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PROGETTO DEFINITIVO DI PIANO ATTUATIVO PRESIDIO OSPEDALIERO-UNIVERSITARIO DI CISANELLO - NUOVA FACOLTA' DI MEDICINA E CHIRURGIA

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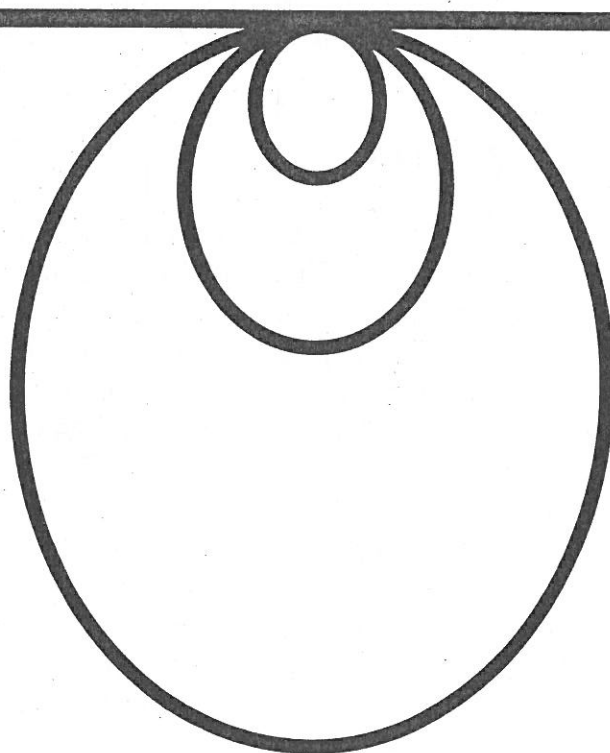


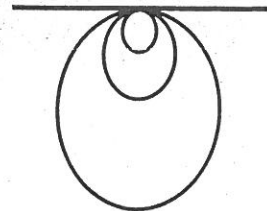
Azienda Ospedaliera Universitaria Pisana
Presidio Ospedaliero di Cisanello

**Studio di fattibilità per l'aggiornamento
del programma di sviluppo**

Indagini geologico-tecniche

Giugno 2005



**Azienda Ospedaliera Universitaria Pisana**
Presidio Ospedaliero di Cisanello**Studio di fattibilità per l'aggiornamento
del programma di sviluppo**

Indagini geologico-tecniche

Giugno 2005

GEOLOGIA APPLICATA

GEOFISICA

GEOTECNICA

IDROGEOLOGIA E
IDROLOGIA

MODELLI MATEMATICI

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Estratto da Crespellani, Madiati, Vannucchi, 2003
"CPT-based liquefaction hazard maps for an Italian coastal area"

0. PREMESSA

Espongo alcune valutazioni di ordine geologico-applicativo relativamente al lavoro

Presidio Ospedaliero di Cisanello (Pisa) Studio di fattibilità per l'aggiornamento del programma di sviluppo

Si tratta di un Progetto a carattere urbanistico, inteso ad orientare la pianificazione complessiva dell'area destinata ai nuovi fabbricati dell'Azienda Ospedaliera Universitaria Pisana (vedi tav. 1).

Ritenendo che la disposizione degli edifici e le relative tipologie fossero da pensarsi anche in rapporto alle caratteristiche dei terreni, il Progettista promosse una campagna sondaggi intesa ad individuare i principali lineamenti stratimetrici e geomeccanici del luogo.

Questa relazione riporta i risultati ottenuti ed individua le problematiche ingegneristiche da considerarsi nel disegno architettonico.

1. INDAGINI DI CAMPO E RELATIVE INTERPRETAZIONI

Vista la connotazione preliminare del lavoro, si ritenne opportuno eseguire le prospezioni mediante un penetrometro statico.

Come è noto, la particolare prova (CPT) permette di determinare in modo rapido ed economico la litologia del terreno attraversato e le sue principali caratteristiche meccaniche.

I 500 m di sondaggio preventivati furono distribuiti secondo una maglia ragionevolmente larga, così da ottenere profondità significative in rapporto all'importanza dell'opera.

In tav. 2 sono indicati i 20 punti di prova; l'irregolarità del passo nel comparto settentrionale deriva dalla circostanza che all'atto della campagna geognostica l'area campita in figura non risultava ancora di proprietà dell'Azienda.

Il puntale impiegato rispecchia lo standard M2 (Friction Jacket Cone); lo strumento rileva le forze tramite un dinamometro idrostatico con tre manometri a scale differenziate.

Le misure della resistenza alla punta (q_p), dell'attrito sul manicotto di frizione (f_s) e del rapporto q_p/f_s sono diagrammate nelle tavole 3 - 22; la tav. 23 inviluppa tutti i grafici.

Le quote piezometriche nei fori di sondaggio pongono la tavola d'acqua intorno a -6 m dalla superficie (le canne permeabili calate nei fori 9, 13 e 19 consentiranno eventuali rilevazioni differite).

Le tavole 24 - 43 sono riservate all'interpretazione delle misure CPT.

La classifica litologica è in accordo con lo schema di Schmertmann, 1978.

Nella derivazione dei parametri meccanici e della storia tensionale il codice informatico utilizzato divide i materiali incoerenti da quelli coesivi con riferimento ad un valore discriminante del rapporto q_c/f_s ; il calcolo riguarda:

- il probabile andamento della tensione litostatica effettiva σ'_{v0}
- l'angolo di resistenza al taglio in termini efficaci ϕ' per le terre giudicate incoerenti;
- la resistenza al taglio in termini totali (ovvero la coesione non drenata c_u per $\phi = 0$) relativa alle terre giudicate coesive;
- OCR (vale a dire il rapporto tra la tensione di preconsolidazione σ'_p e quella litostatica σ'_{v0}) laddove si dispone della stima di c_u .

L'angolo ϕ' è ottenuto in accordo con Durgunoglu e Mitchell, 1975; c_u deriva da correlazioni di Sanglerat, 1972, adattate al caso sulla scorta di nostre esperienze; in particolare, limitatamente all'unità di misura in bar:

$$c_u = \lambda (q_c - \sigma'_{v0}) / (q_c/4 + 15)$$

con $\lambda = 0.75$ per i loams,
 $\lambda = 1.00$ per le argille "magre",
 $\lambda = 1.25$ per le argille "grasse",
 $\lambda = 1.50$ per le torbe.

Relativamente ad OCR, si ricorda che esso esprime lo scostamento tra la consistenza misurata e quella che occorrerebbe attendersi nel medesimo terreno qualora la consolidazione fosse avvenuta per effetto del solo carico litostatico; $OCR > 1$ indica uno stato precompresso.

La relazione utilizzata per la stima del rapporto è la seguente:

$$c_u/\sigma'_{v0} = (0.23 \pm 0.04) OCR^{0.8} \text{ (Ladd e Foot, 1974).}$$

imponendo $OCR = 1$ quale limite inferiore.

Tale condizione si è resa necessaria per evitare che un calcolo acritico portasse a classificare come sottoconsolidato il banco coesivo molle presente in profondità, aspetto questo verosimilmente illogico, considerato che quei terreni hanno avuto tempo e modo di adattarsi alle pressioni litostatiche.

In sintesi, l'interpretazione dei dati indica che il pacco di 35 m oggetto dei sondaggi è suddiviso in 4 unità sovrapposte: suolo, sabbie di tetto, argille mediane ed alluvioni differenziate di base.

Il suolo, spesso circa 2 m, è di natura limo-argillosa e presenta buona consistenza, data da desaturazione ed essiccamenti.

Le sabbie di tetto raggiungono la profondità di 14 - 18 m articolandosi in banchi sabbiosi di media densità e sottili orizzonti coesivi, che localmente si ispessiscono per originare strati di 1 - 2 m.

Le argille mediane si ritrovano fino alla profondità di 22 m; si tratta di terre molli, probabilmente con spiccata componente organica e senza dubbio normalconsolidate.

Le alluvioni differenziate di base possono vedersi come un intreccio di lingue sabbioso-ghiaiose intruse in un corpo coesivo con strati più o meno sovraconsolidati.

Su questo motivo geologico di fondo s'innestano variazioni locali non trascurabili sotto il profilo geotecnico; tali anisotropie, considerate nel prosieguo in forma puramente statistica, andranno meglio definite nelle fasi progettuali più avanzate.

2. NOTE GEOLOGICHE

I terreni descritti fanno parte del corpo alluvionale deposto dal F. Arno presso il suo sbocco a mare.

Terre di bassa pianura dunque.

Lo spessore considerevole del pacco sedimentario in rapporto all'elevazione dell'area di pochi metri trova ragione nelle variazioni eustatiche del livello marino occorse durante i periodi glaciali ed interglaciali del Quaternario antico.

Le differenze in termini granulometrici riscontrate in direzione verticale sono frutto delle divagazioni del Fiume, indotte per un verso dal sistematico sovralluvionamento degli alvei, per l'altro dagli ostacoli frapposti allo sbocco dai depositi eolici costieri.

Le sabbie di tetto dovrebbero così rappresentare l'intreccio dei più recenti paleoalvei, mentre le argille mediane andrebbero ascritte ad un precedente episodio palustre, condizionato probabilmente da un radicale allontanamento del Fiume o da un'ingressione marina.

Le alluvioni differenziate di base ripetono l'intreccio di paleoalvei, ma di corsi d'acqua apparentemente minori e con energia trattiva sufficiente a trasportare ghiaia; quest'ultima caratteristica implicherebbe un mare di diverse decine di metri più basso dell'attuale e quindi un clima nettamente glaciale.

3. NOTE DI SISMICA

Sotto tale profilo, i terreni descritti sono classificabili nella categoria D (secondo la più recente Normativa in materia); si ricorda a tal proposito che il Comune di Pisa risulta ascritto tra le zone 2, vale a dire con accelerazione orizzontale massima convenzionale su suolo di categoria A compresa nel campo 0.15 - 0.25 g.

L'attività tettonica si concentra lungo la dorsale appenninica e nell'Amiata; il "Catalogo dei terremoti italiani" riporta un picco di Magnitudo stimata a 7 (1846 - Toscana settentrionale) e due casi con Magnitudo 6.5 (1767 - Fivizzano e 1920 - Garfagnana)

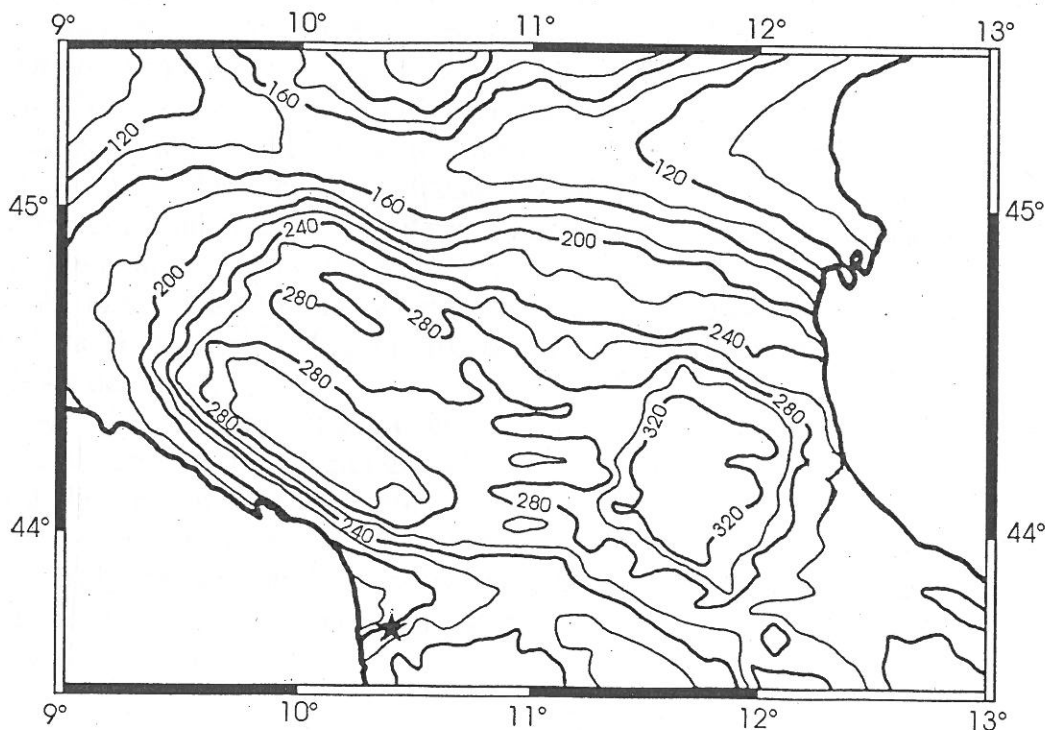
Relativamente ad un cinquantennio, si prevedono a Pisa intensità eccedenti l'ottavo grado MCS con la probabilità di 0.394, 0.033 per il nono grado e 0.001 per il decimo (secondo CNR - GNDT, 1998).

La presenza di sabbie in falda implica una verifica del rischio di liquefazione.

A tal proposito si utilizza la procedura di Robertson e Wride, 1997, basata sulle misure CPT, ed il metodo di sintesi proposto da Iwasaki et al., 1982.

Rimandando alla bibliografia specialistica per la relative descrizioni, qui ci si limita ad allegare l'elenco degli algoritmi (da Crespellani, Madiati e Vannucchi, "CPT-based liquefaction hazard maps for an Italian coastal area", 2003).

Quale accelerazione di picco attesa (PGA) si assume 240 gal, con riferimento alla mappa seguente (da Frassinetti et al., 1997).



Al fine di introdurre criteri conservativi, si pone la Magnitudo pari a 6.5 e si riducono alla metà i valori d'attrito laterale locale dei CPT, assumendo che tale sia il rapporto tra le misure del cono meccanico (utilizzato) e quello elettrico (considerato nel lavoro di R. & W).

I risultati sono diagrammati nelle tavv. 44 - 63; da queste si evince che il rischio di liquefazione delle sabbie locali è basso.

4. ASPETTI IDRAULICI

Il Fiume Arno delimita l'area di studio ad est ed a sud, il fondo dell'alveo risulta alla quota di circa -2m rispetto al livello marino, le sommità arginali intorno a +10 m ed il piano campagna a +7m.

L'ampiezza infra-arginale varia da 200 a 400 m.

L'Autorità di Bacino comprende l'intera area di Progetto tra quelle "di pertinenza fluviale dell'Arno".

In tali aree, ove se ne verifichi la fattibilità e l'efficacia, devono essere realizzati interventi che contribuiscano ad un miglioramento del regime idraulico ed idrogeologico ai fini della difesa del territorio così come definito negli strumenti programmatori e pianificatori di competenza.

L'area è altresì compresa tra quelle interessate da inondazioni eccezionali, dove le opere che comportano trasformazioni edilizie ed urbanistiche ... potranno essere realizzate a condizione che venga documentato dal proponente ed accertato dall'Autorità amministrativa competente al rilascio dell'autorizzazione il non incremento del rischio idraulico da esse determinabile o che siano individuati gli interventi necessari alla mitigazione di tale rischio, da realizzarsi contestualmente all'esecuzione delle opere richieste.

Nel comparto sud dell'area è infine previsto un "Intervento strutturale di tipo B", in particolare una cassa di esondazione; tale indirizzo prevede un vincolo d'inedificabilità subordinato ad ulteriori verifiche da parte dell'Autorità di Bacino, per cui possono prevedersi modifiche alla perimetrazione.

A parte quest'ultimo aspetto, il quadro normativo offre dunque al Progettista la possibilità di proporre interventi atti a mitigare le problematiche idrauliche delineate.

A tal proposito, sembra questa la sede per accennare al drenaggio superficiale delle acque di pioggia, che verosimilmente subirà un'accelerazione a causa delle coperture impermeabili.

In genere si ovvia a tale inconveniente realizzando vasche di laminazione.

Un'alternativa potrebbe essere rappresentata dall'accumulo temporaneo in volumi porosi interrati (ghiaie, ad esempio) e nel rilascio per filtrazione nelle sabbie del substrato.

5. PRESTAZIONI PORTANTI DEL TERRENO

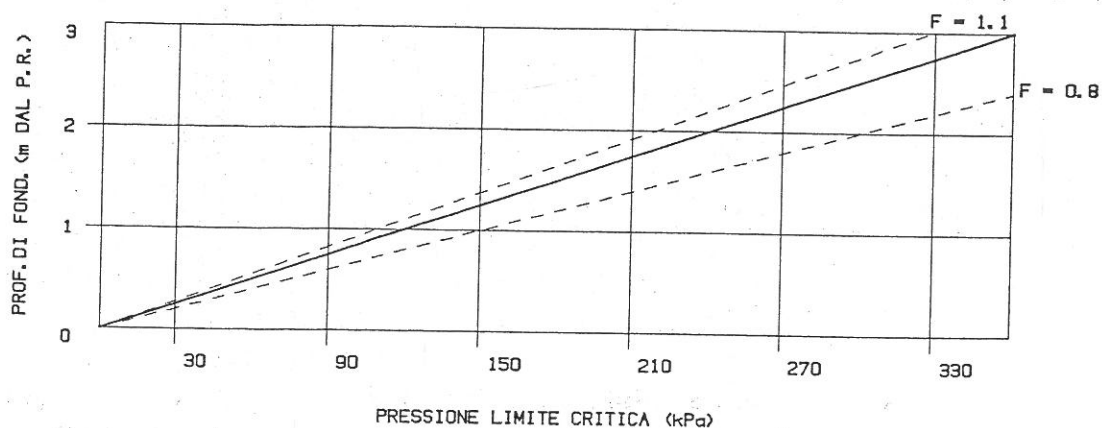
Nell'attuale fase pianificatoria non è il caso di addentrarsi in calcoli che riguardano l'interazione suolo-struttura, visto che di quest'ultima nulla si sa.

Si può tuttalpiù avanzare una stima della pressione limite critica q_{crit} , che dipende dalla resistenza al taglio del terreno in termini di angolo d'attrito (ϕ) e coesione intercetta (c), dal suo peso di volume (γ) e dalla profondità di seppellimento della struttura (H).

La relazione è la seguente (Frölich):

$$q_{crit} = \pi (\gamma H + c \cotg \phi) / (\cotg \phi - \pi/2 + \phi)$$

Attribuendo alle sabbie dell'unità di tetto $\phi = 32^\circ$, $c = 0$ e $\gamma = 1.9 \text{ Mg/m}^3$, per H variabile da 1 a 3 m si ottiene il grafico in figura.



Le linee tratteggiate comprendono il campo consigliato per la pressione d'esercizio.

Occorre far presente che q_{crit} rappresenta semplicemente lo sforzo necessario ad innescare la plasticità del suolo al bordo dell'appoggio, per cui il nesso con la pressione d'esercizio può essere fuorviante: quest'ultima risulterà infatti sottostimata laddove è opportuno assumere a riferimento la rottura globale del masso interagente e sovrastimata laddove sono critiche le deformazioni indotte.

Le discriminazioni in tal senso sono da delegarsi alle fasi progettuali più avanzate.

6. TIPOLOGIE EDILIZIE E FONDAZIONI

A grande scala, il substrato dell'area presenta anisotropie sia in orizzontale sia in verticale: per un verso l'unità sabbiosa di tetto, nel complesso a buone caratteristiche meccaniche, contiene saltuari orizzonti coesivi molli e sicuramente cedevoli; per l'altro incombe l'unità argillosa mediana, che va considerata pessima sotto il profilo della compressibilità.

Pur non escludendo che sondaggi più ravvicinati abbiano ad individuare comparti trasversalmente isotropi dove adottare fondazioni isolate che risolvano la loro influenza nell'unità sabbiosa, in prima istanza occorre ritenere che la tipologia continua (vale a dire il reticolato di travi) sia da preferirsi, soprattutto per ragioni antisismiche.

Contenendo le luci interne e le altezze degli edifici, si renderà fattibile in termini tecnici ed economici la platea generale, che indubbiamente costituirebbe un valido elemento di salvaguardia nei confronti di localizzate liquefazioni delle sabbie.

Con la presenza di un piano interrato, la platea diverrebbe infine "compensata", vale a dire con carico al suolo nell'ordine dello sgravio ottenuto con lo scavo d'alloggiamento; in tale circostanza non occorrerà attendersi cedimenti significativi negli interstrati molli più superficiali e soprattutto nel banco argilloso mediano.

Visto che la falda idrica si colloca a circa -6 m dalla superficie, lo scavo di un piano non comporta particolari problematiche di cantiere, quali emungimenti forzati e relativi riflessi sugli edifici vicini.

La platea e le relative travi-parete di contorno preserverebbero infine il vano sotterraneo da eventuali risalite della quota piezometrica.

Ecco dunque delineata una razionale tipologia edilizia per l'area: fondazione a platea, luci interne ragionevoli, un piano interrato sull'intera superficie coperta, uno spiccato di tutt'al più sei piani con disposizione simmetrica.

Ciò non significa la preclusione di architetture più ardite, ma solo che queste potrebbero avere in quei terreni risvolti economici assai pesanti, soprattutto al livello delle opere fondali.

Grandi luci e carichi concentrati impegnativi richiederebbero infatti palificate d'appoggio, con la prospettiva di ricercare prestazioni portanti a profondità proibitive.



TAV. 1 COROGRAFIA GENERALE

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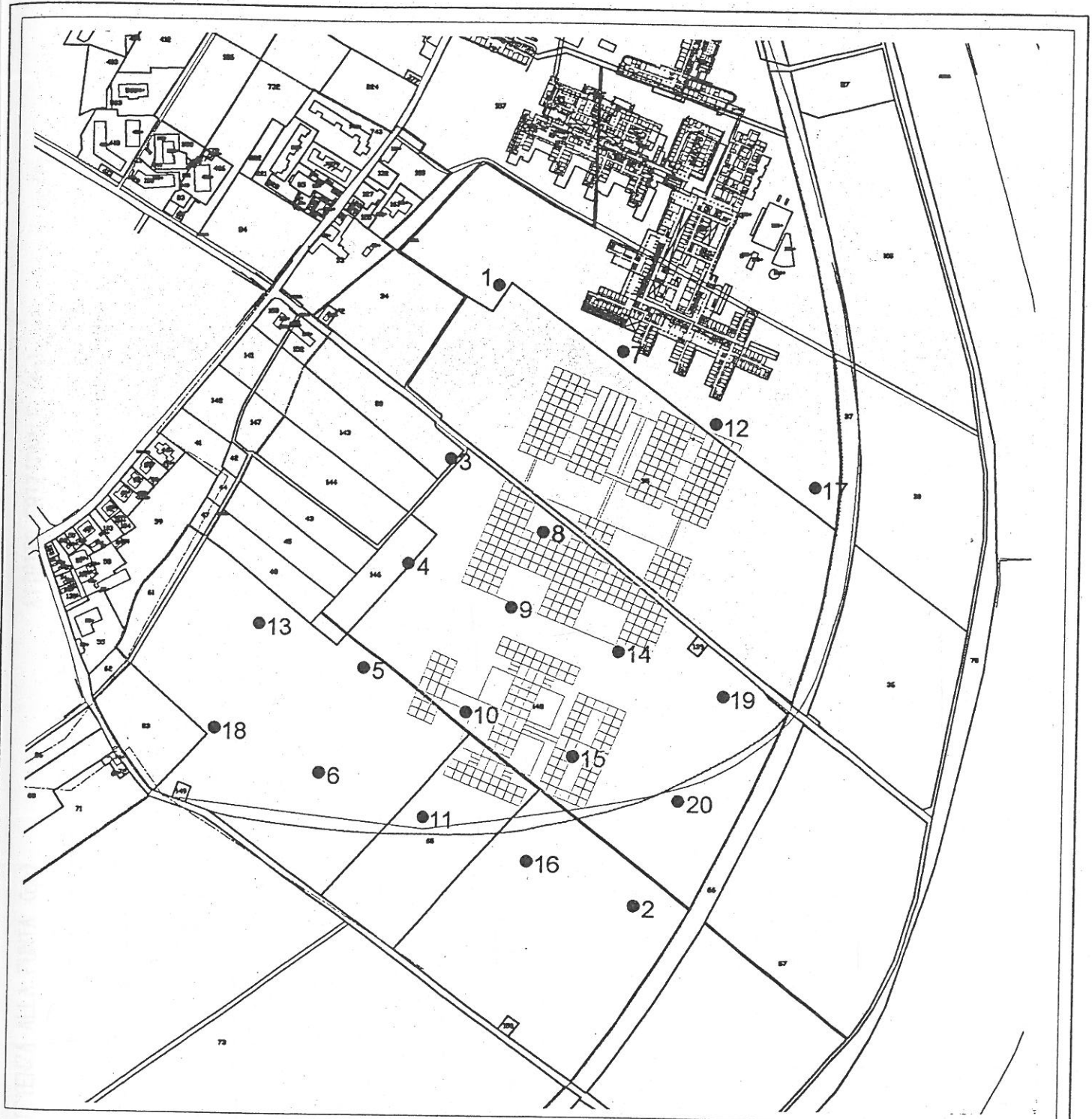


Area di progetto

NORD



LEGENDA



TAV. 2 UBICAZIONE SONDAGGI

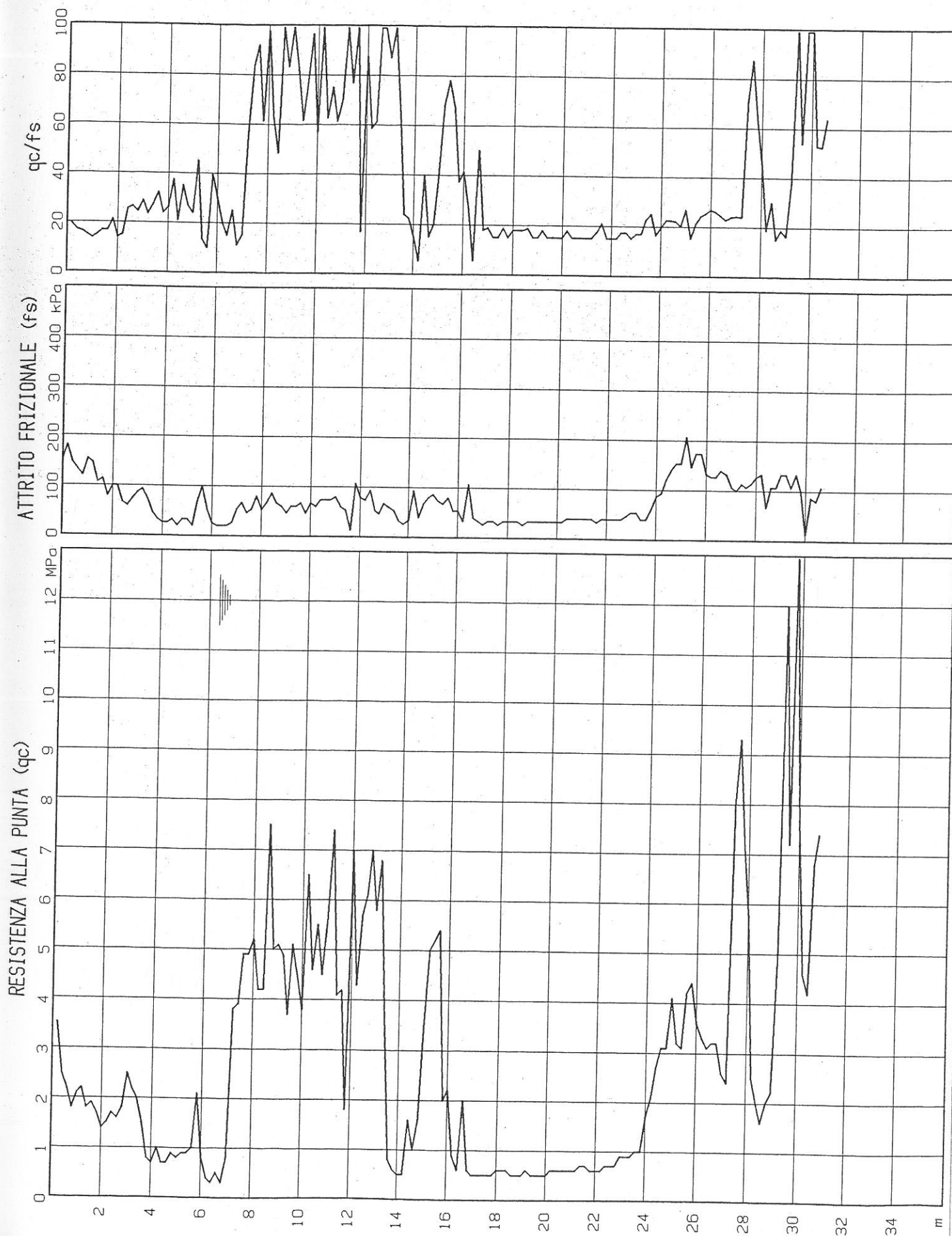
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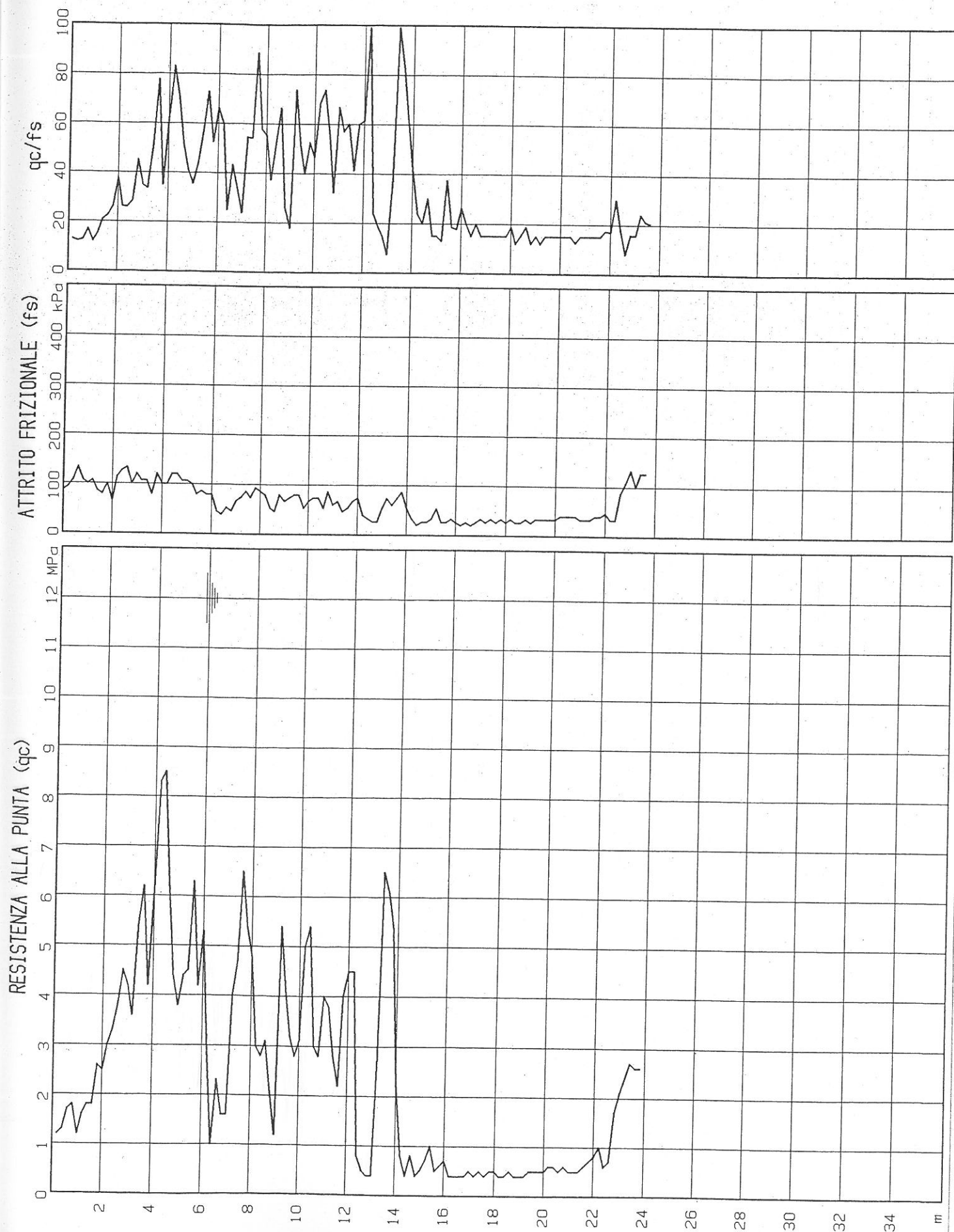
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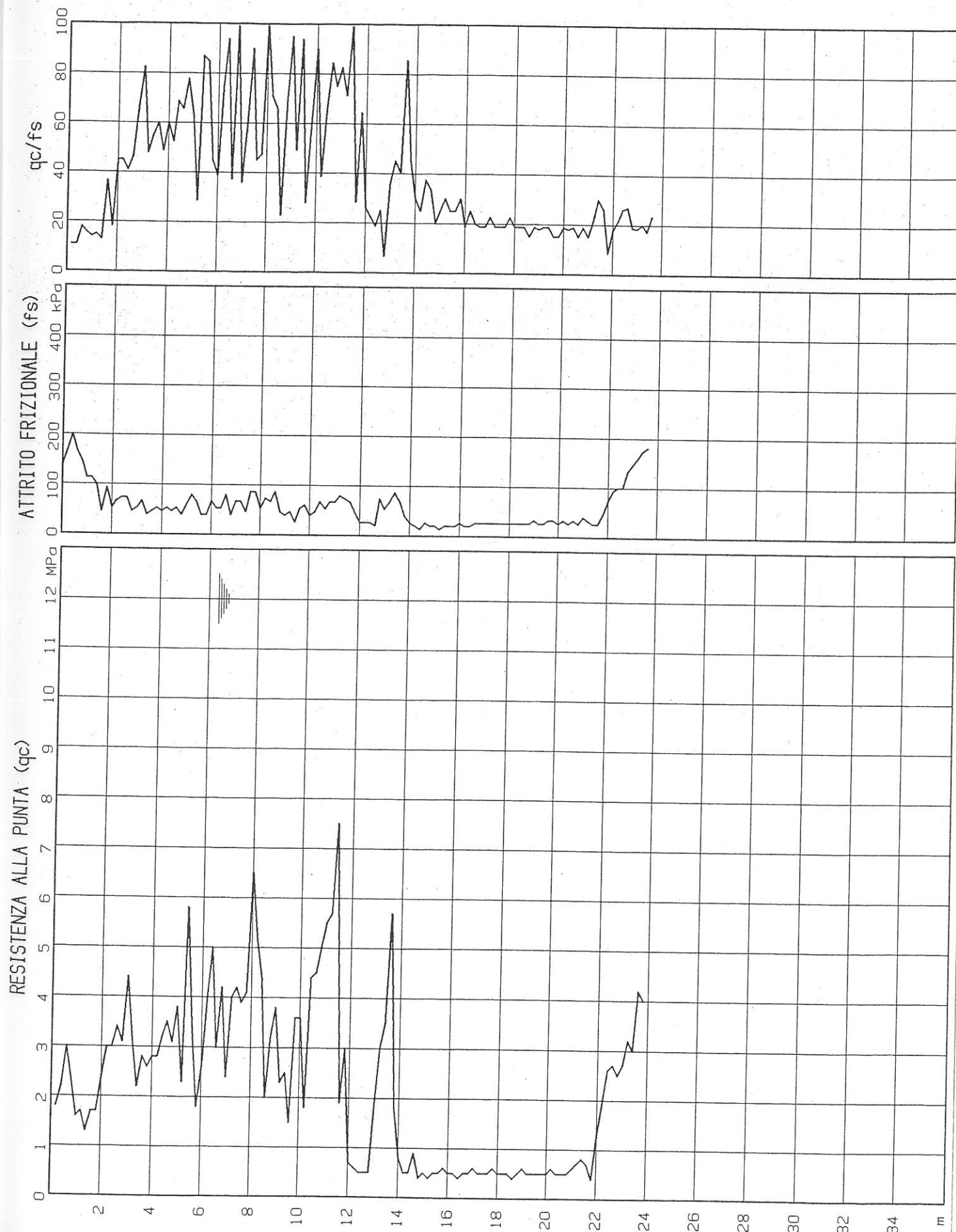


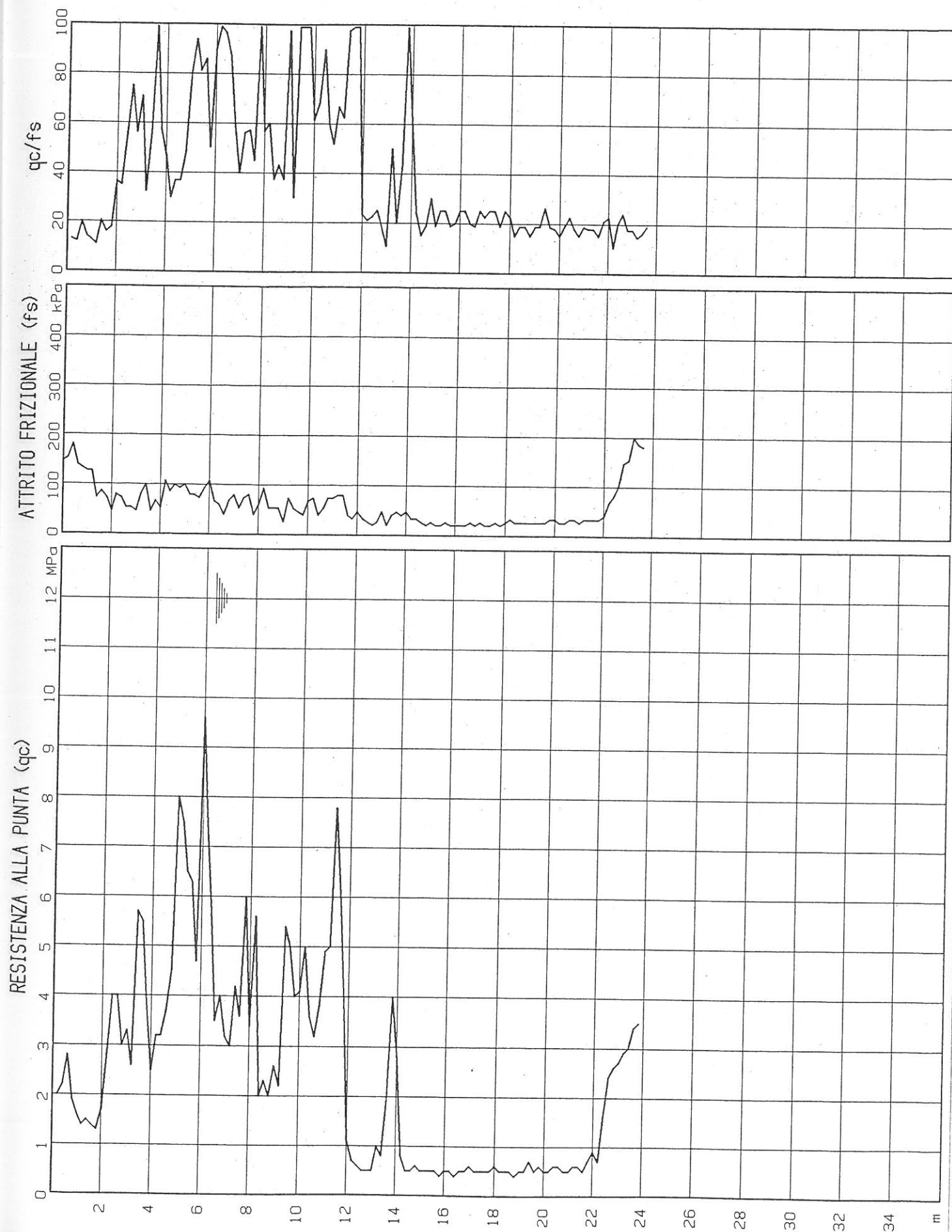
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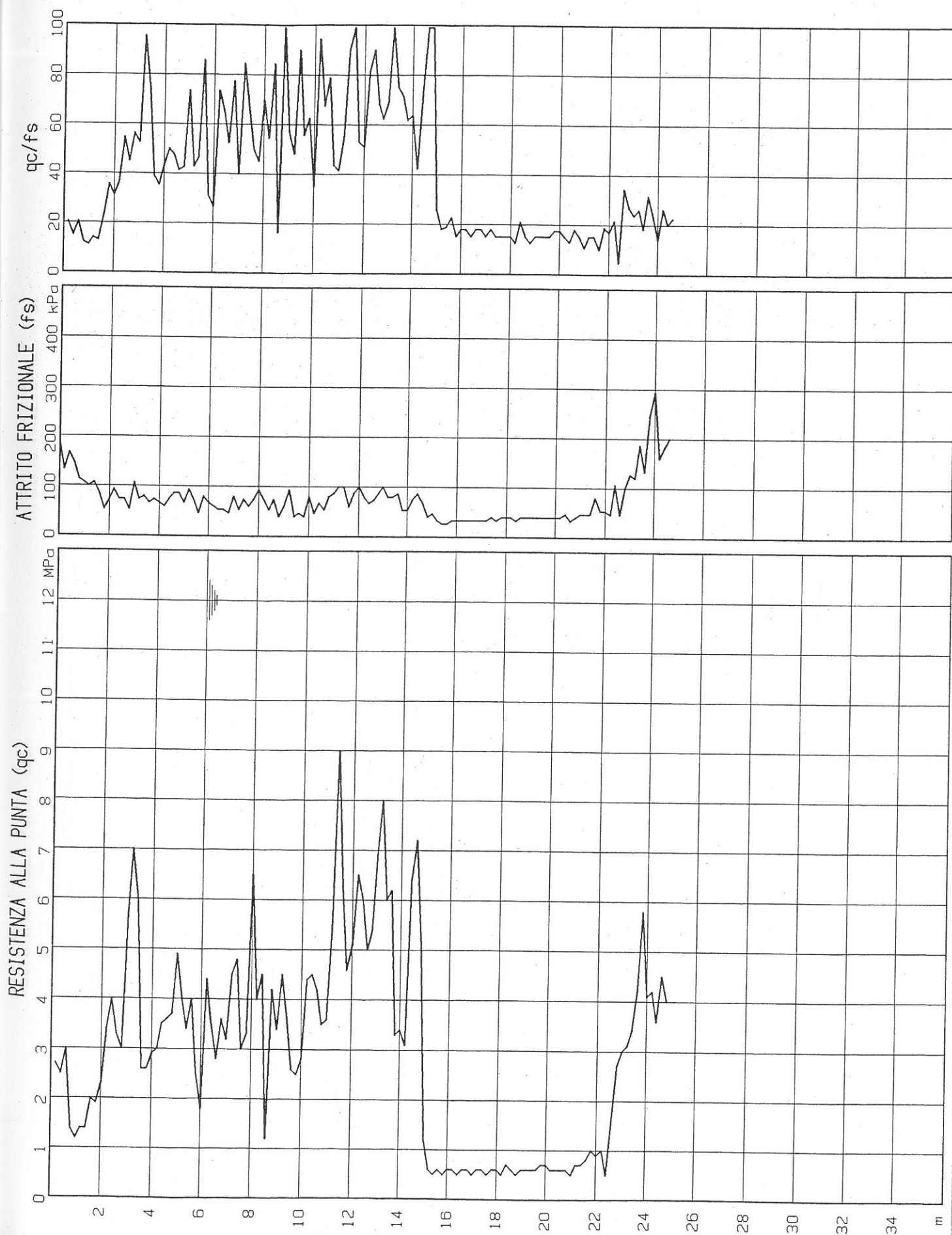
● *Prova penetrometrica statica (CPT)*

