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# Excavation Design and Vibration Analysis for the Crossing Under a Railway Station

# 정거장 구조물 횡단 굴착 설계 및 진동 해석

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요 약

지하철과 같은 지하구조물 설계시 기존 구조물 지하를 관통하는 경우가 있다. 본 논문은 기존 역사의 바닥 슬래브 밑에 새로운 지하철용 박스 구조물 상부 슬래브가 위치하게 되는 특별한 경우의 굴착 방법을 제시하고 제시된 굴착방법이 상부 구조물에 어떤 영향을 미치는지를 FLAC을 이용하여 검토해 본 것이다. 기존 구조물이 위치하고 있는 지반이 경암인 경우 발파가 주의 깊게 설계되고 시행된다면 발파로 인한 상부 구조의 진동 영향은 충분히 제어할 수 있는 수준으로 예상되었다.

## 1. INTRODUCTION

### 1.1 General

The new metro line will be constructed under the station of existing line.

The new line has no rock cover under the existing station with room available only for concrete structures in the roof area. Excavation and temporary supporting of the structures of the station are challenging tasks and need careful detailed design before excavation actually takes place.

In this paper preliminary design has been suggested how the excavation and supporting can be carried out under the station. Excavation takes place in several stages followed by temporary supporting.

The blast induced dynamic responses on the structures were analysed with FLAC code to check whether the maximum particle velocity of the existing structure is under the recommended value.

### 1.2 Description of tools used for analyses

The numerical method used for analyses of the lining design and the blasting simulations was a code called FLAC by Itasca[3].

FLAC is a two-dimensional explicit finite difference code which simulates the behaviour of structures built in soil, rock or other materials which may undergo plastic flow when their yield limit is reached.

FLAC is based upon a "Lagrangian" calculation scheme which is well suited for modelling large distortions. Furthermore, FLAC has several built-in constitutive models which permit the simulation of highly non-linear, irreversible response representative of geologic, or similar, materials. The explicit, time-marching solution of the full equations of motion(including inertial terms) permits the analysis of progressive failure and collapse, which are important phenomena in studies related to underground design.

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With the dynamics analysis option in FLAC, the code can be applied to various engineering dynamics problems. Calculations can also be made to evaluate effects of explosive loading, such as underground blasting.
The general solution procedure in FLAG code is shown in Fig. 1

**MODEL SETUP**
(1) Generate grid, deform to desired shape
(2) Specify boundary and initial conditions
(3) Define constitutive behavior and material properties

↓

Step to equilibrium state

↓

Examine the Model Response

↓

**PERFORM ALTERATIONS**
for example :
- Excavate material
- Change boundary conditions

↓

Step to solution

↓

Examine the Model Response

↓

**REPEAT FOR ADDITIONAL ALTERATIONS**

Figure 1. General solution procedure in FLAC.

## 2. EXCAVATION DESIGN

### 2.1 Principle of blasting design

Blasting design is based on the theory and practice developed originally by Langefors and Kihlström[1] and further developed by Vuolio[2] especially in the field of determining the threshold values and vibration control.
At the stage of blasting design determination of the peak particle velocities for buildings and structures is based on preliminary calculations, experience and engineering judgement.
Preliminary calculations for estimating the permitted peak particle velocities for buildings and other structures have been made using following equation.[2]

$$v = F_k \times v_1$$

where
$F_k$ = structural coefficient (Table 1)
$v_1$ = particle velocity versus distance (Table 2)

Table 1. Structural coefficients $F_k^{(2)}$

| Structural categories (structures in good condition) | | Structural coefficient $F_k$ |
|---|---|---|
| 1. | Heavy structures like bridges, piers etc. | 2.00* |
| 2. | Buildings of concrete and steel, rock caverns with shotcrete reinforcement | 1.50* |
| 3. | Office and commercial buildings of brick and concrete. Wood-frame houses on concrete or stone foundation. | 1.20* |
| 4. | Residential buildings of brick and concrete, no light concrete, limestone-sand brick etc. Rock caverns without shotcrete reinforcement. Curing concrete > 7 days old. Electrical cables etc. | 1.00 |
| 5. | Buildings with light concrete structures. Curing concrete 3-7 days old. | 0.75 |
| 6. | Very vibration sensitive buildings like museums, churches, and other buildings with high vaults and great spans. Buildings of limestone-sand bricks. Curing concrete < 3 days old. | 0.65 |
| 7. | Old historical buildings at the point of collapse, like ruins. | 0.50 |

* These values over one are permitted only when a blasting or vibration specialist is assisting in the work.

