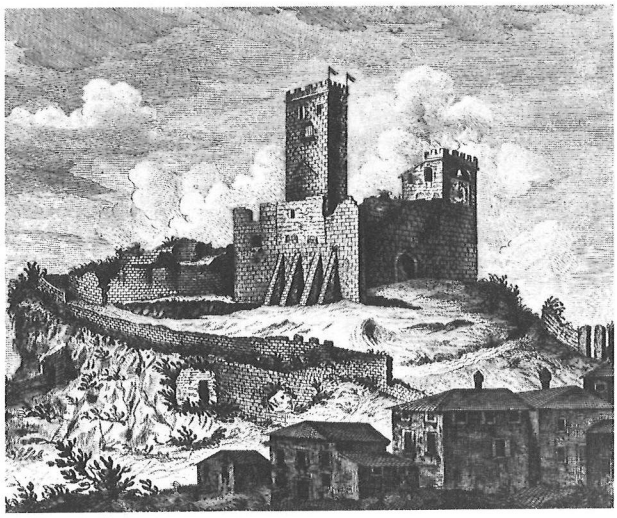
4.27. The demolition of the tower - Castle of Artegna. 4.28.

inside as well as outside in its original form, was severely damaged by the May earthquake. After September 15, its state was such that what remained of the tower had to be demolished.

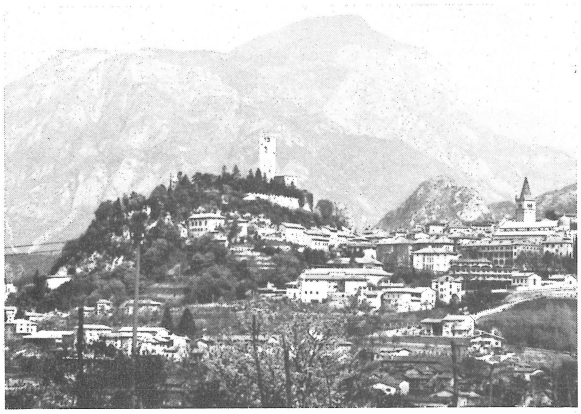
4.9. Castle of Gemona

In the past Gemona, a Longobard fortress, was constantly

The Castle of Gemona



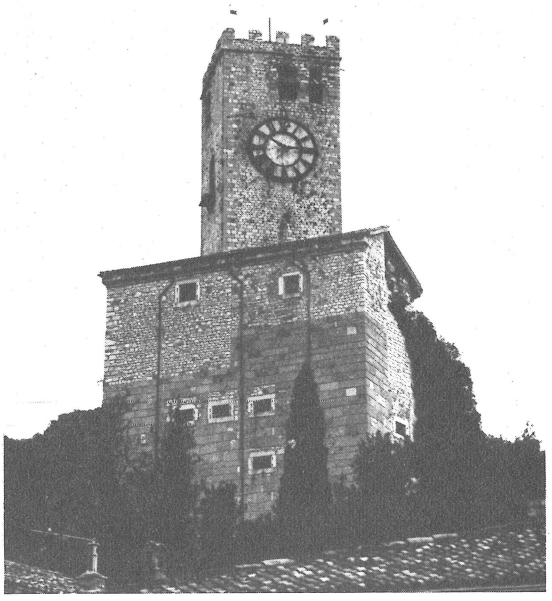
4.29. A 18th century print.

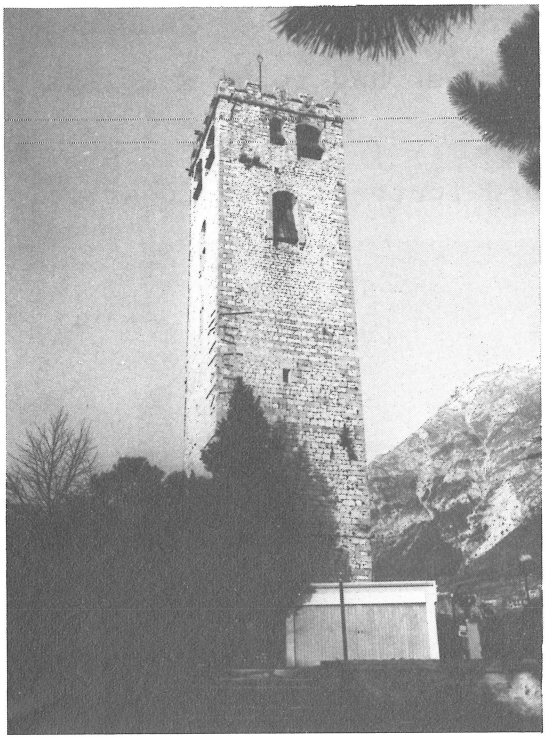


The Castle of
Gemona

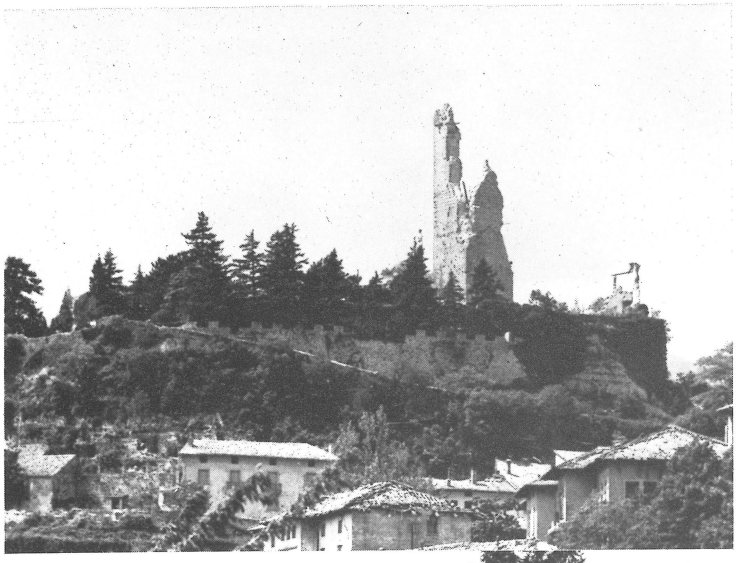
4.30.
Panorama

4.31.
Before the shock.





4.32.
Before the shock.



The Castle of Gemona

4.33.
The ruins of the castle after the May shock.



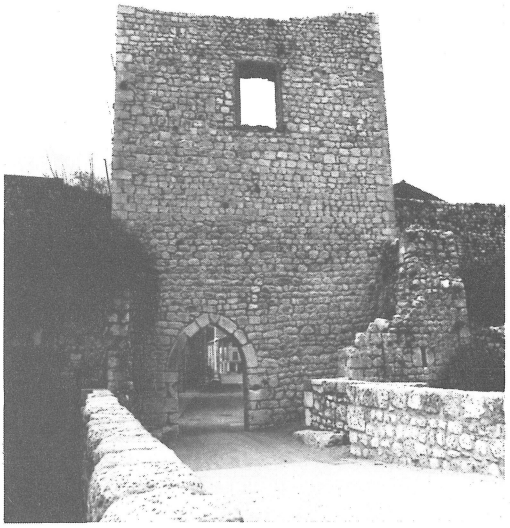
4.34.
The ruins of the castle after the May shock.

in conflict with the nearby town of Venzone, on account of tolls and duties. Of its castle - dating back to the year 611, part of the fortifications of the fortified town walls - only the tower remained. This collapsed during the May earthquake.

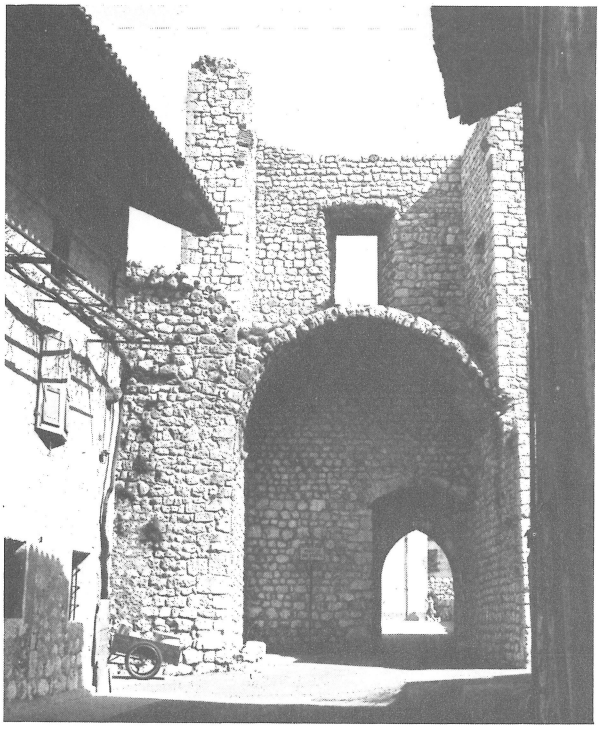
4.10. The Walls of Venzone

Venzone developed in size and importance during the first half of the 13th century under the government of Mels, and was encirled with walls in 1258. Inside a large irregular hexagon, the fortifications enclosed the 14 "insulae" of the old town

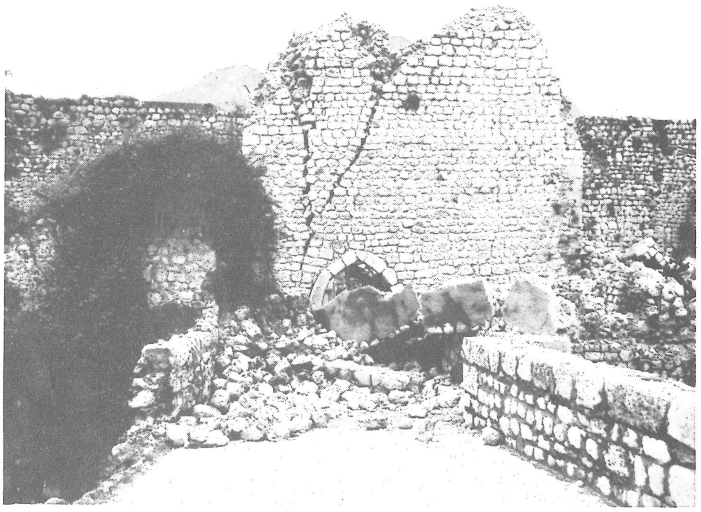
The Walls of
Venzone



4.35. The Tower of S. Genesio before the shock - exterior.

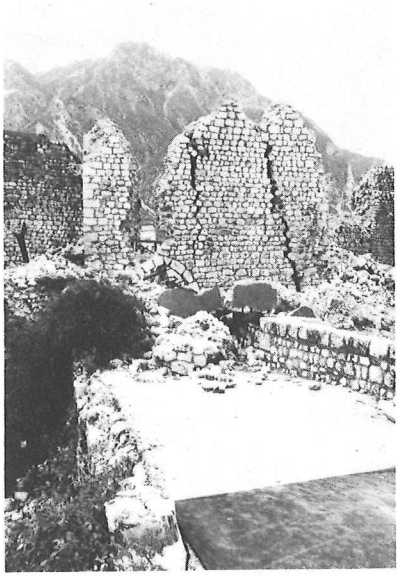


4.36. The Tower of S. Genesio before the shock - interior.



4.37. The Tower of S. Genesio after the May shock - exterior.

The Walls of Venzone



4.38.
The Tower of S. Genesio after the Sept. shock - exterior.



4.39.
The Tower of S. Genesio after the May shock - interior.



4.40.
The walls after the earthquake - detail.



The Walls of Venzone

4.41.
The walls after the earthquake - detail.

centre, composed entirely of hewn blocks of local stone cemented with lime mortar. These towers were destroyed during 1309 after a siege by the troups of the Patriach Ottob no de Radiis and were immediately rebuilt. The gate of S. Genesio dates back to this period. After 1470, with the increase of threats of invasion by the Turks, the walls were fundamentally restored and raised higher.

There were originally 16 towers, distributed at almost regular intervals along the perimeter of the walls. Until the May earthquake as many as 12 remained. Among these were three old gates to the town, of which only the Porta S. Genesio remained standing.

4.11. Observations

It is evident that all the buildings described up till now in this section have undergone severe damage, principally due to the inconsistencies of the walls built of local stone and cemented with lime mortar. After the earthquake, the walls of the castles, like those of the old private dwellings, showed diagonal and vertical fissures beginning at openings; many of these then split owing to the insufficient bonding of the mortar

5. NOTES ON THE EFFECTS OF THE FRIULI EARTHQUAKES ON PUBLIC AND PRIVATE BUILDINGS

5.1. Introduction

These notes comprise some observations on the effects of the seismic shocks occurring in Friuli between May 6 and September 15 1976 on public and private buildings made of masonry in historic town centres. The most important buildings, from the point of view of artistic and architectural interest, such as castles and public buildings of historical importance are not dealt with here.

We deal here with all those dwelling-places of old or even recent construction, made of materials generally obtainable on the site, such as stones or pebbles (sometimes substituted by bricks) and timber.

We also deal with restorations and improvements carried out after the date of building, such as the quite frequent substitution of the old floors and roof frames with structures made of reinforced concrete beams and tiles.

5.2. Analysis of building types

The dwelling-places of the town centres involved in the earthquakes generally fall into the following categories:

a) isolated houses of two or three storeys, often equipped with a large garret serving as a granary (see Photograph no. 5.1.).

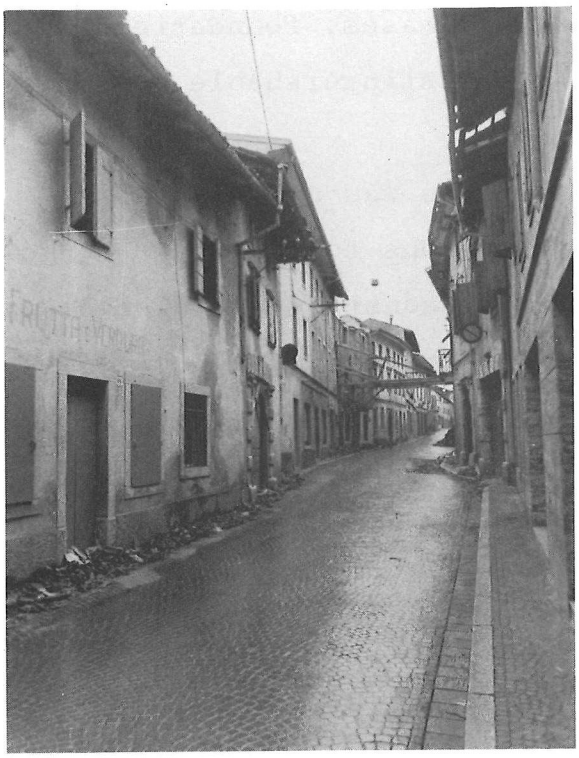
These are almost always built of a load-bearing structure in stone masonry, with floors and roof frames in wood.



5.1. Carnia. Casa Gattolin. As well as the portico and the loggia, note the presence of ties.

Most of these were poorly built many years ago, but have been frequently and often improved both architecturally and structurally (principally reconstruction of floors and roofs).

b) Terraced houses, two to four storeys high, in the principal streets and squares of the town centres (5.2).
These too have a load-bearing structure in stone masonry and floors and roof frames in wood, but they generally show relatively better building techniques.
They very often have a portico giving on to the street or square.



5.2.



5.3.

5.2. Artegna. A street in the town centre.

5.3. Laipacco. Villa Folli Tacelli.

As well as dwelling-places, there are also many examples of larger constructions, such as villas, immediately surrounding the town centre, and palazzi in the urban conglomerates (5.3).

5.3. Structural and architectural elements

Within the great variety of aspects presented by this spontaneous building, closely conditioned by economic and social motives, it is possible to find recurring building types, clearly due to environmental needs.

When considering these elements, we would like to show the possible incapacity of such buildings to withstand strong horizontal movements, when uniformity of behaviour, imputable to the elements themselves, was found.

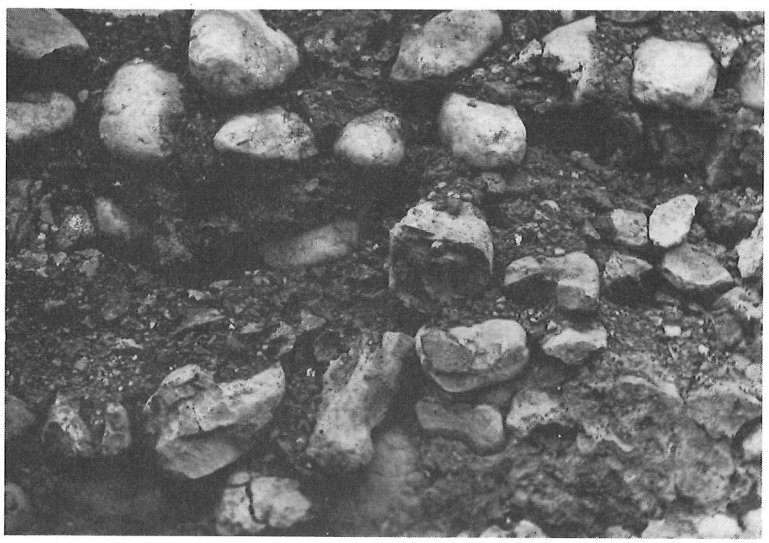
5.3.1. Foundations. In almost all cases, foundations are shallow, direct and continuous, and indistinguishable from the masonry above as regards thickness.

Excessive damage to the foundations was not generally found, nor were these the cause of lesions to the upper structures, except in the case of buildings constructed on slopes.

5.3.2. Masonry. The types of load-bearing masonry most frequently found are the following:

a) masonry of rounded pebbles almost always cemented with poor-consistency mortar, often degraded and even pulverized (natural mortar) (5.4).

This masonry is the result of the juxtaposition of several layers composed of small pebbles (5.5, 5.6). The outside layer is generally better composed, as regards choice of stone, and functions as a container for the internal elements (5.7).



5.4. Frequently-found masonry of rounded pebbles cemented with poor-consistency mortar.



5.5. Vito d'Asio. House. The load-bearing masonry is composed of layers.



5.6. Osoppo. The composition of the masonry is shown clearly here.

Stone elements linking the various layers transversally were frequently lacking.

This type of structure is shown most frequently in the old buildings.

b) Masonry of good-sized hewn stones with sharp corners cemented with mortar (5.8). Walls with better-quality external facings, functioning as containers for internal fillings of small, loosely-cemented pebbles, are not uncommon.

This type, less frequent than (a) above, is generally not plastered externally (5.9).

c) Brick masonry

This is present in more recent buildings (5.10), or in refacing or restructuring operations carried out on masonry of the preceding types (mixed masonry) (5.11).
The thickness of the load-bearing walls is generally considerable (always more than 30 cm) and, in most cases, not related to their static function of transferring the external vertical loads acting on the building to the ground.
Where serious damage occurred, effective linking between orthogonal walls and at the corners was almost always non-existent, so that in some areas the walls were simply leaning together.

The analysis of the damage suffered by a building coincides mainly with that of the behaviour of the load-bearing walls.

5.7.



5.8.



5.7. Forgaria. House.

5.8. Artegna. House. The masonry is of irregular good-sized stones cemented with mortar.

5.9.



5.10.



5.9. Imponzo. House. The upper part of the façade is not plastered.
5.10. Lesions are very regular in this brick masonry.

It should be emphasized that causes connected with the insufficiency of other structural elements, such as scarfing between orthogonal walls and lack of rigidity of floor and roofing, often contribute greatly to the formation of lesions in, or collapse of, the load-bearing walls. These causes will be examined later on, while we wish here to show how the masonry of the first and second types were not strong enough to resist the seismic shocks. Such behaviour can be attributed to the bad scarfing of the stone material supplied by the mortar, and to the lack of elements linking the various wall layers. Under this heading we can justify the "flaking" of walls with detachment of plaster and the outermost layer (5.12). The wall lesions generally show irregular trends, owing to the heterogeneity of their components (5.13).

5.11.



5.11. Gemona. Banca Popolare. On the upper right-hand side, the presence of bricks near the old stone masonry can be noted.

On the other hand, the behaviour of the brick walls was better. Since these were built more recently, higher-quality binders were probably used. The lesions noted showed far more regular trends, quite near the compression isostatic lines of homogeneous materials (5.10).

The behaviour of those walls which had been partially restored (mixed masonry) was not good, due to the diversity of components and the difficulty of good connections (5.11).

Serious lesions and collapse were frequent in all types examined.

d) Partitions. In the rare cases in which the partitions were not composed of load-bearing walls, they were made of plaster-covered wood (5.14) or, in more recent cases, of hollow clay tiles. During the earthquake, these elements generally shared the fate of the load-bearing structure. Sometimes they behaved in an individual manner, due to their precarious connection with the principal structure (5.15).

5.3.3. Roofs. With the exception of a very few cases of restoration carried out later than construction, we find

5.12.



5.12. Building at Gemona. Detachment of the plaster and the outermost layer.

5.13.

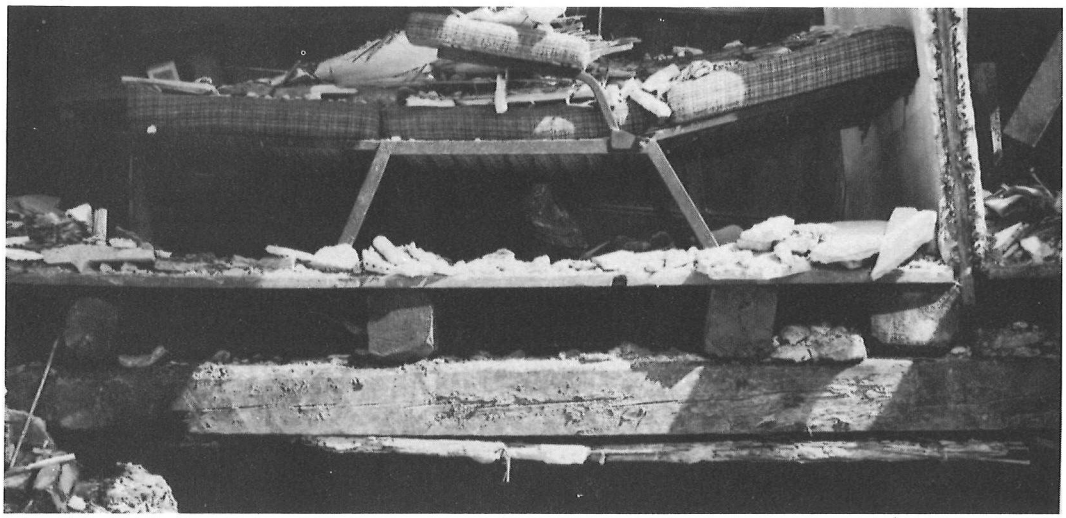


5.13. Artegna. House. Not very regular lesions can be seen to the left. A typical X-shaped lesion can be seen in the middle. The vertical cleft is due to the presence of a chimney.

saddle roofs, made of wood. The main roof frame is composed of beams tilted to their maximum slope, supported by the outside load-bearing walls and by a ridgepole. This last is supported by props supported in their turn by the main inside wall (5.10, 5.16, 5.17). However, it is not rare to find wooden trusses supported by the outside walls (5.18).

A secondary roof frame (purlins) links the principal adjacent elements supporting planking (not always present) and the roof covering (5.19). This last is rarely fixed to the underlying elements.

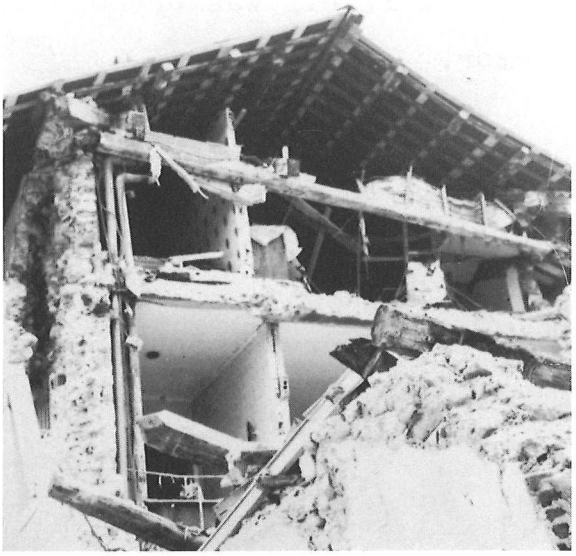
This structure, very rational as regards vertical loads, caused a high number of bad lesions and cases of collapse, owing to its lightness. The commonest type of damage, regard-



5.14.



5.15.



5.16.

5.14. Venzone. The structure of the wooden floor can be clearly seen.
5.15. Buia. The outside wall of this house, certainly more recent than the rest of the building, collapsed entirely.
5.16. Gemona. House with a wooden roof composed of two slopes.

ing the roofing only, was the slipping of the roof-tiles towards the gutters and their subsequent fall (5.20, 5.21). Instead, the secondary structures underwent less frequent damage: their detachment from the main structure was generally found (5.21).

Instead, far more serious were the consequences when the structure of the roof damaged or involved the underlying masonry in its collapse. This happened above all when the number of main beams was small, so that heavy loads were concentrated over small supports, and bad lesions subsequently opened.

A further negative factor was that the scarfing of the main beams with the perimetral walls was very rare or in any case inefficient. Together with the insufficient rigidity of the roof structure, this often led to the collapse of the gables (5.22) and the walls of the last storey (5.23), when the wall itself was orthogonal to the direction of the earth tremor.

The facings of the roofs with reinforced concrete structures were stable in those cases where the link between the

5.17. Venzone. House. The ridgepole is to be noted.





5.18. Artegna. House. The roof is made up of wooden trusses.

roof frames and the perimetral walls was good (5.24). Instead, cases in which the lack of these scarfings caused strong shifting of the roof were not infrequent (5.25, 5.26, 5.27).

5.3.4. Floors. These were generally made of wood: a simple or double frame of beams with their ends fixed into the load-bearing walls supports planking and, rarely, solid flooring (5.14, 5.28). These have often been substituted by tile and reinforced concrete structures.

The wooden floors are generally light-weight structures of little rigidity in their plane, weakly linked along their perimeter with the load-bearing walls.

The floors of this type did not suffer direct damage in the earthquake. Their collapse was only concomitant with that of the load-bearing walls around them, the beams becoming disloged from their housings in the walls (5.28, 5.29, 5.30).



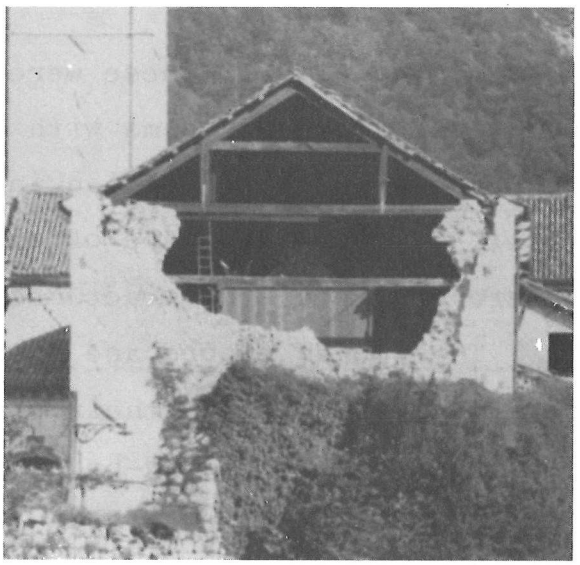
5.19. San Daniele. Villa. The fall of the roof-tiles has brought to light the roof frame.



5.20. Trasaghis. The slipping of the roof-tiles can be seen.



5.21. Carnia. Houses. A typical consequence for roofs after the earthquake can be seen: fall of tiles and partial detachment of the secondary roof structure.



5.22. Venzone. House. Collapse of the gable owing to almost non-existent linking between masonry and roof.

However, it should be noted that the insufficient rigidity of the floor in its plane constituted weakness of the whole structure (5.31). The opposite effect took place in the tile and reinforced concrete floors, when the whole of the floor/wall perimeter was well linked. In the other cases, instead, behaviour was similar to that of the wooden floors, with complete detachment and collapse of walls along the non-linked sides (5.10).

5.3.5. Arches, columns and vaults. Arched structures, very frequent in the area in question, are generally made of

5.23.



5.24.



5.23. Buia. Town Hall. Collapse of the walls of the last storey.

5.24. Gemona. House. The roof, with its reinforced concrete structure, determined the stability of this building.

stone (5.32) or brick quoins (5.33). They abut on monolithic columns with a capitol (5.34) or on rectangular-section pilasters. Steel ties are generally present, to absorb the horizontal thrust, or more massive corner piers, to support the thrust of the last arch (5.62).

The predominant use of these structures is in the construction of arcades and entrances to buildings in the town centres (5.35). They can also be found in dwelling-placesas

5.25.

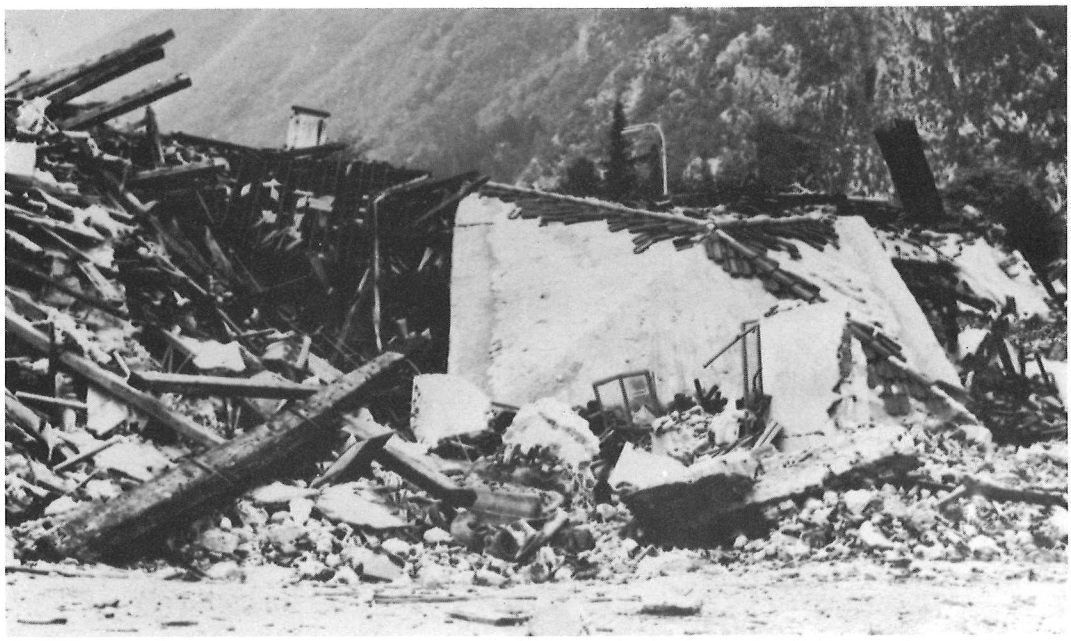


5.26.



5.25. Buia. House. The recent reinforced concrete roof, insufficiently linked to the load-bearing walls, has evidently shifted.

5.26. Venzone. House. The last storey, certainly of more recent construction than the rest, has load-bearing walls of brick masonry and a tile and reinforced concrete roof This shifted strongly, due to insufficient linking with the perimetral walls.



5.27. Gemona. House. The reinforced concrete roof has remained monolithic.

loggias (often together with hanging domes (5.1, 5.34). Isolated arches are frequently used as doorways in boundary walls.

It was only for this last type that the earthquake was totally destructive: the topstones always fell, since the piers underneath them opened (5.36).

In most other cases, in fact, the behaviour of the arches was good, at least during the first shocks. This did not happen in September, when some arches were slightly damaged.Even small lesions were in effect sufficient to damage good arch statics, if the arch itself were stressed by strong horizontal forces. (5.61, 5.62, 5.64).

In some cases, the May shocks caused damaged to the springers (5.38); in others partial collapse resulted, due



5.28.

Gemona. House. In the foreground the composition of the masonry can be seen. The structure of the wooden floors, and their linking to the perimetral walls, is clear.

5.29.

Venzone. Houses. The collapse of the roofs and floors happened because of the collapse of the walls to which their principal beams were joined.

5.28.



5.29.

to the incapacity of the overlying walls to withstand severe loads in their planes (5.34). In this last case, however, the behaviour of the complete, restored structures was positive during the September shocks (Vecchiarutti palazzo, Gemona).

A particularly interesting type of lesion took place when openings were placed near the arches. The lesions branching from them move towards the extrados of the arch, and are arranged according to compression isostatic lines, sometimes damaging the structure of the arch itself (Gurisatti palazzo, Gemona) (5.62).

Arches and vaults inserted into walls singly generally stood up well to the stresses, even if they were surbased, (5.39), while the cross-vaults in the loggias of dwelling-places were badly damaged, sometimes collapsing owing to the



5.30. Venzone. House. The elements making up the floor and the housings of the principal beams are to be noted.



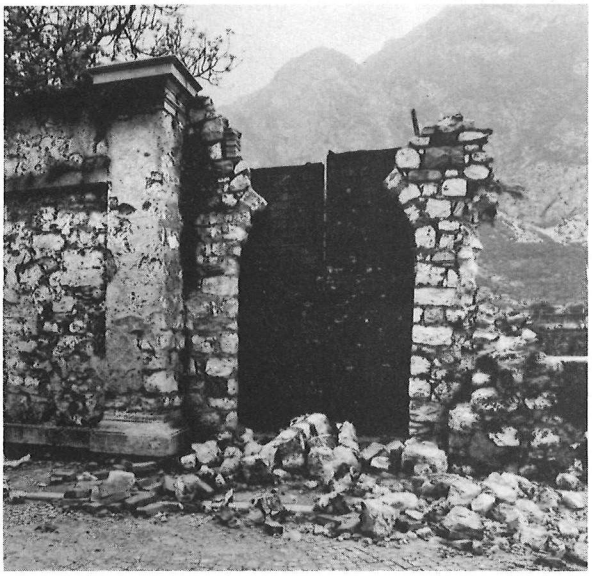
5.31. Ospedaletto di Gemona. House.

absence of steel ties (5.37).

5.3.6. Scarfings, strengthening elements and ties. In order to emphasize the static importance of these elements, it would be useful to use an elementary scheme showing the behaviour of the wall-structures of a building, based on immediate considerations. We believe in fact that it is useless to refer to dynamic analysis for inhomogeneous structures such as those in question.

The constructive elements most subjected to the horizontal forces of an earthquake are the walls. As we have already seen, these are very thick in this case, and of high specific density. Instead, floors and roofs are less important, seeing that in most cases they are of wood and therefore lightweight.

On this occasion too, as in many other cases, the walls were particularly suitable for the transfer to the foundations of the horizontal forces acting in their planes. A free wall behaves as a very rigid and resistant structure when subjected to horizontal forces in its own plane. Instead, if the horizontal forces are orthogonal to the free wall, it is very easily deformed and lacks resistance, and may even collapse,

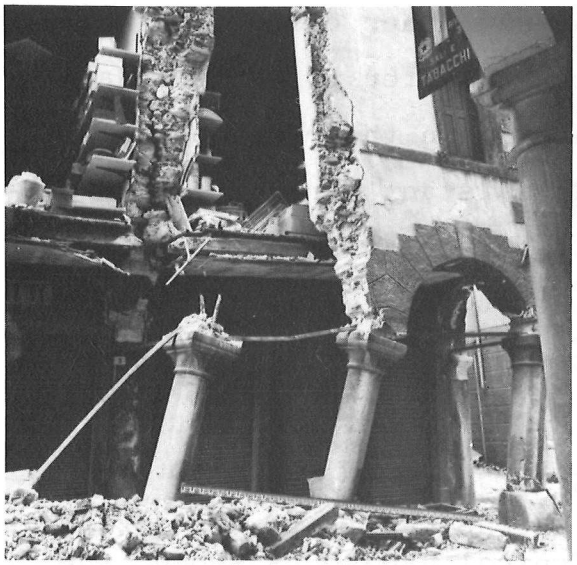


5.32.
Venzone. Walls in Via Nazionale. Typical damage is shown: the topstones of the arch have fallen.

5.33.
Venzone. House. The top of this brick arch is severely damaged.



5.33.



5.34.



5.35.

5.34. Gemona. Casa Vecchiarutti. The brick arches rest on monolithic columns of stone, and are strengthened with steel ties.

5.35. Gemona. Via Bini. Casa Vecchiarutti can be seen on the left-hand side of this porticoed street, an interesting example of the effects of the earthquake. The collapse of the first two porticoes, probably due to the detachment of the masonry above and the consequent breaking of the tie, was the cause of the spectacular detachment of the next two porticoes, as far as the corner of the building.



5.36. Gemona. Note the break in the top of the arch in the wall.



5.37. Gemona. House. The partial collapse of the façade and floors shows the structure of the brick hanging vaults.



5.38. Buia. The Town Hall Square. There is a very evident lesion of the arch, near the corner pilaster.



5.39. Gemona. This very sub-based vault has taken the stresses in an excellent way.

if the forces are strong.

We consider here the load-bearing walls (in two orthogonal directions) making up a building, hypothesizing for the moment the absence of floors and roof. The walls subjected to normal horizontal forces are influenced by the behaviour of the orthogonal walls (which are submitted to forces in their planes). This effect decreases with distance from the intersection lines of the two wall systems, and depends essentially on their good linking. If these links are missing, the walls behave as if they were free (5.40, 5.41, 5.42).

Bearing this in mind, it is clear that the presence of floors which are slightly deformable in their own planes means improved static behaviour of the whole ensemble. However, this is true only if the links with the walls are good for the whole

of the perimeter. Rigid roofs for the walls of the last storey carry out the same function (5.24).

In a properly constructed building, there is therefore a transfer of all horizontal forces to the walls parallel to the direction of the shock, and thus to the foundations. The corners and the joints of the orthogonal walls, together with the areas near them, are subjected to very intense stress.

Many seriously damaged or destroyed houses had transversal and façade walls which were simply leaning together (5.23, 5.43). In other cases, if the jointing had been carried out with large squared-off blocks, there was no effective link between these blocks and the adjacent stone masonry. In this last case, the result was a lesion immediately near the scarfing (5.44, 5.58).



5.40. Gemona. House.



5.41.



5.42.

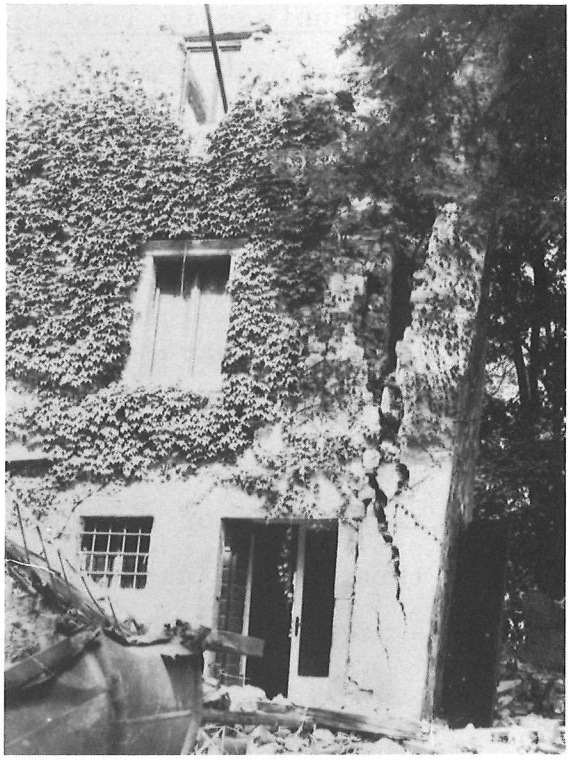
5.41. Gemona. The Old Hospital. The large lesion in the corner can be seen.

5.42. Gemona. House. The effects of the load-bearing façade collapse are clear: the fall of one pitch of the roof and partial collapse of the floors.

In a similar way, the passage from one type of masonry to another, different type, either in the same building, or in terraced buildings, as in the town centre buildings, forms weaknesses. Here, good scarfing being missing, very large vertical lesions sometimes appeared in the masonry of inferior quality, sometimes leading to the collapse of the walls (5.45).

As we have already said, the contribution of floors and roofs to the stability of the building was almost always negligible, since the elements were not rigid or well scarfed to the walls (5.46). In cases where adjacent buildings were





5.43. 5.44.

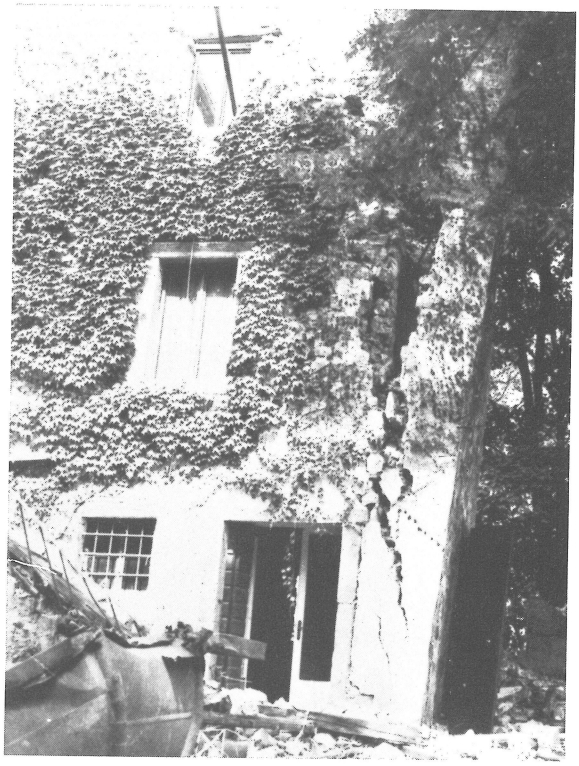
5.43. Venzone. House. The weak scarfing between the side wall on the right, and the façade wall, led to the collapse of the right side of the façade.

5.44. Buia. House. The detachment of the right-hand wall near the corner is due to weak scarfing between the corner and the left-hand wall.

were well connected, the structures behaved well. Similarly, ties - often present in old buildings in places where earthquakes were more frequent in the past - served as good strengthening (5.17, 5.47). The presence of ties in a building, although important for the safety of the building itself, was not generally sufficient to avoid the formation of serious damage (5.58).



5.43.



5.44.

5.43. Venzone. House. The weak scarfing between the side wall on the right, and the façade wall, led to the collapse of the right side of the façade.

5.44. Buia. House. The detachment of the right-hand wall near the corner is due to weak scarfing between the corner and the left-hand wall.

were well connected, the structures behaved well. Similarly, ties - often present in old buildings in places where earthquakes were more frequent in the past - served as good strengthening (5.17, 5.47). The presence of ties in a building, although important for the safety of the building itself, was not generally sufficient to avoid the formation of serious damage (5.58).

5.3.7. Openings in load-bearing walls. These represented very weak zones, and sometimes gave rise to large lesions. As well as visible openings (doors and windows), we must also add vertical channels (principally chimneys) (5.48) and previously walled-up doors and windows (5.49).

Serious damage was done particularly in the case of walls parallel (or quasi-parallel) to the direction of the earthquake, i.e. subjected prevalently to stress in their own planes.

The first type of damage regards elements making up the openings themselves: detachment and fall of balconies, rotation, breaking and fall of lintels and stone piers (5.50, 5.51).

Damage to walls due to the presence of the openings themselves was far more serious. The most frequent types of lesion



5.45.
Houses at Gemona.

were:

a) vertical lesions for one or more storeys, generally from one window to another (5.52, 5.53, 5.54);

b) lesions, often crossed, running at about 45° from the openings of one floor to those of the upper one (5.57, 5.58, 5.65);

c) crossed lesions at 45° between adjacent openings on the same storey (5.55, 5.56);

d) lesions from corner openings and the extrados of arches (5.62);

e) lesions from the openings of the last storey to the roof (5.52, 5.59).

Of particular interest are the 45° lesions, due to shearing stress. These occurred in the areas of wall between openings on different storeys or between openings on the same floor, whichever case represented less resistance to stress. The presence of offsets or floors, rigid and well linked to the walls, prevented the formation of lesions passing from one floor to another.

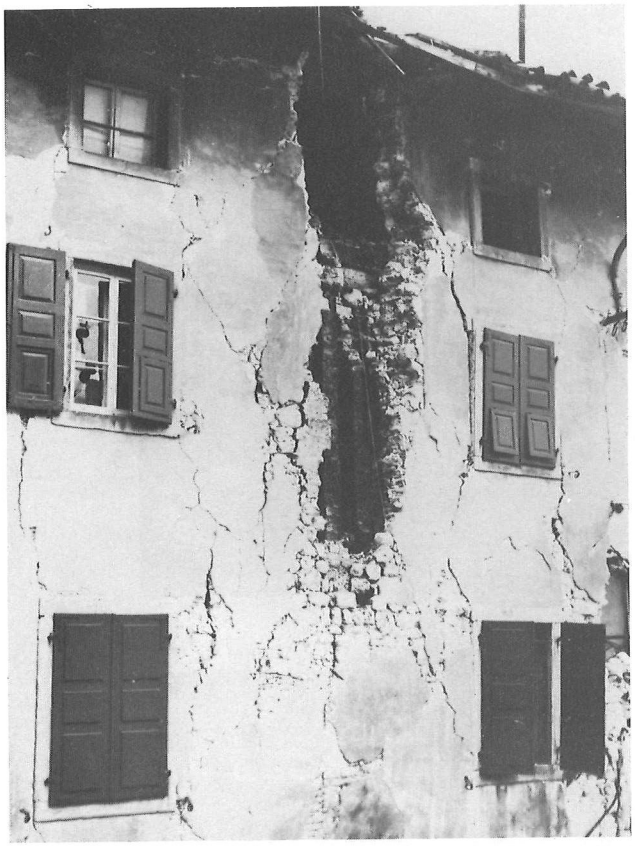
It is interesting to note how the lesions between openings on the ground floor are particularly frequent and large in size, both owing to the greater size of the openings themselves, and to the greater shearing stress (5.55).



5.46. Venzone. The Town Hall Square.



5.47. Cavazzo Carnico. The Stroili palazzo (1765). The ties certainly protected the building against more serious damage.



5.48. The large lesion visible in the centre is certainly imputable to the presence of the chimney.

5.48.

5.49.



5.49.
Artegna. House. The detachment of the external layer of masonry in stone has brought a walled-up window to light.



5.50

5.51.



5.52.



5.50. Venzone. Banca Popolare. Note here the lack of the left pier and the rotation of the lintel of the upper left-hand window, and the shifting of the lower left-hand window pier.

5.51. Venzone. Town Hall.

5.52. Tarcento. The Prefecture. Lesions run vertically along the windows.



5.53. Gemona. House in Piazza del Ponte. The vertical lesions running from the corners of the windows and the door are clearly visible.

5.53.

5.54.



5.54. Vito d'Asio. House. Note the vertical lesion and the central part of the façade, rotated and obviously out of plumb.



5.55. Gemona.House. Note the severe lesions on the ground floor. Also the large tilted fissure, ascribed to shearing stress, and the consequent shifting of the pier of squared-off stone blocks, together with the breaking of the lintel of the left-hand window.



5.56.

5.57.



5.56.
Gemona. Schools. Forty-five-degree lesions are visible between the windows of one floor and, less accentuated, between those of the first and second floors.
5.57.
These lesions are more or less at 45°, starting from corners to openings.

5.4. Conclusion

At the conclusion of these notes, we show a synthetic but interesting photographic series on two buildings in Gemona: the Gurisatti palazzo, and the Pontotti palazzo (head office of the Banca Popolare di Gemona). Built respectively in the 15th and 16th centuries, they are significant examples of the architectural and cultural heritage of this part of Italy, now irremediably lost.

The following documentation shows most of the constructive elements examined here, and the damage they underwent.



5.58.

Palazzo Gurisatti, Gemona.

5.58. In May.
5.59. In July, with shoring.

5.59.





5.60.

Palazzo Gurisatti, Gemona.

5.60. Tuesday, September 14th.
5.61. In October.

5.61.





5.62.



5.63.



5.64.

Palazzo Gurisatti, Gemona.

5.62. Note the corner pilaster, more massive, and the trends of the lesions near the arch.

5.63. The lesions on the façade, in May.

5.64. The façade on September 14.



5.65.



5.66.

Gemona. Palazzo Pontotti (16th century), now the offices of the Banca Popolare di Gemona.
5.65. In May.
5.66. In July, with shoring.
5.67. On September 14.
5.68. In October.



5.67.



5.68.

6. CHURCHES

6.1. Introduction

Churches represent perhaps the most conspicuous part of the history, artistic and environmental heritage of Friuli, both as regards their distribution over the provinces of Udine and Pordenone hit by the 1976 earthquakes, and their dates of construction, which go back over the last thousand years.

In the single province of Pordenone, various types of damage was suffered by 260 churches, and in the province of Udine 58 were destroyed and a further 256 seriously damaged; 341 have lesions. Apart from their architectural style and building techniques which naturally changed over the centuries, from the most modest votive chapel to the most magnificent cathedral, these churches almost always contain pictures, writings and sculptures which allow us to construct or reconstruct important episodes in the history and art of a country or region. It is therefore important that the restoration of these buildings, which represent an extremely valuable collective, cultural and economical asset, be carried out wherever possible. Furthermore, it is essential that similar buildings, wherever they may be, must be protected in the future from the experiences recently undergone in Friuli.

Particular care on the restoration and recovery of single buildings is moreover of great topical interest both in Italy and throughout Europe. The Council of Europe recently proposed a series of interesting initiatives (1975: European Year for Architecture) to safeguard and recover existing artistic treasures with the aim ofgiving old buildings a "vital function in



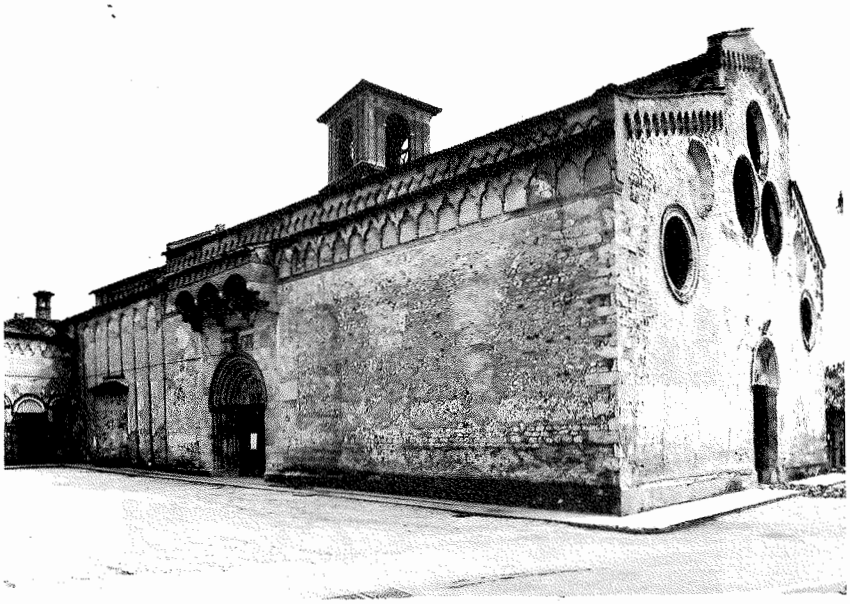
6.1. Colle d'Arba. Parish church. Pebble walls. The church and its bell-tower had to be totally demolished artificially.

contemporary society". We believe it would be suitable to review these initiatives in the light of anti-earthquake precautions.

6.2. General considerations

The churches of Friuli, like its civil buildings, show different building styles according to their dates of construction, greater or lesser importance of the building itself, and the state of the economy at the time of construction, etc.

The walls of the churches are generally made up of hewn stone or variously-sized stones, not always regular in shape, bonded with mortar which has deteriorated with time and is in any case inadequate (Photograph no. 1). Double mortar walls



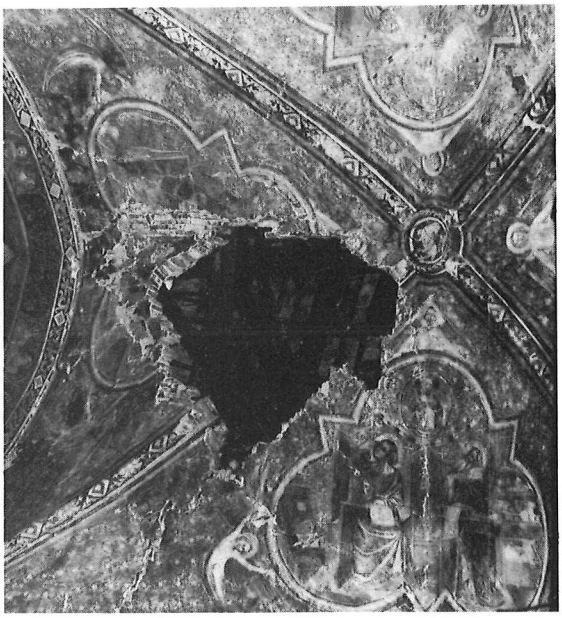
6.2. Spilimbergo. The Cathedral. Note the vertical lesion on the long wall near the corner.



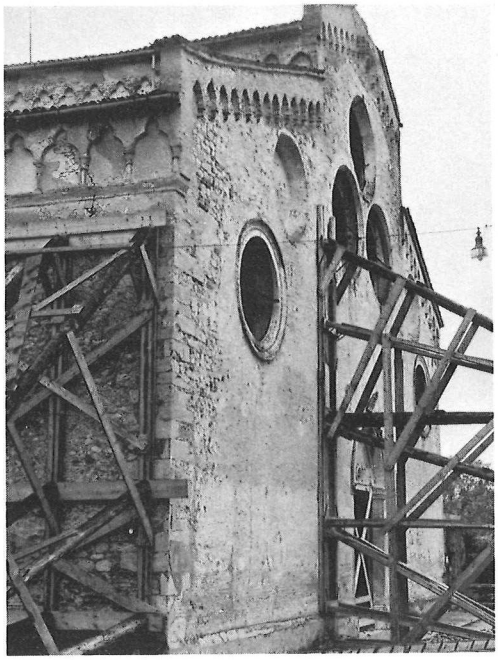
6.3. Spilimbergo. The Cathedral. The tip of the façade has collapsed.



6.4. Spilimbergo. The Cathedral. Lesions at the point of attachment of a side chapel.



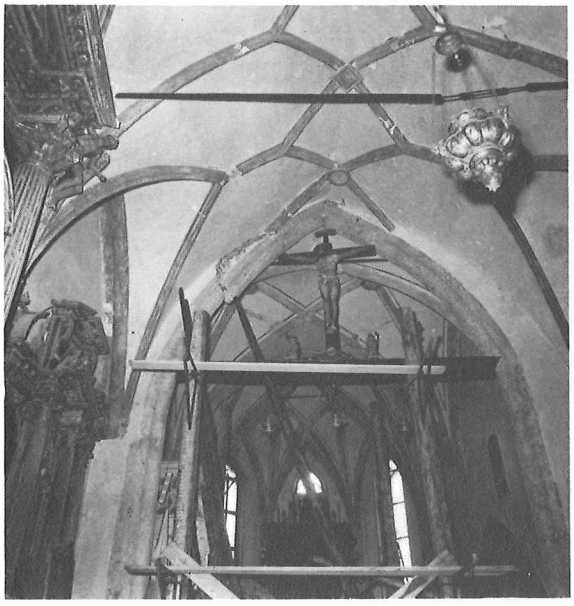
6.5. Spilimbergo. Cathedral. Lesions and partial collapse of the frescoed cross-vault.



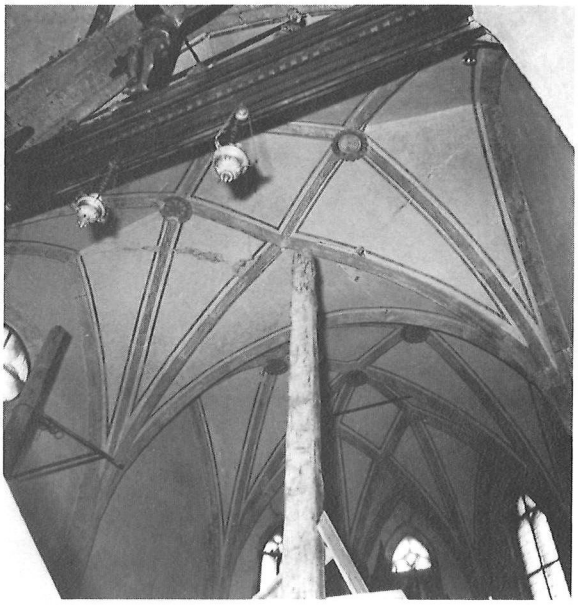
6.6. Spilimbergo. Cathedral. Shoring after September 15 1976.



6.7. Sammardenchia. Parish church. Typical roof structure with wooden beams.



6.8. Zuglio. Pieve di S. Pietro. Cross-vaults.



6.9. Zuglio. Pieve di S. Pietro. Cross-vaults.



6.10.
Valeriano (Pinzano al Tagliamento). Church of S. Maria dei Battuti. Barrel vault.

filled in with pebbles are widespread. The better-quality buildings have walls (sometimes only the main façade) made of hewn and dressed blocks of stone (16, 17, 39, 40, 42, 55, 56 and 59). The use of tiles is more recent and rarer. There are also some old examples of the use of bricks (2-6). Common to all churches is the presence of walls which are long in relation to their thickness, and slender in relation to their height, often lacking in normal wall elements which would otherwise provide stability against shifting movements of the walls themselves.

The most common type of roof is made of wooden trusses holding up a light roof frame, and the roofing itself of curved tiles (7, 11, 27, 28). There are also some roofs made of cross-vaults or barrel vaults (8, 9, 10).

The plan and elevation sizes of the churches are, of course, almost always large with respect to those of the nearby

private dwellings; but the openings (windows, rosettes) are definite points of weakness which reach the point of an intaglio effect at times, so that the"box-like" structure of the whole is lacking in stability. Logically enough, earth tremors are particularly felt by such weak elements, showing up the deficiencies of building materials or techniques in the load-bearing structures.

We therefore believe it is particularly interesting and useful to show - by means of adequate photographic documentation - the most frequent types of lesions found in the Friuli churches, so that future restoration or preventive works can more easily be carried out on buildings of national importance.