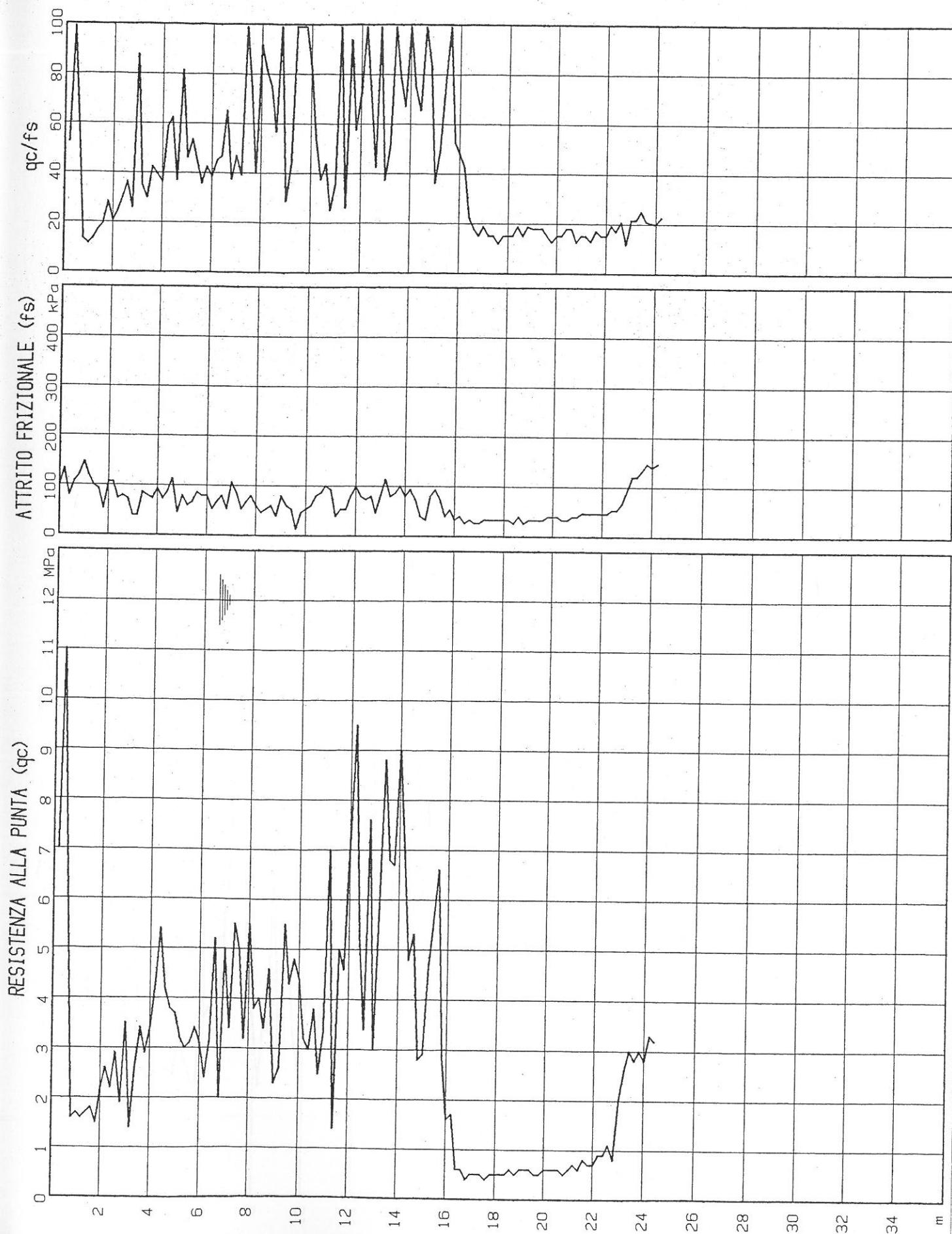


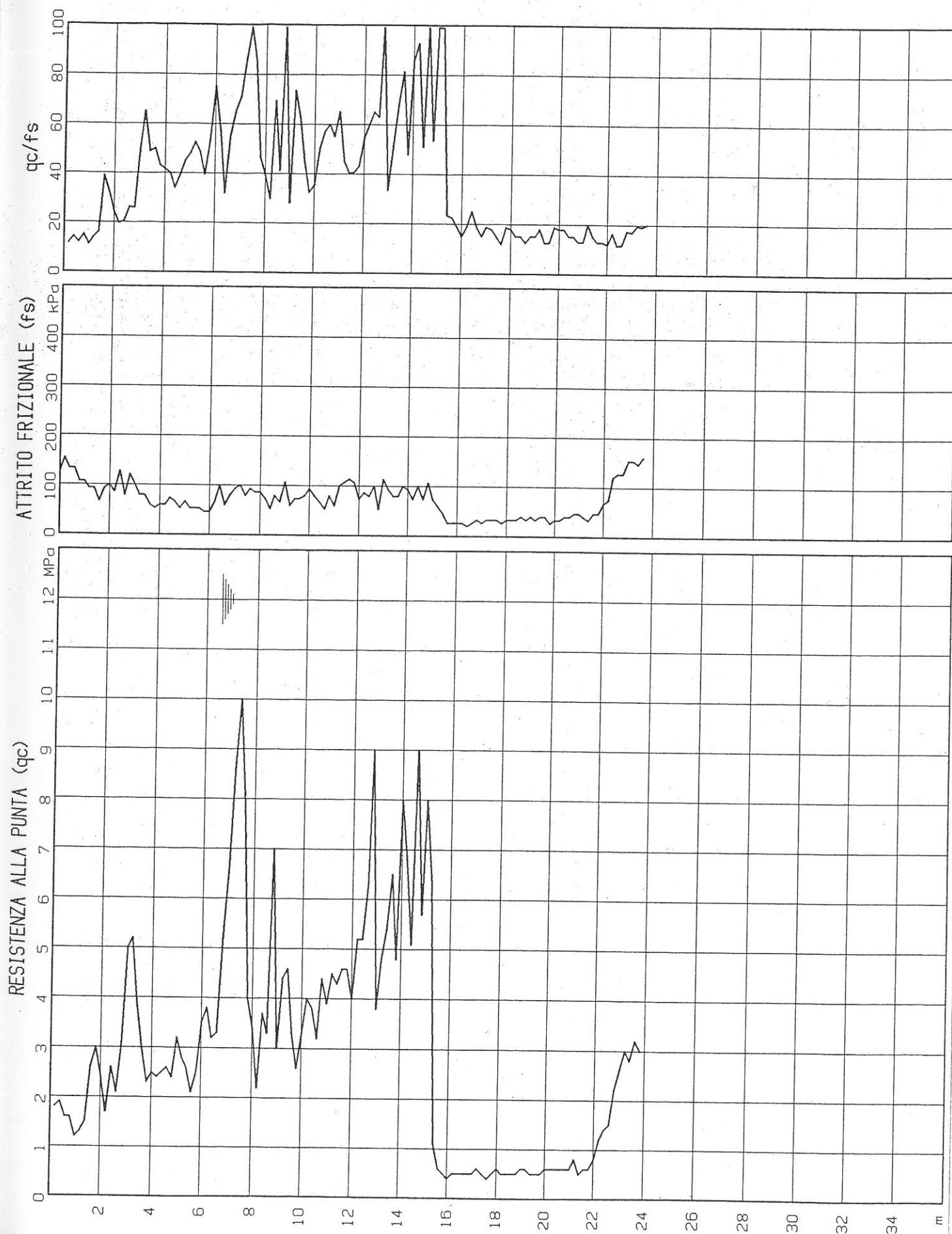


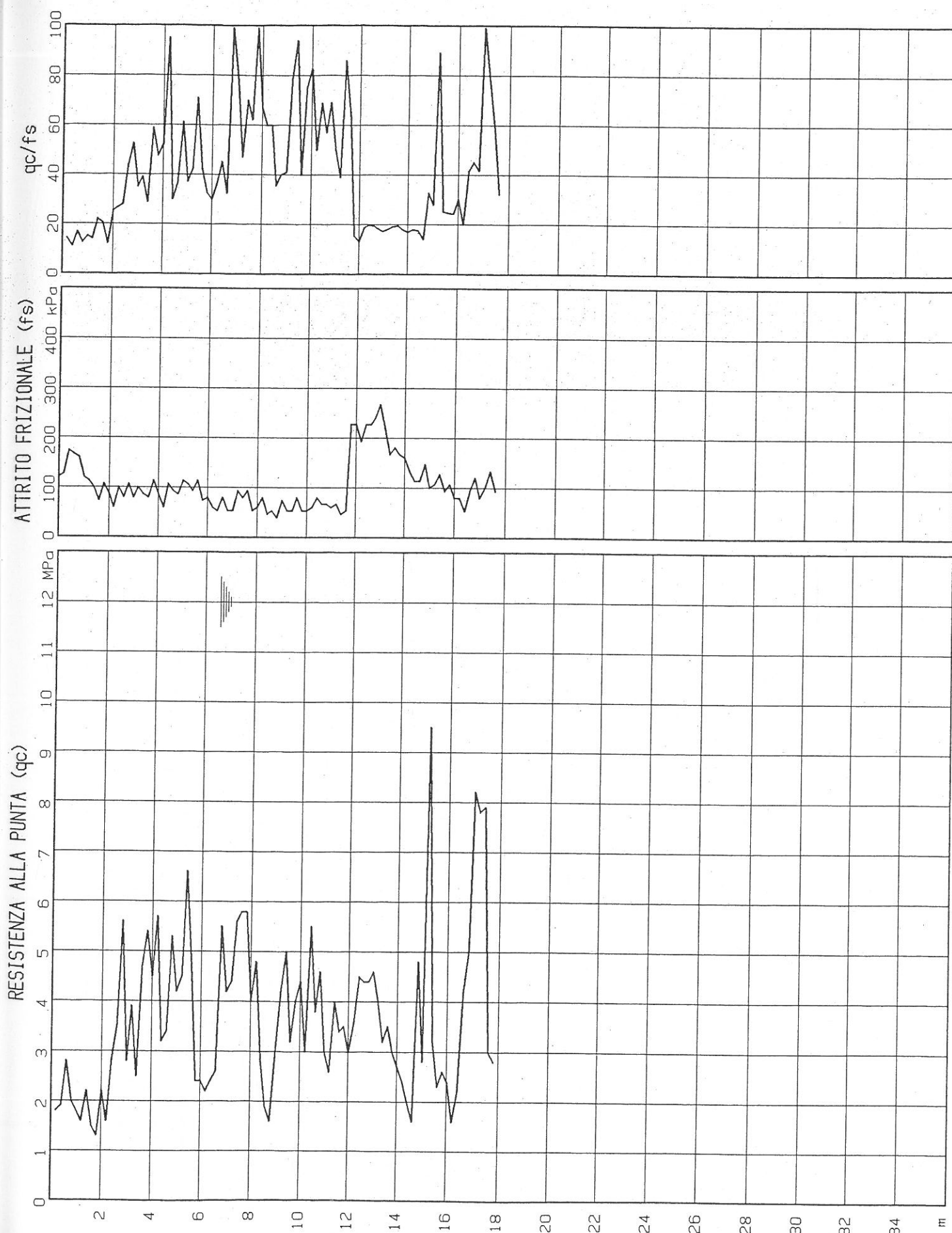


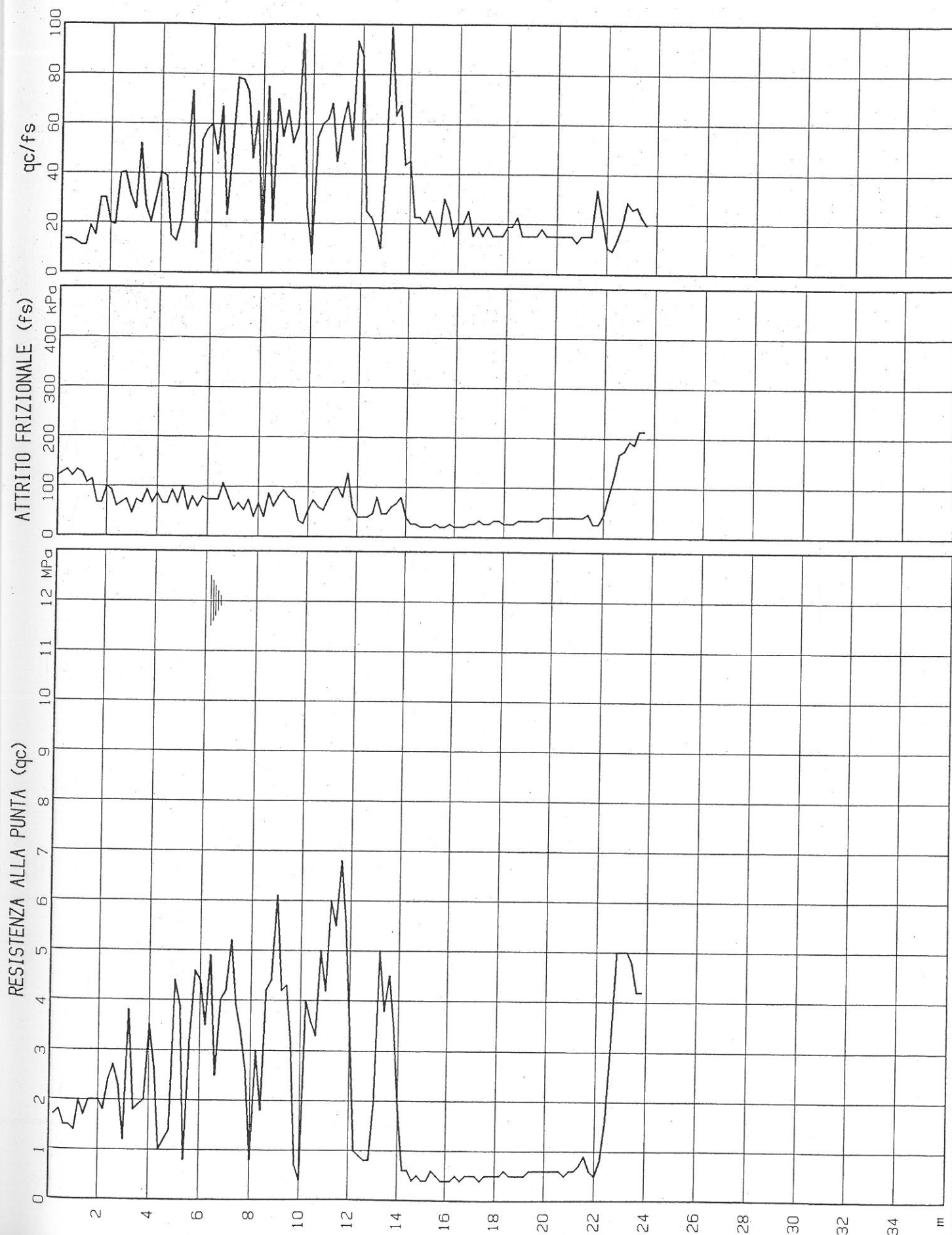


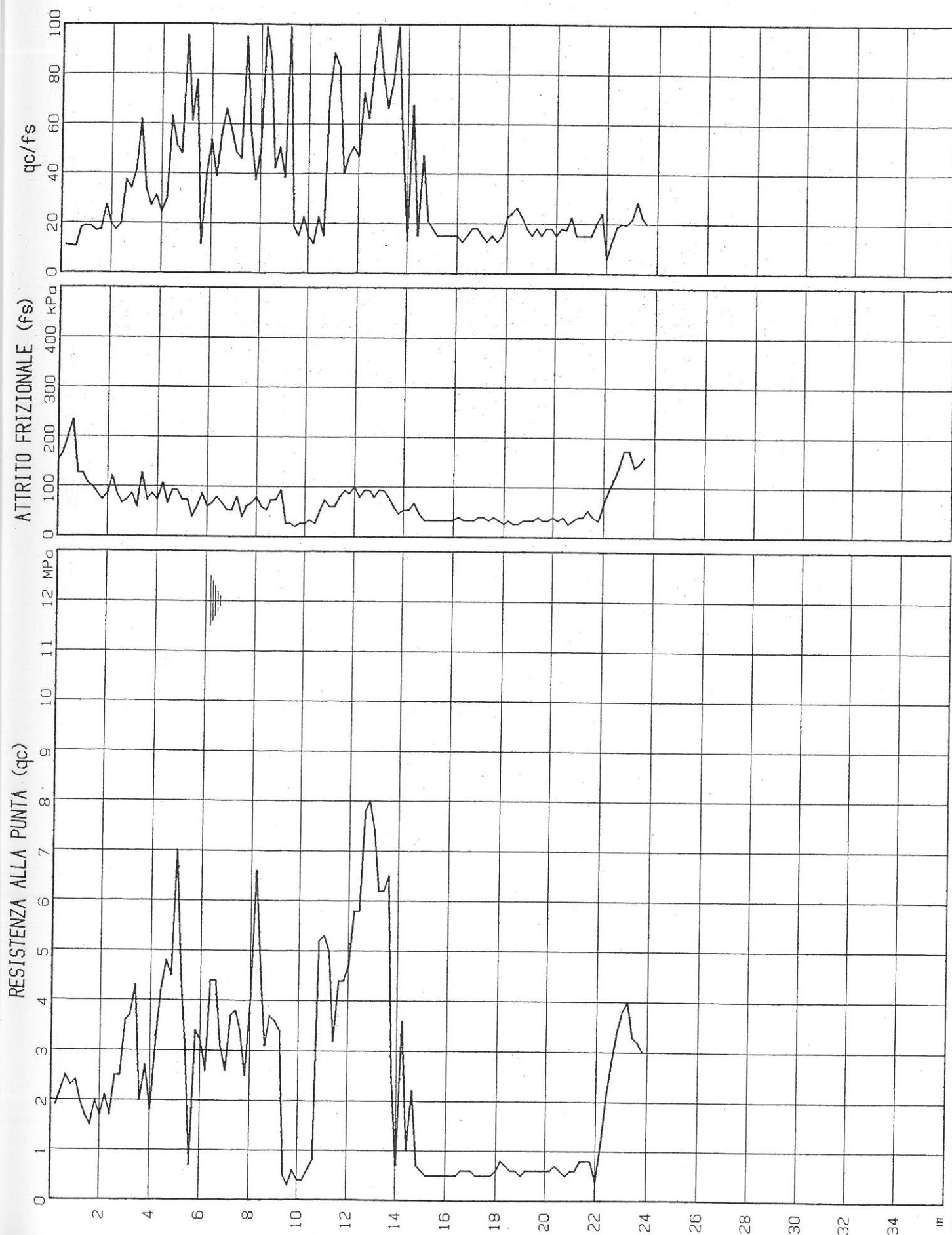


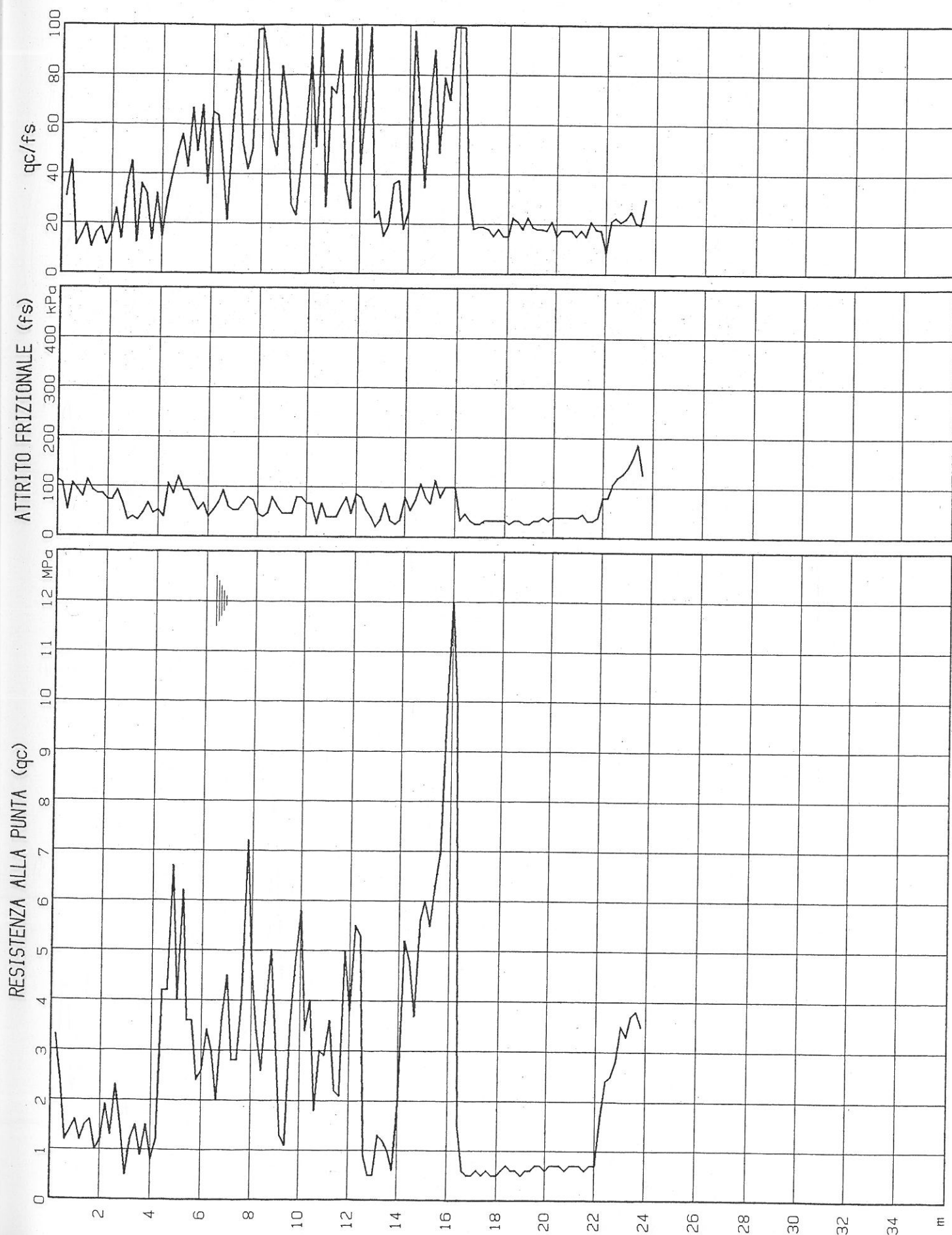


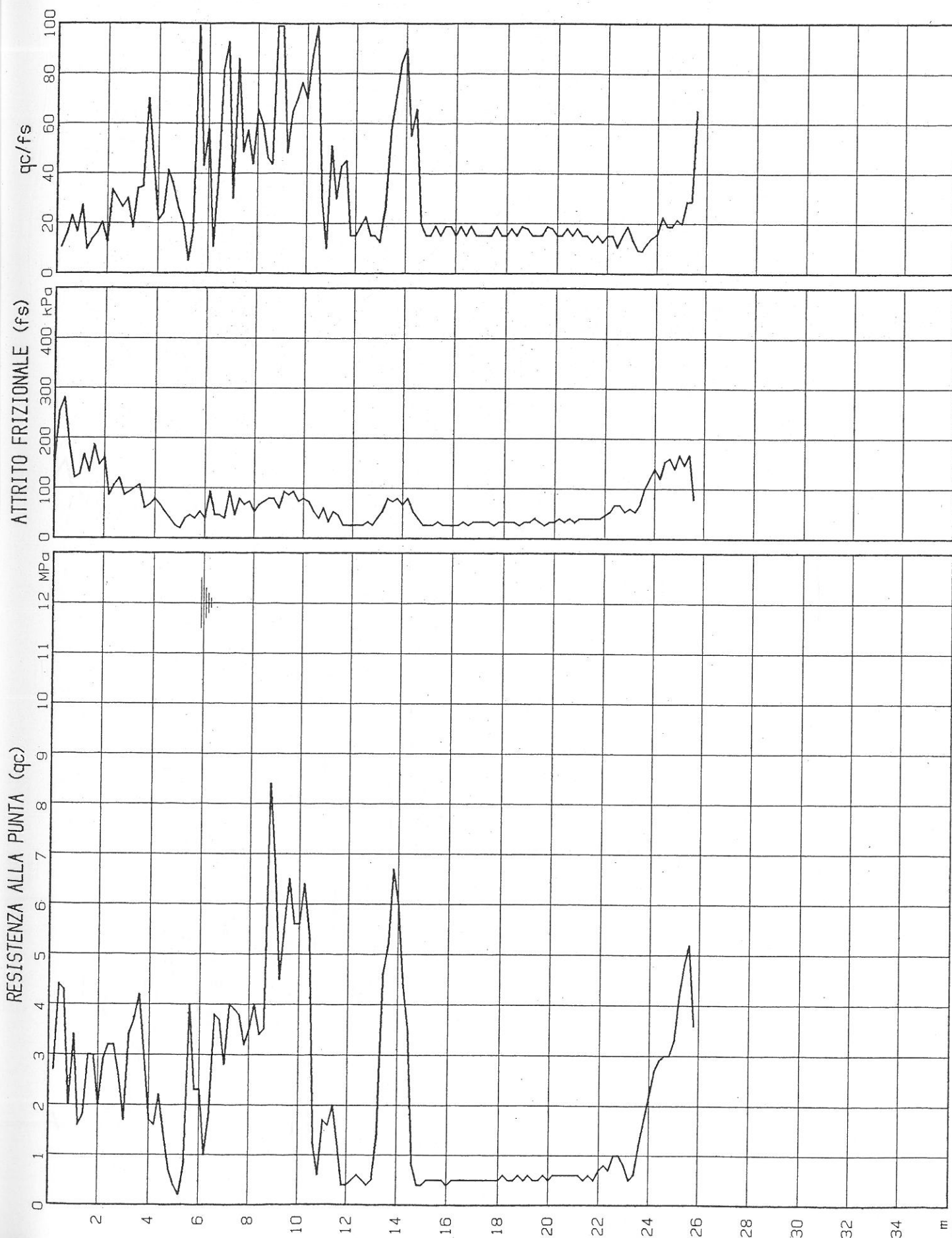


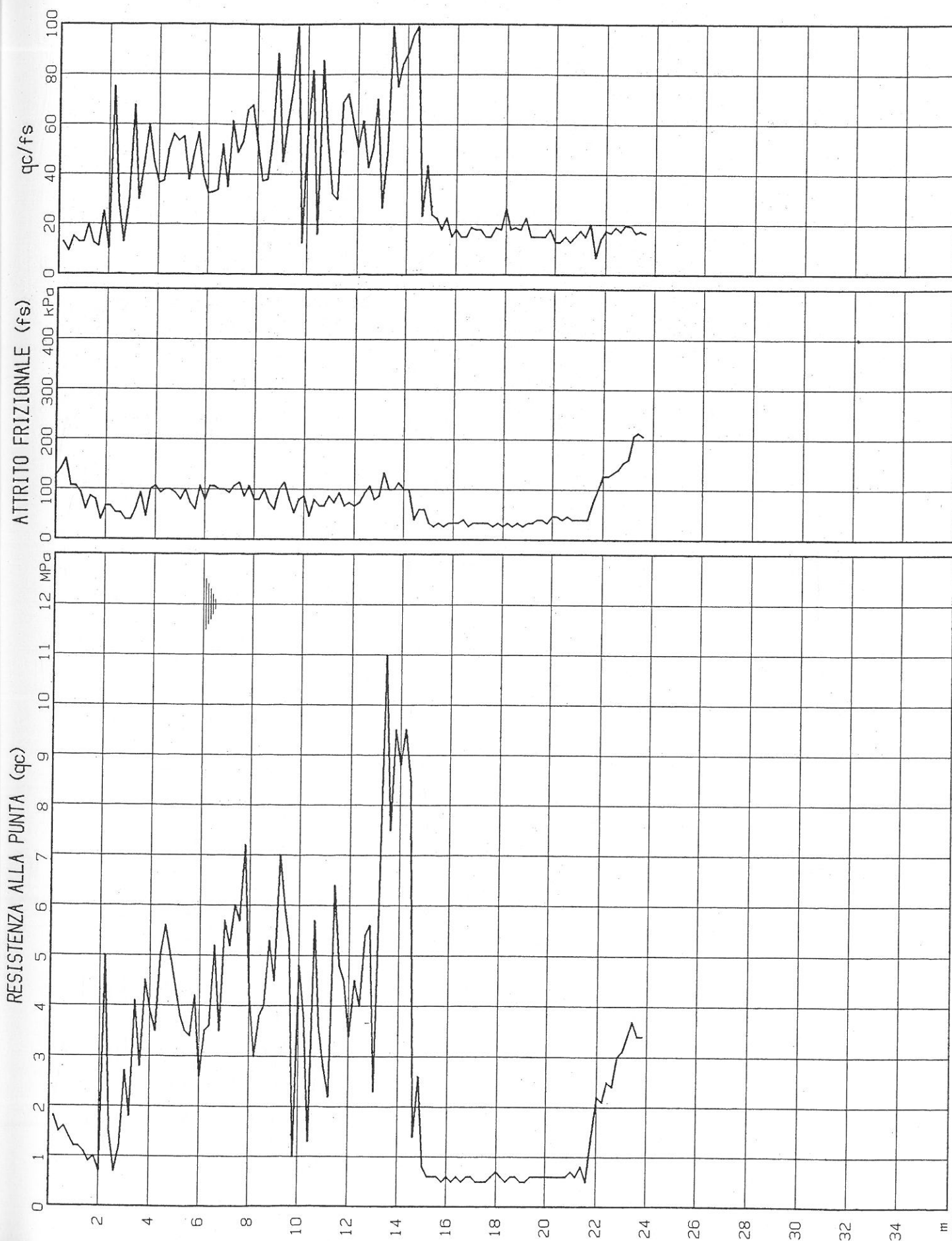


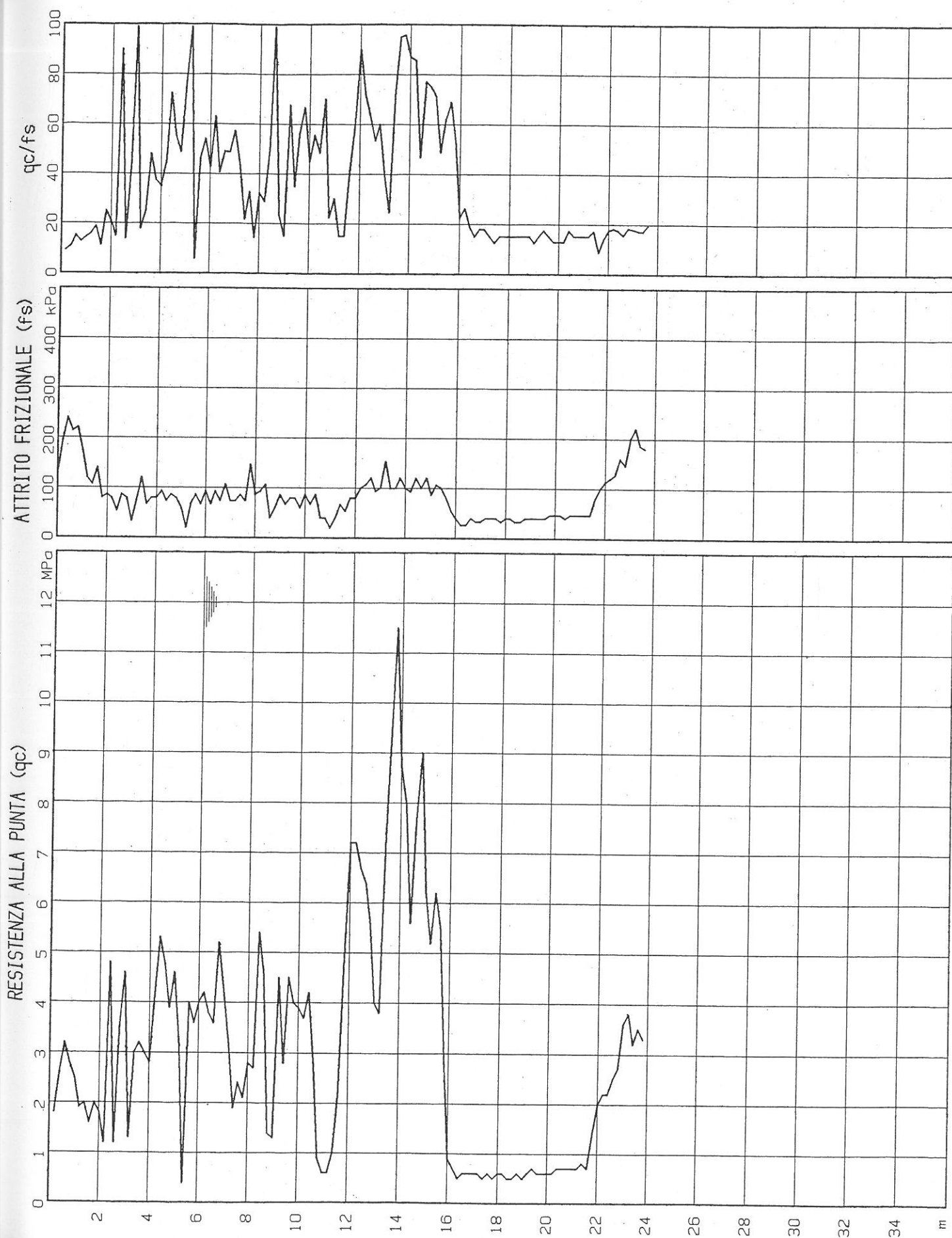


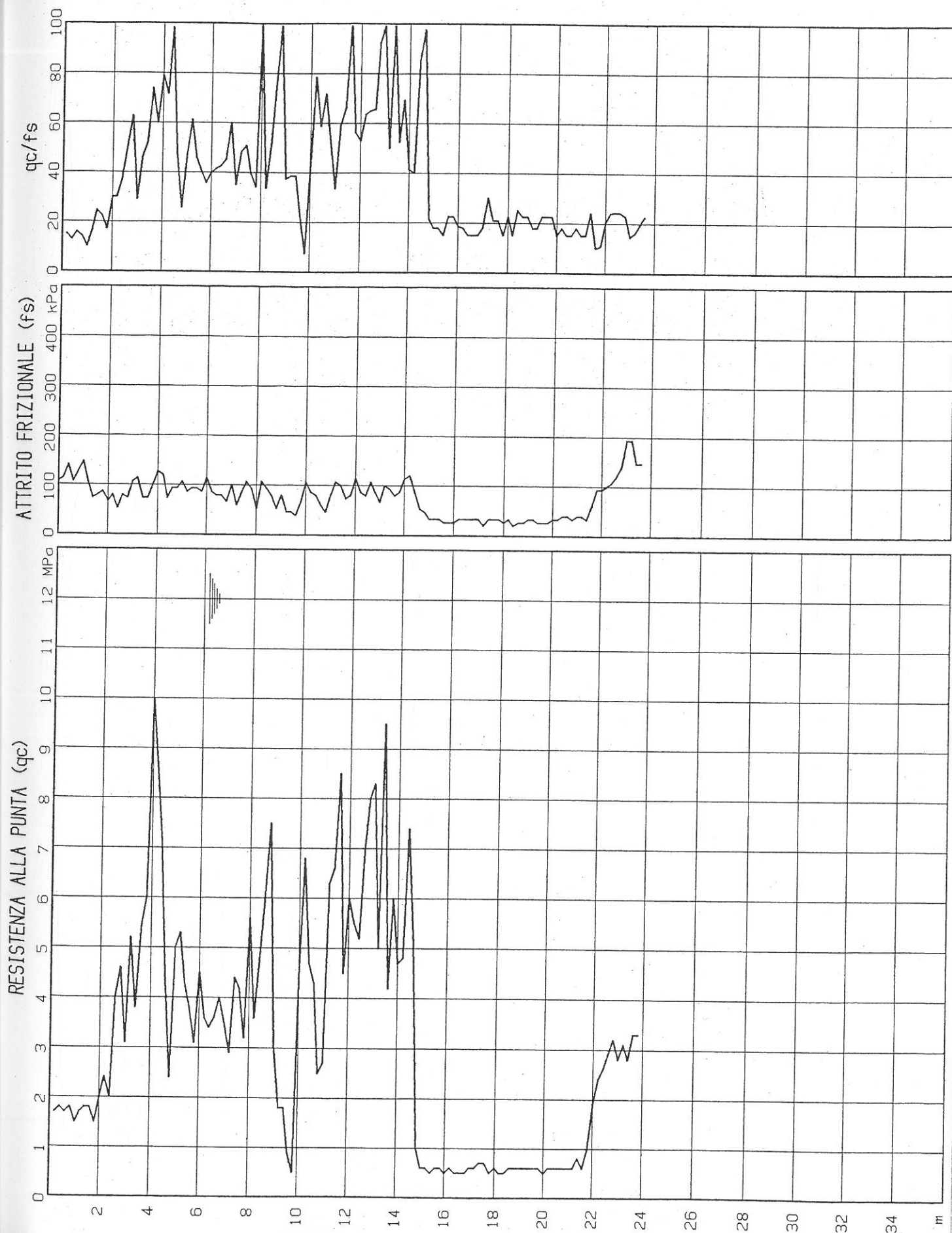


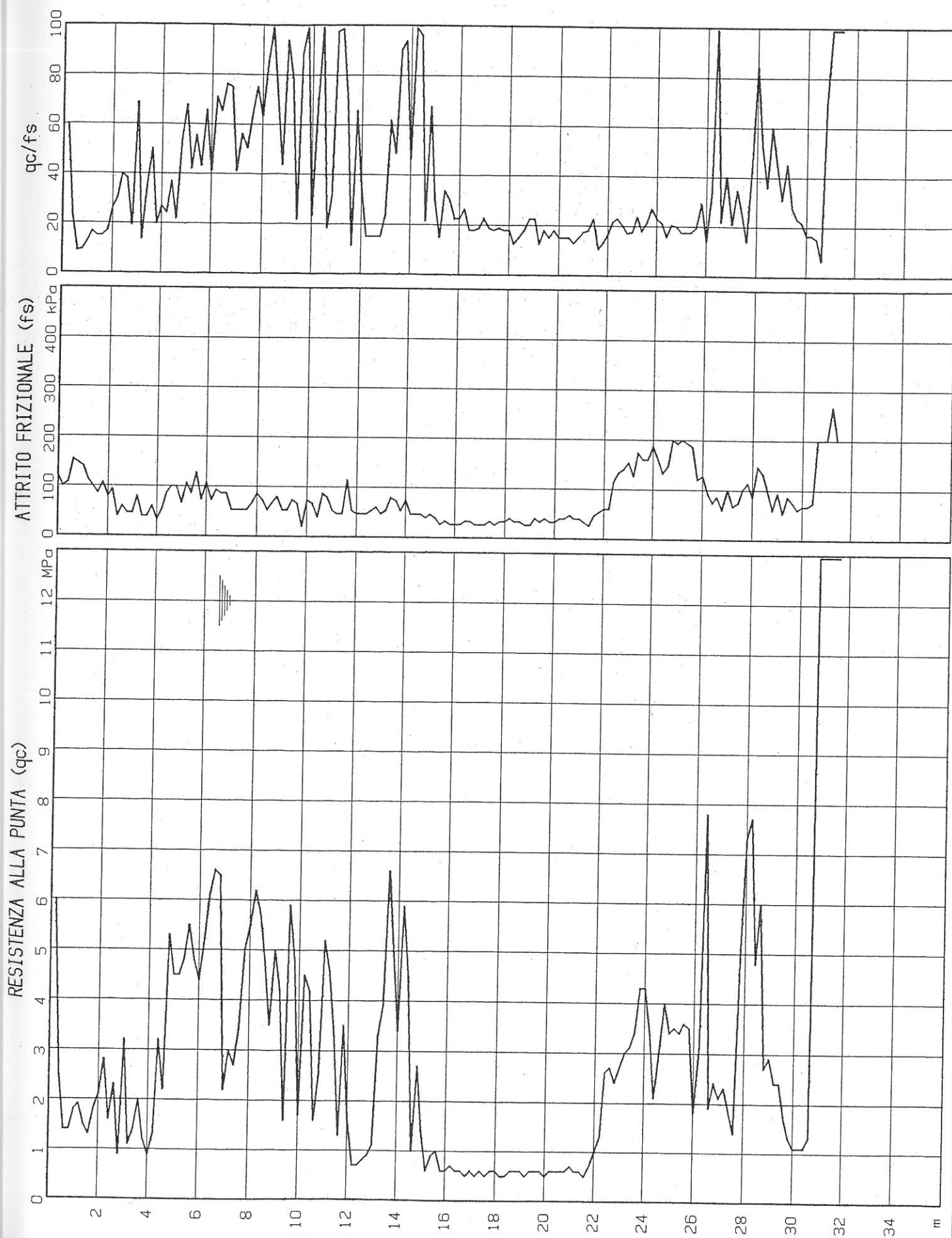


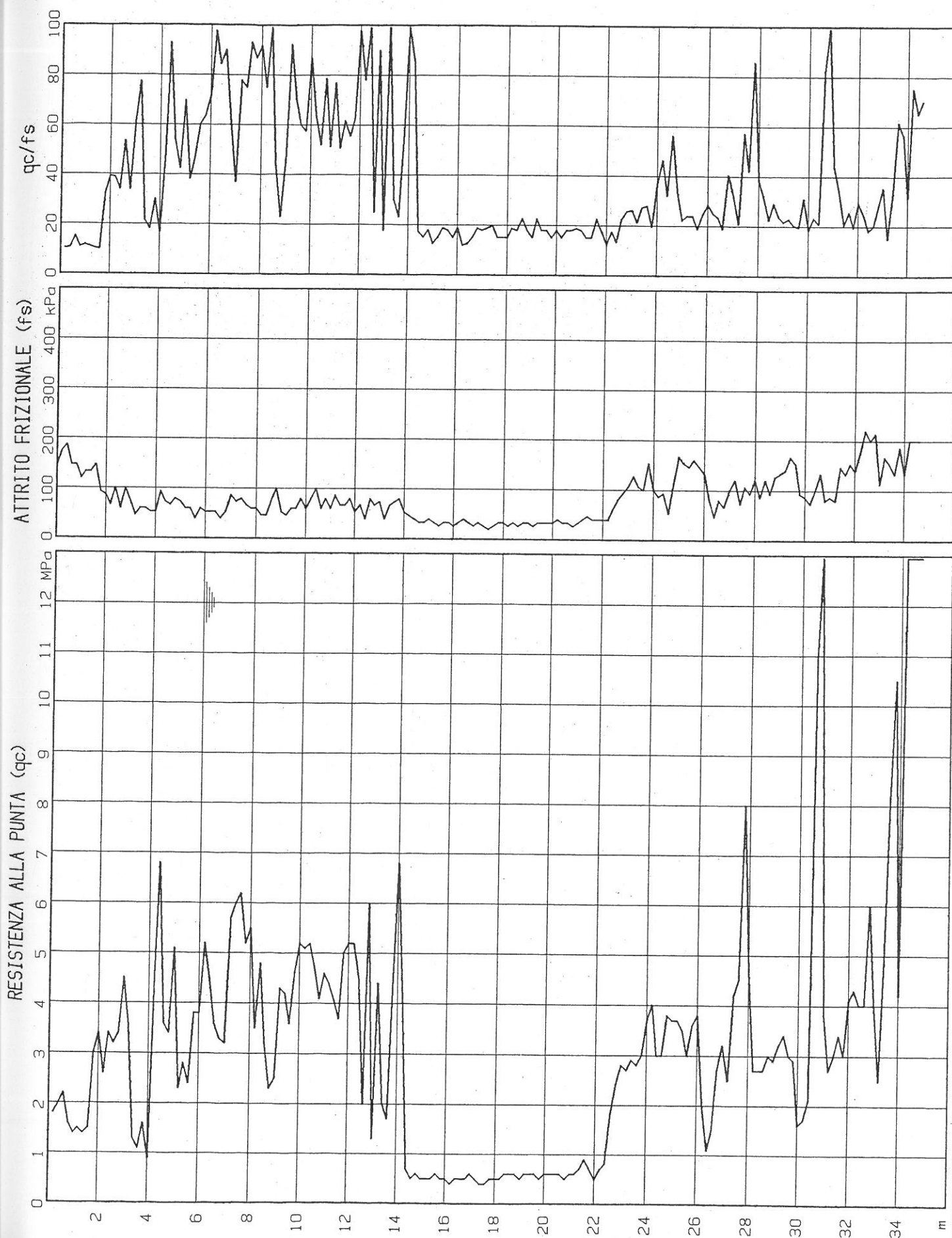


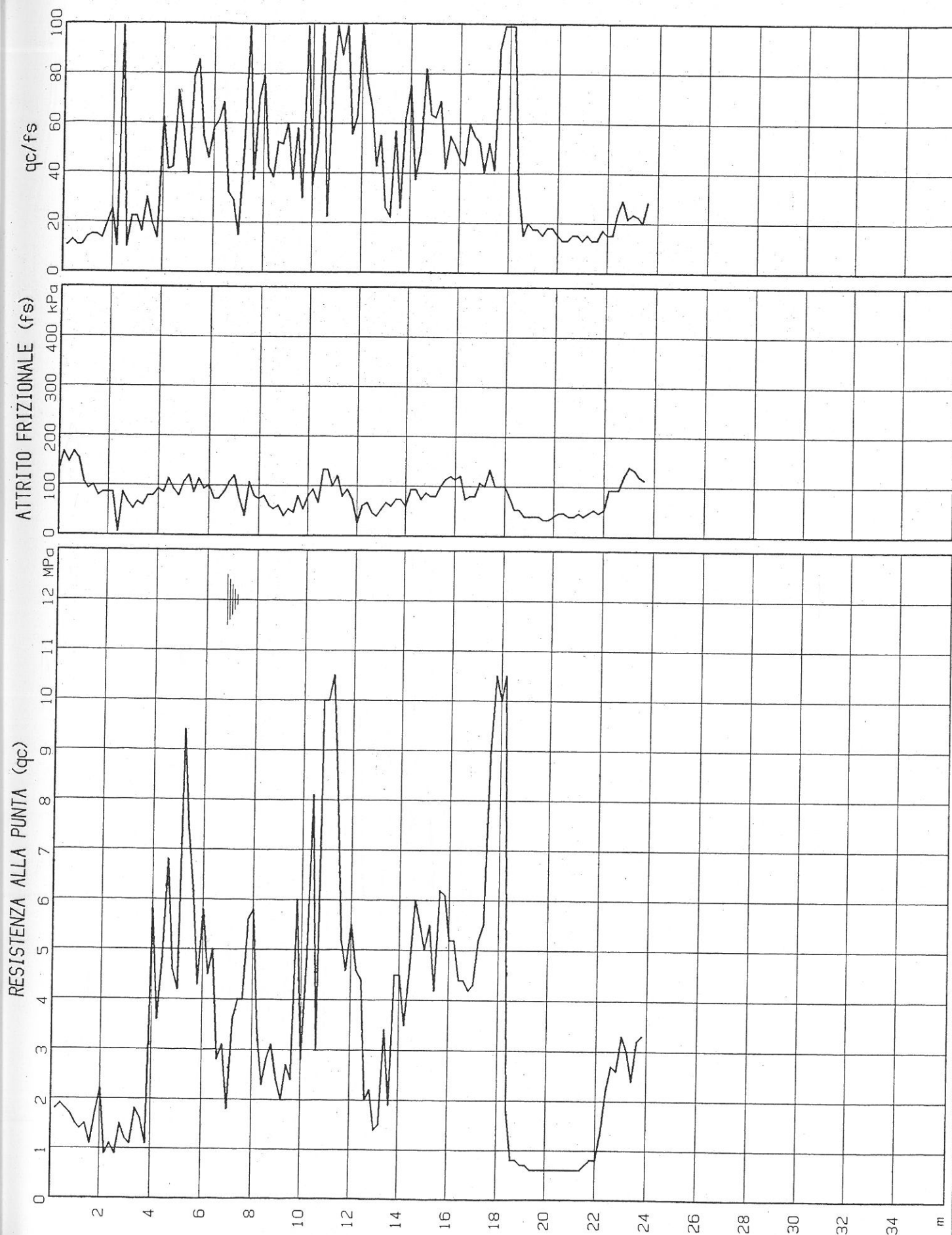


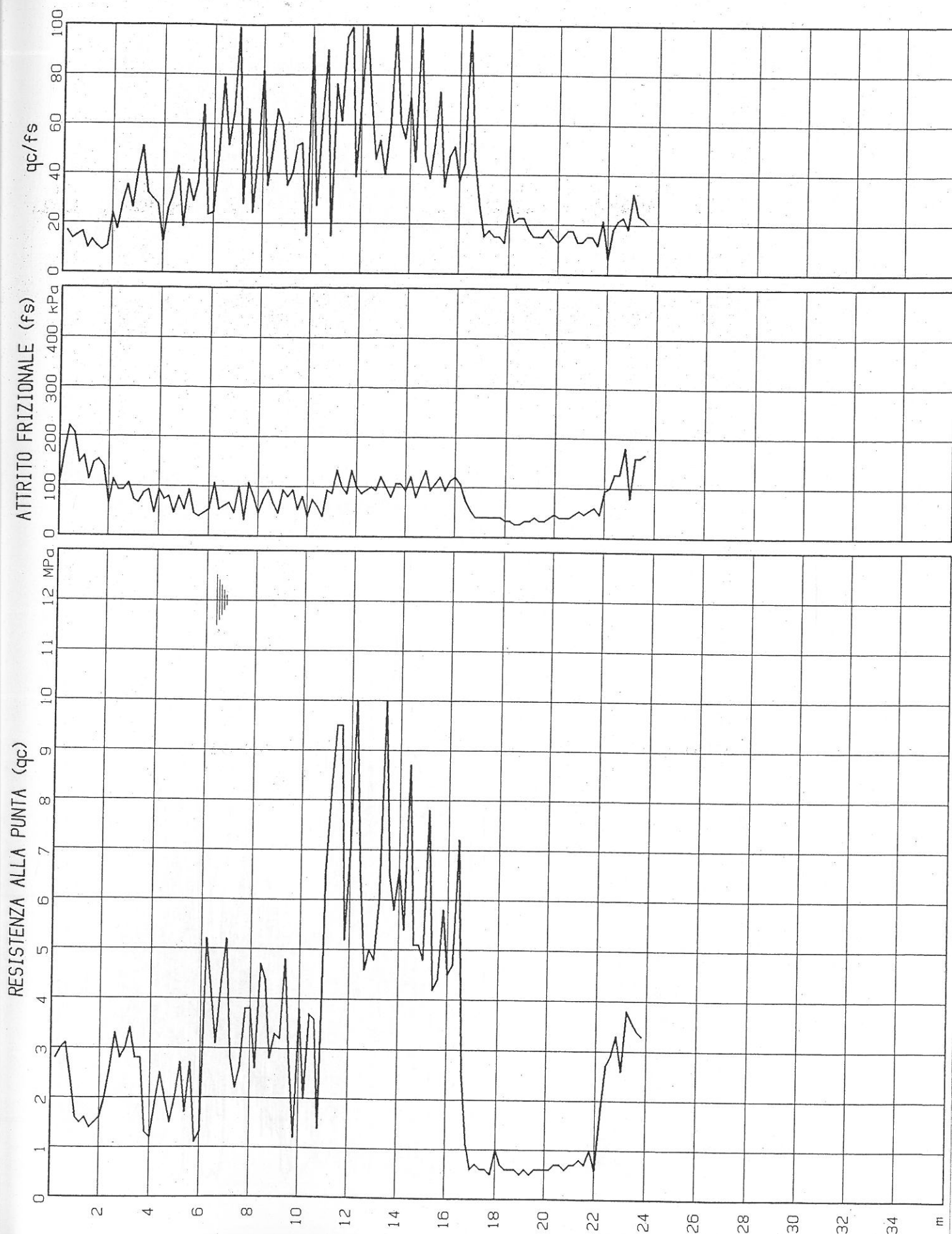


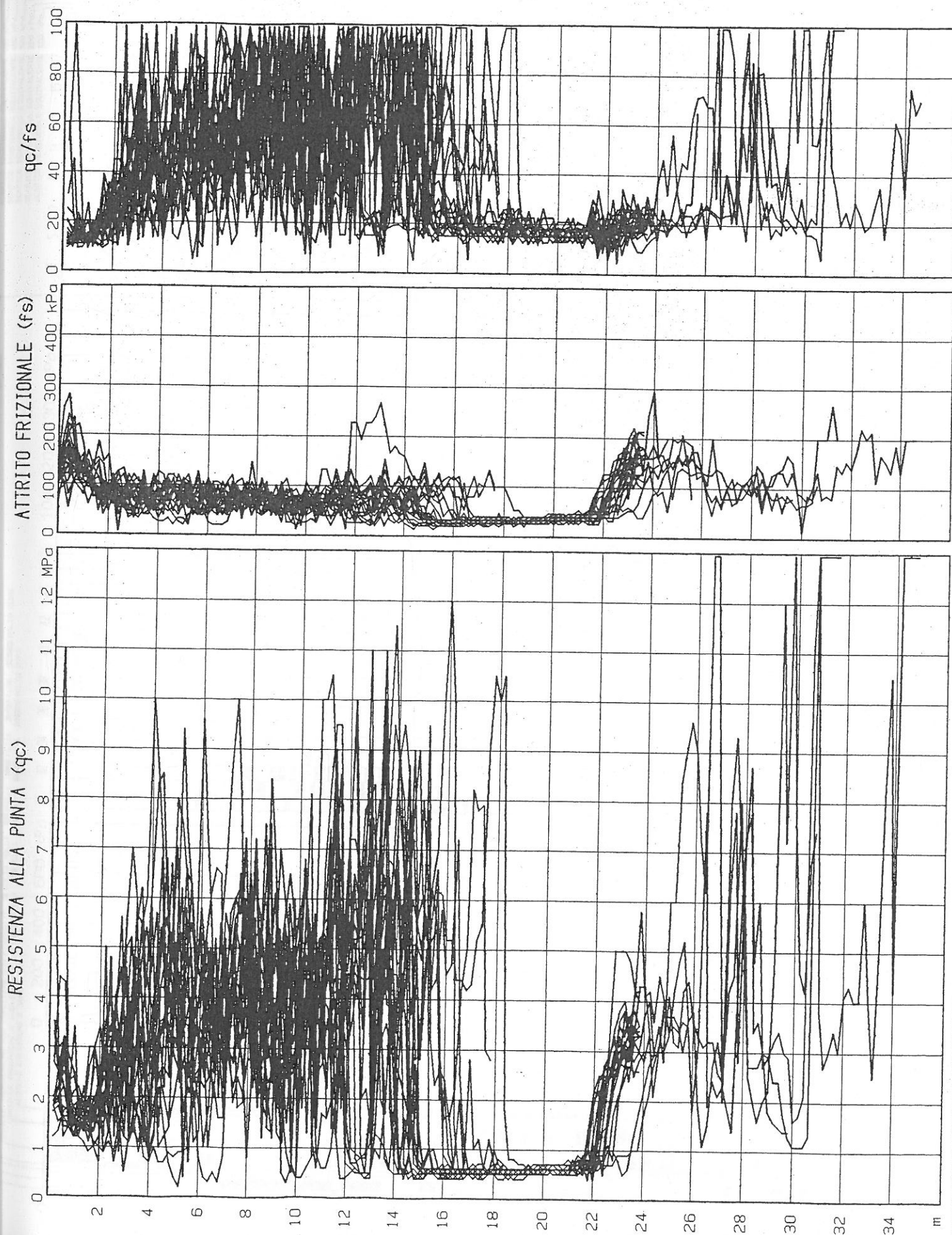


