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# Blasting as an aid for foundation engineers

## Le dynamitage comme aide pour les ingénieurs de fondation

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**SYNOPSIS:** Civil engineers are often faced with considerable difficiulties when erecting structures or buildings on rock or stony ground, which can only be overcome by resorting to mechanical aids.
Blasting is one of these aids, which is commonly accompanied by the negative effects of vibration, noise and damage to existing structures. The following paper illustrates that through the use of special blasting techniques, with the corresponding detonating devices, it is possible to minimize or even eliminate these negative properties.

It is shown by means of several examples that it is possible to economically erect structures in rock, such as steel pile walls using the RSB technique. This procedure proved to be particularly advantageous if the properties of the subsoil changed and these changes were only apparent during construction.

Accompanying vibration measurements on several structures proved that no damage was caused as long as the RSB technique was correctly applied. The vibration measurements also made it possible to optimize the explosive charge and drill hole spacing so that construction costs and time could be reduced to the absolute minimum.

## INTRODUCTION

The forcible breaking up or destruction of hard, natural or artificial materials, in the building and construction industry, is often achieved by blasting.

This possibility of being able to alter the strength of material is applied in foundation work as an aid in the erection of structures on rock or on rock-like consolidated ground. Explosives become a versatile precision tool in the hands of experienced explosive experts.

From the start, outsiders associate the use of explosives with danger to persons and property, as well as annoyance or even damage. These negative properties can be minimized or even completely eliminated, through the use of new blasting methods and detonators. The following advantages can be obtained by the correct implementation of blasting:

- Building time and the accompanying, unavoidable inconvenience caused can be reduced to a minimum.
- Structure can be erected more economically.
- Considerable costs can be saved, even when building in confined spaces, or on difficult ground.

Advances in the development of new blasting techniques, as well as new detonators and explosives over the past decades, have considerably expanded the areas of application of the mode of blasting.

Blasting has thus become an essential engineering tool for the economical erection of structures with minimal environmental damage, as is shown in the following examples.

## EXAMPLE 1

In the course of the extension of the local public transport system in Frankfurt on Main, the underground railway system was planned to pass under the river Main just south of the Ostendstrasse, at the 36.4 km river marking. The construction of the double track tunnel under the Main had to meet the following requirements:-

- Shipping on the Main could only be interrupted for a short time.
- The original flow cross section had to be retained at all times.

It was thus decided to construct this tunnel in 2 sections, 61.5 and 62.0 m long, in building docks close to the banks. These docks would then be flooded and the tunnel sections would be floated over and sunk into a dredged channel crossing the Main.

It should be noted that it was impossible to construct the tunnel by conventional mining methods, because of the nature of the ground and as there was barely 4 m of overburden in the middle of the river (Gerhard Schmidt 1982, 1984). The following substratum, starting from the riverbed, is present at the river Main crossing.

a. Quaternary fluvial deposits of sand and gravel from lower terraces of the Main on the north bank, as well as remnants of valley clays and flood clays on the south bank.
b. Neocene marine deposits made up of an irregular succession of hard limestones and dolomites, friable algal limestones and limestone sinter, quartz lime sands and gravels, lime clays and marls.
c. Paleocene fresh and brackish water deposits (micaceous sands), made up of massive fine sands that lie beneath the base of the tunnel and are only reached by the sheet piling in the vicinity of the south bank.

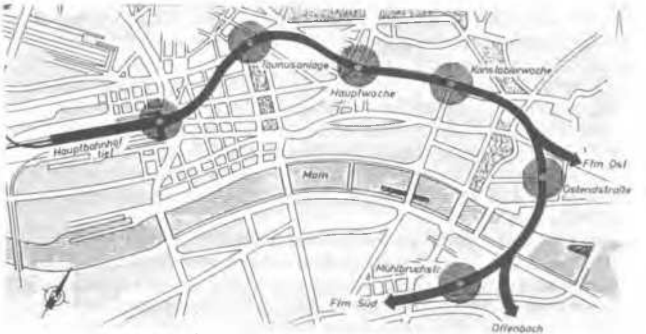

Figure 1: Alignment from underground railway system in Frankfurt/M

The design of the channel was based on model tests carried out at the Darmstadt Technical University. As a result of the tests and economic considerations it was decided to replace the originally planned 17 m deep channel with a channel having an elevation of 82 m above sea level (10 m below normal river level), with the final depth of 17 m below normal river level reached by excavation of a trench between sheet piling at right angles to the flow of the river. For this purpose double piles (Larsen 24) in 14 m lengths were driven into the riverbed.

