RISK ASSESSMENT IN TUNNEL DESIGN

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SUMMARY: With the development and implementation of the ADECO-RS approach, it is now possible to set risk criteria for projects. These are extremely useful in assessing risk probabilities in final designs and are indispensable for costing projects. In fact the procedure proposed not only assesses the risks inherent in a project in terms of construction times and costs, but also clearly and critically identifies the geological and geomechanical parameters that must be acted on and investigated in order to restore variations in the design to financially acceptable values.

1 INTRODUCTION

For some years now there has been growing technological development in the field of underground works that has resulted in design and construction approaches capable of calculating construction times and costs with great accuracy. One of these approaches is known as the ADECO-RS approach, which has been employed in Italy in recent years to design and construct major underground works. There are many examples of its use and one is that of the crossing of the Apennines on the Bologna to Florence section of the High Speed rail network. In this case there was a difference of only a few percentage points between design forecasts and actual results for each tunnel section type, despite the size of the project with over 90 km of tunnel and the very difficult geological and geomechanical conditions in 50% of the works. It can therefore be said that the high degree of accuracy in making predictions typical of civil engineering projects on the surface has also been attained for underground works.

As far as finding finance for projects is concerned underground project may be seen on the same level as other projects when assessing the pros and cons and the risks of construction.

As far as the risks in underground projects are concerned, with new methods not only is it possible to guarantee safe working conditions and to apply the principles of ISO 9000 quality assurance procedures, it is also possible to analyse all the possible variations in construction times and costs for all possible scenarios. This in our opinion is not possible if methods based on excavation or geomechanical classification systems are adopted. In fact in these cases, the main risk factor does not lie in the choice of parameters or in geological uncertainties, but mainly in the difficulty in applying such classification systems to all types of ground and in the overly subjective judgement required in attributing excavation classes during construction and in deciding on the change to the corresponding tunnel section type. Design procedures capable of handling all design uncertainties are required, which employ mathematical methods based on the most probable parameters to calculate the stress-strain behaviour of the cavity (at the face and around the cavity). This is the only way in which the differences between the most probable design and the most pessimistic and optimistic designs can be calculated, depending on the reliability of the survey input, at any stage of the design. The process can be iterated until the final design is approved. Behaviour that was forecast at the design moment can be assessed during construction by measuring actual stress-strain parameters and comparing them with those forecast. Differences from average forecast values can be seen and if necessary appropriate calibration of advance methods and corrective action can be proposed in order to effect a return to the design specifications.

In this paper probabilistic methods are adopted to furnish criteria for identifying not just the risks of a project but also which geological and geomechanical parameters should be employed to force possible variation in construction times and costs within limits acceptable to investors.

The method proposed can be divided into five distinct phases:

a) the acquisition of geological and geomechanical data from the surface or from *in situ* or laboratory tests in order to characterise the ground to be tunnelled;

b) division of the route of the tunnel into sections with uniform geological and hydrogeological characteristics (uniform geomechanical behaviour) (Groups 1-2-3-4-5) and location of joints, overthrusts and discontinuities of tectonic origin which for simplicity we will call "faults";

c) the choice of the most significant geotechnical parameters to attribute to each section to predict the stress-strain behaviour of the tunnel near the face;

d) the attribution of a stress-strain behaviour category to each section that is identified as uniform and the choice of the most probable tunnel section types. Functions with the most significant effect on times and costs are defined at the beginning and a table is compiled, divided into the sections identified in phase b), with the construction times and costs calculated for each section.

e) the results obtained (sum of times and costs required for each section) are shown in a scattergram of times and costs containing 500 tunnel designs (more than sufficient to produce reliable results).

These phases coincide with the main phases of the design moment in the ADECO-RS approach.

2 SURVEY PHASE

In this phase information is collected on the main geomechanical characteristics of the ground and sections with h omogeneous characteristics are grouped together. The purpose of the study is to evaluate and process all the data from the measurements and tests performed during the survey campaigns. Data is obtained by means of the following: geological measurements: measurements of outcropping geological structures, measurements of geological structures using television probes, technical geological measurements, detail stratigraphical maps, geomechanical laboratory tests, *in situ* geomechanical tests, geophysical measurements and tests.

It is essential to obtain sufficient information on the orientation and characteristics of faults and overthrusts if uniform sections are to be identified accurately. A careful analysis of geological structures is therefore performed to ascertain the surface development of discontinuities, their length and the number of outcroppings. The objective is to obtain a sufficient number of sample values of the angle and direction of faults and overthrusts with respect to the centre line (or to North, and in this case one speaks of immersion) of the tunnel both in plan and elevation view.