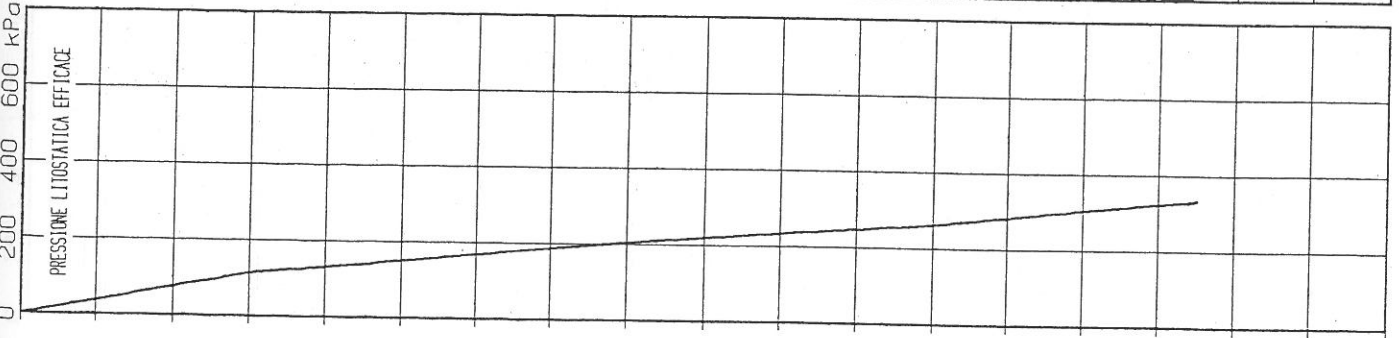
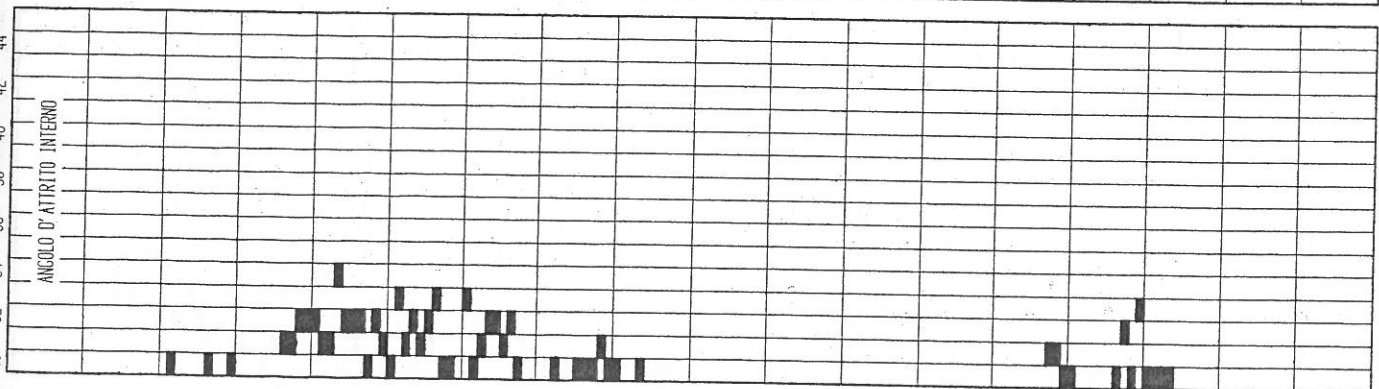
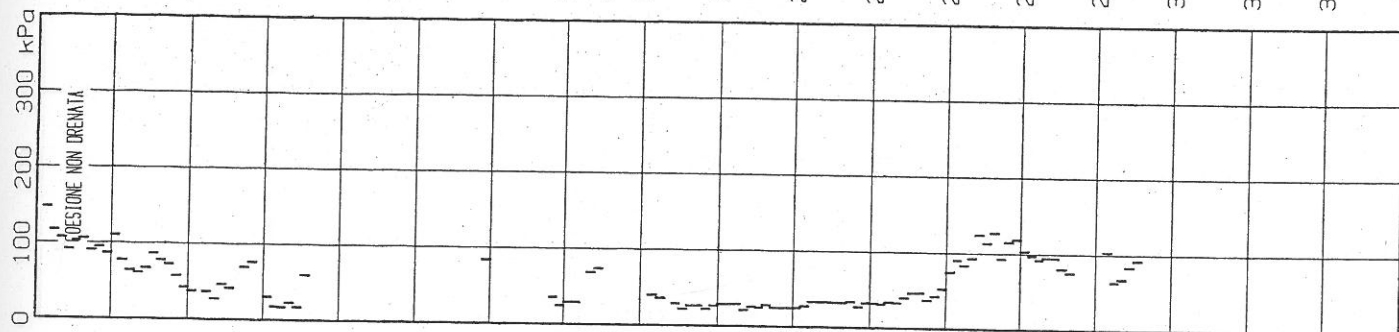
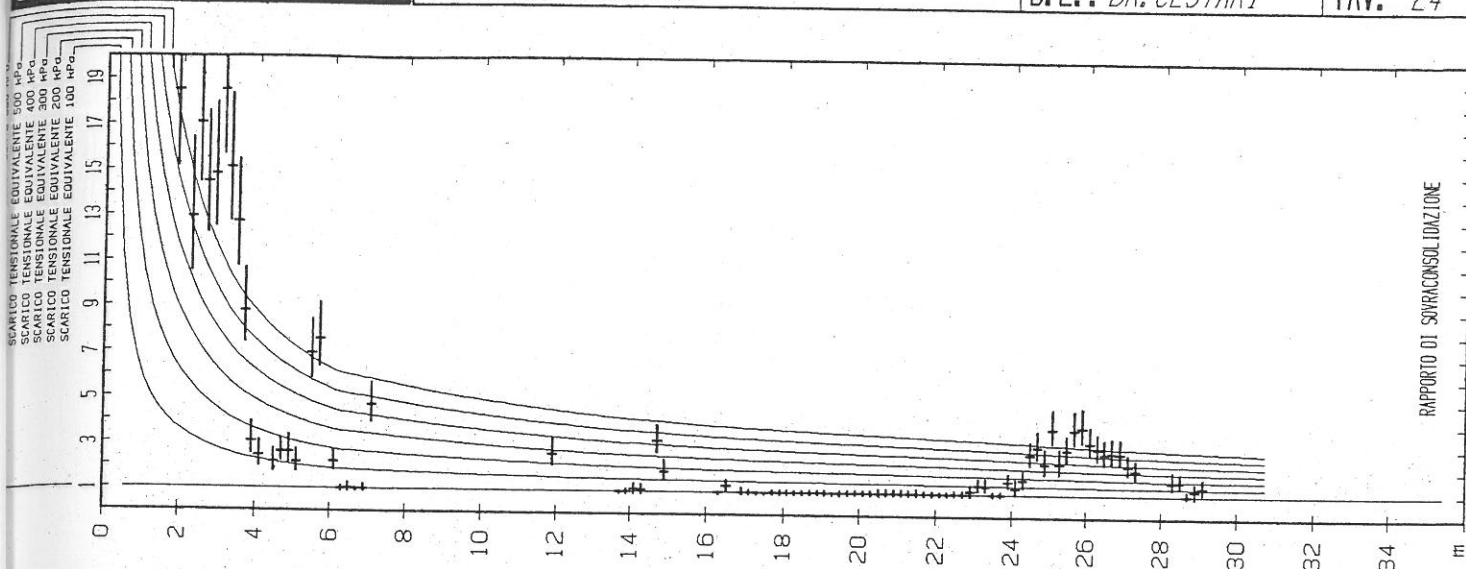






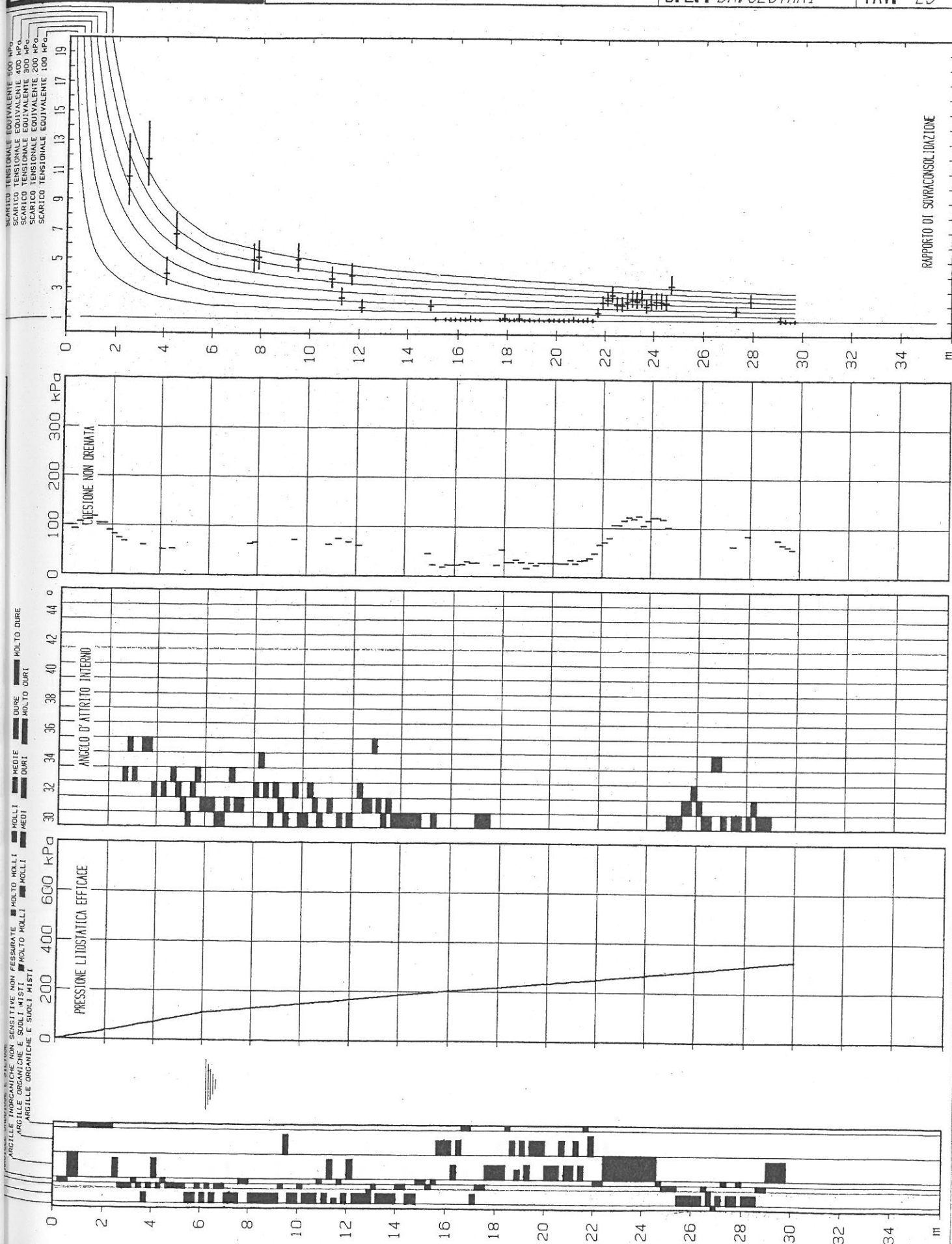


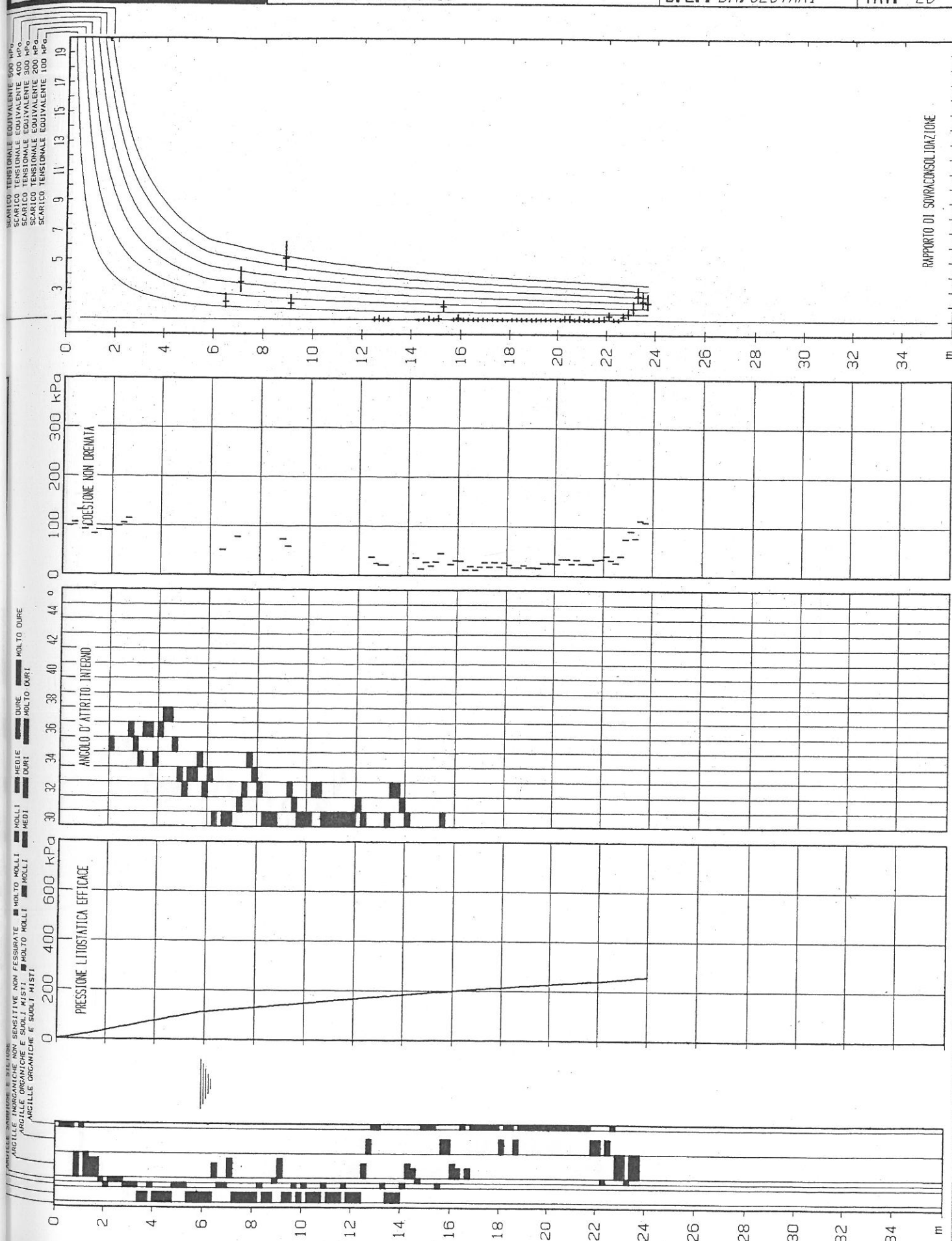


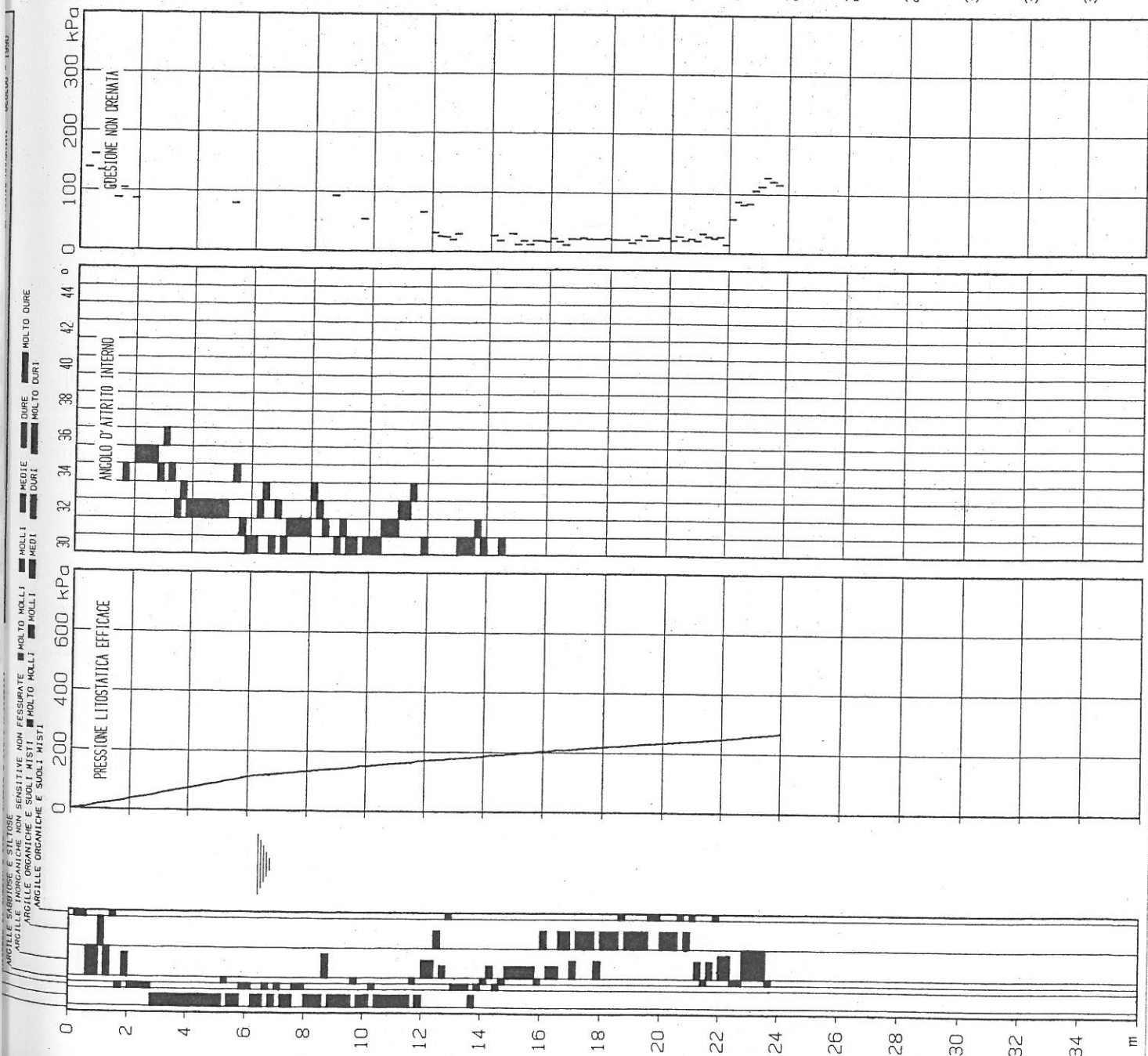
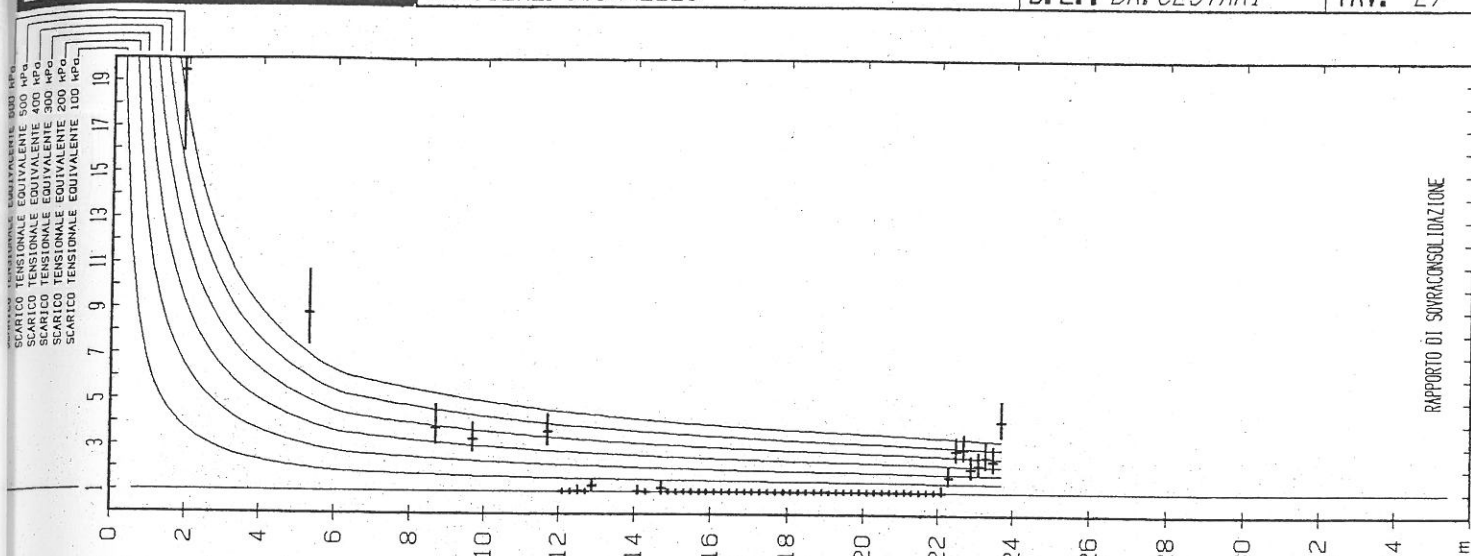


ARGILLE INORGANICHE NON SENSITIVE NON FESSURATE ■ MOLTO MOLLI ■ DURE ■ MOLTO DURE
 ARGILLE ORGANICHE E SUOLI MISTI ■ MOLTO MOLLI ■ DURI ■ MOLTO DURI
 ARGILLE ORGANICHE E SUOLI MISTI ■ MOLTI ■ MEDI ■ DURI ■ MOLTO DURI

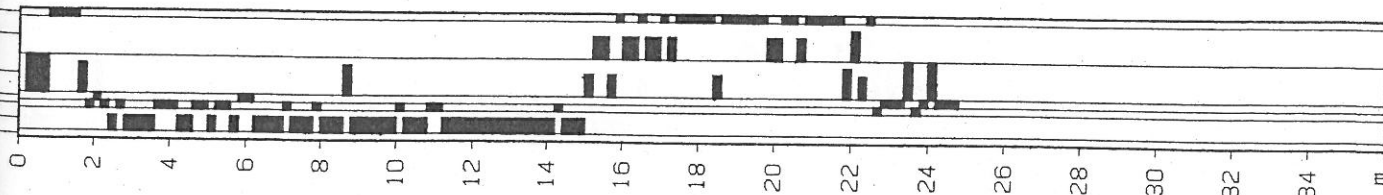
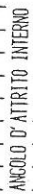
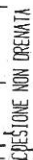