The boundary conditions for the sheet wall ramming were preset as follows:-

a) Very short pile driving time.
b) Profile section based purely on the requirements of statics and not on the technical specifications of pile driving.
c) Pile driving aids must easily adapt to the changing hardness of the substratum.
d) The degree of restraint of the sheet piling having a length of 14 m must not be reduced by the pile driving aid.
e) Environmental influences caused by the pile driving and from vibration caused by the pile driving aid must be minimized.
f) Tremors affecting neighbouring buildings must be kept within the limiting values set by the DIN 4150 standard.
g) The concreting work being carried out simultaneously in the building docks must not be influenced by the pile driving work or the ramming aids. Possible oscillation speeds arising must remain below v = 20 mm/s.

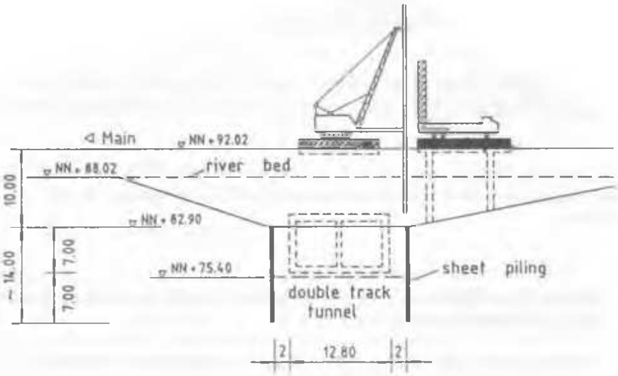

Figure 2: Longitudinal section (from the river Main)

Under these conditions the only ramming aid which can be considered is the RSB procedure (Rosenstock Shock Blasting). The workings of this procedure have been described by J. Dadson 1983, as well as Chr. Eberstadt/B.K. Mayer 1983.

Holes were charged with Super-Cord 40 g/m detonating cord, Ammon-Gelit 3 in cartridge form each 400 gr. and Geosid in cartridge form each 135 gr. (the latter in the areas of harder layers). All subsequent holes weere generally charged according to this procedure. The average charge of the drilled blast holes with a spacing of 1.0 m was 4.6 - 6.2 kg made up of a 12 m long detonating cord and 20 to 25 cartridges.

The pilot tests showed that the chosen method to construct a channel between sheet piling was economical with minimal environmental impact. The following can be concluded:-

- The area of operation in the bed of the Main for the construction of the channel, using the RSB technique, using sheet piling with a length of 14 m could be reduced by half in comparison with the official design using sheet piling having a length of 17 m.
- The excavation volume for the tunnel could be reduced from $V_1$ = 100 800 $m^3$ to $V_2$ = 36 100 $m^3$, which represents a safety of approximately 63%.
- The oscillation speeds of v = 16 mm/s measured in the tunnel building docks lay below the critical value v = 20 mm/s, quoted in the literature, for the strength of concrete.
- In addition to the reduction in building costs for the work on the Main, a considerable reduction in building time was also achieved.

## EXAMPLE 2

A weir was constructed on the river Weser in the suburb of Bremen, Hemelingen, in the years 1906 - 1911, to control the flow of the Weser in the vicinity of Bremen.

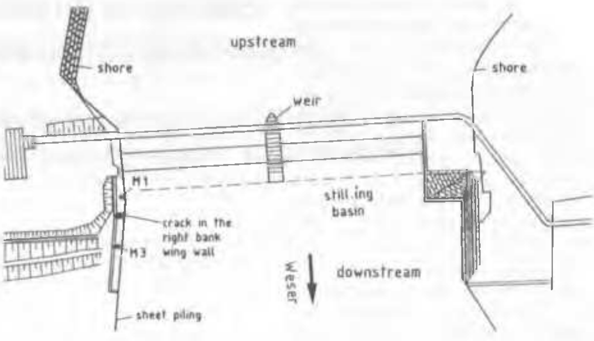

Figure 3: Situation (Weserwehr Hemelingen)

During a routing flushing of the coffer in winter 1980/81, it was noticed that the left sector was damaged and could only partially be used with the help of an emergency lock.