6.3. Survey of churches

One hundred and thirty churches were taken into consideration in this study, all contained over an area of approx. 2000 sq.km. worst hit by the earthquakes. Preliminary surveys were made one day after the first earthquake on May 6 1976, and later extended after the September 15 shocks. The state of the buildings damaged on May 6 worsened after September 15, principally or only in the area inside Venzone, Gemona, Artegna, Osoppo and Bordano; further damage was slight in the more outlying areas. It is interesting to see the progressive settling in some churches in the former area, e.g. some churches in Gemona: Madonna delle Grazie (15th century), (11, 12, 13, 14, 15); SS. Nome di Maria in Fossale (16th century), (16, 17, 18, 19, 20); S. Antonio (21, 22, 23). For the church of S. Chiara in Venzone, we have documentation after May 6 and



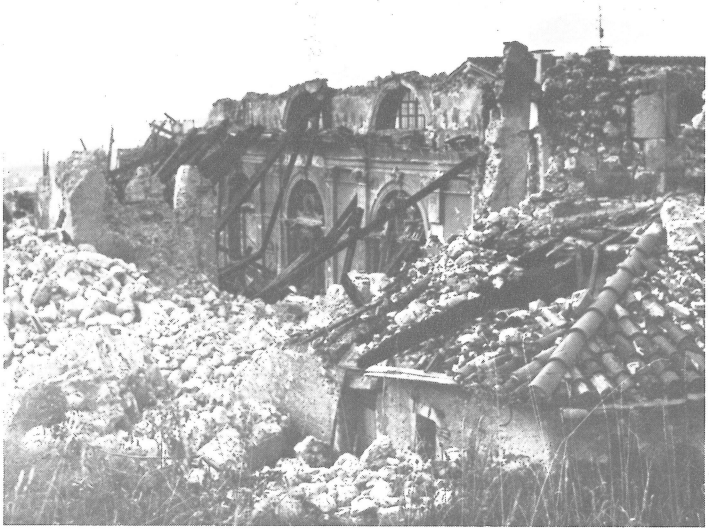
6.11.
Gemona. S. Maria delle Grazie. May 6 1976.



6.12.
Gemona. S. Maria delle Grazie. Situation after the 2nd shock of September 11, 1976.



6.13.
Gemona. S. Maria delle Grazie. Situation after September 15, 1976.



6.14. Gemona. S. Maria delle Grazie.
Situation after September 15 1976.



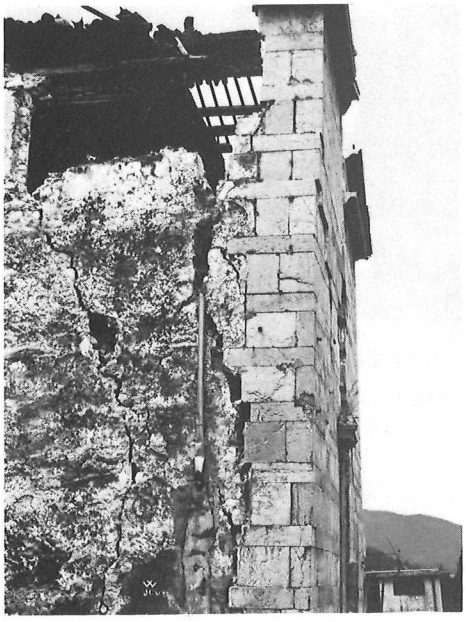
6.15. Gemona. S. Maria delle Grazie.
Situation after September 15, 1976.



6.16.
Gemona. SS. Nome di Maria in Fossale. After May 6 1976.



6.17.
Gemona. SS. Nome di Maria in Fossale. After September 15, 1976.



6.18.
Gemona. SS. Nome di Maria in Fossale. After September 15, 1976.



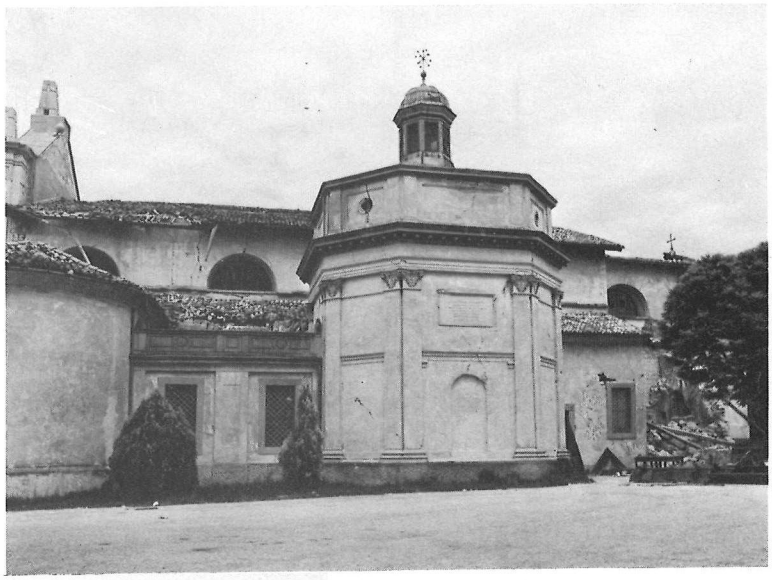
6.19. Gemona. SS. Nome di Maria in Fossale. Side after May 6,1976.



6.20. Gemona. SS. Nome di Maria in Fossale. Side after September 15, 1976.

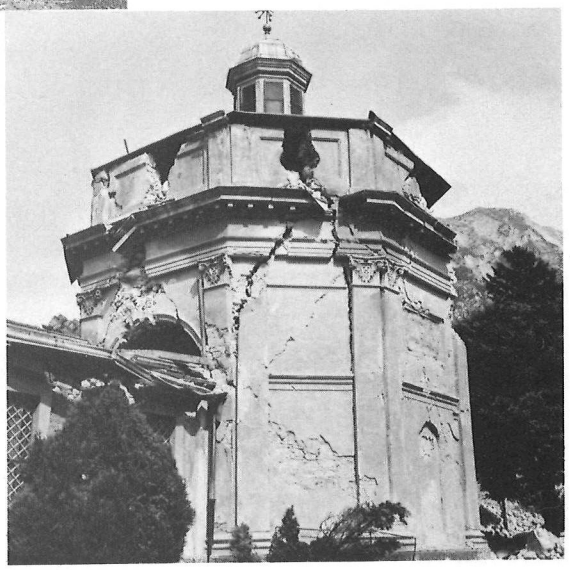
at the present time (24, 25, 26), and for the church of S. Giovanni Battista in Venzone, now partially demolished on purpose, for reasons of safety (27-31).

6.21.
Gemona. S. Antonio. After May 6, 1976.





6.22.
Gemona. S. Antonio.
After September 15,1976.



6.23.
Gemona. S. Antonio. After September 15, 1976.



6.24.
Venzone. S. Chiara after the May shock. Note the split between the façade and the side wall leaning outwards. There are also lesions to the bell-tower and other walls.

6.25.



6.25. S. Chiara. After the September shocks. In the background, Mt. S. Simeone.



6.26. Venzone. S. Chiara after September 15,1976.

6.27.
Venzone. S. Giovanni Battista. After May 6,1976.





6.28.
Venzone. S. Giovanni Battista. After May 6,1976.



6.29.
Venzone. S. Giovanni Battista. November 1976. The church has been partially demolished artificially.

6.4. Damage

We have already seen how the size of the churches renders them particularly vulnerable to earthquakes: construction techniques are faulty, and structural elements are almost insufficiently bonded and without links capable of tolerating the kind of movement encountered during an earthquake. We have also seen how the walls are particularly weak owing to their constitution, and are incapable of absorbing forces acting along their own planes. Without descending into detail, we have tried to classify here the types of damage encountered in the buildings examined, by means of photographs.

4.1. Lesions to walls insufficiently resistant along their planes

These were found in 60% of the churches examined. A large X-shaped lesion (32) (Zomeais) can be seen in the façade, starting at the corners of the door. The longitudinal façade too is damaged: the frontal is detached, with the formation of a cylindrical hinge along its link with the wall. 33 shows the oratory of the Angelo at Clauzetto. 34 (Ciseriis) shows lesions to the façade, together with the collapse of the gable and detachment of the longitudinal wall; 35 shows lesions to the longitudinal walls of the same building, and the partial collapse of the roof. 36, 37 and 38 show the parish church of Forgaria - 36 after the May 6 shock, 37 and 38 at present. Note the collapse of the gable and part of the roof owing to the first shock, and the now total absence of the beamed roof. The façade wall, of considerable thickness, is composed of

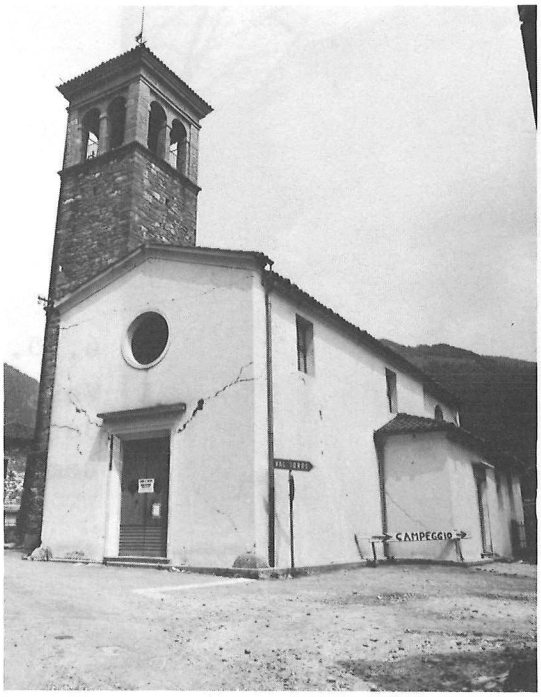


6.30.
Venzone. S. Giovanni Battista, after September 15, 1976.



6.31. Venzone. S. Giovanni Battista. After September 15, 1976.

rounded stones from the nearby Tagliamento River, bound by poor-quality mortar. The bell-tower is different, being built of well-bonded, good-quality stone blocks. 39, 40 and 41 show the façade of the parish church of Cornino, respectively after May 6, and September 15. The façade wall is made up of a facing of squared-off stone blocks standing in front of a



6.32. Zomeais. Parish church. May 1976.



6.33. Clauzetto. Oratorio dell'Angelo, May, 1976.



6.34. Ciseriis. Parish church in June 1976.



6.35. Ciseriis. Parish church in June 1976.

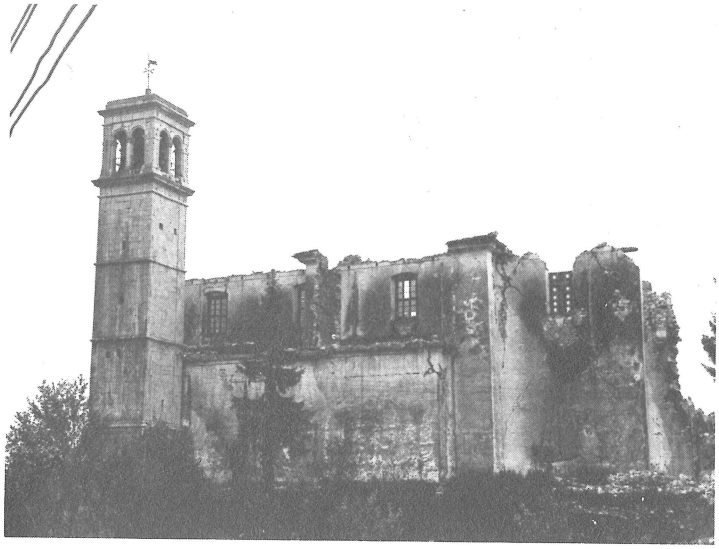
wall of stones bonded with mortar. To be noted here is the relative shifting of the blocks according to almost symmetrical lines with respect to the axis of the façade, the X-shaped lesion starting from the rosette-shaped window above, and the complete displacement of some stones from the façade, by the right nave. Further detachments can be seen after the last series of strong earthquakes and earth tremors. We can see how the shifting of the stone blocks was slight enough to avoid the fall of considerable parts of wall. In 41 we can see a lesion in the façade of the old church of S. Martino in Artegna. It starts at the join between two walls built at different times (see the interesting tourist sign - a translation of the 1519 stone tablet shown in 42, together with 7, 16 and 17).



6.36. Forgaria. Parish church.



6.37. Forgaria.



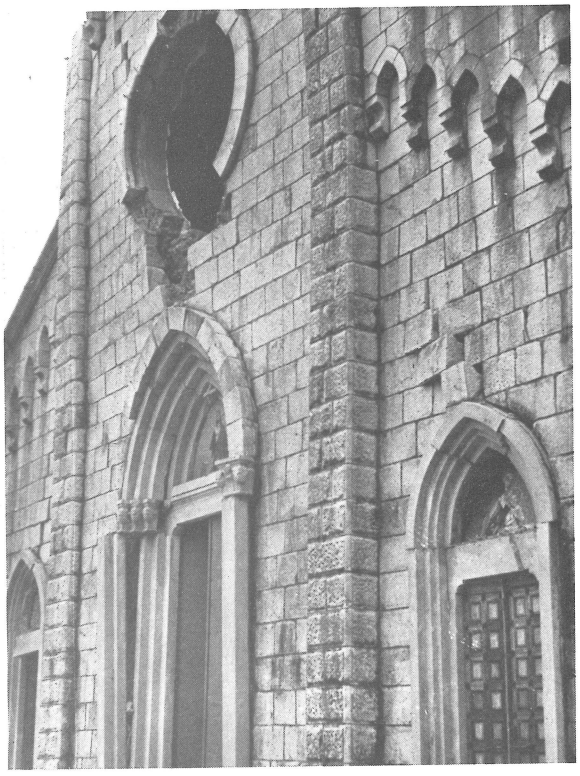
6.38.
Forgaria.



6.39.
Cornino.



6.40. Cornino.



6.41.Cornino.

4.2. Lesions to rosette windows, roof decorative elements and apexes

These lesions are details of the preceding point, of which some examples have been illustrated. They show special points of weakness in the walls, found in 30% of the cases examined. 44: Gemona, the church of S. Rocco; 45: Travesio, S. Antonio; 46: Prossenicco, parish church; 47: Monte Maggiore, church of S. Michele; 48: Lestans, parish church; 49: Anduins, parish church.

4.3. Wall lesions along or near corners

These were found in 29% of the churches examined. They were generally due to the lack of adequate bonding between

6.42. Artegna.

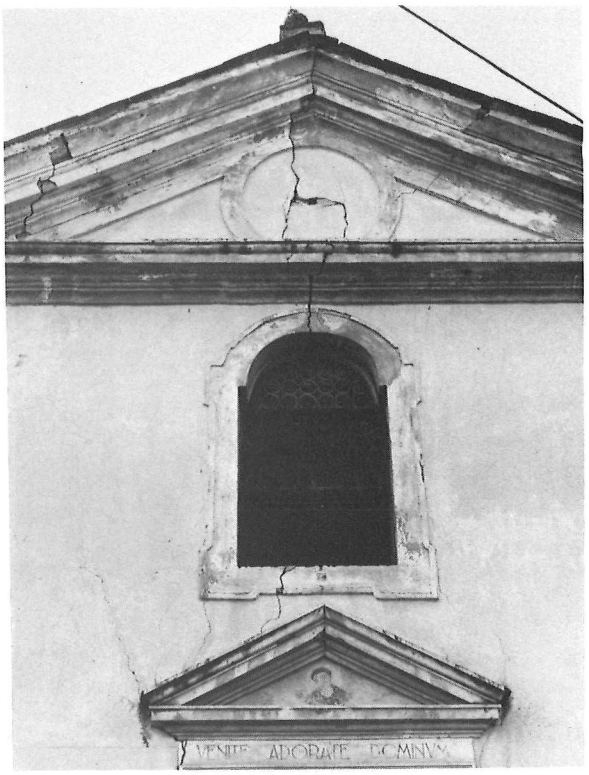


Artegna. 6.43.





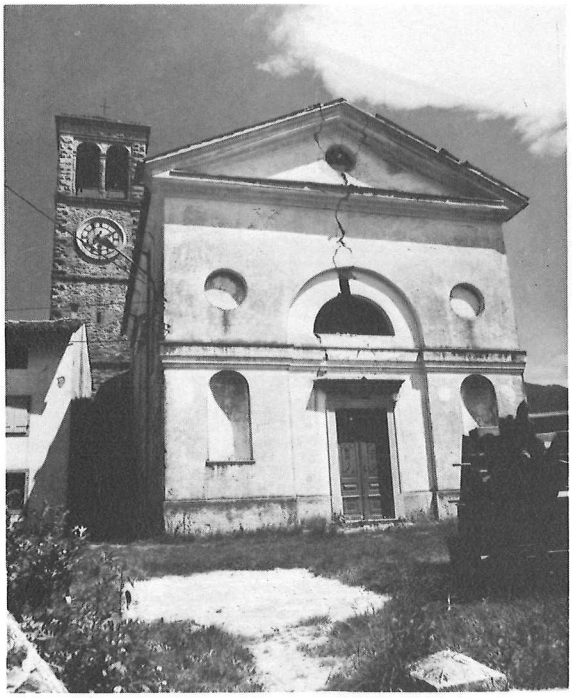
6.44. Gemona.



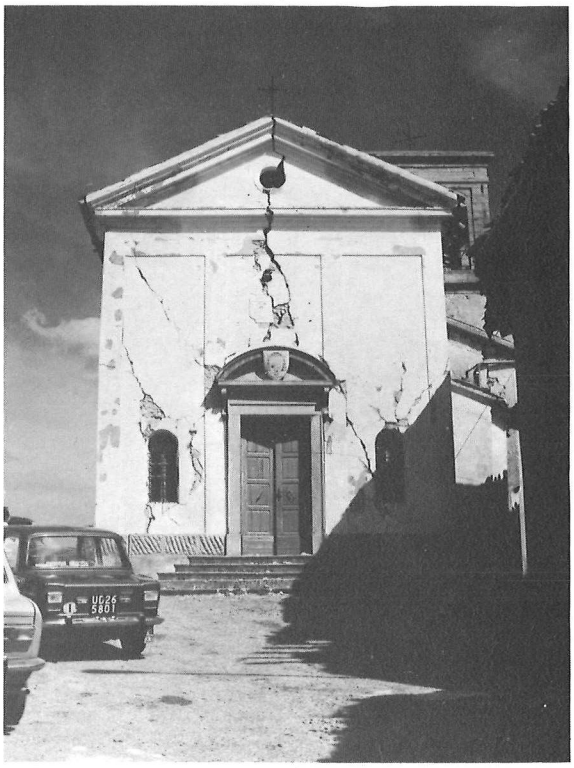
6.45. Travesio.

two orthogonal walls, either because of the different consistency of the walls meeting at the corner, or else owing to the insufficient resistance of a wall in its own plane to the tensions due to inertial actions involving the orthogonal wall, generally to the façade (representing a major mass, made still heavier by trabeations, frontals, etc.).

50: Vito d'Asio, parish church: the cornice stone has fallen; 51: Cesariis (Lusevera), church of the Santi Ermacora e Fortunato; 52: Pradis di sotto, parish church; 53 and 54: Cornino, parish church (see also 39 and 40); 55 and 56: Artegna, church of S. Martino (see also 42 and 43) respectively after May 6 and at the present time: the façade is made of well-dressed, well-bonded blocks of stone. Shifting



6.46. Prossenicco.



6.47. Monte Maggiore.

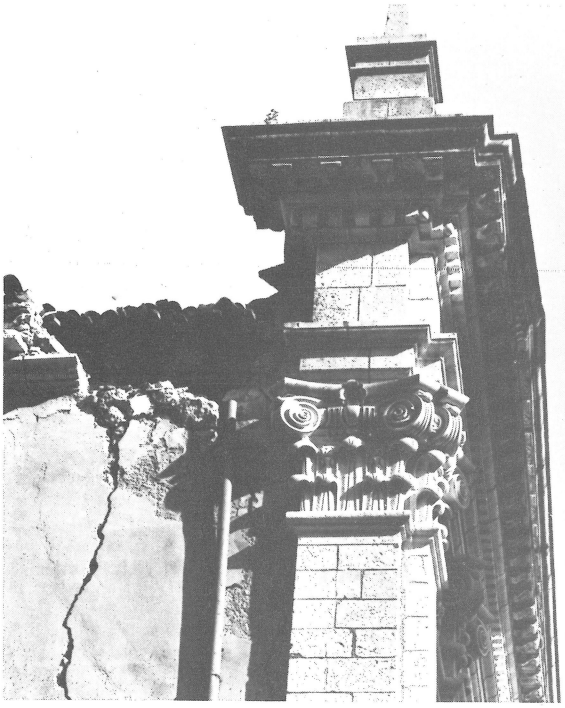
in relation to the wall elements along the bearings of a series of blocks has taken place (the phenomenon has worsened since the September shocks), without any masonry falling. The scarfing of the parallelepiped stone blocks avoided worse damage. The wall cornice has fallen along the flank of the building. 57 and 58: Valeriano, church of S. Maria dei Battuti (see also 10): the lesions correspond to the attachment of the barrel-shaped vault to the back of the façade. In spite of the presence of a steel tie, the façade wall was subjected to the movements of the vault during the earthquake. 59 and 60: San Daniele, church of Fratta: here too, good behaviour was shown by the stone blocks with their regular, square shape. However, they were not sufficient to prevent detach-



6.48.
Lestans.



6.49. Anduins.



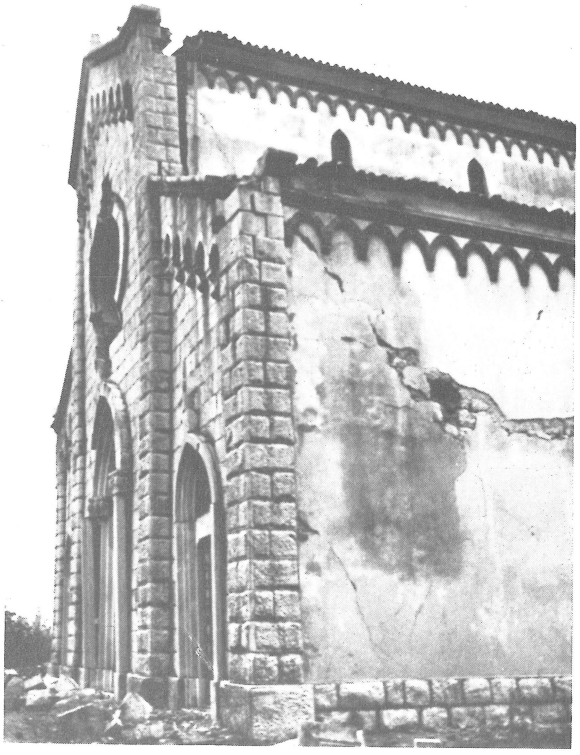
6.50. Vito d'Asio.



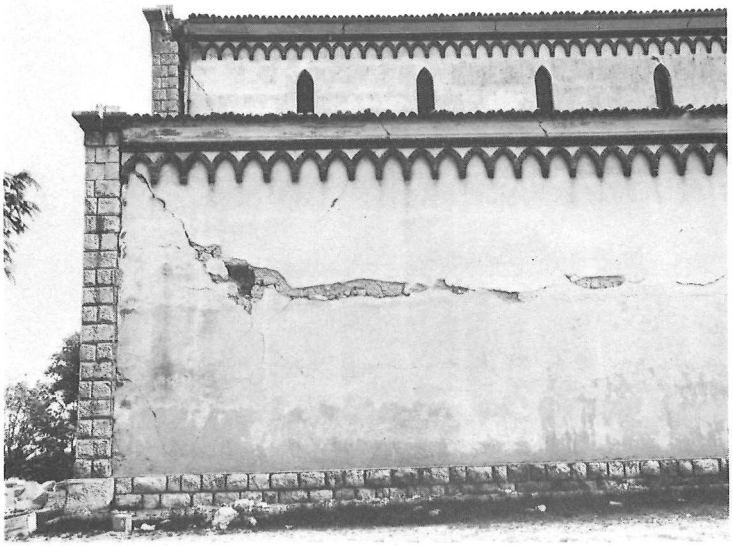
6.51. Cesariis (Lusevera).



6.52. Pradis di sotto.



6.53. Cornino.

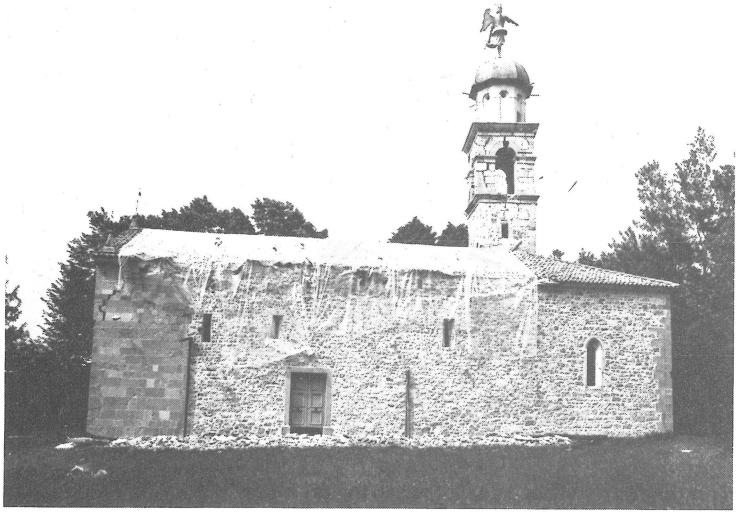


6.54. Cornino.

ment lesions along both corners of the façades; 61: Lestans, parish church; 62: Gaio, parish church; 63: Clauzetto, parish church.

4.4. Partial or total collapse of the roof

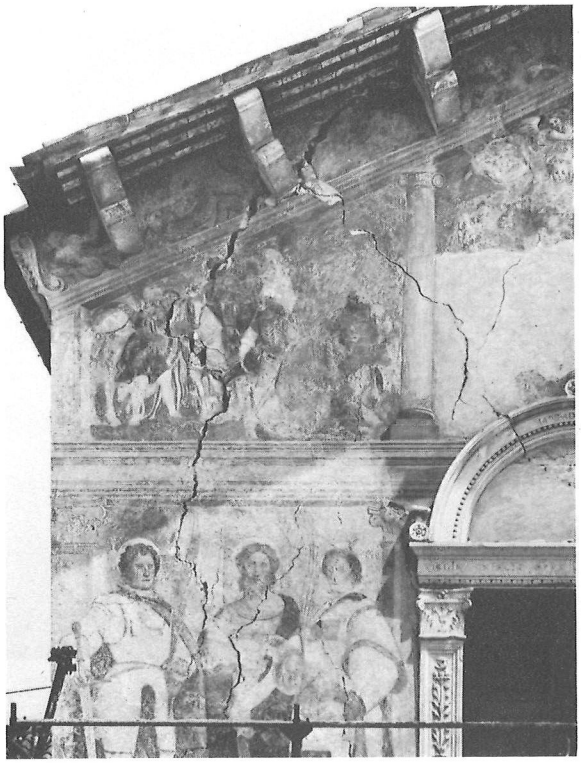
The collapse of the roofs of churches was also a very widespread effect of the earthquake (22% of churches examined). However, even before collapse, the roofs of several buildings were in a very bad state of repair. Damage varies from tiles falling from the roof edges, where they had accumulated during the actual shock, to the detachment of parts of the ceiling during and following the deformation of the roof frame, to the collapse of cornices of the principal elements of the roof frame. This last type of damage, in particular, also occurred in many civil buildings. This type of damage, due to the special formation of decorative elements which, overhanging, were balanced precisely by the weight of



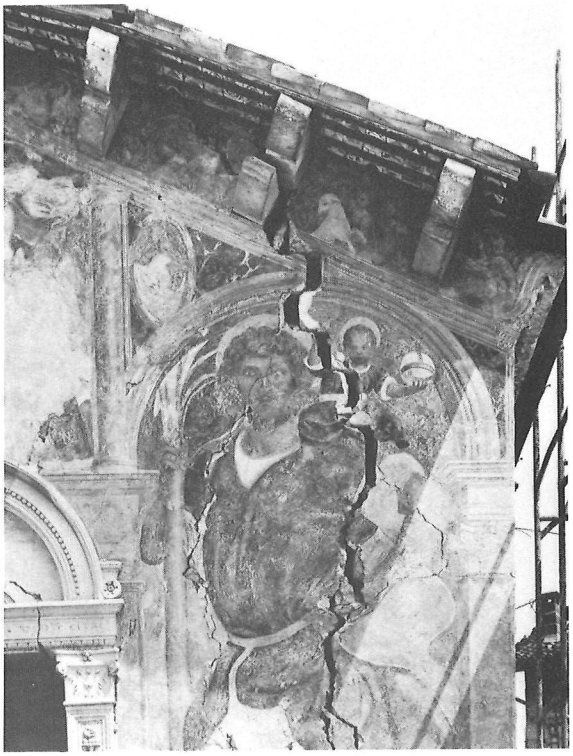
6.55. Artegna.



6.56. Artegna.



6.57. Valeriano.



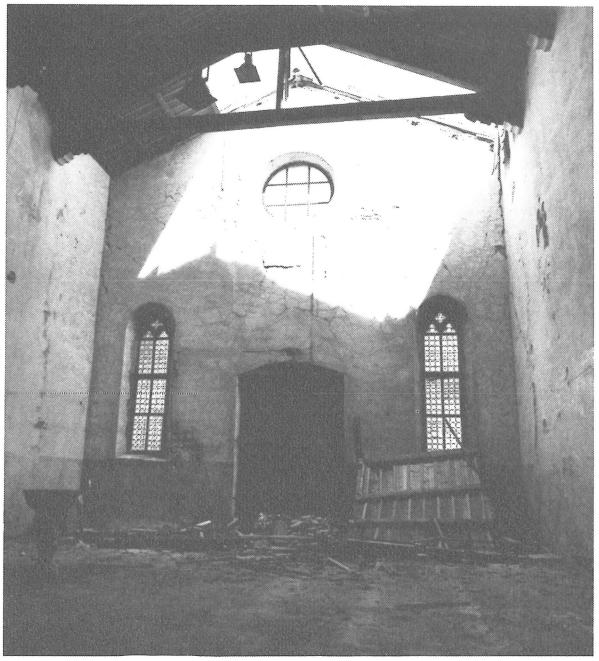
6.58. Valeriano.

the roof beams resting on them (for example, 64: Vito d'Asio, parish church). The collapse of the roof was rarely an isolated phenomenon. It was almost always accompanied by more or less extended collapse of the load-bearing walls, and sometimes of the whole construction. However, much damage could have been avoided with better stiffness of the roof pitches in their planes, better bonded of the bracing of the main beams, with special attention to the load-points of the load-bearing walls, avoiding with even simple methods the fall of small elements inside the church and along its perimeter.

65: Ciseriis, parish church; 66: Venzone, church of SS. Giacomo and Anna; 67-68: Valeriano, parish church: the roof



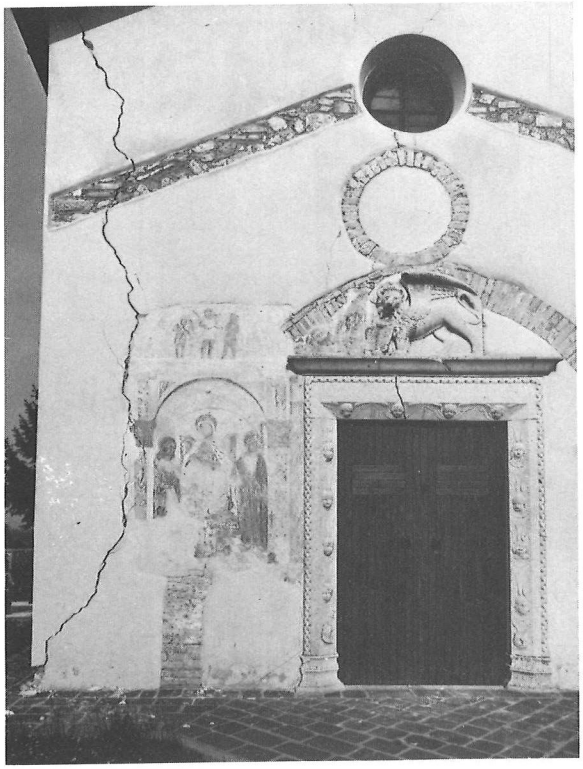
6.59. San Daniele.



6.60. San Daniele.



6.61. Lestans.



6.62. Gaio.



6.63. Clauzetto.

is made of a frame with two pitches of reinforced concrete and light bricks, with a tie. The roof probably fell after the collapse of part of the load-bearing wall, made of low-quality, rounded stones; 68-71: Gemona, church of S. Giovanni. Part of the collapse is due to the fall of the bell-tower; the painted wooden box-ceiling is visible (photograph taken before the September shocks); 72: Madonna di Buia, parish church: part of the roof fell during the demolition (using explosives) of the bell-tower which was left in a dangerous state after the May 6 shock; 73: San Daniele, church of S. Daniele Profeta "Al Castello": part of the wooden roof frame has collapsed. The vaults of plastered wooden strips found in many Friuli churches are visible: the fall of large



6.64. Vito d'Asio.



6.65. Ciseriis.

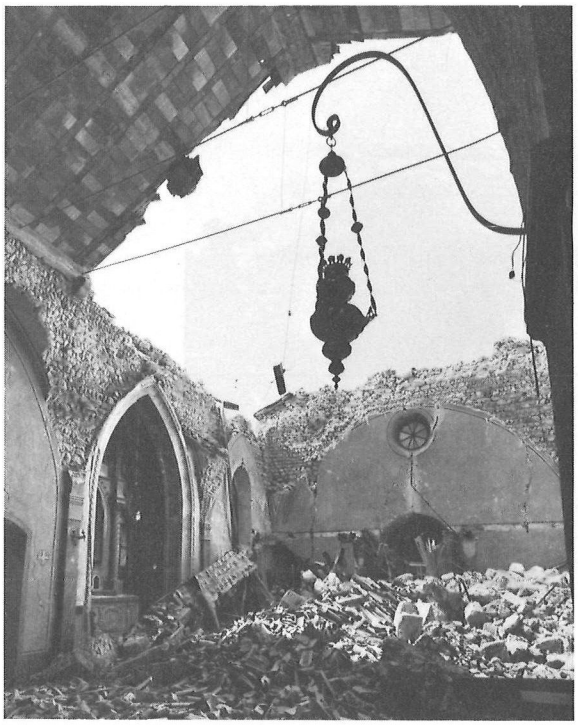


6.66. Venzone.



6.67. Valeriano.

portions of plaster often meant damage and danger; 74: Osoppo, church "Al Forte"; 75: Pers, parish church: the collapse of the roof meant almost total destruction: 76-77: Osoppo, parish church: on May 6, destruction was almost total: the roof followed the collapse of the entire nave, probably not



6.68. Valeriano.



6.69. Gemona.



6.70. Gemona.



6.71. Gemona.



6.72. Madonna di Buia.



6.73. San Daniele.

braced perpendicularly to its long axis. Instead, the transept, better enclosed within its walls, did not collapse, although it was severely damaged (situation on May 6); 78: Majano, parish church, now totally destroyed.



6.74. Osoppo.



6.75. Pers.



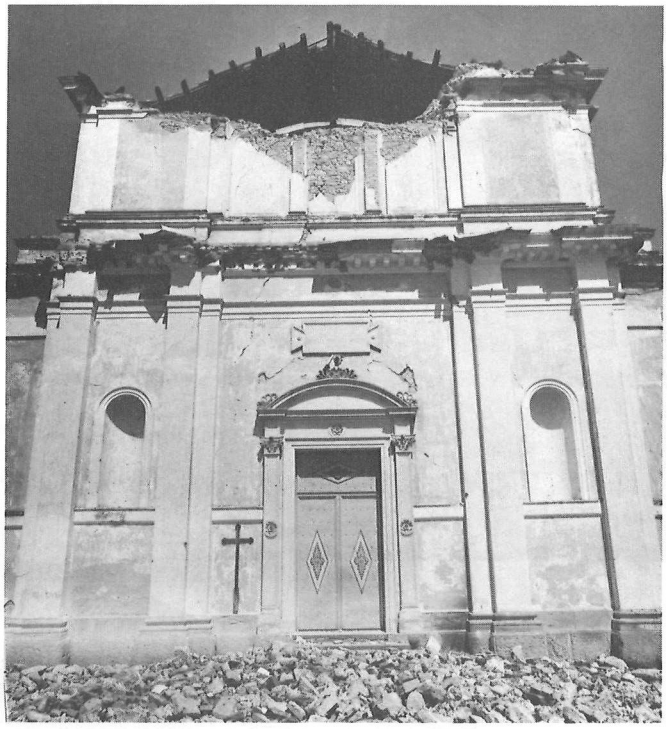
6.76. Osoppo.



Osoppo. 6.77.



6.78. Majano.



6.79. Castelnovo del Friuli.



6.80. Pradielis.



6.81. Montenars.



6.82. Flagogna.



6.83. Montenars.



6.84. Valeriano.



6.85. Flagogna.



6.86. Flagogna.



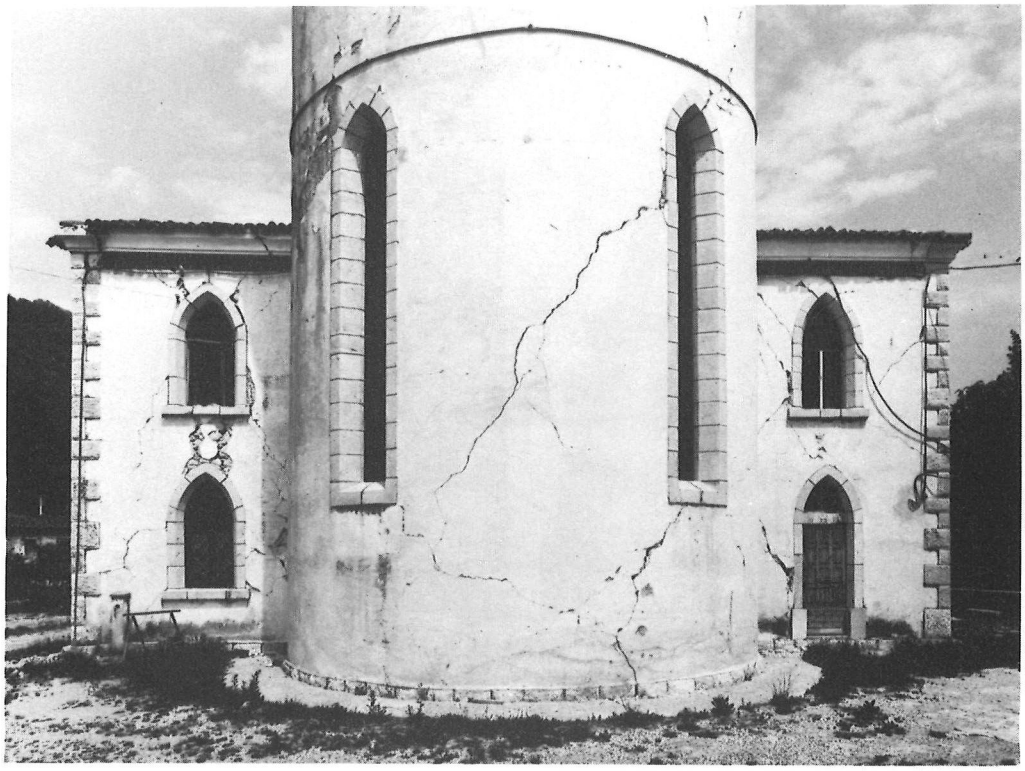
6.87. Castelnovo del Friuli.

4.5. Detachment of gables and frontals

This type of lesion occurred in cases where walls were weak because of lack of support from above, especially if they were subjected to actions perpendicular to their planes. Several examples have already been seen: 79: Castelnovo del Friuli, parish church; 80: Pradielis, parish church; 81: Montenars, church of S. Elena; 82: Flagogna, church of the Madonna di Campagna.

4.6. Lesions to apses and chapels in curved areas

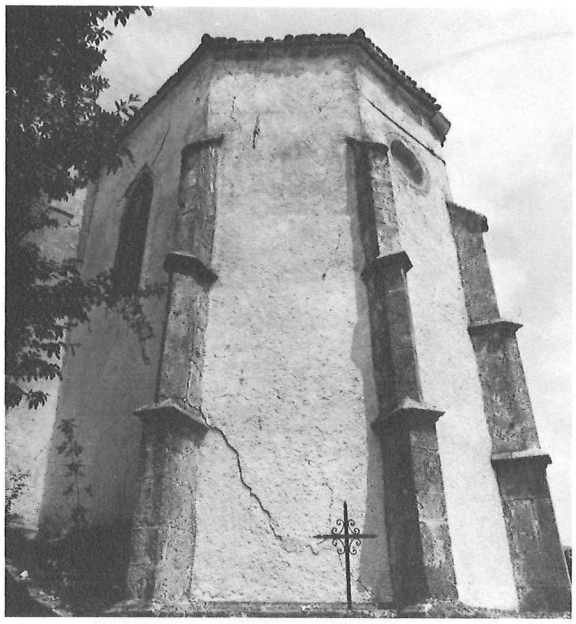
These very frequent lesions are presumably due to the relative shifting of the skewback of the apse. 83: Montenars, church of S. Giorgio; 84: Valeriano, parish church; 85: Flagogna, parish church; 86: Flagogna, church of the Madonna di Campagna; 87: Castelnovo del Friuli, parish church: 88: Cornino, parish church; 89: Zuglio, Pieve di S. Pietro: the beneficial effect of the steel tie along the directrix of the



6.88. Cornino.



6.89. Zuglio, Pieve di S. Pietro.



6.90. Illegio.

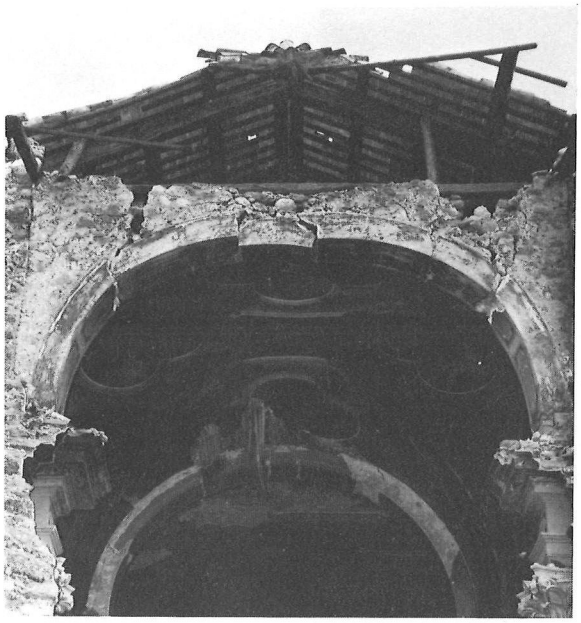


6.91. Illegio.

chapel can be noted; 90-91: Illegio, Pieve di San Floriano: the strengthening made up of buttresses and a steel tie in the upper part can be seen.

4.7. Lesions to arches

Arches set on pillars or over walls parallel to one another are very common in churches, and such structures are of course extremely sensitive to shifting of other lower parts caused by the type of movements encountered in an earthquake. Almost everywhere, arches show lesions in the keystones or the reins. In other cases, the arches collapsed entirely, as some of these photographs show. 92: Flagogna, parish church; 93: Trasaghis, parish church; 94: Prossenicco, parish church; 95: Pinzano al Tagliamento, parish church; 96: Gaio, parish church; 97: San Daniele, church of S. Daniele Profeta; 98-99: Valeriano, church of S. Maria dei Battuti (see also 10,



6.92. Flagogna.



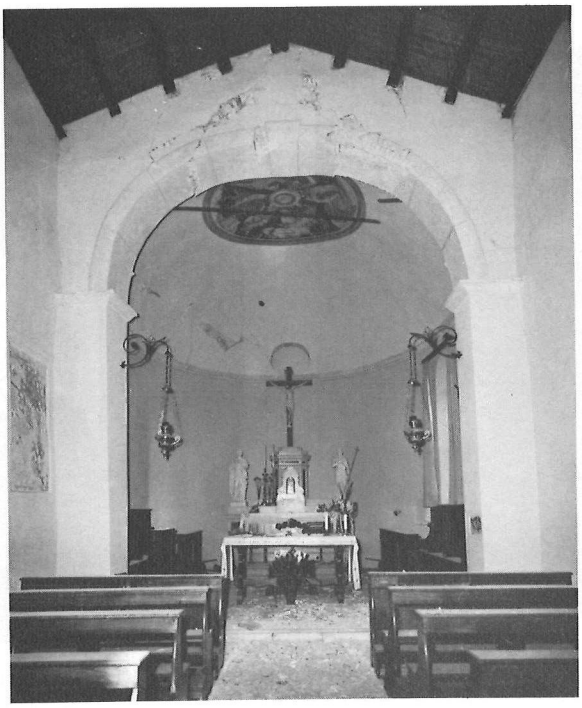
6.93. Trasaghis.



6.94. Prossenicco.



6.95. Pinzano al Tagliamento.



6.96. Gaio.



6.97. San Daniele.



6.98. Valeriano.



6.99. Valeriano.



6.100. Valeriano.

57 and 58): in this case the damaged structure is the barrel-shaped vault.

6.5. Conclusion

In conclusion, we know for certain that we have not been able to illustrate fully and systematically all the lesions, detachments, settlings, and collapses of the churches caused by the 1976 Friuli earthquakes, nor have we gone into deeper diagnoses of phenomena which are very often complicated and dependent on many parameters. In any case, these would be premature. We wished, rather, to offer a panorama of some important effects of the earthquakes on these large buildings, principally as a contribution towards our knowledge of this phenomenon; also as a stimulus to further widening of our knowledge on seismic events, and above all to encourage measures to protect old buildings which make up so large a part of our common artistic, cultural and historical heritage.

Padova, October, 1976.

7. BELL-TOWERS

7.1. General considerations

In this section we refer to the behaviour of "campanili" during the earthquake. These bell-towers are normally at the side of the churches and, together with them, make up - or rather, unfortunately, made up - the centre of a centuries-old tradition and the material symbol, rising above the roofs of any town or village, of the need to remember and show the importance of community life.

From a simple observation of the geometric details of a bell-tower, it may seem that this building is particularly sensitive to earthquakes. This is because, by its very nature, it is always a high building, and in any case higher than its surrounding buildings, so that we instinctively think that any seismic shock will have considerable effects.

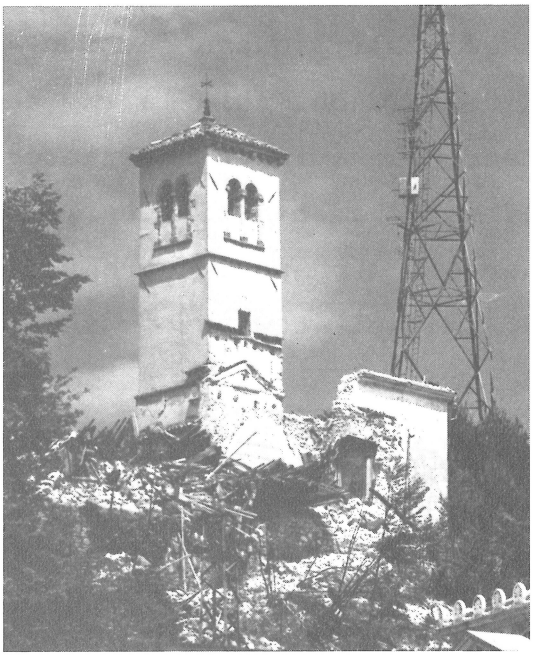
Instead, the particular structure of a bell-tower, and especially the shape of the main shaft - with a hollow, closed and generally square section - often gives it great rigidity and resistance. This is why the resistance of a bell-tower to an earthquake is often greater than that of the adjacent church (although, as can be seen in the section on Churches, these underwent perhaps the greatest damage). In some cases, the fact that the bell-tower took the stress better was due to its having been constructed using better techniques than those used for the church. In effect, in those churches with cobblestone walls, sometimes without brick strips, in which dressed stones were only used for the finishing of the façade, we can find bell-towers whose main shaft is made entirely of well-worked



7.1. Forgaria. Church of S. Nicolò (completely destroyed) and bell-tower



7.2. Pradielis. Parish church (partially destroyed) and bell-tower



7.3. Moggio. Church of the Santo Spirito (completely destroyed) and bell-tower



7.4. Osoppo. Parish church (completely destroyed) and bell-tower (damaged)

cut stone or carefully made brick walls.

Photographs nos. 1, 2, 3 and 4 show some typical cases.

However, it should not be thought that the bell-towers in the area hit by the earthquake did not suffer much damage, even though it may be less than that suffered by the churches. Generally, the structure of the bell-tower often presents some characteristic "weak zones" where damage is concentrated, or in which a breaking sequence for the surrounding structures can begin. The typical parts which are weaker than the main shaft are the belfry (almost always open), all the ornamental superstructures, and the roof. There are also

some causes for weakening of the main shaft, such as the presence of openings, almost always aligned along the midline of a wall or, sometimes, large clocks set in the walls.

The following gives specific information on the typical earthquake behaviour shown by bell-towers and on damage undergone. We would, however, like to preface this with a list of statistics referring to 260 bell-towers over an area of about 1200 sq.km. in a circle round the epicentre.

- Total or almost total collapse of the bell-tower 7%
- Collapse of the belfry or roof 8%
- Serious damage to the belfry and roof 10%
- Serious vertical or diagonal lesions to the main shaft walls 20%
- Various lesions of average severity 20%
- Slight damage 35%

Naturally, these data, referring to a rather large area round the epicentre of the earthquake, change according to whether we consider areas nearer or farther from the epicentre itself. However, it is interesting to note that, while about 45% of the bell-towers in the area suffered serious damage, up to 60-70% of the churches in the same area are severely damaged.

7.2. Documentation of bell-tower damage

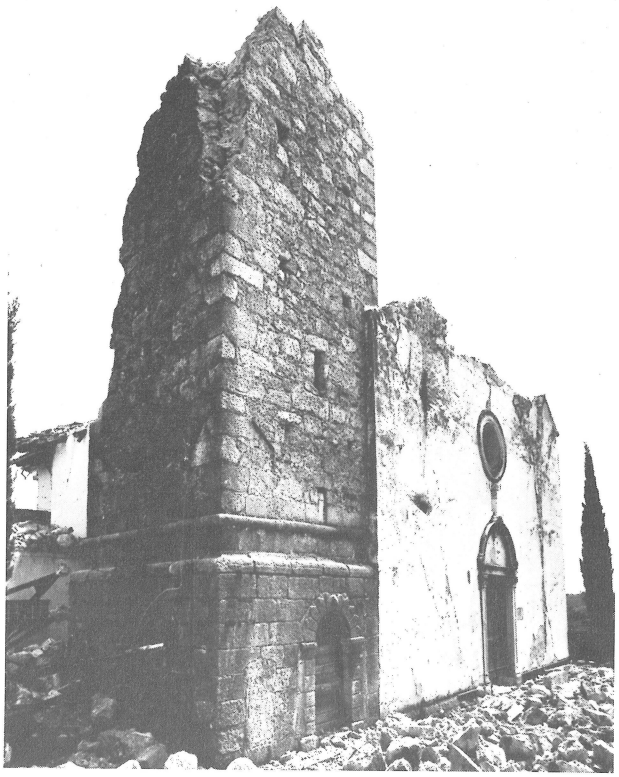
7.2.1. Collapse of the bell-tower. The almost total collapse of the bell-tower occurred in some cases, specially in the presence of very old (though very thick), badly bonded, not very regular, and unplastered masonry.



7.5. Gemona. Bell-tower of the church of S. Giovanni.



7.6. Venzone. Bell-tower of the church of S. Giovanni B. (before the earthquake). It was completely destroyed.



7.7. Valeriano. Parish church and bell-tower



7.8. Travesio. Bell-tower of the church of S. Antonio da Padova



7.9. Madonna di Buia. Bell-tower of the parish church. Partially destroyed by the earthquake, it was demolished with dynamite.

7.2.2. Collapse of the belfry or roof. Very often the belfry has many openings which weaken its walls greatly, so that although the main shaft of the tower often behaved well, the belfry and its roof did not.

We point out here that the danger connected with the collapse of the roof and belfry of a bell-tower is always great, since pieces fall from a great height.



7.10. Flagogna. Bell-tower of the parish church. Note the collapse of the roof and the almost complete destruction of the belfry.



7.11.



7.12.



7.13.

7.11. S. Pietro di Ragogna. Bell-tower

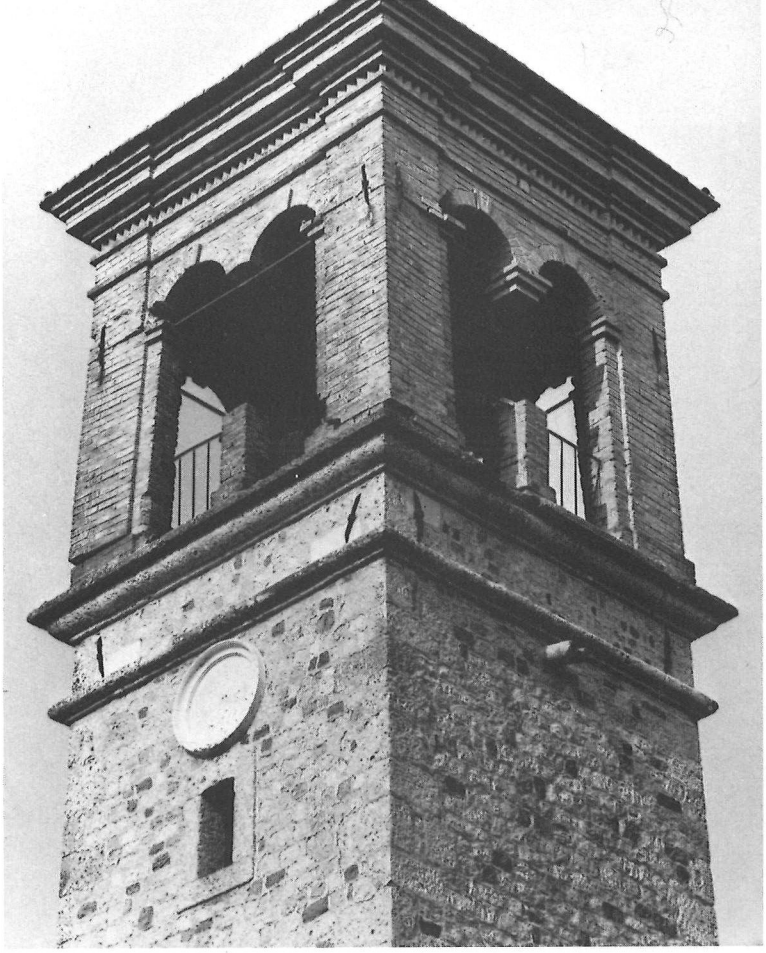
7.12. Aprato. Bell-tower and church of the Madonna del Giglio

7.13. Clauzetto. Bell-tower of the parish church. What remains after the collapse of the roof and of the belfry.

7.2.3. Serious damage to the belfry and roof. In many cases, even if the belfry did not collapse, it underwent a series of lesions which seriously affect its stability to the point where restoration becomes very problematical.



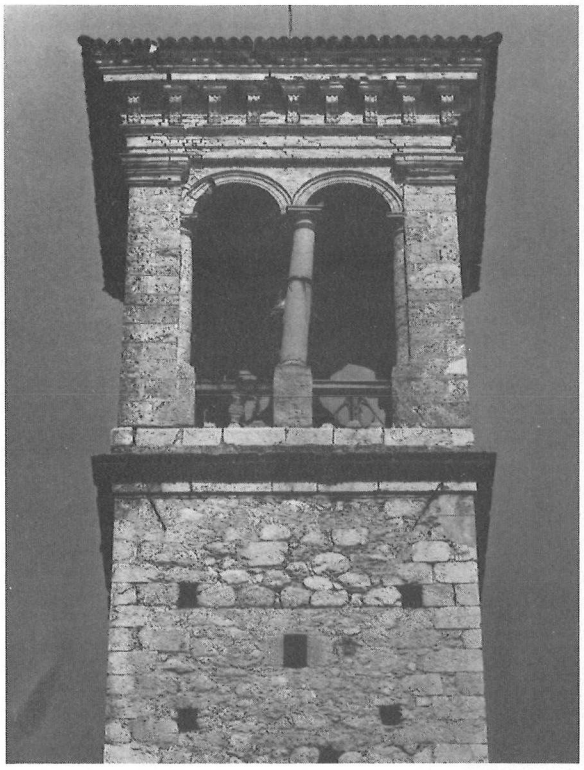
7.14. Cassacco. Bell-tower of the church of S. Maria Assunta



7.15. Gaio. Belfry of the parish church



7.16. Anduinis. Bell-tower of the parish church



7.17. Trasaghis. Bell-tower of the parish church

Artegna. Church of S. Martino and bell-tower

7.18.



7.2.4. Serious vertical or diagonal lesions to the main shaft walls. Serious lesions to the main shaft walls generally appear in bell-towers where building techniques were inadequate (similar to those described in 7.2.1.).

We can also connect lesions with the often aligned openings on the midline of a wall, or with other isolated weakening elements.

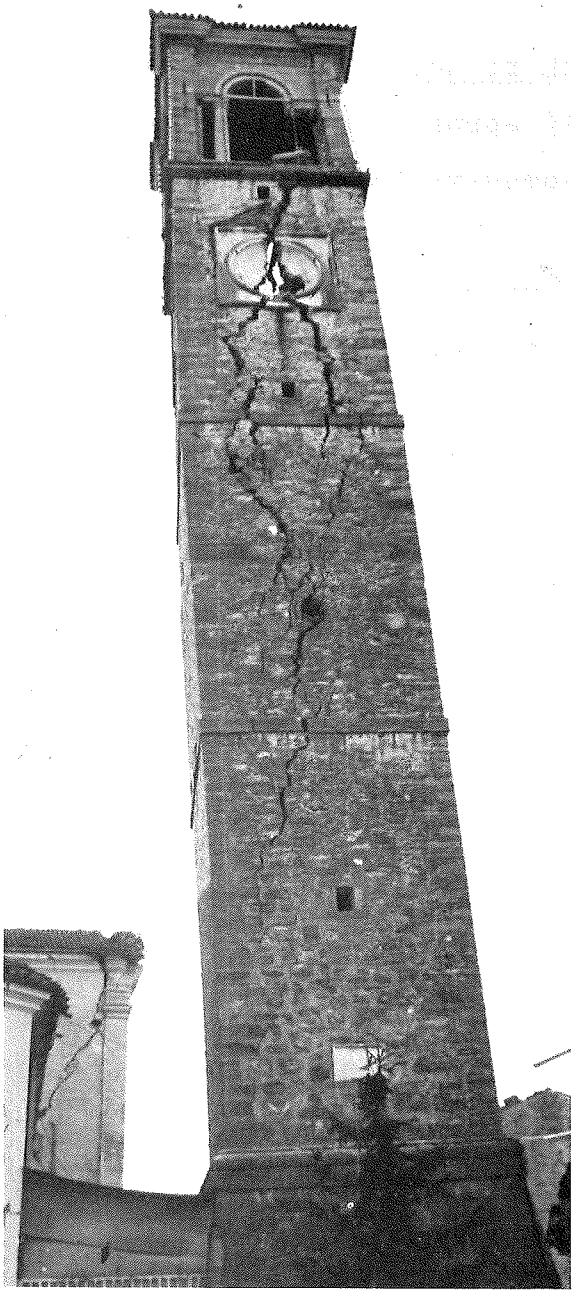
Lesions either run vertically along more or less midline generatrix, diagonally, or in a typical X.