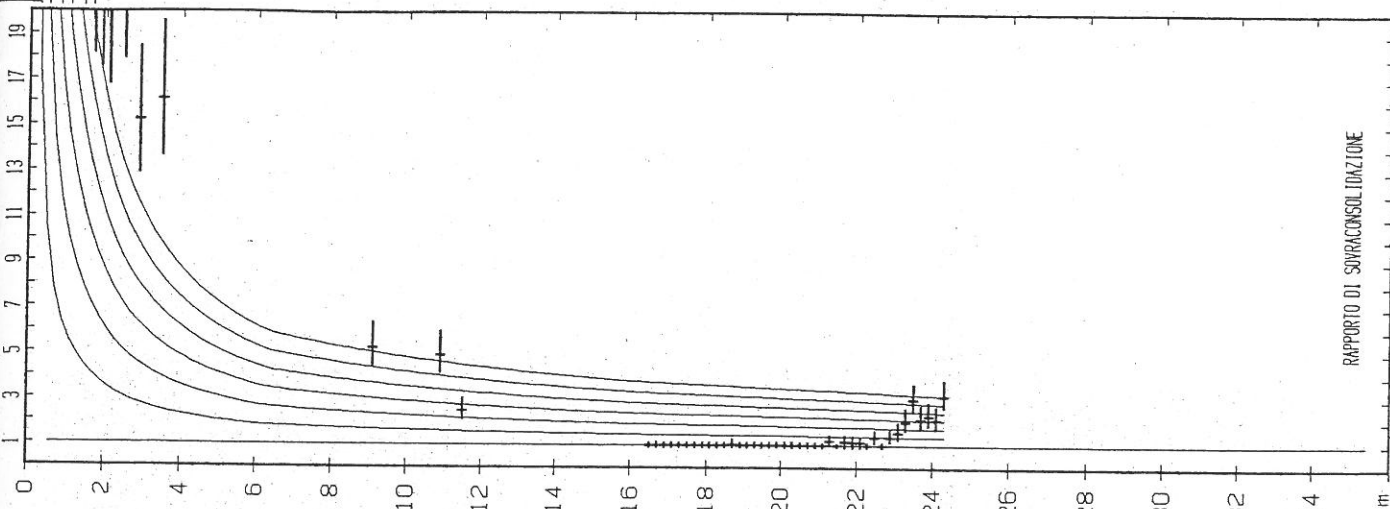




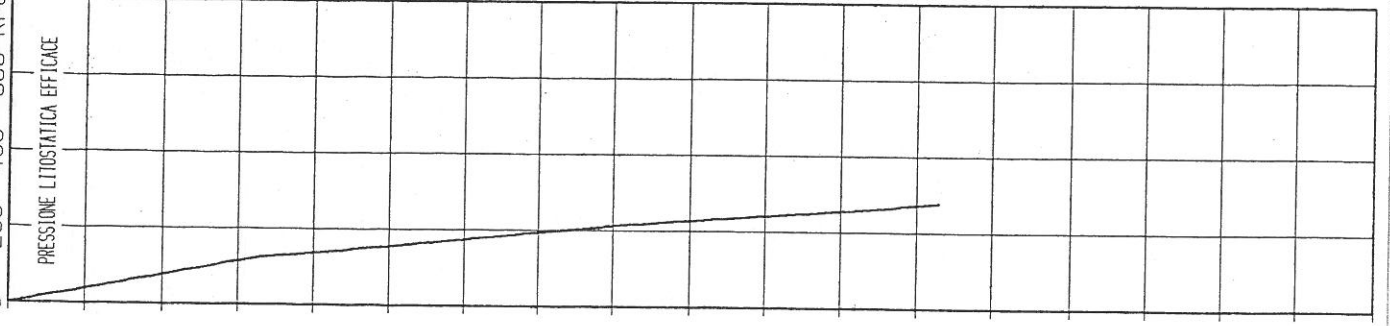
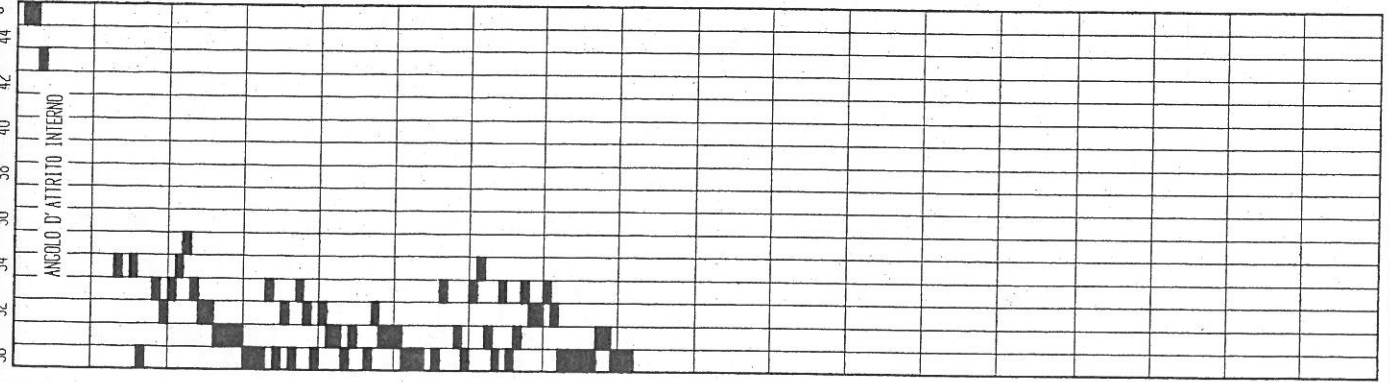
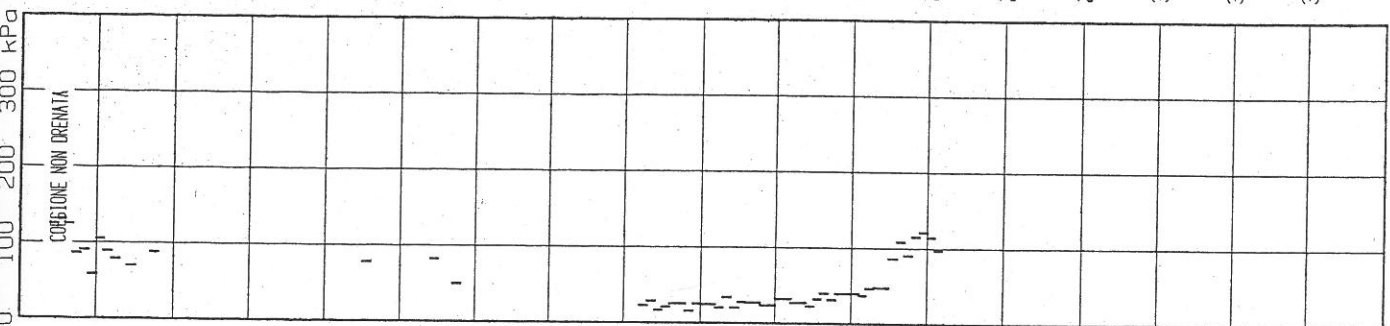




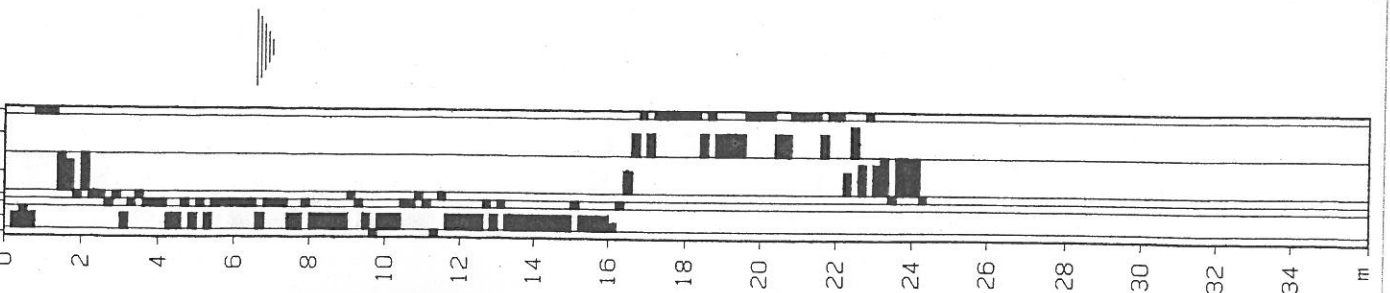
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SCARICO TENSIONALE EQUIVALENTE 400 MPa
SCARICO TENSIONALE EQUIVALENTE 300 MPa
SCARICO TENSIONALE EQUIVALENTE 200 MPa
SCARICO TENSIONALE EQUIVALENTE 100 MPa

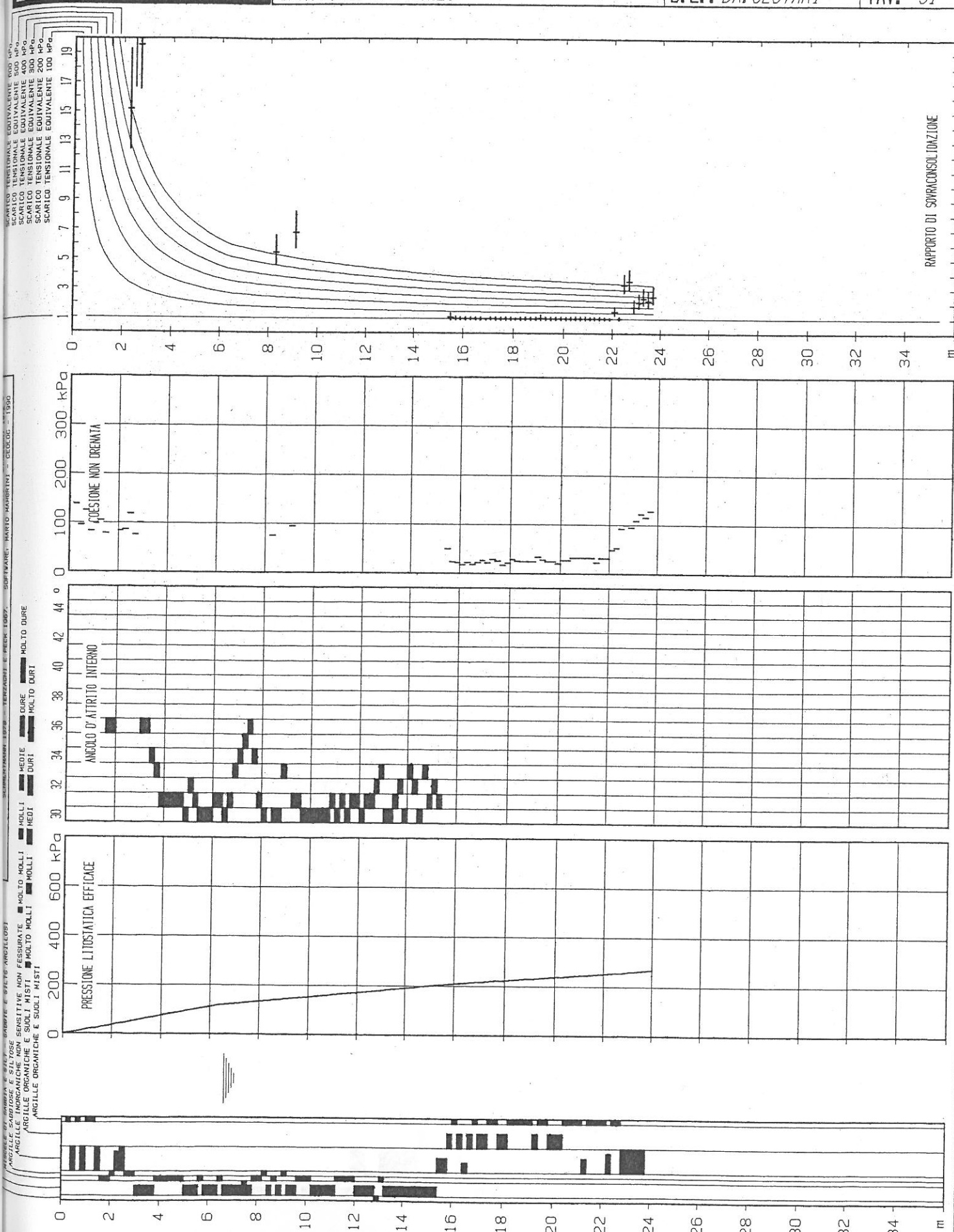


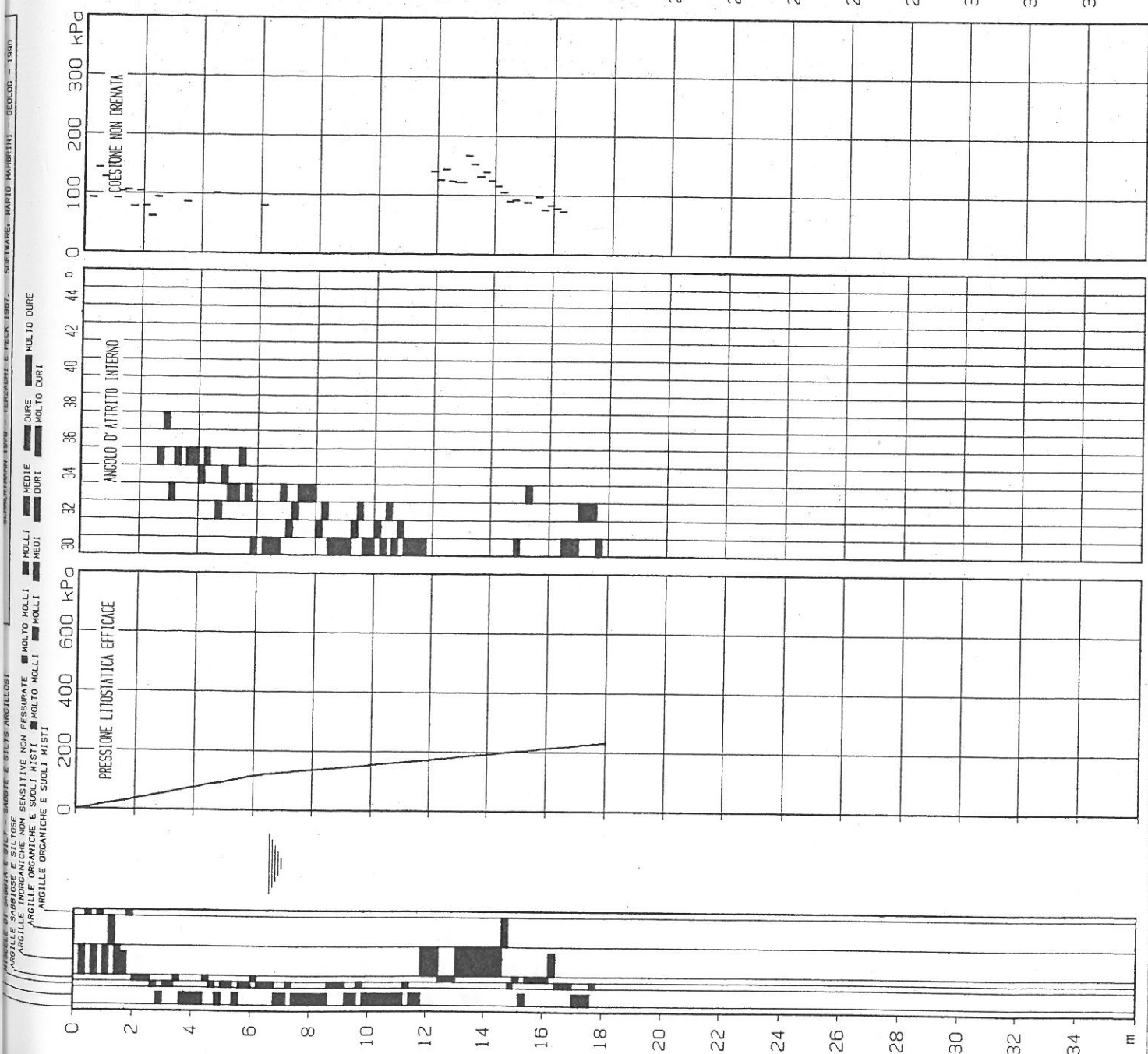
RAPPORTO DI SOVRACONSOLIDAZIONE

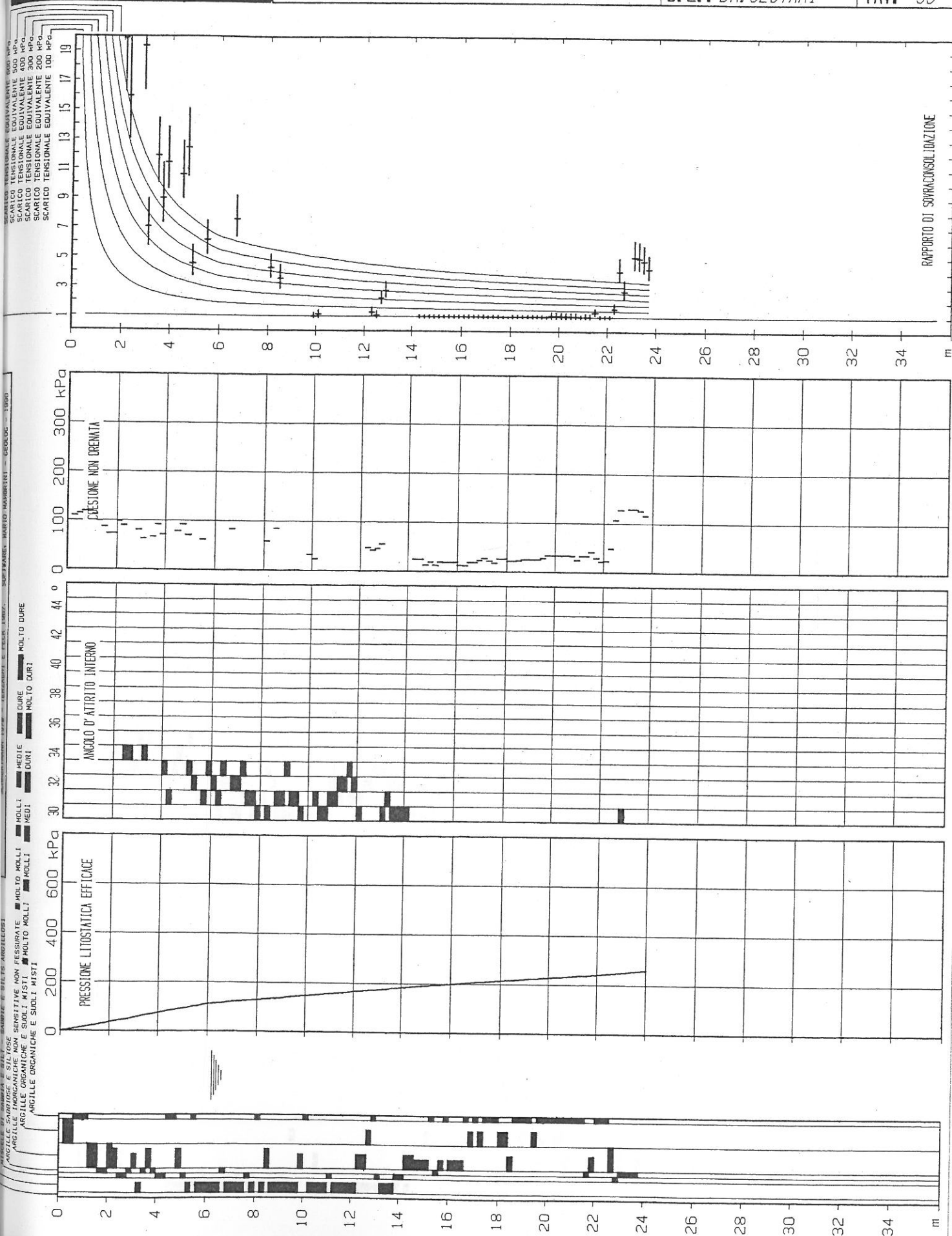


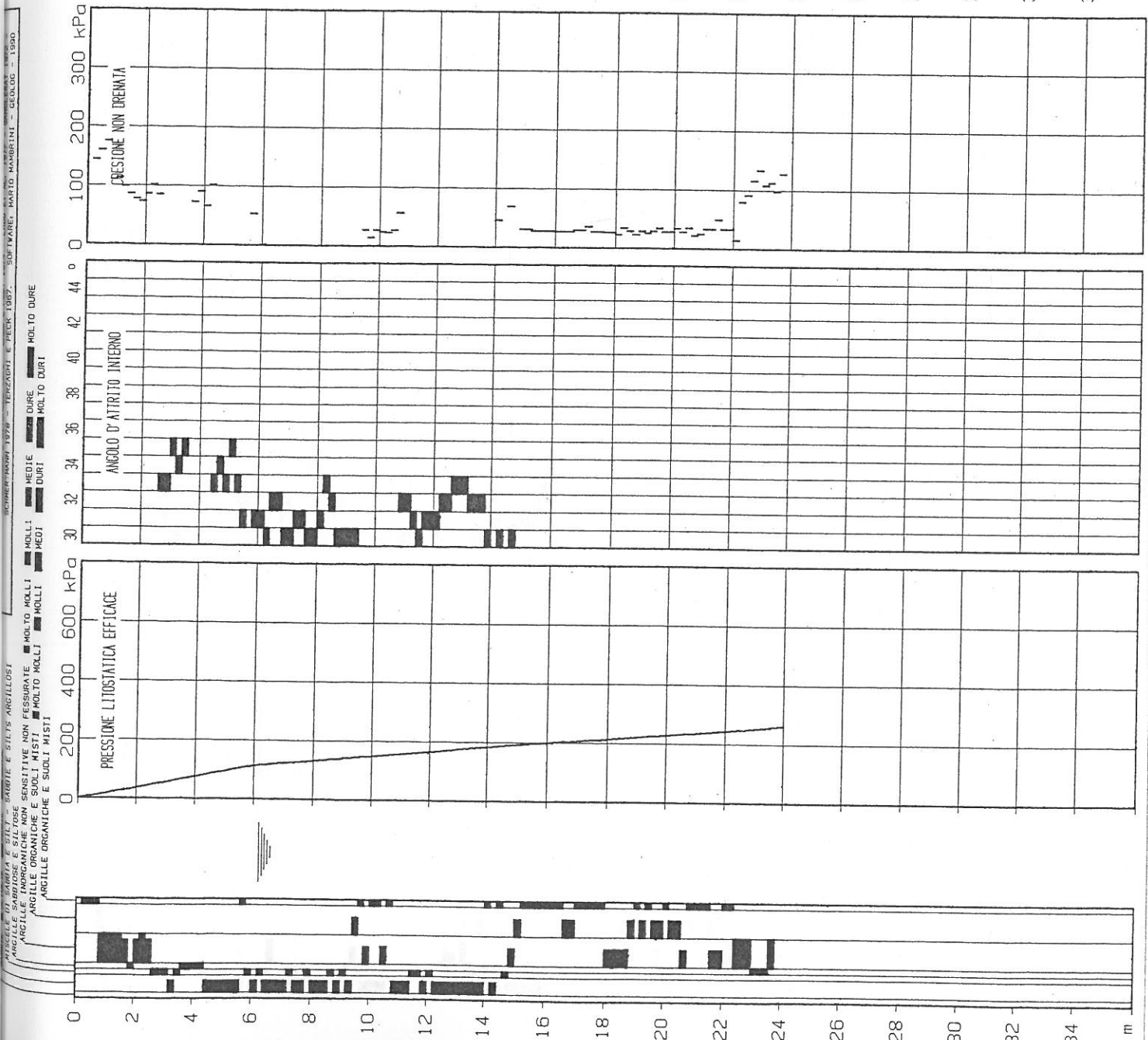
ARGILLE SABBIOSE E SILTLOSE
ARGILLE INORGANICHE NON FESSURATE
ARGILLE ORGANICHE E SUOLI MISTI
ARGILLE ORGANICHE E SUOLI MISTI





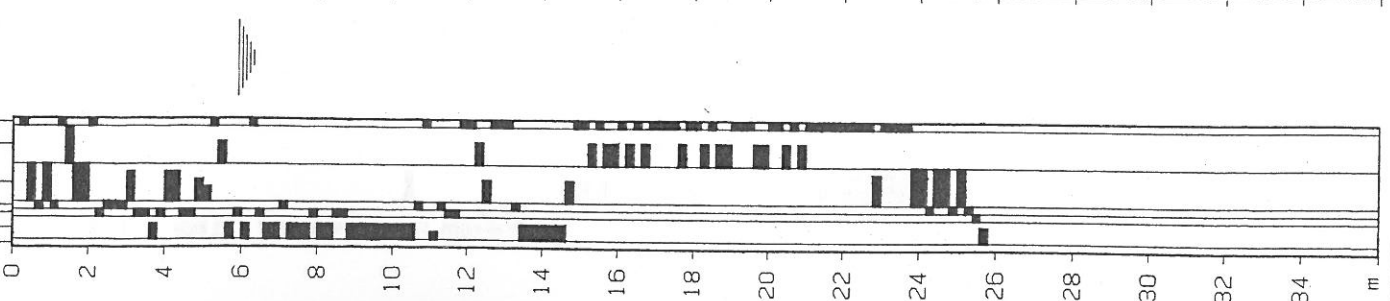
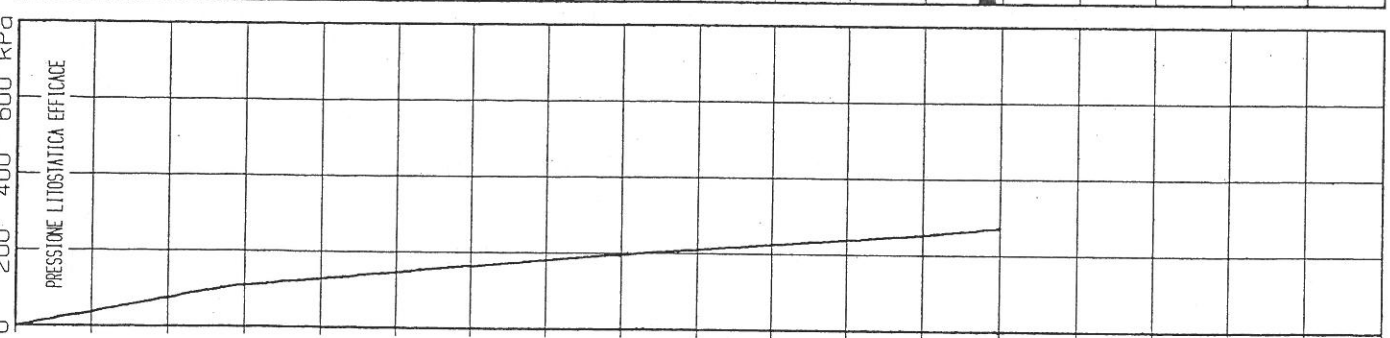
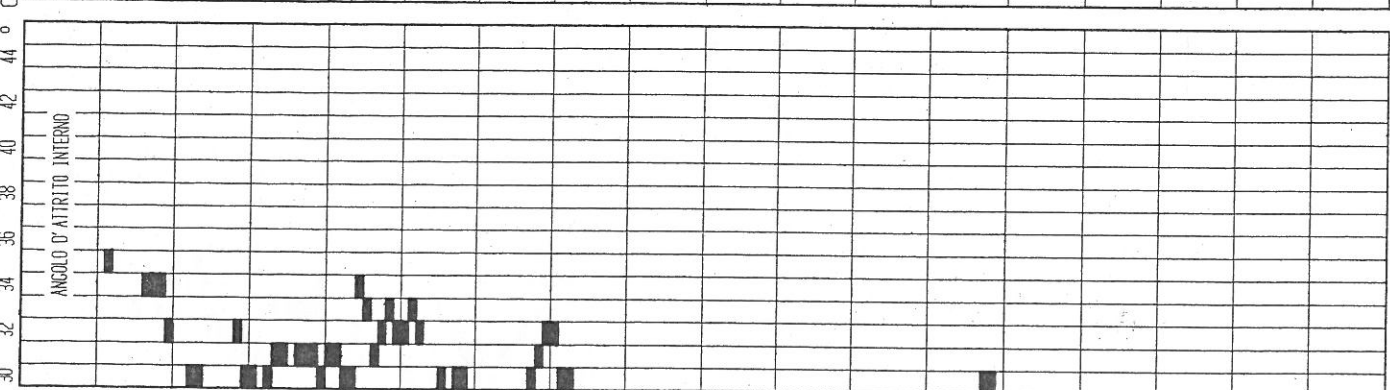
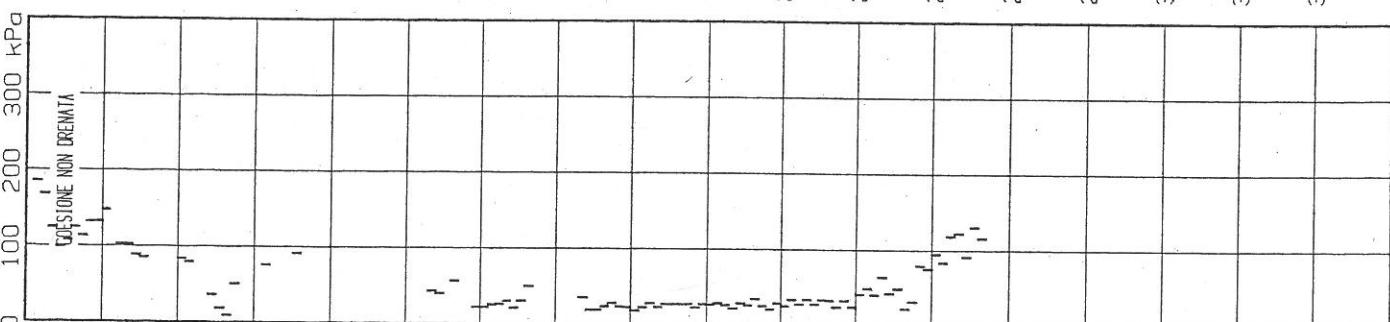
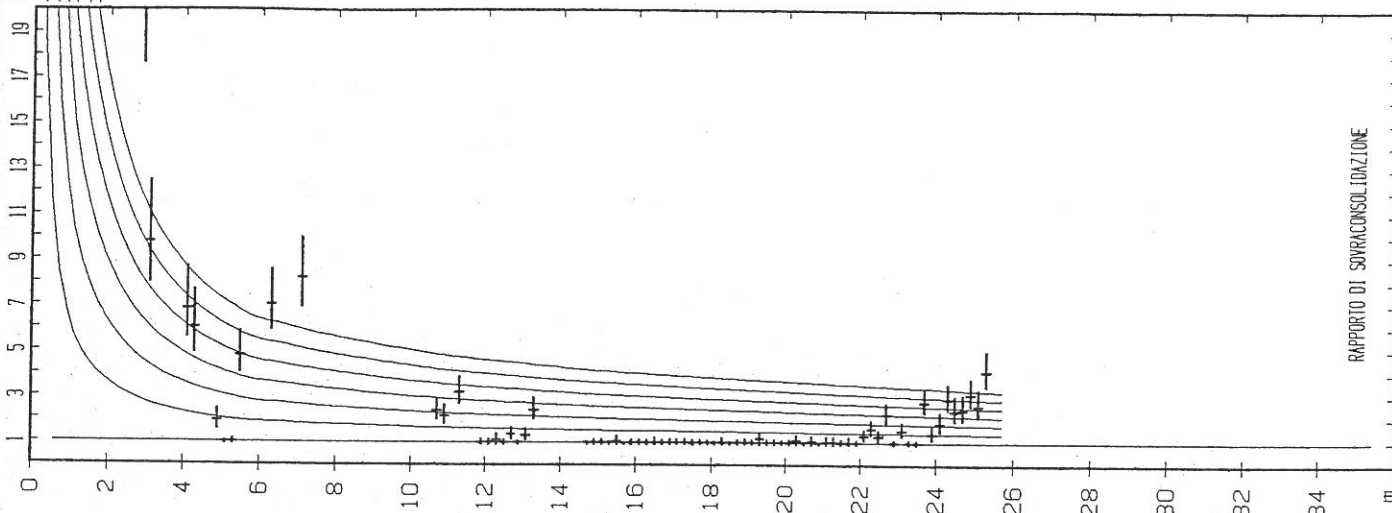


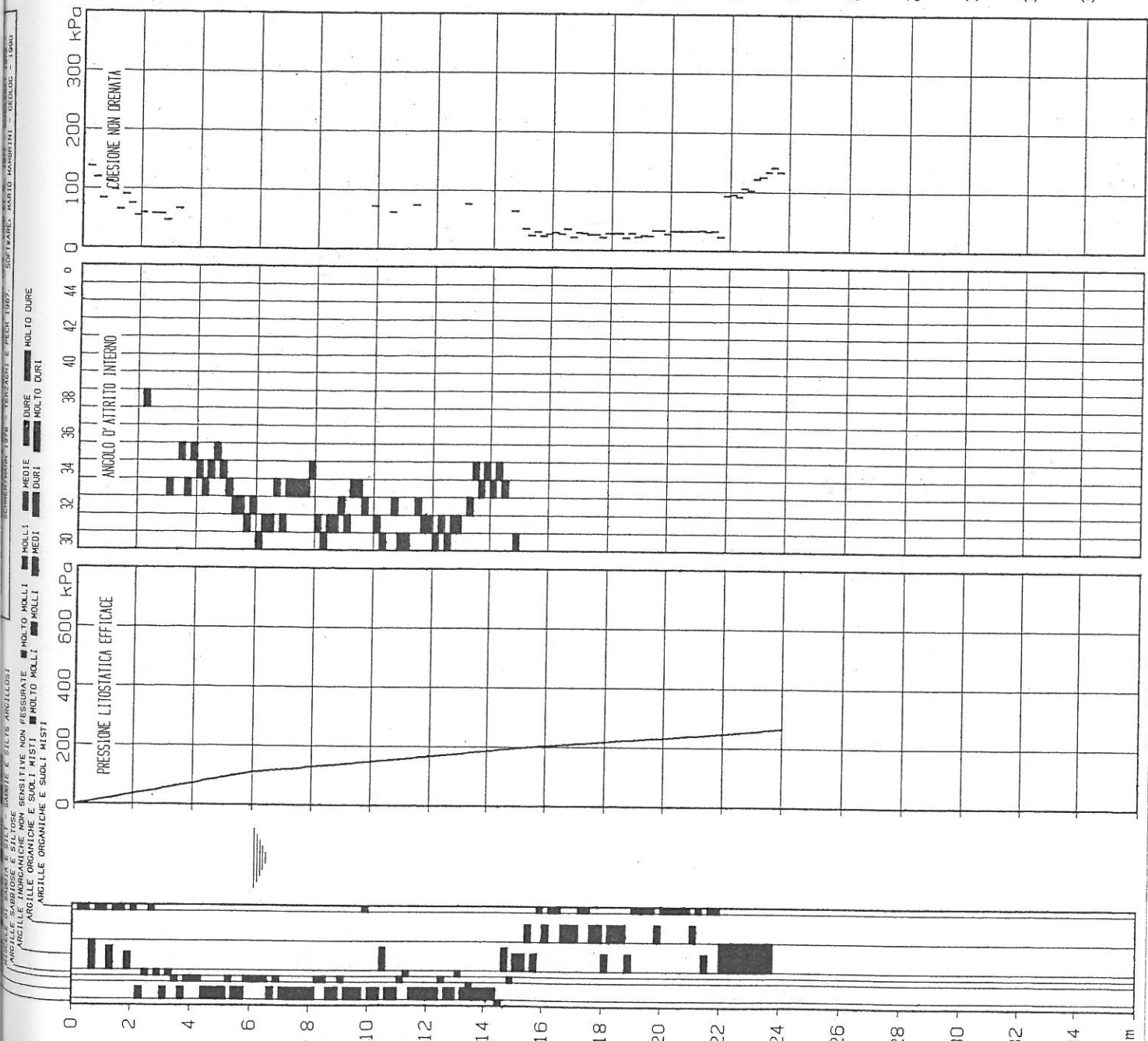


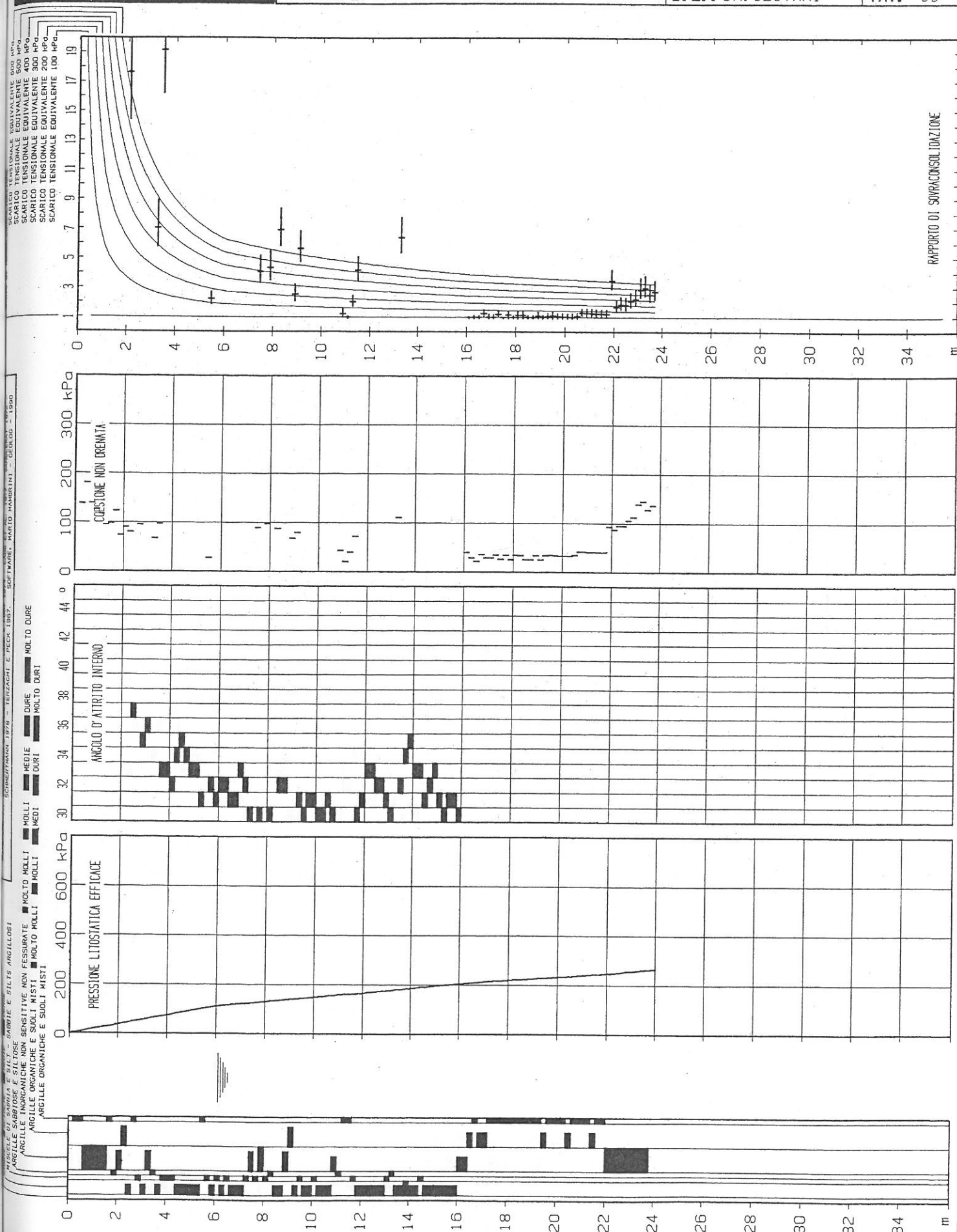


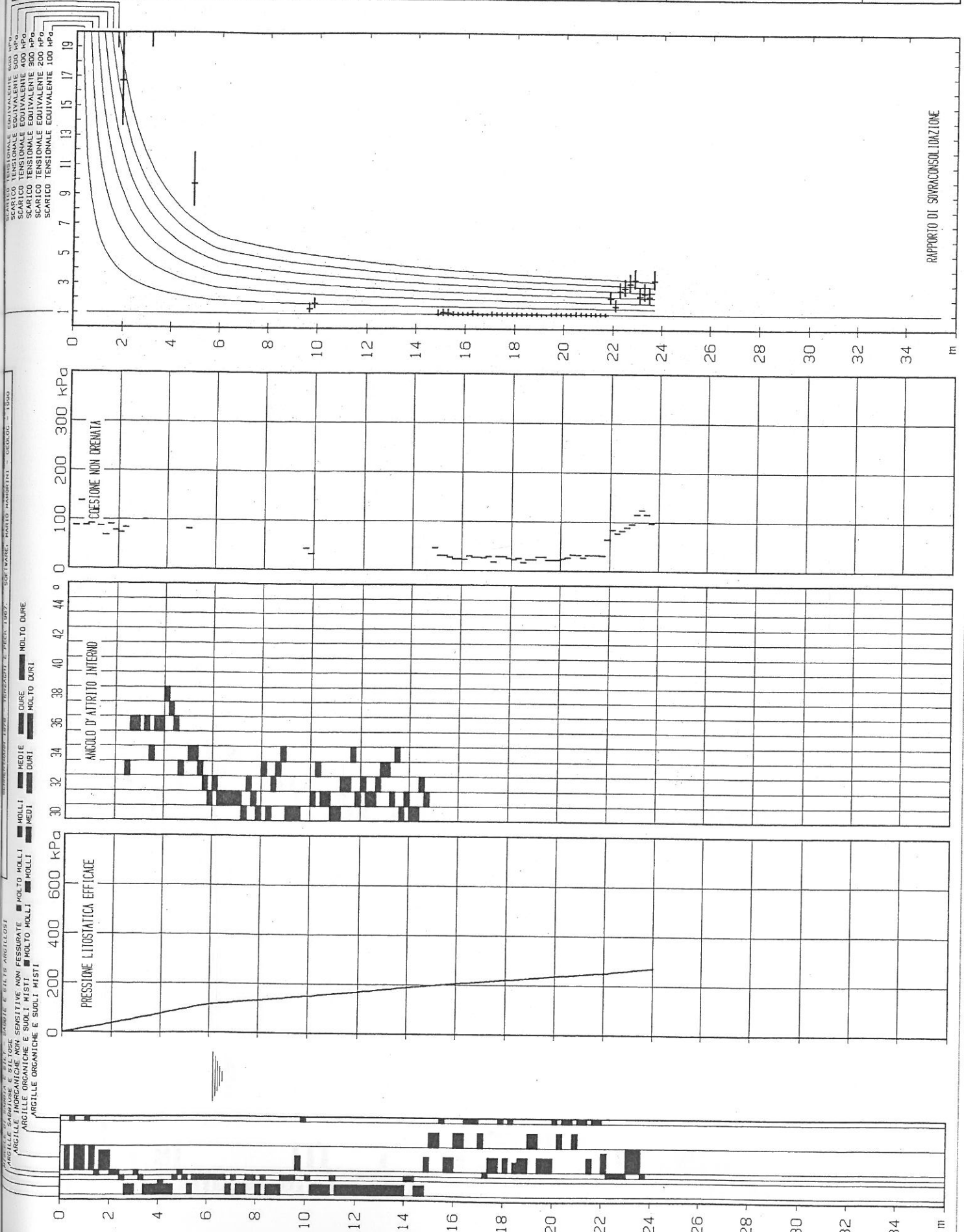


SCARICO TENSIONIALE EQUIVALENTE 600 MPa
SCARICO TENSIONIALE EQUIVALENTE 500 MPa
SCARICO TENSIONIALE EQUIVALENTE 400 MPa
SCARICO TENSIONIALE EQUIVALENTE 300 MPa
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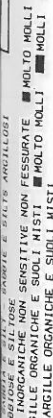