Generally speaking data samples contain errors due to the limited number of samples, the type of sampling and whether the sample is of *in situ* or laboratory size.

It is possible to calculate the number of elements n to be sampled required for an average estimated sample error μ that is not higher than the error ε (figure 1).

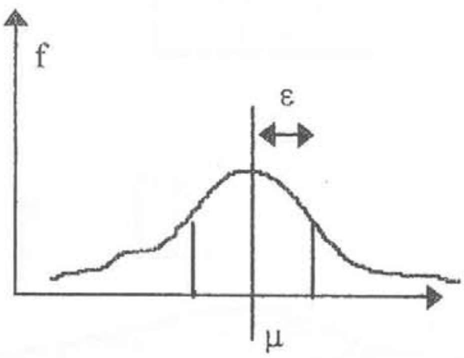


Figure 1: distribution of average samples

For the sake of example, let us assume we have 5 samples (5 angles: $x_1=10°$, $x_2=15°$, $x_3=20°$, $x_4=25°$, $x_5=30°$) for which we calculate the average ($\mu_x = 20°$) and the standard deviation ($\sigma_x = 7{,}90$). In order to be able to state with 95% probability of certainty that the average sample ($\mu_x = 20°$) differs from the average μ (unknown) of the population by not more than the difference $\varepsilon = 1$, the number of elements n required in the sample is calculated as follows:

$$\mathrm{P}\left[\left|\mu_x - \mu\right| < \varepsilon\right] \geq 1 - \frac{\sigma_x}{\varepsilon^2 n} \geq 1 \tag{1}$$

in the case in question:

$$\mathrm{P}\left[\left|20 - \mu\right| < 1\right] \geq 1 - \frac{7{,}9}{n} \geq 0{,}95 \Rightarrow n = 158 \tag{2}$$

The same calculation can be repeated either by setting a value for the probability P and varying the error ε or conversely by setting the error ε as constant and varying the

probability P. Naturally the number n increases considerably as the probability P approaches 100% (for P=98% and $\varepsilon = 1$, then n=395).
Where P=90% and $\varepsilon = 2$, the number of elements required in the sample n = 19.7.
The equation for calculating the standard deviation assuming that the discrete distribution of the elements in the sample approximates a uniform distribution is:

$$\sigma = \sqrt{\frac{n^2 - 1}{12}} \tag{3}$$

If the variance σ_2 of the population is unknown and the distribution can be considered as approximately normal, it can be replaced with an estimate obtained by means of the correct estimator:

$$\overline{\sigma}^2 = S_n^2 = \frac{1}{n-1}\sum_{i=1}^{n}\left(X_i - \mu_x\right) \tag{4}$$

Finally where there is a discrete distribution of a population different from those above, then appropriate equations can be found in the literature.
The survey data mentioned above, together with information on the area in question already available in the literature is then used to divide the entire tunnel into geologically and geomechanically uniform sections with calculation of the average length and identification of the predicted position of faults, overthrusts and joints (Fig. 2).