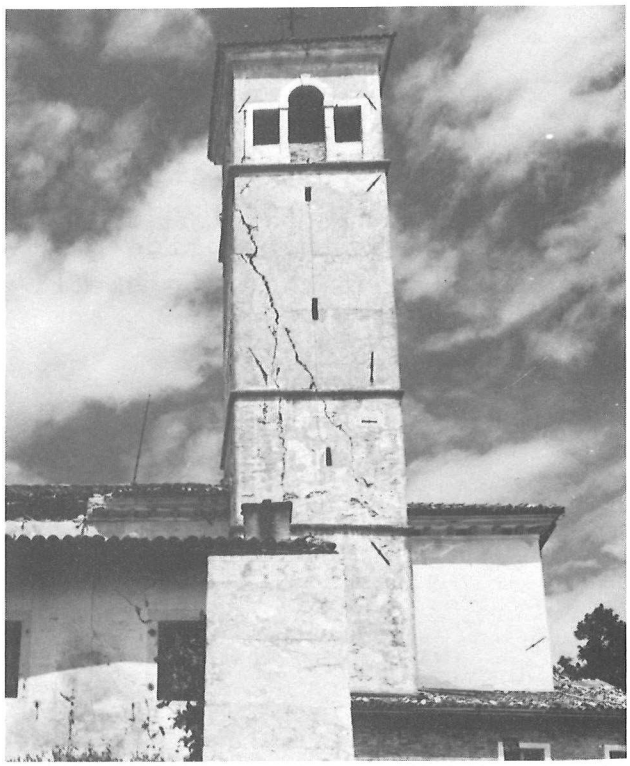
7.19.
Colle d'Arba. Bell-tower of the parish church. The bell-tower is almost completely destroyed by a diagonal lesion.



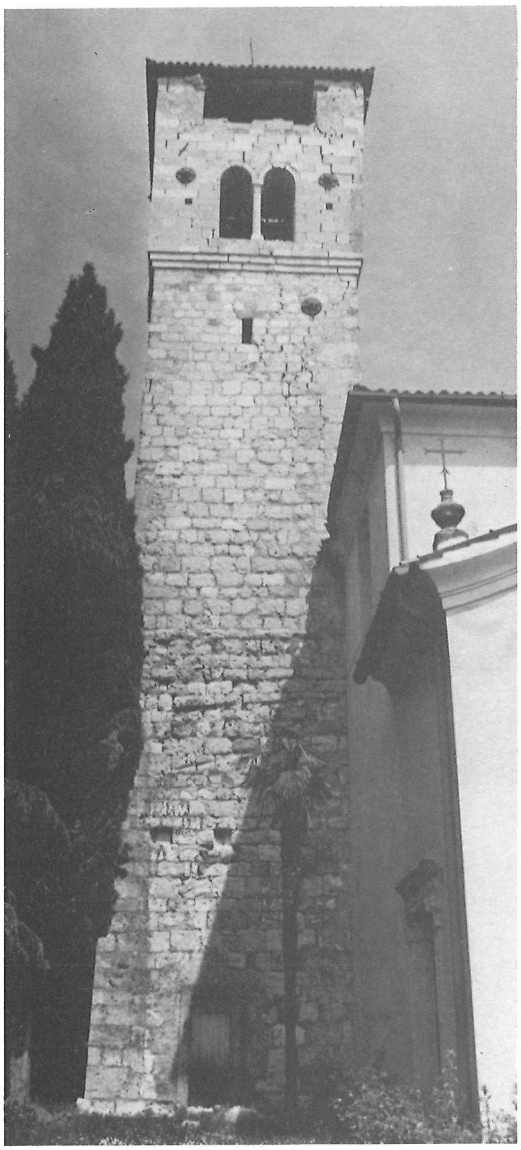
7.20. Montenars. Bell-tower of the parish church. X-shaped lesions by the clock continuing partly vertically through the windows.



7.21. Madonna di Buia. Bell-tower of the parish church. Serious X-shaped lesions and vertical lesions.

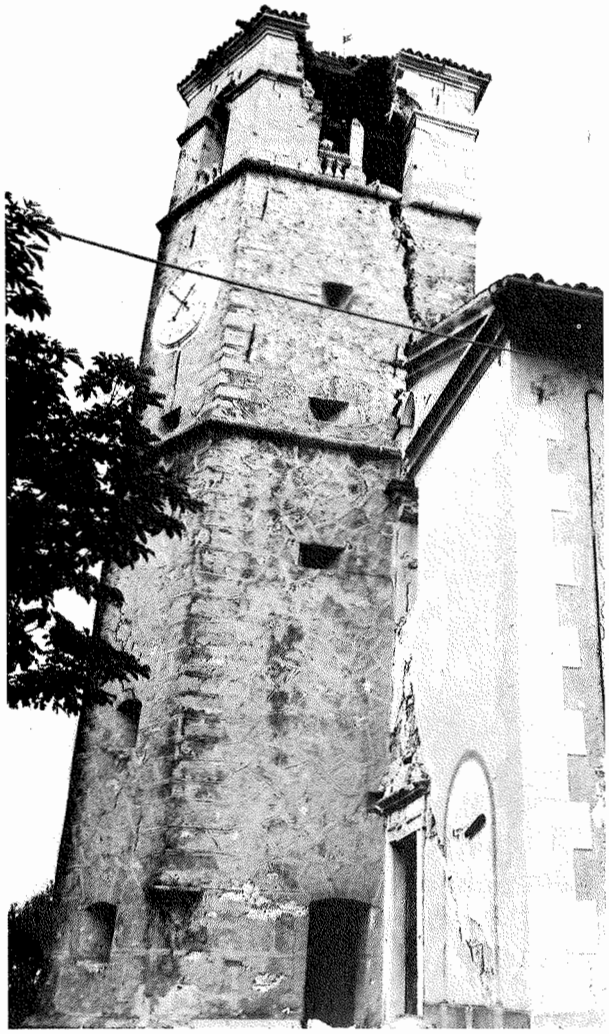


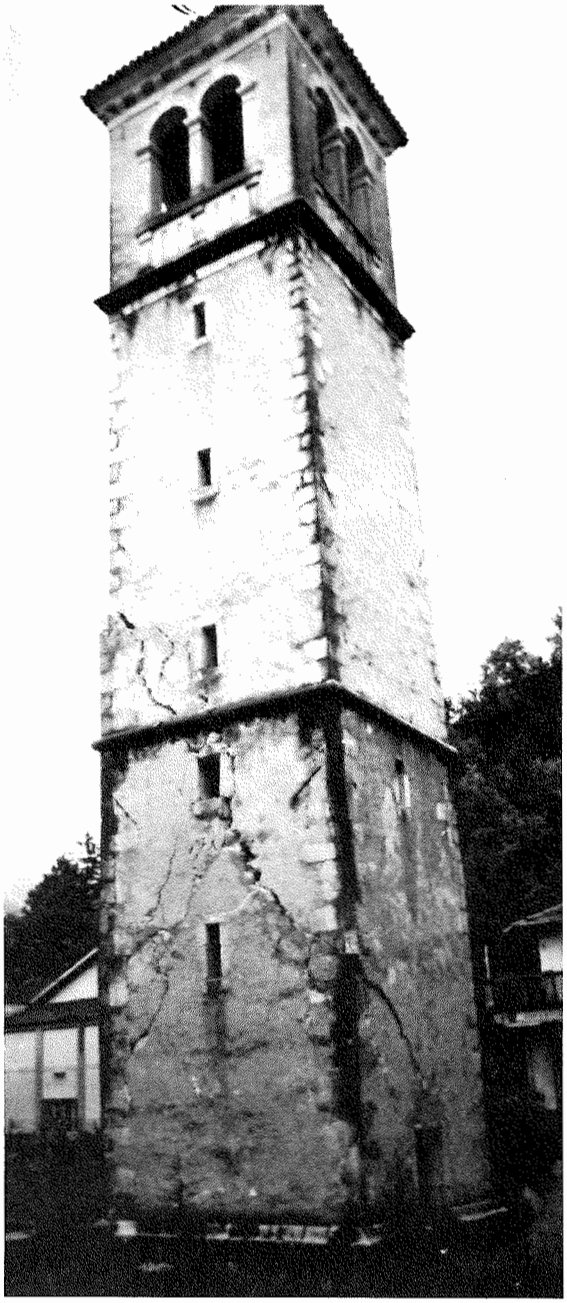
7.22. Moggio. Bell-tower of the Abbey. Diagonal and vertical lesion.



7.23. S. Daniele. Bell-tower of the church of S. Daniele. Vertical lesions starting from the belfry.

7.24.
Resiutta. Bell-tower of the parich church. Helicoidal and X-shaped lesions with rotation of the upper part of the main shaft.





7.25. Monte di Buia. Bell-tower of the church of S.Lorenzo. Serious vertical lesions. During an attempt at removing the bells, the bell-tower collapsed, splitting in two along the lesion.



7.26. Braulins. Bell-tower of the parish church. Diagonal lesion corresponding to the attachment of the bell-tower to the church.



7.27. Illegio. Bell-tower of the Pieve di San Floriano. Horizontal and vertical lesions, corresponding to a cobblestone wall.

7.2.5. Various lesions of average severity. The damage mentioned above can occur with less severity owing to better building techniques or greater distance from the epicentre of the earthquake.

S. Floriano di Buia. Bell-tower of the parish church. X-shaped lesions on the main shaft walls.
7.28.

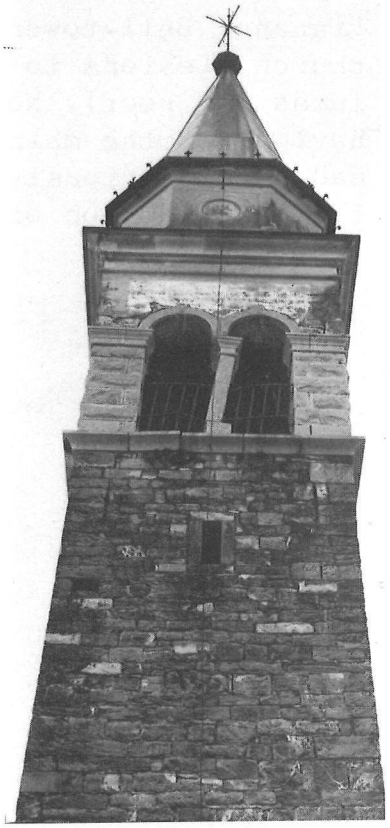


Tarcento. Bell-tower of the parish church. X-shaped and diagonal lesions on the main shaft walls.
7.29.



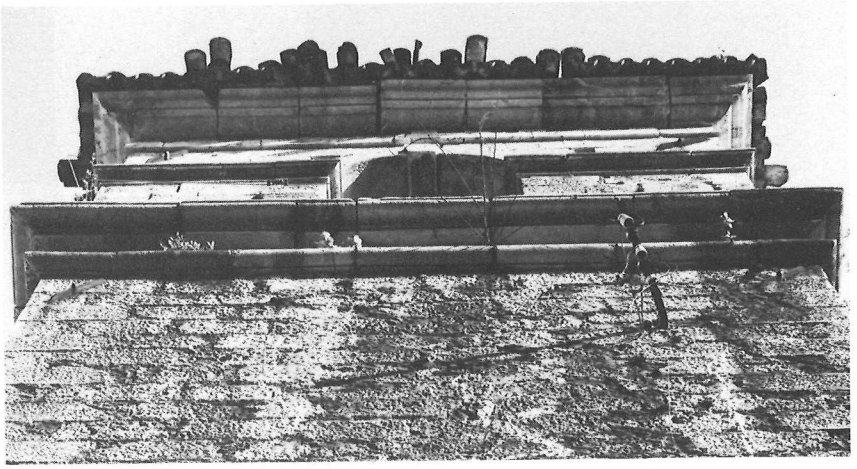


7.30. Valbruna. Bell-tower of the parish church. Vertical midline lesion corresponding to the windows.



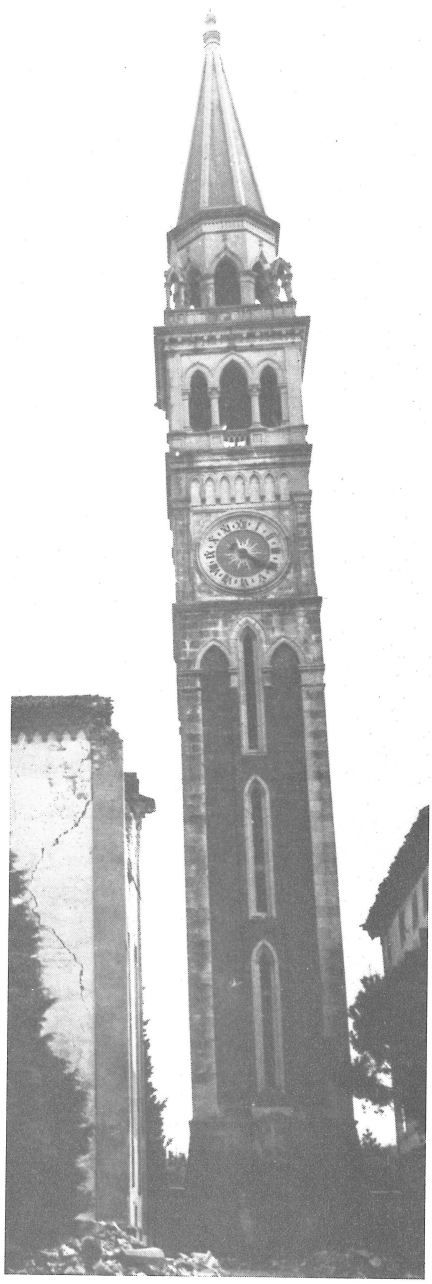
7.31. Aprato. Bell-tower of the church of S. Biagio. Detachment of a column in the belfry.

7.32. Vito d'Asio. Bell-tower of the parish church. Detail of cornice movements (stone) and the roof tiles.



Pignano. Bell-tower of the parish church. Lesions to the belfry (columns and roof). Note the good behaviour of the main shaft, which had been previously restored with the introduction of metal ties.
7.33.

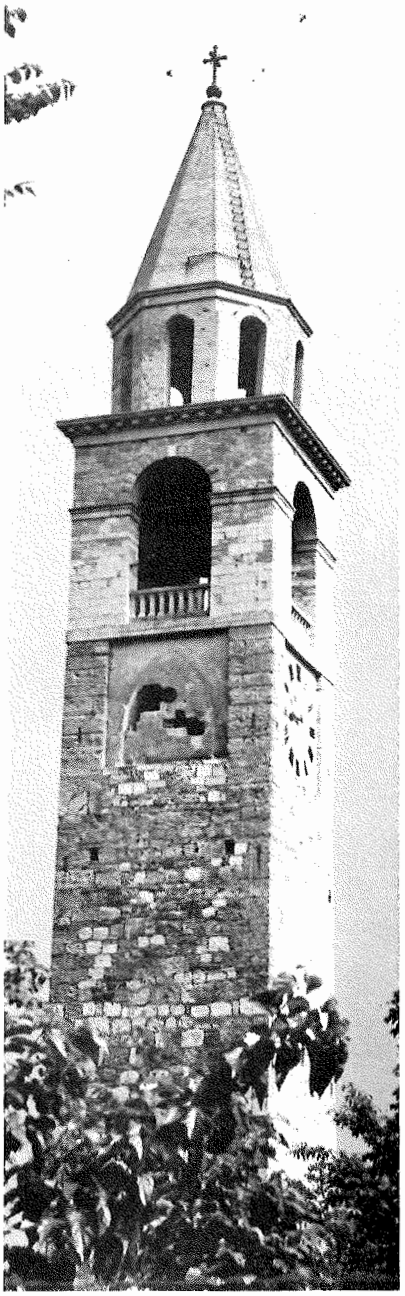




7.34. S. Stefano di Buia. Bell-tower of the parish church. Damage to the belfry columns.



7.35. Udine. Bell-tower of the church of S. Maria di Castello. Lesions to the corner pillars of the belfry and to the octagonal drum.



Artegna. Bell-tower of the parish church. Detachment of the walling of the octagonal drum with rotation outwards.

7.36.

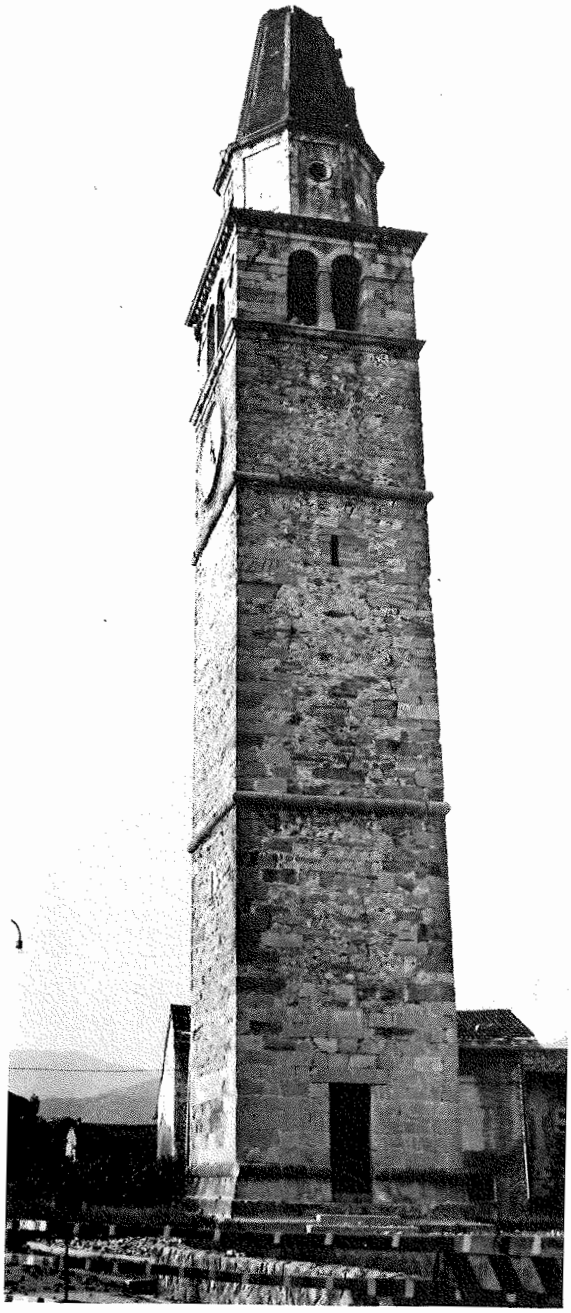
7.2.6. Typical lesions to the spire. We also mention typical damage to the spire of bell-towers in areas quite far from the epicentre. These lesions generally occur at the top of the spire, sometimes detaching it.

S. Giovanni di Casarsa. Bell-tower of the parish church.
7.37.





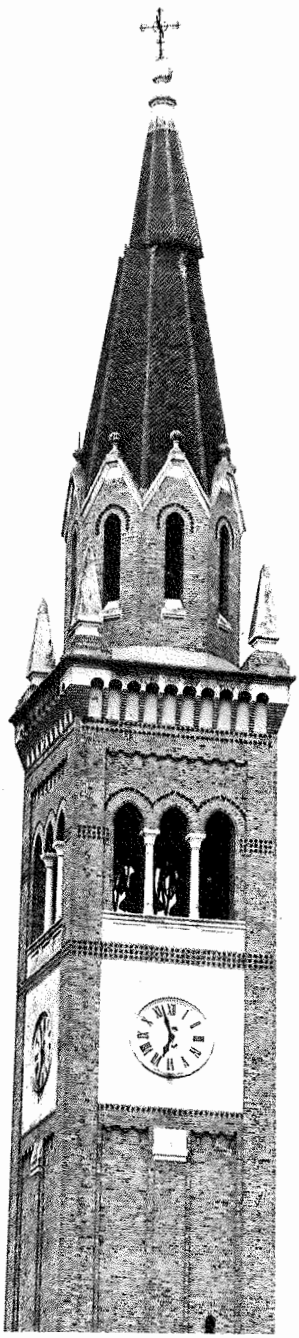
7.38. Domanins. Bell-tower of the parish church.



7.39. Ziracco. Bell-tower of the parish church.

Bannia. Bell-tower of the parish church.

7.40.



7.3. Conclusive observations

Although this is not the place to enter into a discussion on the specific problems connected with the restoration of the damaged buildings, we would like to mention once more how any restoration work, and especially that on bell-towers, should always be planned giving priority to "first static protection". In fact, this immediate work exposes workers to serious risks. On the other hand, the type of work is very important for the security of future restoration work.

In effect, some bell-towers collapsed when preliminary work was started on them. Also, other destructive shocks, even after a long period of time, are always possible, and are even more dangerous in that they act on already damaged structures.





7.41. Madonna di Buia. The photograph on the left shows the church after the earthquake. The bell-tower was severely damaged, and was demolished with dynamite. This operation also caused the destruction of the greater part of the church, as can be seen in the second photograph. 7.42.

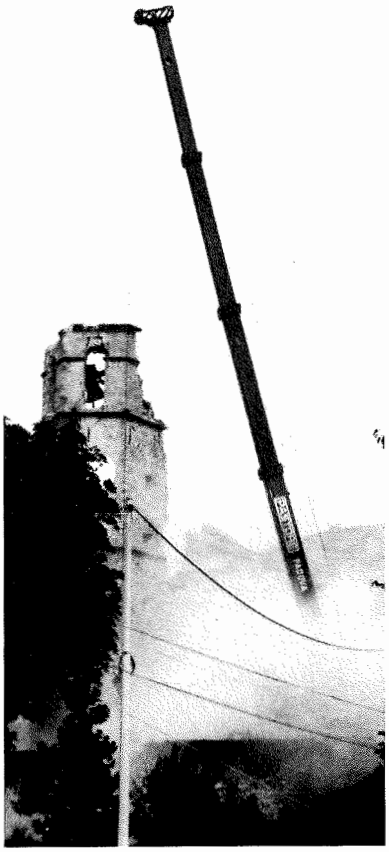
On the other hand, of course, the problem of the demolition of a damaged bell-tower can only be faced after careful considerations on necessity, opportunity and type, since we are constantly dealing with very important and irreplaceable artistic, historical or environmental heritages. When demolition is inevitable owing to danger to human lives, however, demolition techniques must not cause further damage to



7.43.



7.44.



7.45.

Monte di Buia. This sequence shows how, during an attempt at removing the bells from the belfry, the bell-tower itself split along its vertical lesion.

other surrounding buildings (this unfortunately sometimes occurred during the irrational demolition of some bell-towers).

Finally, we would like to consider a point which, although certainly not new, can never be emphasized enough: "preventive action" is always better in seismic areas, aiming at reducing the "seismic risk". This is certainly the most economical and sometimes the only means we have of safeguarding monuments which on one hand have great artistic or historical value, and on the other are particularly vulnerable to even not very serious earthquakes.

To this end, all information given here can be useful on the different seismic behaviour of bell-towers in the Friuli earthquake.

It was possible to see - and the following photographs document this - how bell-towers which had undergone recent restoration with anti-earthquake elements generally took the seismic stress well, and how in all cases damage was not serious.

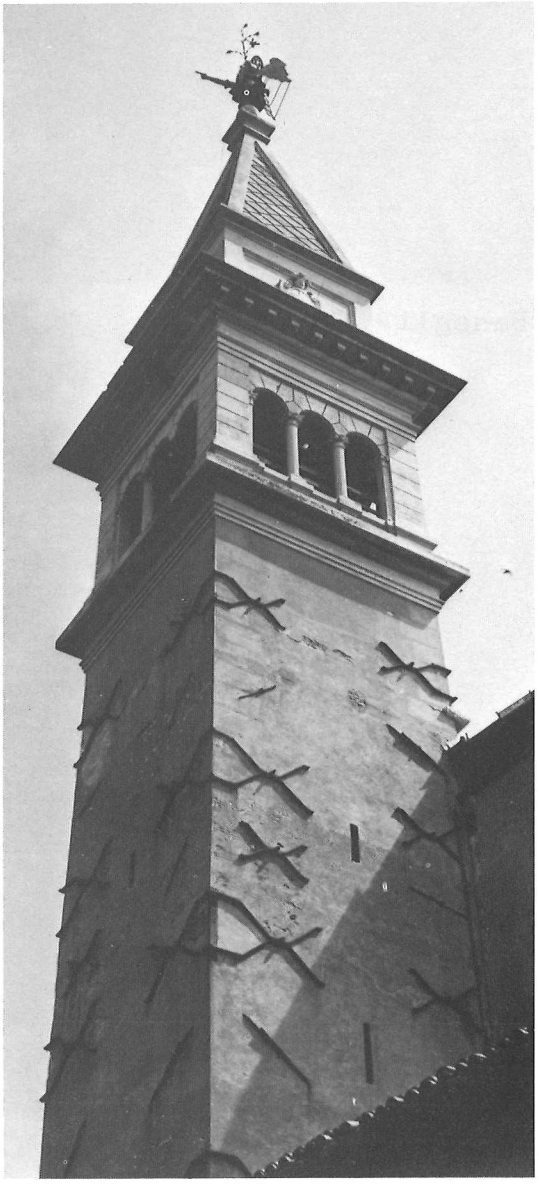


Pignano. Parish church. The bell-tower, with its steel ties, took the stress quite well. 7.46.

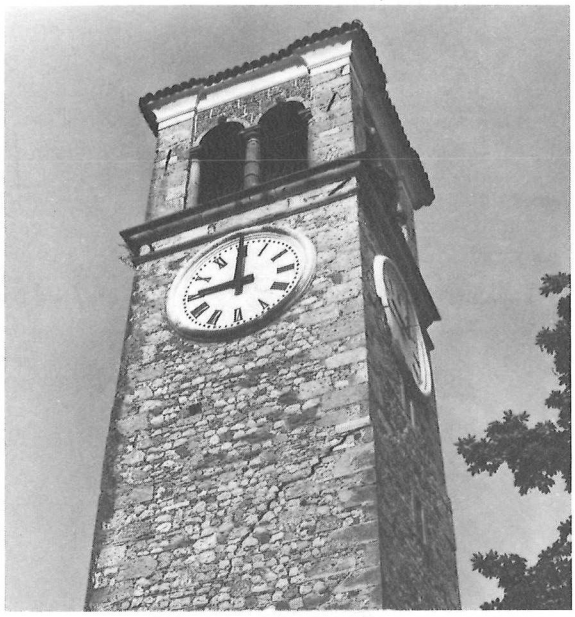


7.47. Tolmezzo. Church of S. Caterina.

Tolmezzo. 7.48.



Bell-tower. Relatively recent restoration (following previous earth tremors) and the use of steel ties led to better behaviour on the part of the building.



Baseglia 7.49.



Mels 7.50.



Colle 7.51.



Cornino 7.52.



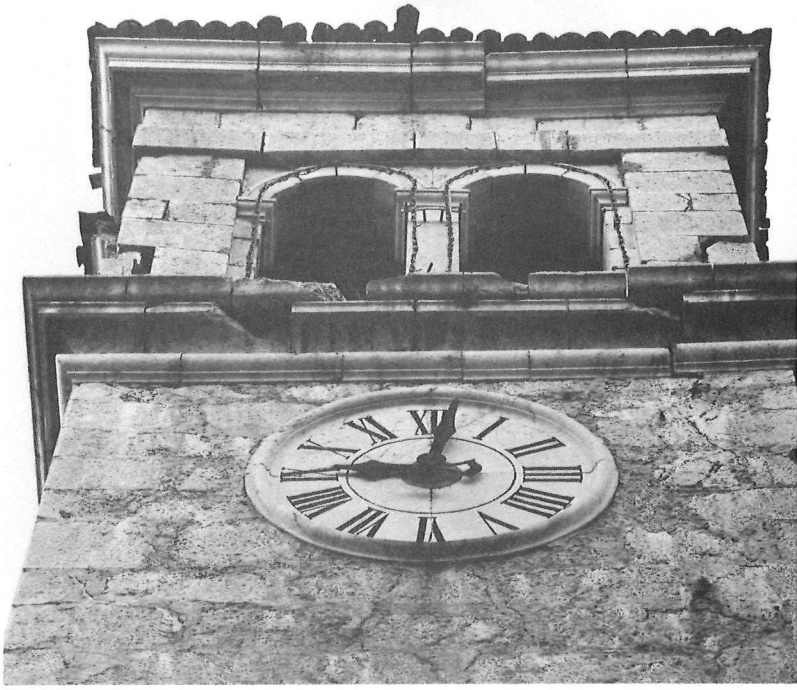
7.53.

Farla

7.54.



Anduins 7.55.

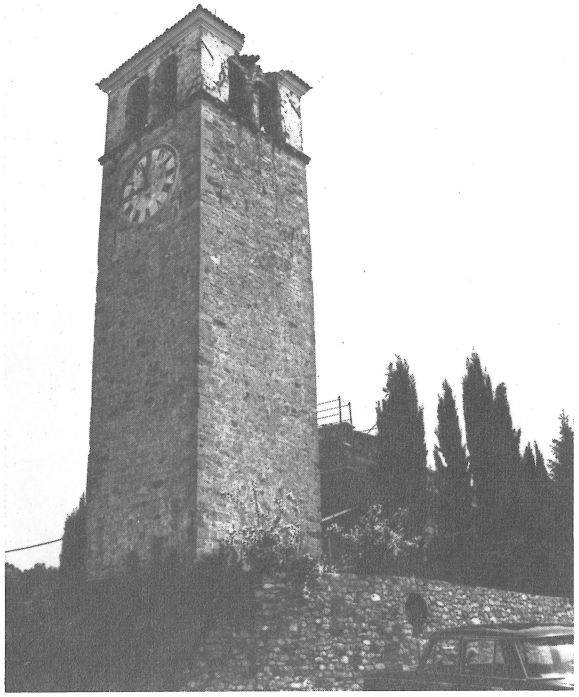




7.56. Mione



7.57. Caneva



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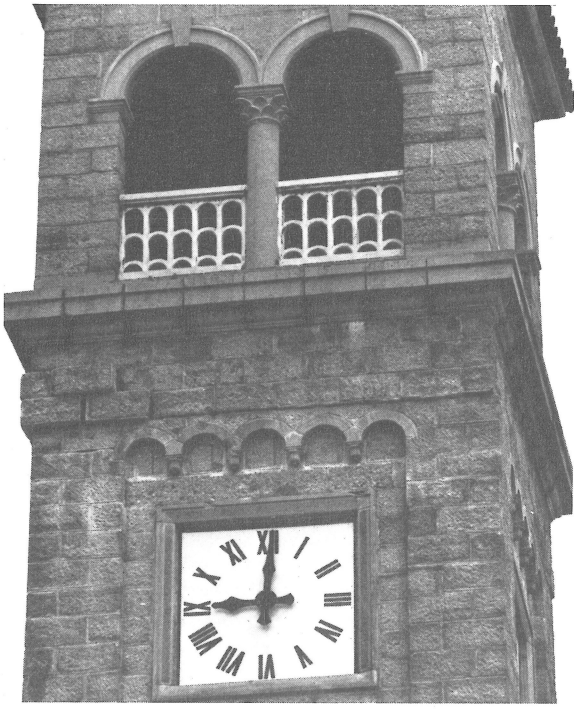

7.59. Forgaria

RECENTLY CONSTRUCTED MASONRY BUILDINGS

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The present paper follows a wider report on the effects on buildings of the earthquake, published on the September issue of "L'Industria delle Costruzioni". This general report was carried out together with a group of the Facoltà di Ingegneria di Ancona under the guide of Professors Gallo Curcio and Tardella.

The earthquake recently happened in Italy pointed out the significance of a research on masonry buildings, which can be considered at least equal with reinforced-concrete buildings, taking into account the wide use of this constructive method. On the other hand, we have noticed that during the earthquake, a number of buildings could endure induced stresses, avoiding first of all injuries to people.

The great importance of such a research is stressed by

the result of a survey and a first analysis of several constructions promptly accomplished days after the earthquake. This result shows a basic lack, mostly with respect to spreading of data and rather exact knowledges, as far as the common behaviour under dynamic action is concerned. This is actually sufficient to justify a specific interest and a need to collect the studies carried out on this argument, so as to undertake a both theoretical and experimental research program. It would be actually a matter of considering the wall a real structure, and relating it with different behavioral-models, in order to find out the various physical features and to focus calculation schemes and procedures sufficiently approaching real behaviour.

The main purpose of this research lies in working out a guide-line to be increased and spread in detail, in revising captiously building systems and materials' quality, which have been considered for centuries ordinary for many "users", very often without a specific expertise. Considering masonry from the point of view of its structural texture, can represent the first broad lines of the problem. By means of the assemblage binder-base element, and in case plastering, it is possible to single out such a texture.

The binder is made up of mortar of different composition and strength, the base element of solid and semisolid bricks, hollow tiles different in dimension and kind, and hollow cement-concrete blocks, lightened in case by means of expanded clay aggregate.

With regard to plastering, it can be made up of mortars

of different components; it is set on the outside and inside walls up to a certain thickness. In any case, it must be considered as a whole, perfectly sticking to the surface of the base element.

The survey of the great majority of damage found after the most recent and documented earthquake, shows first of all an evident correlation between the total energy possible for masonry to take-up under dynamic action without being damaged and the specific strength of the building-texture (Fig. No.1). A quick deterioration in masonry made up of base elements as solid bricks or cement-concrete blocks, because of the forces of inertia induced by earthquake shock, can be easy pointed out in many cases. On the contrary, it has been noticed that masonries made up of base elements as hollow tiles or light concrete blocks weren't strongly or totally damaged (Fig. No.2). Moreover, it has been possible to observe how the presence of a binder more



Figure No. 1 - VITO D'ASIO
Collapse of a solid-brick masonry building. It is quite plain the bad quality of mortar, coming off the bricks.

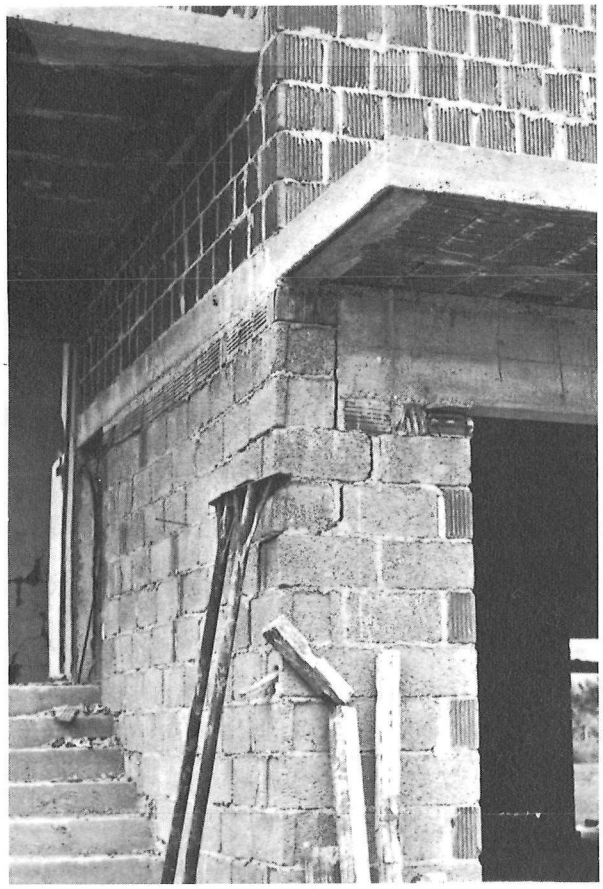


Fig. No. 2 - BUERIIS
Lightened cement-block masonry. It has often proved to be stronger.

or less suitable both from a qualitative and compositional point of view, in connection with the choise of a certain base element, contributed or not to the whole strength of the wall. As far as the presence of plastering is concerned, it has been pointed out in many buildings that plastering improved masonry strength with reference to seismic stress.

Besides, nearly every time, plastering remained perfectly sticking to the surface of the base elements composing the wall, even in seriously damaged masonry (Fig. No. 3). A great variety of intermidiate behaviours has been observed between the two ends, showing after a first inspection a good or a bad total strength of the building-texture against dynamic stress. In this connection, the wall-strength seems to be affected more by particular building-methods rather than by its own composition. We would like now relating the composition of the building-texture with the particular conditions affecting it, as building members, peculiar to masonry or non-masonry structural elements, yet becoming a part of the real



Fig. No. 3 - TARCENTO
Collapses of either outside curtain-walls or indoor partitions are instances of damage occurring very frequently. A correct connection with the roof is therefore essential.

building-texture and interacting with it.

In order to do that, it is necessary a detailed analysis of the more general problem of building-complex just as it is traditionally shaped. By means of this comparison, it is possible to point out structural elements peculiar or not to masonry, which are usually built without a real knowledge of their "weight" and by means of which many damages could be explained. Moreover, it can be characterized a structure-whole made up of masonry, a building "box" so to say, being totally and homogeneously suitable to stand earthquake.

Analysis of the architectural and structural elements of masonry buildings.

Openings represent the first basic element to be consi-



Fig. No. 4 - MEDUNO
The effect of the earthquake is localized in the greatly lightening opening's abutment. It collapsed, causing the total breakaway of the floor from the lower wall.



Fig. No. 5 - BUERIIS
The earthquake's action is evidently localized in the covered entry's abutment. There was consequently the disjunction of two masonry blocks on which a rather stiff reinforced-concrete lintel is fixed.

dered (Fig. No. 4). Openings for doors, windows, building's recess are always a critical point of the building structure (Fig. No. 5). The main thing is just this relationship between the opening's surface and the real extension of the masonry. This means that with

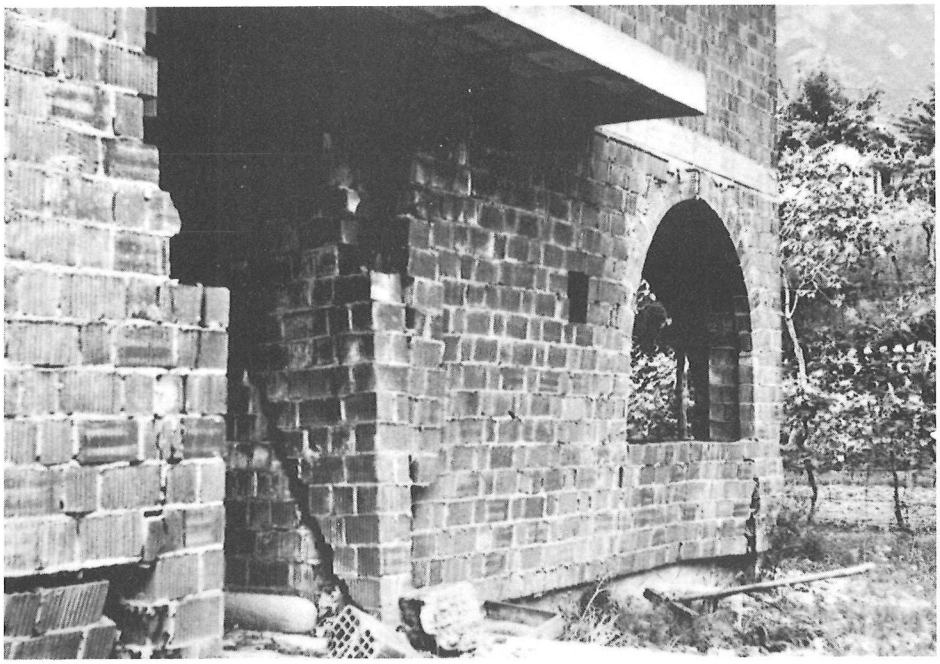


Fig. No. 6 - GEMONA
Partial view of the southern wall: It can be noticed how different shiftings occurred in this building; the first one, as regards the corner, is much more evident because of the presence of thrust elements, as arches, located at the seismic-stress plane.

rather wide openings, which are not fitted out with upper beams, located over the masonry, being sufficiently rigid and appropriately extended at the two extremities, is extremely easy to find out damages in masonries, resulting from the concentration of support pressure on the structure's abutment. When openings are restricted by means of round arches, the behaviour improves considerably, as offsets of compression isostatic-lines in connection with openings, are avoided (Fig. No. 6). No damage at all can even occur, when the arch abut-

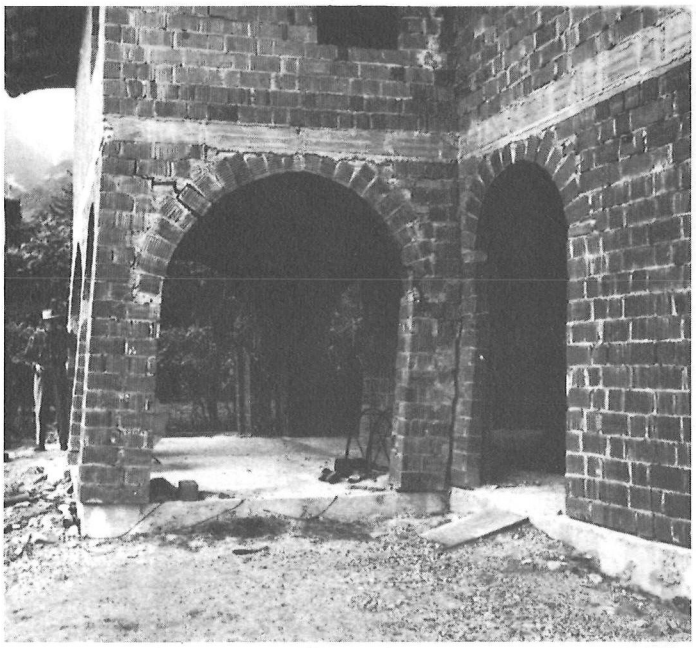


Fig. No. 7 - GEMONA
The lower floor shifted completely, whereas the upper floor was dragged in the movement without suffering any lesions. It can be noticed, on the left, the crack in the arch and, on the right, the breaking on the pier, where the horizontal stress is concentrated.

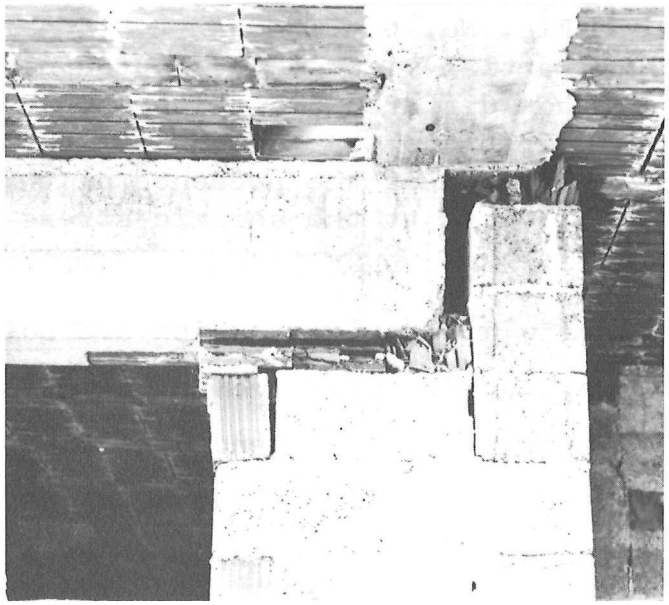


Fig. No. 8 - BUERIIS
Shift of the stiff floor and of the reinforced-concrete beam on the bearing of a pillar and of a masonry abutment: It is evident the crack of the pillar's bearing face, together with the shift of the beam and the breakaway of the abutment's masonry which supports the single-load of the beam.

ment is wide enough to take up the horizontal component of the seismic stress.

The problem of piers and columns can be considered close joined to the presence of arches. The danger comes out from the incapacity of these elements to take in the horizontal

forces (Fig. No. 7). Consequently, in case of horizontal shifts of the upper part of the construction, compression failures will occur at the pier basis and at the upper part of the pier more exactly at the arch reins: The pier works here as a rod.

The situation, in case of isolated columns, is much more critical and dangerous, because they are often located in central position of buildings and are consequently strongly stressed (Fig. No. 8). They support the weight of the main inside beams, which together with the floor stress violently the upper end of the column when the earthquake occurs, cracking almost always the bricks. This situation is quite dangerous, because of a probable eventual collapse.

Serious damages were pointed out, even if these columns were built in reinforced-concrete. The corners are the second significant point for the masonry structures.

We have already mentioned an instance concerning a recess in a building (Fig. No. 9). More generally, we would like now examining the problem from a structural point of view, that is considering the masonry structure a "box" complex, whose corners are obviously the stiffest members (Fig. No. 10). The likelyhood of damages or collaps at the corners has been very often pointed out, especially with regard to the presence of window-openings near the corner itself. The effect of the corner is emphasized by the concentrated stresses due to the opening. We have several examples of such a situation (Fig. No. 11). This is one of the most significant argument to be carefully examined.

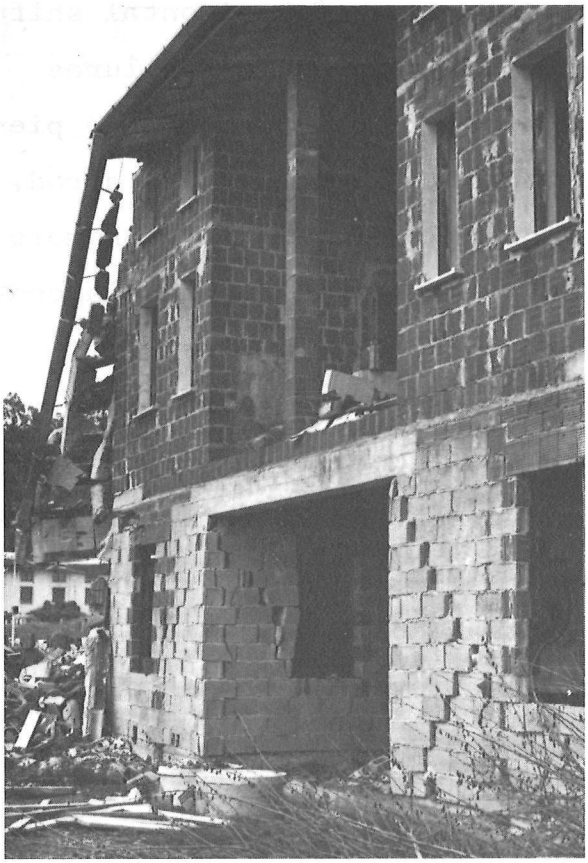


Fig. No. 9 - BUERIIS
The opening masonry-abutments are very often rather vulnerable elements of the building structures.

The prblem of scarfs joining outside with inside walls is similar to that of the corner. Generally, as regards these connections in new constructions, we didn't find serious problems, being the mutual restrain between the two masonry

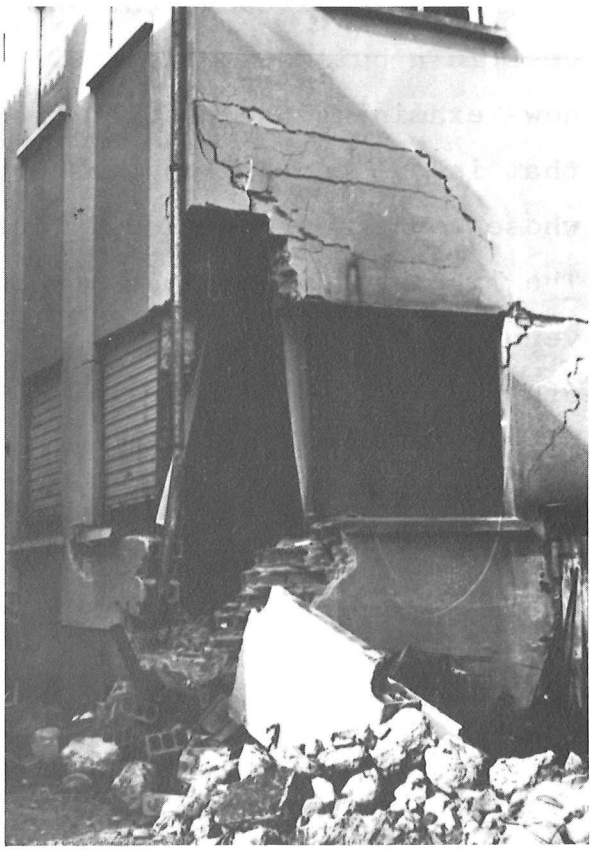


Fig. No. 10 - FORGARIA
The seismic stresses are often concentrated in the corners, particularly when door or window openings are located in the nearby.

texture easy and effective enough.

We would like now to examine now a third kind of problems, peculiar to the connections among the various structural elements of



Fig. No. 11 - FORGARIA
Under the seismic action on a building wall, resulting in several lesions, the corner has proved to be the weak spot; a window opening, too close to it, has made the collapse easier.

the construction. They are, above all connections among the vertical and the horizontal members. One of the main critical points from the seismical point of view lies at the basis of the vertical masonry structure at the groundfloor. Actually (at least in Friuli) the most common kind of residence-building has very often a basement built in a very massive way, that follows closely the ground during the shock. The horizontal force, therefore, acts with great effectiveness just at this level (Fig. No. 12). When, as commonly occurs, because of waterproofing between basement and brickworks, an asphalt layer or bitumenous paper are fitted between them.

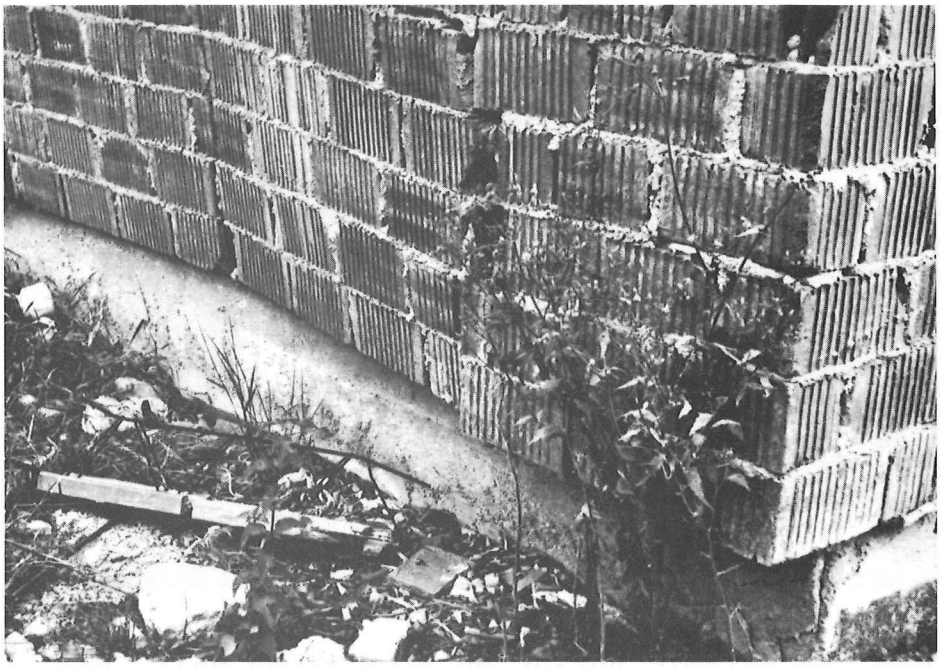


Fig. No. 12 - GEMONA
Southern wall: Detail of the building bearing on the foundation; it can be noticed the shift of the corner different from that of the wall, determining the building-texture breaking.

Their connection is extremely weak and very large shifts occur. These shifts , even if not causing the collapse of the basement, can result in serious problems for the structural members as the pillars (Fig. No. 13), which even if masoned don't follow the shift, since the unit stress at their base is usually really high in comparison with that of a continuous brickwall.

As regards roof, similar problems have arisen because of the lack in connection course with the underlaid masonry.

A similar example was observed in a two-floor house in

Fig. No. 13 - GEMONA
Detail of the arch, where all the horizontal seismic stress are concentrated. It stands to reason how the presence of insulation bitumenous-paper, placed at the base of the wall has made the sliding easier.



Moggio (Fig. No. 14). The building has a structure really monolitic, as far as the good quality of the brickwall (hollow concrete-blocks), the correct construction of the floor and of the beams and a right disposition of the main inside walls are concerned. The only lack which caused a damage, even if not very serious as regards the building static safety, lies in the lack of a top course on the masonry at the roof slab; the whole roof rotated, causing local damages at the corner or at certain walls connected with it (Fig. No. 15).

Finally, damages at intermidiate floors could be pointed out. Often the walls show an horizontal crack at the level of the floors, from which inclined cracks ramify (Fig. No. 16). They emphasize how the horizontal stress exerted by the floors



Fig. No. 14-MOGGIO
No damage at all has been found on the walls of the building (still under construction), but a rotation of the roof as a whole, not being connected to the underlaid masonry, by means of a suitable reinforced-concrete course.



Fig. No. 15-MOGGIO
Detail of the corner bearing-masonry, which sank down and shifted, dragged by the total rotation of the roof.

on the wall concentrates on certain points, as it is not effectively distributed (Fig. No. 17).

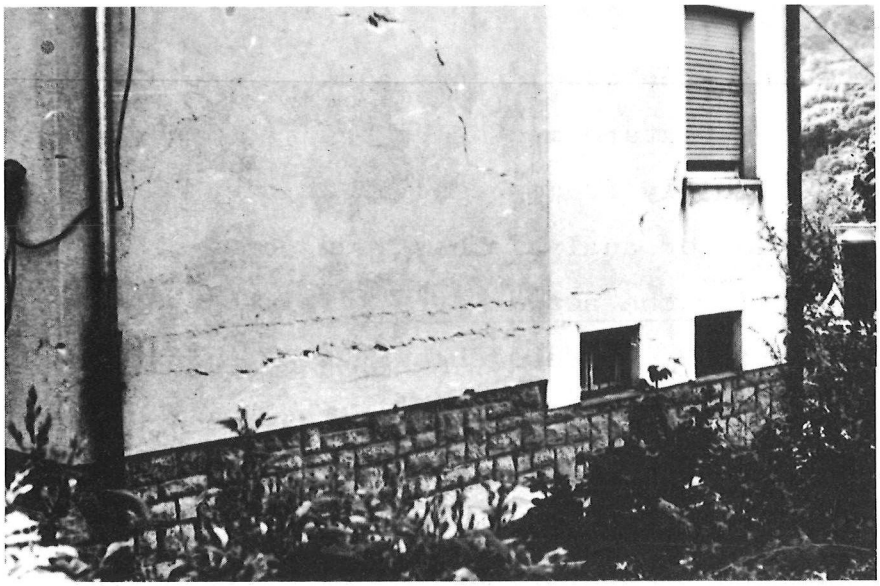


Fig. No. 16
FORGARIA
The stresses induced by earthquake-shocks caused continuous horizontal lesions at a level corresponding to the first floor. The lack in a reinforced-concrete distribution course represents very often the indirect cause of such a damage.

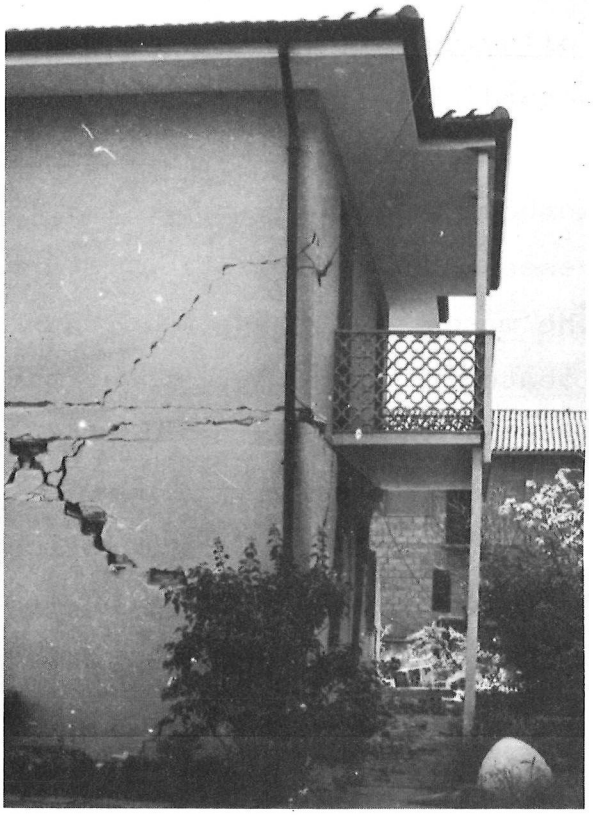


Fig. No. 17 - FORGARIA
There were remarkable damages determined above all by the breaking of the masonry in connection with the floor, lacking in effective courses.

Analysis of the architectural and structural elements of masonry buildings.

Conclusion.

Once the consistency and the various aspects of the building-texture making up the masonry construction as a whole, mostly from the point of view of the damages' description or analysis has been pointed out, we have the need to specify the actual structural function of this complex, very often so heterogeneous; it would be so possible to define a resisting section for the building-texture, so as for reinforced-concrete structures, to specify more accurately the behaviour of a masonry as regards strains or impressed accelerations, range of stability and so on.

Certainly, because of this formulation of the problem, costs of building are likely to increase, as a consequence of the major accuracy of constructive details, often executed following the general rules of "proper building" or "building according to the standard code".

The study we would carry out and whose principles and methods are mostly expressed in this report, is actually a research of optimization, starting from the principle that the masonry, perhaps more than other kind of building, surely located on the outskirts of the big city, must keep costs within certain limits. In this regard, the cost of masonry structure, approaching more or less 40% of the total amount, could tend to increase much more, when restrictive criteria are adopted, so as to reduce the probability of damages resulting from earthquake.

This research, therefore, is to be accomplished utilizing each element present in the building block, in order to

engage the whole masonry, paying particular attention to the inner homogeneity of the building-texture.

An observation of the consequences of the lately occurred earthquake, pointed out an often amazing fact; rather big buildings both as covered area and elevation, weren't damaged at all, even when all the other buildings around it were completely crashed. Very often a good planimetric and volumetric location, but above all the disposition and the optimal dimensioning of masonry panels and an accuracy in scarfing masonries perpendicular to each other, can give an idea of a model masonry enduring earthquake uniformly, by avoiding concentrated stresses and without weak-points.

The procedure of the theoretical investigation and experimental inspection to be optimized, should consist of three stages:

A) Structural schematization of the masonry complex, resulting in a serious of calculation models and eventually in a group of clear rules with a specific theoretical basis, substituting the present "rules of good building".

B) Experimental determination of the physical aspect of the problem: Static and dynamic features of building-texture.

C) Careful study of each constructive detail of brickwork and of the other not really masonry elements, but connected with it.

As far as the first stage is concerned, it can be started considering the already accomplished studies about the masonry complex conceived as a "building box" more or less tightly constrained at the foundation ground.

Conclusion

On the other hand, we would also assume an already existing lack of continuity of connections among orthogonal walls and the consequent distribution of the dynamic stress.

In the second stage, the research must be mostly experimental and carried out for different kinds of masonry, determining for each kind the physical feaures, by means of which the actual behaviour under dynamic action might be described. This stage is really important, as it allows to define the different quality levels of the various brickworks, suggesting above all, for the most common ones, valid criteria in order to improve their "answer" to earthquake. It will be certainly necessary to single out within the masonry the base element resulting the best ones in any case, the most suitable composition both of the connecting mortar among base element and of the inside and outside plastering.