Table 2. Permitted peak particle velocity (vertical component) $v_1(v_1 = F_h \times v_0)$ as a function of distance (R) for structures and buildings founded on different materials. (The structural coefficient is $F_k$ = 1)

| Distance R(m) | Soft moraine<br>Sand<br>Shingle<br>Clay | Moraine<br>Slate<br>Soft limestone<br>Soft sandstone | Granite<br>Gneiss<br>Quartzic<br>Hard sandstone<br>Hard limestone<br>Diabase |
|---|---|---|---|
| | Wave velocities c | | |
| | 1000-1500m/s | 2000-3000m/s | 4500-6000m/s |
| | Permitted peak particle velocity $v_1$ (mm/s) | | |
| 1 | 18 | 35 | 140 |
| 5 | 18 | 35 | 85 |
| 10 | 18 | 35 | 70 |
| 20 | 15 | 28 | 55 |
| 30 | 14 | 25 | 45 |
| 50 | 12 | 21 | 38 |
| 100 | 10 | 17 | 28 |
| 200 | 9 | 14 | 22 |
| 500 | 7 | 11 | 15 |
| 1000 | 6 | 9 | 12 |
| 2000 | 5 | 7 | 9 |

As a threshold value for the permitted peak particle velocity 50-70 mm/s has been applied in the design for heavy concrete structure of the existing station. These applied values are quite low and conservative as well compared to the values calculated by the equation(2), which would suggest values in the range of 80 - 100 mm/s for heavy structures like bridges, piers etc.
Measured values of rock factor k were provided as background material for blasting design. Values that are used in the blasting design were estimated also according to the distance between the blasted zone and the structure as well as according to the previous experience.

## 2.2 Design of blasting schemes

### 2.2.1 Blasting under the existing station

The new line has no rock cover under the existing station with room available only for concrete structures in the roof area. Excavation and temporary supporting of the structures of the station are challenging tasks and need careful detailed design before excavation actually takes place.
In this paper, preliminary design has been suggested how the excavation and supporting can be carried out under the station. Excavation takes place in several stages followed by temporary supporting. Following stages, which refer to plan and cross section have been suggested.

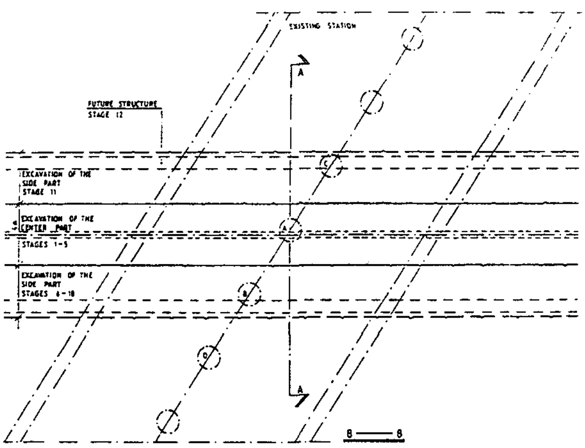



figure 2. Plan of Existing Station and New Line

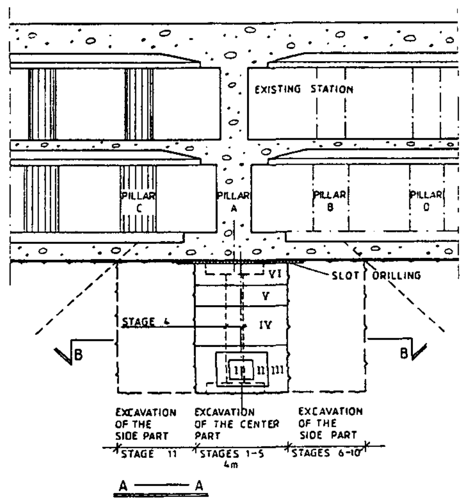



figure 3. Gross Section and Excavation Stages

### 2.2.2 Excavation stages under the station

**Stage 1.** Forces of the pillar A are transferred to pillars B and C. Future rock walls are anchored through the floor slab with outwards inclined rock anchors (if possible). Pregrouting of the future rock walls.