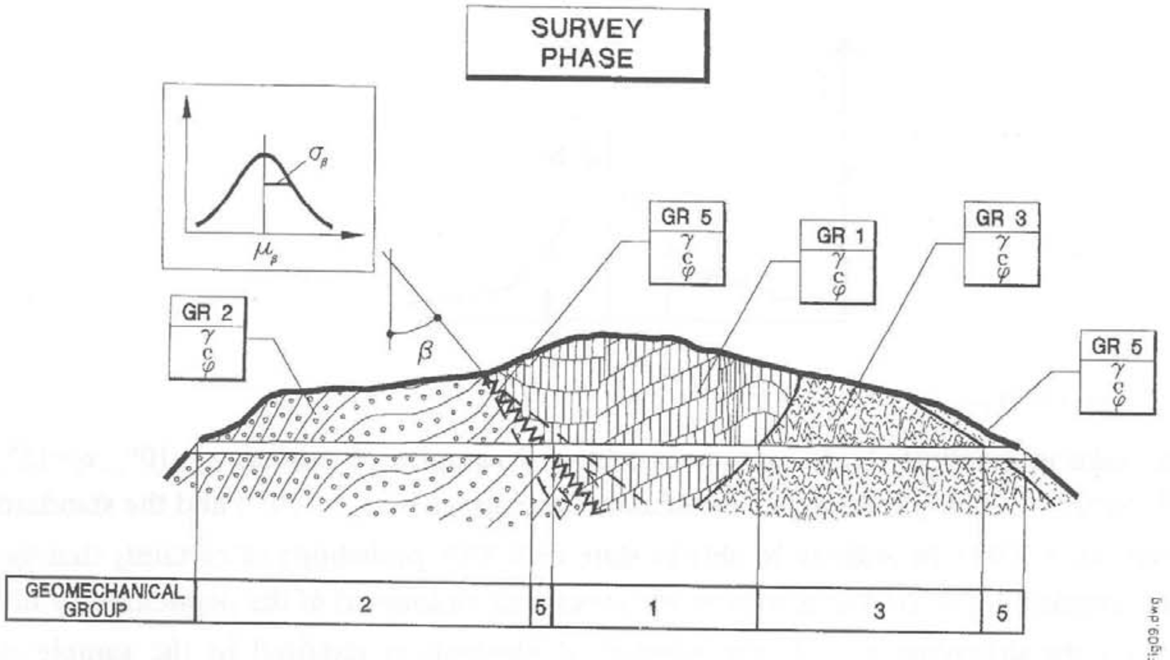


Figure 2. Survey Phase

The most significant parameters for the risk analysis in question were felt to be: the position of faults and overthrusts (angle of intersection in plan and elevation view), the number of bore-holes per kilometre and from a geomechanical viewpoint, the deformation modulus E, cohesion C, the angle of friction ϕ and also the overburden H, the weight of the ground γ and the Poisson coefficient v.
Variation in the size of these sections will depend on the location of faults, overthrusts and joints, defined by the parameter that we will call L_{fault} given by the equation:

$$L_{fault} = \mathrm{MAX}\left(2L_\beta, 2L_\alpha\right) + \text{Area of disturbance} \tag{5}$$

where the *Area of disturbance* is defined by the geologist and

by $x=\dfrac{\frac{D}{2}}{tg(90°-\beta)}$; $y=\dfrac{\frac{D}{2}}{tg\alpha}$ indicated in figures 3 and 4 below (6)

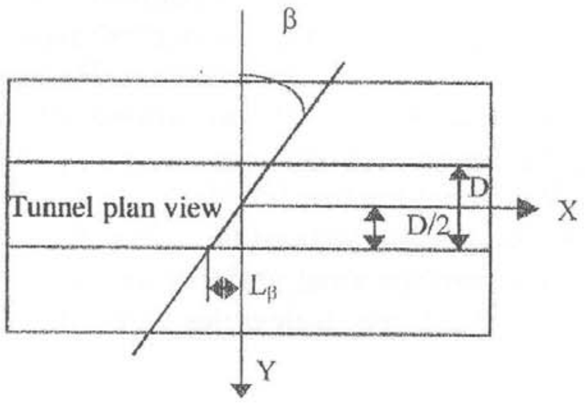


Figure 3

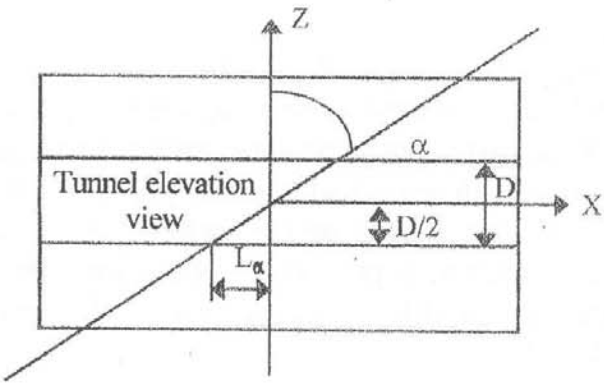


Figure 4

The parameters, in addition to persistence, felt to be significant for determining the lengths of the sections in fault, overthrust and joint zones are: the angle of intersection with the centre line of the tunnel in plan (β) and elevation (α) view. The angle σ is expressed with a normal distribution of the type:

$$\alpha = RiskNormal\ (\mu_\alpha ; \sigma_\alpha) \tag{7}$$

where σ_α is the standard deviation for the angle of the plane (x,z) of the fault, overthrust or joint that delimits the section. The value for σ_α is defined on basis of structural geological measurements made on each tectonic feature that intersects the tunnel (faults, overthrusts and joints between strata present along the profile for each tunnel in question or for part of it). For each geological structure in stratigraphical contact a range of variation in the plane x, z is assessed in order to establish the average value and the maximum variation from that value (σ_α).
The normal function RiskNormal is obtained by using the programme @Risk but this does not exclude the use of other programmes that make similar use of probability functions.
The angle β is expressed by a normal distribution of the type:

$$\beta = RiskNormal\ (\mu_\beta ; \sigma_\beta) \tag{8}$$

where μ_β is the average angle and σ_β is a reasonably prudent value for standard deviation obtained from the sampling of the structural geology described previously (phase b) in the introduction) subtracting half of the lengths L_{fault} that mark the limits of each section (on the left and on the right):

$$Varied\ length\ = Medium\ length - \frac{L_{fault\ left}}{2} - \frac{L_{fault\ right}}{2} \tag{9}$$