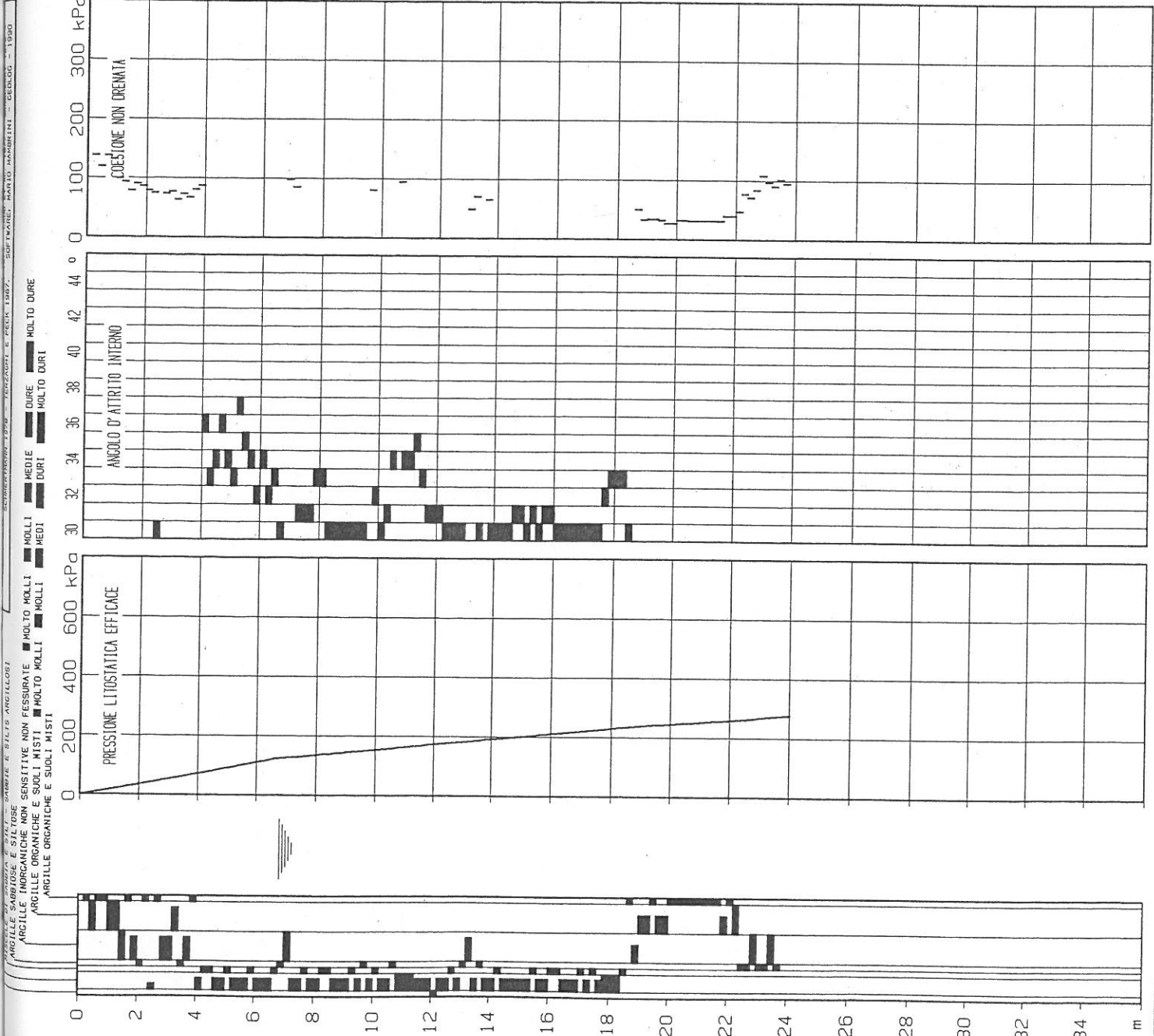


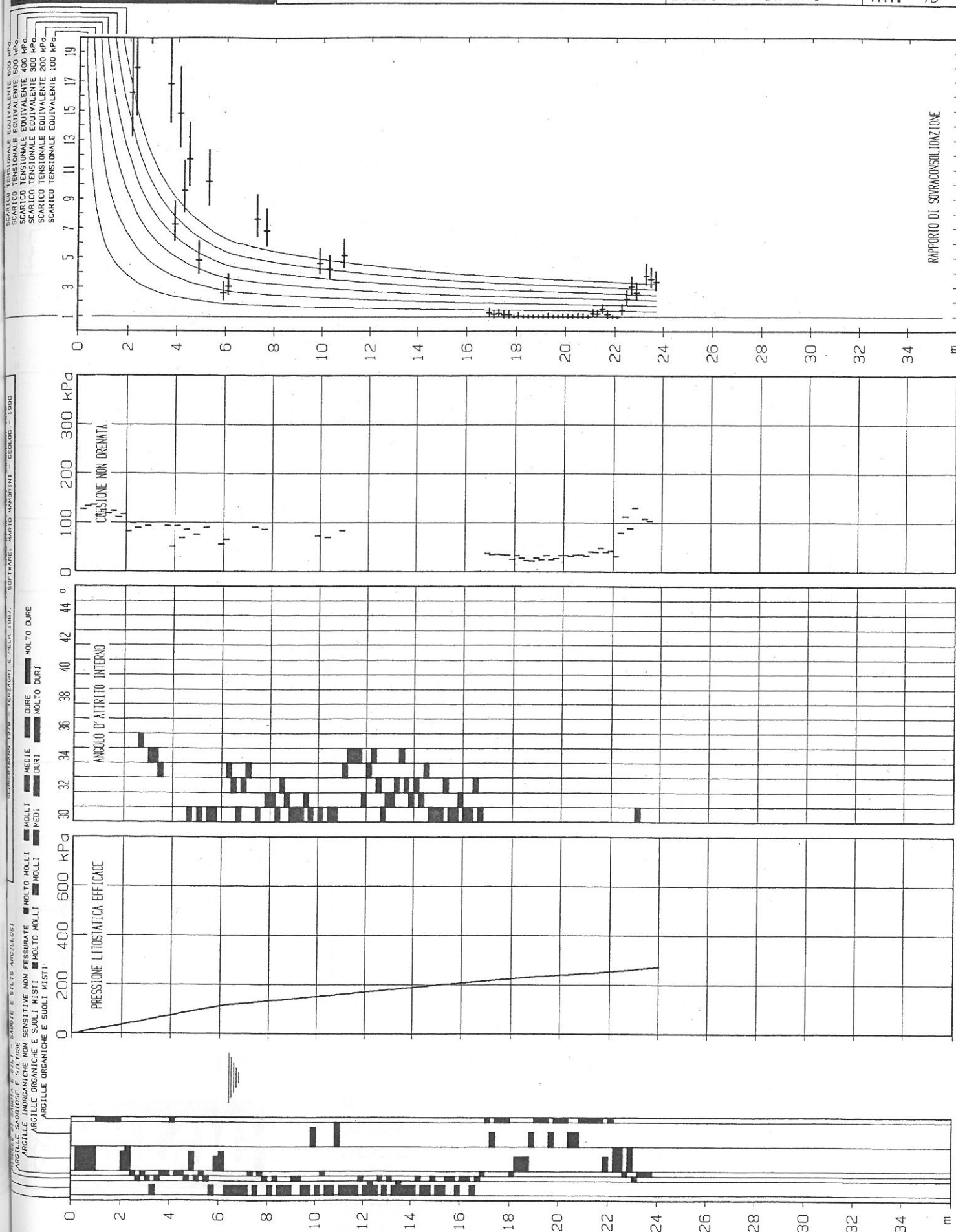


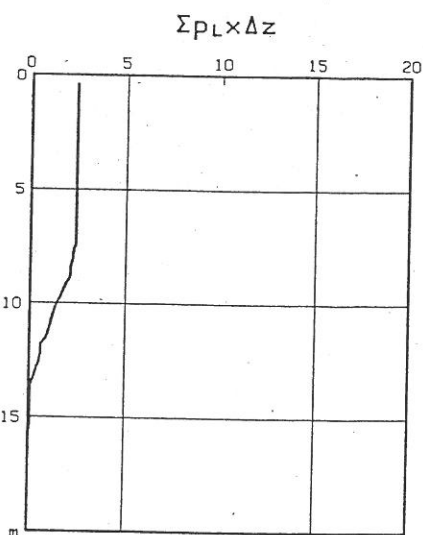
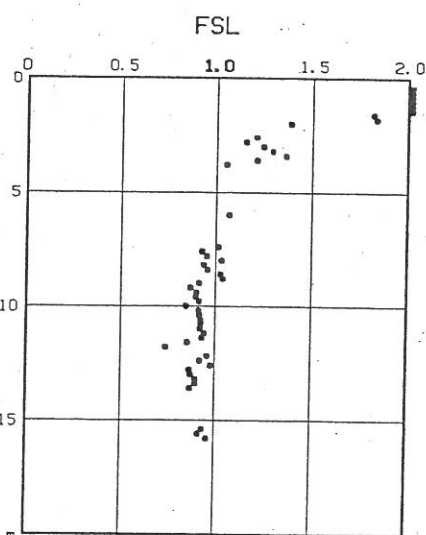
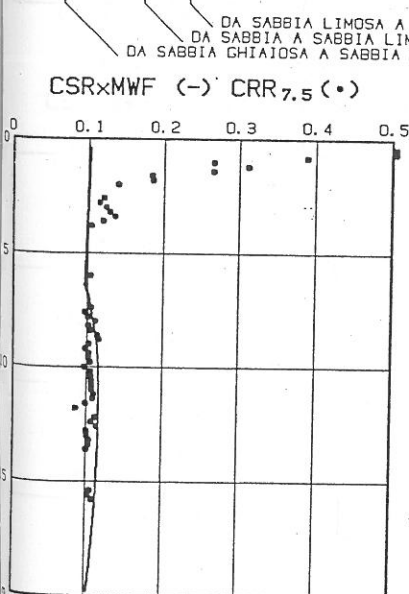
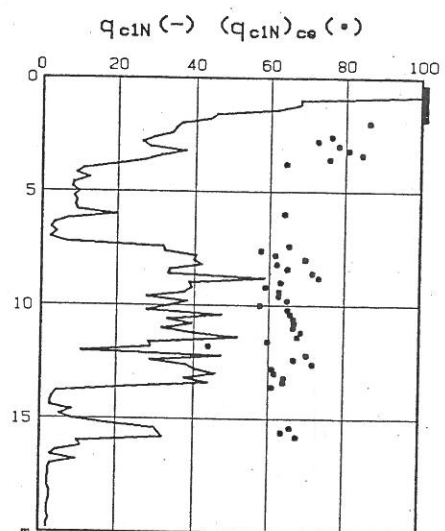
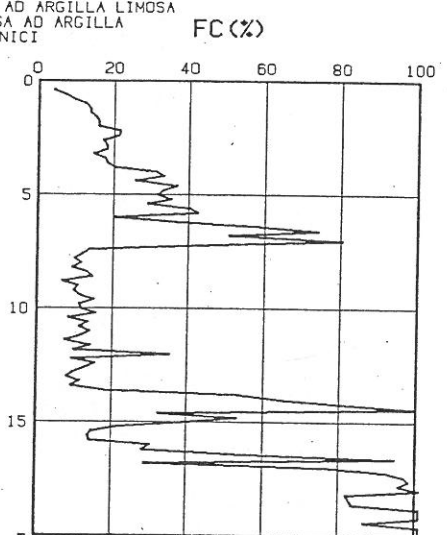
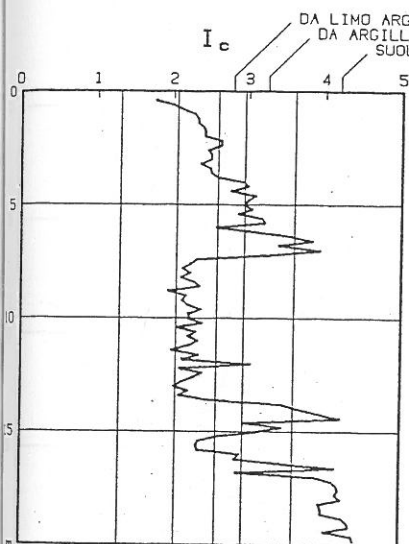
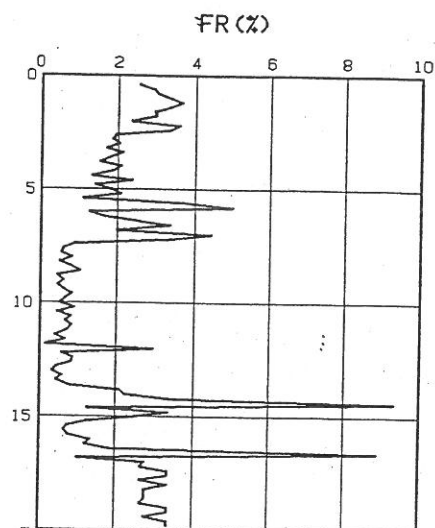
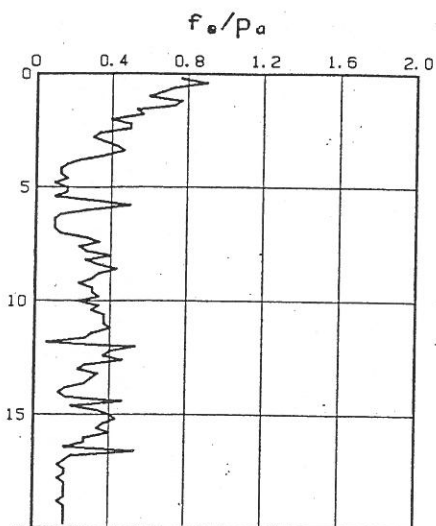
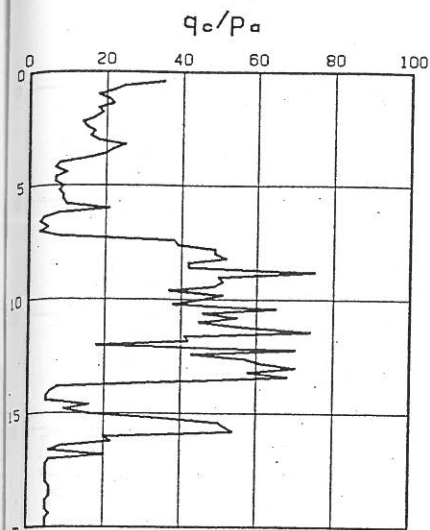










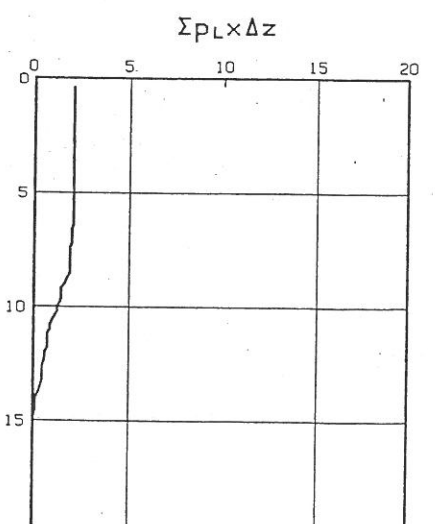
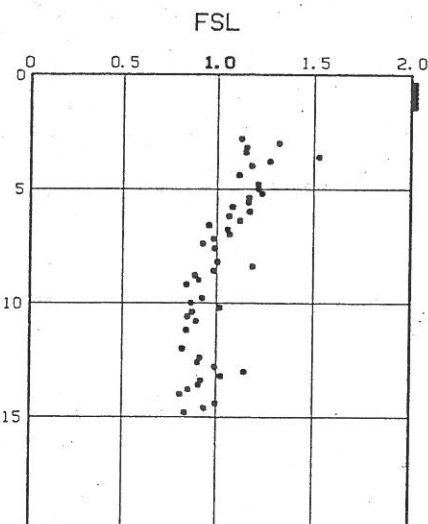
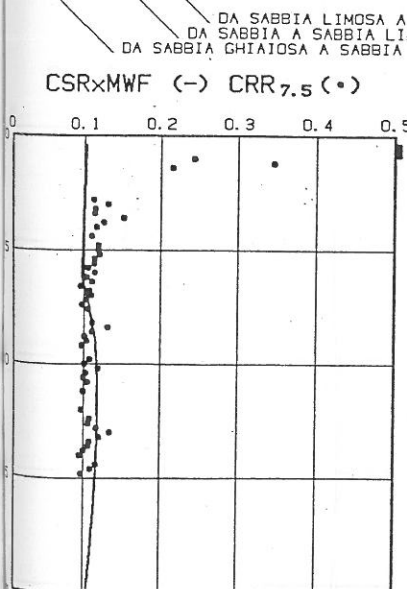
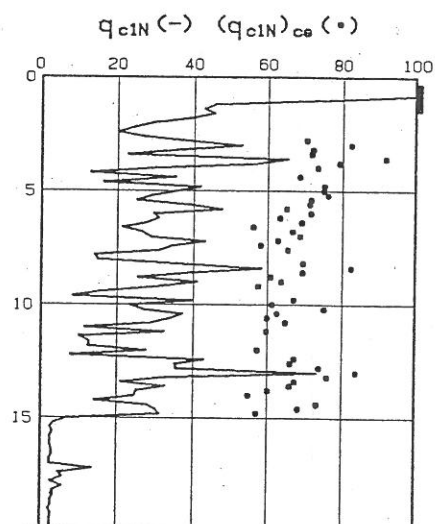
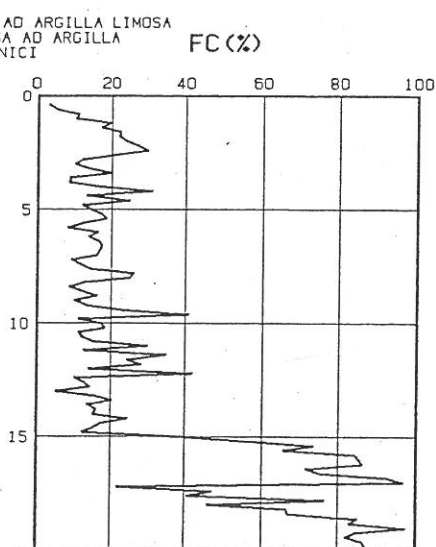
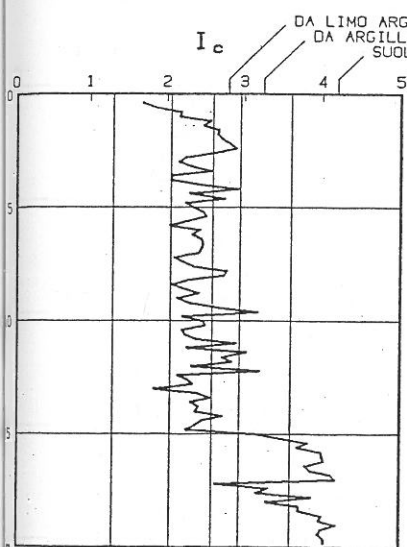
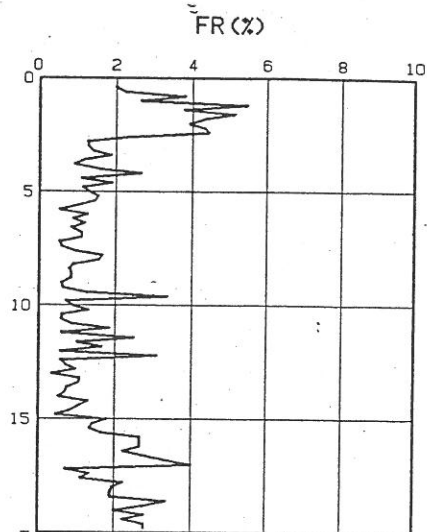
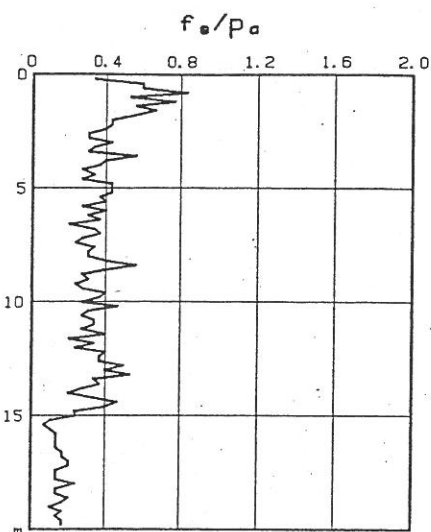
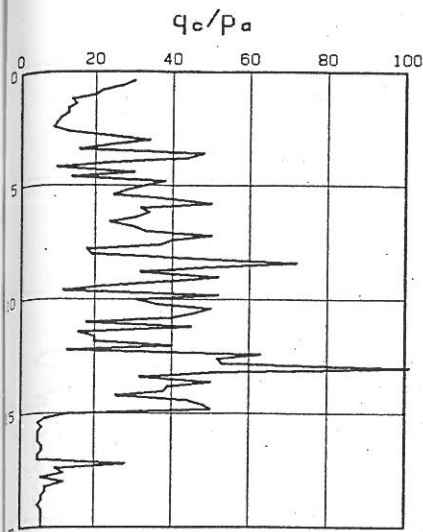


BASSO ALTO MOLTO ALTO
RISCHIO DI LIQUEFAZIONE

geoLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT1
LOCALITA: PISA
CANTIERE: CISANELLO

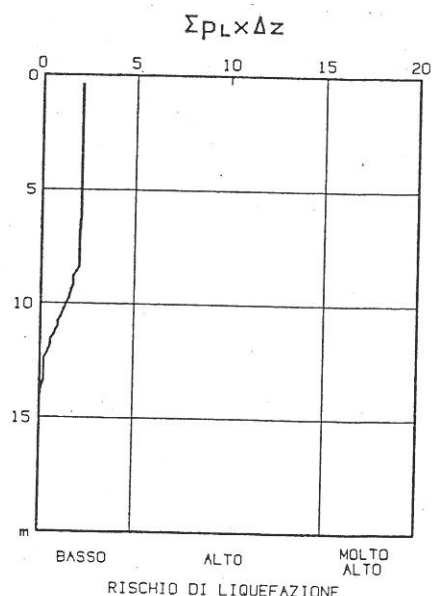
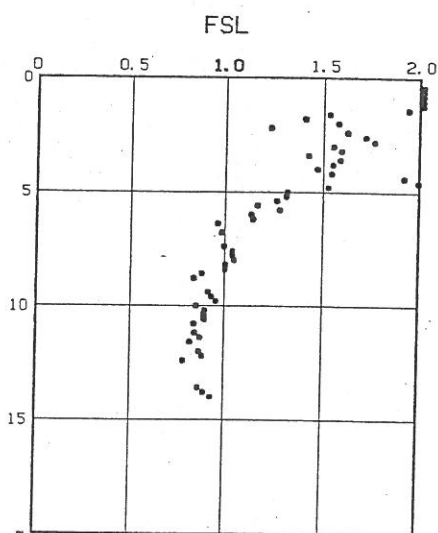
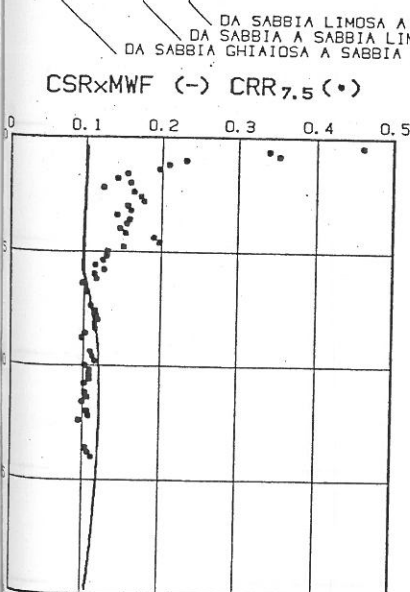
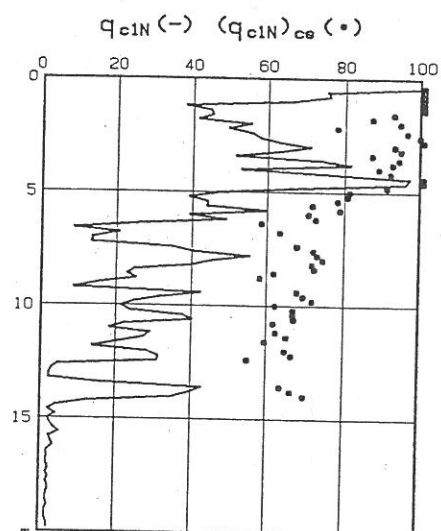
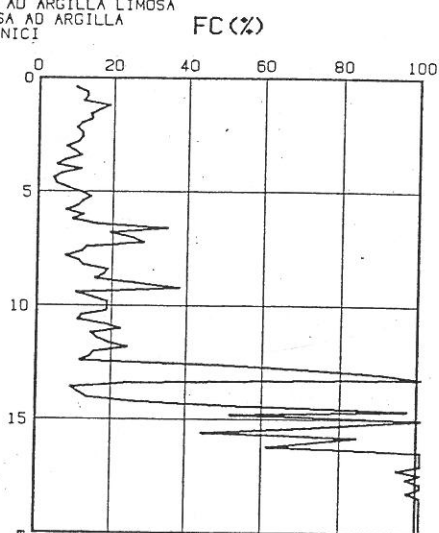
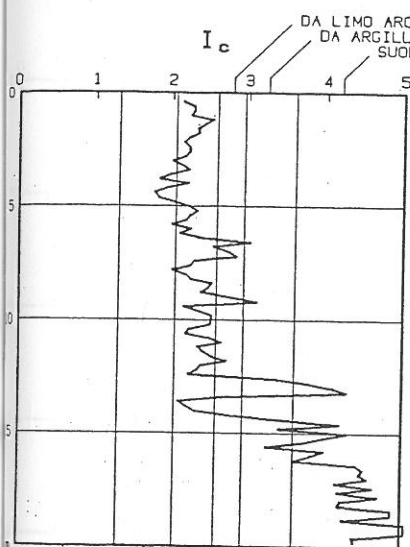
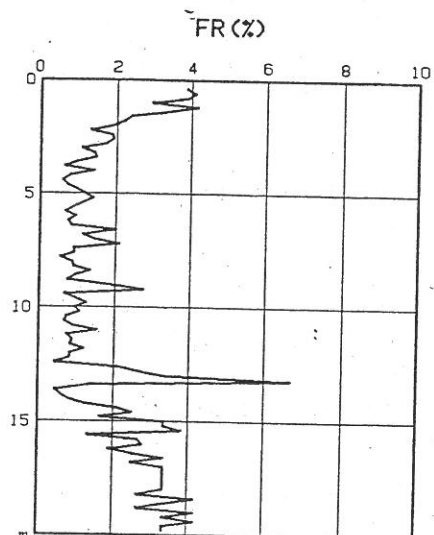
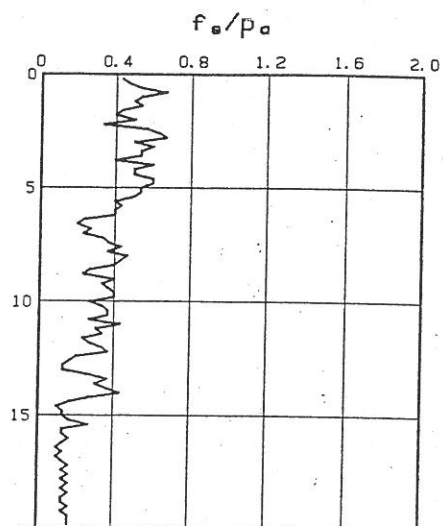
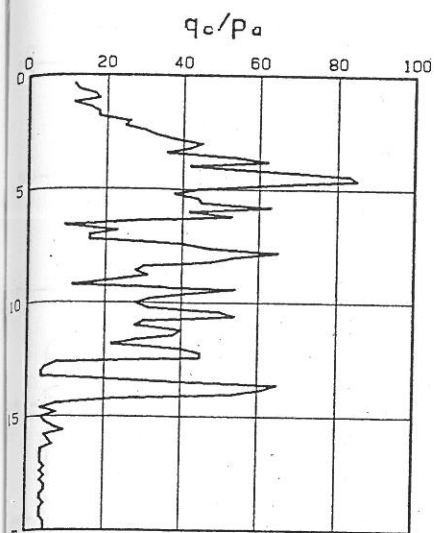
TAV. 44



geoLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT2
LOCALITA: PISA
CANTIERE: CISANELLO

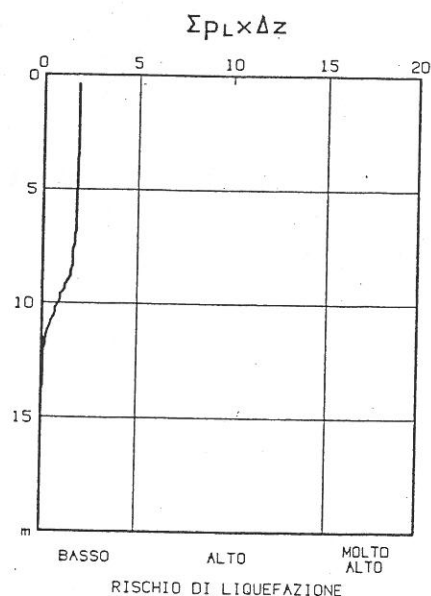
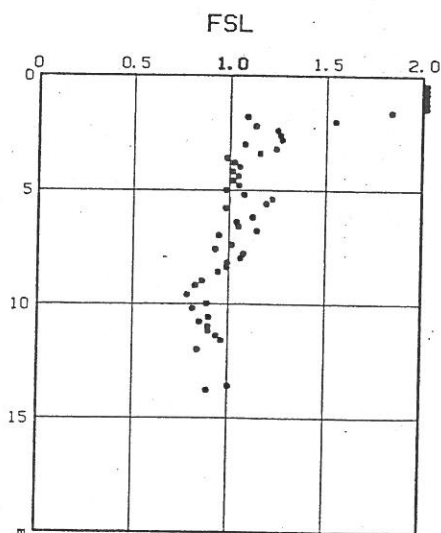
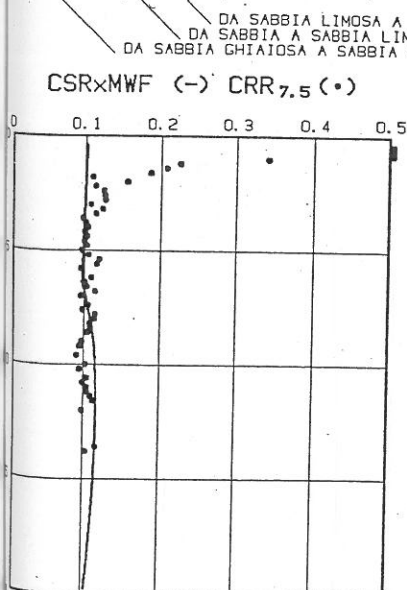
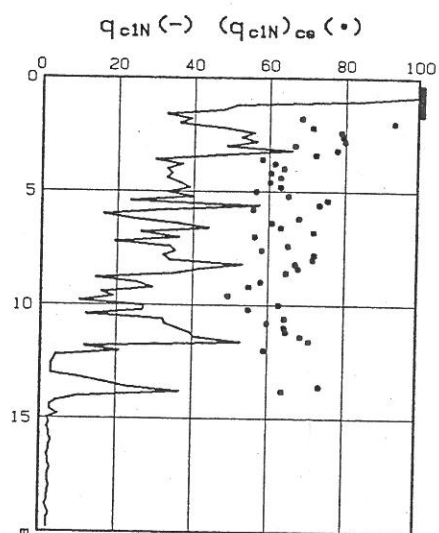
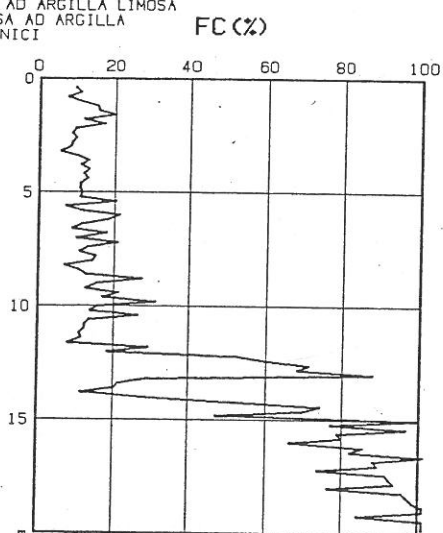
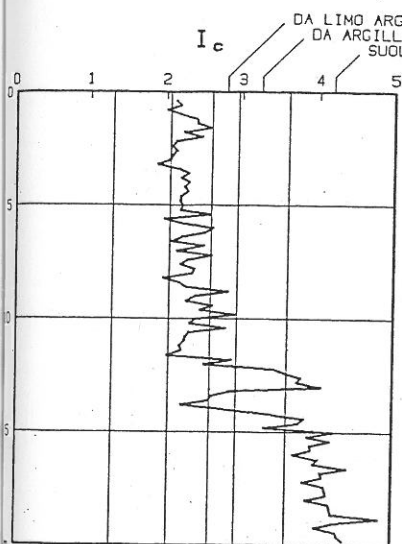
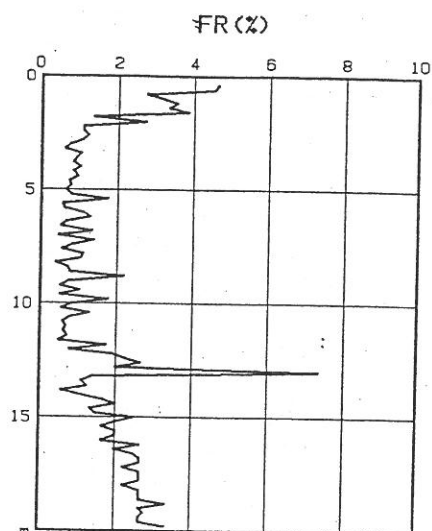
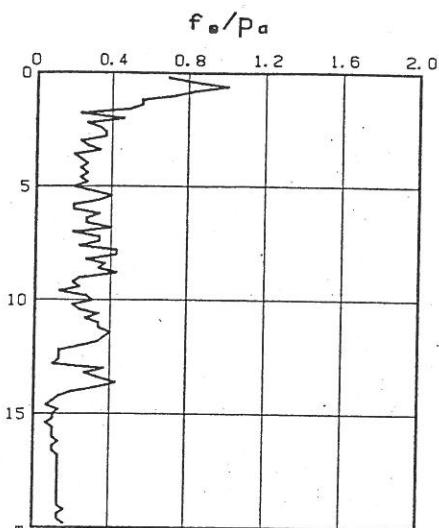
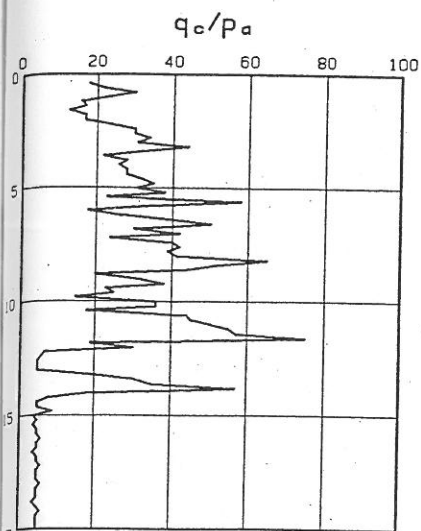
BASSO ALTO MOLTO ALTO
RISCHIO DI LIQUEFAZIONE



geoLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT3
LOCALITA: PISA
CANTIERE: CISANELLO