In the last stage, it will be mostly a question of selection among all the constructive elements, which traditionally are a part of the masonry complex and also a question of analysis for each element of its location or of its constructive detail, most suitable to the assumed structural model.

In accomplishing these stages, the main target will be the definition of a range of minimum according to cost criteria, material availability and general social charges. The result this procedures would like to obtain is to make masonry constructions fit for earthquake endurance;

Conclusion.

this not only for buildings located in officially declared seismic areas. The various earthquakes we are informed of, as a consequence of a systematic observation , make us assume that the seismicity attributed to several regions is actually wider and in any case less defined than one can think.

Eventually we would like examining the problem of restoration of lately built masonry constructions, carried out so as to give these buildings the same range of seismic risks conceived for new constructions: In this regard, a repair procedure is not so different to the calculation of a new brickwork but only in the effectiveness to be restored.

One of the most important argument is how to avoid new damages resulting from the same restoration elements added to the already damaged masonry structure, as it was observed in Friuli with regard to some aids given after the earthquake occurred in May 1976 and that after the further shock happened on 15th of September, proved to be inefficient or even worse the cause of other damages.

It would be a significant goal for the society and particularly for technicians to succed in building all the constructions so as to obtain always the same level of safety against earthquake and in this regard our research could result in working out an effective code of practice: The comparison of the Italian present code of practice regarding building in seismic areas (DM 331965) with the damages and defect observed, shows clearly a number of lacks. This could be avoided by means of a more careful analysis of the events.

Conclusion

REPORT ON DAMAGES AND PROVISIONS ATTAINING TO EARTHQUAKE CONCERNING THE ELECTRICAL SYSTEM

G. F. FAORO - G. FERRAZZA
SETTORE PRODUZIONE E TRASMISSIONE
ENEL - VENEZIA

1 - ZONE INVOLVED IN THE EARTHQUAKE

The subject of our research is the zone which converges in the Friuli-Venezia Giulia region whose territory has been subdivided into areas of seismic intensity, according to the provision of the regional law no. 15 - May, 10 1976.

1.1 - Enel plants in the region

The Enel production plants are represented by the steam generating station in Monfalcone (365 MVA), the hydroelectric power plants in Somplago (180 MVA), Ampezzo (78 MVA) and by other hydroelectric power plants of minor size.

In the area there are 11 interconnecting substations with a total conversion capacity of 1190 MVA and a network composed by 292 Kms of 220 KV voltage, 566 Kms of 132 KV and 85 Kms of 60 KV.

The distribution system is fed by the production plants, by the interconnecting stations, and by 8 distribution sub-stations.

The radial system has 397 pole mounted outdoor transformation points and 686 indoor installations, 1000 Kms of 10÷20 KV network and 3000 Kms of low voltage network.

2 - RESULTING DAMAGES FROM THE EARTHQUAKE

2.1 - Chronology of the events

The phenomena started at 8.59 p.m. on May 6th with a first waveshock which registered 6.5 points on Mercalli scale reaching the maximum at about 9.01 p.m. with a shock of 9 points on Mercalli scale for less than a minute. The acceleration of oscillatory motion was 0.4÷0.5 g in horizontal direction and 0.1 g in vertical direction.

Following the earthquake all the generating plants in the region tripped, including some in the outside area as Torbole (130 MVA), Pederobba (3.5 MVA) and Croce del Gallo (3.5 MVA) located at 180, 100 and 90 Kms from epicentral area. The same happened to the interconnecting and distribution transformers, particularly to the ones installed in the switchyards of Dolo and Dugale (380/132 KV), Scorzè and Vellai (220/132 KV) and Caerano (132/60 KV) located at about 100÷150 Kms from epicentral point.

The restoration of system was gradually done during the same night and the following days : in the morning of May

7th 165 distribution cabins were left unfed, and by the evening of May 8th the number dropped to 69; in the evening of May 10th most of them were switched in the network.

2.2 - Damages in the central area

Somplago plant : with regard to the buildings there were cracks in the roof of the switchboard room, of the workshop, of the dining hall and the storehouse; there have been also damages due to landslide action involving the road to the butterfly valves (see picture no. 1).

The structures for the hydraulic system have been heavily damaged in the covering of the tail water canal (see picture no. 2); we also had the fall of two beams from a bridge of a new highway under construction (see picture no. 3), the settling of the left bank of the canal and cracking in the concrete of the dissipating device (see picture no. 4). On the other hand there have been no damages to the Ambieste and Lumiei dams. The height of the two dams is respectively 59.23 m and 36.50 m and the crests' length is 144.64 m and 152 m.

The electrical switching devices have been severely damaged : 18 single pole oil circuit breakers - of a total of 21 - came down; the same happened to 209 insulator elements of a

total of 580 due to porcelaine cracking, we also had breaks in the contact sections of the disconnecting devices, in the pneumatic operating mechanisms and associated pressure lines and overstressed in the joints of the busses (see pictures no. 5 - 6 - 7).

Campagnola plant : the main damages were located in the brickworks and in the hydraulic structures with splits and displacements along the head race, in the wicket gates of the overtaking duct, in the control building; the electrical battery felt from the stand off insulators. The upsetting of a transformer 10/20 KV (1.5 MVA) and the displacement of another 6/10 KV (1 MVA) transformer was also noticed.

Pineda plant : there have been breaks along the wall of the by-pass canal and in the control building besides lesser damages to the equipment.

Campolessi plant : breaks took place in the building and the battery has been damaged.

Savorgnana plant : considerable breaks occured along the head race bed (see picture no. 8) besides damages to the 10/20 KV 3 MVA transformer.

Minor plants : some minor power stations suffered considerable damages; these plants are sometimes used as distribution cabins or warehouse for equipment and other activities.

High voltage transmission lines : no damage has been noticed.

Distribution installations : it is worth to point out that in the S. Daniele and in the Buia primary cabins the HV transformers weighting 70 and 100 tons were displaced and derailed (see pictures no. 9 - 10); in many other substations the destruction of insulating elements and the overturning of the baskets containing batteries was almost complete. The distribution cabins have suffered substantial damages; in fact 152 of them (45% of the total) have been involved; 77 were completely destroyed and 36 heavily damaged (see pictures no. 11 - 12 - 13). The preassembled cabins suffered no damages.

The damages to distribution feeders came from an indirect source : landslides, rocks falls on the supports, falls of cabins where the lines were connected; the cable lines suffered the worst damages in the terminals inside of the destroyed cabins. The pole mounted transformers, as well as the other supports, suffered no particular damages although in some cases there has been the displacement and the overturning of the transforming equipment.

The low voltage network outside the central areas suffered no great damages whereas the network located inside suffered considerable damages.

2.3 - Damages in the median zone

There are less damages than in the central zone :

- in production and transformation plants damages occured to the buildings of some minor power houses and to the Tolmezzo station
- in distributing plants, 31 cabins have been damaged, equal to 5% of the total, 4 of which demolished and 9 heavily damaged; 20 Kms of feeder lines have been destroyed and 870 Kms damaged; of the low voltage lines 700 Kms have been destroyed or damaged and 19,000 consumers'connections have been destroyed.

2.4 - Damages on the peripheric zone

The damages here are almost negligible, but for the heavy breaks found in the piezometric towers of the S. Foca and Villa Rinaldi power houses; the height of the two towers is 51 m.

The two stacks of the Monfalcone steam generating station did not suffer, their height is 90 and 66 m.

3 - INTERVENTIONS TO RESTORE SYSTEM OPERATION

Production and transmission plants

The interventions carried out in the damaged plants can be summarized as follows :

Somplago plant : in order to restore the operation of the hydraulic system the banks and the bed of the last segment of the tail water canal have been given a lining of screened concrete providing afterwards the reconstruction of the opening into the Leale stream. In addition the construction has been strengthened by cement injections and by setting micropoles.

At the same time effort has been made in order to restore the electromechanical equipment of the switchyard by replacing the damaged units with the ones recovered from other plants. The plant has been put in service gradually, some services of particular necessity coming first.

Ledra Plants : essentially the restoration of the hydraulic system has been reached by lining the banks with screened concrete and by patching and plastering the buildings' cracks. The electromechanical system has been restored with a relocation of the transformers displaced by the quake.

S. Foca and Villa Rinaldi plants : repairs of the cracks in the piezometric towers have been made first by internal sealing with epoxy compounds of the horizontal breaks in order to keep the hydraulic system at work. After that a special treatment has been given to the towers by exterior reinforcement with metallic armature bolted to rings and a concrete kerb in the tower foundations.

Distributing plants : the damaged plants have been put in operation after repairs of the dorsal lines of the medium voltage network and of the supply lines. The demolished or unusable cabins have been put out of the network; where necessary they have been replaced by pole transformation points or by preassembled metal cabins and with the new transformation points the low voltage network was switched in gradually, facing heavy problems, since most networks have been destroyed and the few operating ones did interfer with the demolition and construction activity.

Altogether the restoration activity required the installation of 77 new transformers, 82 cabins, 2,300 poles and 412 Kms of low voltage cable.

From a chronological point of view it is possible to realize the progress reached in the repair work through the analysis of the load of the transformers as referred to in the Table 14 where the data pertinent to a few days after the earthquake are compared to the previous day ones.


All. 1

All. 2

1.484

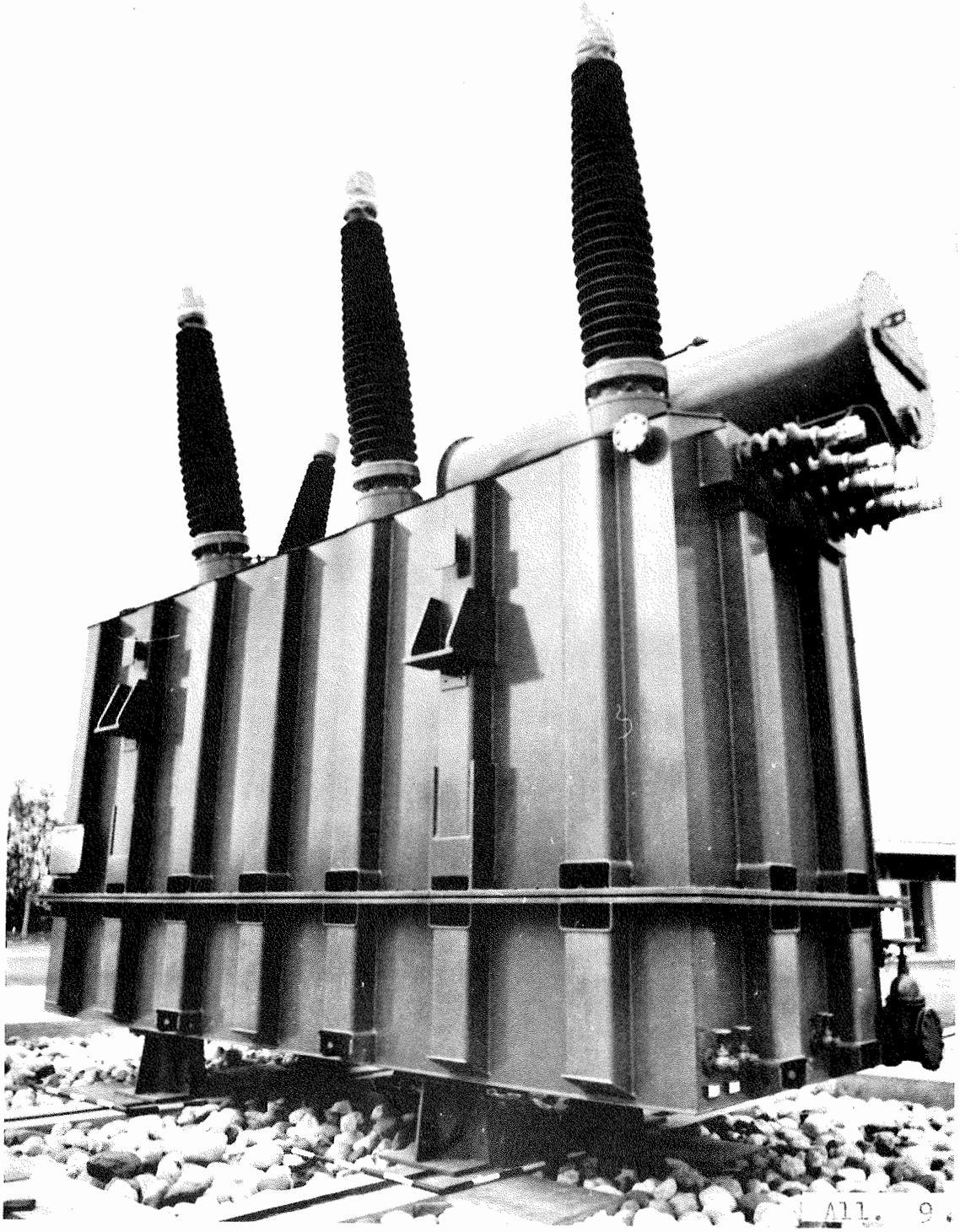
All. 4

All. 5

A11. 6

All. 7


A11. 3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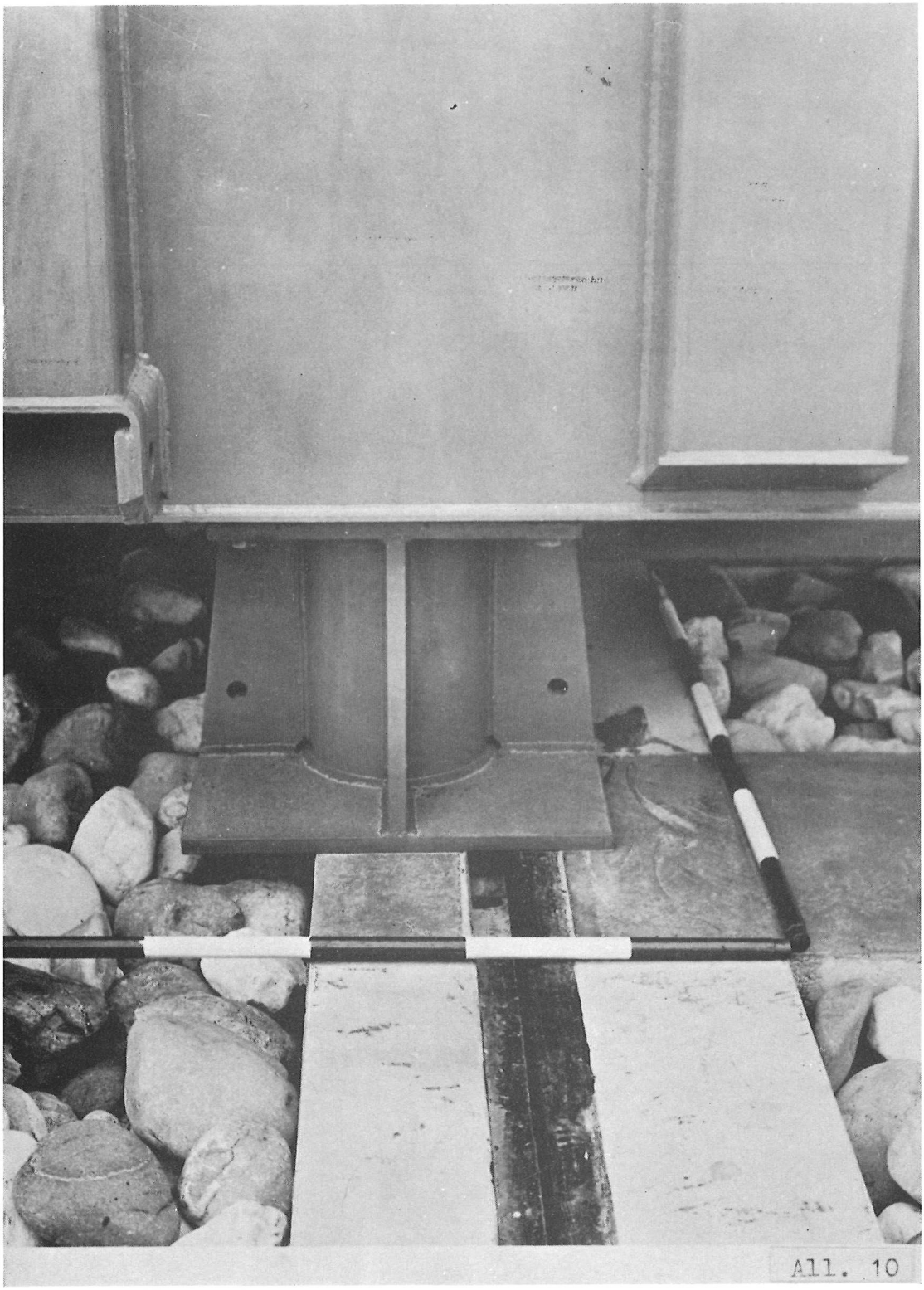
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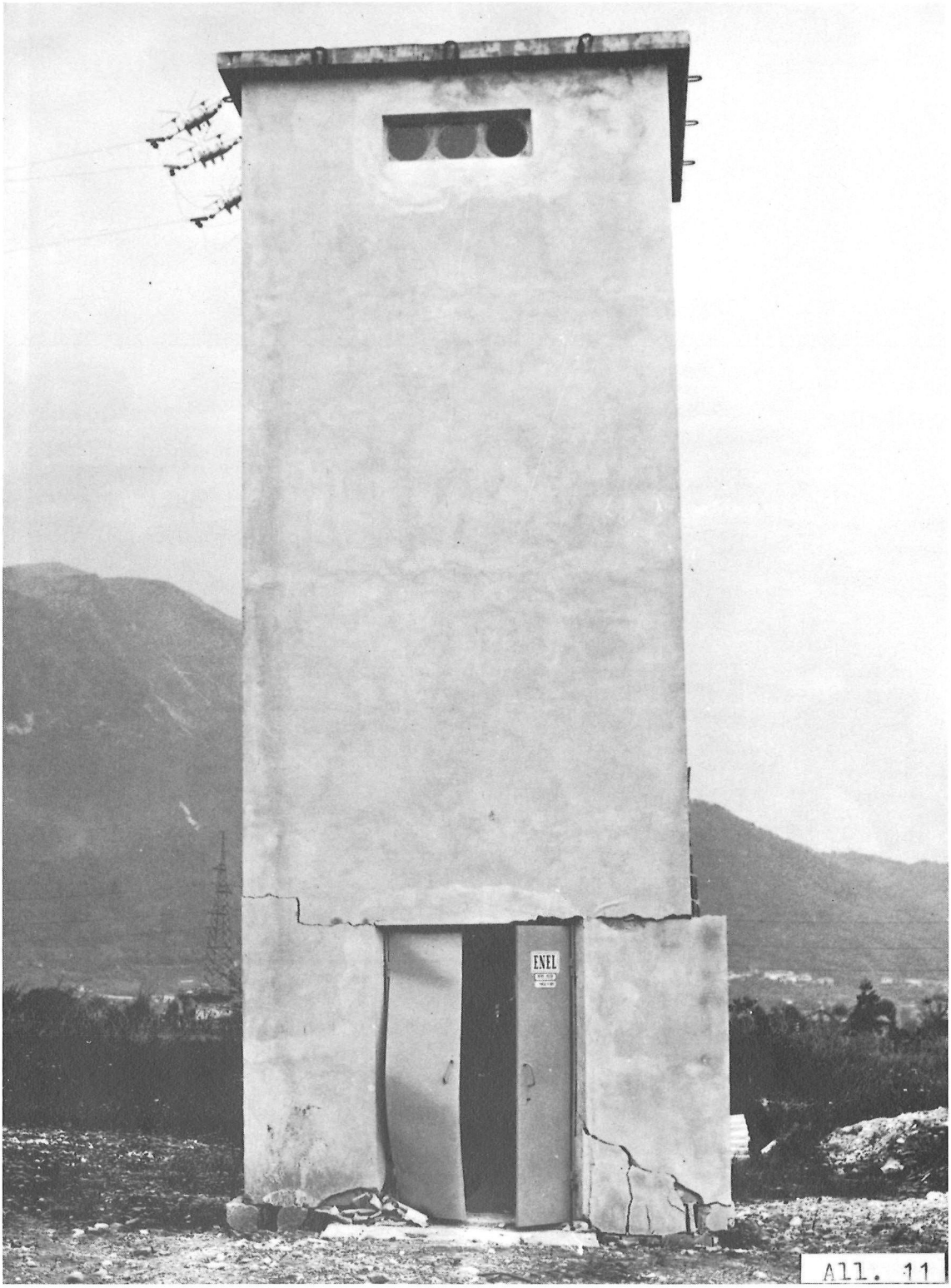
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ELECTRICAL LOADS (MW) IN THE AREA

Cabins feeding mixed loads

Cabins	May 5	May 7	May 12	May 19	May 26	June 9
S. Giovanni	25	9	22	23.8	24	25
S. Daniele	20.5	4	7	12	14	17
Udine Rotonda	23.5	10	20	23	27	26
Udine H. E.	31	10	22	27	25	30
Maniago	9.9	2.2	7	9.3	9.9	10.5
Tolmezzo	9.5	0.7	3	5.5	7	7.5
Pradamano	4.6	2.7	4	4.3	4.3	4.5

Industrial load tied to high voltage system

Lines	May 5	May 7	May 12	May 19	May 26	June 9
Ferio	9.1	0.5	0	8	1.7	8
Safau	13	1	12	14	15	15
Bertoli	10	0	10	10	2	10
Cementificio Travesio	9.6	3.5	6.7	6.3	7.6	8

Table 14

COLLAPSE OF A NON SEISMIC RESISTANT R.C.BUILDING DURING THE FRIULI EARTHQUAKE: THE "ASTRO" BUILDING IN MAIANO

by

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SYNOPSIS

The ASTRO building in Maiano, like many other similar structures,collapsed totally during the strongest shock of the Friuli earthquake in May the 6th 1976. By means of a dynamic elastic and a static inelastic analysis, the most likely mechanism of failure is established, and the associated seismic resistance is evaluated. The adequacy of the existing code in providing safeguard against failure at the accurence of very intense ground motion is commented.

INTRODUCTION

The Friuli earthquake struck over an area which for its largest part was previously considered as seismically non active. As a consequence of this, many modern r.c. buildings which had not been designed for earthquake resistance suffered unrecoverable domage or collapsed.Typically,the number of stories of residential or office buildings in this region ranges from three to six, their structure consisting of cast-in-place frames connected by hollow tile-r.c. ribs floors. One of them, six stories high,

has been chosen in order to demonstrate a frequent mechanism which has led these structures to failure. This can be instructive, since many of the failures could have been averted simply by a more suitable arrangement of the structural system, without any strengthening.

The overall ductility of the structure is then assessed with reference to a one-degree-of-freedom model suggested by the results of the elastic dynamic analysis. Non linear material behavior, cracking, and P - Δ effects are included in the analysis of the model. From the ductility measure, the maximum earthquake intensity that the building could have sustained is inferred

STRUCTURAL SYSTEM

The plan of the ASTRO building and the elevation in the N-S (longitudinal) direction are shown in fig.1. Along this direction, the structure was made of five continous frames, while the lateral stiffness in the E-W direction was provided only by two frames and by the system formed by the floor slabs, the N-S beams,and the columns. Since the torsional rigidity of the beams was rather low, both the stiffness and the strength of the structure along this direction would have been not much higher than that of the sum of all the isolated columns. However internal partitions and exterior infills panels, made of hollow brick masonry, exerted a stiffening effect which was considerable in the initial phase of motion. In the ASTRO building, the first floor was open along the contour; internally there were only two infilled brick walls in the direction E-W, running at both sides of the staircase. At the upper stories, the central longitudinal frame (N-S direction) was infilled with masonry, while in the transverse direction (E-W),in addition to the staircase walls, the side frames of the building were also infilled.

Due to this arrangement of structural and non structural elements, the longitudinal direction resulted as the weakest fot the building, the first story in this direction being "soft" with respect to all the others.

GROUND MOTION

At the time of the most intense shock no accelerograph was placed in Maiano, the site where the ASTRO building stood. The nearest instrument was in Tolmezzo, some 20 km distant. However since both sites are at approximately the same distance from the epicenter (~ 15 km), and since no major modifications of the ground shaking are expected as a consequence of local geology conditions, the accelerogram recorded at Tolmezzo can be taken as representative for Maiano too. The N-S component of the ground motion is presented in fig.2. The peak acceleration is 0,367 g, the dura

tion of the intense phase is no longer than 10 secs. The E - W component has similar caracteristics, with a peak of 0,315 g, and the vertical component a peak of 0,277 g. The acceleration response spectrum of this earthquake is plotted in fig.3, for the damping ratios of 0,02 and 0,05. The range of periods for which amplification occurs is 0,15 - 0,5 seconds, with the largest peaks at about 0,3 seconds. Beyond the value of 0,5 sec there is a sharp decrease of the response, indicating that the prevailing frequencies of the earthquake were above the 2 Hz. The spectrum of the recorded motion is compared in fig.3 with the design spectrum official in Italy for the sites of high seismicity. To evidentiate the differences in the form, the design spectrum has also been scaled to approximate the actual one (for 0,05 damping) in the zone of maximum response. The scaling factor adopted in the diagram is 0,75/0,1 = 7,5, i.e. the actual elastic response would be 7,5 times larger than the design one. The factor 7,5 corresponds also to the ratio of the maximum accelerations of the actual and design earthquake: 0,367/0,05 $\cong$ 7,3 (0,05 g is approximately the max acceleration corresponding to the response value of 0,1 g).

EARTHQUAKE RESPONSE

Elastic Analysis

The mode shapes and associated periods of the building obtained using the computer program ETABS (1) , are illustrated in fig.4. For the analysis beams and columns have been assigned area and moment of inertia based on their initial section, and a common value of the modulus of elasticity: E = 250.000 kg/cm^2. The infilled panels have been modeled as reacting by shear only, with a modulus G = 80.000 kg/cm^2, and a reduced thickness (s = 2,5 cm), derived in order to roughly reproduce their hollow section and the reduced stiffness caused by some openings.

The symmetry of the building yields modal shapes which are uncloupled with respect to longitudinal and transverse axes. The first mode is rotational, then follow the two fundamental modes in the N-S and E-W directions, and the higher ones in the same order. There is a marked difference in the form of the two orthogonal fundamental modes: the E-W has approximately a "shear type" form, due to the prevailing effect of the shear panels which exist all over the height. The N-S instead, shows a concentration of the deformation at the first story, amounting to nearly half the displacement at the top.

The maximum stresses and deformations for both directions have been calculated by vectorially summing the modal maximum responses: about 98% of the total response, however, is contributed by the fundamental modes. In applying the response spectrum it was considered not appropriate to fol

low the sharp discontinuities of the particular spectrum available,which would have made the results too sensitive to small variations of the structural properties. Within the interval between the two fundamental periods of the structure the smoothed spectrum can be approximated by an horizontal line. The actual constant value adopted in the analysis is: S_a = 0,1 g : by inspection of fig.3 one concludes that the values of the forces developed in the structure if it remained elastic were proportionally higher by a factor 7,5.

Results of elastic analysis. The value S_a = 0,1 g is that prescribed by the national code for high seismicity sites. It is thus interesting to investigate the state of stress produced by such an earthquake in a structure which was not designed for it. From the dynamic analysis, the maximum base shears developed in the E - W and N - S directions are : T_{max} = 130 t and T_{max} = 122 t respectively. In the E - W direction, the two infilled walls could offer a resistance five or six times greater than 130 t before attaining the cracking stage. Along the N - S direction on the contrary, the base shear is resisted by five frames, the central one absorbing nearly half of the total force. In fig.5 it is shown the bending moment diagram along the height of a column: the stresses at the ground floor are quite high, and fall off rapidly at the upper stories due to the restraint offered by the panels. For the column shown the stresses in the concrete and in the steel at the base section are σ_c = 125 kg/cm^2 and σ_s = 1150 kg/cm^2, to be compared with the strength values of R_c = 250 kg/cm^2 and R_s = 3.600 kg/cm^2, respectively. These results, which are similar for the other columns, indicate that the longitudinal frames can withstand the design spectrum, even if the stresses exceed the admissible level. However, the reserve strength of the building in the same direction is certainly not compatible with the elastic forces produced by the real earthquake.

Inelastic analysis.

Due to the significant difference both in stiffness and in strength,it is clear that even in presence of equal and simultaneous ground motions in both directions, the building would fail in the N - S direction. The mechanism of collapse is suggested from fig.5. The first story of the frame, having the bays free from the panels, is highly stressed, while the upper part of the structure can be comparatively considered as undeformed. With the increase of lateral actions, the first floor would continue to drift rigidly, until plastic hinges would form at the base and at the top of the columns.

Inelastic response. Based on the preceding remarks, the mechanism of collapse of the building has been simulated by a single-degree-of-freedom model, illustrated in fig.6. A rigid diaphragm connects the five frames, which are truncated at the first floor level. The actions transmitted by the upper part however, are included in the model; they consist of: vertical gravity loads on each column, vertical loads on the columns deriving,

by virtue of frame action, from the horizontal forces acting at the upper stories, and a lateral force applied at the first floor level representing the resultant lateral action. The normal forces in the columns due to frame action are taken proportional to the horizontal resultant, keeping the same factor of proportionality given by the elastic analysis.

A static analysis of this model, for monotonic increase of the lateral force up to collapse, has been performed by means of the computer program NLRCFR (2), suited for the non linear analysis of r.c. plane frames. The program deals with the cracking and the plasticization of the sections by subdividing them in a number of strips and following step by step the anelastic stress path of each one. It includes a "geometric matrix" which accounts for the nodal deformations, up to the level of "finite displacements". The following properties were assumed in the computer runs: the beams remain elastic; the column behavior is described by that of nine internal cross-sections; the deformability is assumed to vary linearly between the values calculated at each of them; the number of concrete strips in a cross section varies from 20 to 30, according to the section height. The σ-ε law for concrete is the C.E.B. parabola rectangle; no consideration has been paid to degradation of stiffness due to cyclic behavior and spalling. A bi-linear curve has been used for steel, the strain hardening modulus being 2% of the initial modulus E.

Results of inelastic analysis. The diagram of the lateral resultant F versus lateral displacement Δ of the model is plotted in fig.7. The first part of the curve, near the origin, reproduces the results of the elastic analysis of the entire structure: for the value: F = 122 t the displacement of the model is 2,5 mm and the max concrete stress of the base of a column is 125 kg/cm^2 (see fig.5). For higher values of the lateral force, cracking and non linearity of the concrete first affect the shape of the curve. Subsequently, for values of Δ in between 9 and 12 mm all the columns undergo yielding of steel; then the curve continues further with a very flat slope, due to strain hardening of steel.However the slope is progressively decreased by the 2nd order effects, until collapse occurs for crushing of the concrete.

The collapse value of F is 300 t, which corresponds to a load factor of 300/122 = 2,46 with respect to the elastic checking value.

The ultimate displacement is situated presumably between 30 and 35 mm: the first value corresponds to 50% of the columns having concrete strains in compression larger than 3,5/1000, the second larger than 5/1000.

A reference bi-linear F - Δ diagram appears in the figure: the knee of the curve has been conventionally chosen at the point Δ = 11,5 mm,this point marking the onset of noticeable unreversible deformation and corresponding to 50% of the columns having steel yielded. The maximum ductility of the structure has been evaluated from it, resulting something less than 3 (including the P - Δ effects).

Following a well established approximate criterion, spectrum intensities can ben related to ductility ratios: to the range of ultimate values found for Δ there corresponds a range of spectrum intensities from 6,5 to 7,5 times larger than the design one. The Friuli earthquake spectrum intensity is at the top of the range. This result justifies on one hand the collapse of the ASTRO building, but explains as well why some other buildings of similar caracteristics could stand, even if their structure was unrecoverably damaged.

CONCLUSIONS

From the analysis of the lateral behavior of the ASTRO building a value of 3 was obtained for the ductility ratio, which proved insufficient to prevent the collapse of the building. While the analysis made did lack in some features, as for example the degradation and deteroriation of the concrete due to cyclic straining and to spalling, it is believed that the indications obtained are accurate and dependable enough to base on them some general conclusions. First it is once again confirmed that structural systems leading to yielding mechanisms with plastic hinges at the first floor columns only are unsuited, because of the difficulty of endowing them with sufficient ductility.

For a non ductile structure a higher elastic design spectrum is required. The building which has been examined was almost complying with the seismic code prescriptions, so that its collapse reveals some code inadequacy. A ductility check should be prescribed and a finer subdivision of the design spectrum intensities should correspond to the calculated ductility ratios.

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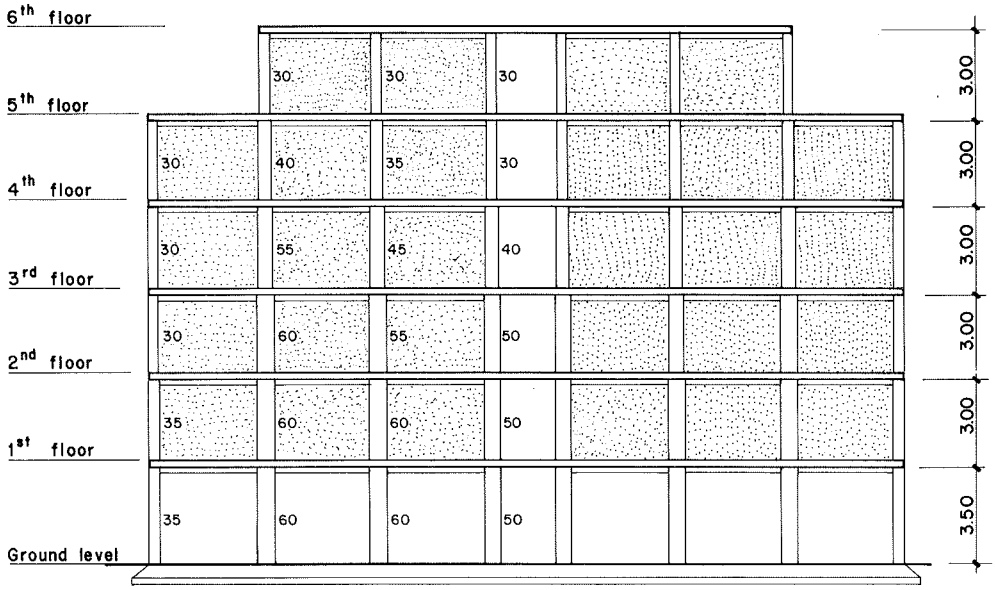
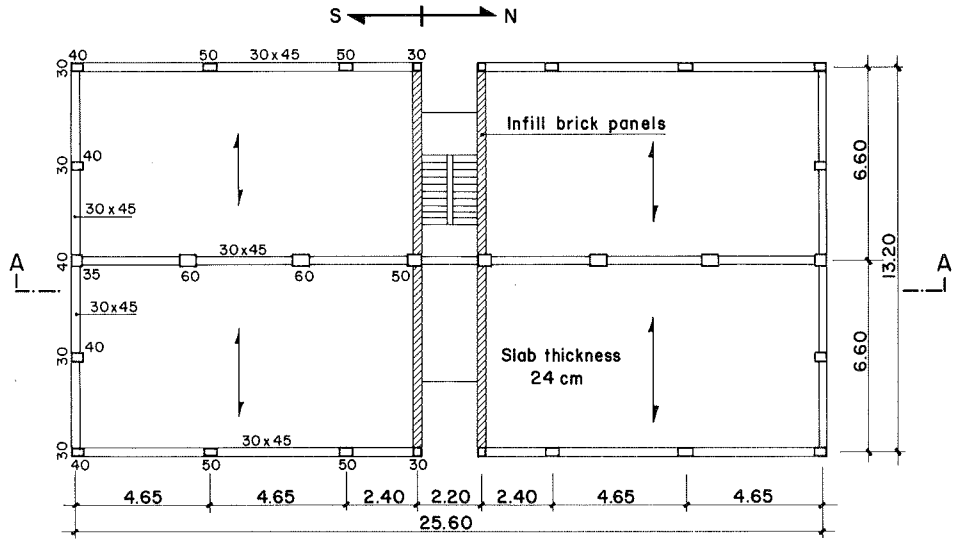


Section A-A



Ground floor

Fig. 1 - PLAN AND ELEVATION OF "ASTRO" BUILDING

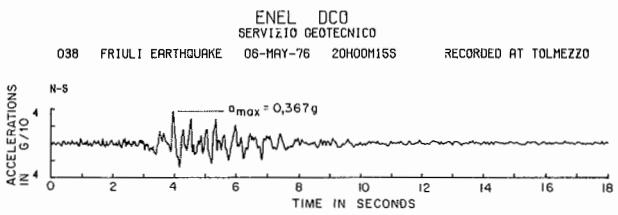


Fig. 2

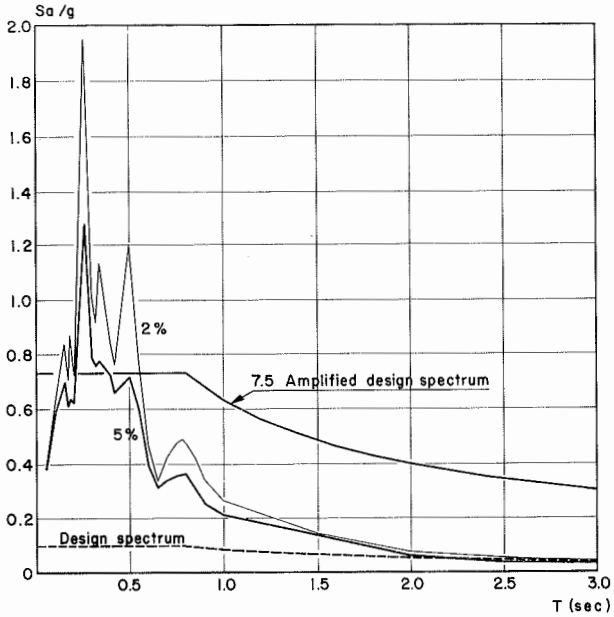


Fig 3 - FRIULI EARTH. AND DESIGN ACCELERATION RESPONSE SPECTRA

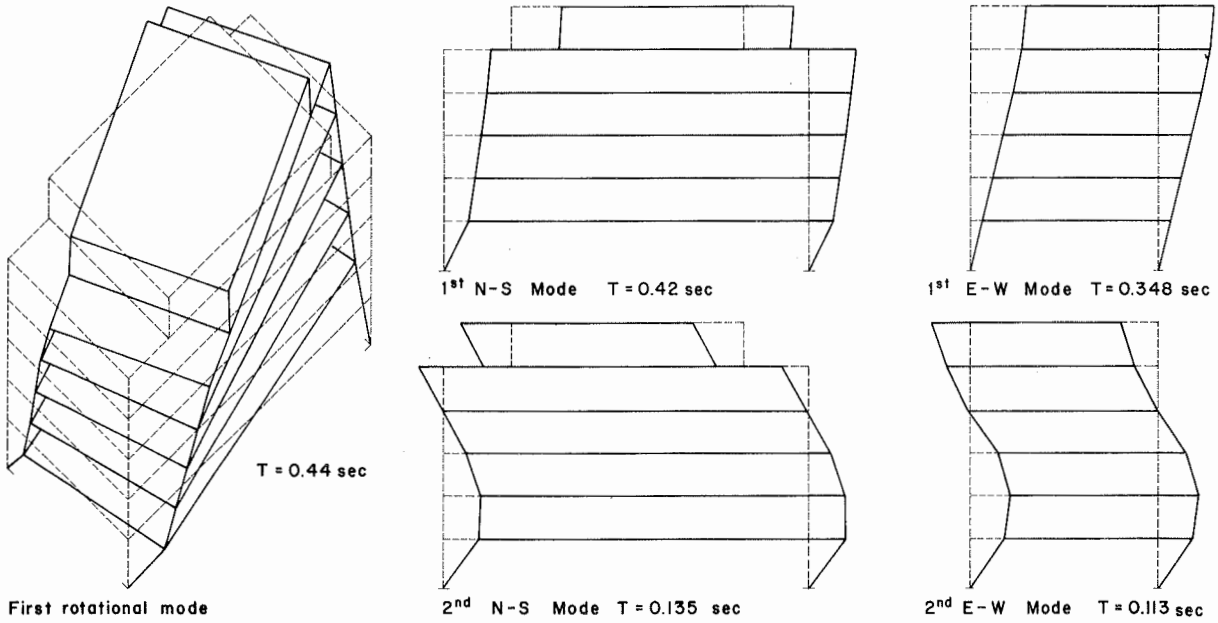


Fig. 4 - MODE SHAPES AND ASSOCIATED PERIODS

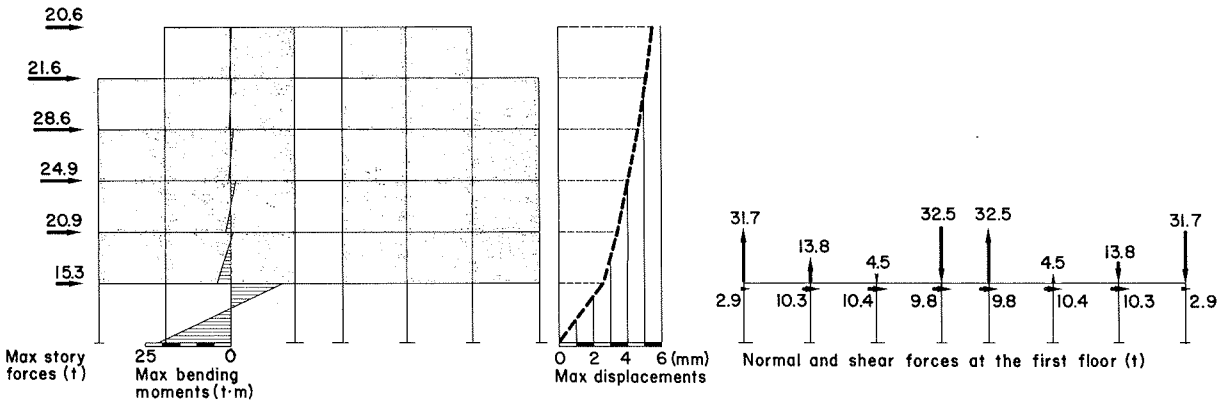


Fig. 5 - N-S CENTRAL FRAME - RESULT OF CODE DYNAMIC ANALYSIS

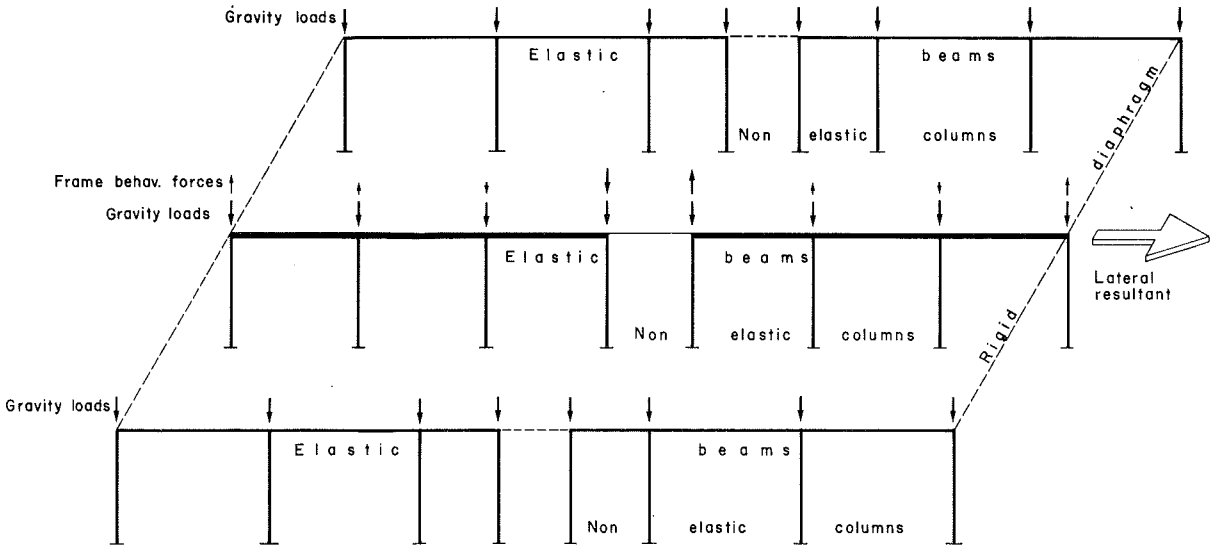


Fig. 6 – O.D.F. MODEL FOR INELASTIC ANALYSIS

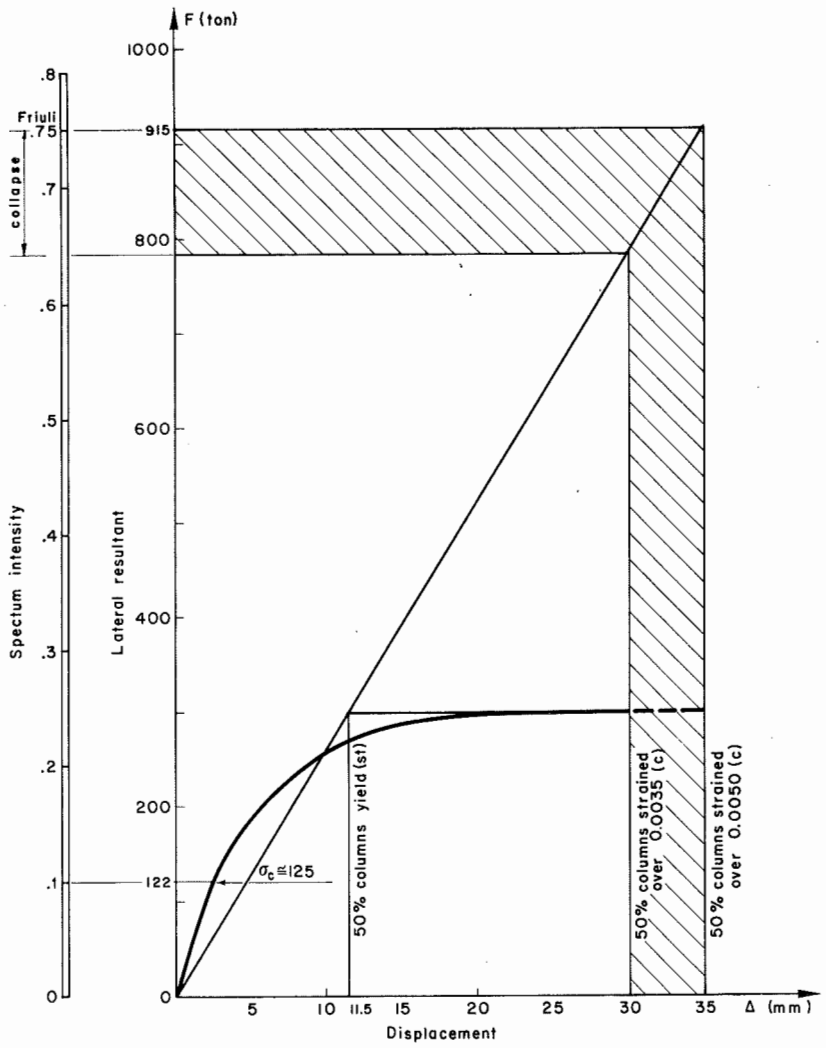


Fig.7- CALCULATED FORCE-DISPLACEMENT RELATIONSHIP OF THE 1st STORY- DUCTILITY AND EVALUATION OF COLLAPSE SEISMIC INTENSITY

ASSESSMENT AND INTERPRETATION OF BUILDING DAMAGE

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ABSTRACT

The area of Friuli hit most severly by the May 6th, 1976, earthquake was investigated by a group of Swiss engineers four weeks after the event occurred.

The main goal was to study the effects of the tremor on various types of structures with respect to current code and design practice.

Information on all types of damages occurred was collected and evaluated. For typical structures and structural elements, rules for improved earthquake resisting design were found and will be presented. The damage investigation provided a data basis for a macroseismic map.

A detailed description and photographic documentation are available in a summarized form in Ref. (1).

SIGNIFICANCE OF DAMAGE ASSESSMENT

A science evolves only when its conceptions and theoretical models can continually be put to the test on the research object itself. This is relatively easy to do in the laboratory. It is, however, much more difficult to perform when the object must be observed in nature. It becomes even more difficult when the natural phenomenon to be studied occurs only seldom and then without prior warning to when or where.

The earthquake specialists assembled at the second international UNESCO Conference in Paris (Ref. 2) were of the unanimous opinion that, in order to reach a better theoretical understanding of the effects of earthquakes, more basic knowledge must be procured. This can be achieved only by thorough observation of the effects of the natural phenomenon itself. Only on the basis of profuse and reliable data is it possible to develop theoretical methods of effectively reducing the immeasurable losses of future earthquakes.

Engineers have tried to avoid this extensive and time-consuming field work by exposing structures to artificially produced shocks or by subjecting models to simulated strong vibrations on a shaking table. The value of such model tests is strongly disputed. The interpretation of the findings fails, in the main, because no exact rules can be formulated either for earthquake propagation in the soil or for the interaction of the structure with its subsoil.

The energy induced by an earthquake is considerable. According to an empirical relation by Gutenberg and Richter, the energy released by the earthquake in Friuli was about 30 million kWh. A nuclear power plant of the 300 MW class, for instance, would therefore require about four days to produce the same amount of energy as was released by the earthquake in a few seconds. Another comparison shows that an atomic

bomb with a blasting power of 1 megaton TNT would have a similar effect as the main earthquake on May 6th. All this demonstrates how difficult it is to ascertain experimentally the way an actual structure will behave during an earthquake.

It is no less difficult to come to conclusive results by means of model tests. The dynamic behaviour of a structure is influenced significantly by its many components which, in a static analysis, are taken into consideration only globally as loads.

Façades, dividing walls, piping, machinery and furniture can have a considerable influence. It is not at all clear, however, whether this influence is positive or negative. The same can be said of the interaction of buildings and their subsoil. On the other hand, it is known that mutual friction of the building elements and the elastic-plastic deformation of building materials are able to reduce the stresses drastically. However, it would be difficult, if not impossible, to realistically incorporate such facts in a model test.

PURPOSE OF THE FIELD TRIP

On May 6th, 1976, at 9 p.m., many people in Switzerland were aware of a prolonged trembling of their surroundings. Depending on their location, crockery rattled, water rippled, chairs rocked and lamps swayed. In some tall buildings, the movements were so strong that the frightened occupants fled outside.

It was learnt the next morning from the news media that the origin of the shocks, felt over wide areas of Europe, lay in the N.E. Italian

region of Friuli and that the area north of the town of Udine had been hit by a violent earthquake. Despite the fact that news of the scope of the damage could only sparcely filter through because of the interruption of road and telecommunications, it was soon realized from the large number of victims that an extensive natural catastrophe had taken place at the southern foot of the Alps.

For engineers daily confronted in their jobs with the structural problems of earthquake safety, the area, which had seen much construction activity over the past years, was expected to provide an abundance of illustrative material. To miss the rare opportunity of examining on the spot the ultimate carrying capacity of diverse types of structures all of which had been damaged or destroyed by the same occurence was unthinkable. The following were the aims of the expedition :

- critical examination of the earthquake characteristics together with their effects on the earth surface,
- inspection of typical structural damage and destruction, particularly ultimate carrying capacity.

It was out of the question to conduct a complete survey of the damage. The emphasis lay, therefore, on examination of the most important and typical examples of structural damage, and not on the description of the extent of the destruction.

The sojourn in the earthquake area lasted from Monday, May 31st unitl Friday, June 4th 1976. The interval of four weeks from the main earthquake was still deemed acceptable. Nevertheless, clearance and repair work was well under way over large areas with the advantage that one's freedom of movement was not greatly hampered and one could already observe the work of reconstruction.

A great deal of information and documentation was collected, including notes on conversations, sketches, several hundred photographs

and newspaper clippings, which, together with the participants' own impressions formed a richly documented, far reaching report for those authorities directly interested.

From the engineering office of Basler & Hofmann, E. Glauser, E. Heimgartner, A. Branger, H. Gugerli and B. Rast took part in the examination of the area. This group was joined by R.J. Hopkirk, Electro-Watt Engineering Inc. and H. Stauber, Motor-Columbus Consulting Engineers Inc.

FRIULI AS AN EARTHQUAKE AREA

A glance at a map showing the severe earthquakes which have occured in central Europe indicates that there are areas in Italy and in bordering Yugoslavia which are as a whole considerably more active than Friuli. However, Friuli is well known as an earthquake area where numerous severe seismic occurrences have been registered for over 800 years. The strongest earthquakes have mostly hit the northern part of Friuli but also, in the area damaged by the most recent earthquake, shocks with an intensity of VII (MSK) have previously been observed. On the other hand, the southern part of Friuli exhibits no significant seismic activity. Figur 1 comprises a list of the earthquakes with epicentral intensities of VII and more which are known today to have occured in Friuli (Ref. 3).

The strongest known earthquake in the region, that in Villach (January 25th, 1348), caused the destruction of over half the buildings in Gemona and, in Villach itself, over 1000 people were killed. During

Figure 1 List of earthquakes in the Friuli Region with an Intensity of VII (MSK) and higher (Period: 1000 - 1975).

Year	Intensity (MSK)	Epicentral Area * exact location unknown	Year	Intensity (MSK)	Epicentral Area * exact location unknown
1000	IX	Villach	1839	VII	Arta
1170	IX	Villach	1841	VII	Tolmezzo
1278	VIII	Cividale	1853	VII	Amaro (E Tolmezzo)
1279	VIII	Cividale	1857	VII	Rosegg (E Villach)
1348	X	Villach	1873	IX	Soverzene (N Belluno)
1389	VII	Resiutta	1889	VIII	Tolmezzo
1403	VIII	*	1889	VII	Tolmezzo
1451	VII	*	1892	VII	Claut
1455	VIII	*	1895	VII	Claut
1511	IX	Tricesimo	1896	VII	Pontebba
1514	VII	Gemona	1898	VII 1/2	Cividale
1529	VII	Udine	1908	VIII	Arta
1690	IX	Villach	1920	VII	Venzone
1700	IX	Enemonzo (W Tolmezzo)	1924	VII	Ampezzo
1750	VII	Pordenone	1928	IX	Verzegnis
1776	VIII	*	1931	VII	Gemona
1788	IX	Tolmezzo	1936	IX	Sacile (W Pordenone)
1789	VII	*	1959	VII	Sauris-Forni-Zuglio
1790	IX	Tolmezzo	1959	VIII 1/2	Arta
1794	IX	Tramonti	1960	VII	Carnia Alps
1794	VIII	Tramonti			
1812	VIII 1/2	Pordenone-Sequals			

the earthquake of March 26th, 1511, many buildings, castles and churches collapsed in the epicentral area of the latest earthquake. Since 1511, the strongest earthquakes in the region have occurred more to the north and west of this epicentral area, so that in Gemona, for example, no particularly severe damage (building collapse) has since been recorded. It can be reckoned, therefore, that the oldest buildings destroyed by the latest earthquake were 4 to 5 centuries old.

The measure of the activity of a region is characterized by the number of earthquakes occurring in it which have reached or exceeded a certain intensity, within a given period of time (cumulative frequency). In the frequency-intensity diagram the frequencies N ($I_o \geq I_o^*$) were applied for the various intensities I_o^*. Figure 2 shows the frequency-intensity diagram for the Friuli region. Accordingly, approximately 40 earthquakes with an intensity $I_o \geq$ VII have occurred within the period 1000 - 1975.

Gutenberg and Richter (Ref. 4) found a linear relationship between the logarithm of the frequency N and the intensity I_o in the known form :

$$\log N = a - b \cdot I_o$$

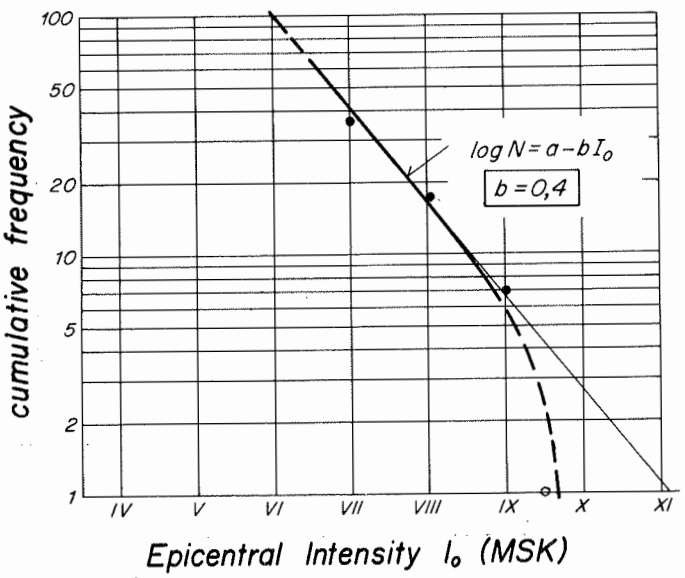

Figure 2
Cumulative earthquake frequency N as a function of Intensity I_o for the Friuli Region, excluding Villach
(Period: 1000 - 1975).

Whilst the parameter "a" depends on the time period observed and the area, the "b"-value, i.e. the slope, is an independent figure which becomes important when comparing seismic regions. A small "b"-value means a large ratio of severe, destructive earthquakes to relatively weak occurences - the warning effect of smaller earthquakes is lost. The value b = 0.4, shown in Figure 2, is smaller than the values in other areas of central Europa (0.5 - 0.7). In other words, it can be said that Friuli, as a seismic area, is somewhat insidious.

From the above data, average recurrence periods for a given intensity can be readily found. Figure 3 gives the curve for Friuli, relating return periods with epicentral intensities. Statistically, it can be concluded that an earthquake of the intensity IX1/2 (MSK) can be expected, on the average, every 250 years.

This is a simple rule indicating the overall activity. Its usability is limited since topical differences are not included. Only detailed risk analyses can encounter for different contributions to the total risk

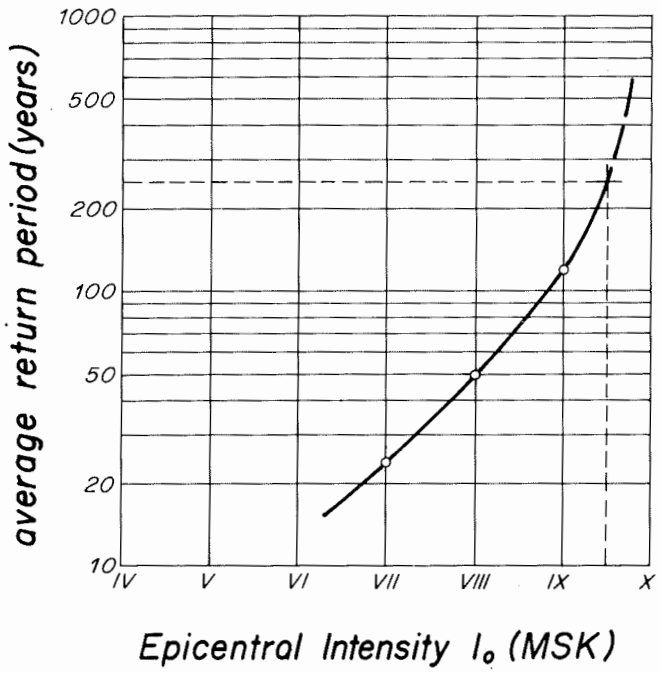

Figure 3
Average recurrence period for earthquakes in the Friuli Region (excluding Villach) with an Intensity $I_O \geq$ VI (MKS).

on a given location, stemming from background seismicity, influence from nearby sources and attenuated influence from distant sources.