**Stage 2.** Slot drilling of the centre part in the contact between the existing foundation and the rock. The purpose is to create a continuous slot to damp the blast vibrations.

**Stage 3.** The centre part is excavated in several stages under the existing station.
Temporary rock support for working safety reasons is installed.

**Stage 4.** Temporary foundation and temporary steel pillar are constructed under the pillar A.
**Stage 5.** Forces are transferred to pillar A and to the temporary pillar. The temporary pillar is protected for instance by filling the surrounding with sand.
**Stage 6.** Forces of the pillar B are transferred to pillars A and D.
**Stage 7.** Slot drilling in the contact between the existing foundation under the pillar B and the rock.
**Stage 8.** The side part is excavated in several stages under the existing station.
Permanent rock support is installed to support the side walls.
**Stage 9.** Temporary foundation and temporary steel pillar are constructed under the pillar B.
**Stage10.** Forces are transferred to pillar B and to the temporary pillar. The temporary pillar is protected.
**Stage11.** The other side is constructed the same way.
**Stage12.** Final concrete structure of the new tunnel is constructed.
**Stage13.** Temporary pillars are removed.

The concrete walls of the existing station can temporarily be supported during the excavation using the same principle as with the pillars. If the structural capacity of the existing bottom slab of the station is not sufficient enough to carry the traffic loads selfweight etc. in different construction stages extra temporary support should be installed.
4m wide heading with 1.5m advance per round is excavated first under the station in stages 1-5. According to the following specifications:

| Excavation | k | v(mm/s) | R(m) | Qmom(kg) |
|---|---|---|---|---|
| Cut of the heading | 250 | 50 | 8 | 0.9 |
| Enlargement no. 1-2 of the heading | 250 | 50 | 6 | 0.6 |
| Enlargement no. 3 of the heading | 300 | 70 | 4 | 0.4 |
| Enlargement no. 4-5 of the heading | 300 | 70 | 2 | 0.1 |

$$v = k\sqrt{\frac{Q}{R^{1.5}}}$$

v = Allowed Velocity
k = Coetticient of the rock
Q = Amount of Explosive
R = Distance

When the excavation stages of the heading are completed the excavation of the other side part follows. It comprises the stages 6-10. The distance between the contour holes of the final rock wall is 400 mm and advance per round is 1.5m. In this paper, blast specifications of the side part were not included since, in the dynamic analysis, only the cut blast of the pilot tunnel was modelled.

### 2.3 Simulations of blast induced dynamic responses

The blast induced dynamic responses in the structures were analysed with FLAC code at the section where the metro tunnel undergoes the slab of the existing station building.
In the section the tunnel is in hard rock.
The input data FLAC analyses were based both on the site investigation data available at the site and on the experience on similar rock conditions. The site investigations consisted of core drilling loggings, pressurementer tests, down hole tests and laboratory tests.
Parameters used for rock were, $E=500\times10^5 t/m^2$, $\upsilon=2.65t/m^3$, $c=50t/m^2$ and $10t/m^2$, $\Phi=35°$ and $30°$ , $\sigma$

$_t$=50t/m² and 10t/m².

Stress measurements were not conducted at the site. As in-situ state of stress, Ko value(ratio between horizontal and vertical stress)of 0.5 was used.