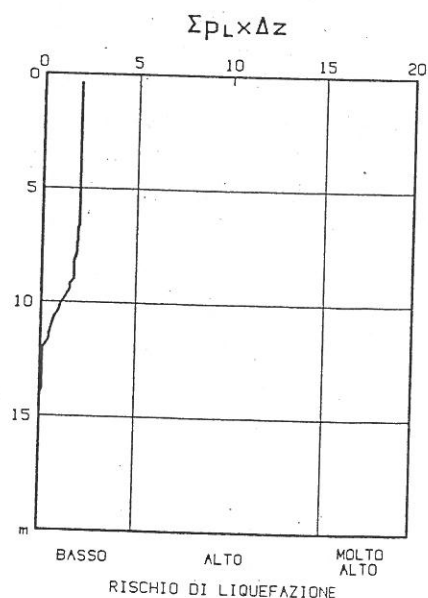
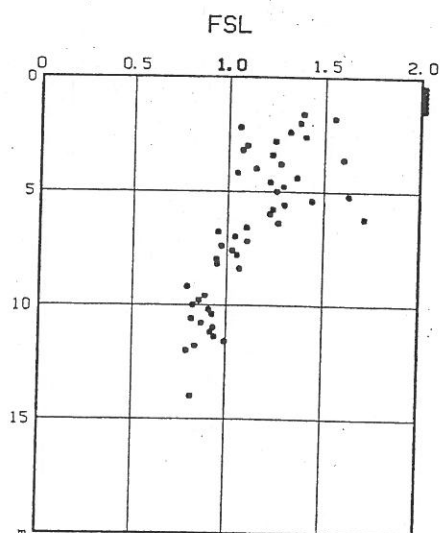
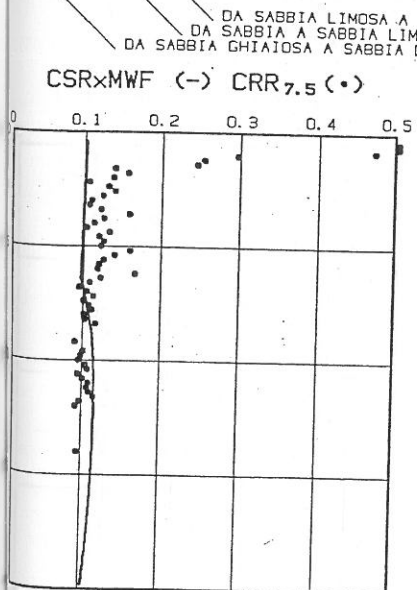
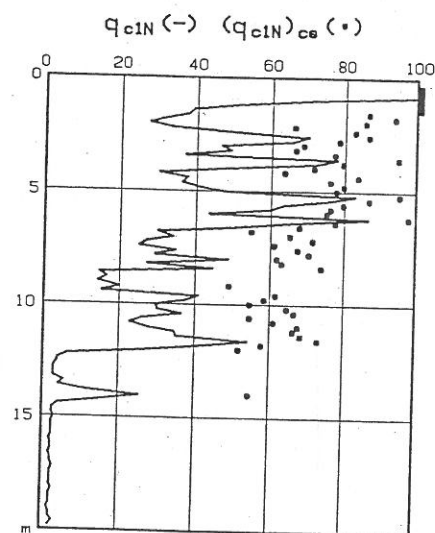
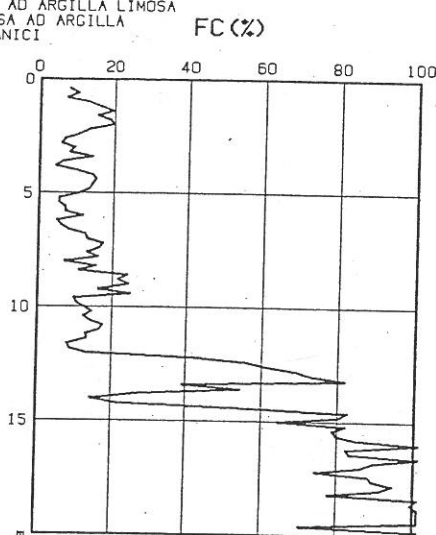
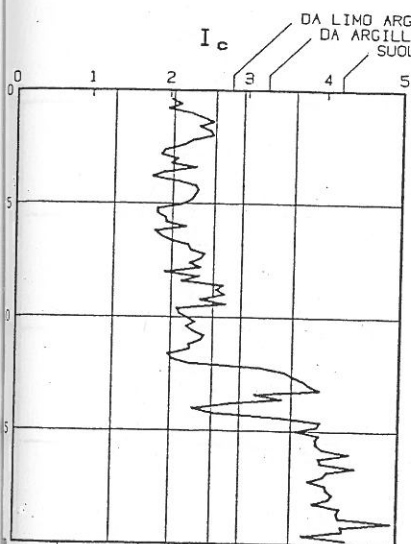
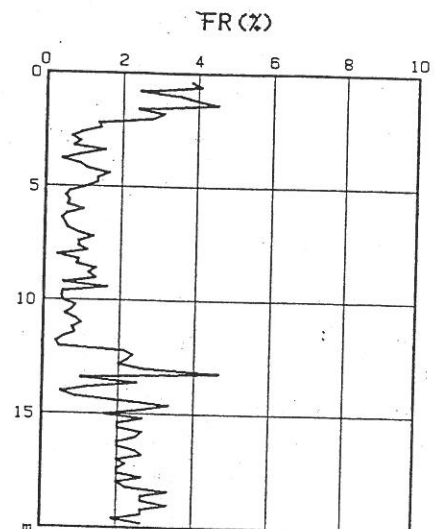
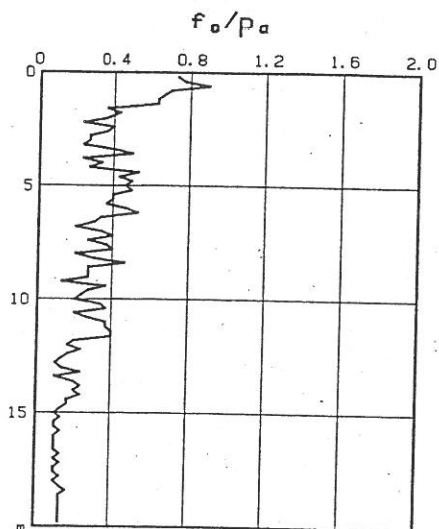
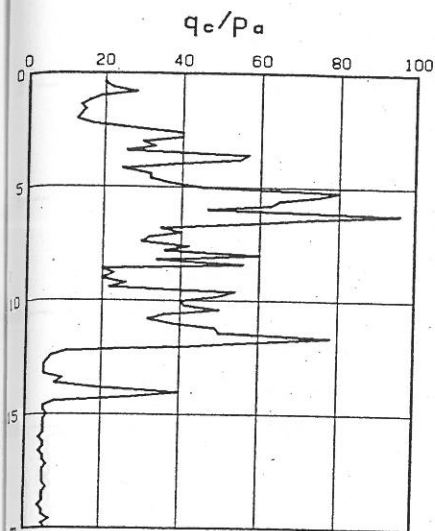
TAV. 46



geoLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT4
LOCALITÀ: PISA
CANTIERE: CISANELLO

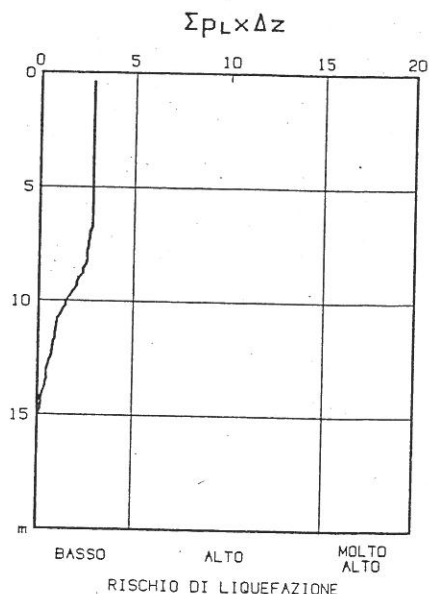
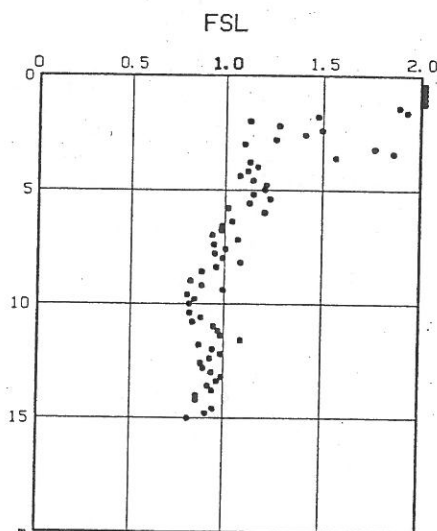
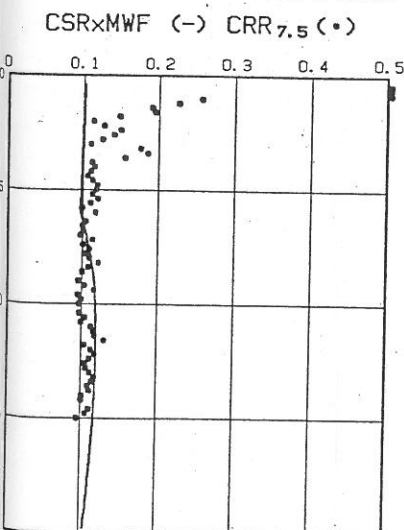
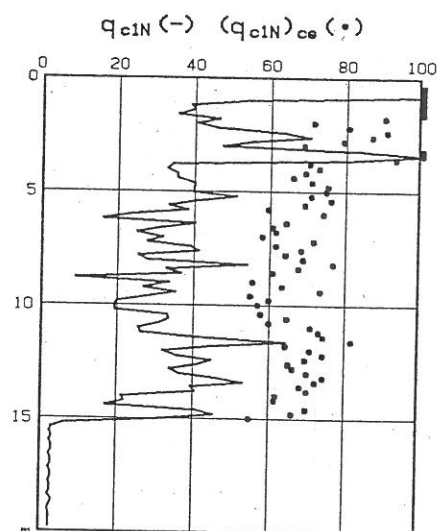
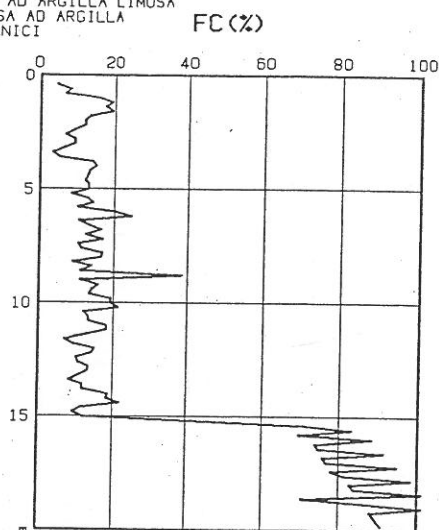
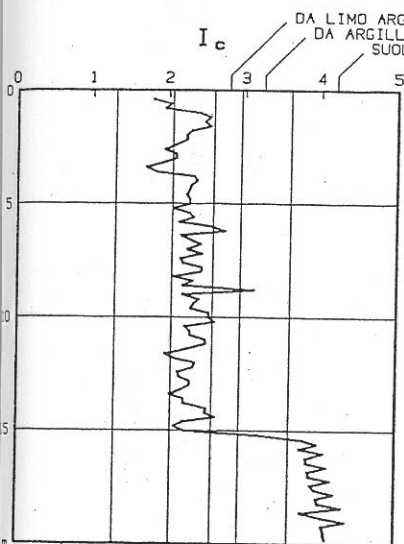
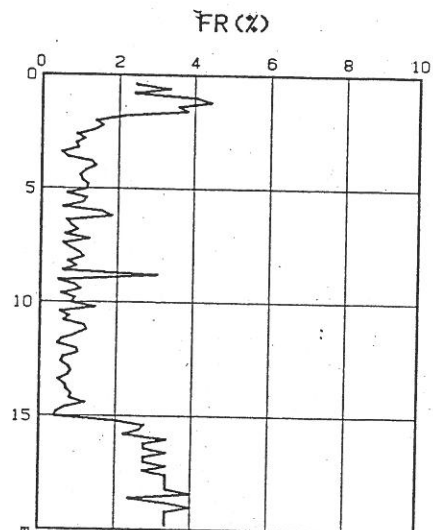
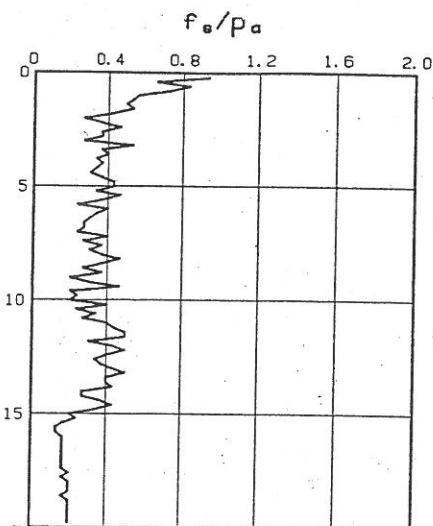
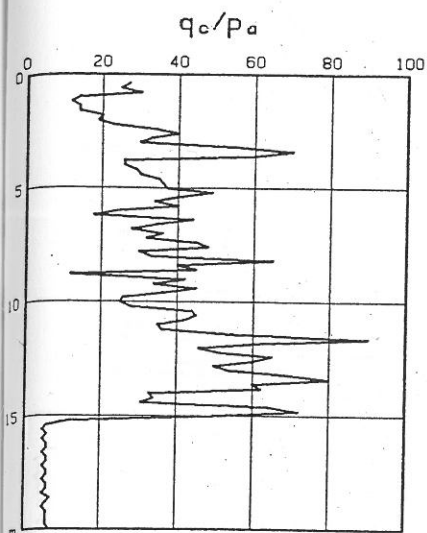
TAV. 47



geoLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT5
LOCALITA: PISA
CANTIERE: CISANELLO

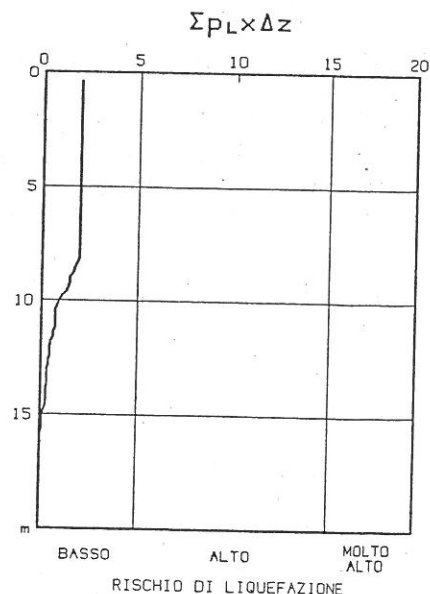
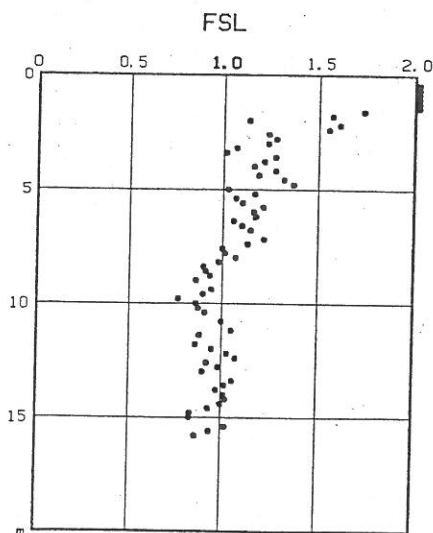
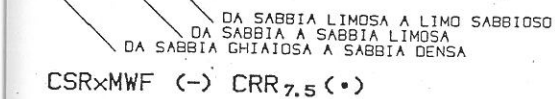
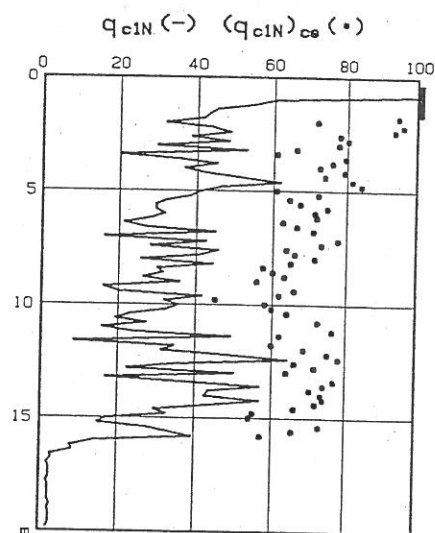
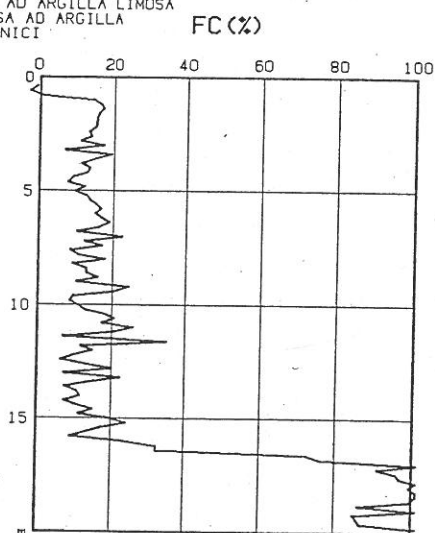
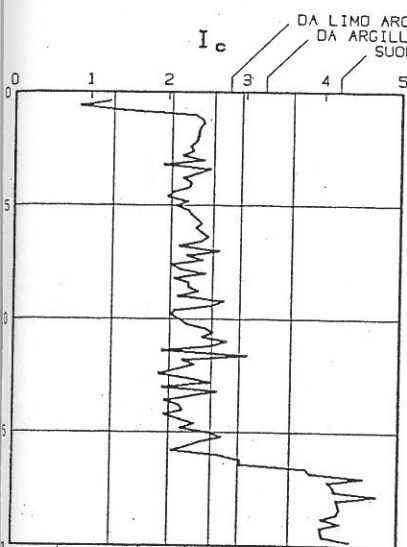
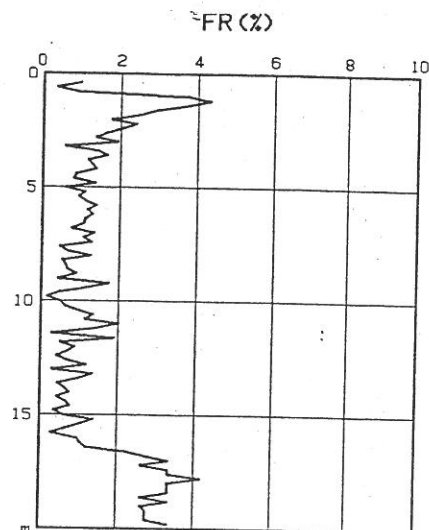
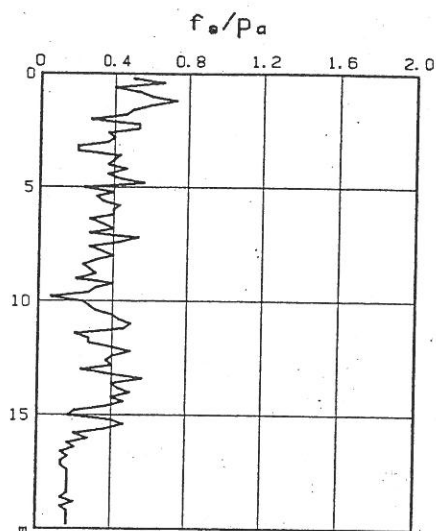
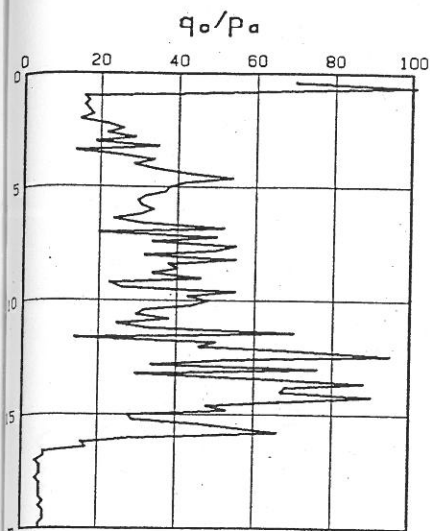
TAV. 48



geoLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT6
LOCALITA: PISA
CANTIERE: CISANELLO

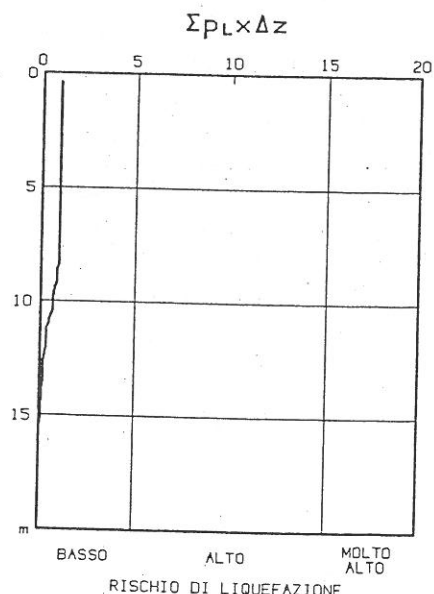
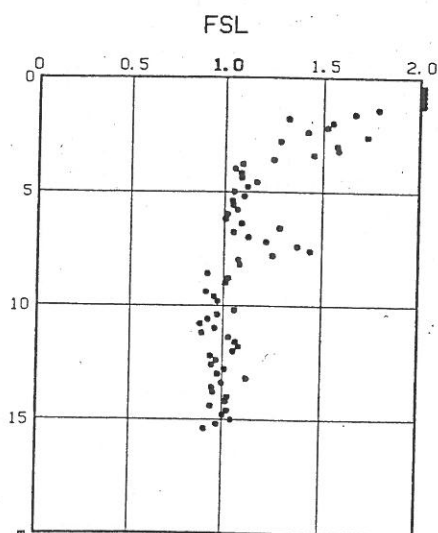
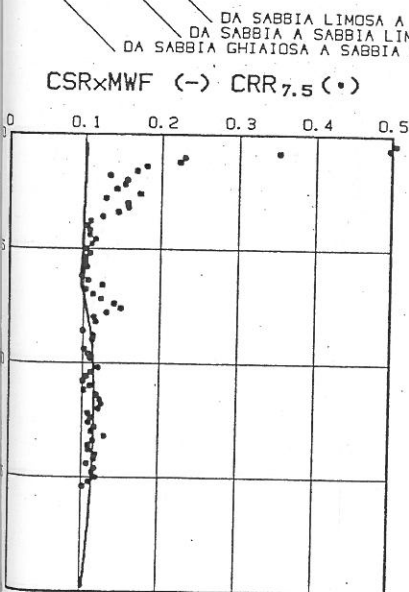
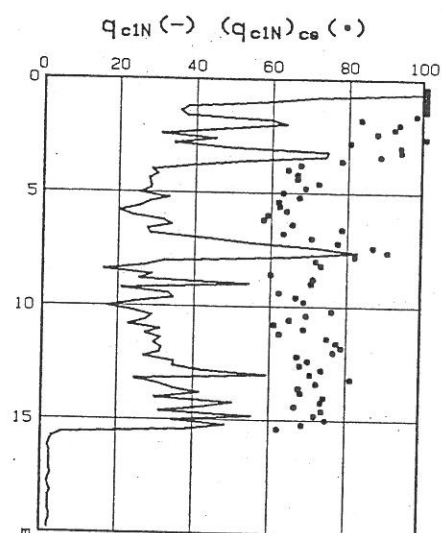
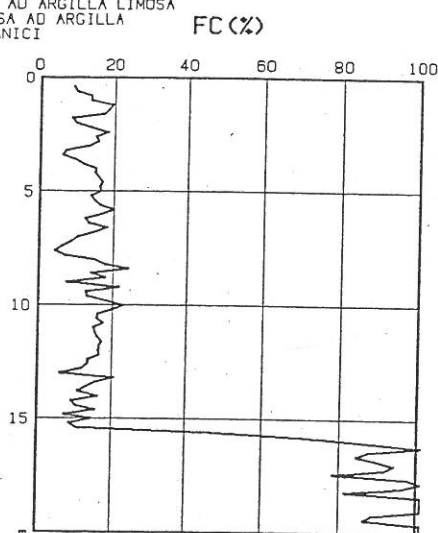
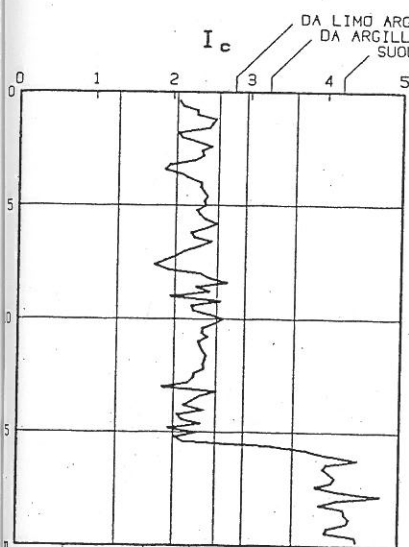
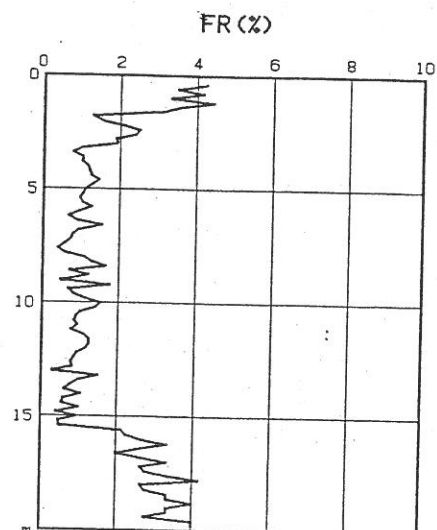
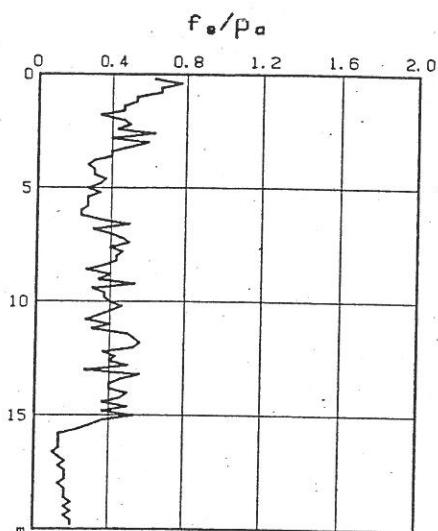
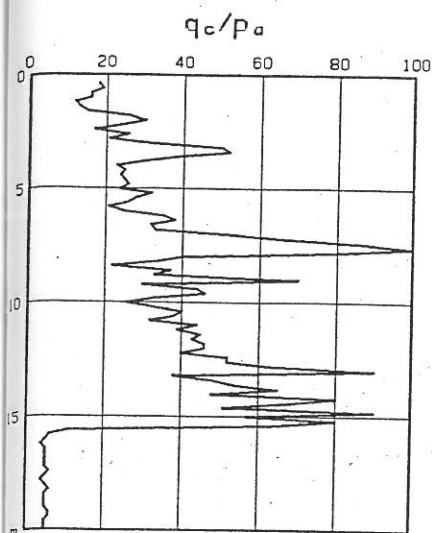
TAV. 49



geoLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT7
LOCALITA: PISA
CANTIERE: CISANELLO

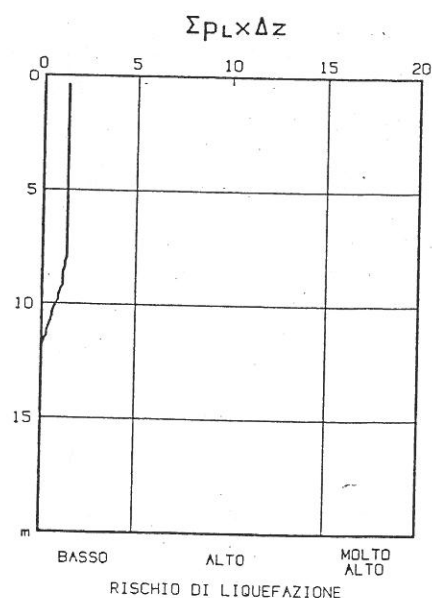
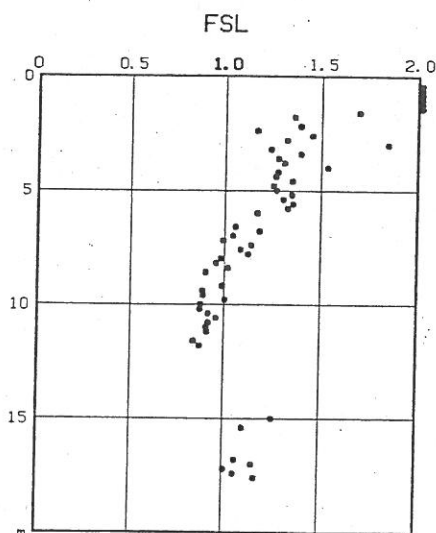
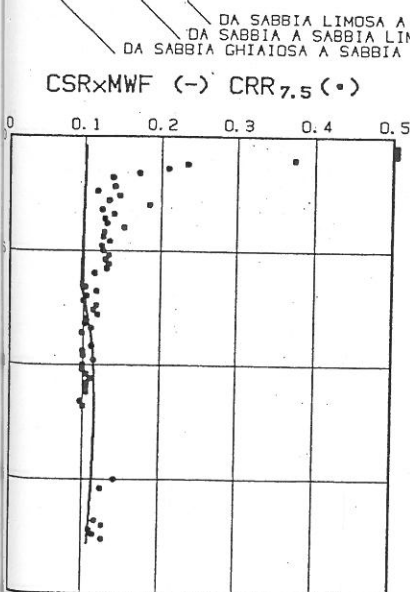
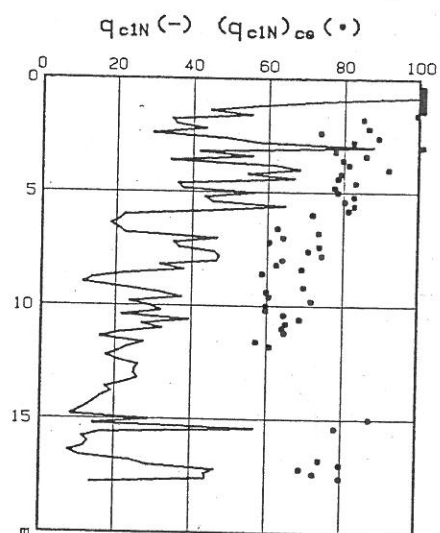
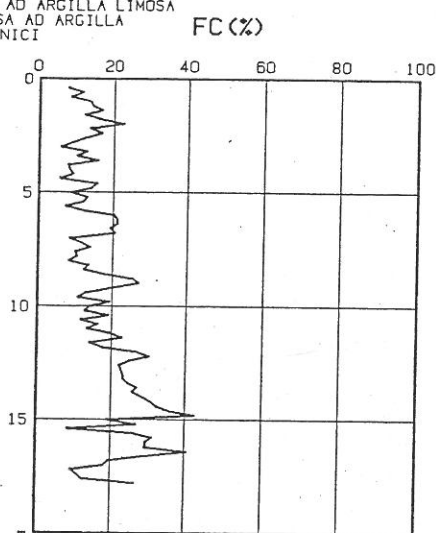
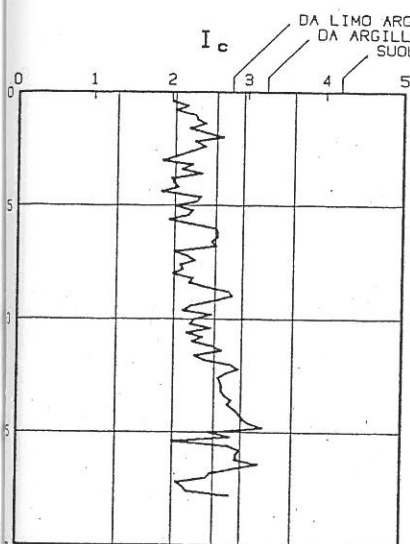
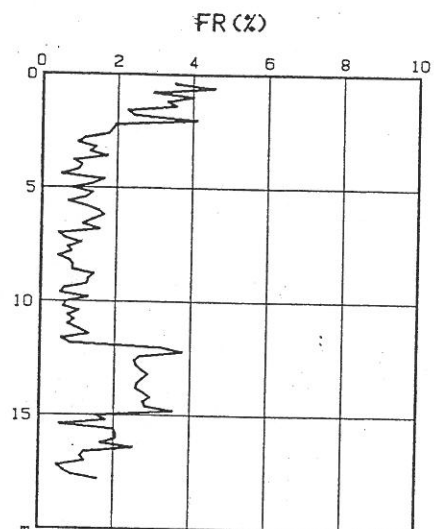
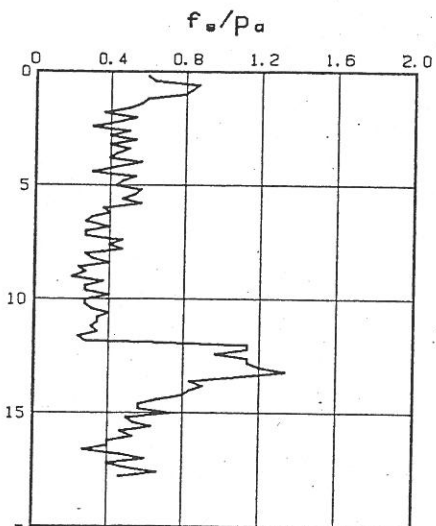
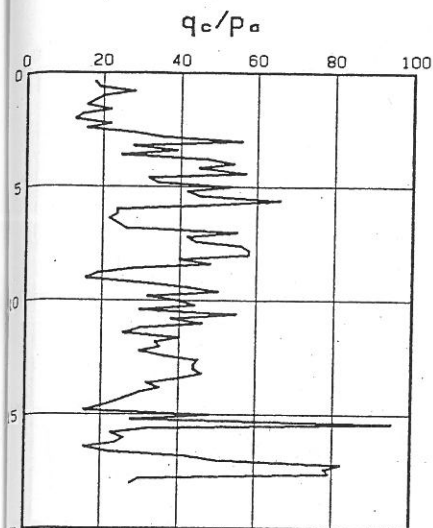
TAV. 50



geoLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT8
LOCALITA: PISA
CANTIERE: CISANELLO

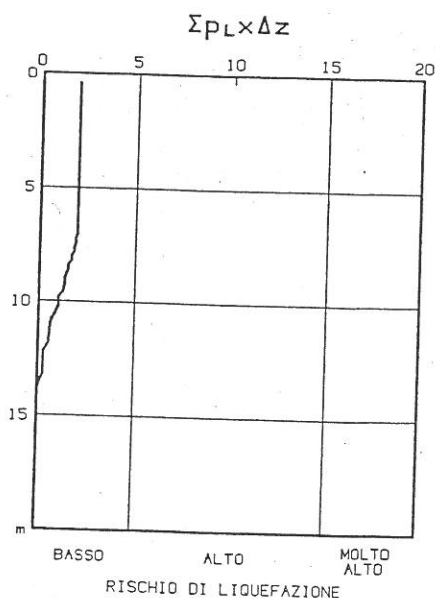
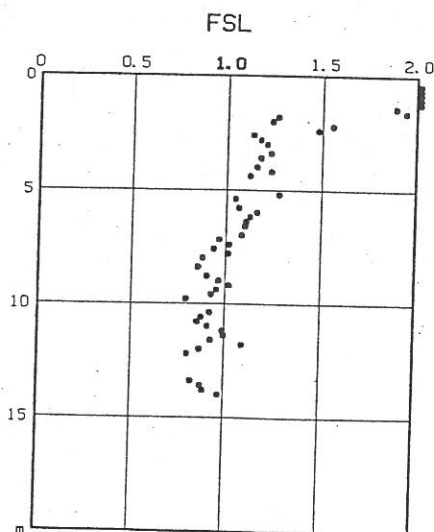
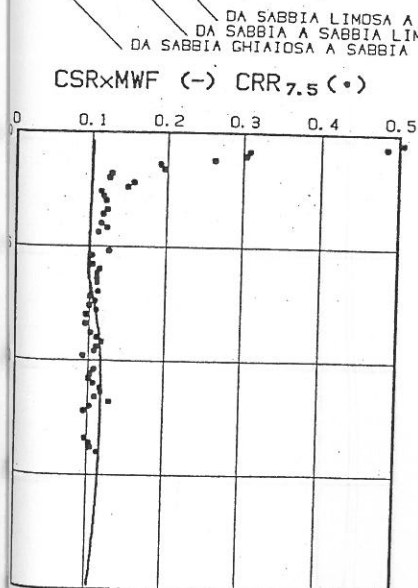
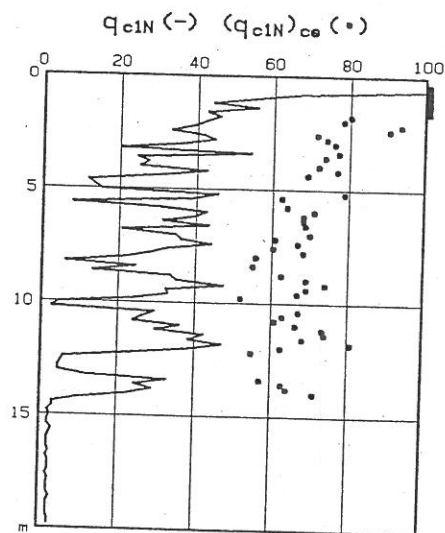
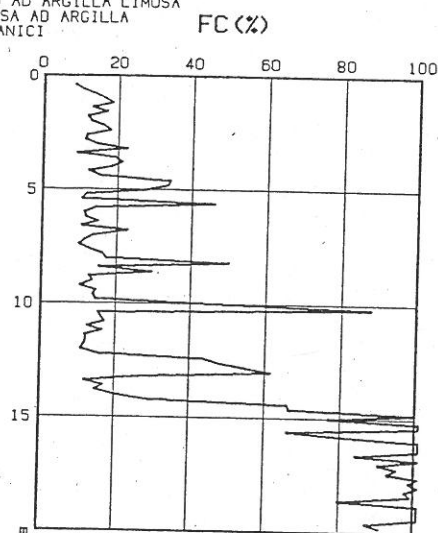
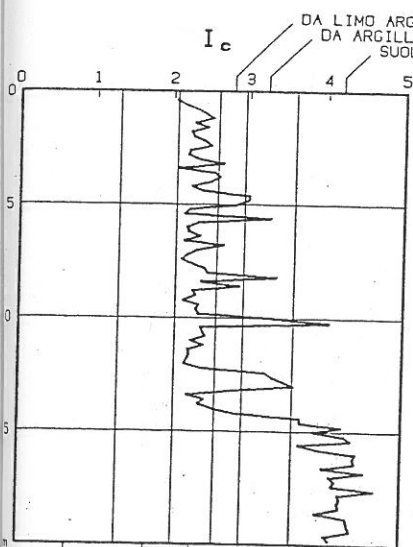
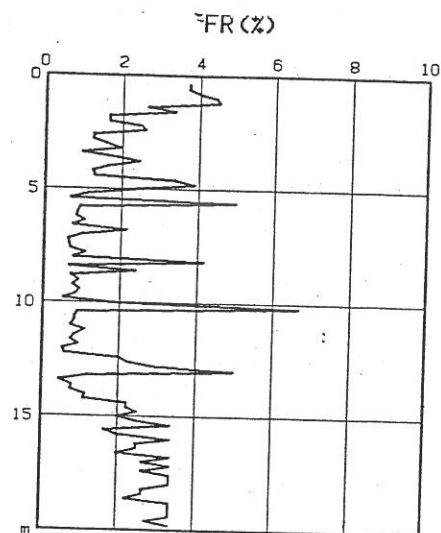
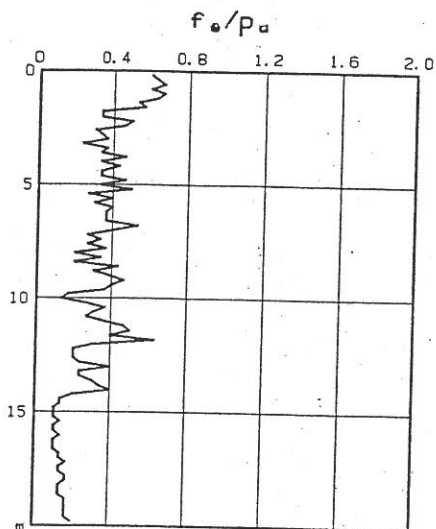
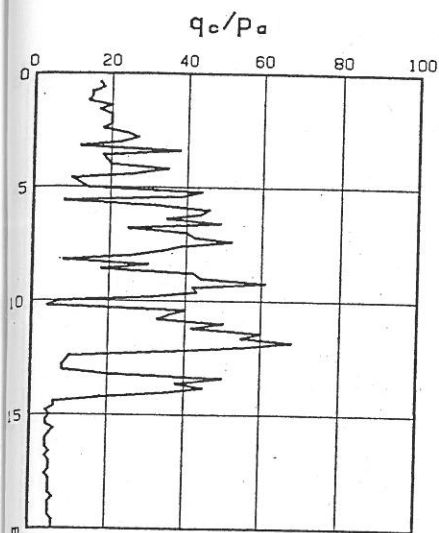
TAV. 51



geoLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT9
LOCALITA: PISA
CANTIERE: CISANELLO