The importance of comprehensive risk studies is recognized by many countries. In Switzerland too, an earthquake risk study is being made under the auspices of the Swiss Division for the Safety of Nuclear Installations. The results will be published in 1977.

PROPERTIES OF THE MAIN SHOCK

The accelograph nearest to the epicentrum belongs to the Italian State Electricity Company (ENEL) and is located in Tolmezzo (Lat. 46.38°/Long. 12.98°). It is stationed approximately 15 km from the epicentrum and lies in a zone with an earthquake intensity of I = VII - VIII (MKS). The apparatus is a strong motion recorder, type Teledyne RFT-250. The acceleration recordings begin on the film when a trigger acceleration of 0.01 g (g = acceleration of gravity 9.81 m/s^2) is reached. These recordings consist of three components - two horizontal and one vertical. Figure 4 shows clearly the phase of the strongest

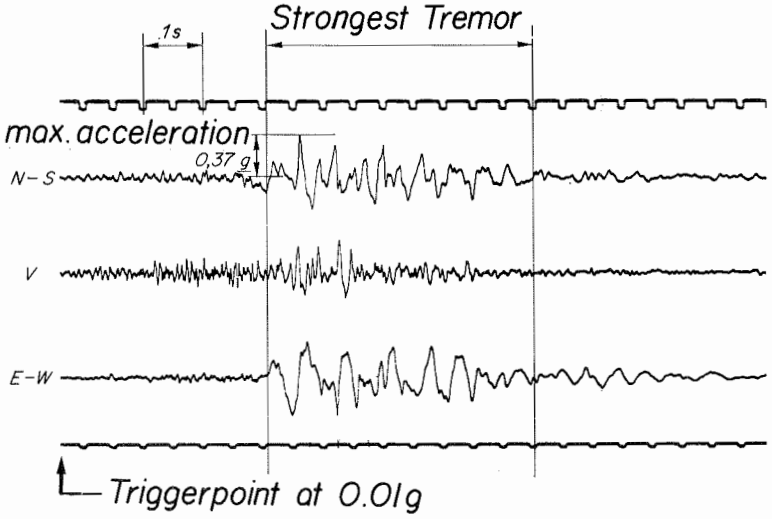

Figure 4
Strong motion record from a three-component acceleration recorder type Teledyne RFT-250 (Tolmezzo Station). Technical data: Eigen-frequency 22.5 Hz, critical damping 50 %, sensitivity on original film 1 cm ≙ 1 sec ≙ 0.52 g.

tremor which lasted about 4 1/2 seconds. The total duration of the main shock was approximately 40 seconds.

The Swiss Earthquake Service computed the strength of the earthquake at M = 6.5 (Richter scale). The beginning of the recording (vertical component only), as recorded by the station in Basel, is shown in Figure 5. Already here the distortion in time is clearly revealed. The various velocities of longitudinal, transversal and surface waves as well as the mixture of the direct and reflected waves caused a signal of several minutes in the Basel station located 450 km from the source. The recording essentially reproduced the pre-shocks. When the first stronger amplitude of the main shock set in, this highly sensitive seismograph could no longer record properly. This recorder is connected to a velocity pick-up. Here, as opposed to the above-mentionned accelerographs, single maximum amplitudes cannot be used by the civil engineer because of their frequency coupling.

Maximum ground acceleration has only a limited value for the design of structures. In most cases, a structure is destroyed not by a single shock to its foundation but by amplified shaking caused by repetitive input motions. Therefore a repetitive amplitude is more represent-

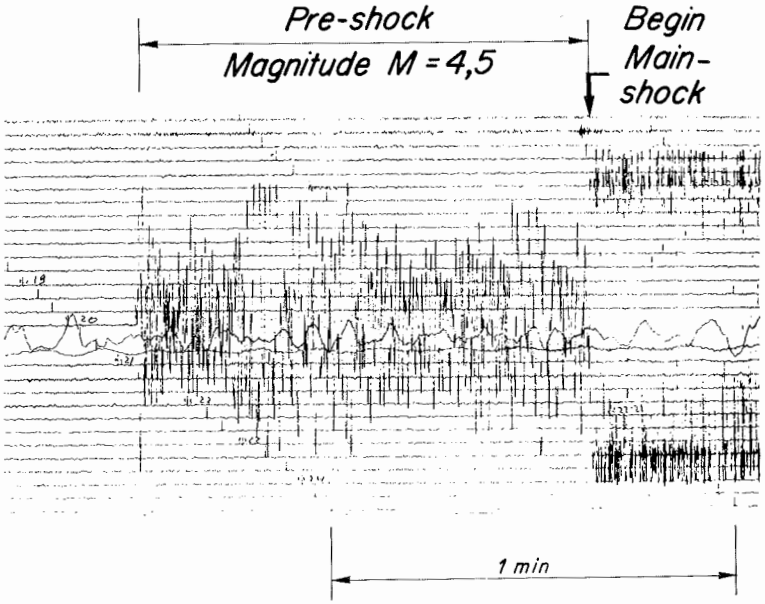

Figure 5
Recording made by the highly sensitive electrodynamic seismograph at the Basel station.

Properties of the Main Shock

ative as a design value. In addition, duration and frequency-content are important.

Digitized response spectra from the existing recordings are not yet available. From experience, it can be concluded that the maximum ground acceleration at the epicentrum was about 0.5 g. The representative design acceleration could then be put at about 0.35 g on bedrock. With this 0 period ground acceleration, the desired design spectra can be constructed. (Example: design spectra according to Wash 1255 - see Figure 6.)

The majority of the buildings in the Tagliamento Valley are founded on quaternary deposits. In these cases the desired design spectra have to be adapted accordingly. The magnification effects for modal accelerations, compared with those on rock, are small in the 30 - 6 Hz range, but very significant below 6 Hz.

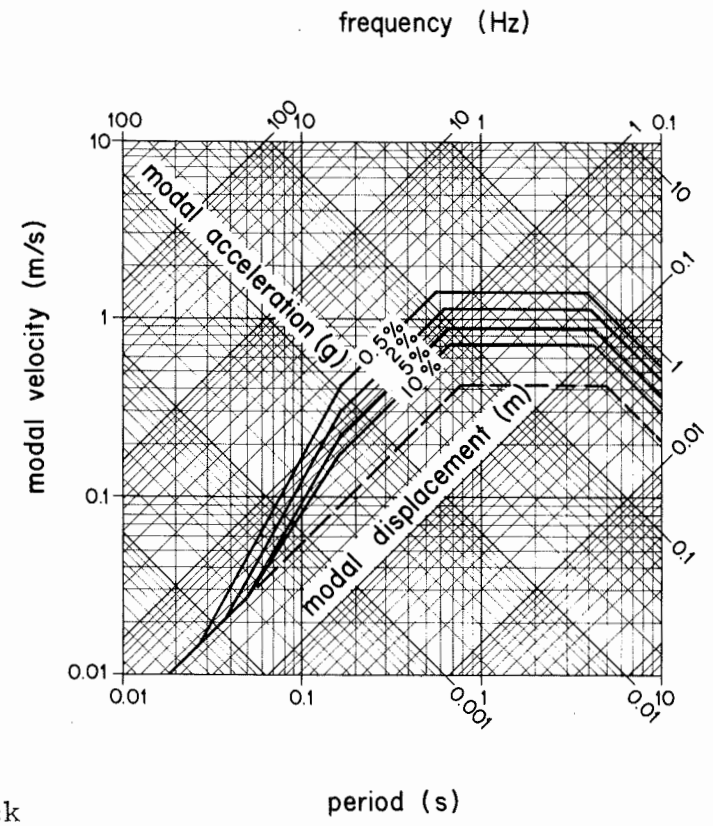

Figure 6
Representative Response Spectra for typical gravel deposits overlying bedrock in the Epicentral Area.

TYPICAL STRUCTURAL DAMAGE

The large number of structures with their various functions showed substantial differences in behaviour under earthquake motion. Through observation and examination of damage to many dissimilar structures, it became clear how damage and collapse of varying severity showed common characteristics, as seen from the following selected examples.

Brick and Masonry Structures

House with Workshop near Artegna. The three-storey building comprised a cellar in reinforced concrete, and a workshop with a living area above it in concrete blocks. One of the longitudinal walls of the workshop floor containted three doors, the other a row of windows. The uppermost floor, on the other hand, was stiffened by several dividing walls and contained relatively few windows. The cellar ceiling was built in reinforced concrete, that of the ground and the roof as a hollow block slab.

Whilst the cellar and upper floor showed only slight damage, the workshop floor was more seriously destroyed. In the cross wall, in which the entrance was located, approximately 10 cm wide diagonal cracks opened in a typical X-pattern (Figure 7). The window and door jambs were badly damaged, whilst parts of the walls fell out or were crushed. The concrete block walls of the ground floor, weakened by the openings in them, were overstrained and unable to transfer the horizontal shear caused by earthquake excitation of the comparatively heavy upper floor to the rigid cellar floor.

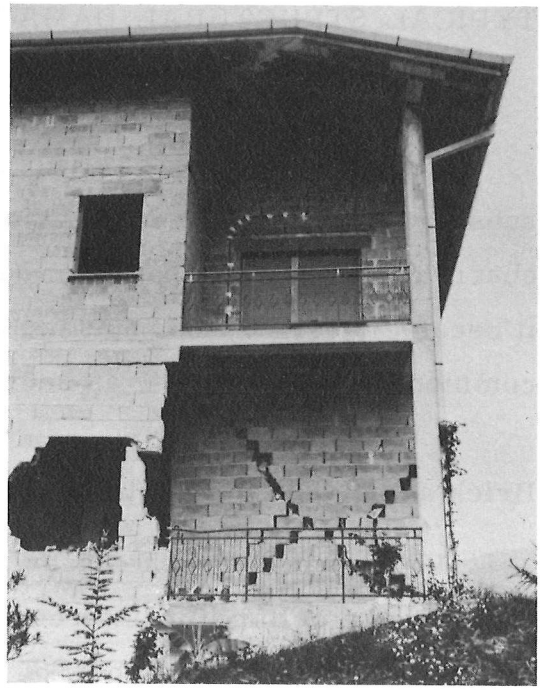

Figure 7
House with workshop near Artegna with cellar level in reinforced concrete and two storeys in masonry. Gaping X-shaped diagonal cracks and parts of wall broken on one front due to high shear force.

Hotel near Venzone. The five-storey (plus one cellar level) hotel building was erected some 20 years ago. The outer walls were built out of a mixture of brick and hewn stone. Due to the wide windows and doors only narrow areas of wall remained in the front wall of the ground floor. The front part of the building was founded on rock, the rear part on gravel. Also in the case of this multi-storey building, the



Figure 8
Five-storey hotel building near Venzone with walls of hewn stone and brick. Damage in the ground floor manifested by typical X-cracks between the windows of the front façade and a diagonal crack on the side wall due to heterogeneous foundation conditions.

upper floors remained pratically undamaged and the destruction was restricted mostly to the ground floor (Figure 8). The wall sections between the windows of the front façade were totally broken by the shear forces and one of the side fronts showed a diagonal crack. Apart from the weakening of the ground floor, all of this damage may be attributed to different foundation behaviour.

After long discussion between the owner and a consulting engineer, it was decided to demolish the building since there seemed to be no easy and economical way to repair the structure.

Typical Behaviour. Many brick and masonry buildings showed damage in the ground floor. The massive construction of the closed upper floors led to large horizontal shear forces which could not be resisted by the few walls in the ground floor area.

The most common damage were diagonal cracks, which, due to oscillating acceleration, always appeared in an X-pattern (Figure 7). These X-cracks appeared particularly often where horizontal - also, however, in isolated cases, vertical - rows of windows were located. Also horizontal cracks could be observed in the area of weak mortar pointing. Such damage was not only to be found on window jambs (Figure 12) but also whole upper floors shifted slightly. Diagonal cracks in only one direction can be traced to another source, namely the different degree of settling in non-homogeneous subsoil.

Reinforced Concrete Frame Structures

Living and Business Building near Artegna. The three-storey reinforced concrete frame building was still under construction. Since the ground floor was planned as a sales area, only the longitudinal rear wall was in brick. The front and side walls were reserved for large windows (Figure 9). The staircase, located in a corner, was enclosed



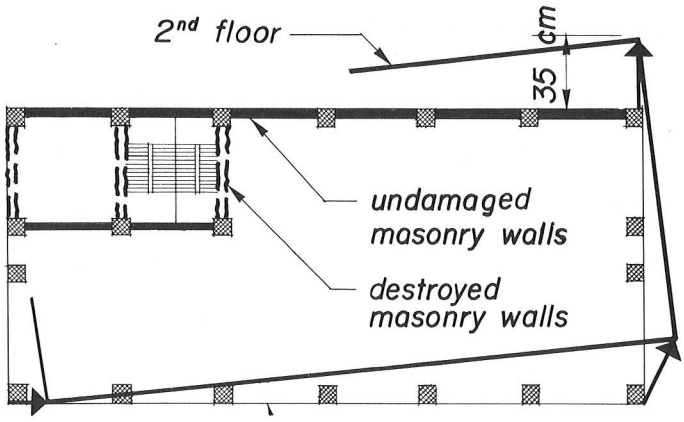

Figure 9
Living and business building near Artegna. Reinforced concrete frame building with brick partitioning walls in the upper storeys. In the open-fronted ground floor only the rear longitudinal wall and the staircase were braced. The rotationel movement of the building was caused by the stiffening effects of the staircase walls.

in brick walls. In contrast to the ground floor, both the upper living floors were stiffened by continuous brick walls. The foundation consisted of a single reinforced concrete slab.

The damage here was also limited entirely to the ground floor. The reinforced concrete columns tilted sharply (maximum deviation from the vertical position was up to 10 % of the height of the columns). At both ends of the columns, plastic hinges could be observed with cracked off concrete and buckled longitudinal reinforcement. The brick walls of the staircase, acting as a small core, were widely destroyed, the bracing of the longitudinal rear wall, however, remained intact.

The movements which occurred between the floor and ceiling of the ground floor were a result of the arrangement of the brick walls. The staircase became the centre of rotation, since the firm longitudinal

wall and the core, despite its considerable destruction, did not shift. In addition to the low translational resistance of the unbraced parts of the ground floor, the extremely excentric arrangement led to considerable torsional excitation, to which the construction could offer only insufficient resistance.

New Hospital in Tolmezzo. The new hospital in Tolmezzo was built during the period 1969-1974. It is a reinforced concrete frame building with brick filling. Since Tolmezzo was already classified in earthquake category 2 because of the 1928 earthquake, the building was designed in accordance with the earthquake regulations valid at the time. The building height was therefore limited to 7 storeys. In the seismic analysis conducted by the Centro Nazionale delle Ricerche (CNR) a ground acceleration of 0.10 g was taken as a basis. The various parts of the building were purposely constructed as separate units, thus allowing for free vibration of the individual segments. The 5 cm wide, continuous joints were filled with Styropor.

The supporting frame and the brick walls survived the earthquake undamaged. In many rooms, however, cracks appeared in the plaster, mostly along imbedded cables. The torn off covering along one of the joints running through all storeys was caused by large relative displacement, which again can be explained by the individual modes of vibration of adjacent building segments.

Despite the fact that the Prefecture ordered the evacuation of the hospital after the earthquake, the damage can be described as negligible. It will be comparatively inexpensive to restore the building. The building survived the earthquake with minor damage because of its distance from the epicentrum but, more importantly, because its supporting structure was designed to resist seismic forces. The even distribution of rigidity over the entire height of the building and the joints also played an important role.

Typical Behaviour. Much of the damage to the concrete frame structures may be attributed not to the design of the framework itself but to the placement of the brick walls, for which the dissimilar behaviour was not taken into consideration. The reinforced concrete frame can absorb large deformations both in the elastic and inelastic range. On the other hand, brick walls are both rigid and brittle and are therefore able to sustain only very moderate deformations without collapse. Every brick wall - both the outside and the otherwise unimportant inner walls, has a substantial influence on the behaviour of a building during an earthquake. Open ground floors with practically no shear walls and very minor shear resistance are therefore often the reason for collapse. Damage in the supporting framework springs mostly from insufficiently designed details, for example, the connection of beams and columns (Figure 10).

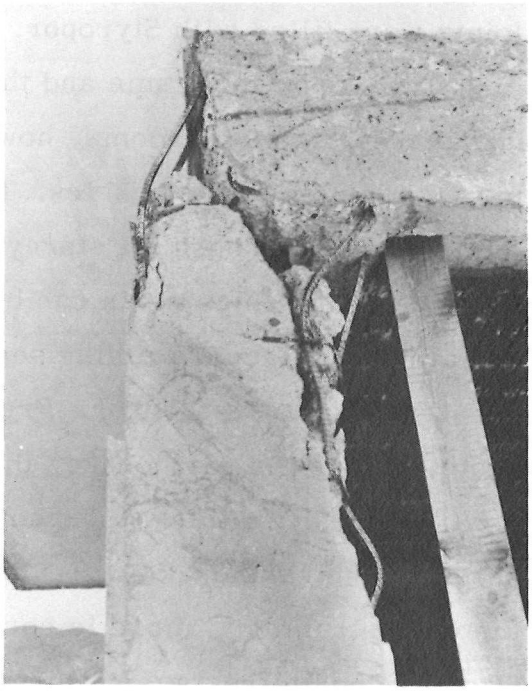

Figure 10
Shearing off of a column at its connection with the crossbeam due to insufficient stirrups.

Timber Storage Shed south of Magnano. The double-naved open shed (Figure 11) consisted of columns held together by longitudinal girders upon which short columns supported the roof trusses and concrete roof slabs. Tension rods carried the horizontal shear of the triple-jointed arched trusses.

In one of the naves, three roof sections collapsed. The tops of several truss supports were destroyed and showed cracks at their base. The cause of the damage lay in the generally very light design of the supporting structure which, with its tension rods and friction joints, was not suitable to carry abnormally large horizontal forces. In addition, the supports were insufficiently designed.



Figure 11
Open shed made of prefabricated reinforced concrete elements south of Magnano. Roof trusses as triple-jointed arches with tension rod borne on columns. Collapse of one roof section and damage to other segments due to the sliding of the trusses from their supports.

Fantoni Furniture Factory in Rivoli di Osoppo. The one-to-two storey shed complex was built in three parts and had been in operation for about a year. The same construction principle was applied throughout the framework, namely columns bearing longitudinal girders upon which the rafters rested. These elements were not connected but were

placed over one another with a relatively large support depth of 30 cm. The façade consisted of glass plates in metal frames.

The most severe damage to the framework occurred in one of the sheds were a large number of the roof beams fell from their supports. In other areas, cracked off concrete with buckled reinforcement rods could be observed on some of the columns. The glass fronts and the interior fittings were, to a large extent, damaged or destroyed.

Since the sliding of the rafters from their supports can only be explained by a considerable divergent motion of the opposite longitudinal girders and since no noticeable inelastic deformation of the columns could be found, the major deformation can be traced back, in the main, to displacements in the foundation. The damage to the columns near the insufficiently segmented enclosing wall demonstrated how secondary elements in the supporting framework can have unforeseen destructive effects.

The damage to the glass façades and the interior fittings, which also occurred in areas where the framework remained intact, can be attributed to the large deformation of the weak shed construction.

Warehouse near Gemona Railway Station. The reinforced concrete structure of the warehouse (Figure 12) consisted of individual frames with roof elements laid over them. The façade was built of prefabricated concrete elements, which were well fixed to the supporting framework and sealed with a plastic material. On one of the front walls, an annex made of brick and concrete blocks was built directly on to the warehouse.

The framework of the warehouse showed only insignificant damage and scattered, fine diagonal cracks were perceptible on the façade elements. It can be deduced, however, from the misshapen sealing material, that large shear deformation occurred between the elements

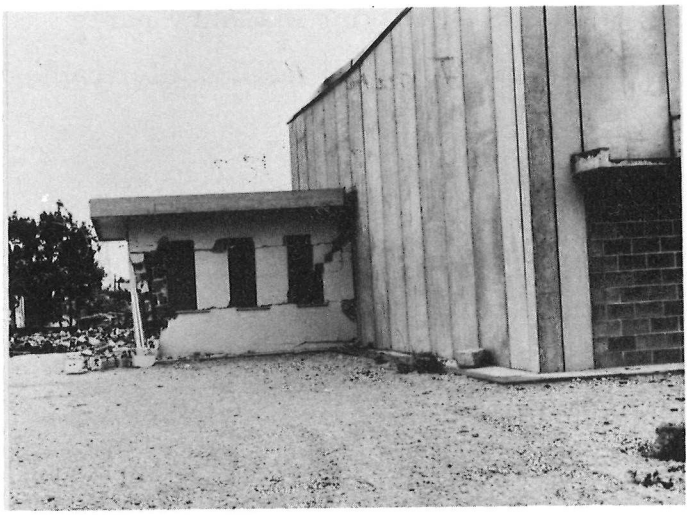

Figure 12
Prefabricated storage shed with brick annex in Gemona. Supporting framework of the shed undamaged. Side wall slightly caved in by impact with rigid annex. Roof of the annex shorn off and pushed over by the force of the impact due to insufficiently wide joints.

of the façades. Moreover, the façades were slightly caved in the area of the annex. The annex itself was shorn off around the window jambs and the roof shifted over about 20 cm.

Only unimportant damage occurred in this type of construction, as was confirmed by other shed structures and also by a single-family house. The main reason for this lay in the stiffening effect of the façade element and in the friction action of the joints. The destruction of the insufficiently detached annex can be traced back to the greatly differing modes of vibration of the two buildings. The impulse resulting from the impact pushed the roof of the annex several times further than the movement of the wall itself.

Typical Behaviour. In many cases, precast reinforced concrete frame buildings showed destruction which can be traced back to the connection of the individual elements. The numerous joints and supports produce frameworks which can offer only minimal resistance to horizontal loads and show very little possibility for plastic deformation. Because of the use of purely friction joints, the stability of the supporting framework was often not guaranteed. On the other hand, well-tied concrete elements for the façade contributed substantially to the satisfactory

behaviour of several structures. Smaller stabilizing masonry parts could not, to any large extent, impede the motion of the flexible framework and were often destroyed.

Stone Arch Bridges

Road Bridge over the Tagliamento River between Gemona and Trasaghis. The stone arch bridge spans the Tagliamento River in 17 spans of approximately 13 meters each, with massive pillars in the river bed and large abutments. The greatest damage to this long structure was a fissure in one of the abutment walls. Otherwise, the earthquake caused only scattered fine cracks.

Typical Behaviour. No large damage to stone arch bridges could be observed. This massive, unjointed type of structure was well able to withstand earthquake shocks. Occasionally, however, walls added to the abutment were destroyed as a result of the agitation.

Reinforced Concrete Bridges

Freeway Bridge over the Buia-Maiano Roadway. The two adjacent three-span bridges, with end spans of 10 meters and a center opening of about 15 meters, were extremely skew supported. The deck girder, conceived as a low-built hollow box, was longitudinally prestressed. The intermediate supports each consisted of a massive cross beam on four pillars. The bridge girder floated on the abutments and piers on Neoprene pads.

The bridge girder itself remained intact. It was, however, considerably twisted and displaced as a result of the skew supports (Figure 13). Consequently, the supports incurred extensive damage, whilst the bridge was partly displaced from the supports. Both abutments were pushed apart and badly damaged by the impact of the main girders.

Typical Structural Damage

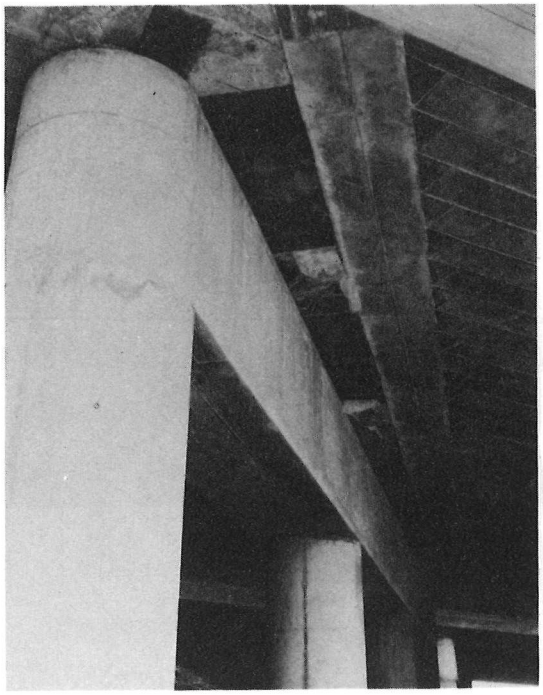

Figure 13
Three-span freeway bridge over the Buia-Maiano roadway. Bridge girders fallen from their Neoprene pads due to strong sideward displacement.

Freeway Bridge over the Tagliamento River near Osoppo. The freeway bridge over the Tagliamento River numbers 32 spans, each about 30 meters wide. The two separate bridges consist of simple prefabricated beams of prestressed concrete over which the precast deck slabs were laid. Two of the four longitudinal girders of each bridge were connected by two cross girders. The individual bridge sections were jointed at each pillar and glided on Teflon pads. The pillars themselves consist of large walls. The box-shaped abutments were closed off by means of high side walls.

Here again the earthquake damage concentrated on the supporting areas which were badly affected. The large longitudinal displacement in the individual bridge sections led to cracking of the concrete on the longitudinal girders and destroyed supports and impact brackets (Figure 14). The high abutment walls were badly lacerated by the impact of the end girders.



Figure 14
Freeway bridge with 32 spans over the Tagliamento River. Prefabricated girders borne on massive pillar walls. Cracked off concrete on the girders and completely destroyed supporting brackets due to the impact of the girder. Left bridge girder slipped from its bearing.

Typical Behaviour. Whilst the superstructure was generally only slightly damaged, many beam and slab bridges suffered extensive destruction at their support points. The heavy bridge girders were accelerated by the effects of the earthquake and struck against the abutment walls. Because of their oblique angle, some bridges were twisted due to the back and forth blows and pushed off their supports. Due to the enormous rebound forces the concrete of the bridge girders crushed, whilst the less stable abutments were pushed away and generally badly destroyed.

Tower Structures

Italian State Railway Water Tower in Gemona Railway Station.
The mushroom-shaped structure consisted of a hollow, cylindrical shaft supporting a water reservoir with a capacity of approximately 75 m^3. The bottom and walls of the 10 m diameter storage tank were constructed of reinforced concrete, the roof of a hollow block slab. The shaft had an outer diameter of 2 m with 20 cm thick walls. Orthogonal reinforcement was used both inside and out. A small entry was to be found at the foot of the shaft.

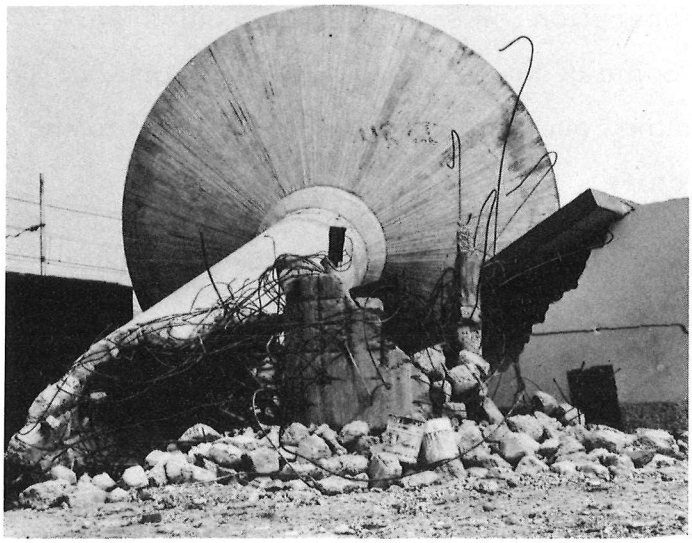

Figure 15
Overturned water tower belonging to the Italian State Railway in Gemona station. Foot of the shaft completely destroyed over a length of 2 - 3 meters and concrete shattered.

The overturned water tower afforded one of the most spectacular cases of earthquake damage (Figure 15). The top-heavy mass of the reservoir caused a huge bending force at the foot of the shaft which led to the destruction of the concrete and the collapse of the entire construction. The weakness created by the opening at the foot of the shaft must have accelerated the collapse.

Canals

Power Station Canal ENEL near Trasaghis. The canal served as an outlet for the Lago di Cavazzo, a storage basin of the Somplago hydro-electric power station, and consisted of a closed and an open free surface channel. The damaged open section of the canal had a trapezoidal cross-section and was made of non-reinforced 15 cm concrete slabs, jointed every 3 x 3 meters. The bottom consisted of one slab, both sides of two slabs each. A considerable number of the slabs broke, mostly horizontally in the middle of the slabs. The uppermost row of slabs was generally more affected than the others. The slabs around the abutments of overpasses were completely ruined.

Due to considerable leakage from the outlet structure, which was also destroyed, the canal could no longer be used, putting the undamaged

Somplago power station out of operation for some time. Repairs were therefore quickly undertaken. Some of the broken slabs were torn out and replaced. Concrete was poured over the remainder and fixed to the damaged slabs by shear connectors.

INTERPRETATION OF STRUCTURAL DAMAGE

Closer examination of the effects of the earthquake on various types of structures reveals that much of the damage originates from just a few basic structural defects. In this chapter, therefore, several ground rules are laid down for the design of structures. Although these conclusions can also largely be confirmed by observation of other earthquakes, the damage described here is peculiar to this particular earthquake and epicentral region.

The main cause of the extensive damage is certainly the fact that the effects of an earthquake did not have to and therefore were not taken into consideration in the design of the buildings. If engineers had only visualized that their structures would have to undergo the displacement, velocity and acceleration of earthquakes, the few selfevident consequences in the layout of the structures would have avoided most of the damage even without a proper earthquake design.

Structural Elements

Walls. Walls and partitions consisting of brick and masonry are both rigid and brittle. Where the walls are insufficiently strong or where many openings exist, the masonry walls are no longer able to

absorb horizontal forces. When overstrained, these walls crack mostly crosswise under 45°, the cracks spreading, either along the mortar joints or in the bricks. Because of the brittle nature of masonry constructions, the cracks widen, joints gape open or the walls concerned even collapse. In a skeleton construction the reinforced concrete framework can, in certain cases, continue to uphold the weight of the building. For purely masonry constructions, on the other hand, at least a partial collapse of the building is unavoidable.

Reinforced Concrete Columns. Since reinforced concrete columns are, in general, considerably more flexible than walls, the latter carry practically the entire earthquake force. However, in open constructions the entire stress is carried by the columns. The free-standing reinforced concrete columns of one-storey storage sheds were mostly strong enough to absorb the stress without being destroyed, in many cases even without incurring lasting cracks. By partial stiffening, for instance by means of an annex or installations or heavy, rigid upper floors, higher stress results, which usually leads to plastic deformation at both ends of the columns. Hence, the reinforcement, overstressed by tension, can buckle as a result of the alternating action. The related cracking of the concrete and the buckling cannot be significantly reduced even by means of closely spaced stirrups. The ability of the resulting plastic joint to rotate is, however, increased and a shear failure prevented.

In the event that it is impossible to calculate the columns, taking actual earthquake forces into consideration, then at least the plastic deformation of the columns in all directions must be guaranteed. The movements should not be hindered by any secondary elements. An improved building method for the prevention of collapse could, therefore, be to shape the columns that the plastic hinges, necessary to absorb energy, are formed in cross-beams.

Floor Slabs. Usually, floor slabs are designed with considerable safety factors for vertical loads. Therefore the vertical force of an earthquake can be absorbed with no substantial overstrain. On the other hand, floors present overly large concentrations of mass, which, in the event of an earthquake, produce horizontal inertia forces which must be borne by the upright structural elements.

Structural Systems

Open Ground Floors. Open or slightly stiffened ground floors, mostly for commercial use, are particularly vulnerable. The locally severe destruction in the area of such weak spots caused the collapse of entire buildings or made their demolition necessary, even with otherwise only minor damage, because restoration would have been to difficult.

Greatly differing conditions of rigidity in a supporting structure result in local weak spots which are the first to be overstrained in an earthquake and plastically deformed. Hence the stronger parts of the building are no longer irreversibly deformed and energy absorption is limited to the weakest building parts. Consequently, an evenly distributed plastification of the entire structure is necessary to ensure that destruction remains within acceptable limits.

Torsional Action. Many, from the point of view of the structural system, symmetrical structures suffered damage due to the additional twisting motion of the building around its vertical axis. As a result of the superimposed movement, some parts of the building are relieved whilst others are considerably more deformed than they would be due to translational movements only. Torsional loads are caused by the unsymmetrical layout of the structural system and the walls but also by contingencies arising in the rigidity and execution of partitioning walls

and additional fittings. The consequences of torsional strain can only be met by appropriate consideration in the design including provision for sufficient torsional rigidity of the building. Due to the incalculable influence of secondary elements, which are not designed to carry vertical loads, an asymmetry in the ground plan can hardly be excluded.

Attached Buildings. Severe damage could be located in structures composed of building sections with greatly differing rigidity due to diverse types of construction (for instance, reinforced concrete frame and pure brick) or which varied considerably in their design. This damage occurred because the individual deformation of each component was obstructed.

This problem can be overcome by so arranging the joints as to divide the structure into sections, each with its own clearly distinct vibration behaviour. The joints should be made adequately wide since numerous uncertainties make an exact calculation impossible. It must be taken into consideration, for example, that the deformation usually provided for in a homogeneous supporting structure can turn out to be considerably larger due to the formation of cracks or plastification. An adequate freedom of movement, therefore, allows for greater plastification and a larger capacity to carry earthquake stress.

Special Structures. Special structures (e.g. bridges and water towers), because of their unusual form and distribution of mass, necessitate a dynamic analysis which takes the vibration behaviour of the structure and the real properties of an anticipated earthquake into consideration.

Secondary Structural Elements. All the components and fixtures, in particular, dividing walls, attached façade slabs, coverings, pipes and other fittings, which form part of a structure, influence the

response of the supporting structure. These secondary structural elements are generally not included in the analysis of the supporting structure and therefore not designed against earthquake forces. They can, even when subjected to only slight movements, suffer damage which produces an increasing alteration in the vibration behaviour. It cannot be predicted whether this influence will prove to be positive due to greater absorption of energy or negative due, for example, to added torsional motion. As far as possible, in order to ensure that secondary elements survive earthquake loads without substantial damage, they should be analysed and designed together with the supporting system.

Joints and Supports

Jointing of Structural Elements. If prefabricated structures are designed only in accordance with the Standards laid down for earthquake forces or these forces are overlooked altogether, then the result is greatly underdimensioned connections of the structural elements. Load bearing connections should be properly designed against the expected dynamic forces. Purely friction-type joints are no longer sufficient to transfer the forces that arise, even from only weak earthquake loads.

Fixation of Secondary Structural Elements. Building parts (such as prefabricated façade slabs and dividing walls and fittings, particularly machines, storage racks and pipes), which are not part of the supporting structure, are usually either directly or indirectly connected to it. Due to the action of the earthquake, much damage occurred through the displacement or collapse of façade slabs which were unconnected or insufficiently secured. The actual displacement occurring at the fixation point, which can be considerably larger than those of the ground shock, must be taken into consideration in the fixation of secondary elements.

Interpretation of Structural Damage

Bridge Bearings. As opposed to the old stone arch bridges, a good number of newly erected reinforced concrete bridges incurred damage which was chiefly limited to the bearings and abutments. From the point of view of mass arrangement, bridge constructions come under the heading of special structures, since here the generally heavy girders in beam and slab bridges rest on comparatively high and therefore flexible piers. It is impossible to achieve with conventional methods a stabilization of the huge mass forces that occur when the structure is subjected to earthquake shocks. It should be noted that the reinforced concrete bridges hit by the earthquake in Friuli suffered severe damage which will be difficult to repair but they were to our knowledge by no means in danger of collapse.

MERITS AND LIMITATION OF EARTHQUAKE DESIGN SPECIFICATIONS

The region at the southern foot of the Alps hit by the May 6th earthquake has been known for centuries as an earthquake area. However, in the major part of the epicentral area, no laws existed for the design of structures. Such laws applied only for new buildings in a small part of the area.

The Italian State Administration has enacted special regulations for earthquake-prone areas and has repeatedly brought them up-to-date. Since May 8th, 1975, the following regulations have been in force for structures in earthquake-prone areas:

- Law No. 64 dated February 2, 1974

- Decreto ministeriale dated March 3, 1975 (Ref. 5)
- Norme tecniche per le costruzioni in zone sismiche (Ref. 6)
- Regulation No. 3797.

The classification of zones is included in the above. Most of the area hit by the latest earthquake lies outside the Friuli earthquake zones (Figure 16), which, in any case, are classified as being of low frequency and force. Only the northern part of the most recently damaged area lies within one of these zones.

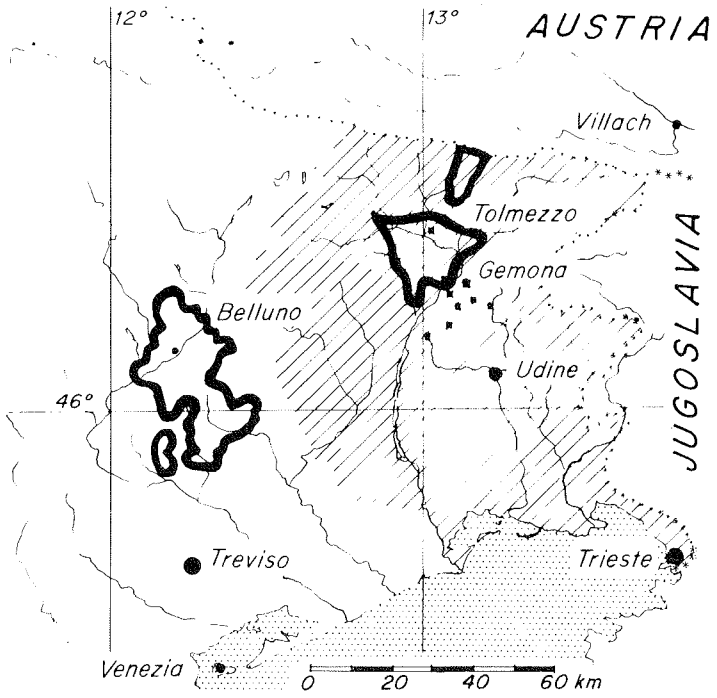

Figure 16
Earthquake zones in Friuli. The areas outlined in black indicate those regions in which the structures must conform to earthquake codes.

Using the Code in the above-mentioned zones, design earthquake loads are determinded by statical or dynamic analysis. An average horizontal acceleration will result which is about 7 % of the gravity acceleration g. In addition, the kinematic behaviour of the type of structure and foundation as well as a series of special structural rules are included.

Merits and Limitation of Earthquake Design Specifications

Comparison of horizontal design accelerations given by the Code with those produced by an earthquake with the Intensity IX (Figure 17) shows large discrepancies. The code values are significantly smaller because it is assumed that strong energy absorption will occur due to inelastic behaviour of materials and elements. But this means that the structure must be capable of absorbing the appropriate energy. Consequently, plastic deformation and therefore damage or maybe even collapse can result.

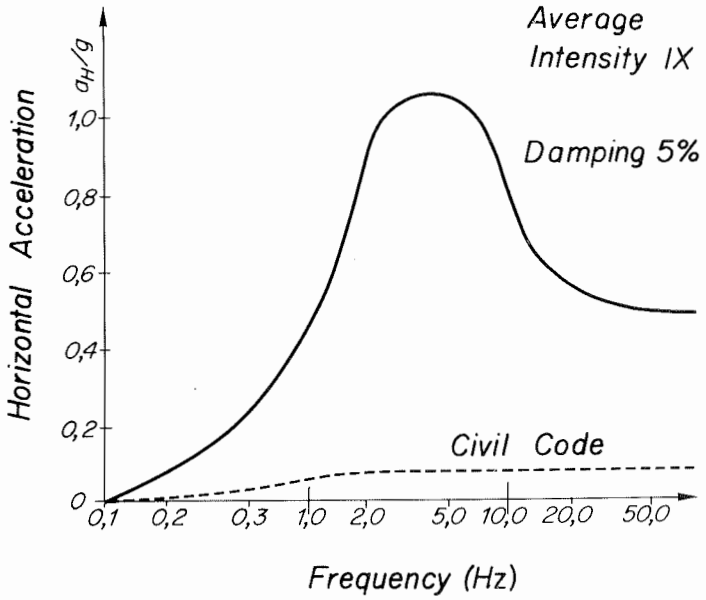

Figure 17
The horizontal acceleration curve of earthquakes with an intensity of IX (MM) according to Ref. 7. The dotted line shows the typical design curve used in conventional earthquake codes (Ref. 5).

On present day standards this is not good enough. Originally, Codes were drawn up merely to prevent the collapse of a structure and thus save lives. Today, our more highly developed society demands that at least life-lines (i.e. hospitals, water supplies, electricity, etc.) continue to function after an earthquake. As a result, it is imperative that a Code be introduced incorporating design specifications which distinguish between the various functions for which structures are built.

ACKNOWLEDGEMENTS

The examination of the building damage in Friuli took place under the auspices of the Division for the Safety of Nuclear Plants and with the support of the Kernkraftwerke Leibstadt AG, the Kernkraftwerke Kaiseraugst AG, the Bernischen Kraftwerke AG and the Energie de l'Ouest-Suisse S.A. The work in the earthquake area was furthered by a survey of the extent of the damage supplied by Signore Jannece of the Regional Building Commission in Udine. The earthquake accelerographs were supplied by Dr. Giuliani of the Comitato Nazionale per l'Energia Nucleare in Rome. The Swiss Earthquake Service of the Geophysical Institute of the ETH Zurich contributed to the seismological evaluation of the earthquake.

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BEHAVIOUR OF DAMAGED BUILDINGS UNDER STRONG EARTHQUAKES

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ABSTRACT

The region of Friuli in northeastern Italy was, after the May 6th earthquake, hit again on September 11th and 15th 1976, by an equally strong tremor with the intensity IX (MSK).

Observations of the effects by the September serie of earthquakes on already damaged buildings - repaired and still unrepaired ones - revealed the characteristics of structural behaviour under repeated large deformations.

Previously damaged and unrepaired structures were further damaged, many to the point of total collapse. Previously damaged and repaired structures resisted the second strong earthquakes only in cases where repair work included true strengthening of the structure, as opposed to mere patchwork or rebuilding of broken partitioning walls, in which cases further damages resulted also. Typical cases are presented.

INTRODUCTION

On September 11, 1976, after a relatively long period of calm, Friuli was hit by another strong earthquake with an epicentral intensity VII (MSK). This earthquake was exceeded, on September 15, by two further shocks with a force almost equal to that of May 6. The first shock with intensity VIII occurred at 5.00 a.m., the second one with intensity IX (MSK) at 11.00 a.m.

During the second stay in the earthquake-ravaged area of Friuli from September 24 - 26, the following aspects were examined:

1. The process of damage expansion on previously damaged structures (see Ref. 1, 2).
2. Effects of various repair measures on the behaviour of buildings during renewed earthquakes.

In order to distinguish the damage and destruction caused by the two series of earthquakes of May and September from each other, only those more modern structures were examined which had been closely investigated during the first trip. The medieval buildings in the villages and towns were mostly so badly destroyed after the May earthquake that examination of further damage was pointless.

Only a small part of the structural damage studied can be attributed with certainty to any one of the large number of shocks. Already during the first field trip, building damage was analysed which, apart from the first strong shock, had been subjected to a series of aftershocks with intensities up to I_0 = VII (MSK). Earthquakes of this force can cause noticeable damage to intact structures and can cause previously damaged buildings and building elements weakened by their brittle material behaviour to collapse. After the strong earthquakes of September 11 and 15, the investigations were even further handicapped by this uncertainty.

STRUCTURAL BEHAVIOUR DURING FURTHER STRONG MOTION EARTHQUAKES

A superficial inspection of the damaged area gives the impression that all the buildings left standing after the first earthquake had been destroyed by the strong shocks in September. In particular, many older structures, located mostly in centers, collapsed. On the other hand, in comparison, only few people were killed since the majority of the population was living in tents and mobile homes.

Closer examination of the effects of the earthquake disclosed the following distinct differences in behaviour :

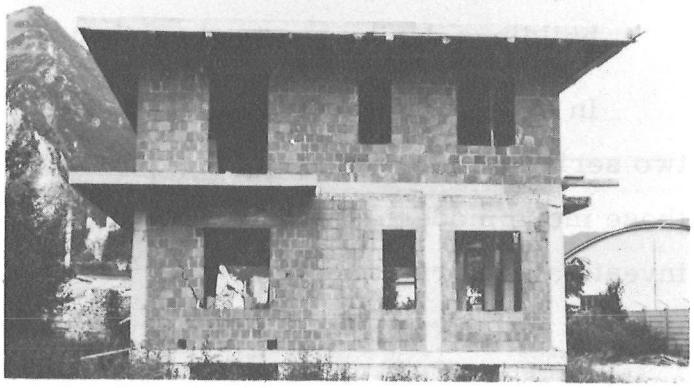

Figure 1
(June 1, 1976)
Uncompleted building in Gemona with partially open ground floor. The almost undamaged upper floor had rotated around the slightly cracked brick transverse wall in the ground-floor.

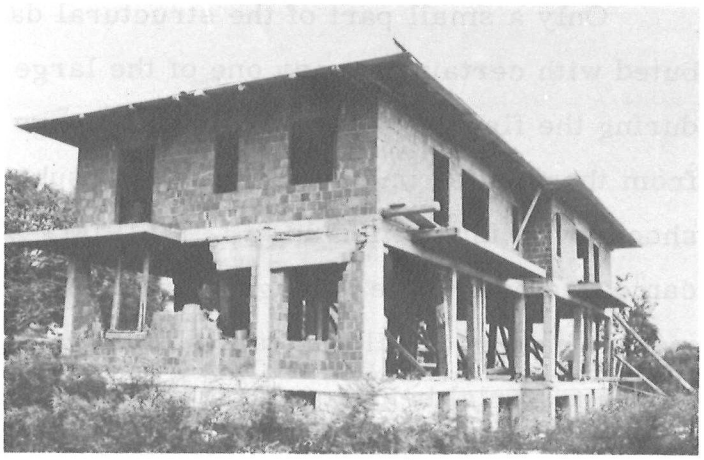

Figure 2
(Sept. 25, 1976)
View of the transverse wall destroyed by the September earthquakes. The massive wooden struts in the ground floor have saved the building from collapse.

- Buildings which showed no or only minimal surface cracks after the first strong earthquake, suffered no further damage or at most single masonry cracks. The buildings are in no danger of collapse.
- Buildings in which the brick walls were already cracked, suffered, in many instances, greater masonry damage. In the case of skeleton structures, damage to the supporting frame could result (Figures 1 and 2).
- Buildings with previous severe masonry destruction or damage to the supporting structure were so weakened that they suffered further deformation which often led to local collapse or even the collapse of the entire building (Figures 3 and 4).



Figure 3
(June 1, 1976)
Living and business building in Artegna. The two undamaged upper floors have rotated around the staircase core in the open ground floor.

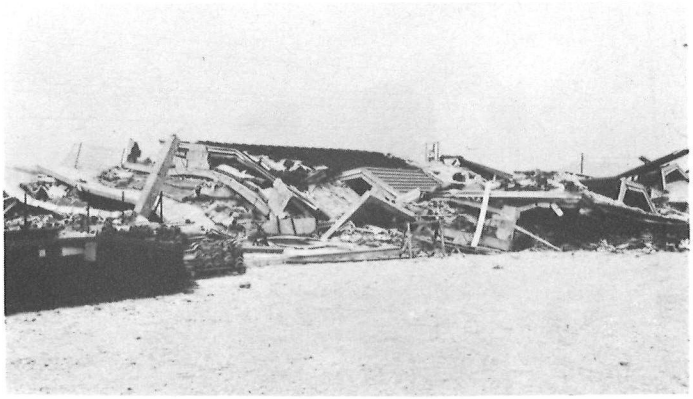

Figure 4
(Sept. 25, 1976)
View of the building which collapsed during the September 11 earthquake.

BEHAVIOUR OF REPAIRED BUILDINGS

Many buildings, slightly damaged by the first earthquake, had, in the meantime, been repaired. Priority was given to those buildings urgently required to keep life-lines open, such as the hospital in Tolmezzo, and to keep industry and trade moving. The reconstruction work was interrupted by the September earthquakes, thus giving the opportunity of judging the merits of the various repair work and to plan further reconstruction from the point of view of a threat of further strong earthquakes.

The building repairs inspected by us can be grouped as follows :



Figure 5
(June 1, 1976)
Living and business building near Bueris. Typical damage due to a reinforced concrete skeleton construction with brick dividing walls.

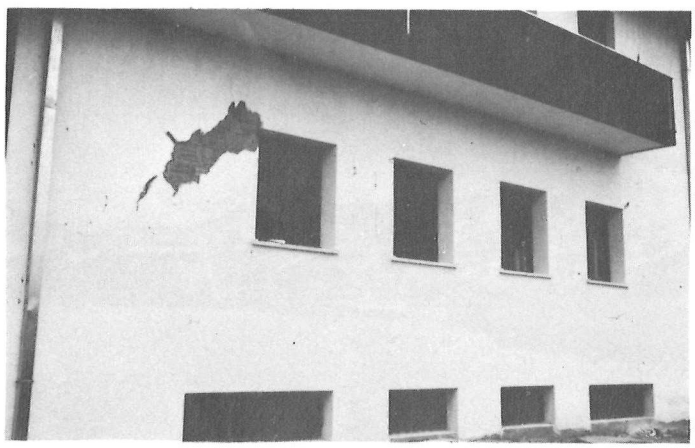

Figure 6
(Sept. 25, 1976)
View of the repaired building. Because the repairs did not include strengthening measures, the same kind of damage occurred during the new strong earthquakes.

Surface repairs: Façade and plaster cracks and even minor masonry cracks were superficially repaired. Under renewed earthquake strain the same kind of damage occurred, sometimes to a larger extent, in the areas of non-repaired masonry cracks (Figures 5 and 6).

Restoration: Where damage was limited to only a part of the supporting structure, that part was restored as far as possible to its original condition (Figure 7). It was not intended to strengthen the supporting frame. Naturally the same building parts were damaged in the same way as before. In addition, it should be noted that a repair can hardly restore the original carrying capacity and in most cases a weakness remains.

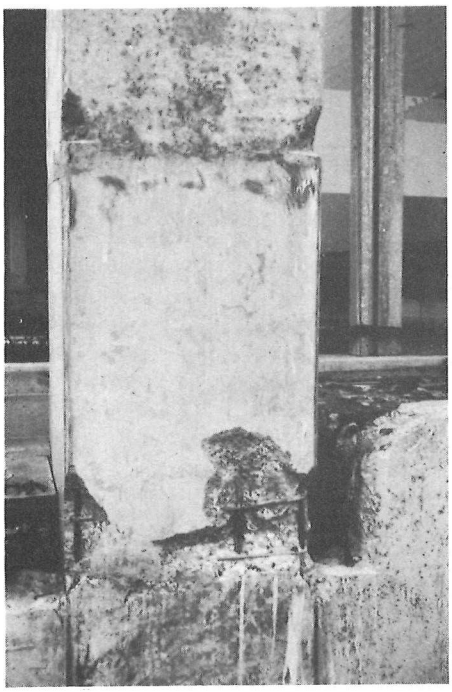

Figure 7
(Sept. 25, 1976)
Repaired column-foot of the Fantoni trade hall in Osoppo. The new earthquake load caused the same damage in the repaired areas as did the first.

Strengthening: In a few cases, it was hoped to achieve an improved carrying capacity by strengthening the supporting structure. For this purpose, for example, the cross-section of reinforced concrete columns were enlarged or the weak areas in brick walls were eliminated

by extensively bricking up the windows and doors (Figure 8). As was to be expected, distinctly less damage occurred than during the first shock.

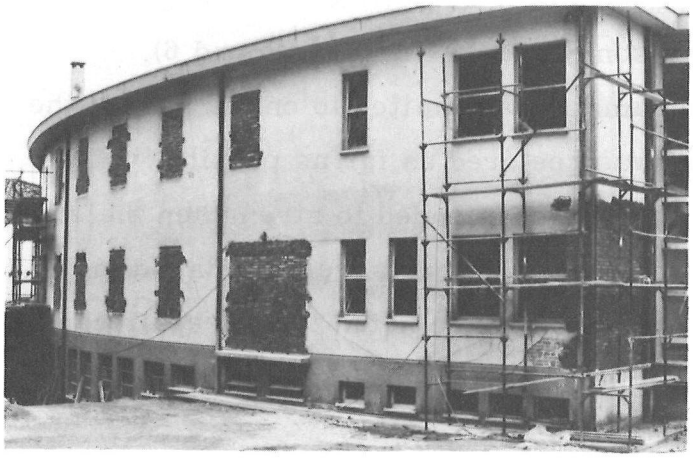

Figure 8
(Sept. 25, 1976)
Business building SBUELZ in Tricesimo. In order to strengthen the carrying capacity during earthquake load, the openings in the rear wall were bricked up.

Reducing of affected masses: The load acting on the supporting structure of a building is influenced not only by its vibration behaviour but also by the mass of the building parts affixed to it. A reduction in the weight of the building elements and fixtures will proportionally reduce the cross-sectional forces. For instance, heavy reinforced concrete façade slabs can be replaced by lighter steel façades. In the repair of a factory in Artegna (Figure 9), a reduction in load (steel façades instead of concrete elements) and a strengthening of the columns was undertaken. After the new strong earthquakes no damage could be seen which compared with that after the first earthquake.

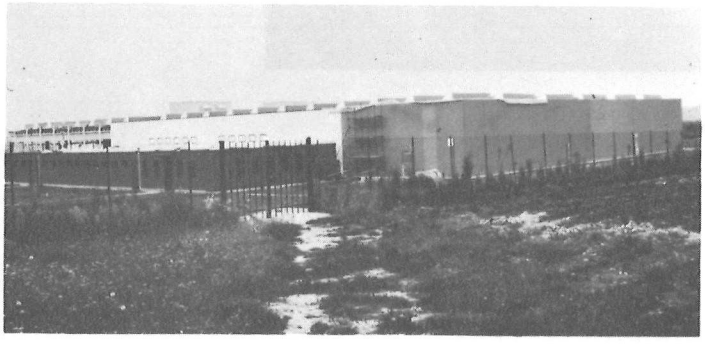

Figure 9
(Sept. 25, 1976)
SOLARI watch factory in Artegna. The heavy concrete façade elements were replaced by light steel façades in order to reduce the weight.

EFFECTS OF THE EARTHQUAKES OF SEPTEMBER 11 AND 15 ON RECONSTRUCTION

The new strong earthquakes in the first half of September interrupted the reconstruction work and further planning was halted in order to reconsider whether the measures which had been taken were indeed appropriate. From the standpoint of the engineer not participating in the decisions concerning reconstruction, a few conclusions for civil design can be drawn. These are not influenced by financial and social considerations and do not, therefore, necessarily correspond to the measures actually taken.

The most important lessons to be learnt can be summarized as follows :

Many more buildings must be razed and built anew than was at first assumed. Damage to the supporting structure cannot be easily repaired and, in most cases, represents a weakening of the carrying capacity. In addition, an already badly damaged building remains a constant threat.

Repair work should be so carried out that the behaviour under earthquake load is obviously improved. At least the original carrying capacity must be aspired to. Therefore, for example, badly cracked walls must be rebuilt. Structural improvements can be achieved by local reinforcement and by reducing the affected masses through the use of light building materials, wherever possible, instead of heavy ones.

It is obvious that the civil engineer must incorporate earthquake load in the calculation and construction of buildings to be newly erected. As demonstrated by the positive behaviour of the newer buildings in the area of Tolmezzo, design in accordance with the now valid earthquake Code offers sufficient safety for the life of the occupants. At the same

time, it should be noted that it is accepted that building damage will occur and the protection of the inhabitants is guaranteed during a limited number of strong earthquakes only. If it is required that a building (e.g. hospitals, utilities) be able to continue functioning after an earthquake, then the normal earthquake building code is insufficient. A dynamic analysis of the building behaviour should be made and the structures should be designed in accordance with the loads actually occurring during an earthquake. An earthquake risk study can serve as a basis for the selection of the appropriate earthquake input parameters.

ACKNOWLEDGEMENTS

The examinations of the building damage in Friuli took place under the auspices of the Division for the Safety of Nuclear Plants and with the support of the Kernkraftwerke Leibstadt AG, the Kernkraftwerke Kaiseraugst AG, the Bernischen Kraftwerke AG and the Energie de l'Ouest-Suisse SA.

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Building Damage in Friuli Earthquake 1976
— Buildings in Osoppo —

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Abstract

Old masonry houses were severely damaged due to their heavy roofs and brittle masonry walls. Many new reinforced concrete (RC) buildings were also destroyed because they were in a district where aseismic building codes were inapplicable. Bearing RC walls were very effective, but hollow brick walls did not contribute much to resisting the earthquake force.

Introduction

Despite the fact that Italy is the first country where aseismic building codes have been applied, the damage district in Friuli Earthquake 1976 has not participated in the benefits of the regulations - it being one of the regions where the regulation were inapplicable.

Consequantly most of both old and new buildings were constructed with no regard for earthquakes. The Tolmezzo Earthquake of 1928 might have been a good warning; following that earthquake many bell-towers were reinforced by iron hooks, and some of the buildings and houses behaved well during the recent shocks.

Earthquake-fire did not spread, and prompt actions were taken by the Italian Government after the earthquake. Because of the fairly good ground, the damage to civil engineering structures such as roads, railways, bridges, dams, water supplies and sewers was not severe despite the high intensity of the earthquake. But the damage to buildings was very great.

Stones and bricks are easily obtainable, and massive materials can form comfortable spaces against severe weather, so heavy masonry houses of two or three storeys are very common in this area. Recently, hollow bricks have been widely used. They are good heat insulators, light and easily laid. They are often combined with RC frames, but are too weak to be used for bearing walls. RC buildings with plenty of RC bearing walls suffered little damage in the recent earthquake. Many precast concrete (PC) structures collapsed due to insufficient strength and joint-rigidity.

The characteristics of ground motion plus the particular topology seem to be closely related to the damage criteria and distribution. At the top and foot of hills and on slopes, the vibration might be stronger than that on a wide plane. Rigid masonry structures on firm ground were at a dynamical disadvantage. On the other hand, many long period structures such as bell-towers and campanile survived.

Osoppo is an old town located at the foot of a hill where, probably,

the old Tagliament river once flowed. The town has been spreading to the plane.