The dynamic input load was calculated according to references.[1][4] The dynamic input load was given as a pressure pulse with time. The summary of the calculations are as follows ;

Detonation pressure = $\sigma_d = \rho \dfrac{V_d^2}{4}$ ,

| $\sigma_d$(Pa) | $\rho$ | $V_d$ | |
|---|---|---|---|
| 1.06E+10 | 1400 | 5500 | (Dynamite) |

$\rho$ = density of explosive

$V_d$ = propagation velocity of explosive

Transmitted stress = $\sigma_h = \sigma_d \left(\dfrac{r_p}{r_r}\right)^{\gamma}$ ,

| $\sigma_d$(Pa) | $\sigma_d$ | $r_p$ | $r_r$ | $\gamma$ |
|---|---|---|---|---|
| 2.91E+08 | 1.06E+10 | 1.01 | 2.55 | 1.3 |

$\sigma_d$ = detonation pressure

$r_p$ = radius of the charge

$r_r$ = hole radius

$\gamma$ = polytropic coefficient

The radius of the charge($r_p$) in the station ,

$V = k\sqrt{\dfrac{Q}{R^{1.5}}}$ , $Q = \left(\dfrac{V}{k}\right)^2 R^{1.5}$

| Q(kg) | V | k | R |
|---|---|---|---|
| 0.905097 | 50 | 250 | 8 |

V = allowed velocity , The volume of the explosive = $Q/\rho$ = 0.905/1400 = 0.65dm³

k = coefficient of the rock , Hole length $\ell$ = 2m

Q = amount of explosive , R = distance

Radius of the charge :

$$Vol = \pi r_p^2 l \Rightarrow r_p = \sqrt{\frac{Vol}{\pi l}} = 1.01cm$$

Transmission force : $F_h = \sigma_h A$

A = area of the hole mantle : $A = 2\pi r_p \ell$ = 0.13m²

$F_h$ = 3.71E+07N

Transmission pressure in the model : $\sigma_{hm} = \dfrac{F_h}{A_{mod}} = 1.16E+0.7$

$A_{mod.}$ = area of the model opening mantle = 3.2m²

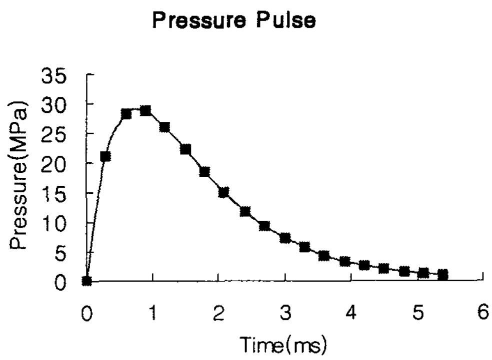
The pressure pulse $\sigma = \sigma_h(e^{-\alpha t} - e^{-\beta t})$

$\sigma_h$ = transmitted stress

t = time

$\alpha$ = coefficient(0.9)

$\beta$ = coefficient(1.8)



Based on the experience on such rock blast only, the cut blast of the pilot tunnel was modelled, since the cut blast are known according to measurements to cause the strongest response on the structures. Fig 5 shows the modelled mesh of the station building and ground.
In Fig 6 to Fig 8, the filled grey shaded plots with the velocity vectors were made to demonstrate the transmission of pressure pulse after some milliseconds. From the figures it can be seen that after 5.8millisecond velocity is transmitted to the top of the bottom slab and after 8.2millisecond velocity is transmitted to the top of the of second floor slab.

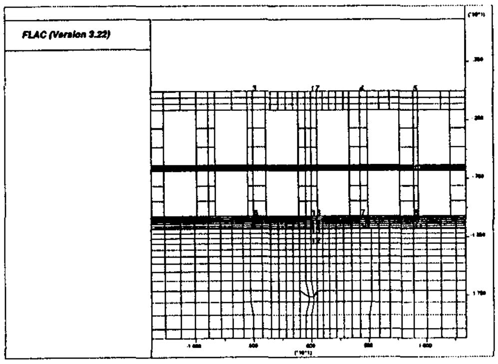



Fig 5. Modelling of the Station and Ground

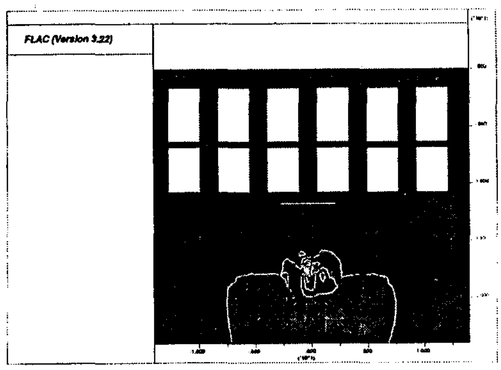



Fig 6. Velocity(after 2.9ms)