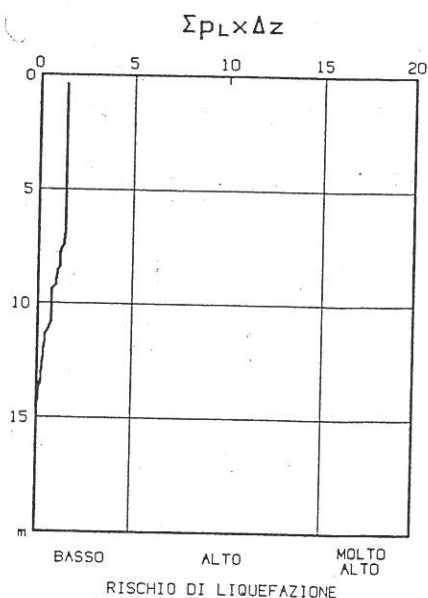
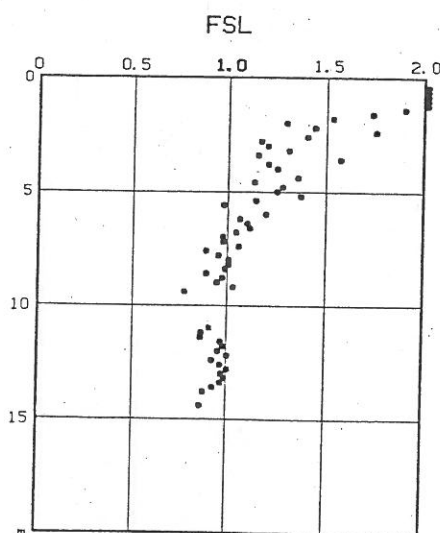
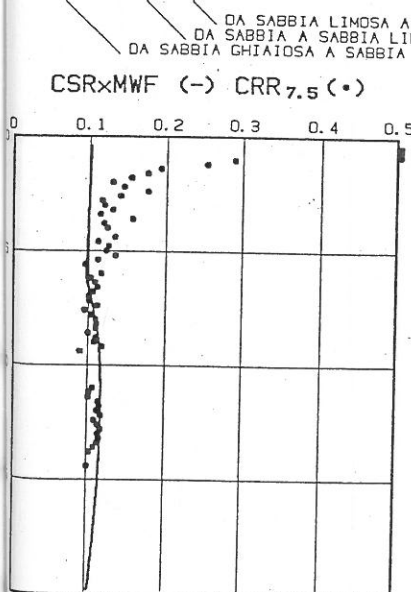
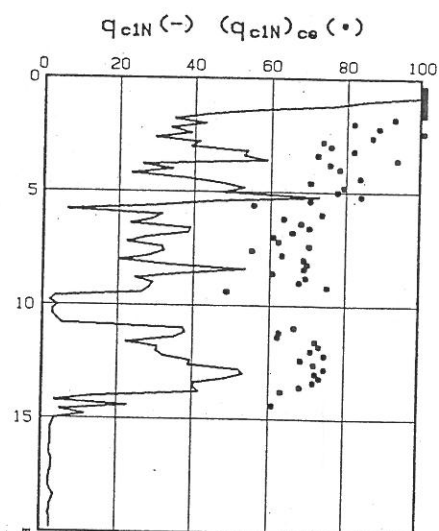
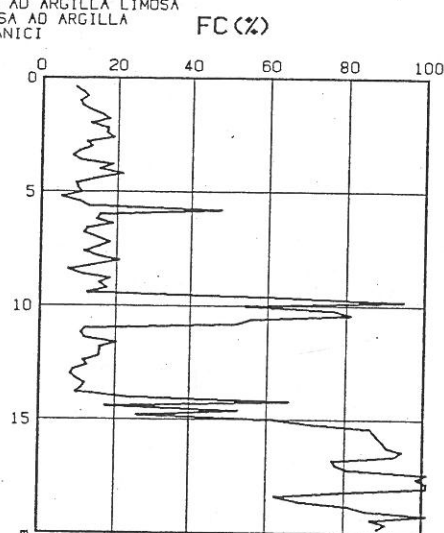
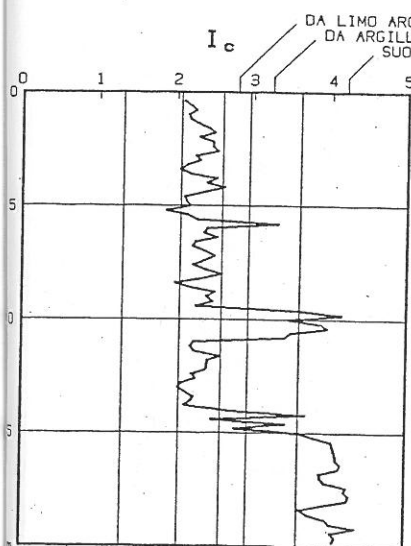
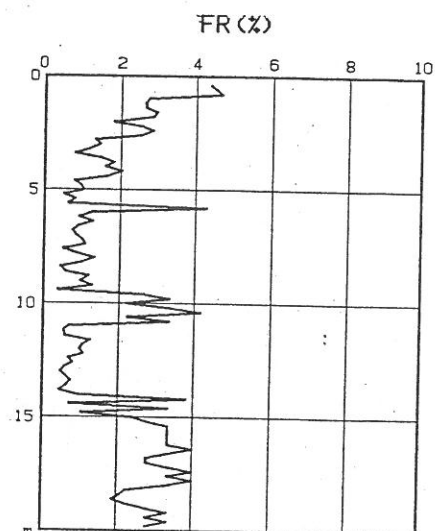
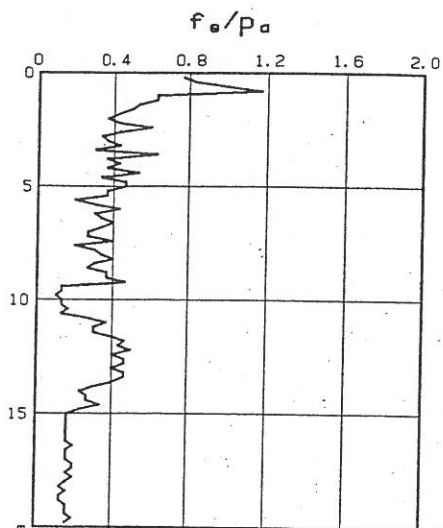
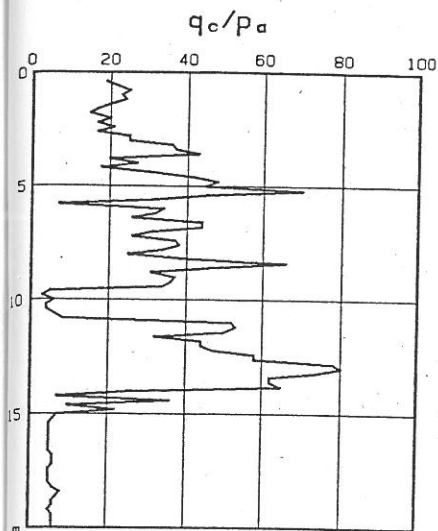
TAV. 52



geoLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT10
LOCALITÀ: PISA
CANTIERE: CISANELLO

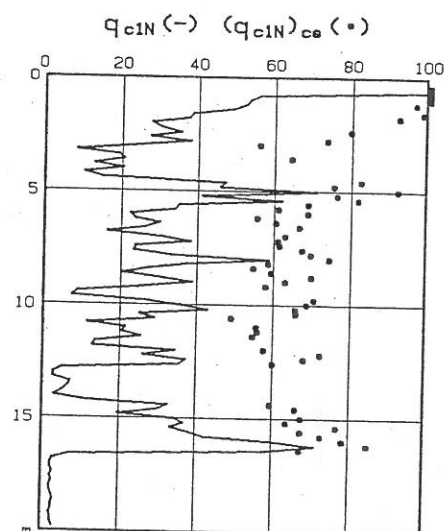
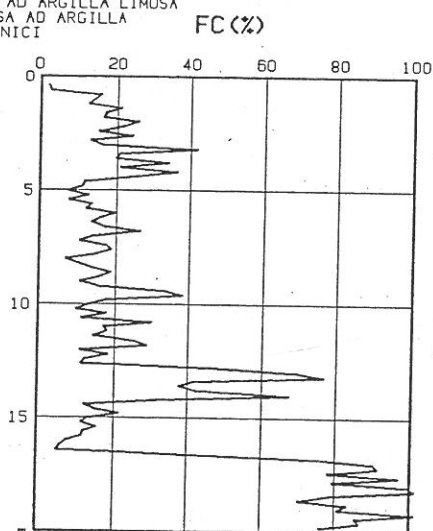
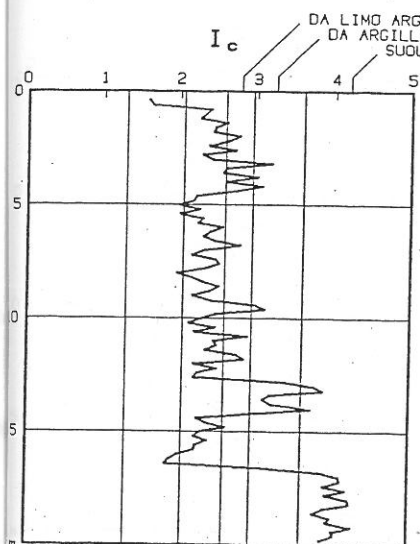
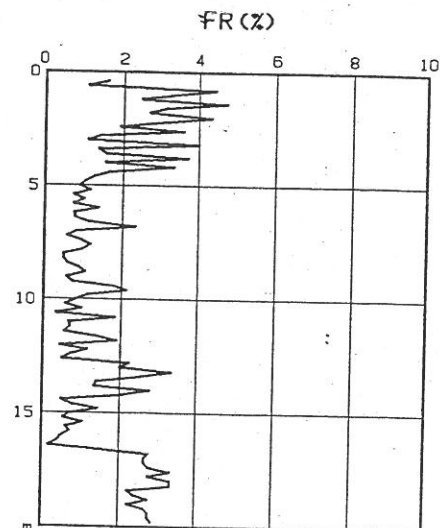
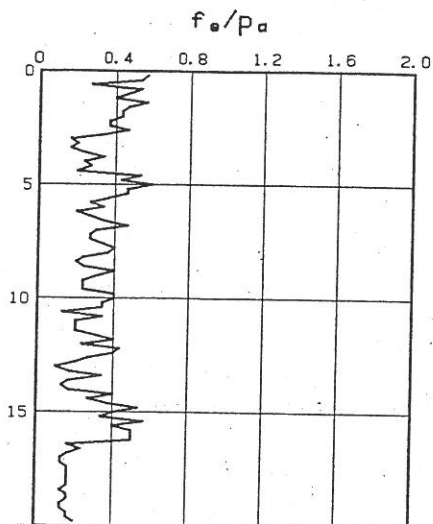
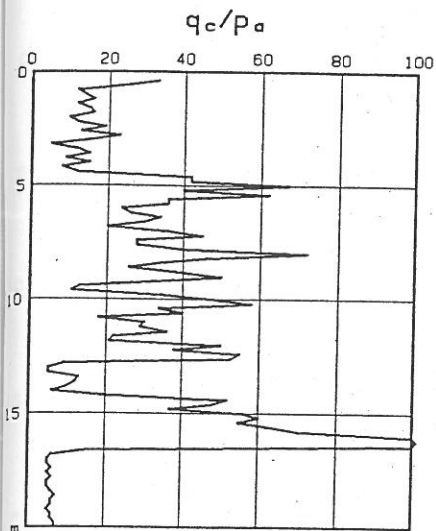
TAV. 53



geOLOG

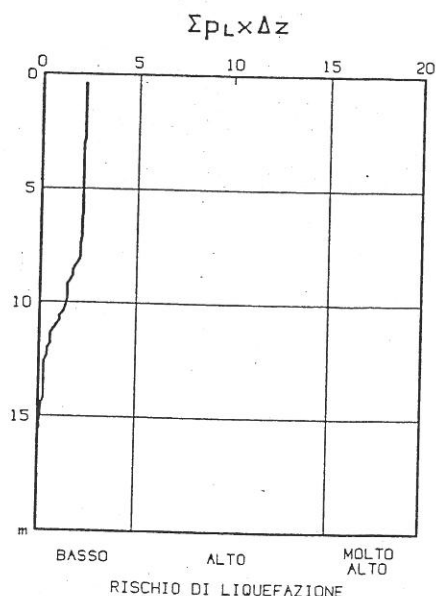
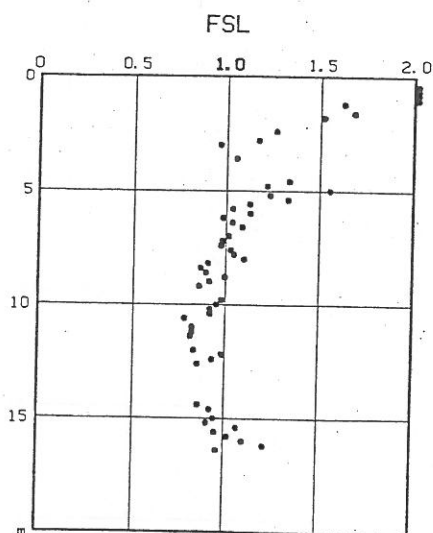
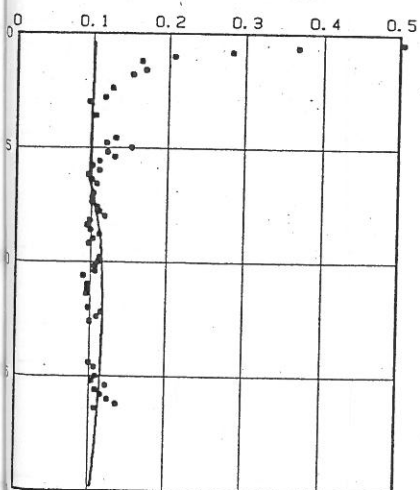
MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT11
LOCALITA: PISA
CANTIERE: CISANELLO

TAV. 54



DA LIMO ARGILLOSO AD ARGILLA LIMOSA
DA ARGILLA LIMOSA AD ARGILLA
SUOLI ORGANICI

CSR x MWF (-) CRR_{7.5} (•)

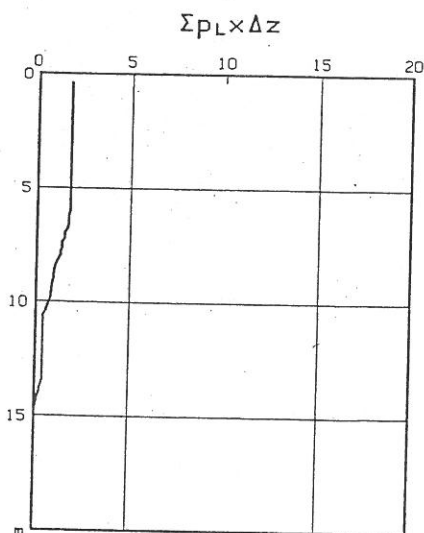
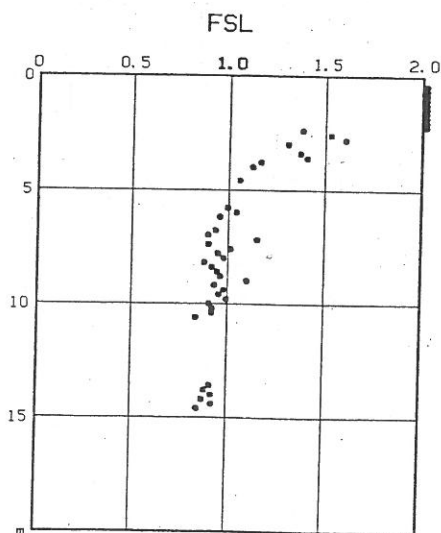
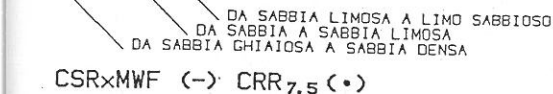
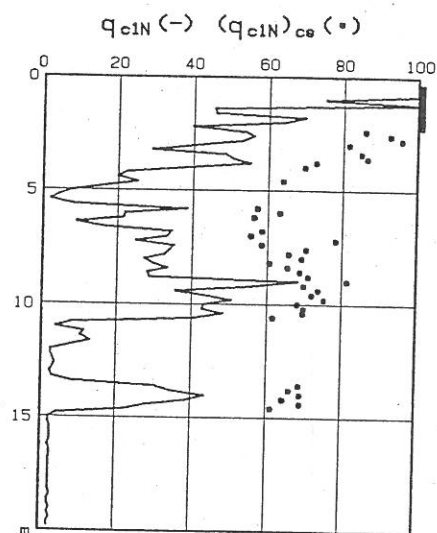
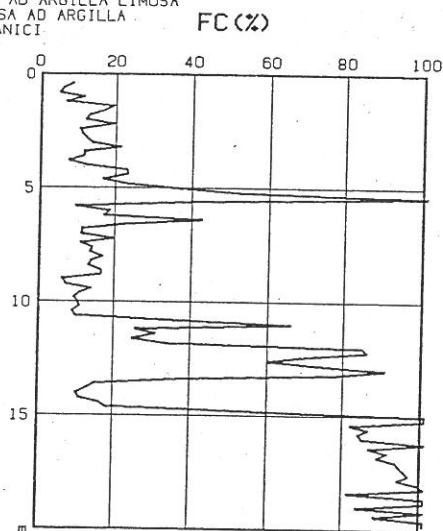
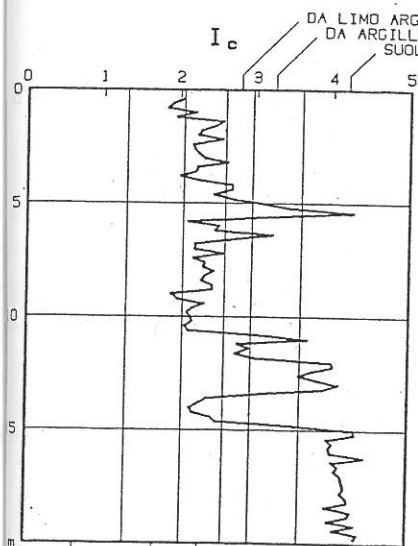
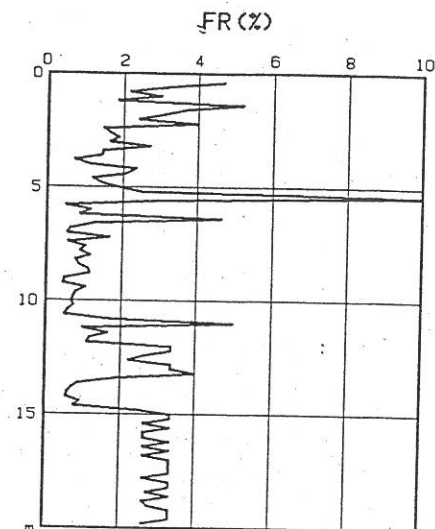
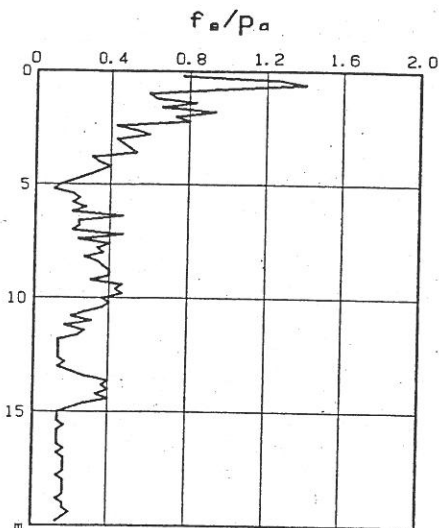
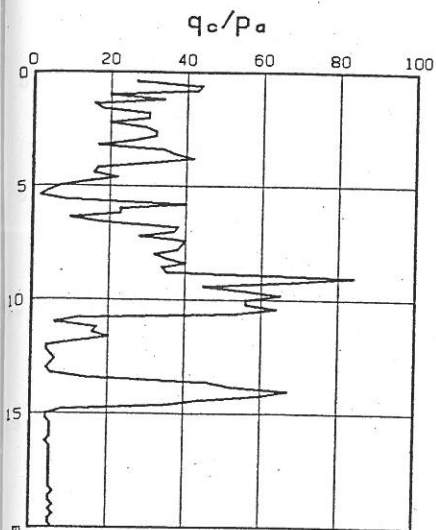


BASSO ALTO MOLTO ALTO
RISCHIO DI LIQUEFAZIONE

geoLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT12
LOCALITA: PISA
CANTIERE: CISANELLO

TAV. 55

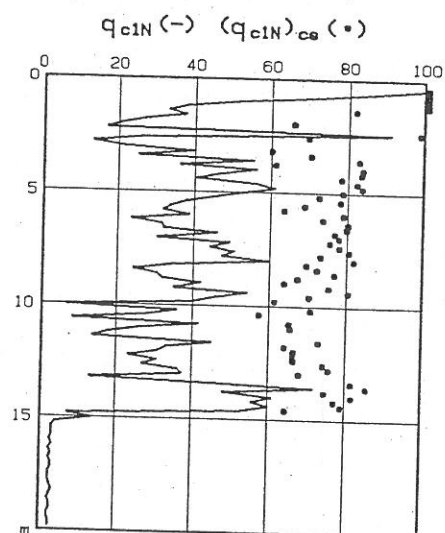
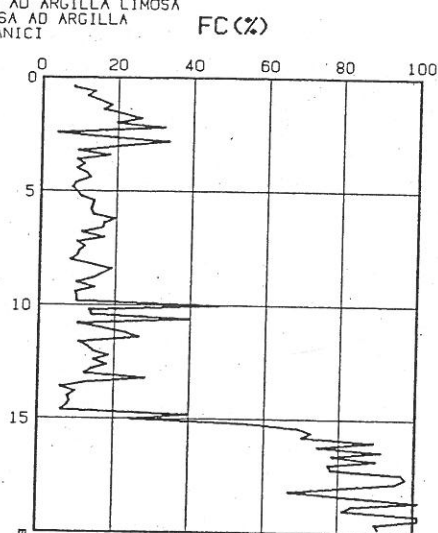
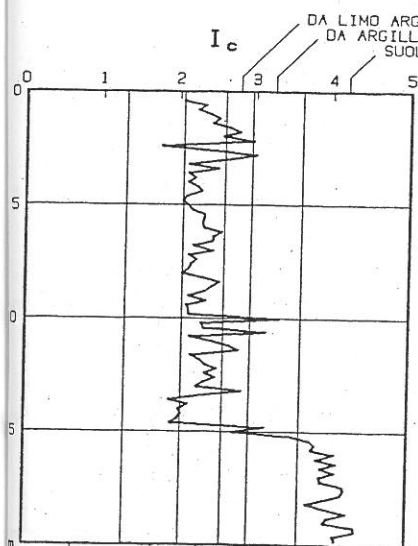
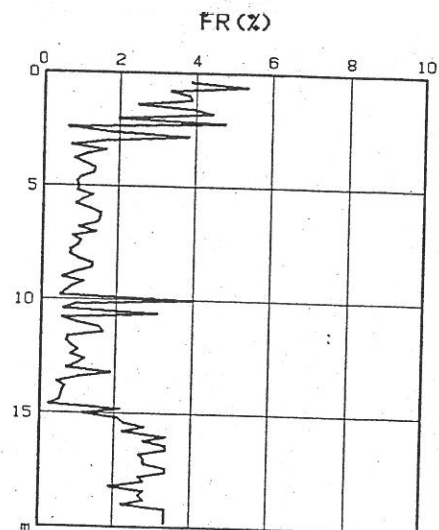
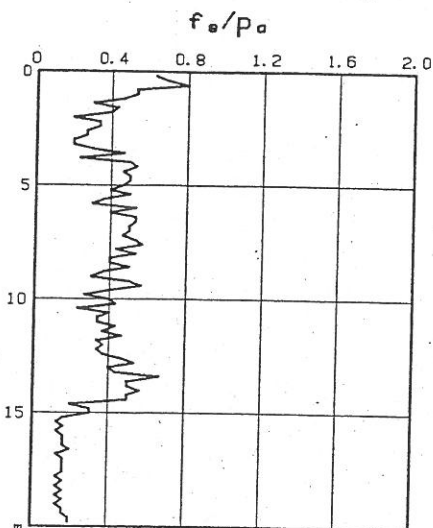
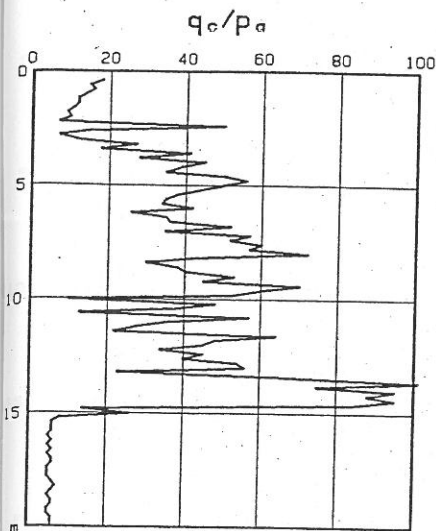


BASSO ALTO MOLTO ALTO
RISCHIO DI LIQUEFAZIONE

geoLOG

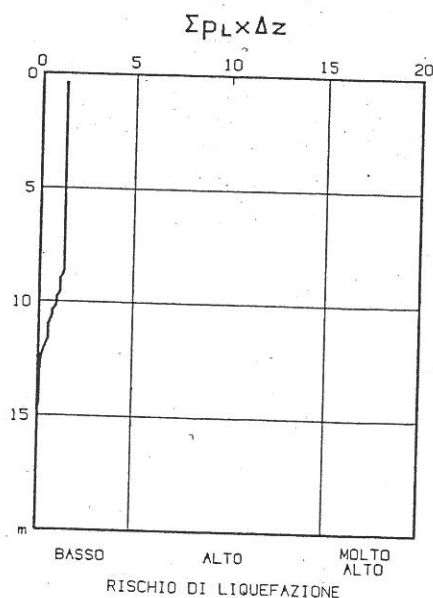
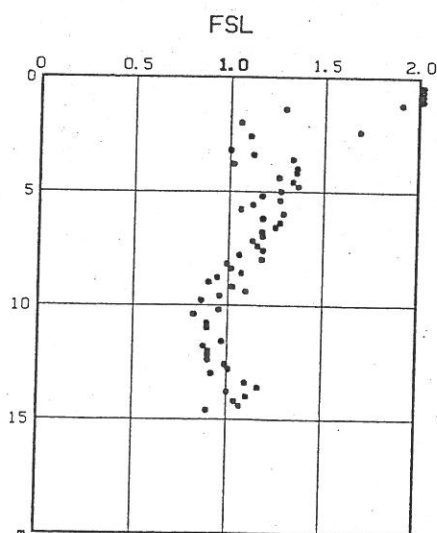
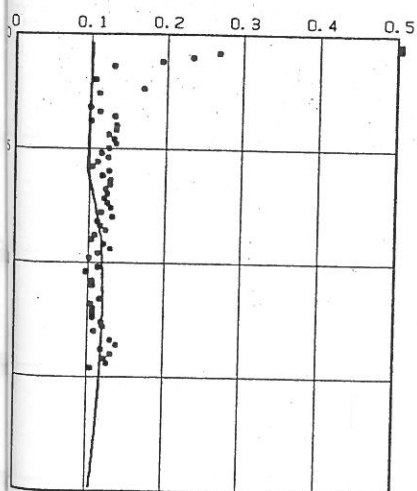
MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT13
LOCALITA: PISA
CANTIERE: CISANELLO

TAV. 56



DA LIMO ARGILLOSO AD ARGILLA LIMOSA
DA ARGILLA LIMOSA AD ARGILLA
SUOLI ORGANICI

CSR_xMWF (-) CRR_{7.5} (•)

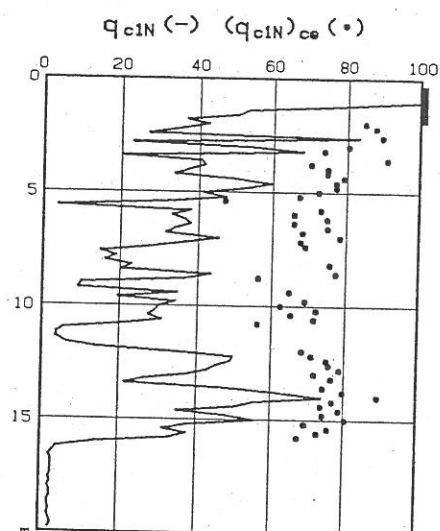
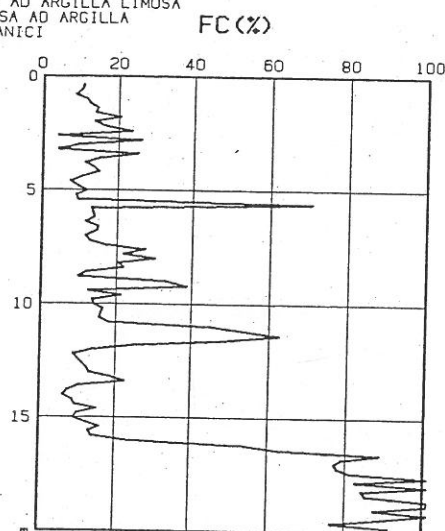
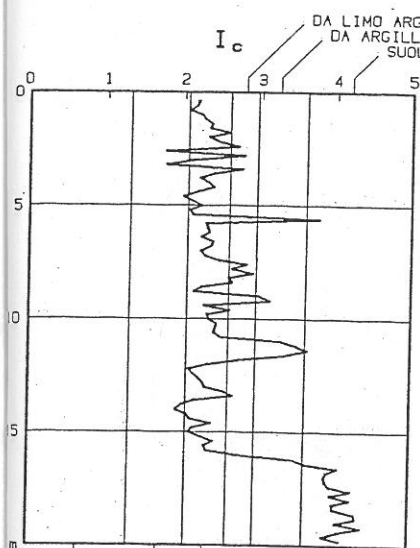
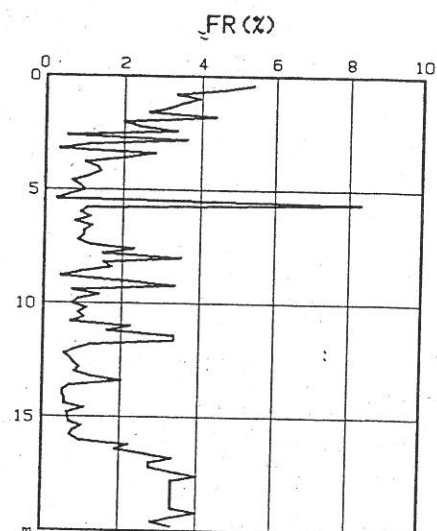
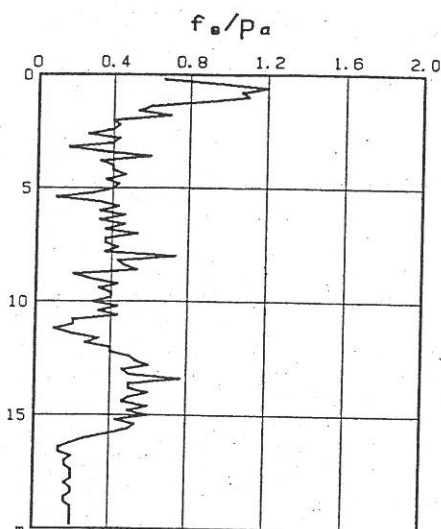
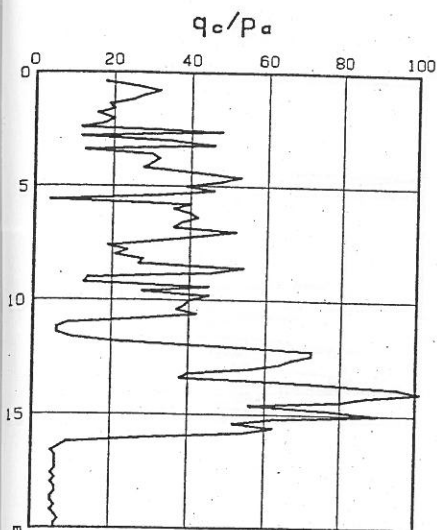


BASSO ALTO MOLTO ALTO
RISCHIO DI LIQUEFAZIONE

geOLOG

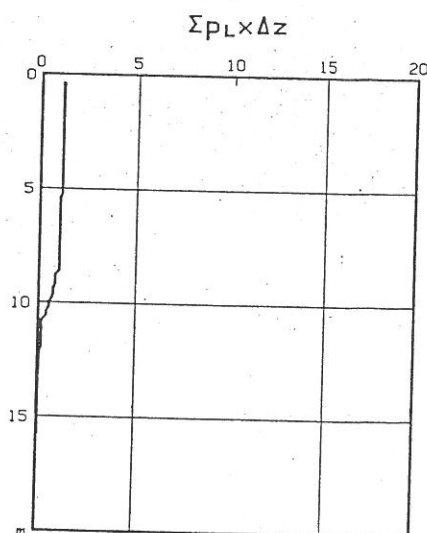
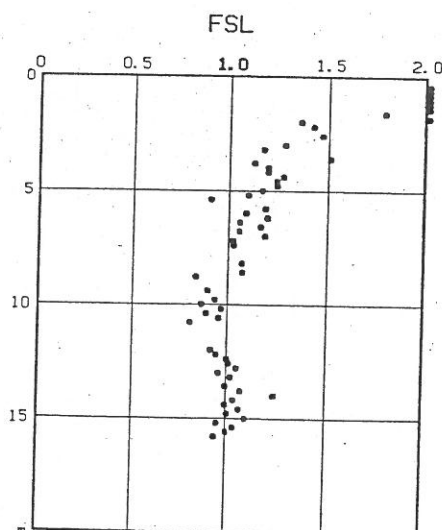
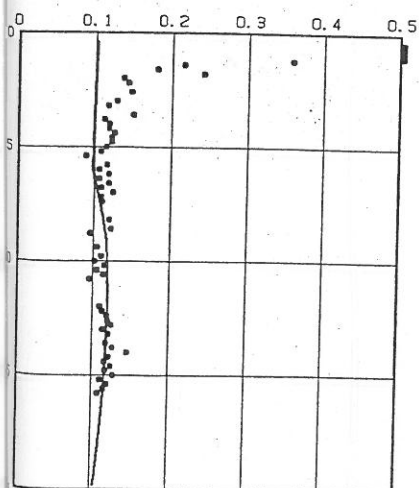
MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT14
LOCALITA: PISA
CANTIERE: CISANELLO

TAV. 57



DA LIMO ARGILLOSO AD ARGILLA LIMOSA
DA ARGILLA LIMOSA AD ARGILLA
SUOLI ORGANICI

CSR x MWF (-) CRR_{7.5} (•)

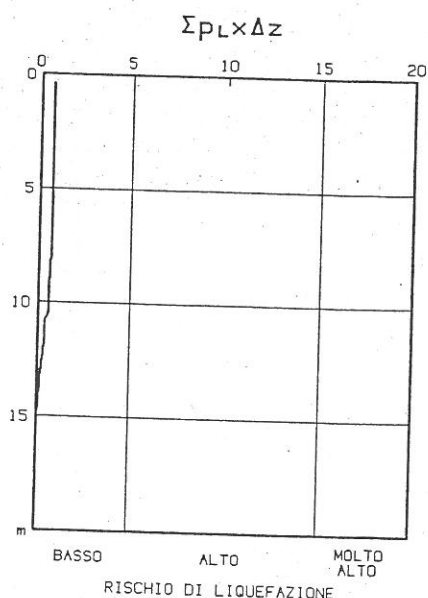
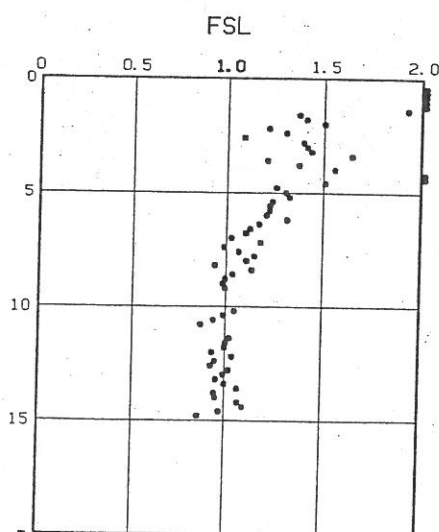
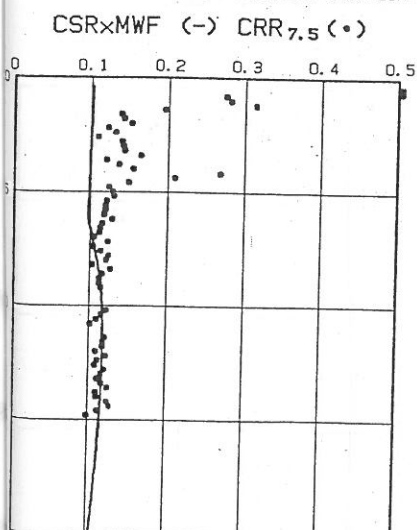
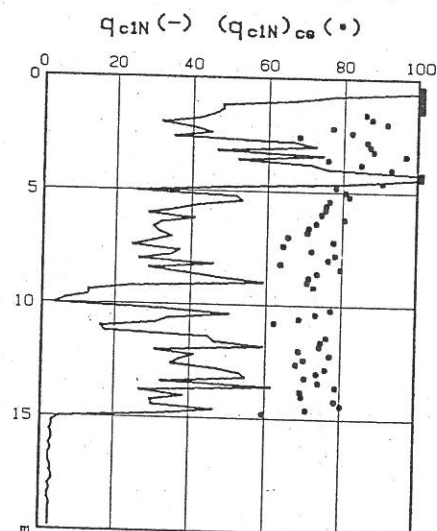
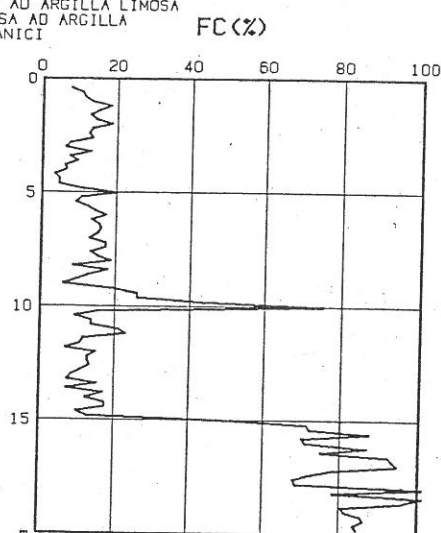
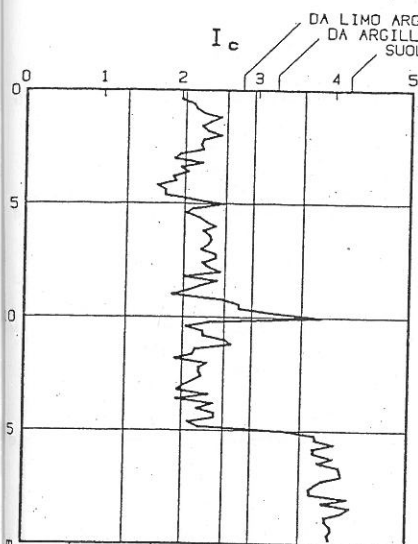
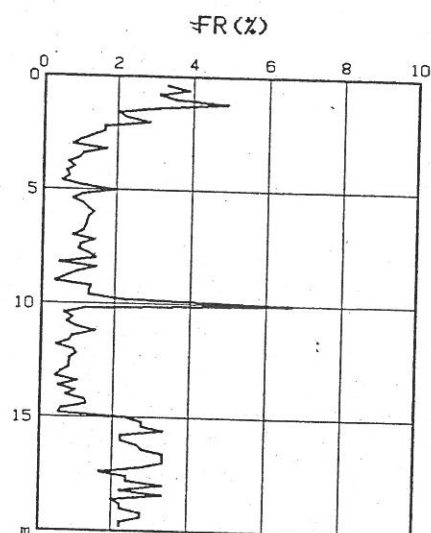
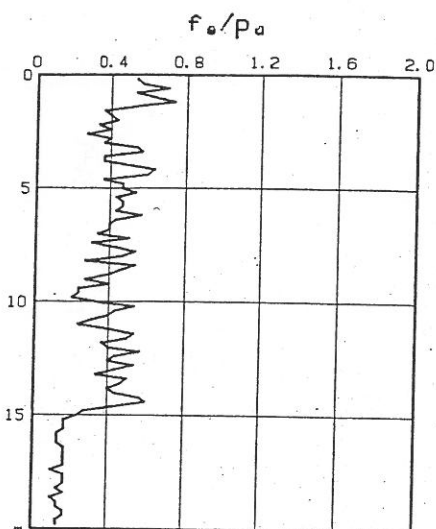
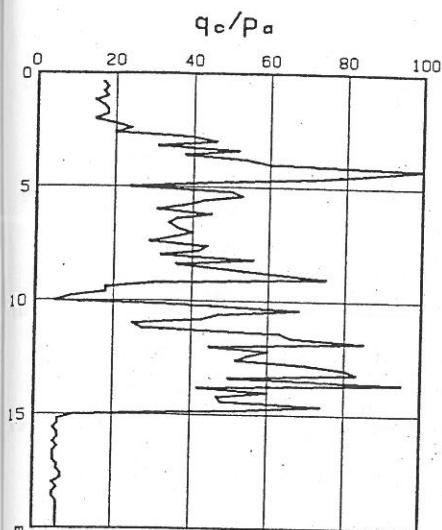