The old town was mostly destroyed, but a few houses and buildings suffered only slight damage. Consequently, this was selected as one of the towns where we would focus our damage investigations.

Intensity of the Earthquake

The intensity of the Earthquake in Osoppo was not measured directly, but might be IX or X on the MM scale. A simple RC structure near Dumo (Fig.1) shows that the maximum acceleration response was around 330gals. Its calculated natural period was about 0.85sec., and the acceleration response spectrum is estimated as shown in Fig.2. (The ground was assumed to be horizontally deposited sand and gravel layers. The shape of the response spectrum calculated from the Tolmmezzo Records, is also illustrated.

Building Damage in Osoppo

Most of the houses in old Osoppo were masonry structures with heavy roofs. As the epicentre was very close to the town, the effect of Vertical ground motion on the behaviour of the roofs was great. The detail of a roof is illustrated in Fig.3. Two kinds of tiles are supported by a kind of post truss or by a simple beam. In the case of the truss, the horizontal beam member (the lower cord) was broken by a bending moment or shearing force caused by the weight and additional vertical force produced of the quake. In some cases the king post was broken and the load bearing

capacity of the truss decreased. For simple-beam type roof-structures, damage was produced either by bending and shearing failure of the beams or by failure of the supporting walls. In any case, the damage to the roof structures thrust additional weight on the walls which were also subjected to horizontal earthquake force, thus the upper parts of the masonry walls were broken. Damage to 214 houses out of the 764 damaged houses investigated in Osoppo was caused by failure of the roof structure.

Typical failures of masonry walls were: shearing diagonal cracks, horizontal bending cracks and separation of wall-plates. These failures combined and caused the whole structure to collapse. Walls with small openings remained, and the concept of the "wall ratio — wall area/floor area" can be used for an estimate of the strength of the houses. Gable walls broke easily except where partition walls perpendicular to the gables existed. Corner walls fell down when shearing cracks running in two different directions. Therefore houses with wide openings in walls facing directions collapsed from the corner.

Old houses seemed to be built by the owners and their helpers who were neither building engineers nor skilled constructers. The buildings had first been built and only later connected to each other, and the structures had beocme complicated. Stresses were concentrated on jonted parts at random dynamic loading. Therefore houses in Osoppo which suffered only slight damage were:

A) Those with light roofs

B) Those with simple plans

C) Those with enough walls with Small Openings (large wall ratio)

Most of the new houses designed by architects and building engineers satisfy conditions A adn B and, in many cases, c also. Thus, they survived in the damaged town. The reason why the houses along the street were generally less damaged might be that the outerwalls perpendicular to the street had small openings and the corner walls thus did not fall down.

Damaged houses were almost equally distributed throughout the old city. No major differenses in damage distribution due to topological effects were observed, because the whole area of the old town was probably within the boundary area where the vibration was augmented by topology. The sites on the extended crack line of the ground (in N 30° direction) might have been a little more severly damaged.

Building Damage in Detail

Some of the damage to houses and buildings we investigated in Osoppo is described.

"A School Building"(No.1 on the map; Fig.4)

Two storeyed brick masonry building used for a school suffered only a little damage. The plan and elevations are shown in Fig.5 and 6. The roof chimney was broken, and the canopy above the entrance fell down. Cracks on the walls showed that the main ground motion was in the north direction. This structure remained standing because of its sufficient wall ratio (4%) and its simple plan. The roof and ceiling were typically heavy. (see Fig.6, 7 and 8)

An Apartment House near the Dumo (No.2)

A three storeyed house (plus ground floor) was slightly damaged, and had been repaired within one month after the earthquake. Two blocks were connected by a stair case where stresses were concentrated and window glasses were broken. The main vibration direction seemed to be N-S. If it had been E-W, then short thin columns at the ground floor might have collapsed. The joints between RC beams and brick walls were seperated by shearing force. (Fig. 9, 10 and 11)

A stone house in front of the school (No.7)

It was surprising that this house remained. It had been a two-storeys stone house, and the third one was added later with bricks. Though there were no openings in the east façade and only small ones in the north and south, the wall facing west has wide opening, and it might be disadvantageous to the lateral forces to NS direction. The partition walls were probably effective. (see Fig.12)

A small brick church (No.9)

A big stone hit the wall. (see Fig.13)

A brick house at a corner (No.12)

A shutter showed that the ground once extended, and vertical cracks were procuced at footing-beams of NS directions. (Fig.14)

A typical stone house (No.15)

It was often experienced that a house seemed to be slightly damaged from outside was actually damaged much. The partition block wall collapsed, and the truss scarcely supported the roof. (Fig.15)

Five-Storeys RC point House (No.51)

It behaved quite well against earthquake, as it had enough walls at each storey. Only a few finishing tile fell down from the outside walls. (Fig.16)

A factory for car-parts (No.75)

The old part of the building pushed the new part of PC structure to the south. No effective walls were built in NS direction. I-shaped (I-9x18x6x8) steel columns were bent and joints of PC beams and PC columns were rotated. (Fig.17 and 18)

Conclusive Remarks

Fig.4 shows the damage distribution of old Osoppo town. The most of the survived houses were simply shaped and independent house with light roofs. Brittle masonry houses were common, and there evidently appeared the difference of the strength of the structures. If they had been ductile, continuous differences from slight and severe damage shoud have been observed. Consequently, the distribution of slightly damaged houses was quite random, and it was difficult for people to escape from collapsing houses. RC structure with walls behaved well[1], and necessity of RC walls

were strongly felt in all over the damaged area. The short period wave components were predominant in the ground motion, it was disadvantage for rogid masonry structures. But a bell tower at the foot of the hill was also damaged and later artificially destroyed.

Acknowledgement

We are very much abliged to Italian people for their cooperation during our investigation. We thank very much Prof.G.Grandori, Dr.L.Corbo, Eng.Colcerasa, Miss E.Dellapietra, Miss M.Piva and other people, who kindly share their time for us.

1) RC walls are about three times stronger than RC columns of an equal sectional area.
(T.Shiga "Wall ratio of RC Buildings and their aseismic capability" AIJ, Tohoku Branch, Nov., 1976)

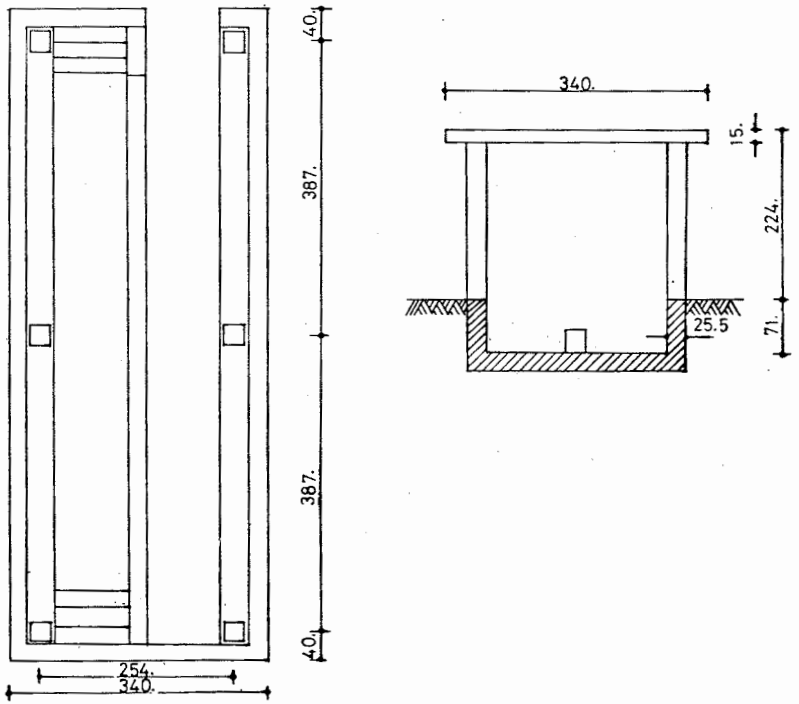

Fig.1 Simple RC Structure in Osoppo

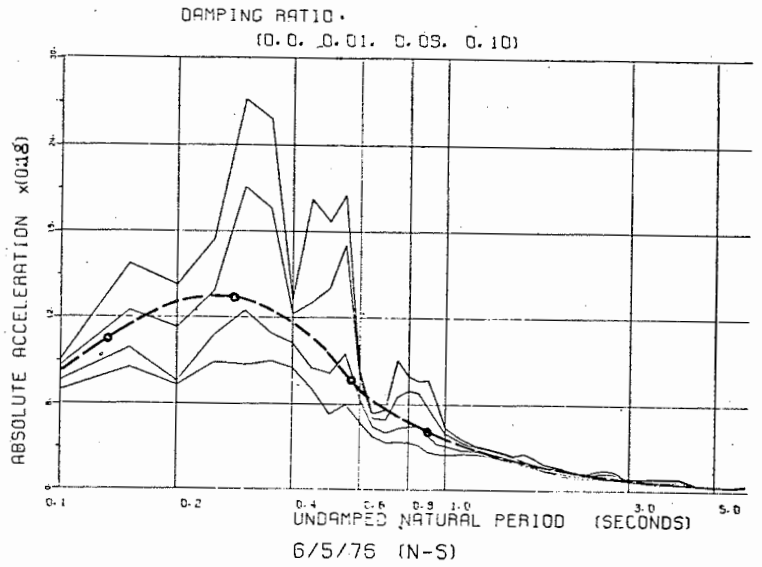

Fig.2 Response Spectrum Estimated from the Structural Damage(h=0.03)

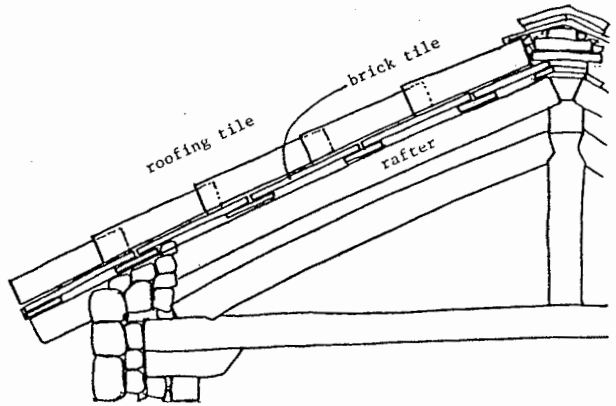

Fig.3 Details of Roof

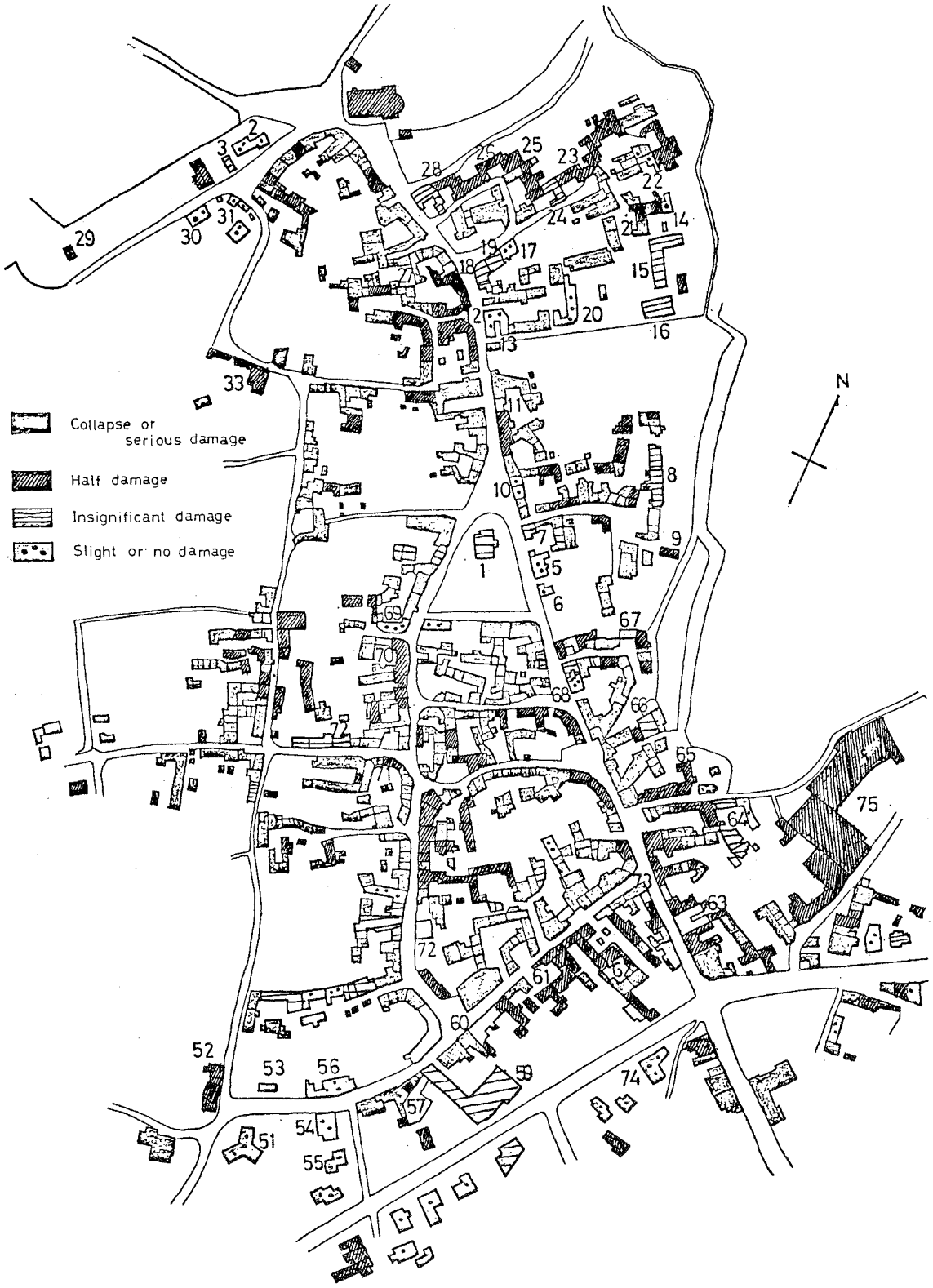

Fig.4 Damage Distribution of the Old Part of Osoppo

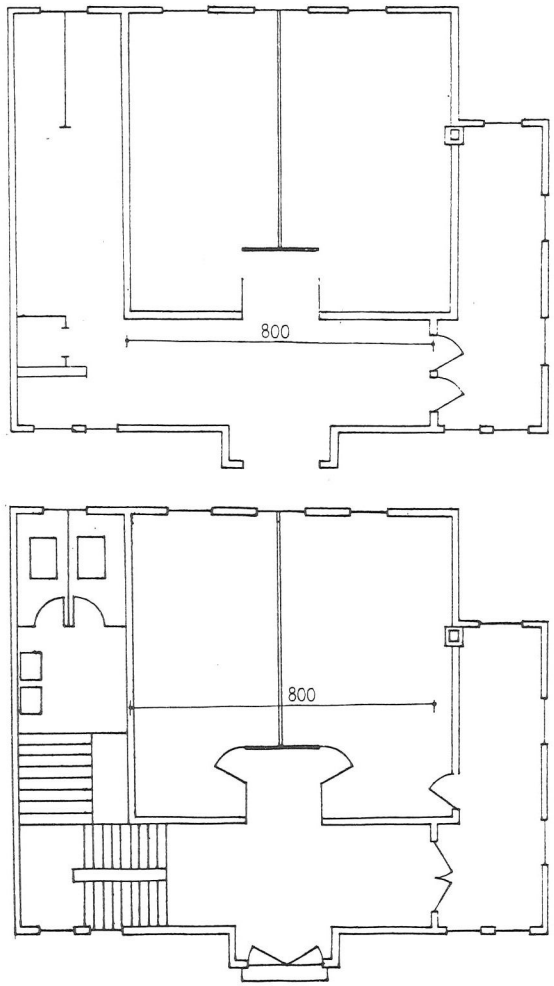

Fig.5 Plan of a School Building(No.1)

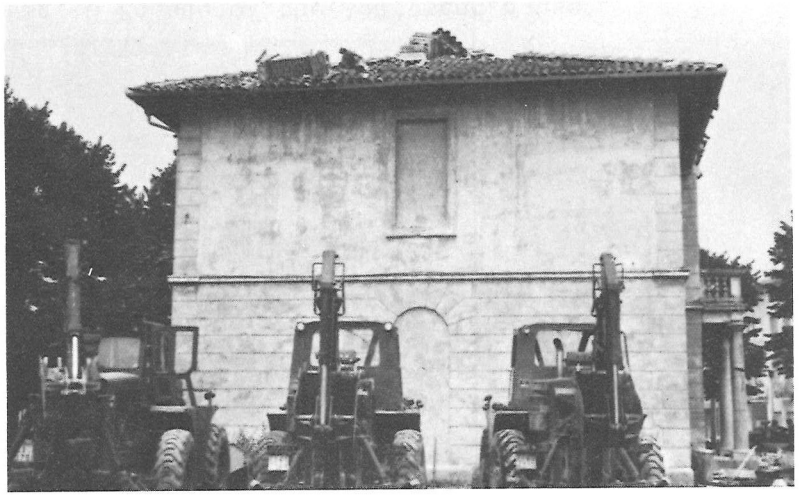

Fig.6 Façade of the School Building



Fig.7 Crack of a Wall Facing West

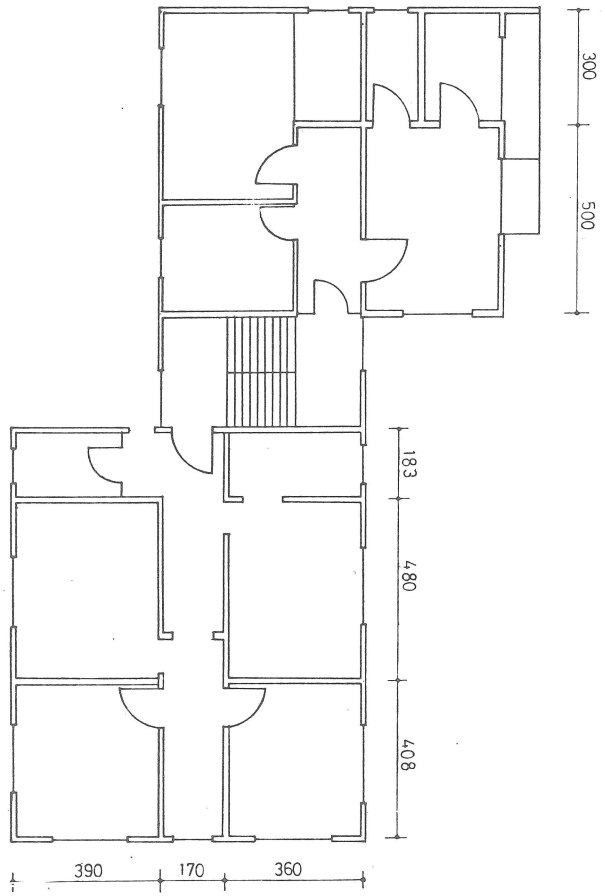

Fig.9 Plan of an Apartment House(No.2)



Fig.8 Capony Fallen Down



Fig.10 Façade of the Apartment House

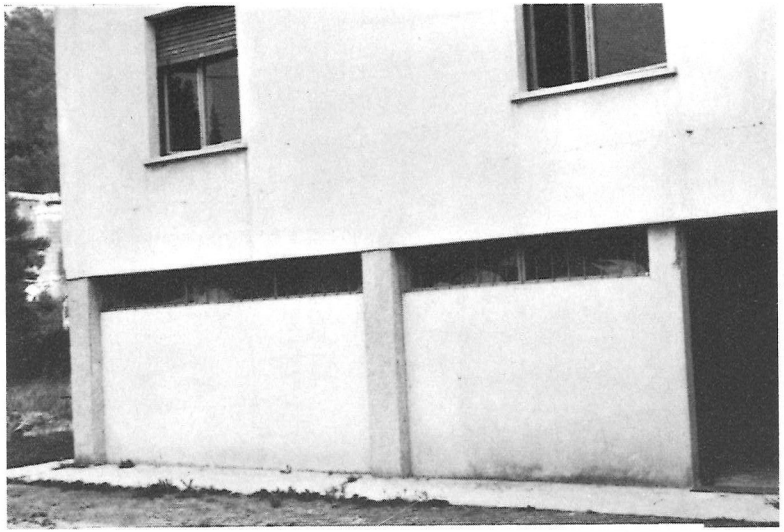

Fig.11 Short and Thin Columns at the Ground Floor



Fig.12 Facade of a Stone House



Fig.13 A Stone Hit a Small Brick Church

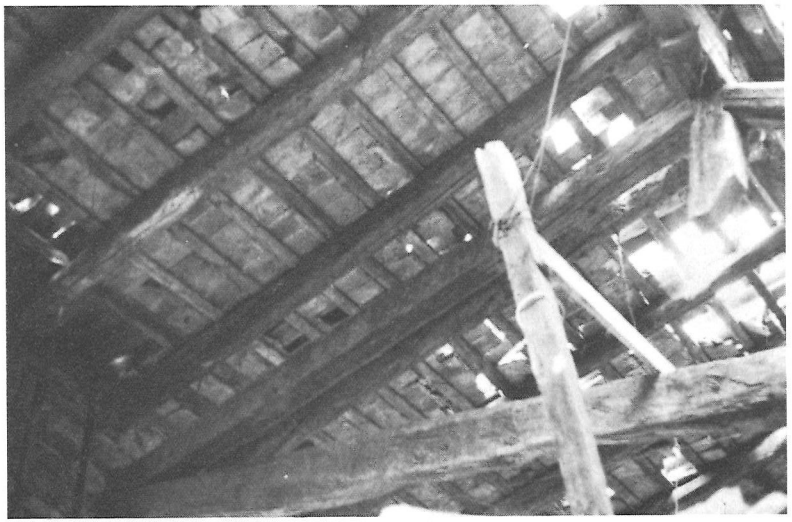

Fig.15 A Roof Structure(No.15)

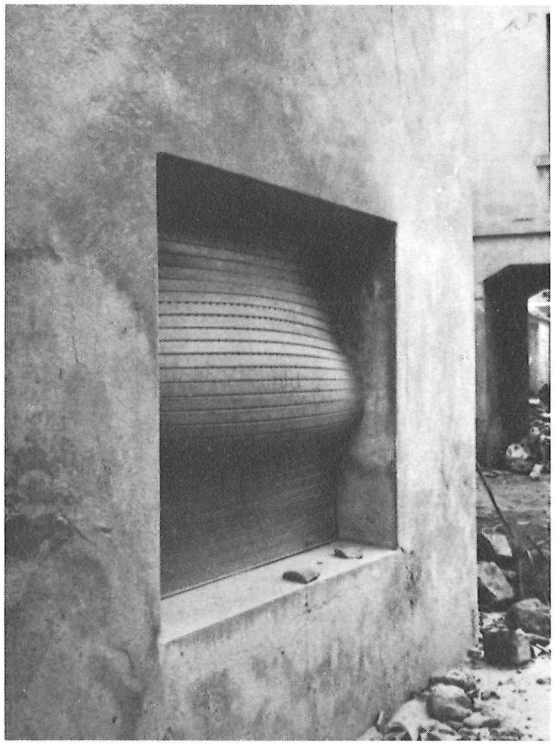

Fig.14 A Brick House(No.12)



Fig.16 A RC Point House(No.51)

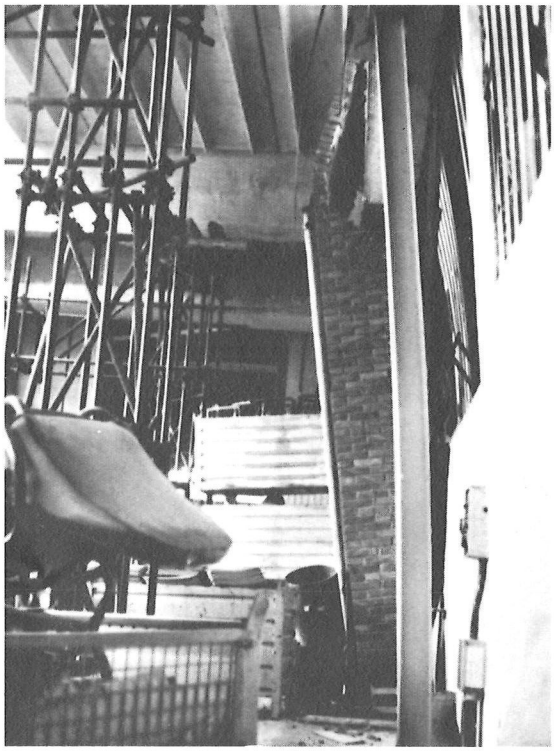

Fig.17 A Factory for Car-Parts(No.75)

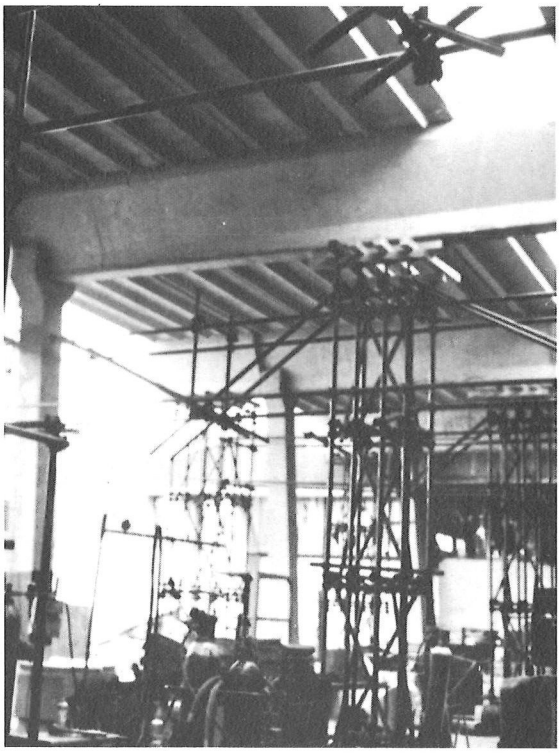

Fig.18 The Factory for Car-Parts(No.75)

ON THE INELASTIC BEHAVIOR OF STRUCTURES DURING EARTHQUAKES AS EXEMPLIFIED BY THE FRIULI CATASTROPHE 1976

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I

The dramatic consequences of the Friuli earthquake are well known. The human and material losses were extremely heavy. It is clear that everything should be undertaken to avoid similar tragedies in the future.

However, having been offered the tragic possibility of observing the earthquake-induced phenomena, it is our duty to carefully collect all the accessible data, to critically examine and analyze their character and, finally, to draw the necessary conclusions so as to be able to design in the future our structures in a manner to safely withstand all the disastrous effects with a reasonable probability.

Normally, the engineering structures are designed in such a way as to remain, under the usual working conditions, in the elastic range of their response. Obviously earthquakes produce, as a rule, unusual conditions. Consequently, in such extreme situations, inelastic deformations of the structures may occur. They should, however, always remain

on the safe side (cf. Sec. VI).

In this aspect, the classical theory based on the assumption of physical linearity of the material properties is no more adequate for rational design. The classical elastic theory should, therefore, be replaced by a more general approach taking into account the inelastic phenomena.

For convenience, the inelastic behaviour can be classified into two groups, viz. viscoelasticity and plasticity, depending upon the source of mechanical energy dissipation.

Now, what are the essential differences between the elastic and the inelastic material responses?

An elastic (or reversible) deformation is, as we all know, the response of a material in the first stage of the loading process. It may be linear or nonlinear. It is not accompanied by energy dissipation. (We confine ourselves to discussing isothermal processes only).

Viscous flow is observed when the material is maintained for a long period of time under the action of external forces. This may be either reversible or irreversible; it is always connected with energy dissipation.

A plastic deformation is a kind of defence (self-defence) of the material against overloading. It is always irreversible and is always connected with energy dissipation.

Thus it may be seen that there is not only a quantitative, but, essentially, also a qualitative difference between elastic, plastic, and time-dependent phenomena.

The above remarks hold good, as a rule, for any material; they are also true for the engineering materials from which our structures are made.

In consequence, the response of our engineering structures to vari-

ous kinds of external agents depends (1) on the duration of the application of loads, and (2) on their intensity.

The theory of elasticity deals with reversible phenomena, and is not interested in and, therefore, cannot account for such effects like the time-dependent deformation processes which are generally called the rheological (or viscous) phenomena as, e.g. creep, relaxation, etc.; but it also cannot account for plastic effects.

On the other hand, the designer — a conscientious designer — wants to know what really is going to happen to his structure in the course of its existence, let us say, in a year, or two, or five; and obviously also, what is going to happen if the structure — by accident or purposely — is overloaded, overloaded in comparison with the originally planned design load.

All of us know that, e.g., concrete, our most popular material, exhibits very important changes in its behaviour, when it is loaded over long periods of time. This time-variable (rheological) phenomena are well known as its creep, relaxation etc.

But also steel exhibits rheological properties which, however, are of a quite different type than those of concrete.

On the other hand, overloading processes are always possible. They may be accompanied by plastic deformations.

Thus we have to develop theories which account for these facts. According to the above general scheme, the mechanical behaviour of structures may be governed:

(1) by the rheological properties of the materials, or

(2) by their plastic properties.

For these two types of response the common denomination of inelastic behaviour is generally being used.

Of course, the two types of response, (1) and (2), may also occur simultaneously. The corresponding general theory of elastic-visco-plastic bodies, obviously more complex, has also recently been developed and its first results are available for engineering applications.

II

As we know, the earthquake induced impulses, are — as a rule — of short duration. Therefore, engineering theories of earthquake-proof structures may, at least in the first approximation, disregard the rheological effects. They should, however, in any case take into account the plastic material properties.

The theory of plasticity was born in France, about a hundred years ago. Two or three excellent papers by Barré de Saint-Venant [1] and his co-worker Maurice Lévy [2], (1870, 1871), based on the experimental work by H. Tresca [3] (1868), were the first publications in this domain. But afterwards, for about thirty years, nobody was interested in plasticity. Only in 1904 Maksymilian Tytus Huber [4] formulated his well known yield condition which later on was, independently, reformulated by Richard von Mises [5] (1913) and Heinrich Hencky [6] (1924) in the same form. In the twenties and thirties, interesting research was going on in Europe and America. But a really significant gradient in plasticity research appeared only after World War II.

Now the theory of plasticity is already a well established discipline with its basic formulations and theorems, as well as countless engineering applications carefully verified by experimental evidence. However, it took many years before it was officially admitted to engineering practice. Especially the codes of practice were rather reluctant, and there

was a lot of discussion and contestation before the methods of the theory of plasticity were accepted not only by scientists, but also by the official authorities.

To-day the elastic-plastic analysis approach has already found very wide applications in structural design. In this connection, the notion of plastic hinges is being currently used with considerable success.

Let us consider a very simple but interesting example confirming that the plastic analysis often leads to straightforward results. Figure 1 presents a skeleton structure, the upper part of which can be considered to constitute a rigid block (stiffened by the walls). The loads are supposed to be vertical (dead and working loads) and horizontal (earthquake accelerations), V and H respectively. Theoretical considerations indicate that, with the increase of V and H, plastic hinges as indicated in Figure 2 will develop. I would like to insist, once more, on the fact that this notion when initially advanced and recommended for engineering applications, has severely been critisized and contested as being non-realistic. Experimental work on small scale models which clearly confirmed the inherent phenomena, was declared not to be reliable nor adequate.

Figures 4 and 5 dramatically confirm the theoretically predicted behaviour on a model in scale 1:1.

III

In order to arrive at straightforward conclusions, we have, so far, discussed a considerably simplified mechanical model. However, the kinematical scheme of the plastic movement is somewhat more complex than that resulting from the rigid-body translation indicated at Figure 2. As a matter of fact, there exists an additional kinematical constraint: a

stiff and resistant staircase located close to the left-hand rear corner of the building (cf. Figure 3). Consequently, the rigid block constituted by the two wall-stiffened upper stories effectuates not only a translation (as already discussed; cf. Figure 2), but simultaneously also a rotation with respect to a vertical axis passing through the fix staircase (cf. Figure 3) [7, 8]. Thus, in order to find the actual movement, both velocity fields have to be vectorially added. This enables us to correctly evaluate the inelastic performance of the structure and is also of importance when, to this effect, we proceed to properly establish the power balance. This balance, in such a situation, should be expressed in terms of the above resulting velocity field, compatible at present with the prevailing kinematical boundary conditions, as well as of the corresponding interior forces.

The Figures 4 and 5 confirm the existence of the phenomena discussed. The hinges can be easily located.

Note: Also small vertical (second order) movements occur, which together with the plane pattern previously discussed, finally result in a three-dimensional kinematical field. The vertical displacement may, however, in the first approximation be disregarded.

IV.

Here we have to add some additional explanations. It is known that in structures made of elastic-perfectly plastic materials, two types of plastic collapse may occur (cf. e.g. [9]). The first one takes place under a single loading programme as a result of unlimited increase of plastic deformations, in spite of the constancy of the ultimate load. This phenomenon is linked with the notion of the ultimate load carrying capacity of

the structure and the corresponding notions of limit analysis and limit design.

In the limit analysis, three elements are of basic importance: the intensity of the ultimate (limit) load; the distribution of the internal forces occurring in the ultimate stage of the structures; and the description of the ensuing plastic motion (plastic mechanism).

For engineering purposes, however, the most interesting quantity looked for is the intensity of the ultimate (limit) load, because this enables us to evaluate the safety of the structure against collapse and the corresponding coefficient of safety.

For this purpose, we have at our disposal the two well known fundamental theorems of limit analysis, the significance of which consists in providing the lower and the upper bounds of the collapse load. Both of them, the "statical" theorem, which directly leads to the lower bound for the collapse load, and the "kinematical" theorem, which provides its upper bound, have a clear and definite physical meaning. Therefore they have been readily accepted and are successfully applied in structural design.

On the other hand, the second type of failure occurs under time-variable loads acting in certain repeated cycles and, although during any particular cycle the structure does not collapse, it may do so after a certain number of cycles. If, on the contrary, the structure "adaptes" itself to the variable loading programme, it is said to have shaken-down.

Consequently, it is of basic importance to formulate a precise criterion which separates the above two alternatives. In a somewhat analogous way as in the above mentioned theory of the ultimate load carrying capacity, we are able to fix the bounds for the variability of the particular cycles in order to enable the structure to finally adapt itself to the

variable loading programme (to shake-down). If so, the cyclic loads may act upon the structure in any number and at arbitrary sequence; thus after final adaptation, no more plastic deformations will occur, the structure reacting further on in a purely elastic manner only.

The first formulation of the shake-down theorem was published in two famous papers by Ernst Melan [10] (1936, 1938). Later it has been generalized by Warner T. Koiter [11] (1956).

Both approaches, the ultimate load theory and the shake-down theory, have already found very wide applications in structural design. One of the extensions of the shake-down approach leads to the idea of prestressed structures implemented with such a success by Eugène Freyssinet and his numerous followers.

V

Let us return to our example now. We have to recall the fact that in Friuli two earthquakes occurred: the first started on May 6, 1976; the second on September 15, 1976. The structures were acted upon by severe earthquake shocks. During the first earthquake (May-June) over 250 shocks were recorded, during the second one (September-October), somewhat less. The structures were acted upon in a way which corresponds to cyclic loads.

The photographs 4 and 5 were taken in June 1976. We see that the skeleton adapted itself to the earthquake-generated shocks (it has shaken-down). In this form, it existed during more than 4 months successfully withstanding the influence of a long sequence of earthquake-shocks in May and June 1976. The plastic hinges (may be, more correctly, the brittle-plastic hinges) are clearly to be seen on the photographs. Soon,

comparatively simple means were conceived to neutralize the hinge-effects and there was every hope to recover the building for its normal housing and commercial purposes.

Unfortunately, the second earthquake, very violent and dangerous (Mercalli scale ~9) annihilated this scheme and the measures taken. This time, the structure did not shake-down. The final act of the tragedy is to be seen at Figure 6.

Finally, one example more. Figure 7 shows the formation of hinges in a continuous girder on columns with voutes. By theoretical analysis it may readily be found that the hinges, in this case, should develop below the voutes. It is seen once more that the earthquake clearly confirmed the predictions of the theory to be true. In this particular case, already the first earthquake was destructive. The collapse pattern to be seen on the picture follows from the special geometry of the structure.

VI

What conclusions are to be drawn from our observations and statements? They are of two kinds.

(a) In seismic areas, all possible effects of the earthquake-shocks have to be taken into consideration, evidently also the horizontal shock-generated forces. Their intensity should be evaluated on the basis of geophysical observations and the local geological conditions. The excitations being of random character, the stochastic approach is appropriate.

(b) The inelastic material properties generally constitute an additional reserve of safety of structures. In some situations, however, the inverse statement is true: they may reduce the margin of safety (e.g., when instability occurs). Therefore, their influence should, by an adequate analysis, carefully be examined.

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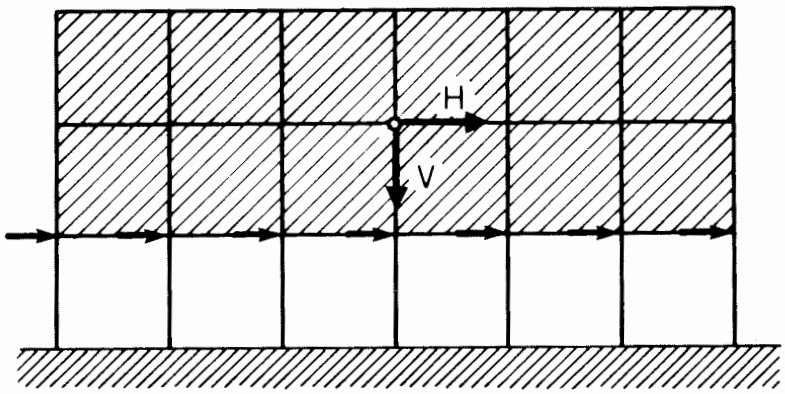

Fig. 1. Skeleton building (partially wall-stiffened)

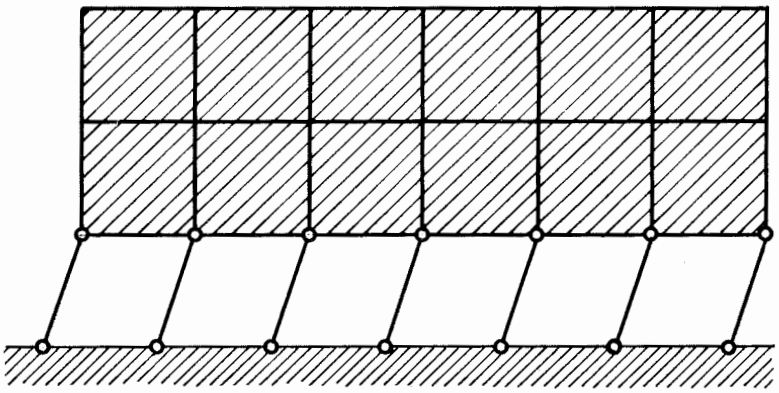

Fig. 2. Simplified plastic mechanism

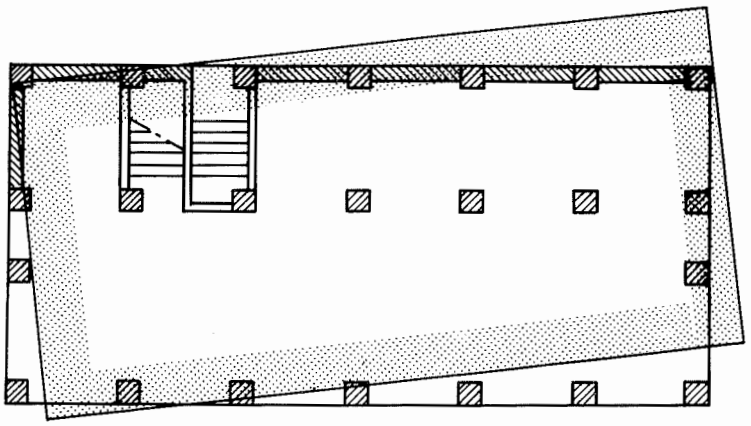

Fig. 3. Actual plastic mechanism

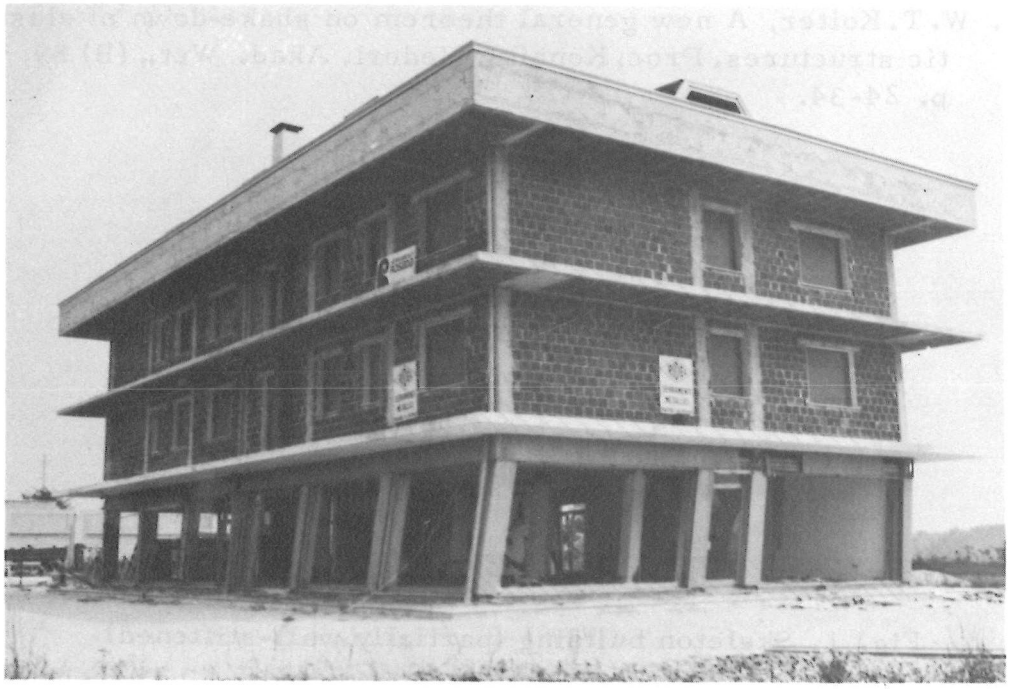

Fig. 4. Front elevation (Artegna)



Fig. 5. Side elevation (Artegna)



Fig. 6. Collapse of the building



Fig. 7. Collapse of r. c. framework (Maiano).

ON THE BEHAVIOUR OF A R.C. BUILDING WITH SUSPENDED FLOORS DURING THE FRIULI EARTHQUAKE ON MAY, 1976 (°)

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1 - INTRODUCTION

The office building of the Snaidero S.p.A. in Maiano is made of reinforced concrete; its middle floors, hang from the roof structure to a large extent, (fig. 1). This particular solution was chosen to meet both an architectural (as well and as) an urban requirement. The architectural requirement is met by providing a large area, free from columns, where to move on the ground floor. The urban requirement is met by not making the level of the top floor exceed the height limitations of habitable top floors: technical volumes above it were not subjected to such limitations.

In this way, a structure particularly unfit to withstand earthquakes have been conceived, because the main bulk of the building was placed on the top.

(°) Research carried out in the frame of the Italian Geodynamics project.

Research carried out in the frame of CNR's Italian Geodynamics Project. Pubbl. n. 29.

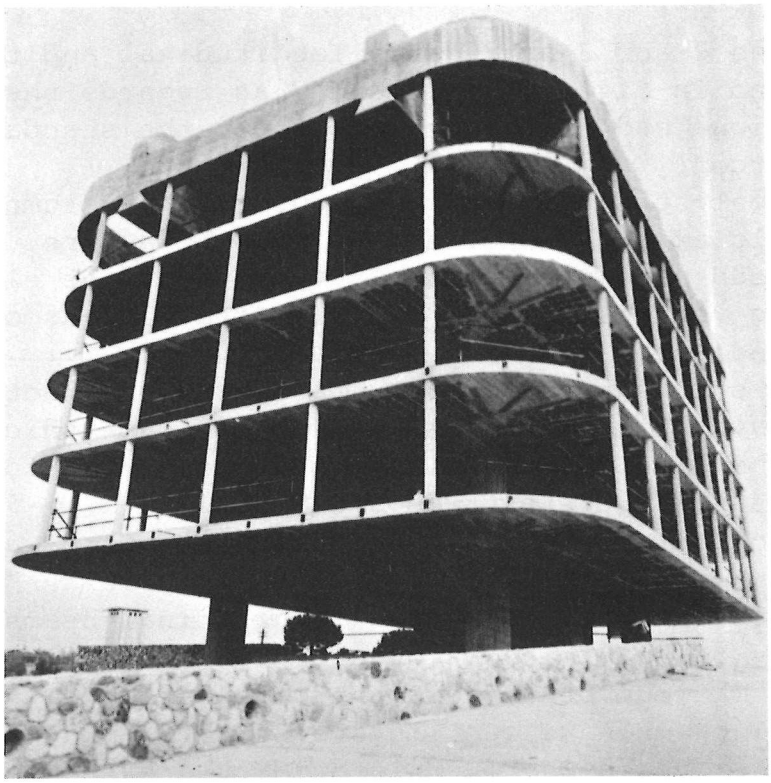

Fig. 1

The designers of the structure had accepted such a solution since the area was not considered a seismic zone, according to the seismic law in force in 1974.

At the time of the earthquake the structure was completed but the finishes not yet been added. It is therefore interesting to briefly present an explanation of the behaviour of this structure derived from a survey of the damages and its interpretation by means of a dynamic analysis. Such an analysis will also allow us to make a comparison with the regulations in force in seismic areas.

2 - DESCRIPTION OF THE BUILDING AND DESIGN CRITERIA

The parts of the building of greater interest to us are

shown in figures 2 and 3 as regards longitudinal and transversal sections and in figures 4,5,6 and 7 as regards the plans of the foundations and of the ground floor, the standard floor and the roof.

The vertical structure is constituted by 4 columns (each designed to hold axial load of about 1,200 t) a core, composed of reinforced concrete walls, which contain the elevators, the stairs, the vertical piping of the installations of air-conditioning and bathrooms.

The floor structure is a concrete joist construction with burned clay fillers; the beams have the same thickness of the slab; the middle floors lay on the core, the four main columns and a series of prestressed ties.

The horizontal structure of the roof is made of a network of 4 main beams which only lay on the 4 columns (this it is isolated from the core). The two longitudinal beams have two cantilevers of 7.50 m and a span of 18.75 m; they were designed for a maximum bending moment of about 1.700 tm. The two transversal beems have cantilevers of 5.625 m and a span of 11.25 m; other beams lay on this grid; the extremes of all cantilevers are connected by an edge beam distributing the loads from the prestressed ties.

The footings rest on piles of various diameters.

The connection of the footings of two of the big columns with that of the core is of notice.

As far as the design criteria are concerned, the building was designed for the vertical loads and was verified for only slight horizontal forces (about 1% of vertical loads),

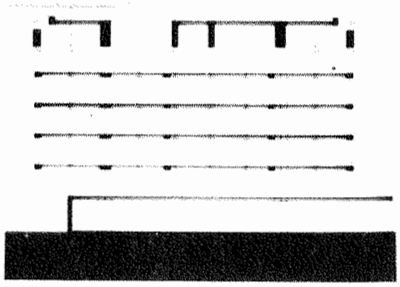

Fig. 2

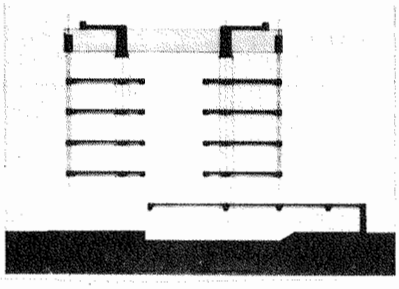

Fig. 3

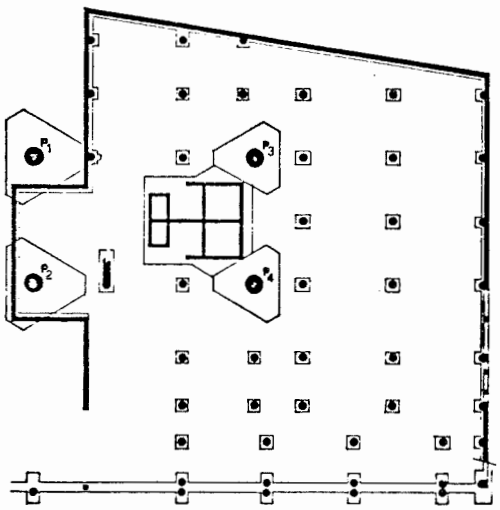

Fig. 4

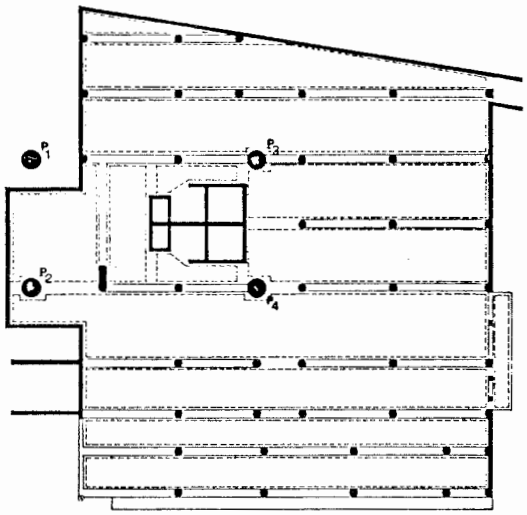

Fig. 5

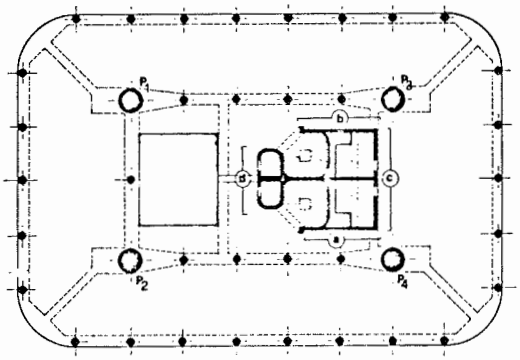

Fig. 6

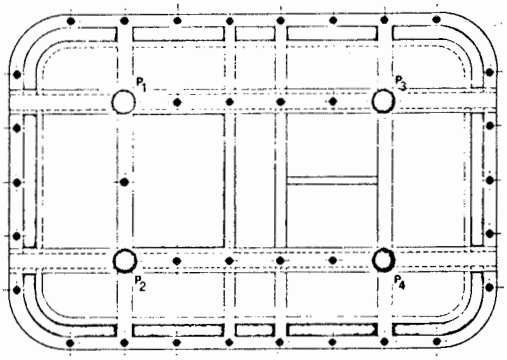

Fig. 7

because the site was considered in non seismic area. Not much importance was therefore given to the placing of the core which was not coincident at all with the centre of mass of the structure. This displacement magnified the effects of the earthquake by causing a singificant torque. In the same way the placing of the openings in the core walls was set up by purely architectural and installations requirements. The slight horizontal forces envisaged in the design allowed for any localized weakening of bracing structures.

Greater care was given to the study of stress and strain in the various steps of the construction of the building and to the effects of prestressing.

During the construction the thies were shored and were used as compression members to support the weight of the building.

After dropping the shores, the elastic deformations of the roof resulted as predicted by calculations and no cracks appeared.

3 - DESCRIPTION OF DAMAGES

The principal damages in the building, useful to identify its behaviour, are the following.

Damages to the main columns

In the four columns we note cracks due to compression located at the beam-column connection at roof level (fig. 8). They should have logically formed since the are didn't connect the roof, weighting roughly 2.000 t, and therefore the corresponding seismic forces only involved the four columns.

There are two similar cracks at the base of the two columns with independent footings. A spiral cracks runs through the column not attached to the slab of the ground floor, starting from the base and going up for 2 m.



Fig. 8

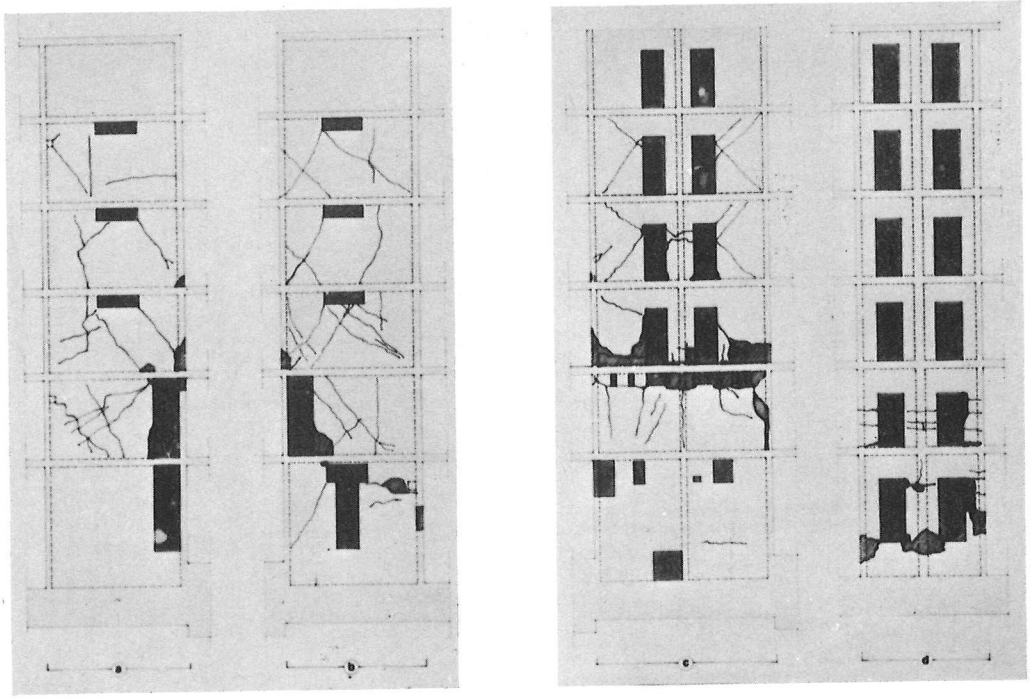

Fig. 9

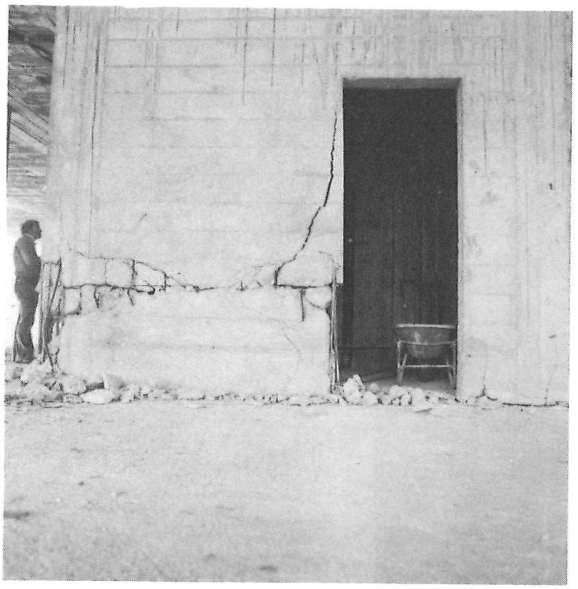

Fig. 10

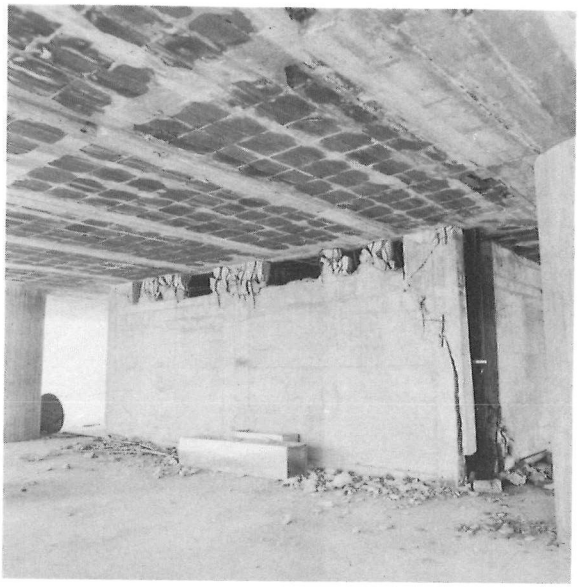

Fig. 11



Fig. 12

Damages to the prestressed ties

No significant damage to the ties except for small cracks, surrounding them on the edge beams, probably due to slight vertical effects, occurs.

Damages to the roof beams

No cracks are visible in the horizontal beams holding up the building.

Damages to the middle floors

No significant damage except for two transversal cracks in the burned clay fillers, the former starting from the big square hole of the slab, the latter corresponding to the core, occurs. They clearly show the in plane behaviour of the floors which mainly appears at the weakest and at the most stressed points. The break of the fillers in the areas close to the core is however remarkable.

Damages to the core

Because of their importance they are shown in fig. 9. Starting from the top we notice marked diagonal cracks in the walls delimiting the bathroom area; on the contrary the walls of the lidt space are undamaged.

At the second floor the breaks become more important just by the doors (fig. 10); the broken zone correspond also to the openings left for ventilation piping (fig. 11). There the core was too weak to withstand shear and torque due to seismic forces.

From the first floor down to the footing the walls of the bathrooms area are slighty cracked while important cracks appear in the lift area (fig. 12).

Damages to the basement

Horizontal cracks occurred along the external walls, holding up the slab of the ground floor; they show that shear and torque at this level were distributed between the core and the external walls even if they were not strong and had weak foundations.

4 - SEISMIC ANALYSIS OF THE BUILDING ACCORDING TO THE ITALIAN REGULATIONS

The seismic analysis of the structure is based on the dynamical model shown in figures 13 (longitudinal section), 14 (transversal section), 15 (roof plan), 16 (tipical floor plan) and 17 (ground floor plan). The roof is sketched by a grid of beams closely following the layout of the real structure, while the other floors are represented by rigid in plan elements but perfectly flexible.

The vertical elements correspond to those of the real structure only but the core represented by a beam whose section fulfil equivalent geometric properties. External walls below the ground floor are skatched by spring having an elastic constant equal to the shear stiffness of the walls. The inertia of the roof is represented by lumped masses placed at the intersections of the beams with the vertical element. To deal with the floors skatched with rigid inplan elements we have considered the kinetic energy associated with horizontal displacements through a mass having a moment of inertia around its vertical axis, concentrated in the centre of mass of the floor. We have treated the kinetic energy due to vertical displacements by a series of lumped masses placed at the intersections with vertical elements. Finally effects caused by deformations of the ground have been taken into account by springs of suitable stiffness situated in the center of mass of the lower face of the main footings.