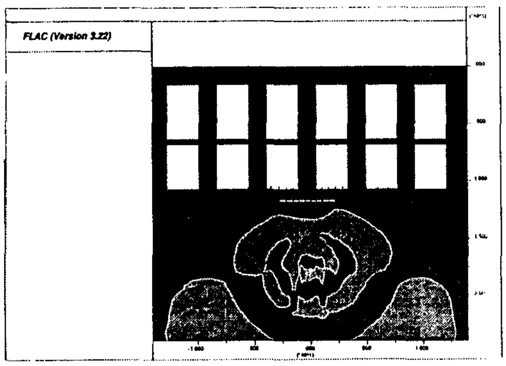



Fig 7. Velocity(after 5.8ms)

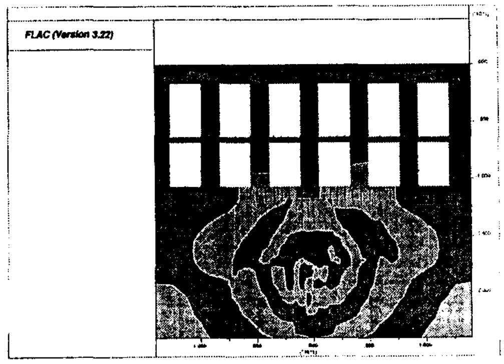



Fig 8. Velocity(after 8.2ms)

The history plots tell the dynamic response on specific points.
Fig 9 and Fig 10 shows the dynamic response history at the point on the bottom and on the top of building respectively.

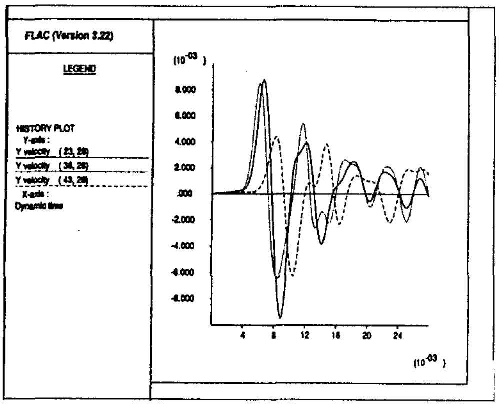


Fig 9. Velocity History at Bottom Floor

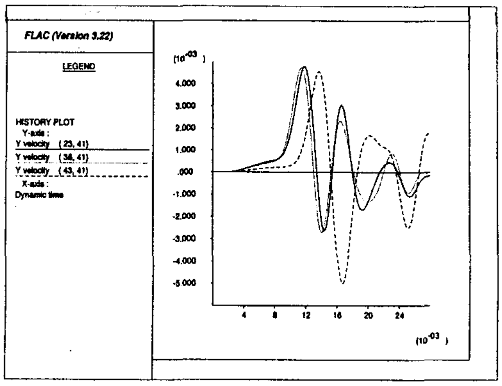


Fig 10. Velocity History at Top Floor

In the analysis the maximum velocity received in history points on the bottom of the building is only about 9㎜/s. The value is considerably lower than the peak velocity(50-70㎜/s) recommended in chapter2.1. The slot drilling under the slab maybe have a decreasing effect on the blast induced response or the rock factor (k) needs to be adjusted.

**Conclusion and recommendations**

The excavation of the tunnel line under the station can be carried out using drill and blast method. Vibrations caused by blasting can be reduced to low level. This was also demonstrated by numerical dynamic analyses.

The blast induced dynamic responses can be demonstrated and analysed with FLAC code. The damping in rock mass (e. g. rock factor k) is one of the most uncertain parameters in analyses and that's why controlled test blasting with velocity measurements would be needed.

Higher peak particle velocities could probably be permitted if risk analysis would be carried out along the line before excavation starts. The practice would include also continuous vibration measurements and installation of shock isolators under sensitive equipment. Higher values would allow longer advance per round and faster and more economical excavation as well.

## REFERENCES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(5) OY FORCM AB. 1990, "FORCIT EXPLOSIVES MANUAL"