BASSO ALTO MOLTO ALTO
RISCHIO DI LIQUEFAZIONE

geOLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT15
LOCALITÀ: PISA
CANTIERE: CISANELLO

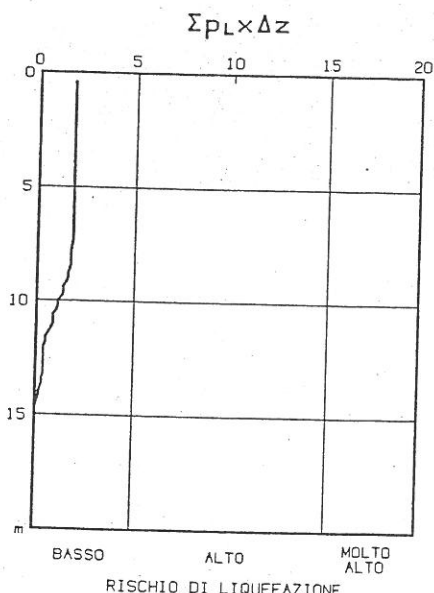
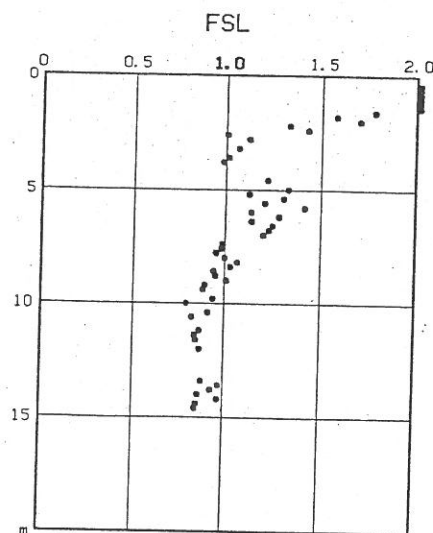
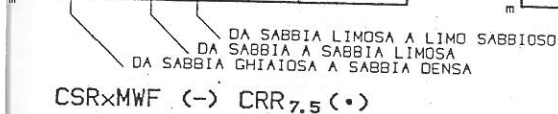
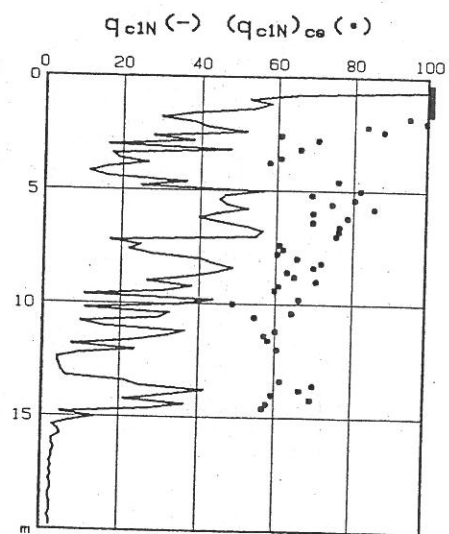
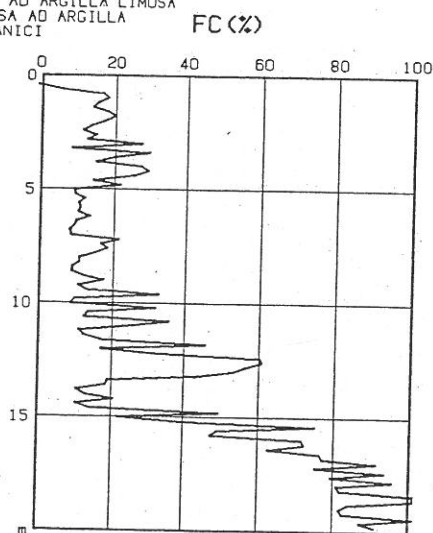
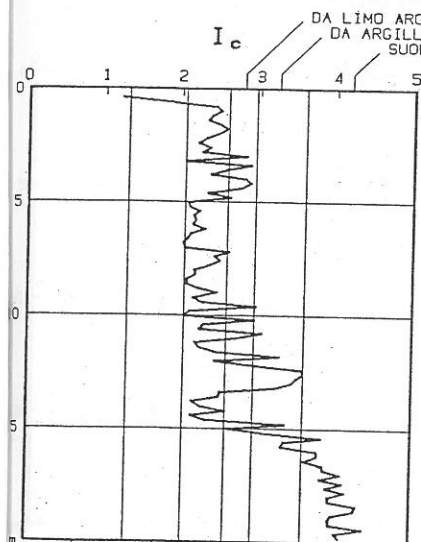
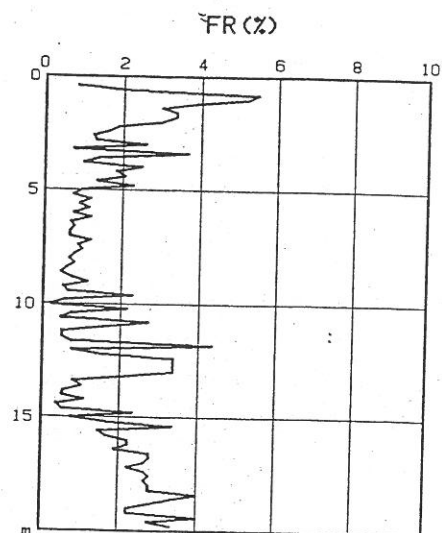
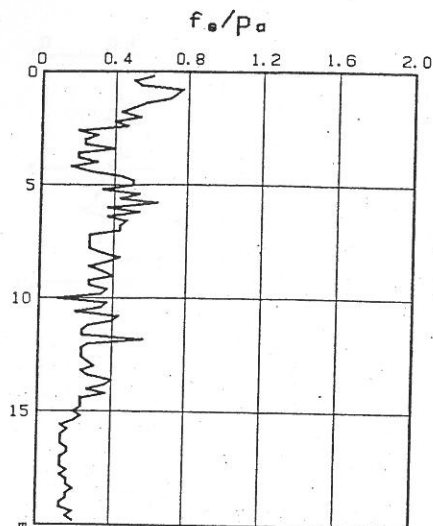
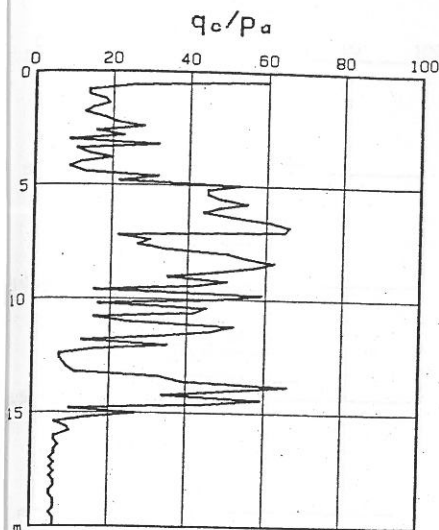
TAV. 58



geOLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT16
LOCALITA: PISA
CANTIERE: CISANELLO

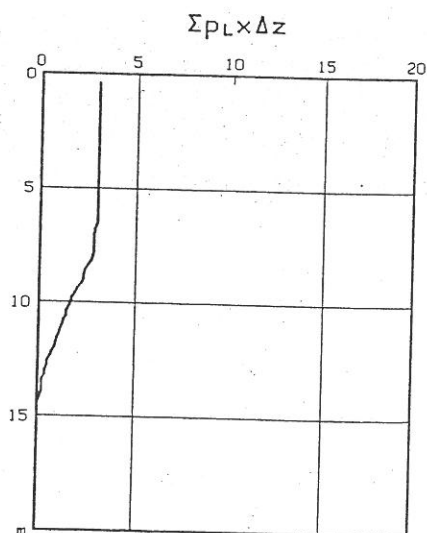
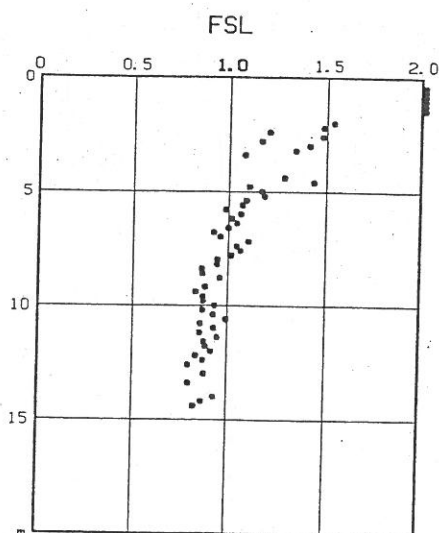
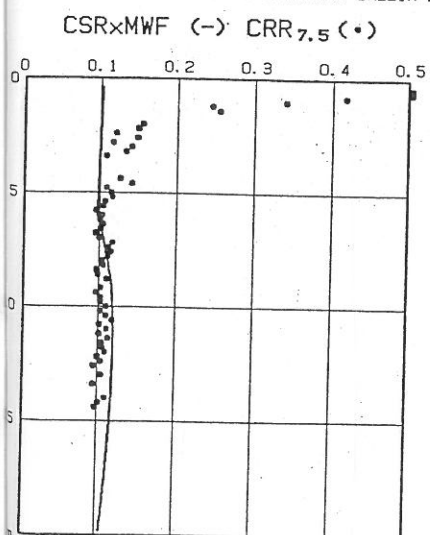
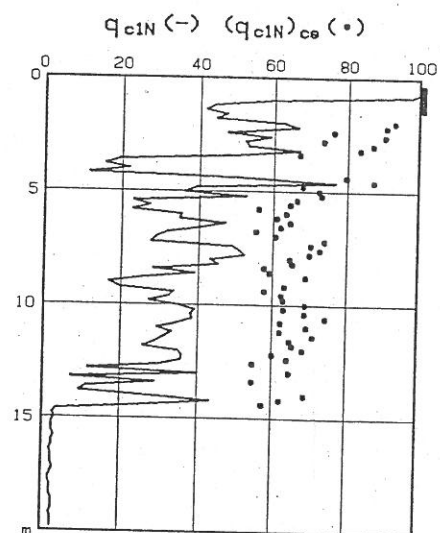
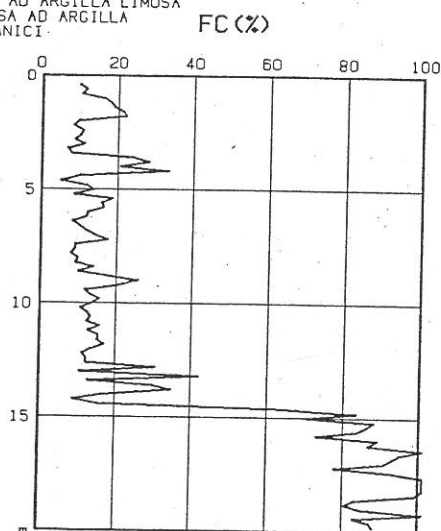
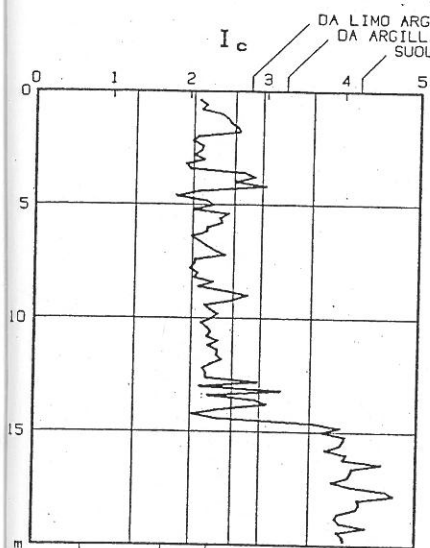
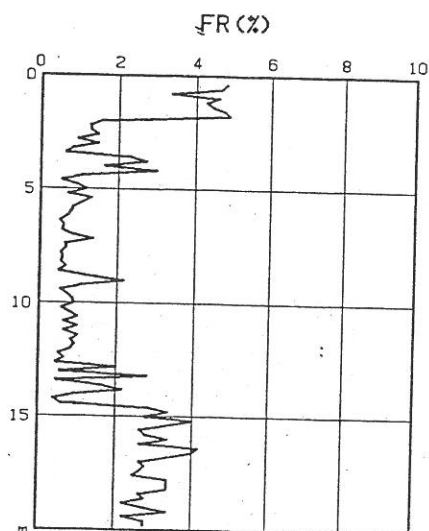
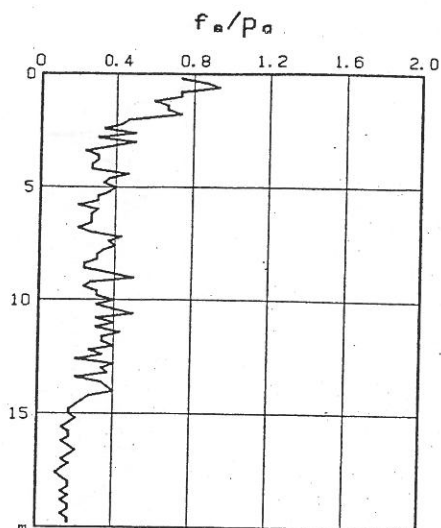
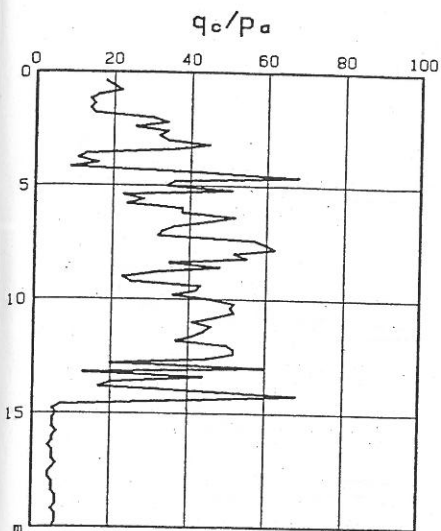
TAV. 59



geOLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT17
LOCALITA: PISA
CANTIERE: CISANELLO

TAV. 60

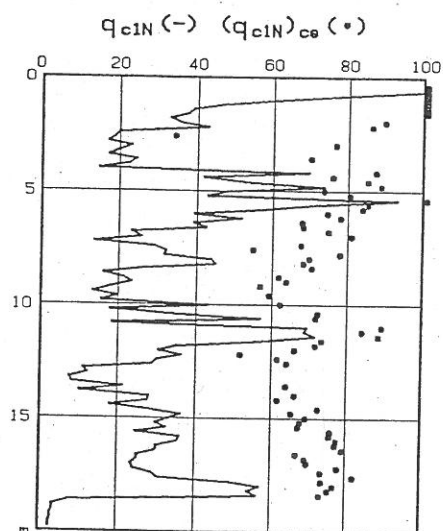
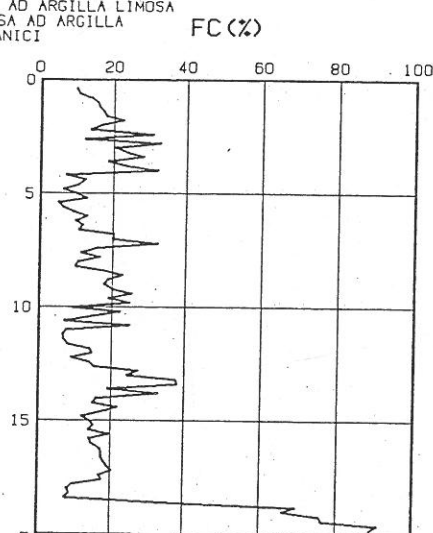
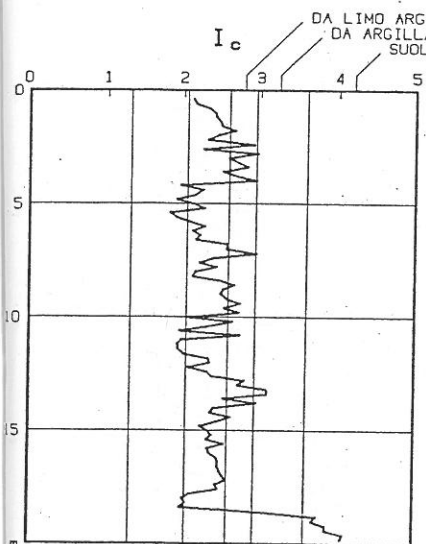
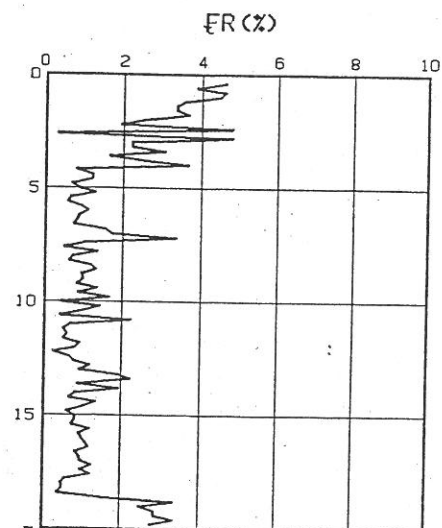
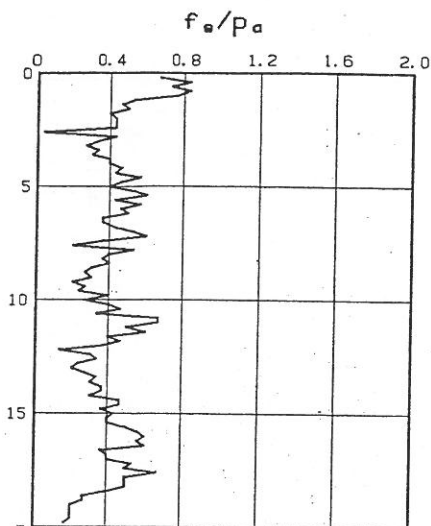
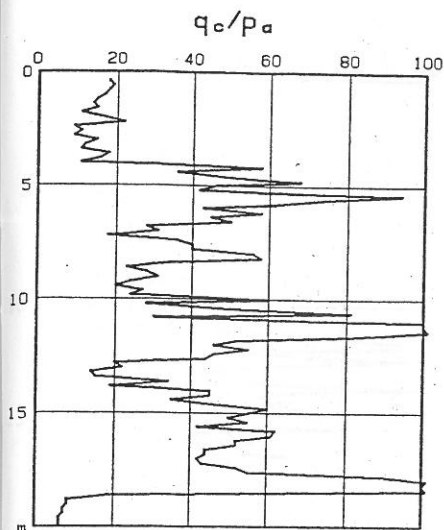


BASSO ALTO MOLTO ALTO
RISCHIO DI LIQUEFAZIONE

geoLOG

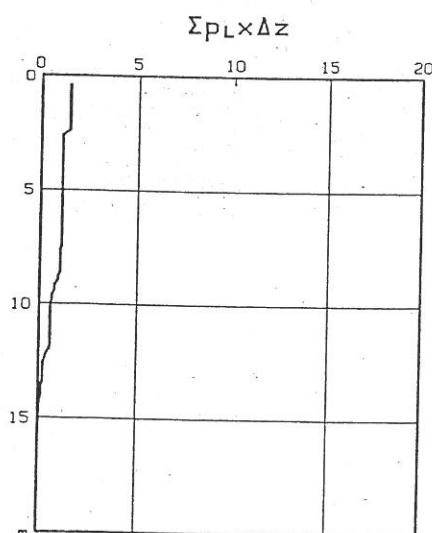
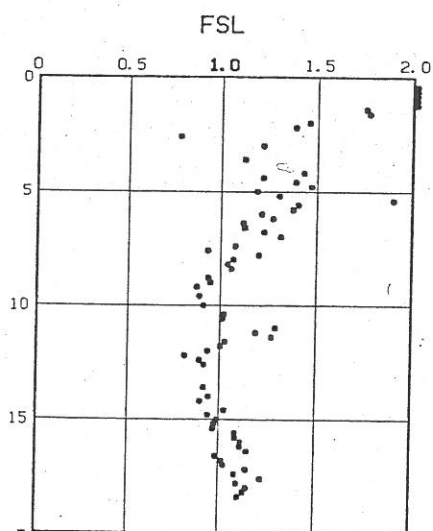
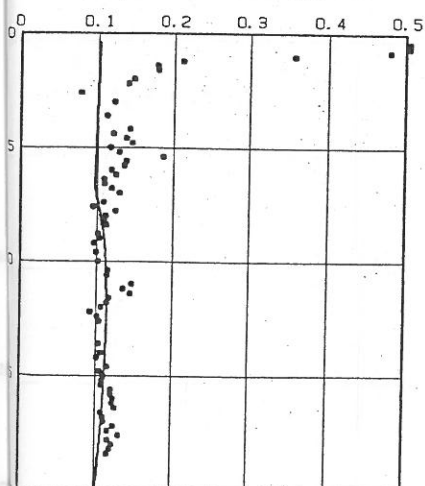
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ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT18
LOCALITA: PISA
CANTIERE: CISANELLO

TAV. 61



DA LIMO ARGILLOSO AD ARGILLA LIMOSA
DA ARGILLA LIMOSA AD ARGILLA
SUOLI ORGANICI

CSR_xMWF (-) CRR_{7.5} (+)

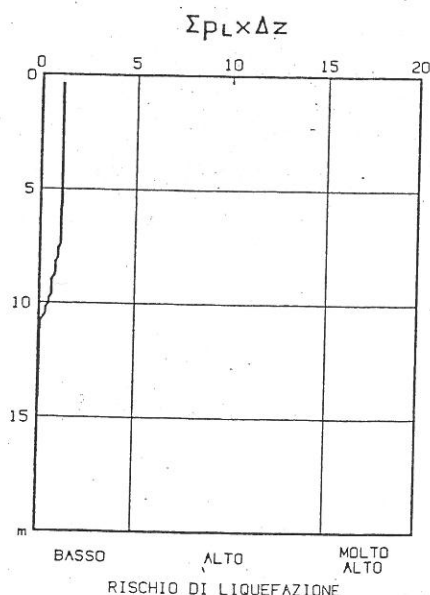
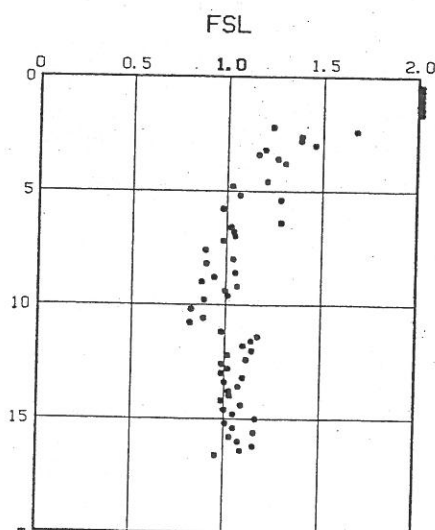
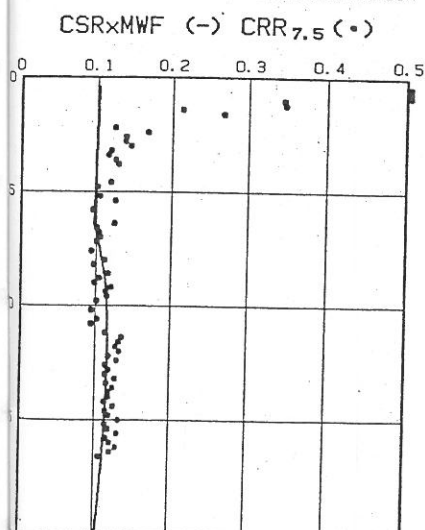
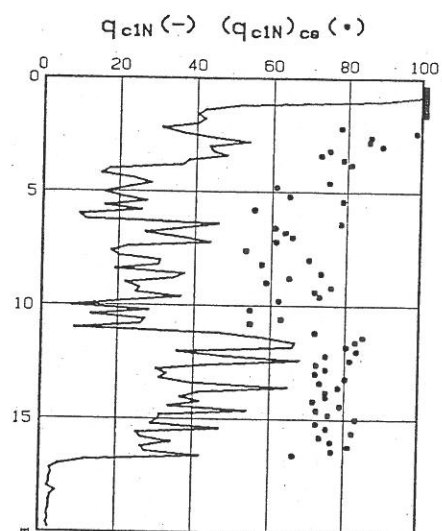
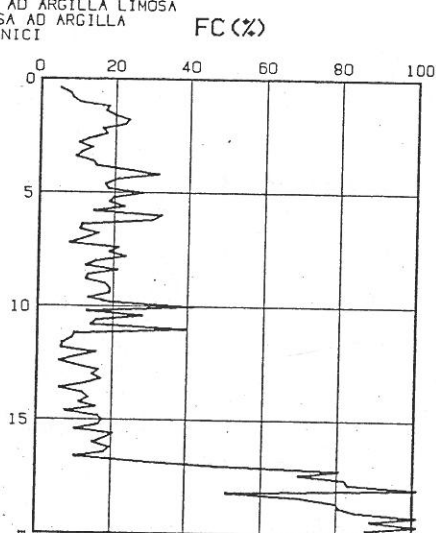
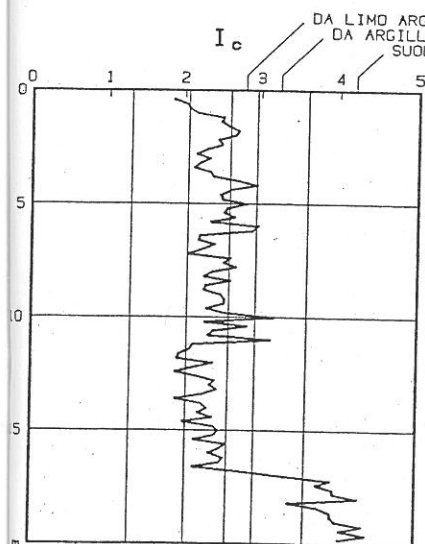
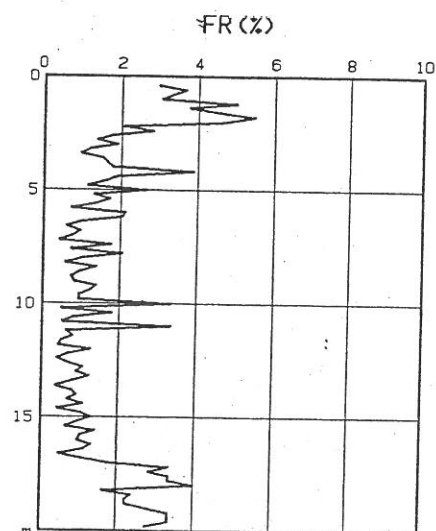
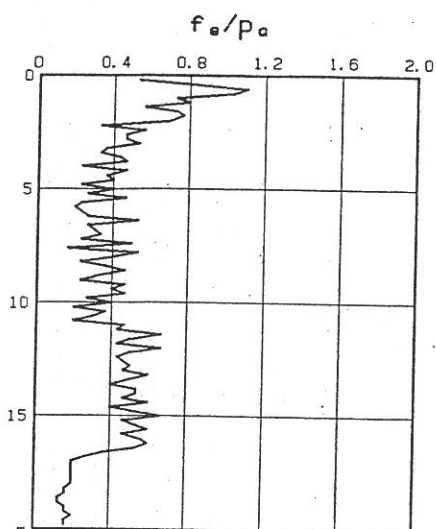
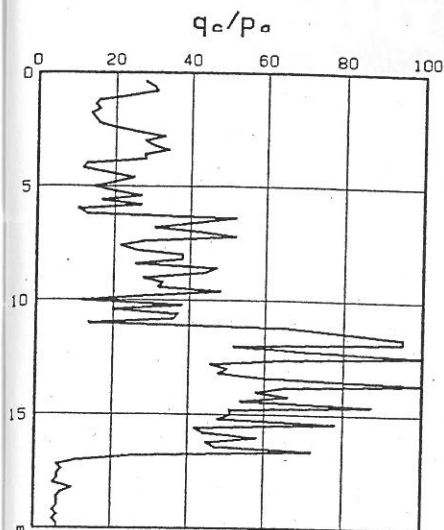


BASSO ALTO MOLTO ALTO
RISCHIO DI LIQUEFAZIONE

geoLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT19
LOCALITÀ: PISA
CANTIERE: CISANELLO

TAV. 62



geOLOG

MAGNITUDO: 6.5
ACCELERAZIONE ORIZZONTALE: .24g
RIFERIMENTO: CPT20
LOCALITA: PISA
CANTIERE: CISANELLO

TAV. 63

APPENDICE

Estratto da:
Crespellani, Madiari, Vannucchi, 2003

"CPT-based liquefaction hazard maps for an Italian coastal area"

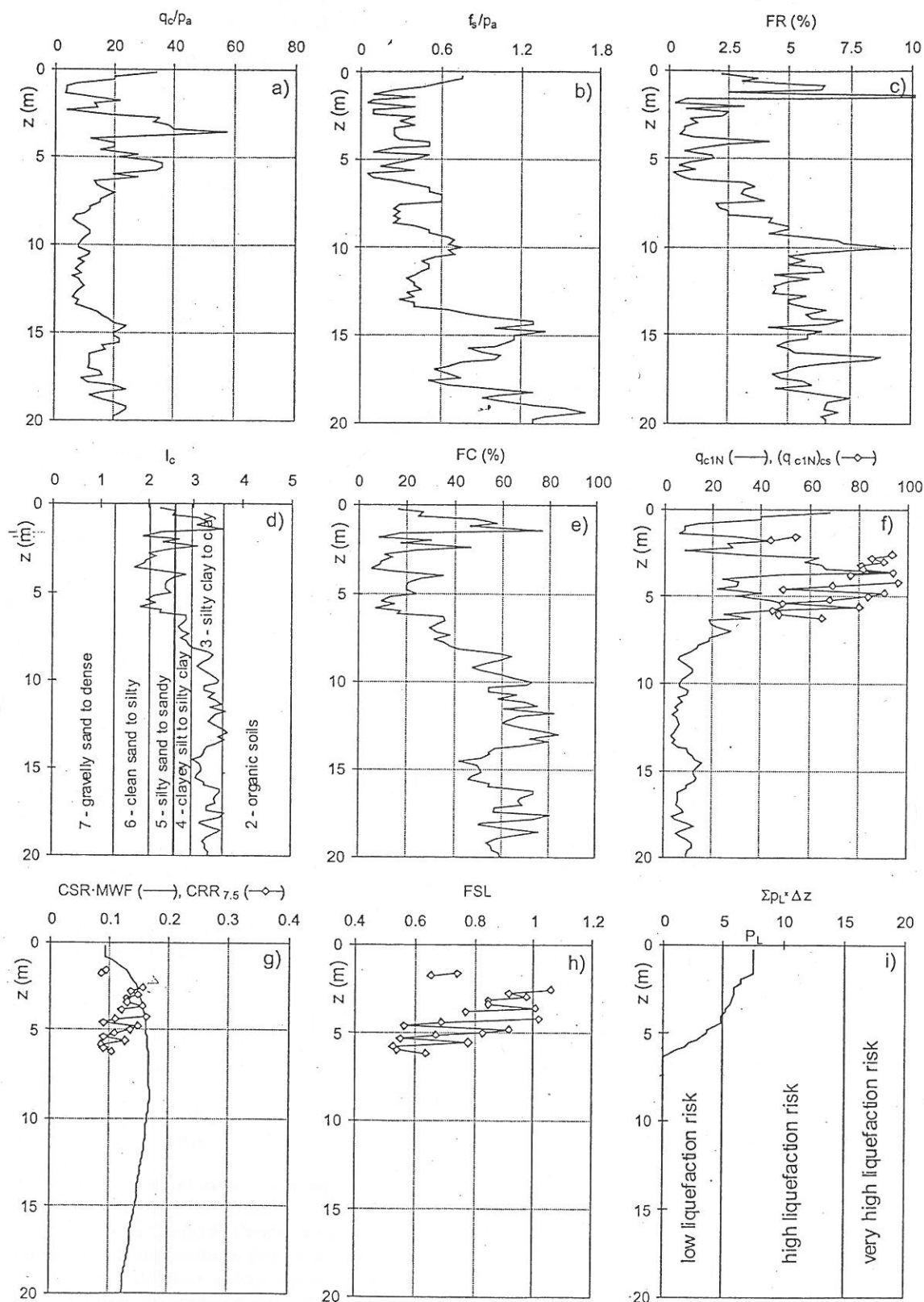


Fig. 8 - CPT 108: q_c and f_s profiles and trends with depth of FC , $CRR_{7.5}$, weighted CSR , FSL and P_L .

Fig. 8 - CPT 108: profili di q_c ed f_s e andamento con la profondità di FC , $CRR_{7.5}$, CSR corretto, FSL and P_L .

6. Numerical analyses by means of the Robertson and Wride Procedure

The R&W procedure was applied to 1009 CPT-profiles, by calculating for every explored vertical:

- the profile with depth of the liquefaction resistance, expressed in terms of cyclic resistance ratio (CRR);
- the profile of the expected seismic action expressed in terms of the cyclic stress ratio (CSR);
- the profile of the liquefaction resistance factor, FSL, expressed as the ratio between capacity of the soil to resist liquefaction, CRR, and seismic demand on the soil deposit, CSR:

$$FSL = \frac{CRR}{CSR} = \left(\frac{CRR_{7.5}}{CSR} \right) \cdot MSF = \left(\frac{CRR_{7.5}}{CSR} \right) \cdot \frac{1}{MWF} \quad (2)$$

where $CRR_{7.5}$ is the cyclic resistance ratio for magnitude 7.5 earthquakes, CSR is the cyclic stress ratio, MSF is the "magnitude scaling factor" and MWF is the "magnitude weighting factor".

a) Cyclic resistance ratio $CRR_{7.5}$

Synthetically, for every soil layer of each CPT, the following computations were carried out:

- Normalisation of the penetration cone resistance by means of the relationship:

$$q_{c1N} = \left(\frac{q_c}{p_a} \right) \cdot \left(\frac{p_a}{\sigma_{v0}} \right)^{0.5} \quad (3)$$

- Soil classification, through the soil behaviour type index, I_c , given by:

$$I_c = \sqrt{(\log F + 1.22)^2 + (\log Q - 3.47)^2} \quad (4)$$

where Q and F are the normalised cone penetration resistance and the normalised friction ratio, respectively defined as:

$$F = \frac{f_s}{q_c - \sigma_{v0}} \times 100 \quad (5)$$

$$Q = \frac{q_c - \sigma_{v0}}{\sigma_{v0}} \quad (6)$$

For soil classification reference was made to the bilogarithmic diagram [ROBERTSON, 1990], which, on the basis of Q and F , divides soils into 9 classes ranging from sensitive, fine grained soils (class 1) to very stiff fine grained and heavily overconsolidated or cemented soils (class 9). By excluding classes 1, 8 and 9, which are not of concern for liquefaction, in order to identify the soils of classes 2 to 7, it is sufficient to calculate I_c at each layer and verify the class to which it be-

longs on the basis of Tab. I, in which the soil behaviour type boundaries are indicated.

It should be noted that the value of $I_c = 2.6$ represents, in the R&W method, a threshold above which the soil is considered as not being susceptible to liquefaction.

- Estimate of the fines content, FC, from the values of the I_c index, by means of the relation:

$$FC = 1.375 I_c^3 - 3.5 \quad (7)$$

- Correction of the normalised penetration cone resistance for taking into account the presence of fine particles, by means of a factor, K_c , dependent on the I_c index, in order to evaluate the equivalent clean sand normalised penetration resistance $(q_{c1N})_{cs} = K_c \cdot q_{c1N}$
- Evaluation of the cyclic resistance ratio, using the equation:

$$CRR_{7.5} = 93 \cdot \left(\frac{(q_{c1N})_{cs}}{1000} \right)^3 + 0.08 \quad (8)$$

where $30 < (q_{c1N})_{cs} < 160$

b) Cyclic stress ratio CSR

The cyclic stress ratio CSR was estimated by means of the following semi-empirical equation [YOUNG & IDRIS, 2001]:

$$CSR = \left(\frac{\tau_{av}}{\sigma_{v0}} \right) = 0.65 \cdot \left(\frac{a_{max}}{g} \right) \cdot \left(\frac{\sigma_{v0}}{\sigma'_{v0}} \right) \cdot \gamma_d \quad (9)$$

in which:

- a_{max} is the horizontal peak acceleration of the design earthquake at the surface of the deposit;
- g is the acceleration of gravity;
- σ_{v0} and σ'_{v0} are, respectively, the total and the effective vertical overburden stresses;

Tab. I - Boundaries of soil behaviour type index, I_c , for soil classification [ROBERTSON, 1990].

Tab. I - Valori limite dell'indice del potenziale di liquefazione I_c per differenti classi di terreno [ROBERTSON, 1990].

Soil Behaviour Type Index, I_c	Zone	Soil Class
$I_c < 1.31$	7	Gravelly sand to dense sand
$1.31 < I_c < 2.05$	6	Sands: clean sand to silty sand
$2.05 < I_c < 2.60$	5	Sand mixtures: silty sand to sandy silt
$2.60 < I_c < 2.95$	4	Silt mixture: clayey silt to silty clay
$2.95 < I_c < 3.60$	3	Clays: silty clay to clay
$I_c > 3.60$	2	Organic soils: peats

- r_d is a stiffness reduction coefficient which, in simplified procedures, is assumed to vary only with the depth.

At the site, $a_{\max}$ was assumed 0.275g. The coefficient r_d , which provides an approximate correction for flexibility of the soil profile, was determined using bi-linear functions [YOUD & IDRIS, 2001] as follows:

$$r_d = \begin{cases} 1 - 0.00765 z & \text{for } z \leq 9.15 \text{ m} \\ 1.174 - 0.0267 z & \text{for } 9.15 < z \leq 23 \text{ m} \end{cases} \quad (10)$$

where z is the depth below ground surface in metres.

c) Liquefaction safety factor FSL

As far as the magnitude weighting factor is concerned, for earthquakes of a magnitude greater than 7.5, the recent NCEER recommendations [YOUD & IDRIS, 1997] suggest using MSF values given by the equation:

$$MSF = \frac{10^{2.24}}{M^{2.56}} \quad (11)$$

and for earthquakes of a magnitude of less than 7.5, intermediate MSF values between those of equation (11) and those calculated with the following equation [ANDRUS & STOKOE, 1997]:

$$MSF = \left(\frac{M}{7.5} \right)^{-3.3} \quad (12)$$

In the case under study, the magnitude of the design earthquake being $M = 6$, MSF value was 1.93 and MWF = 0.518.

By way of example, some results of the application of the Robertson & Wride procedure to soil profiles from CPT108 (Fig. 6) are reported in Fig. 8. In this figure, the values of FC and $CRR_{7.5}$ are plotted against depth as well as the values of the resistance demand, CSR, corrected with the magnitude weighting factor, MWF, and of the safety factor, FSL.

In particular, Figs. 8e and 8g evidence the good match between FC and $CRR_{7.5}$ values and the laboratory test results from the samples retrieved in borehole S10 close to CPT108, described above in paragraph 4.

Moreover, it should be noted that soil was considered as not being susceptible to liquefaction if $I_c > 2.6$ (see paragraph 5 point 2) and, in this case, CRR and FSL were not calculated.

7. Criteria for drawing up liquefaction hazard maps

To map the liquefaction hazard, a single numerical value must be associated with each profile examined.

Even if other ways of defining the severity of liquefaction have been proposed [SHINOZOUKA & OHTOMO, 1989], the most popular synthetic index, representative of the extension of liquefaction along a vertical, is the liquefaction potential index, P_L , introduced by IWASAKI *et al.* [1978; 1982] and defined as follows:

$$P_L = \int_0^{20} F(z) \cdot w(z) dz = \int_0^{20} p_L dz \quad (13)$$

in which z is the depth below the ground surface, measured in metres and defined up to a value of 20 m, beyond which liquefaction phenomena can be excluded, $F(z)$ is a function of the liquefaction resistance factor, FSL, which assumes a value of zero for $FSL > 1$, and is the complement to 1 for $FSL < 1$; $w(z) = 10 - 0.5 z$ is a linearly-decreasing function of depth z .

The P_L values range from 0 to 100, but IWASAKI *et al.* [1978] demonstrated that P_L values greater than 15 identify a high susceptibility to liquefaction damage. The liquefaction indexes associated to liquefaction risk levels are shown in Tab. II.

This index, which can be applied to all the methods that involve calculation of the liquefaction resistance factor, FSL, and that many authors have used for compiling liquefaction hazard maps [IWASAKI *et al.*, 1982; KUSANO *et al.*, 1988; YASUDA & WAKAMATSU, 1999] has been adopted in the case under study.