With such a model the seismic analysis according to the italian regulations for the area of second category have been carried out, by separately considering the effects of shocks

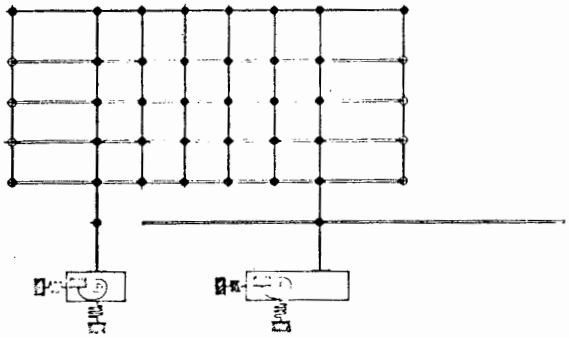

Fig. 13

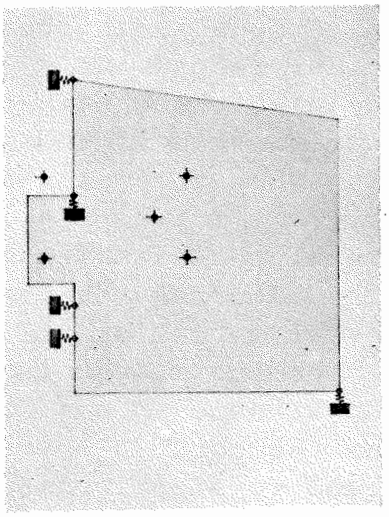

Fig. 17

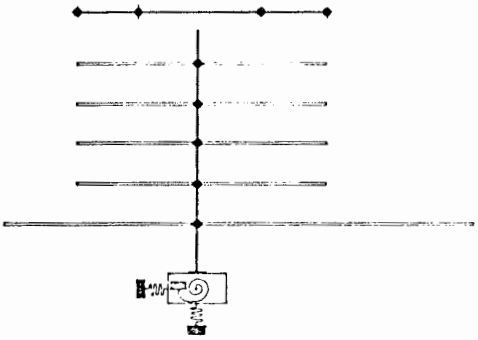

Fig. 14

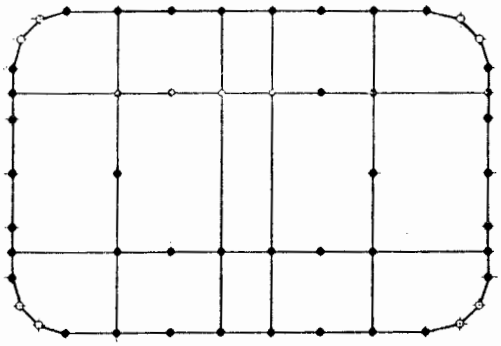

Fig. 15

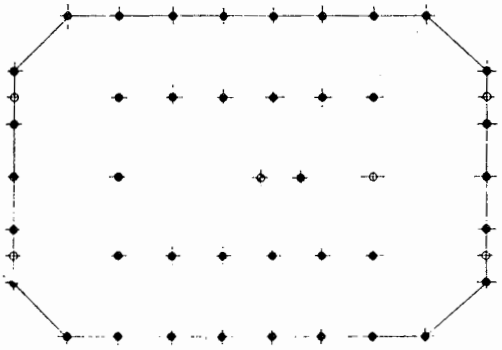

Fig. 16

acting in longitudinal and transversal directions (°). The vertical ecitation has also been studied. Calculations have been performed for two different situations: first the external walls soustaining the ground floor have been supposed as effective, second their effects have been completely overlooked. We have thaught that it was intersting to verify the structure in the second case to analyze the bahaviour of the building when the walls are not able to withstand horizontal forces. This may happen either for its own failure or for failure of the slab itself.

The main results of the computations for excitation in trasversal direction are presented in fig. 18 e 19. In fig. 18 the bending moment in the columns with indipendent footing (P_1 and P_2) and in one of the two columns with the same footing as the core (P_3) are plotted. In fig. 19 bending moment, shear and torsional moment for the core are shown. The continous diagrams refer to the results obtained under the hypothesis that external walls are effective, those with dashed line refer to the second case.

The seismic analysis according to the above regulations has determined the following conclusions. The highest stresses due to seismic effects on the roof beams are about 10% of those due to static loads. The design of these beams fulfil the italian regulation as well as the prestressed ties.

The two main columns (P_1 and P_2) are at the verge of the building code. Particularly the compression stress in the concrete are 140 kg cm^{-2}.

The condition of the core is completely unsatisfactory because of openings mainly at the holes for air conditioning pipes. The stresses would only be admissible where the horizontal forces were negligible (maximum sectral acceleration equal to 0.03 g).

(°) We have considered the response spectrum defined by the following function:

$$S_a(T) = \begin{cases} 0.07\ g & \text{when } T \leq 0.8\ s \\ 0.07\ g\ (0.862/T^{2/3}) & \text{when } T \geq 0.8\ s \end{cases}$$

where T is the natural period.

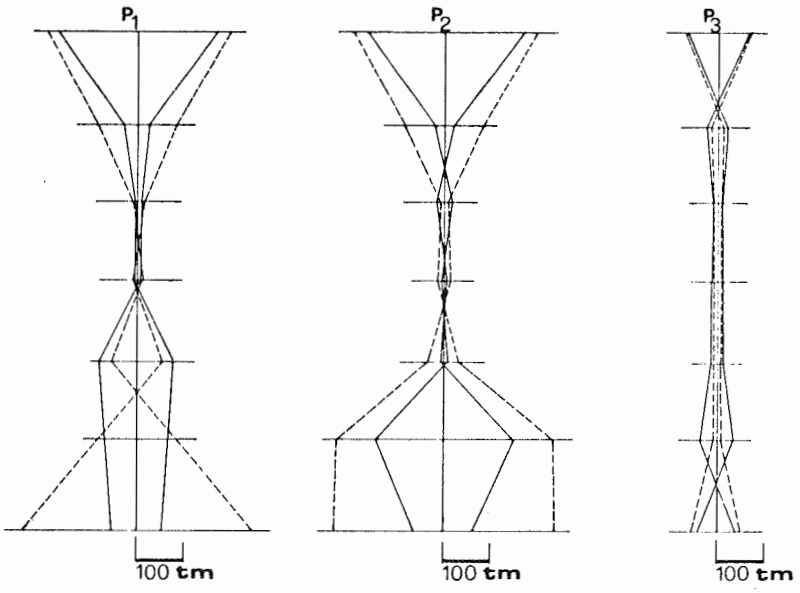

Fig. 18

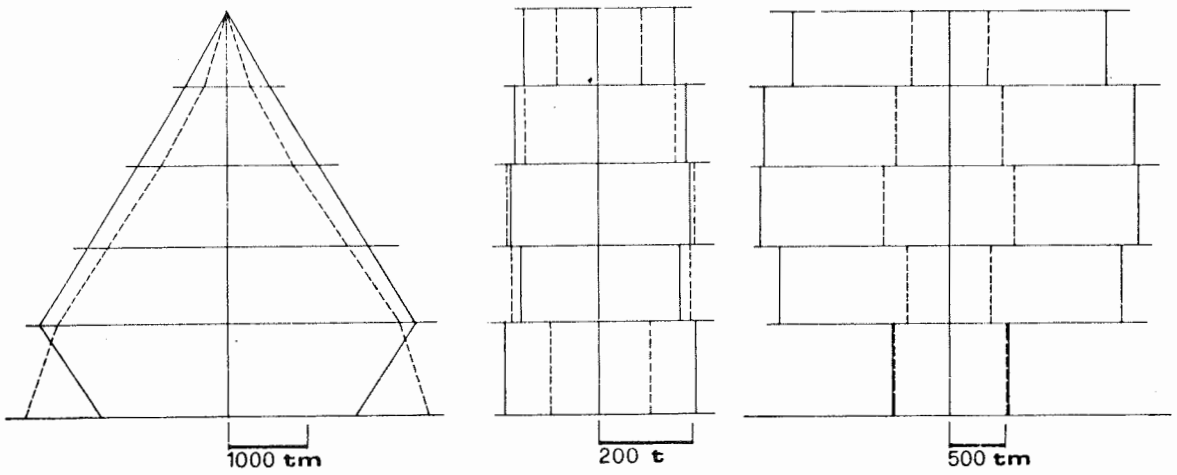

Fig. 19

5 - ANALYSIS OF THE DYNAMICAL BEHAVIOUR OF STRUCTURES

Unfortunatly we have not accelerograms of the main shock recorded near the building; therefore it is only possible to look for some approximate idea of the actual behaviour studying the response of the dynamical model to the accelograms recorded near Tolmezzo.

At the time of the earthquake the structures were completed but no finishes were added,as previously said. Therefore the real masses of the building were those corresponding to the dead load of structural member, while all other dead loads and live were not present. Then the real behaviour must be studied with a model having reduced masses.

The analysis is performed using the response spectrum of the N-S component of the accelerograms recorded in Tolmezzo; a damping factor of 0.05 is assumed for all modes.

In figures 20,21,22 and 23 the most significant internal forces obtained using the response spectrum of Tolmezzo record (dashed lines) are compared with those computed assuming the regulation spectrum (°) (continous lines). The plotted quantities are the same as in fig. 18 and 19. Figures 20 and 21 refer to computations carried on under the hypothesis that the external walls are effective, while the results of fig. 22 and 23 are obtained neglecting this elements.

It is possible to see from figures that the effects of the real earthquake in the columns and in the core are about fivetime larger than those given by regulations, when the external walls are considered (fig. 20 and 21); if the walls are neglected the ratios between the maximum values of internal forces range from about 3.5 for the bending moments in columns (fig. 22) to about 5 (locally also 8) for the torsional moment in the core (fig. 23).

(°) These values are slightly different from those shown in fig. 18 and 19, due to the different values of the masses and then of the natural periods.

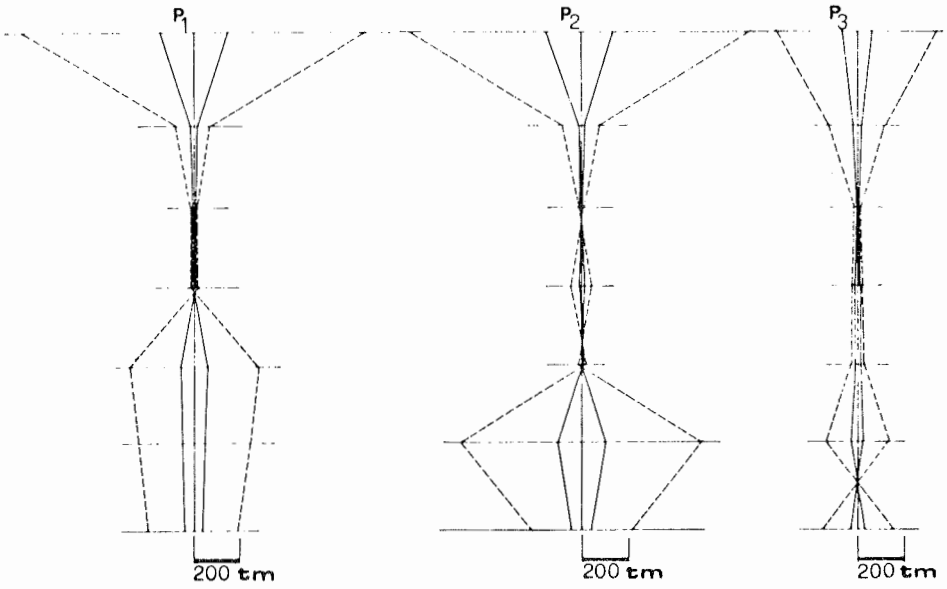

Fig. 20

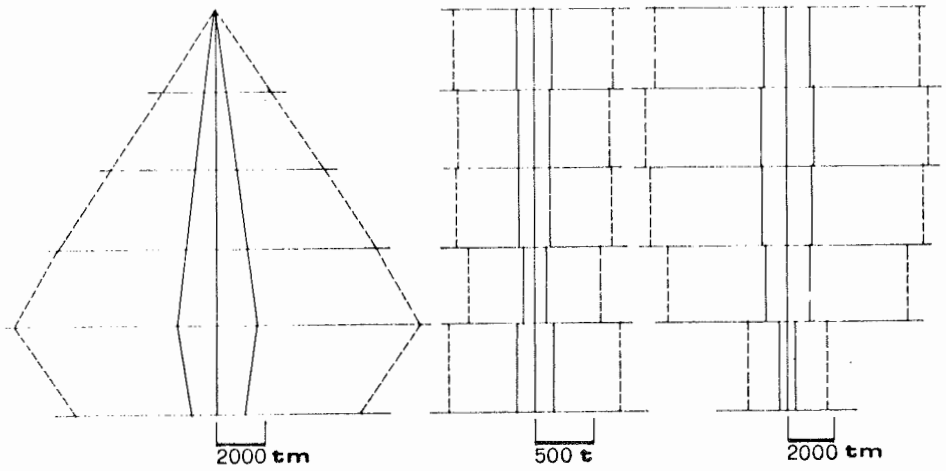

Fig. 21

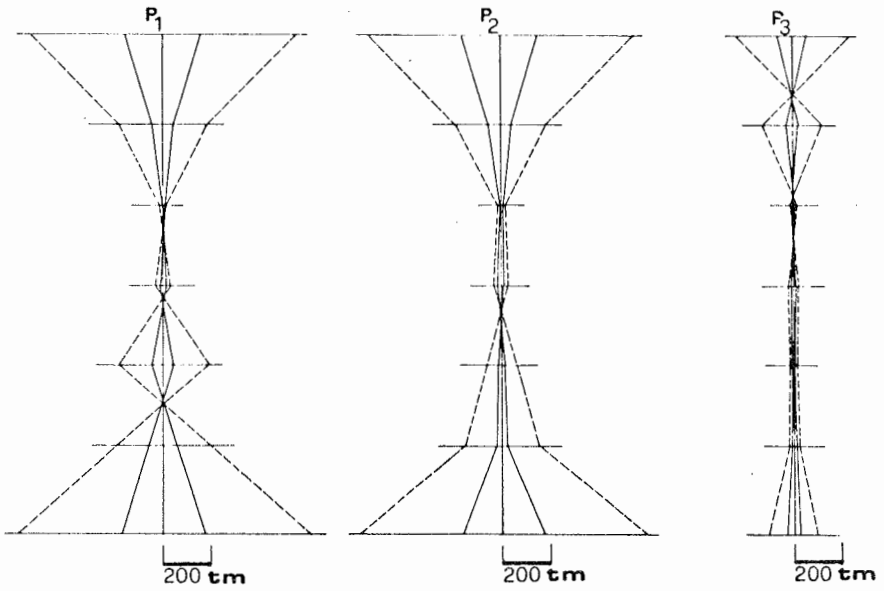

Fig. 22

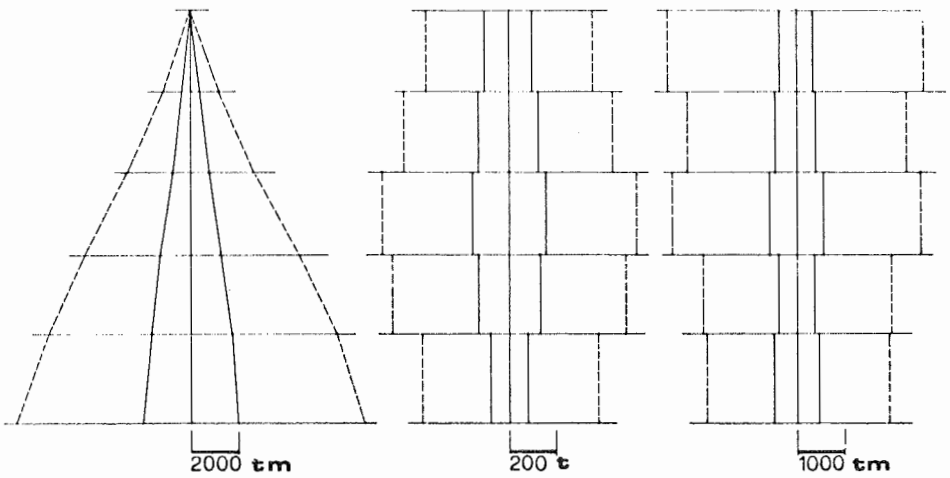

Fig. 23

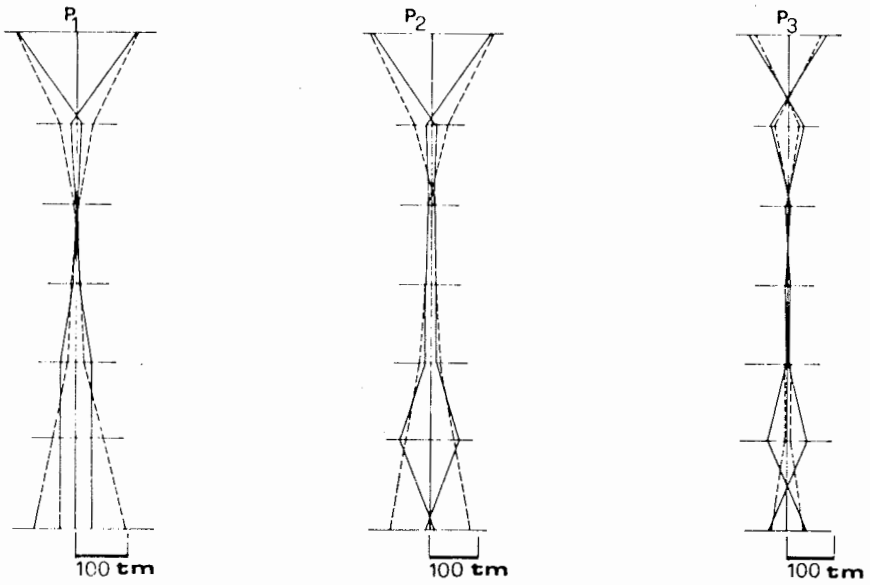

Fig. 24

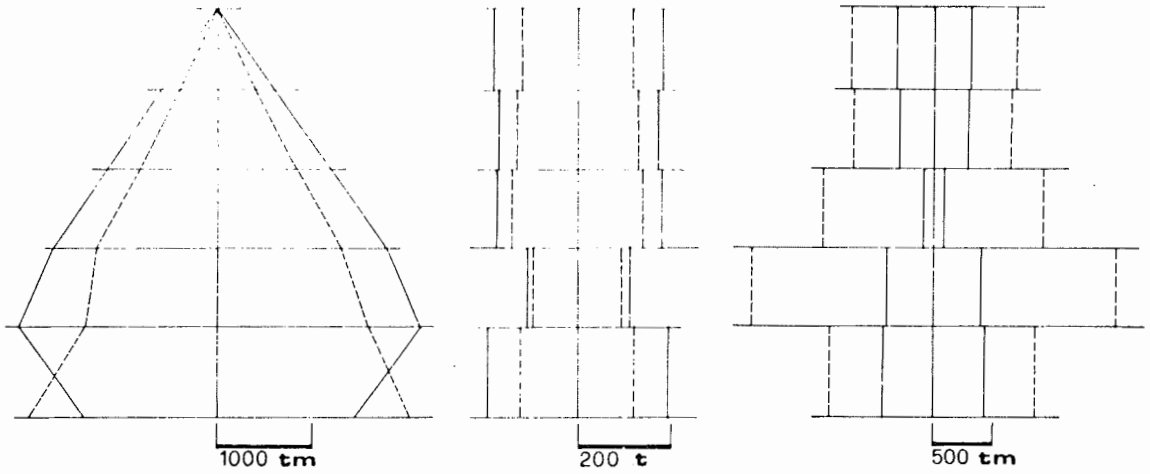

Fig. 25

6 - CONCLUSIONS

The dynamic analysis and the survey of the damages show that all structural member so designed to satisfy the seismic code requirements, did not suffer important damages (the most severe, even if negligible, being the cracks at the top of the main columns that are at the verge of the code). On the other hand, the structural element, and particular the core, that can not withstand the seismic forces specified by the code with allowable stresses, suffered damages large enough to compromise the future behaviour of the structure.

If the building was conceived from the beginning with a vertical element apt to reduce the torsional effects and with a different layout of the openings in the core, no significant damages had been occurred. This assertion is justified by the results shown in fig. 24,25 and 26 (°). They were obtained analyzing a modified structure where a shear wall between columns P_1 and P_2 was added and the stiffness of the core was increased. The figures show a reduction of the values of internal forces such to make them in agreement with the requirements of seismic code.

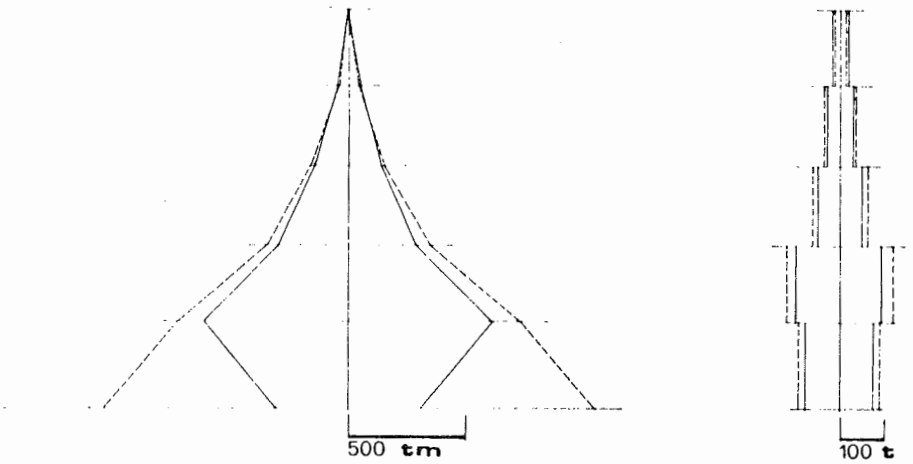

Fig. 26

(°) Continous line plots refer to the case of efficient external walls while dashed line plots are obtained disregarding these elements.

DYNAMIC ANALYSIS OF FRIULI R .C.DAMAGED BUILDINGS

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Italy.

1. INTRODUCTION

After the shocks of may 6th 1976 a field investigation was performed by the authors in the more damaged area of Friuli. A number of residential buildings having a reinforced concrete main structure was investigated in detail. Among them, four buildings have been selected, all having the following features: a) the structural type and scheme was the most usual for recently erected buildings in the Friuli area (or more in general in northern and central Italy), b) none was designed under the previsions of a seismic code, c) all were at the point of collapse (and actually some collapsed during the following shocks), d) all suffered similar structural damages and exibited similar collapse mechanisms. The upper storeys had minor damages and they behaved as a rigid

Research carried out in the frame of the Italian Geodynamics Project. Publication No.

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body in comparison to the ground floor, where more flexible vertical structures were present and large deformations and plastic hinges at columns took place.

A more detailed description of the buildings considered can be found in ref. [1] .In the same reference and in ref. [2] investigations concerning the capability of the buildings to withstand seismic loads have been performed. The following conclusions can be pointed out:

a) all buildings, even if not explicitly designed for seismic loads, could withstand a quite large amount of horizontal forces. A figure for the seismic coefficient ranging from 0.05 to 0.10 has been obtained, according to the admissible stresses defined by italian codes for reinforced concrete.

b) the ductility of the buildings was rather constant and large. The ratio between the displacements at the limit state and the displacements at admissible stresses (both in the framework of the italian codes for r.c.) ranged from 10 to 13.

As a matter of fact all the above considerations are useful to correlate the experimental event "collapse" to theoretical statements of a building code. No explicit reference has been made to the actual earthquake and to the actual history of the structure during that earthquake. The present paper is a first attempt to make an analysis of the dynamic behaviour of the buildings considered in the above mentioned references.

Only simplified models of the structures will be adopted in the sequel: in particular only the linear behaviour of the structures will be taken into account, despite the fact that the actual structures went deeply into che non linear range. This is done basically because the aim of the paper is not to explain with theoretical considerations why structures collapsed, but rather to correlate the results of conventional calculations to experimental evidence. In the authors' opinion the following calculations represent the deepest theoretical analysis that practitioners would make to design structures of few storeys dwellings. On the other hand, even remaining at this level of simplification, some of the

input data for linear structural dynamic are not available with a sufficient degree of reliability: damping of the structure, effects of deformability of foundations and ground, stiffness of non-bearing elements and so on. The present paper reports (as a small advance outside the domain of the dynamic analysis that is conventionally used in the design practice) some parametric analysis with respect to variations of the above mentioned uncertain input data.

2. SEISMIC LOADS

The only record of the main shock of may 6th has been published in ref. [3] , from which fig.1 is reproduced. Digital data obtained from that figure have been adjusted by the procedure described in ref. [5] . In order to allow comparative calculations of different research works, the significant first seconds of the digitalized records are listed in table 1. In the sequel the ground motion of tab.1 is used as input for all the time-history responses considered. No analytical seismological study has been published up to now for the entire Friuli area, therefore a reliable correlation between the recorded ground motion and the actual motion at the base of the buildings examined is not available. However some considerations on this point can be made:

a) the building are situated at a distance ranging from 8 to 15 Km sourthwards from the epicenter, while the record has been taken at about 8 km north-west from the epicenter (see map in ref. 1);

b) the area of the buildings examined was the most damaged in the region of the hitten by the shock of may 6th. This is probably due to the geological condition of the Tagliamento river valley that moves sourthward from the epicenter to the zone where the buildings stood

c) all the buildings examined rested on nearly similar ground: alluvium, having a thickness of about 50 meters at least. The record was taken on rocks.

The above considerations suggest that an actual ground motion lower than the recorded one seems to be unlikely for the buildings. Lacking a more detailed seismological analysis (now in progress by other specialists) the authors will

(text follows at page 11)

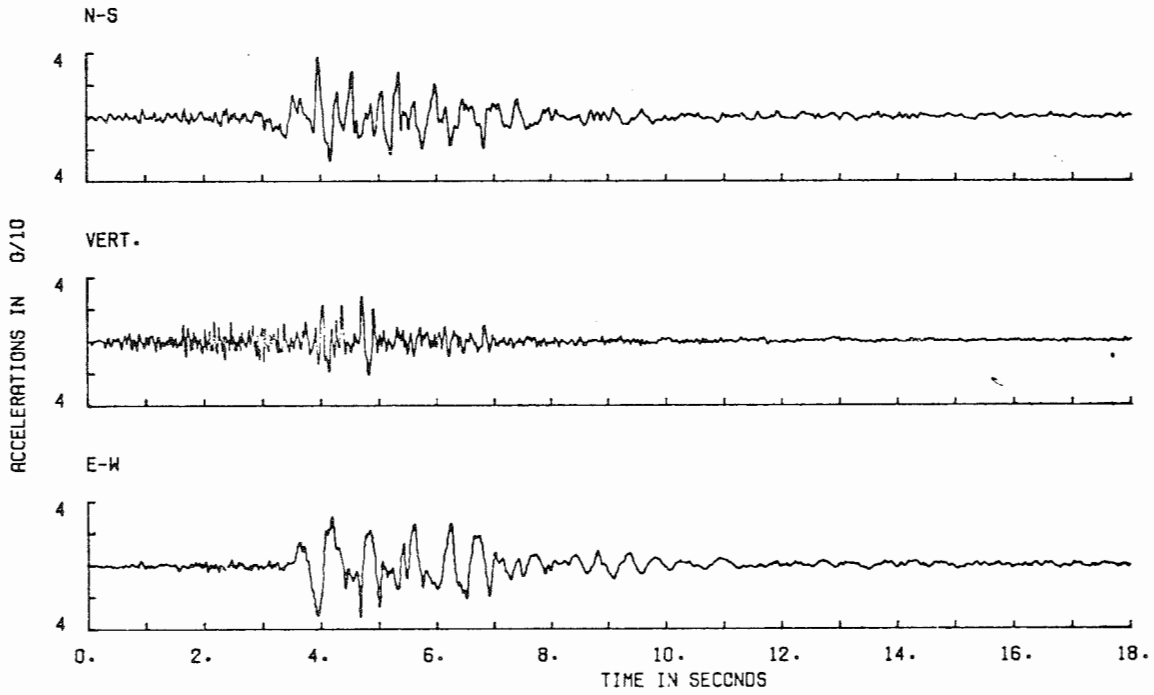

Fig.1. Friuli earthquake on may 6,1976 recorded at Tolmezzo(first 18 second of record). From reference 3.

TABLE I-ADJUSTED ACCELEROGRAMS

TOLMEZZO, may 6, 1976, N - S

t	a/g	t	a/g	t	a/g	t	a/g
.0000	.002695	.0120	-.009214	.0380	-.001834	.0610	.006548
.0640	.014531	.1130	.008509	.1330	-.001906	.1560	-.010824
.1790	-.009341	.1940	-.000453	.2110	.007035	.2370	.012915
.2630	.009895	.2690	-.006424	.3070	-.010938	.3330	-.019857
.3500	-.025870	.3590	-.015377	.3670	-.004083	.3760	.014311
.3900	.024700	.4080	.029187	.4370	.018765	.4510	.005355
.4660	-.005056	.4920	-.006576	.5060	-.000586	.5210	-.002097
.5350	.009792	.5520	.015680	.5760	.011262	.5930	.003749
.6100	-.006663	.6250	-.011074	.6450	.000811	.6620	.017099
.6800	.030485	.6850	.014182	.7140	.011160	.7350	-.008155
.7490	.000735	.7690	.002220	.7870	-.005293	.8040	-.005306
.8180	.006584	.8270	.015578	.8440	.011065	.8500	-.000839
.8620	-.015748	.8820	-.017262	.8990	-.026175	.9170	-.026188
.9250	-.005294	.9340	.011000	.9430	.030393	.9570	.055683
.9780	.037768	.9920	.024358	1.0040	.000549	1.0150	-.003859
1.0580	-.003875	1.0670	-.002496	1.0870	.007989	1.1080	.001974
1.1250	-.008438	1.1390	.007952	1.1510	.019843	1.1660	.037633
1.1800	.021323	1.1860	.001918	1.1940	-.018687	1.2150	-.006502
1.2320	-.011514	1.2490	-.007027	1.2640	.007863	1.2810	-.001049
1.2900	-.016956	1.3070	-.026368	1.3270	-.017482	1.3500	-.007098
1.3640	.010792	1.3730	.027185	1.4010	.031566	1.4180	.018154
1.4300	.001845	1.4530	.006229	1.4670	.000319	1.4930	.007701
1.5100	.022589	1.5330	.032973	1.5520	.000259	1.5790	-.010159
1.6040	-.017077	1.6240	-.025091	1.6410	-.007203	1.6490	.023992
1.6640	.041881	1.6780	.059672	1.6920	.020962	1.7040	-.017646
1.7270	-.036782	1.7410	-.035572	1.7720	-.026694	1.7840	.007598
1.7920	.032896	1.8120	.003079	1.8350	-.022237	1.8750	-.006865
1.8920	.000024	1.9090	.016412	1.9320	.032796	1.9520	.030683
1.9690	.035871	1.9860	.010460	2.0060	-.013354	2.0320	.000028
2.0520	-.008985	2.0690	-.002997	2.0890	-.016411	2.1000	.002882
2.1090	.011870	2.1200	-.006032	2.1460	-.022449	2.1660	-.025363
2.1770	-.004570	2.1860	.020724	2.2000	.034114	2.2260	.016297
2.2370	-.001610	2.2490	-.029919	2.2660	-.025430	2.2800	-.036839
2.3030	-.050755	2.3200	-.001666	2.3430	.044418	2.3540	.029611
2.3690	.013401	2.3890	.044388	2.4000	.060780	2.4110	.026573
2.4230	-.019535	2.4340	-.030042	2.4540	-.003255	2.4710	.020533
2.4800	.002727	2.4890	-.010679	2.4970	.011616	2.5110	.020607
2.5310	.029494	2.5490	.008682	2.5600	-.009225	2.5910	-.006246
2.6090	-.021156	2.6230	-.028567	2.6400	-.000378	2.6660	.022005
2.6800	-.004804	2.6940	.021987	2.7110	.026375	2.7310	.007062
2.7460	-.021247	2.7710	-.012364	2.7890	-.022775	2.8090	-.030188
2.8260	-.024199	2.8490	-.007914	2.8690	.023373	2.8970	.041155
2.9140	.015844	2.9310	.036733	2.9430	.015825	2.9510	.005420
2.9690	.032208	2.9910	.023294	3.0110	.000981	3.0200	-.022824
3.0260	-.049628	3.0430	-.074939	3.0570	-.054148	3.0660	-.019954
3.0800	-.009463	3.1060	-.025879	3.1230	-.007990	3.1370	-.003599
3.1510	-.021406	3.1630	-.049715	3.1770	-.076524	3.1890	-.083932
3.2090	-.070545	3.2260	-.058655	3.2400	-.049764	3.2570	-.063175
3.2740	-.075086	3.2860	-.054193	3.3060	-.039406	3.3140	-.054211
3.3260	-.072118	3.3340	-.094423	3.3520	-.104635	3.3800	-.109352
3.3920	-.116700	3.4060	-.125768	3.4170	-.100475	3.4380	-.049886
3.4520	-.015597	3.4750	.002188	3.4920	.061778	3.5060	.113769
3.5230	.137559	3.5390	.119749	3.5550	.091439	3.5720	.063128
3.5870	.052719	3.5980	.033412	3.6090	.024506	3.6180	.063200
3.6350	.095090	3.6520	.119679	3.6700	.093668	3.6900	.066056
3.7040	.039047	3.7240	.019935	3.7610	.011012	3.7700	-.003893
3.7960	-.008309	3.8160	-.005421	3.8360	-.024733	3.8470	-.051540
3.8590	-.069447	3.8730	-.094656	3.8870	-.091764	3.9050	-.021775
3.9160	.080918	3.9250	.173113	3.9300	.249010	3.9390	.311504
3.9450	.342601	3.9530	.369496	3.9760	.309982	3.9910	.247473
4.0150	.122458	4.0310	.061449	4.0420	-.002556	4.0540	-.053165

TABLE I (follows)

t	a/g	t	a/g	t	a/g	t	a/g
4.0650	-.105171	4.0740	-.133477	4.0910	-.154287	4.1190	-.160304
4.1340	-.181113	4.1400	-.207916	4.1480	-.234721	4.1600	-.257028
4.1710	-.268934	4.1850	-.222843	4.2000	-.152852	4.2110	-.084458
4.2170	-.045762	4.2250	.004833	4.2370	.055426	4.2460	.089621
4.2630	.104511	4.2770	.134303	4.2910	.159595	4.3030	.122387
4.3110	.059683	4.3170	.006279	4.3430	-.029436	4.3600	-.027946
4.3710	-.045652	4.3880	-.066662	4.4030	-.078571	4.4260	-.032485
4.4450	.027104	4.4540	.064299	4.4770	.113386	4.4970	.169874
4.5110	.214566	4.5250	.254658	4.5450	.201446	4.5540	.256141
4.5570	.221939	4.5600	.159438	4.5620	.102936	4.5680	.046333
4.5710	-.005769	4.5770	-.053372	4.5880	-.093579	4.6020	-.062287
4.6110	-.033992	4.6200	-.019197	4.6280	-.047402	4.6340	-.077205
4.6420	-.105910	4.6590	-.126319	4.6880	-.119836	4.7220	-.096555
4.7420	-.029667	4.7680	-.004382	4.7820	.026910	4.7990	.010501
4.8140	-.008068	4.8340	.011981	4.8540	.062570	4.8710	.086360
4.8850	.022352	4.9000	-.059456	4.9140	-.102564	4.9250	-.114571
4.9540	-.072687	4.9680	-.031195	4.9910	.066992	5.0060	.111584
5.0200	.147376	5.0430	.151763	5.0540	.162257	5.0660	.129450
5.0770	.043144	5.0860	-.020861	5.0940	-.066965	5.1090	-.099673
5.1320	-.139886	5.1520	-.162197	5.1720	-.177108	5.1890	-.211318
5.2030	-.230726	5.2200	-.189035	5.2290	-.068540	5.2410	.014854
5.2520	.074347	5.2720	.145737	5.2920	.162126	5.3180	.211211
5.3520	.243904	5.3470	.276695	5.3580	.223089	5.3640	.111486
5.3670	.040085	5.3750	-.044820	5.3900	-.080528	5.4040	-.013536
5.4240	.038054	5.4440	.014743	5.4640	.028132	5.4840	.002821
5.5020	-.032889	5.5220	-.074599	5.5390	-.029909	5.5500	.011685
5.5580	.062476	5.5930	.072662	5.6190	.097948	5.6280	.053344
5.6450	-.012165	5.6590	-.049373	5.6910	-.073190	5.7140	-.097002
5.7230	-.137407	5.7400	-.177316	5.7570	-.193725	5.7890	-.140142
5.8090	-.083652	5.8340	-.049466	5.8570	-.006278	5.8690	.016016
5.8910	.041904	5.9110	.094894	5.9310	.120184	5.9570	.166270
5.9630	.203456	6.0060	.138045	6.0260	.047234	6.0460	-.031676
6.0740	-.016791	6.1090	-.031709	6.1230	-.006416	6.1430	.041274
6.1630	.065064	6.1830	.039753	6.1970	-.039154	6.2170	-.116064
6.2510	-.173071	6.2540	-.171583	6.2710	-.156692	6.2910	-.106102
6.3060	-.064509	6.3260	-.051120	6.3490	-.063031	6.3750	-.068944
6.3920	-.046653	6.4030	-.006458	6.4150	.053036	6.4290	.082829
6.4550	.108116	6.4750	.078306	6.4920	.045597	6.5040	.060491
6.5180	.076784	6.5410	.054473	6.5700	.038058	6.5900	.061948
6.6130	.060437	6.6240	.069331	6.6530	.033617	6.6670	-.003590
6.6870	-.036500	6.7050	-.052708	6.7300	-.033321	6.7530	-.037832
6.7760	-.070643	6.7850	-.118248	6.7990	-.164354	6.8220	-.191166
6.8400	-.127174	6.8570	-.063183	6.8720	.014210	6.8830	.049905
6.9150	.054389	6.9440	.034975	6.9580	.055669	6.9780	.084059
7.0100	.070644	7.0480	.048326	7.0590	.063220	7.0800	.081110
7.0970	.048302	7.1200	.021491	7.1490	-.020122	7.1690	-.048432
7.1690	-.058641	7.2180	-.046955	7.2500	-.061870	7.2850	-.069286
7.3140	-.041000	7.3310	.000592	7.3460	.036385	7.3720	.067573
7.3920	.101764	7.4100	.107756	7.4330	.069045	7.4710	.020828
7.5000	-.000080	7.5530	-.044103	7.5610	-.076814	7.5960	-.060430
7.6200	-.066440	7.6430	-.041151	7.6690	-.050063	7.6920	-.051573
7.7130	-.041183	7.7270	-.033689	7.7510	-.038200	7.7740	-.038210
7.8030	-.023523	7.8230	-.021832	7.8460	.007958	7.8780	.042144
7.8960	.034636	7.9160	.022727	7.9340	.037619	7.9650	.055505
7.9630	.035096	7.9970	.012291	8.0120	-.017415	8.0320	-.021924
8.0610	.012263	8.0620	.030154	8.1020	.043546	8.1250	.021236
8.1480	.012526	8.1770	.004813	8.2180	.007796	8.2560	.001879
8.2620	-.013032	8.3140	-.017546	8.3370	-.014555	8.3650	-.011567
8.3630	-.004175	8.4260	-.001193	8.4580	-.002706	8.4950	.013678
8.5180	-.005731	8.5320	-.022037	8.5640	-.036951	8.5870	-.050360
8.6090	-.035482	8.6650	-.005692	8.6820	.016601	8.7140	.040388

TABLE I (follows)

t	a/g	t	a/g	t	a/g	t	a/g
8.7570	.019578	8.7420	-.001324	8.7630	-.020632	8.7860	.001658
8.8000	.018553	8.8200	.026944	8.8520	.010631	8.8700	-.014676
8.8640	-.022181	8.8990	-.008787	8.9160	.015006	8.9300	.026900
8.9620	.012087	8.9630	.001679	9.0030	-.016229	9.0230	-.023637
9.0550	.004550	9.0780	.029841	9.1040	.053731	9.1280	.034322
9.1510	.020915	9.1770	.016502	9.2000	.004593	9.2230	-.013315
9.2460	-.035624	9.2700	-.046034	9.2990	-.040145	9.3250	-.044555
9.3570	-.043067	9.3980	-.016383	9.4160	-.008690	9.4570	-.014905
9.4980	-.000021	9.5450	.019362	9.5830	.047647	9.6130	.026736
9.6390	.005926	9.6690	.001415	9.6980	-.001495	9.7210	-.022404
9.7450	-.028513	9.7740	-.043223	9.7980	-.037232	9.8240	-.020942
9.8530	-.016452	9.8860	-.016464	9.9270	-.019479	9.9500	-.008987
9.9830	.007299	10.0150	-.010510	10.0410	-.019420	10.0730	-.015031

TOLMEZZO, may 6, 1976, E - W

t	a/g	t	a/g	t	a/g	t	a/g
.0000	-.004207	.0170	.003092	.0490	-.002710	.0810	.001688
.1040	-.004414	.1300	.000184	.1620	.000182	.1940	-.005620
.2140	-.002721	.2370	-.007023	.2550	-.007024	.2830	-.006526
.3610	-.002927	.3240	.000271	.3440	-.004130	.3640	-.002632
.3980	.000287	.4110	-.007035	.4310	-.012836	.4600	-.007038
.4970	-.012341	.5210	-.006942	.5520	-.005544	.5840	-.004047
.6100	-.011346	.6300	-.001059	.6590	.004846	.6800	-.002553
.6970	-.000354	.7170	-.002556	.7430	.012143	.7660	.004841
.7570	.005340	.7980	-.003961	.8210	-.005362	.8440	-.003964
.8590	-.005365	.8730	-.011266	.8820	.004634	.8970	.019533
.9240	.029631	.9310	.019530	.9430	.006330	.9630	-.005372
.9780	-.009770	.9920	-.019974	1.0090	-.012675	1.0350	.000524
1.0610	-.011176	1.0870	.000520	1.1130	-.008182	1.1370	-.012583
1.1550	-.009685	1.1920	-.002387	1.2180	.007912	1.2410	.003510
1.2640	.010909	1.2810	.002108	1.3130	.003606	1.3410	.013804
1.3640	.005002	1.3960	.005100	1.4270	-.002302	1.4530	-.000803
1.4640	.000695	1.5040	.007993	1.5150	-.000707	1.5240	-.012508
1.5440	-.011009	1.5530	-.022709	1.5640	-.034410	1.5870	-.025612
1.5980	-.015412	1.6100	-.002213	1.6320	-.000715	1.6550	.000784
1.6780	-.008017	1.6950	-.015318	1.7180	-.008020	1.7410	-.012421
1.7460	-.005022	1.7690	.012477	1.7840	.002276	1.8010	-.006525
1.8150	-.000626	1.8320	.008173	1.8460	.019872	1.8630	.000871
1.8830	.006170	1.9010	.011069	1.9180	.019868	1.9260	.003767
1.9400	-.012333	1.9550	.006766	1.9750	-.013736	1.9860	-.003536
2.0090	.006762	2.0150	.021362	2.0290	.025761	2.0380	.005261
2.0400	-.007940	2.0490	-.024040	2.0570	-.038641	2.0720	-.022541
2.0860	-.015242	2.1030	.002357	2.1170	-.004944	2.1290	-.013745
2.1490	-.001946	2.1600	-.007847	2.1720	-.015148	2.1890	.006251
2.2030	-.006049	2.2230	-.013651	2.2320	-.029851	2.2430	-.040052
2.2510	-.019552	2.2570	-.009253	2.2740	.006646	2.2860	.020046
2.3090	.021545	2.3140	.008344	2.3200	-.007756	2.3260	-.019557
2.3460	-.022458	2.3600	-.029759	2.3710	-.013659	2.3830	-.001960
2.4060	-.009201	2.4290	.006937	2.4460	.018636	2.4510	.024536
2.4690	.034735	2.4830	.023034	2.4890	.008434	2.4940	-.009167
2.5090	-.006267	2.5200	-.001868	2.5340	-.017969	2.5570	-.013570
2.5710	-.001871	2.5830	.011328	2.5940	.020128	2.6030	-.000373
2.6140	.009927	2.6340	.011325	2.6460	.005525	2.6600	.011424
2.6710	-.004777	2.6860	-.017878	2.7030	-.007679	2.7200	-.019380

TABLE I (follows)

t	a/g	t	a/g	t	a/g	t	a/g
2.7430	-.022281	2.7600	-.016382	2.7740	-.000283	2.7910	-.014984
2.8000	-.032484	2.8170	-.016385	2.8260	-.001786	2.8460	.007013
2.8630	.001212	2.8710	-.001688	2.8890	.010011	2.9000	-.003190
2.9060	-.017790	2.9200	-.026591	2.9260	-.006091	2.9510	-.007593
2.9690	-.001694	2.9860	-.010495	2.9970	-.022195	3.0110	-.008996
3.0340	-.011897	3.0630	.001301	3.0770	-.000200	3.0910	.015899
3.1090	.032098	3.1170	.014498	3.1230	-.000202	3.1370	-.013403
3.1510	-.019204	3.1710	-.007505	3.1830	.008594	3.1910	-.000106
3.2060	-.007507	3.2230	.007192	3.2400	.020391	3.2540	.020390
3.2630	.004290	3.2740	-.010411	3.2900	-.026511	3.2940	-.020612
3.3060	-.023512	3.3260	-.016213	3.3400	-.026514	3.3570	-.033815
3.3630	-.020015	3.3720	-.004516	3.3950	-.001517	3.4060	.007282
3.4200	.021881	3.4400	.011680	3.4520	.007280	3.4810	.013178
3.5030	.018977	3.5260	.017576	3.5520	.035174	3.5610	.051274
3.5690	.067373	3.5780	.084973	3.5870	.104072	3.5890	.121572
3.6070	.134771	3.6180	.145071	3.6440	.143569	3.6500	.127469
3.6610	.109869	3.6670	.089368	3.6780	.083568	3.6900	.105567
3.7040	.121066	3.7210	.109965	3.7330	.076265	3.7360	.060165
3.7380	.041065	3.7530	.027864	3.7730	.024963	3.7840	.004462
3.7930	-.018936	3.8040	-.052639	3.8100	-.073139	3.8190	-.101040
3.8210	-.127440	3.8330	-.142040	3.8470	-.155241	3.8620	-.169842
3.8700	-.190342	3.8870	-.218243	3.8900	-.238743	3.8990	-.253344
3.9070	-.269444	3.9160	-.285645	3.9220	-.298745	3.9360	-.311946
3.9450	-.314846	3.9620	-.301747	3.9700	-.285548	3.9760	-.266548
3.9820	-.251848	3.9850	-.234248	3.9910	-.216749	3.9990	-.204949
4.0080	-.191750	4.0130	-.177150	4.0250	-.134650	4.0310	-.095051
4.0340	-.065451	4.0390	-.004251	4.0420	.038249	4.0480	.071948
4.0510	.096548	4.0620	.148148	4.0710	.183347	4.0790	.215547
4.0910	.205746	4.1020	.199445	4.1080	.193545	4.1190	.209745
4.1250	.225945	4.1400	.227344	4.1540	.230243	4.1650	.241942
4.1670	.256042	4.1710	.274242	4.1800	.293842	4.1880	.303541
4.1970	.285941	4.2030	.268341	4.2110	.246340	4.2200	.230240
4.2230	.212940	4.2280	.183339	4.2370	.161439	4.2430	.129139
4.2540	.107236	4.2710	.098437	4.2800	.104237	4.3030	.110136
4.3260	.089034	4.3370	.067634	4.3460	.051533	4.3540	.033933
4.3680	.014932	4.3770	-.002668	4.3880	-.030469	4.3940	-.053969
4.4000	-.086169	4.4030	-.108169	4.4080	-.133070	4.4230	-.146170
4.4280	-.118571	4.4400	-.083171	4.4430	-.062671	4.4450	-.042171
4.4570	-.024572	4.4740	-.021673	4.4830	-.039273	4.4880	-.053874
4.5000	-.065574	4.5200	-.077275	4.5340	-.093476	4.5480	-.099277
4.5650	-.088977	4.5740	-.071478	4.5880	-.055278	4.6000	-.043579
4.6140	-.037680	4.6280	-.062580	4.6370	-.088981	4.6420	-.112481
4.6510	-.143182	4.6590	-.178382	4.6620	-.197382	4.6650	-.226682
4.6680	-.254582	4.6710	-.280883	4.6790	-.323583	4.6910	-.250183
4.6940	-.190084	4.6970	-.146084	4.6990	-.119784	4.7050	-.084584
4.7080	-.049384	4.7140	-.018685	4.7220	.007715	4.7280	.050215
4.7340	.081014	4.7450	.116214	4.7480	.135214	4.7510	.156714
4.7650	.182113	4.7850	.179212	4.7940	.195312	4.8160	.198211
4.8310	.201410	4.8480	.218809	4.8590	.199709	4.8770	.174808
4.8820	.152807	4.8970	.130907	4.9050	.111606	4.9140	.089906
4.9240	.068605	4.9310	.002005	4.9340	-.024395	4.9430	-.055195
4.9510	-.081596	4.9630	-.118196	4.9710	-.150397	4.9800	-.186997
4.9880	-.216697	4.9940	-.247098	5.0000	-.258798	5.0140	-.235399
5.0170	-.213099	5.0200	-.185599	5.0230	-.151899	5.0260	-.126899
5.0290	-.094699	5.0310	-.058100	5.0340	-.022900	5.0460	.004900
5.0600	.022499	5.0660	.007899	5.0740	-.008202	5.0800	-.025802
5.1030	-.011203	5.1170	-.027304	5.1230	-.049204	5.1290	-.066804
5.1370	-.075604	5.1520	-.056605	5.1630	-.044806	5.1800	-.037506
5.2000	-.046307	5.2090	-.030208	5.2290	-.015509	5.2460	-.038909
5.2610	-.074110	5.2780	-.113711	5.2920	-.135612	5.3120	-.153212
5.3240	-.145913	5.3440	-.144414	5.3490	-.110714	5.3550	-.087214

TABLE I (follows)

t	a/g	t	a/g	t	a/g	t	a/g
5.3510	-.043515	5.3670	-.003715	5.3700	.029985	5.3750	.056385
5.3950	.090084	5.4070	.116383	5.4180	.135483	5.4330	.106182
5.4360	.076582	5.4390	.050482	5.4470	.009481	5.4530	-.027119
5.4590	-.053519	5.4700	-.076920	5.4840	-.085720	5.4930	-.050621
5.5100	-.013921	5.5130	.016779	5.5190	.051978	5.5220	.082778
5.5270	.109176	5.5330	.138478	5.5420	.166277	5.5500	.194177
5.5620	.210276	5.5850	.226375	5.5930	.242475	5.6080	.257174
5.6250	.241074	5.6340	.216173	5.6360	.198573	5.6450	.161973
5.6480	.125273	5.6540	.091572	5.6590	.071072	5.6770	.037371
5.6830	.012471	5.7020	-.015330	5.7110	-.032930	5.7250	-.054931
5.7540	-.032731	5.7400	-.103231	5.7480	-.122232	5.7600	-.141332
5.7710	-.110533	5.7740	-.087133	5.7800	-.066633	5.7860	-.046033
5.7940	-.034334	5.8170	-.034335	5.8310	-.043135	5.8460	-.059236
5.8600	-.070937	5.8910	-.084138	5.9030	-.100238	5.9140	-.114839
5.9510	-.111940	5.9540	-.109041	5.9630	-.123641	5.9740	-.133941
5.9690	-.145642	6.0110	-.151443	6.0310	-.138244	6.0460	-.123644
6.0550	-.110445	6.0630	-.092846	6.0860	-.067946	6.1000	-.038647
6.1030	-.016647	6.1110	.008253	6.1170	.025853	6.1260	.044852
6.1340	.069852	6.1430	.091752	6.1630	.107951	6.1770	.129850
6.1630	.153350	6.1940	.175349	6.2030	.195649	6.2110	.216349
6.2200	.239848	6.2290	.257348	6.2430	.260347	6.2600	.251547
6.2660	.235446	6.2710	.211946	6.2770	.188546	6.2800	.160746
6.2860	.137246	6.2910	.119645	6.2940	.091645	6.2970	.068445
6.3030	.040545	6.3090	.015645	6.3140	-.010755	6.3260	-.035656
6.3290	-.056156	6.3340	-.073656	6.3430	-.091257	6.3520	-.110357
6.3660	-.127956	6.3750	-.116158	6.3860	-.107356	6.4030	-.114659
6.4180	-.129960	6.4260	-.142560	6.4410	-.163061	6.4460	-.170261
6.4640	-.171862	6.4750	-.161562	6.4950	-.177663	6.5040	-.190863
6.5150	-.200964	6.5270	-.208464	6.5300	-.186464	6.5390	-.158564
6.5410	-.132265	6.5440	-.108765	6.5470	-.072165	6.5610	-.039965
6.5640	-.013565	6.5780	.015734	6.5840	.037734	6.5960	.050933
6.6070	.071433	6.6150	.099332	6.6270	.128632	6.6300	.152032
6.6360	.175532	6.6550	.184331	6.6700	.175530	6.6820	.169630
6.7050	.177029	6.7230	.181428	6.7450	.182627	6.7590	.165327
6.7760	.140426	6.7850	.115526	6.7960	.092025	6.8020	.074425
6.8170	.052525	6.8340	.015824	6.8400	-.010476	6.8480	-.041276
6.8600	-.001777	6.8690	-.089577	6.8740	-.120377	6.8860	-.149678
6.8950	-.176076	6.9090	-.195079	6.9260	-.180479	6.9380	-.146780
6.9470	-.115980	6.9520	-.088080	6.9580	-.064681	6.9610	-.041181
6.9670	-.019281	6.9730	.001219	6.9780	.023219	6.9840	.037918
6.9930	.051518	7.0070	.074517	7.0220	.076017	7.0300	.059917
7.0360	.048216	7.0480	.030616	7.0620	.017415	7.0770	.010115
7.0940	.017414	7.1060	.027714	7.1200	.035013	7.1430	.042312
7.1630	.020412	7.1660	.007212	7.1780	-.013289	7.1840	-.030889
7.1890	-.041189	7.2130	-.044090	7.2410	-.063091	7.2650	-.079192
7.2680	-.090893	7.3020	-.076293	7.3140	-.054294	7.3340	-.030794
7.3520	-.010295	7.3630	.005804	7.3750	.017504	7.3980	.029303
7.4240	.038102	7.4420	.023402	7.4480	.011701	7.4510	.004401
7.4540	-.008799	7.4620	-.024899	7.4710	-.043999	7.4770	-.058600
7.4830	-.073300	7.5120	-.071801	7.5350	-.055702	7.5440	-.042502
7.5550	-.020502	7.5640	-.005803	7.5850	.005697	7.5990	.021996
7.6140	.036596	7.6310	.055695	7.6570	.060094	7.6840	.057193
7.7040	.063093	7.7300	.065992	7.7560	.052891	7.7740	.045490
7.7910	.036790	7.8030	.023589	7.8230	.005989	7.8440	-.007212
7.8550	-.021513	7.8670	-.036513	7.8780	-.048213	7.8900	-.058514
7.9050	-.051114	7.9130	-.036514	7.9160	-.030615	7.9250	-.017415
7.9360	-.004215	7.9420	.001685	7.9530	-.013016	7.9740	-.023216
7.9830	-.037917	8.0030	-.032017	8.0120	-.015918	8.0190	.000182
8.0260	.013382	8.0470	.023681	8.0700	.026580	8.0870	.020780
8.0990	.013379	8.1190	.003179	8.1340	.009078	8.1480	.009078
8.1720	.006077	8.1920	.009076	8.2060	.019276	8.2240	.023775

TABLE I (follows)

t	a/g	t	a/g	t	a/g	t	a/g
8.2470	.023775	8.2670	.016374	8.2850	.007674	8.3050	.001773
8.3220	.003272	8.3340	.016472	8.3450	.028172	8.3570	.042871
8.3600	.053071	8.4090	.057470	8.4370	.061869	8.4750	.042868
8.4950	.028467	8.5150	.017966	8.5290	.006266	8.5440	-.005434
8.5580	-.017135	8.5730	-.028835	8.5900	-.040636	8.6070	-.050836
8.6300	-.056737	8.6470	-.047938	8.6710	-.042038	8.6850	-.034739
8.6910	-.024439	8.7020	-.015639	8.7310	-.005340	8.7450	.001960
8.7570	.012259	8.7650	.022459	8.7710	.035659	8.7860	.054758
8.7940	.066458	8.8060	.078158	8.8230	.068457	8.8350	.070857
8.8520	.053256	8.8640	.038656	8.8720	.021056	8.8810	.009356
8.9010	.000555	8.9190	.007854	8.9510	.000554	8.9740	-.017047
8.9940	-.033146	9.0120	-.049248	9.0410	-.063649	9.0610	-.071150
9.0840	-.054350	9.1100	-.087251	9.1190	-.072651	9.1360	-.057952
9.1540	-.044752	9.1650	-.031552	9.1770	-.016953	9.1910	-.005153
9.2120	.007946	9.2380	.013846	9.2640	.018245	9.2870	.035844
9.3040	.050544	9.3220	.062243	9.3430	.069543	9.3660	.072542
9.3780	.076942	9.4040	.066641	9.4130	.053541	9.4250	.043241
9.4340	.028040	9.4450	.013940	9.4660	-.002160	9.4780	-.009461
9.4920	-.021161	9.5130	-.029962	9.5280	-.041662	9.5420	-.053362
9.5630	-.056363	9.5800	-.053363	9.6040	-.054864	9.6360	-.038765
9.6480	-.025965	9.6630	-.013765	9.6860	.002334	9.7070	.014033
9.7530	.025833	9.7510	.030232	9.7680	.033132	9.7860	.040432
9.8070	.049231	9.8330	.043430	9.8530	.036130	9.8800	.036129
9.8970	.028629	9.9120	.018528	9.9270	.011228	9.9470	.005328
9.9650	-.003473	9.9820	-.012173	10.0000	-.013674	10.0170	-.022474

suggest to take the input of tab. 1 as reasonable (or perhaps under-estimated) input for the buildings.

3. DYNAMIC STRUCTURAL MODEL

The buildings here considered are number 1,2,3,4 of ref. [1] ; they have respectivelly 4,7,4,3 storeys.

For the reasons mentioned in the introduction each of them can be adequately represented by a rigid body supported by beam-type elements (columns). The shear panels (unfilled brick walls) are taken into account by equivalent truss type elements acting as a bracing into the main frame. Where not otherwise specified, the columns are supposed to be built in the rigid ground (or foundation), and in the upper slab. The stiffness of the columns has been calculated assuming the uncracked r.c. section and a Young modulus of 200000 kg/cm^2. The mass of the building has been assumed as uniformely distributed at floor level. The rigid body has six degrees of freedom, however allowing the large axial stiffness of the columns, only three degrees of freedom are significant: the two horizontal components of displacement and the in-plan rotation. Three principal modes of vibration are therefore significant.