3 CAVITY STRESS-STRAIN BEHAVIOUR PREDICTIONS (DIAGNOSIS PHASE)

Each uniform geomechanical zone is divided into sections belonging to the same behaviour category on the basis of the parameters defined above.
First of all the samples of the various geomechanical parameters are analysed for a given uniform section. They are assumed to be normally distributed and the averages and standard deviations are calculated. At this point the averages of the various parameters are used to establish the average overburden values at which the different face behaviour categories change ($H_{1average}$ between A and B and $H_{2average}$ between B and C with $H_{2average} > H_{1average}$ since the stress state and face instability increases as the overburden increases).
These two average heights can be calculated by the design engineer using characteristic curves with analytical equations or with a finite element programme (FEM). Notes $H_{1averag}$ and $H_{2average}$ it is to divide uniform geomechanical sections into sections of average length (expressed as a percentage of the total length of the corresponding uniform section and termed %average) characterised by the same behaviour category (categories A, B and C) (Fig. 5).

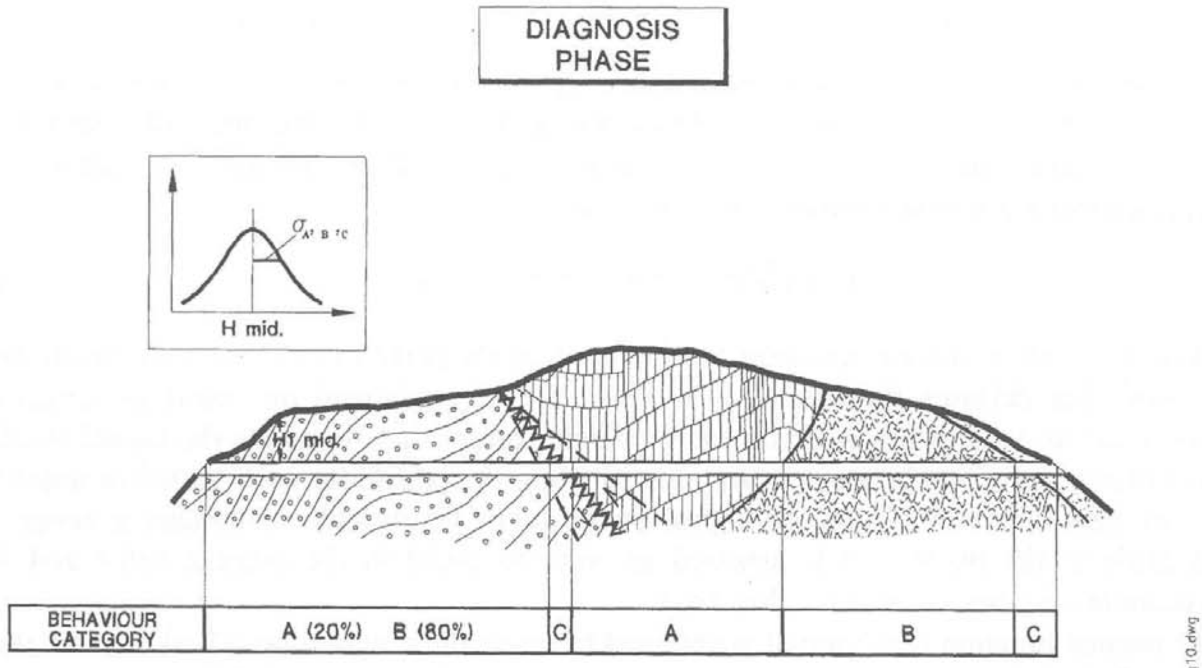


Fig. 5. Diagnosis phase

Finally the variable percentage of the length of the subsection with uniform face behaviour can be expressed by the following normal function:

$$\%\ Variation = RiskNormal\ (\%\ average;\ \sigma_{A,B,C}) \tag{10}$$

where %average is the value defined above and $\sigma_{A,B,C}$ is the standard deviation and depends on the variation of the geomechanical parameters felt to be most significant: E, C, ϕ.
The method used to define that standard deviation is given below.
From the equation

$$R_{CM} = \frac{2 \cdot C}{tg(45° - \frac{\varphi}{2})} \tag{11}$$

it was possible to obtain a single parameter (the *in situ* mono-axial compressive strength R_{CM}) from the two initial parameters ϕ e C.
The respective averages (μ_E, μ_R) and standard deviations (σ_E, σ_R) can be obtained from the variation in E and in R_C,
The direct relationship given in the equation below is assumed to obtain a single standard deviation (σ_{TOT}) from the standard deviations of E e di R_{CM}:

$$\sigma^2_{TOT} = \sum_i W_i^2 \cdot \sigma_i^2 \tag{12}$$

where σ_i consists of σ_E, σ_R and where W_i consists of the weights attributed to those standard deviations and which for simplicity were considered equal to one.
Finally, $\sigma_{A,B,C}$ is calculated taking account of σ_{TOT}, of a coefficient K and of the number of bore-holes per kilometre:

$$\sigma_{A,B,C} = \frac{K_1}{K_2}(\sigma_{TOT}) \tag{13}$$

where K_1 = number of uncertain parameters (in this case E and E e R_{CM}) and $K_2 \leq 1$ a function of the number of surveys (at the discretion of the design engineer).

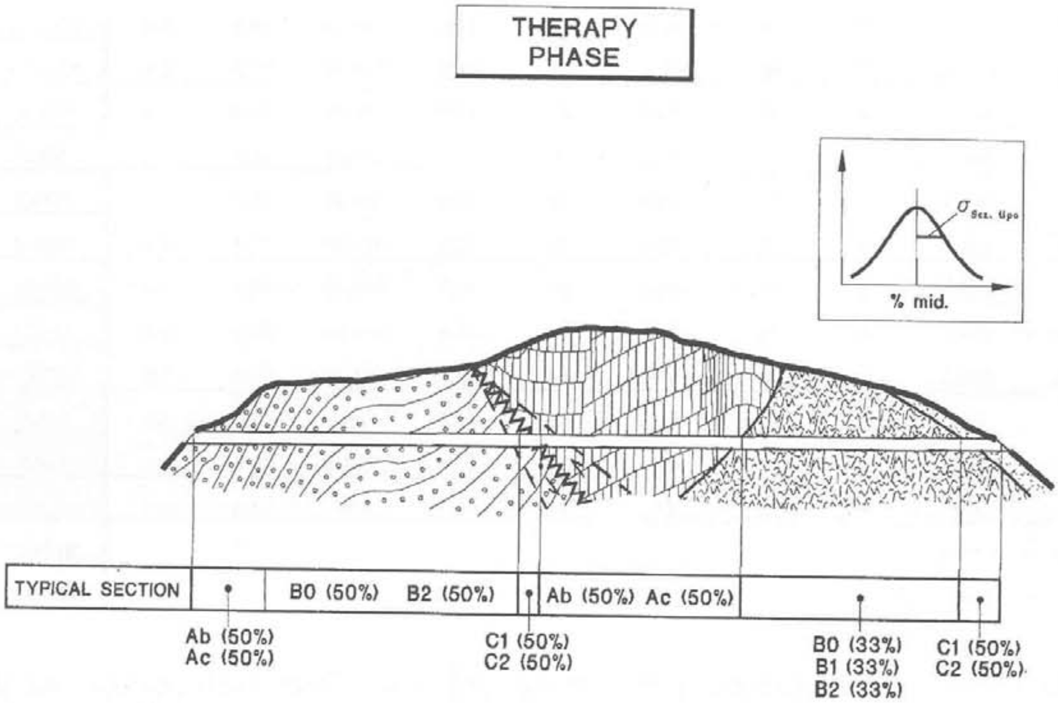


Fig. 6. Therapy phase

4 INTERVENTION FORECASTS (THERAPY PHASE)

Appropriate tunnel section types are attributed to each section with the same behaviour category and the most suitable face and cavity stabilisation techniques are decided along with the range within which these may vary as cavity behaviour varies. Their size is expressed as a percentage of the length of the section considered (Fig. 6).
Finally the percentage variation in the lengths of the subsections in which the same section type is employed must be calculated assuming a normal distribution.
As in the previous case, a similar normal function is adopted:

$$\%\ Section\ type\ variation = RiskNormal\ (\%\ average\ of\ section\ type;\ \sigma_{section}\ type) \quad (14)$$

where % average section type depends on the position and the size of the plasticised zone (using the analysis of characteristic curves or finite element analysis) and the size of the solid load and therefore on the different types of intervention required for stability and where standard deviation $\sigma_{\text{section type}}$ is equal to $\sigma_{A,B,C}$.

5 SUMMARY OF THE ANALYSIS PERFORMED

The table below gives a summary of the analysis performed.
The results, which is to say the time required in months and the cost of the whole tunnel or part of it, are given at the bottom right of the table.