Three types of response have been calculated for each building:

1) spectral response, according to the italian seismic code for second class seismic zone, i.e. each stress component is the r.m.s. of the three modal responses, calculated according to the ground acceleration spectrum

$$a_c = 0.07 \times g \quad ; \quad 0 < T \leq 0.8 \quad \text{sec}$$

$$a_c = 0.07 \times 0.862 \times T^{-\frac{2}{3}} \times g \; ; \; T > 0.8 \quad \text{sec}$$

2) same as above but with reference to the acceleration spectrum of the N-S component of the accelerogram in fig.1;

3) time history response through step by step integration of the ground accelration of fig.1. The significant results of this analysis are the maxima of each stress com-

ponents.

The following section lists the significant results for the four buildings. The numerical results have been obtained by the computer program SAP-IV.

4. DYNAMIC ANALYSIS RESULTS

Building 1 - 4 storeys apartement building at Avilla. As it has been pointed out in ref. [1] ,the presence of two important shear walls at two sides of the ground floor avoids in-plan torsion. The significant motion is only a one-directional displacement. Within the hypotheses on the stiffness listed in sec.3, the foundamental period of the building is:

$$T_o = 0.13 \text{ sec}$$

Fig.2 and 3 plot the significant results for this building. In absissa there is the fundamental period, and therefore different values of the absissa correspond to different hypotheses on the column-foundation stiffness, and/or on the mass of the building. The range $0.07 < T_o < 0.20$ sec is likely to cover the realistic range of the building.

In ordinate there is the mass maximum acceleration in g unit . The upper lines refer to the response spectrum of the two components of the ground horizontal acceleration of fig.1, for the damping ratio: $\nu = 0.05$, $\nu = 0.07$ and $\nu = 0.10$.

In the figs. 2 and 3 are also indicated three reference levels of maximum mass acceleration corresponding to:

1) the italian seismic code: a_{II};
2) the max.gr. acc. at which the building reaches the allowable stresses in the shortest columns (that are the most heavily stressed, see ref. [1]): a_a)
3) the max.gr.acc. at which the building reaches the ultimate state in the shortest columns: a_u.

The definition of a_a, a_u is made within the framework of the italian r.c. codes, see ref. [1] .The following table summarizes the variability of the ratio actual/design values within the chosen range variability of T_o and ν .

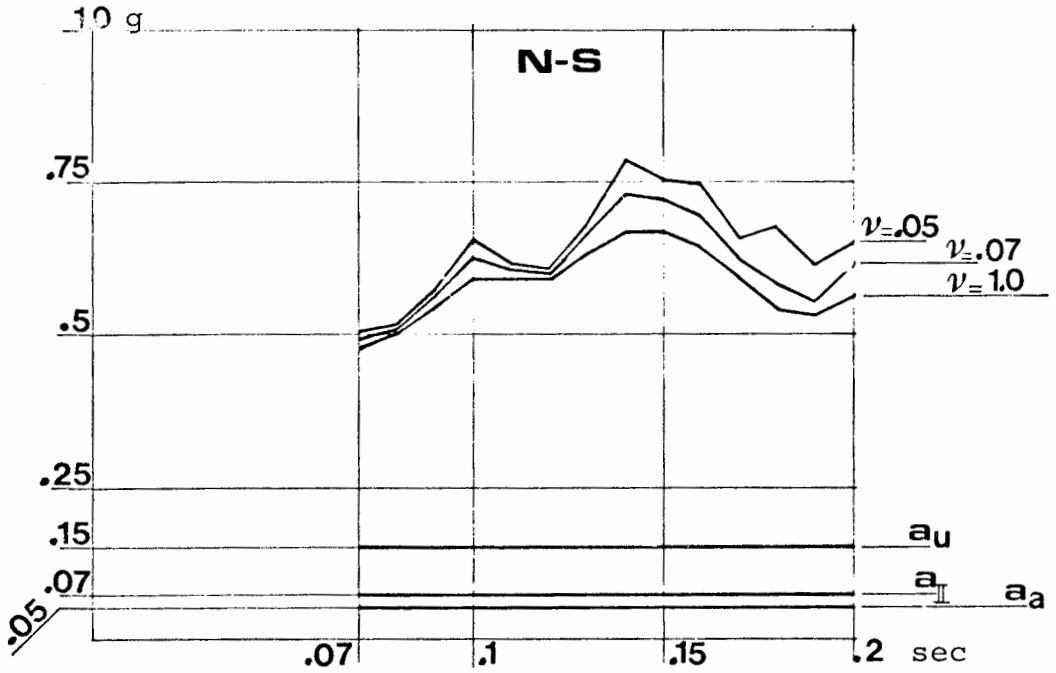
10 g
N-S
.75
.5
.25
.15
.07
.05
ν=.05
ν=.07
ν=1.0
au
a||
aa
.07
.1
.15
.2
sec

Fig.2.

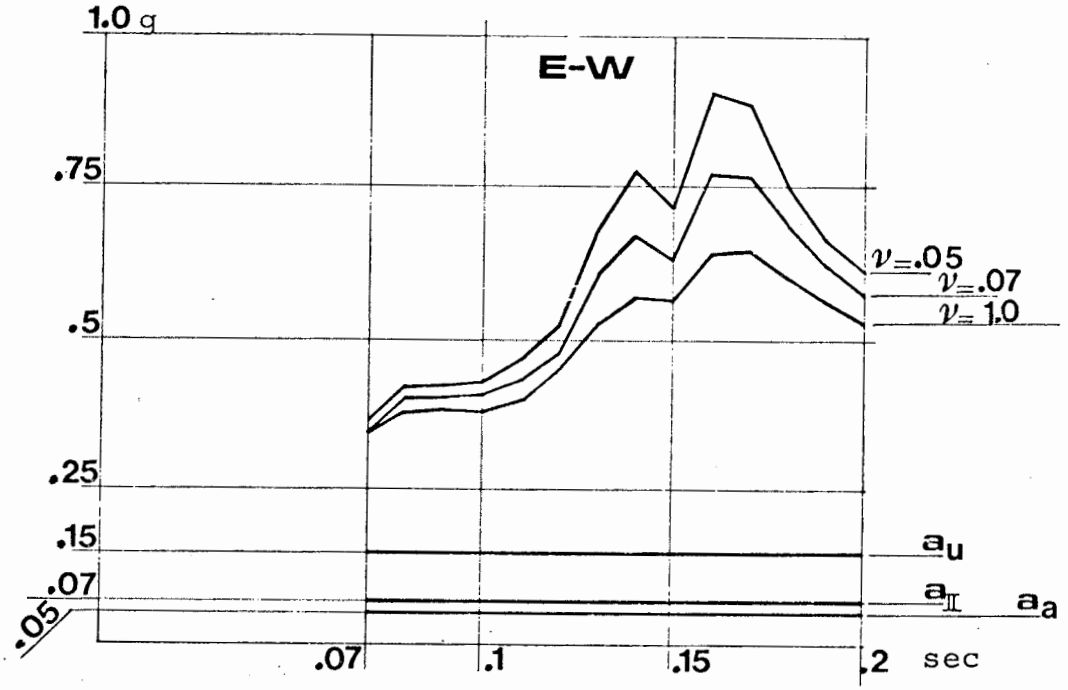
1.0 g
E-W
.75
.5
.25
.15
.07
.05
ν=.05
ν=.07
ν=1.0
au
a||
aa
.07
.1
.15
.2
sec

Fig.3.

TABLE II

		a_c/a_{II}	a_c/a_u	a_c/a_a
N-S	max	11.26	5.25	15.76
	min	6.84	3.19	9.58
E-W	max	12.99	6.06	18.19
	min	5.04	2.35	7.06

Building 2 - 7 storeys apartment building at Maiano. The following calculated values for each column are taken as significant:

T_a = shear force in a column for which allowable stresses are reached (unit:tons)
T_n = idem: ulitmate stress
T_{II} = idem: adopting the italian seismic codes
T'_c = max shear forces in a column obtained through step by step calculation
T''_c = idem at T'_c but with one half of the stiffness in all the columns
T'''_c = idem as T'_c but obtained with the response spectrum of the N-S component of the accelerograms shown in fig.1.

The following table summarises the result (max,min are taken among all the columns of ground floor)

TABLE III

	T'_c/T_{II}	T'_c/T_a	T'_c/T_u
max	30.530/2215= = 15.5	30530/3140= =9.7	30530/4900= =6.7
min	18.370/1477 =12.4	18.370/2920 =6.3	18.370/4750 =4.0

$T'_c/T''_c = 30.530/20.260 = 1.5$

$T'_c/T''_c = 30.530/24.960 = 1.2$

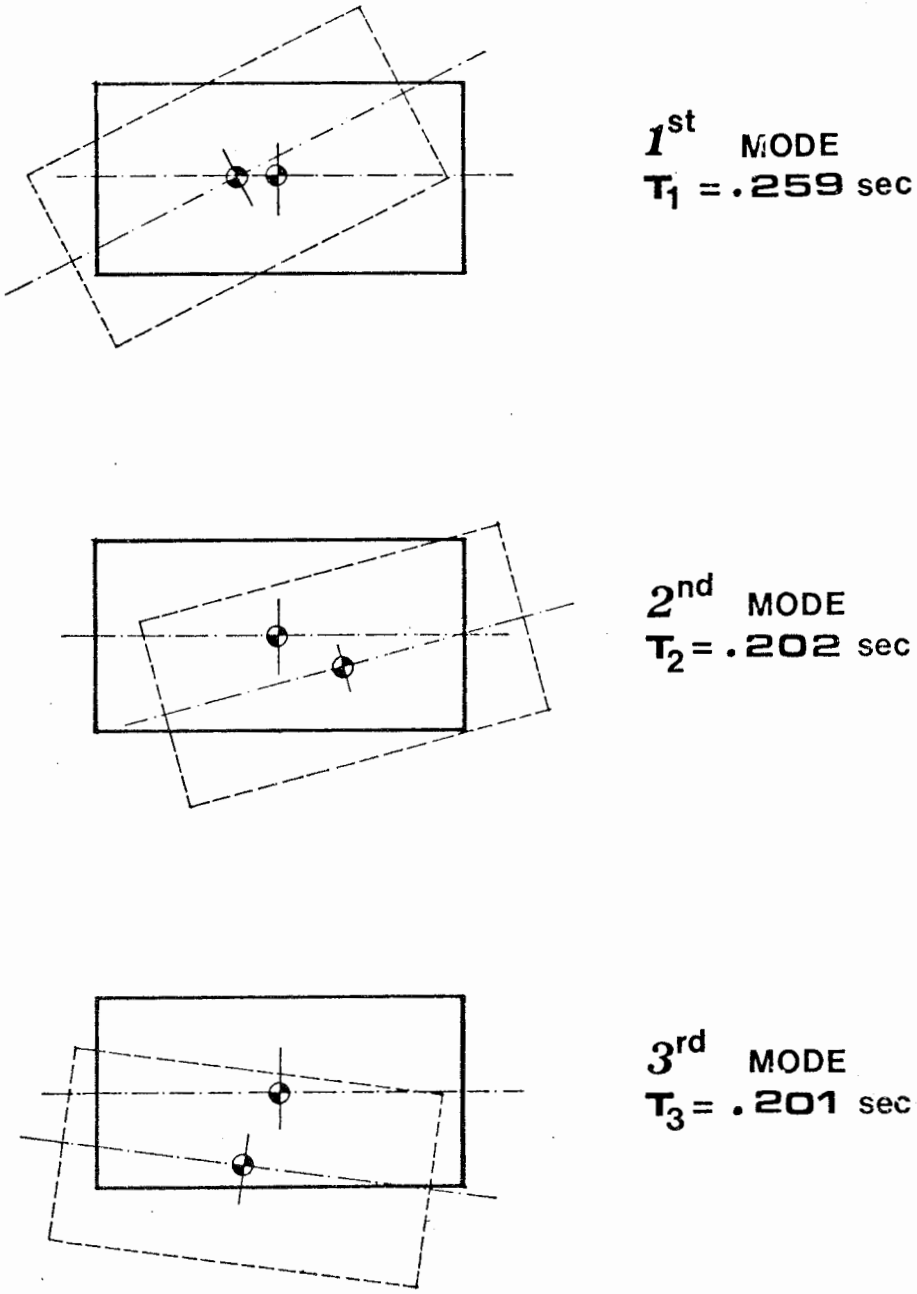

Fig.4. Building 3. Most significant principal modes.

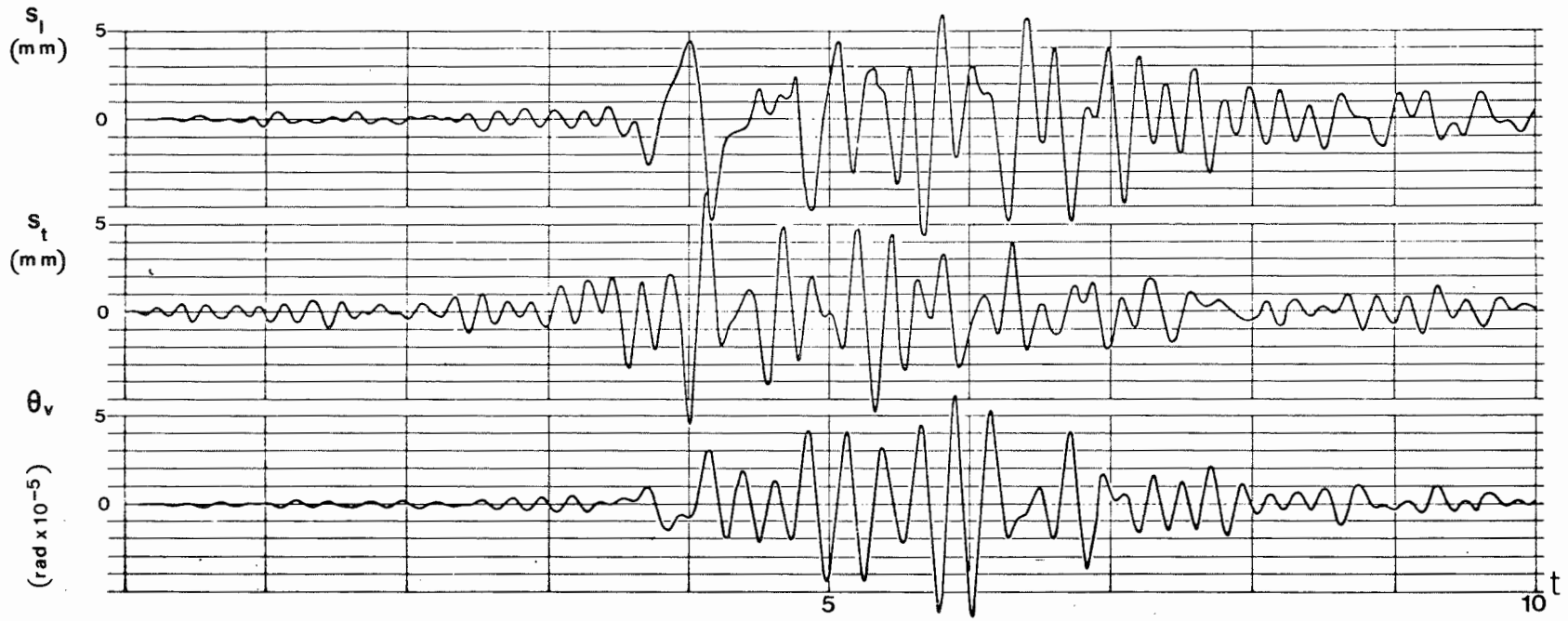

Fig.5. Building 3. Longitudinal (s_l) and trasversal(s_t) displacement of the center of mass of the floors and rotation around the vertical.

Building 3 - 3 storeys building at Artegna. As it is pointed out in ref. [1] and in other research-work [4] the main feature on this building is to have a highly dissymmetric plan due to the presence of unfilled walls and stair boxes. The stiffning effect of these walls is highly uncertain, and therefore the input data depend mostly on the designers judgement. In obtaining the following results, the stiffness of the walls was computed assuming a thickness equal to 1/10 of the total wall thickness and assuming a shear modulus G=43,000 Kg cm^{-2}.

The reader can compare the results here obtained to the results of other research (ref. [4]).

Such a comparison can be a useful example to see how the judgement on safety in a building may be influenced by the design hypotheses and structural scheme assumptions.

In fig.4 the three foundamental modes of vibration of the building are plotted. The following table summarizes the results obtained. Symbols have the same meaning as in building 2.

TABLE IV

	T'_c/T_{II}	T'_c/T_a	T'_c/T_a
max	7739/541= =14.3	7739/980= =7.9	7739/590= =13.1
min	4025/353= =11.4	4025/980= =4.1	4025/590= =6.8

$T'_c/T''_c = 7739/5126 = 1.5$

Building 4 - 3 storeys apartement in Gemona. The building was under construction at the moment of the shock and only an unfilled wall at one side was cast.

Three different hypotheses on the stiffness of the shear wall have been made:

1) infinite stiffness
2) thickness of the wall 1/10 of the total wall thickness and shear modulus G=43,000 Kg cm^{-2}

3) thickness of the wall 1/10 of the total wall thickness and shear modulus G=21,500 kg cm^{-2}.

The following table summarises the main results

Stiffness hyp.		1)	2)	3)
T_c/T_{II}	max	9788/809= =12.1	9581/611= =15.6	9588/617= =15.6
	min	5625/450= =12.5	6661/407= =16.3	6648/409 =16.3

5. CONCLUDING REMARKS

The above results may suggest the following considerations:

1) The stresses derived from a linear dynamic analysis using a step-by-step integration of the earthquake records are much higher than the admissible design stress values. Useless to say that this can be a quite trivial conclusion.

2) The ratios between linear response and seismic design stress are quite constant for all the buildings: they range from 12 to 15. It is interesting to note that the above figures are quite similar to the ductility ratios mentioned in the introduction.

3) The ratios between linear dynamic response and limit state design stresses are still rather large (ranging from 5 to 8). This can suggest that even designing with reference to limit states, ductility must be taken into account.

4) Torsional effect are very evident for some buildings as static analysis has predicted. This can be clearly seen from figs 4 and 5, referred to building 3. In addition for this particular building displacements and rotation have quite the same dominant frequency. This can produce an unpredictable summing up of effects on shear forces in the external columns.

6. ACKNOWLEDGEMENTS

Thanks are due to P. Setti for cooperation in computation. C.N.R. has given the financial support.

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NOTES ON OLD MASONRY BULDINGS STRUCTURAL BEHAVIOUR
PROPOSAL OF INVESTIGATION

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Summary

The intervention intends to explain the study on masonry buildings behaviour under recents seismic events.

The documentation we have produced intends to give a contribution for the characterization of particularly interesting constructive and structural typologies that may be individualizated in static order progressive evolution.

This study wants also to give indications on possible previous strenghtening of monumental buildings, so as a larger discussion could start about our historical and artistic patrimony protection.

The problem of conservation of historical centres and monumental buildings has reproposed as topical question, but still it didn't find a good solution for the shortage of data and indications about old masonry buildings behaviour under earthquake.

The application of dynamic analysis for masonry structures, for now, can't rely on well determinate hypotesis and it finds these real difficulties:

- extreme trouble to isolate from structural context simple schemes to be investigated with dynamic analysis;
- problem of a structural relief sticking to realities and of determination of strength of materials;
- restrected elastic field and non homogeneous material;
- arising of failure mechanisms that quickly prevent masonry to dissipate energy with an elastic behaviour.

While it is possible to suppose linear structural schemes for steel and reinforced concrete structures, for example "frame", "shear" or "bending type", this way is impraticable for masonry buildings: it is hard in these to isolate the resistant units, their restraints, and the possible interactions.

These interactions have a particular influence because in the various parts of a same building, both the amplitude and the vibration period are varying in discordance; it occures because the vibrations are function of many parameters: structures height, floor distance, mesh sizes, continuity and importance of masses, number and wideness of the openings, walls floors and vaults structural characteristics.

In solidly jointed structures the cohesion might nullify the vibration discordance of same elements; this doesn't happen in degraded and cracked structures, in which the discordant oscillations of elements

Fig. 1 Duomo of Gemona - Cross section

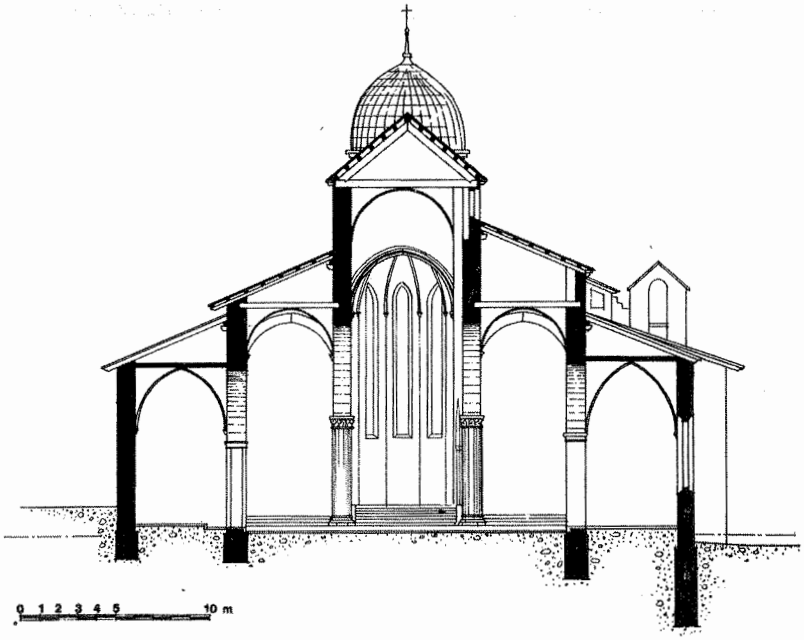


Fig. 2 Duomo of Gemona - Lateral view

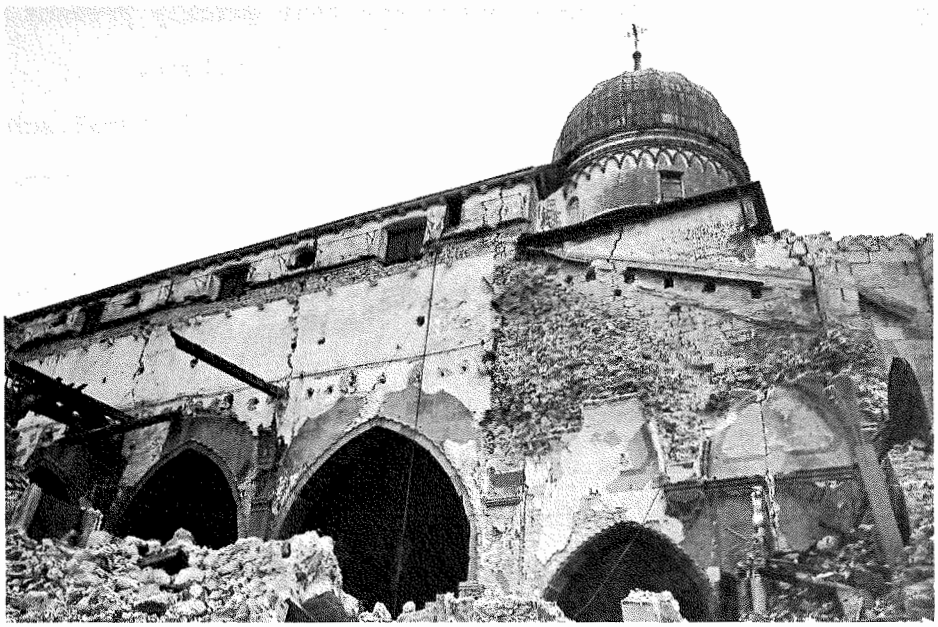


Fig. 1 - 2

The complexity of the scheme is due to the different height where are placed the masses of the naves longitudinal panels, to the different restraints at the ends, to uncertain valuation of importance of crossing connections, to presence of overhanging masses, for example the dome, that can have a various interaction with the other parts of the building.

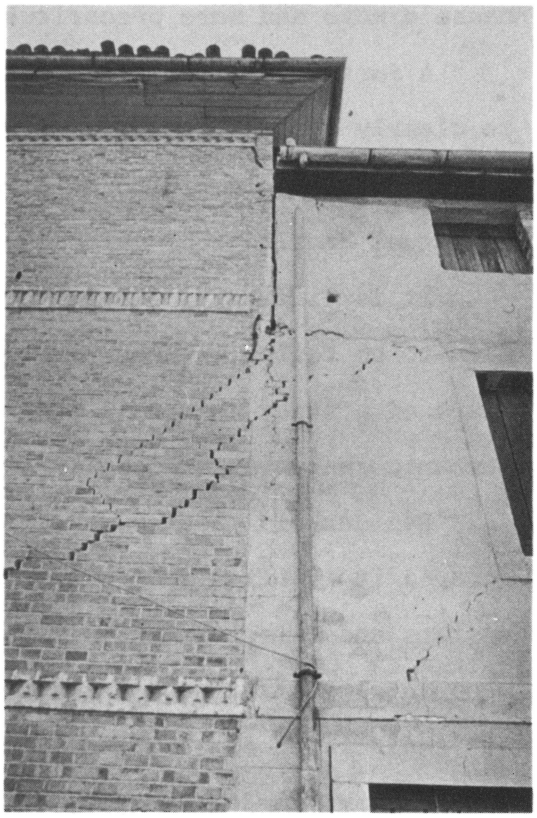


Fig. 3 - 4 - House at Forgaria.

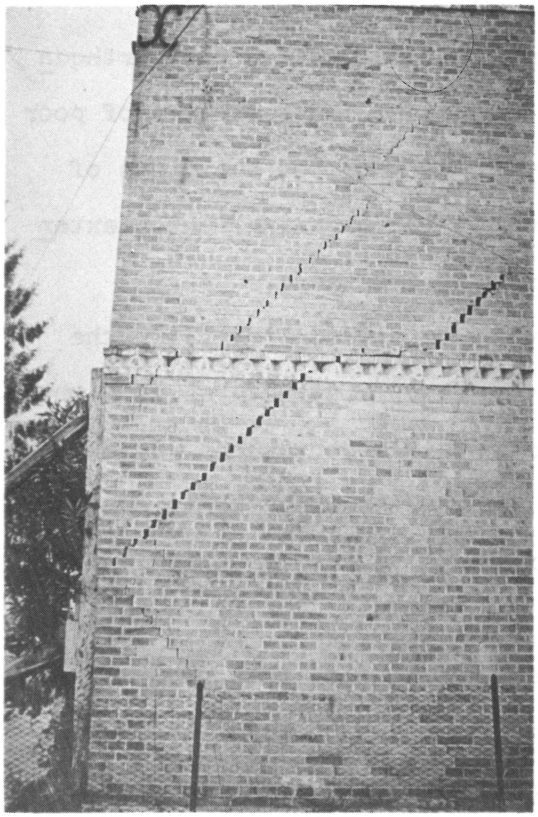


Fig. 3 - 4

Examples of insufficient connection in a masonry building; the superior part has become indipendent from the lower.

It is possible see a considerable horizontal translation.

cause a more and more precarious stability: (fig. 1-2)

A further difficulty consists in a structural relief that be able to clearly describe the geometry of the buildings, for instance, because of the possible hiding of structural elements and that be based on an actual knowledge of material properties.

It is difficult, indeed to obtain inalterated and significant samples to be tested in laboratory, so that it would be desiderable the start of a study to obtain equipements for determination "in situ" of mechanic characteristics.

Besides old masonry materials is not homogeneous: in the earthquake area it is often made with scarcely consistent conglomerates of poor mortars and limestone units: the difference between the buildings of monumental importance and the ordinaries, is the organization of external superficies sometimes well constructed in freestone.

The arising of damages, also under low stresses, limits, on the other hand, the elastic field of answer in masonry structures as a whole. The appearance and progression of cracks entail a continuous variation of internal stress and make changes in the comprehensive behaviour that sometimes does not cause the complete collapse because masonry is able to find subsequent stability schemes.

In other words, until structures are monolithic, they reply elastically to ground vibrations and dissipate the kinetic energy as deformation work: with the increasing of damages they stray from an elastic behaviour and take up more and more less energy.

Over this limit some structures became labile and behave like kinematic motions: at this point the horizontal ground vibrations are not trasmitted to the upper masses: (fig. 3-4)

We have another example of such behaviour in the transformation of pillars into connecting rods.



Fig. 5 - Duomo of Gemona - Internal view.

Fig. 5- 6

The pillars connecting rod behaviour limited the transmission of the ground oscillations to the upper masses.

The considerable cross section of the pillars though subject to a high combined compressive and bending stress, still holds the loads resultant.



Fig. 6 Duomo of Gemona - Particular of column base.



Fig. 7 - Loggia funeraria at Gemona - Frontal view.

Fig. 8 - Loggia funeraria at Gemona - Plan

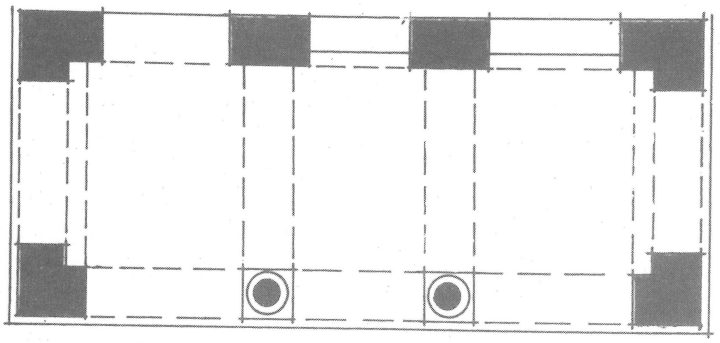






Fig. 9 - Loggia funeraria at Gemona - Particular.

Fig. 7 - 8 - 9

A kinematic motion took place in the anterior part of the loggia by forming a hinge at the first arch springer (A - B), and a sliding plane at the other end (C).

The horizontal stress has been mainly absorbed by the very stiff masonry walls, on the back.

The difference in behaviour between r.c. frame buildings and masonry pillars buildings is due to the importance of displacements compared with sizes of resistent elements, with consequent static effects.

This might justify the good behaviour of buildings on masonry arcades. (Fig. 5-6-7-8-9-10)

For the difficulties and indeterminations we meet in dynamic analysis for masonry buildings, we think that these investigations are very interesting:

- search of data about behaviour under fatigue of masonry elements;
- study on building comprehensive behaviour as static scheme and resistent mechanism development under earthquake.

To justify the first point, we presume that masonry structures have been damaged not only by the strongest dynamic stresses, but, at the same time with chemical-physical properties decline, they held a progressive decay also for low iterating oscillations.

It would be interesting, for instance, to test in laboratory old masonry units under dynamic stresses that may vary both for cycles and oscillation amplitude, and to verify the passage from en elastic period to following stages till to failure.

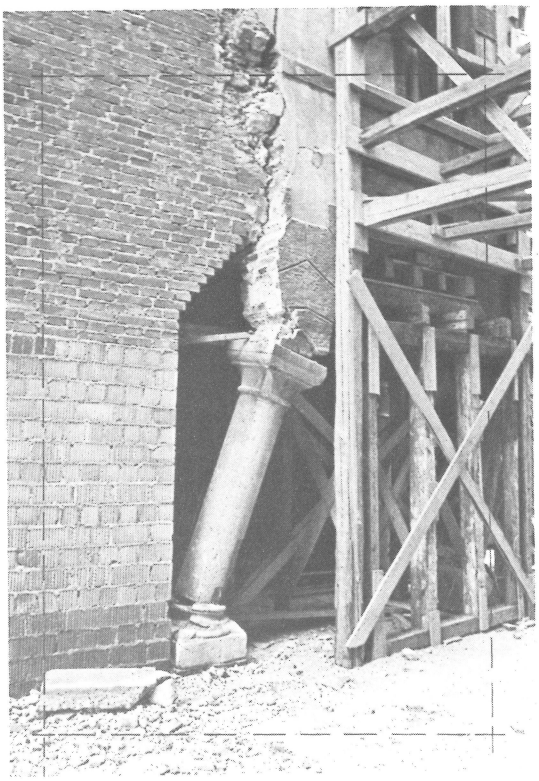
About the second point it needs to isolate from the comprehensive behaviour the structural schemes that follow inside. This study might be developed on the grounds of existing documents examination, in order to caracterize the progressive evolution of static scheme.

For example we have:

- structure that activate a comprehensive behaviour: (fig.11-12-13)
- structures that get efficiency through the collaboration of other elements: (fig. 14-15-16)
- structures that may cause the whole or a part building collapse: (fig. 17-18-)

Fig. 10 - Gemona - via Bini.

Limit situation in connecting rod behaviour of pillars.





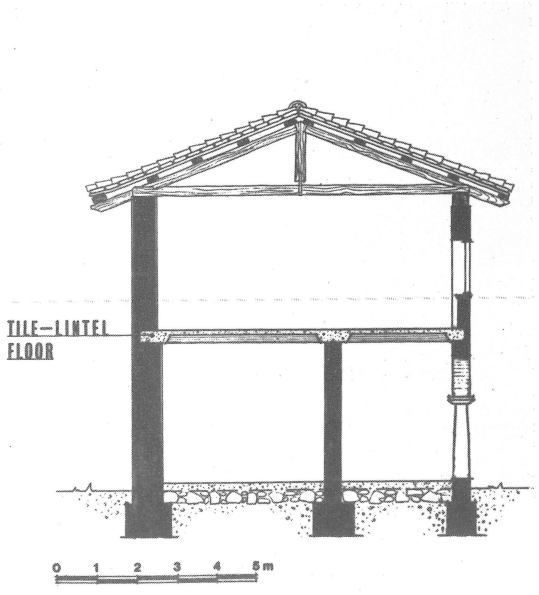


Fig. 11 - 12 - St. Antonio's convent - Internal view and cross section.

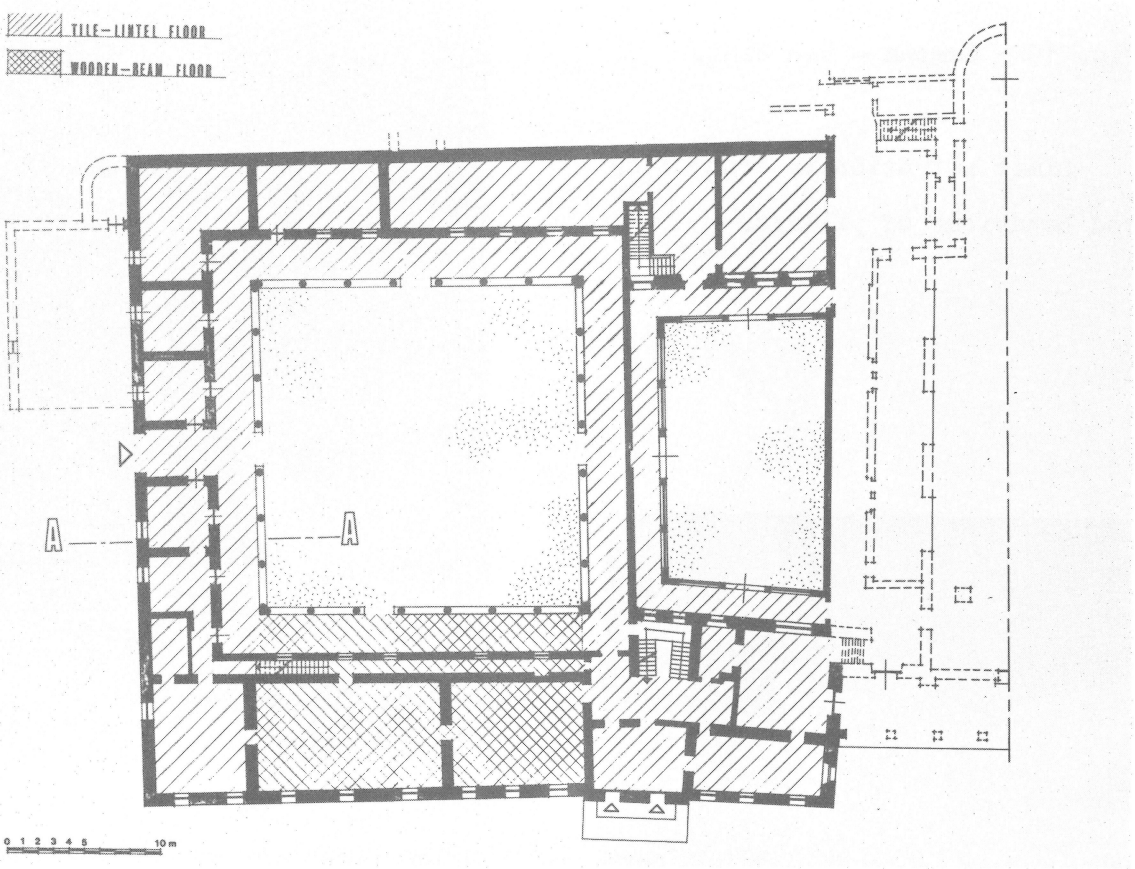


Fig. 13 - St. Antonio's convent - Plan.

Fig. 11 - 12 - 13

Some years ago the building has been partially rebuilt.

The introduction of r.c. first floor has activated the collaboration of the whole building under earthquake; the crack at the floor bearing shows its stifness.

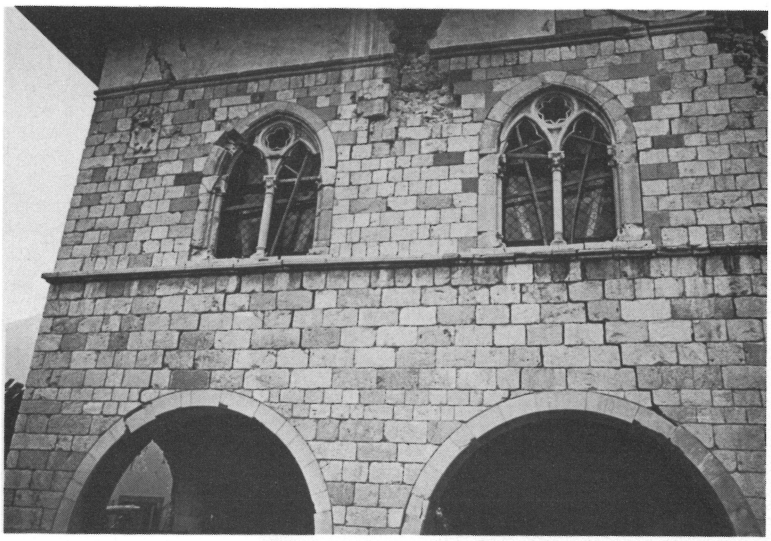
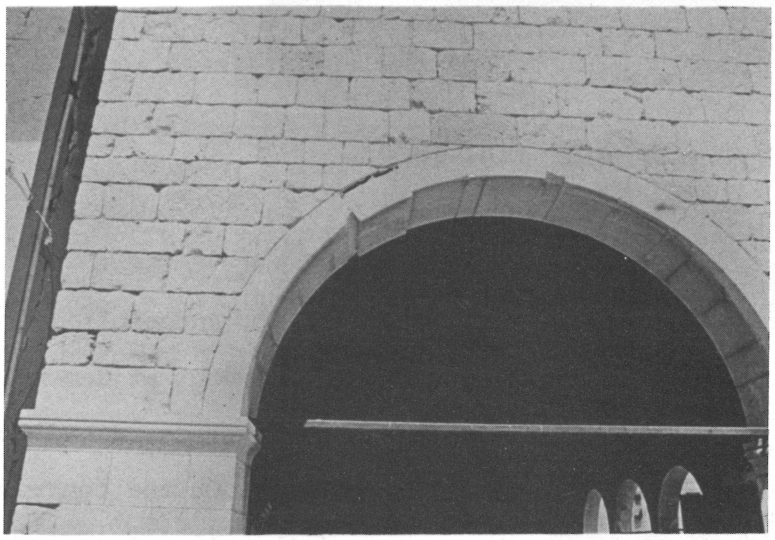


Fig. 14 - 15 - 16

Municipio of Venzone



Good behaviour of a very slender arch due to an inherent capacity to buckle without losing carrying capacity.

The arches that were well restrained at the springes, haven't been damaged when upper wall stifness contributed to centre the loads funicular.

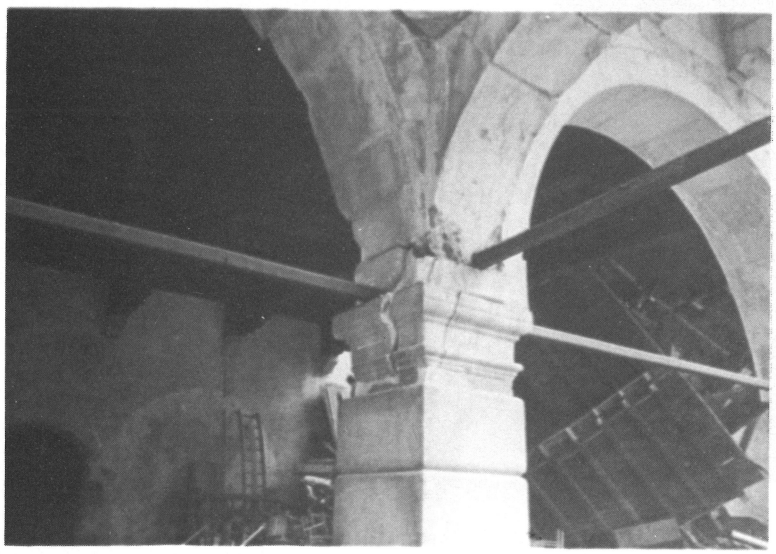




Fig. 17



Fig. 18

Fig. 17 - 18 - St. Antonio's convent at Gemona.

The corner trusses caused some walls collapse. On the contrary, the cross not thrusting trusses have been a restraint for walls top end.







Fig. 19 - House at Forgaria.

Fig. 20 - Parish church of Forgaria.

Fig. 21 - St. Maria's church at Gemona.

Fig. 19 - 20 - 21

Examples of scarcely restrained structures. The damaged walls were free at the top end.

- Less restrained structures that can't find a second equilibrium configuration: (fig. 19-20-21).

At conclusion we pose the question if it is possible an investigation to find criteriums to follow the static scheme evolution of a structure under seismic events.

This study may give an evaluation of building security degree and so useful indications for previous strenghtening of monumental buildings with the acknowledgement of apter elements to oppose seismic moviments.

INDICE DEL VOLUME XVIII

AMATO A., BARNABA P.F., FINETTI I., GROPPI G., MARTINIS B., MUZZIN A.: *Geodynamic Outline and Seismicity of Friuli-Venezia Giulia Region* . . . . pag. 217

AMBRASEYS N.N.: *Crustal Deformation after the Friuli Earthquake, 1976* . . . " 257

ARIC K., GIESE P., MILLER H., MORELLI C., NICOLIC R.: *Crustal Structure and Seismicity of Northern Italy* . . . . . " 273

ARIC K., GUTDEUTSCH R.: *Remarks about the Structure of the Earth's Crust Near the Friuli Area* . . . . . " 279

ARSOVSKI M., MIHAILOV V., CVIJANOVIC D., KUK K.: *The Relation between Seismological and Neotectonic Characteristics of Friuli and Neighbouring Areas* . . " 285

BARTOLE R., BERNARDIS G., GIORGETTI F., RUSSI M., NIETO D.: *Earthquake Catalogue of Friuli-Venezia Giulia Region (from Roman Epoch to 1976)* . . . " 299

BASILI M., BERARDI R., MUZZI F., POLINARI S., TINELLI G., ZANETTI L.: *Strong Motion Records Relative to the Friuli Earthquake Accelerogram Processing and Analysis* . . . . . " 335

BASILI M., IACCARINO R., TENAGLIA G.: *Macroseismic Aspects and Seismological and Statistical Considerations* . . . . . " 349

BAYÜLKE N.: *Lateral Force Analysis of a Simple r.c. Structure Damaged in May 6 1976 Friuli Earthquake* . . . . . " 1.029

BEMMELEN (van) R.W.: *Note on the Seismicity of North Eastern Italy (Friuli Area)* . " 357

BENEDETTI D., VITIELLO E.: *Ductility of r.c. Buildings during the Friuli Earthquake* " 1.037

BENEDETTI D., VITIELLO E.: *Implicit Design Loads of Friuli r.c. Damaged Buildings* " 1.052

BERNARDIS G., GIORGETTI F., NIETO D., RUSSI M., SUHADOLC P.: *Seismicity of Friuli-Venezia Giulia Region. Space Epicenter Distribution from Roman Epoch up. to 1976. Maximum Observed intensities of the Last Century* . . . . " 365

BO G.M., DE STEFANO S., MANCUSO P.L., MANZONE G.: *Behaviour of Some Simple Structures during the Friuli Earthquake* . . . . . " 1.082

BONAFEDE M., BOSCHI E., MULARGIA F.: *Stress Corrosion Theory of Crack Propagation Applied to the Earthquake Mechanisms. A Possible Basis to Explain Two or More Large Seismic Shocks in a Geologically Short Time Interval* . . . . " 377

BOSTJANCIC J., SHEPPARD P., TERCELJ S., TURNSEK V.: *Use of a Modelling Approach in the Analysis of the Effects of Repairs to Earthquake-Damaged Stone-Masonry Buildings* . . . . . " 1.091

BRAGA F., PARDUCCI A.: *Elastic-Plastic Degrading Oscillators Behaviour under Earthquakes - Preliminary Results* . . . . . " 1.117

BRANCALEONI F., PETRANGELI M.P.: *Dynamic Investigation of the Effects of Non-structural Elements on a r.c. Building in Friuli* . . . . . " 1.134

BRISEGHELLA L.: *Damages to Bridges and Roads in the 1976 Friuli Earthquake* . . " 1.150

BRISEGHELLA L., CAPELLARI L., DALL'AGLIO B., D'EREDITÀ R., GORI R., SIMONI L., TURRINI G., ZAUPA F.: *Earthquake in Friuli (Italy), 1976. Damage to Historical Monuments and other Buildings of Artistic Interest* . . . . . " 1.203

CAPUTO M.: *The Area of the Fault, the Dislocation, the Stress Drop and the Seismic Moment of the Friuli Earthquake of May 6th, 1976* . . . . . " 405

CARDAMONE P., CAVALLIN A., MARINO C.M.: *Linears Study in Friuli by Means of Landsat 2 Images* . . . . . " 411

CARTAPATI E., CHERUBINI A.: *Recently Constructed Masonry Buildings* . . . " 1.453

CHIARUTTINI C., ZADRO M.: *Horizontal Pendulum Observations at Trieste* . . " 441

COLAUTTI D., FINETTI I., NIETO D., PUPIS C., RUSSI M., SLEJKO D., SUHADOLC P.: *Epicenter Distribution and Analysis of 1976 Earthquakes and Aftershocks of Friuli* . . . . . " 457

COLUCCI P., MICHELATO A.: *An approach to the study of the "Marrubbio" phenomenon* . . . . . " 3

CONSOLE R.: *Focal Mechanisms of some Friuli Earthquakes, 1976* . . . . " 549

CORTECCI G.: *Tracciamento isotopico dell'inquinamento dell'aria e delle acque* . . " 11

COSENTINO P.: *Difficulties and related criticism in applying the Gutenberg & Richter relation to the seismic regions in statistical seismology* . . . . . pag. 79

COSENTINO P., LUZIO P., RIGGIO A.: *Geomagnetic surveys for siting a magnetic observatory in Western Sicily* . . . . . " 143

EBBLIN C.: *Orientation of Stresses and Strains in the Piedmont Area of Eastern Friuli, NE Italy* . . . . . " 559

EMSC (European Mediterranean Seismological Centre) WORKING GROUP: *Revised Hypocenters and Magnitude Determinations of Major Friuli Shocks, 1976* . . " 581

FAORO G.F., FERRAZZA G.: *Report on Damages and Provisions Attaining to Earthquake Concerning the Electrical System* . . . . . " 1.472

FINETTI I.: *Mediterranean Ridge: a young submerged chain associated with the Hellenic Arc* . . . . . " 31

FINETTI I., FRINZI U., SCOTTI A.: *Preliminary Spectral Analysis of Aftershocks of the 1976 Friuli Earthquakes* . . . . . " 587

FINETTI I., GIORGETTI F., HAESSLER H., HOANG T.P., SLEJKO D., WITTLINGER G.: *Time-Space Epicenter and Hypocenter Distribution and Focal Mechanism of 1976 Friuli Earthquakes* . . . . . " 637

FRASSETTO R.: *Altimetria del centro storico di Venezia - Implicazioni urbanistiche di difesa dall'acqua alta e dalla subsidenza* . . . . . " 151

GASPARINI G.: *Space and Time Distribution of 1976 Friuli Earthquakes* . . . " 657

GEBRANDE H., MILLER H., SCHMEDES E.: *An Attempt of Focal Depth Determinations for Friuli Aftershocks by Means of Lateral Arrivals Using Realistic Models of Crustal Structure* . . . . . " 677

GIORGETTI E.: *Report on the Hydrogeological Survey carried out by the Government Special Delegation for Friuli* . . . . . " 689

GIORGETTI F.: *An Example of Microseismic Zoning in the Madonna di Buia Area* . " 695

GIORGETTI F.: *Isoseismal Map of the May 6, 1976 Friuli Earthquake* . . . . " 707

GIORGETTI F., SIGNANINI P.: *Nuovi contributi al problema delle relazioni tra parametri geofisici ed idraulici nello studio degli acquiferi alluvionali* . . . . " 24

GIORGETTI F., SIGNANINI P.: *Nota sull'uso della resistività differenziale longitudinale nella risoluzione di problemi connessi con le cavità carsiche* . . . . . " 165

GIUFFRÈ A., MENEGOTTO M., PINTO P.E.: *Collapse of a Non-seismic Resistant r.c. Building during the Friuli Earthquake: ASTRO Building in Maiano* . . . . " 1.496

GLAUSER E.C., HEIMGARTNER E., SÄGESSER R.: *Assesment and Interpretation of Building Damage* . . . . . " 1.506

GRAF R., MAYER ROSA D.: *Macroseismic Investigations and Maps for the Friuli Earthquake of May 6, 1976* . . . . . " 715

HAKUNO MOTOHIKO: *Topographical Effect on the Distribution of Earthquake Damage* . . . . . " 723

HEIMGARTNER E.: *Behaviour of Damaged Buildings under Strong Earthquakes* . . " 1.540

IZUMI M.: *Damage in Friuli Earthquake 1976 - Buildings in Osoppo* . . . . " 1.549

KARNIK V., PROCHÁZKOVÁ D., RUPRECHTOVÁ L., SCHENKOVA Z., DUDEK A., DRIMMEL J., SCHMEDES E., LEYDECKER G., ROTHÉ J.P., GUTERCH B., LEWANDOWSKA H., MAYER ROSA D., CVIJANOVIC D.: *Macroseismic Effects of the Friuli Earthquake of May 6, 1976 in Austria, Czechoslovakia, F.R.G., France, Poland and Yugoslavia* . . . . . " 729

LIVIERATOS E., ZADRO M.: *Analysis of data from two sea-tidal stations of the Aegean Sea* . . . . . " 172

LO BASCIO A., NAPPI G., SCARPA R.: *Seismicity of Etna during November 1973* . " 92

LODDO M.: *The gravitational attraction of a right polygonal prism* . . . . " 184

MAGRI G., MITTEMPERGHER M.: *Research on the Relationships between Surface Geology and Seismically Active Tectonic Structures* . . . . . " 739

MAISTRELLO B., MARCELLINI A., PETRINI P., RAMPOLDI R.: *Response Spectra Analysis for Different Sites in Friuli* . . . . . " 745

MARTINIS B.: *The Friuli Earthquake, May 6, 1976: Geology* . . . . . " 755

MAUGERI M.: *Geotechnical Aspects on the Recent Friuli Earthquake* . . . . " 809

MAYER ROSA D., PAVONI N., GRAF R.: *Aftershocks Statistics and Fault-Plane Solutions on Friuli Earthquakes 1975 and 1976* . . . . . pag. 827

MIHAILOV V.: *Strong-Motion Records of the May 6 and September 15, 1976 Friuli Earthquakes Obtained by the Yugoslav Strong-Motion Instrument Network* . . " 835

MITAL G.S., BHALLA M.S., SAXENA R.S.: *Application of paleomagnetic studies in estimating the age of Deccan Traps from Rajkot, Chotila and Wankaner areas of Kathiawar peninsula, India* . . . . . " 98

MOSETTI F.: *Prof. Mario Picotti (1887-1976)* . . . . . " 138

OLSZAK W.: *On the Inelastic Behaviour of Structures During Earthquakes as Exemplified by the Friuli catastrophe 1976* . . . . . " 1.563

PALMIERI S., FINIZIO C., COZZI R.: *The contribution of meteorology to the study and prediction of high tides in the Adriatic* . . . . . " 191

PAPASTAMATIOU D.: *May 6, 1976 Friuli Earthquake: Field Measurements* . . . " 851

PAPAZACHOS B.C.: *Evidence of crustal shortening in the northern Aegean region* . . " 66

PETRINI V., SETTI P., COLOMBO G., RAFFA C.: *On the Behaviour of a r.c. Building with Suspended Floors during the Friuli Earthquake in May, 1976* . . . " 1.576

PETRINI V., VITIELLO E.: *Dynamic Analysis of Friuli r.c. Damaged Buildings* . . " 1.593

PISTONE G., ROCCATI R.: *Notes on Old Masonry Buildings Structural Behaviour* . " 1.612

PRAKASA RAO T.K.S., KRISHNA MURTHY A.S.: *A direct method of interpreting magnetic anomalies caused by long horizontal cylinders* . . . . . " 72

RIBARIČ V.: *The Earthquake in Friuli May 6, 1976 - A contribution to the Study of Seismic Fields in Slovenia* . . . . . " 861

RITSEMA A. REINIER: *Preliminary Analysis of Friuli Earthquake Records* . . . " 875

RIZKALLA R.I.: *A trial for depth estimation to the origin of a magnetic anomalous source-body* . . . . . " 110

RIZKALLA I.: *Magnetic anomalies as a tool for the determination of basement rock types* " 114

ROULAND D., PETERSCHMITT E.: *On the Computation of Focal Mechanisms of Friuli Earthquakes* . . . . . " 889

SANCIN S.: *Convolution and Fourier's transform in the times series filtering with time variant operators* . . . . . " 199

SCARASCIA S., COLOMBI B., GUERRA I., LUONGO G., SLEJKO D.: *Preliminary Report on the Recording of Some Aftershocks along the Profile Gemona-Trieste* . " 895

SIRO L.: *Liquefaction of Sands in Friuli during the May 6 and September 15, 1976 Earthquake* . . . . . " 909

SCHMEDES E., MILLER H., GEBRANDE H.: *vp/vs Ratio and Implications for Hypocenter Determinations of Aftershocks of the Friuli Earthquake, May 6, 1976* . . " 933

TOKUYAMA A.: *Crustal Deformation after the Friuli Earthquake, 1976* . . . " 945

VERMA S.K.: *Optimum pulse shape selection in time-domain electromagnetic method* . " 124

VUKASHINOVITCH M.: *The Level of the Macroseismic Field for the Territory of Friuli* . " 953

VUKASHINOVITCH M.: *The Soil Acceleration Excited by the Earthquake of May 6* . " 959

VUKASHINOVITCH M.: *Seismic Shakeability and Seismic Pisk for The Territory of Udine* . . . . . " 964

VUKASHINOVITCH M., NEDELJKOVITCH S.: *The Duration of Oscillation as a Factor in the Earthquake Intensity Determination* . . . . . " 973

VUKASHINOVITCH M., NEDELJKOVITCH S.: *The Estimation of the Degree of Building's Damage According to the New Seismic MSK-64 Scale* . . . . . " 980

BRAMBATI A., CORSI M., GIORGETTI F., ONOFRI R., STEFANINI S., ULCIGRAI F.: *Geological Emergency Interventions in the Earthquake Stricken Zones of Friuli: the Example of Osoppo Town* . . . . . " 989

Book review: *Remote Sensing for Environmental Sciences* . . . . . " 136

Book review: *Remote Sensing for Environmental Sciences* . . . . . " 137

Book review: *Remote Sensing for Environmental Sciences* . . . . . " 214

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INDICE

BAYÜLKE N.: *Lateral Force Analysis of a Simple r.c. Structure in May 6, 1976 Friuli Earthquake* . . . . . pag. 1.029

BENEDETTI D., VITIELLO E.: *Ductility of r.c. Buildings during the Friuli Earthquake* . . . . . " 1.037

BENEDETTI D., VITIELLO E.: *Implicit Design Loads of Friuli r.c. Damaged Buildings* . . . . . " 1.052

BO G.M., DE STEFANO S., MANCUSO P.L., MANZONE G.: *Behaviour of Some Simple Structures during the Friuli Earthquake* " 1.082

BOSTJANCIC J., SHEPPARD P., TERCELJ S., TURNSEK V.: *Use of a Modelling Approach in the Analysis of the Effects of Repairs to Earthquake-Damaged Stone-Masonry Buildings* . . " 1.091

BRAGA F., PARDUCCI A.: *Elastic-Plastic Degrading Oscillators Behaviour under Earthquakes - Preliminary Results.* . . . " 1.117

BRANCALEONI F., PETRANGELI M.P.: *Dynamic Investigation of the Effects of Non-structural Elements on a r.c. Building in Friuli* . . . . . " 1.134

BRISIGHELLA L.: *Damages to Bridges and Roads in the 1976 Friuli Earthquake* . . . . . " 1.150

BRISEGHELLA L., CAPELLARI L., DALL'AGLIO B., D'EREDITÀ R., GORI R., SIMONI L., TURRINI G., ZAUPA F.: *Earthquake in Friuli (Italy), 1976. Damage to Historical Monuments and other Buildings of Artistic Interest* . . . . . " 1.203

CARTAPATI E., CHERUBINI A.: *Recently Constructed Masonry Buildings* . . . . . " 1.453

FAORO G.F., FERRAZZA G.: *Report on Damages and Provisions Attaining to Earthquake Concerning the Electrical System* . " 1.472

GIUFFRÈ A., MENEGOTTO M., PINTO P.E.: *Collapse of a Non-seismic Resistant r.c. Building during the Friuli Earthquake: ASTRO Building in Maiano* . . . . . " 1.496

GLAUSER E.C., HEIMGARTNER E., SÄGESSER R.: *Assessment and Interpretation of Building Damage* . . . . . " 1.506

HEIMGARTNER E.: *Behaviour of Damaged Buildings under Strong Earthquake* . . . . . " 1.540

IZUMI M.: *Damage in Friuli Earthquake 1976 - Buildings in Osoppo* " 1.549

OLSZAK W.: *On the Inelastic Behaviour of Structures During Earthquakes as Exemplified by the Friuli catstrophe 1976* . . . " 1.563

PETRINI V., SETTI P., COLOMBO G., RAFFA C.: *On the Behaviour of a r.c. Building with Suspended Floors during the Friuli Earthquake in May, 1976* . . . . . " 1.576

PETRINI V., VITIELLO E.: *Dynamic Analysis of Friuli r.c. Damaged Buildings* . . . . . " 1.593

PISTONE G., ROCCATI R.: *Notes on Old Masonry Buildings Structural Behaviour* . . . . . " 1.612