1 Section	2 Length	3 L.@risk	4 Group	5 Category	6 %	7 Section Type	8 %	9 S.T. length	10 Cost	11 Dayly produc.	12 Cost L.@corr.	13 Time months
1	1778	1753,0	2	A	20,0	Ab	50,0	175,30	16,5	3,8	2892,6	1,6
1	1778	1753,0	2	A	20,0	Ac	50,0	175,30	17,5	3,3	3067,8	1,8
1	1778	1753,0	2	B	80,0	B0	50,0	701,20	18,0	2,3	12621,6	10,4
1	1778	1753,0	2	B	80,0	B2	50,0	701,20	23,0	1,8	16127,6	13,4
2	0	50,0	5	C	100,0	C1	50,0	25,00	32,0	1,4	800,0	0,6
2	0	50,0	5	C	100,0	C2	50,0	25,00	34,0	1,3	850,0	0,6
3	888	863,0	1	A	100,0	Ab	50,0	431,50	16,5	3,8	7119,8	3,8
3	888	863,0	1	A	100,0	Ac	50,0	431,50	17,5	3,3	7551,3	4,4
4	990	934,0	3	B	100,0	B0	33,3	311,33	18,0	2,3	5604,0	4,6
4	990	934,0	3	B	100,0	B1	33,3	311,33	20,0	2,0	6226,7	5,2
4	990	934,0	3	B	100,0	B2	33,3	311,33	23,0	1,8	7160,7	5,9
5	0	56,0	5	C	100,0	C1	50,0	28,00	32,0	1,4	896,0	0,7
5	0	56,0	5	C	100,0	C2	50,0	28,00	34,0	1,3	952,0	0,7
6	0	0,0	5	C	0,0	C1	100,0	0,00	32,0	1,4	0,0	0,0
	L.tot	L.@tot									Sum£.	Sum t
	3656	3656									71869,7	36,6

Table 1

There are two faces in the example given in the table (one from each portal) and the metres driven on those faces are shown by different colours in order to be able to show the presence of gas (the lighter rows). This means adopting lower coefficients for daily advance rates (e.g. 0.75 in column 11). The question is exactly the same if the presence of water or other types of problem that slow tunnel advance are envisaged.

The table is created with a variable number of rows depending on the number of sections with uniform geomechanical behaviour (faults and overthrusts are also considered sections with geomechanical behaviour), according to the group that the sections belong to (fault and overthrust sections are categorised in group 5) and to the tunnel section types specified.

In table 1 for example, the first section is in group two and has 4 rows because section types Ab, Ac, B0 and B2 are specified. The second section in table 1 is in a fault zone and is therefore in group 5 (the worst). The section occupies 2 rows because the tunnel section types specified in this case are C1 and C2. There are a total of 6 sections and 14 rows.

Let us now proceed to a description of the content of the various columns:

1. division of the total route (phase b in the introduction) into sections of uniform geomechanical behaviour (groups 1 to 5 shown in column 4);
2. average length in metres of the individual sections of column 1 considering, at the time, the joints, overthrusts and faults of zero length located previously (phase b);
3. the length of the faults, overthrusts and joints is defined for simplicity as L_{faults} as calculated under section 2 above;
4. groups 1-2-3-4-5 with uniform geomechanical behaviour;
5. division into sections with the same face behaviour (behaviour categories: A (face stable) B (stable in the short term), C (unstable);
6. percentage variation in the length of the sections with uniform face behaviour with the normal distribution centred on the average (%average) and with standard deviation ($\sigma_{A,B,C}$).
7. section types adopted (Ab, Ac, B0, B1, B2, C1, C2, C3, C4) within the different subsections with uniform face behaviour;
8. percentage variation, with normal distribution, in the length of the additional subsections in which the same tunnel section type is adopted;
9. lengths in metres in which the same tunnel section types are adopted;
10. Costs per linear metre of tunnel section type calculated as follows:

$$Cost\,/\,m = (n_2\ average\ cost\ /\,m)\cdot\left(\frac{n_2\ \cdot average\ cost/m\ \cdot dayly\ average\ production\ rates}{Dayly\ production\ rates}\right) \qquad (15)$$

   where n_1 and n_2 respectively give the percentage cost of materials and labour for a given tunnel section type.

11. daily advance rates of the face in metres (rates predicted on the basis of data in the literature) expressed by means of a uniform distribution that takes account of the variation in different section type methods.

   The following function is inserted in the cells:

$$RiskUnifom(a\,;b) \qquad 16)$$

   according to which the probability that all the rates included in the range a,b will occur is the same. The function is multiplied by a coefficient that takes account of possible slower rates (e.g. because of water heads, gas, etc.).

   An example of daily rates for different section types is given below:

SECTION TYPE	RANGE (a,b) OF DAILY PRODUCTION RATES (m)
Ab	3,5 - 4
Ac	3 - 3,5
B0	2 - 2,5
B1	1,75 - 2,25
B2	1,5 - 2
C1	1,3 - 1,5
C2	1,2 - 1,4
C3	1,1 - 1,3
C4	1 - 1,2

Table 2

12. cost of driving the metres of tunnel contained in column 9 using the corresponding tunnel section type;
13. time required to drive the length of tunnel in column 9 using the corresponding tunnel section type.

At this point the Monte Carlo procedure is applied to the normal distribution that best approximates the variation in the angles of the faults, overthrusts and joints. The same procedure is also applied to the normal distribution adopted to determine the division of the geomechanically uniform behaviour sections into subsections (shown as a percentage of the total length of the section) of uniform face behaviour (A,B,C). The same procedure was also applied to the uniform distribution assumed for average daily advance rates.
The procedure, and therefore the calculation of the total time required and cost of the project or part of it, was repeated *n* times by using a macro shown below:

```
Sub Galleria A()
'
' Galleria A Macro
'
Dim i As Integer
Worksheets("foglio1").Activate
        For i = 1 To 500
                Calculate
                Range("N15").Select
                Selection.Copy
                Cells(i + 1, 16).Select
                Selection.PasteSpecial      Paste:=xlValues,       Operation:=xlNone,
                SkipBlanks:= _
                False, Transpose:=False
        Next i
End Sub
```

There now follows a brief demonstration that the number n of iterations chosen is sufficient to guarantee that the results are not affected by excessive approximation.
Harr (1987) used the Monte Carlo procedure to reconstruct the possible slip surfaces of slopes. He assumed that the normal distribution was known with a margin of error equal to ε, and obtained a relationship of this type:

$$n = \left(\frac{h_{\alpha/2}^{2}}{4\varepsilon^{2}}\right)^{m} \tag{17}$$

where α is the reliability of the estimate of n, and $h_{\alpha/2}$ is given by:

$$h_{\alpha/2} = \psi^{-1}\left(\frac{1}{2} - \frac{\alpha}{2}\right) \tag{18}$$

with Ψ obtainable from tables associated with the normal distribution and finally m is the number of variables considered.

If, for example, one variable only is considered (m =1) with a margin of error for the initial normal distribution of 5% (ε = 0.05) and with the reliability attributed to the value n to be obtained equal to 95% (α = 0.05), then the result $h_{\alpha/2}$ = 1.96 is obtained from which it follows that n = 384.

The decision to work with n = 500 therefore guarantees sufficient accuracy.

The result of the analysis is expressed in a time and costs scattergram where the number of points is equal to the number of iterations, each of which represents a different possible design resulting from the application of the Monte Carlo procedure (Fig. 7).

The most probable design is that least distant from the average point (in the centre of the group) which does not necessarily coincide with one of the actual 500 points in the figure.

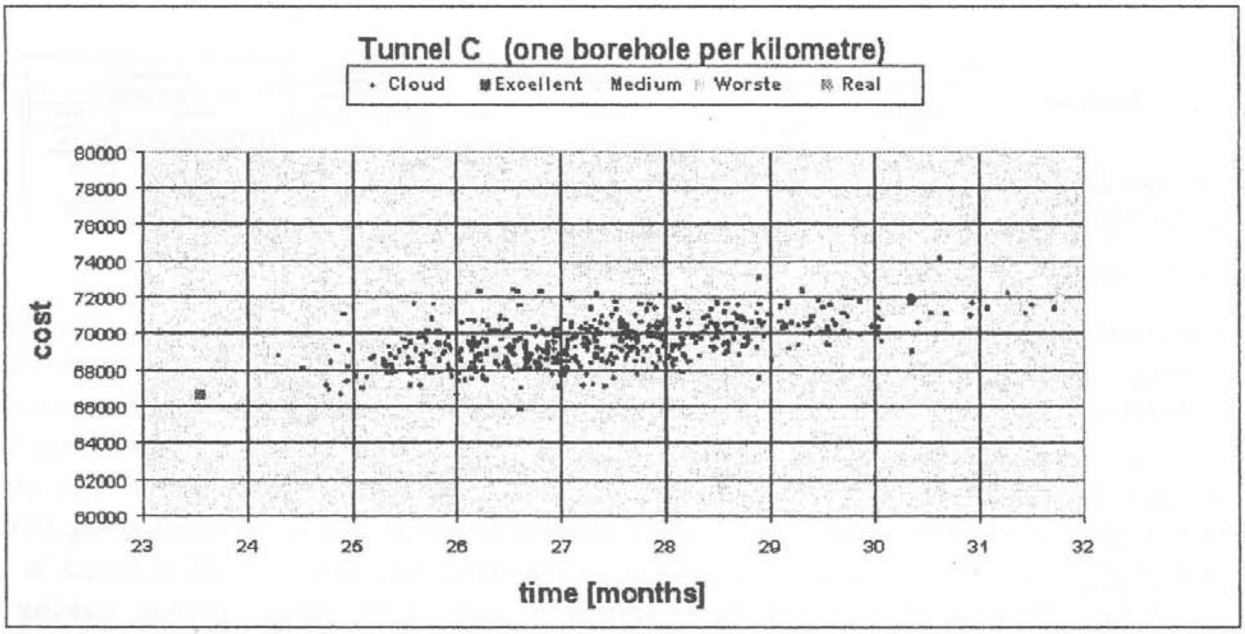


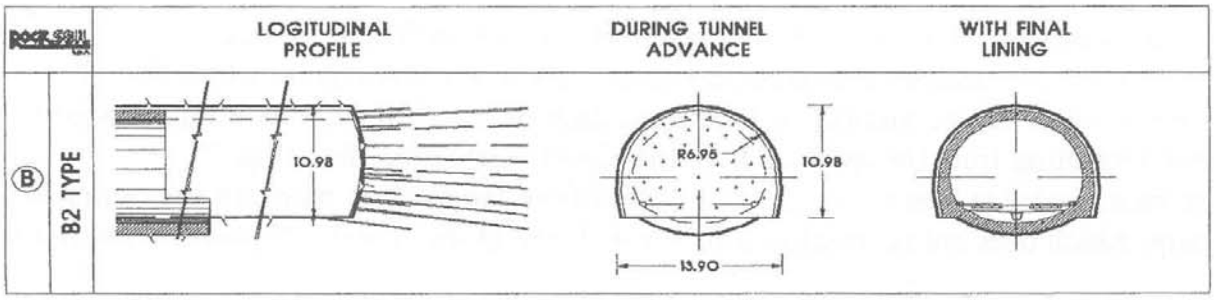
Figure 7 Time and cost scattergram for tunnel C

6 CONCLUSIONS

The risk analysis proposed can be used in the probability sphere to assess which of all possible designs is the most probable in terms of construction times and costs both for an entire project and for a part of it and also to find the difference between this and the most pessimistic and the most optimistic designs.

Account is taken of the geological and geotechnical parameters felt to be most significant in tunnel design to arrive at the scattergram of points felt to be significant, in which each point represents a different design for the tunnel in question. The scattergram not only gives the most probable design but also the variation in possible construction times and costs.

The objective is a design which for each geologically uniform section of the tunnel identifies sections with the most probable uniform behaviour within which it is possible distribute the most probable tunnel section types selected by the design engineer on the basis of the geomechanical conditions and the behaviour of the cavity. What may vary during tunnel advance are the geological and geomechanical characteristics and they may vary within a range of forecast values. The deformation behaviour of the face and the cavity may also vary and these must be responded to with various action by either making adjustments in the implementation of the tunnel section type or by changing the section type to another of those specified for that section of uniform face behaviour (Fig. 8).



GEOLOGY	GEOMECHANICS	OVERBURDEN (m)		DESIGN SECTION TYPES			FORECAST DEFORMATION RESPONSE		VARIABILITY	
					INTERVENTION				INTERVENTION	
	Intrinsic band	From	To	Type	Primary	Final			Primary	Final
Sandy-silty complexes in massive or not well stratified from slightly to moderately cemented facies. Low permeability at the mass scale.	φ =14÷32 c=0.25÷0.5 MPa E=0.5÷3.0 GPa	50	100	B2	45 glass fibre structural elements into the ground advance core Overlapping >5.00m Lattice steel ribs every 1.20 m Shotcrete >30cm	90 cm concrete lining 100cm invert	<10cm	5÷18cm	35÷55 glass fibre structural elements cp advance core Overlapping 5÷7m Lattice steel ribs every 1÷1.4 m Shotcrete 25÷30cm	90 cm concrete lining 100cm invert

Fig. 8 B2 section type and relative variation

The combination of all possible scenarios from the most pessimistic to the most optimistic provides a set of designs in which the average design is also the design with most probability of being implemented. Therefore by evaluating and combining all the possible scenarios identified during the different design phases it is possible to calculate the differences (in terms of times and costs) between the average design and the most pessimistic and the most optimistic. If the difference is acceptable to those investing in the project then we can consider the information on which the design itself is based to be acceptable, otherwise it is necessary to return to each of the design phases seeking to understand which of the design parameters must be refined to reduce the range of variation in designs.

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