Regulation of the weir was no longer possible at this time. During the March 1981 floods caused by rain and melting snow with flow velocities of up to v = 4 m/s and flow volumes at the weir of Q ≥ 1900 $m^3/s$, forming potholes up to 8 m deep which threatened the stability of the weir, so that a new construction was considered. Until that time the weir had to be maintained and repaired. A site inspection revealed that in the underwater area of the right weir section, potholes and scouring of the right bank wing wall up to a depth of 8 m below the riverbed had been formed, leading to subsidence and tilting of the wall. As a remedy for the potholes formed in the basin, both solid and chemical injection of the wing wall back filling and the substratum, made up of medium grained coarse sand, were initially carried out. The injections with a cement bentonite suspension served as a filling for possible cavities and unconsolidated zones, where as the chemical injections, according to the Monodur process, served to seal the weir against scouring from the bank side.

Following these provisional protective measures the right bank below the water level was reconstructed of partially remodelled, using anchored sheet piling made of Larssen III sections. It was planned to extend this sheet wall up to the werir, in order to replace the massive retaining walls. In order to avoid restricting the form of the pile wall, a 0.6 m slab, above the foundations of the retaining wall, was blasted away using a previously tested process described by W. Rosenstock/H. Schulz 1979.

Piles could not be driven through the base of the wall. It was necessary to make the concrete of the spur amenable to pile driving, by means of blasting according to the RSB technique. The following blasting operations were necessary:

- Splitting off of a concrete slab, 0.6 m thick, 9.5 m high over a length of ca. 12 m.
- Shock blasting in the spur of the embankment over a width of 2.0 m and a length of 15 m.

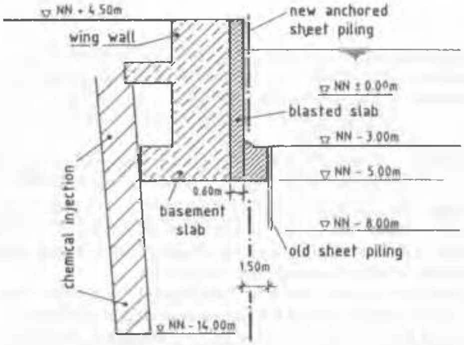

Figure 4: Cross section (right bank wing wall)

Both blasting operations, cleaving off the slab and shock blasting of the spur, were coupled and deployed blast holes as follows:-

Length l = 9.5 m spacing a = 0.8 m diameter D = 89 mm

The holes were charged with detonation cord Nitropenta 100 gr. for the cleaving operation. An additional charge of L = 760 gr. was applied at the foot.

The following vibrations measured in a vertical direction at the measuring point M$_3$ (on a washed away foundation section, see Figure 3), were noted:-

| | | | |
|---|---|---|---|
| Blast 1 | v = 17,3 mm/s | f = 15 Hz | b = 1635 mm/s$^2$ |
| Blast 3 | v = 14.0 mm/s | f = 400 Hz | b = 27468 mm/s$^2$ |
| Blast 4 | v = 15.8 mm/s | f = 276 Hz | b = 27468 mm/s$^2$ |

(v = highest vibration speed, f = frequency, b = acceleration)

Figure 5: Right bank wing wall

At measuring point M$_1$ (a wing wall still connected to the weir), it was possible to take measurements in 3 directions , Z$_1$ = vertical, x$_1$ = horizontal direction, downstream, and y$_1$ = horizontal direction towards the bank.

| | z$_1$ | x$_1$ | y$_1$ |
|---|---|---|---|
| Blast 3 | v = 14 mm/s | v = 24 mm/s | - |
| | f = 267 Hz | f = 182 Hz | - |
| Blast 4 | v = 15 mm/s | v = 82.5 mm/s | - |
| | f = 333 Hz | f = 90 Hz | - |
| Blast 6 | v = 12 mm/s | v = 89 mm/s | v = 36 mm/s |
| | f= 333 Hz | f = 90 Hz | f = 125 Hz |

As there is a crack in the massive wall between measuring points M$_3$ and M$_1$, it was not possible to deduce a trend from the differing distances of these points from the blast point. The high x$_1$ values at the measuring point M$_1$ of v = 82.5 and 89.0 mm/s are worthy of comment. These shocks however, only last for a max of t = 25 ms. The following reliable standard values for the structure present could be established from the above readings:-

z direct. v = 30 mm/s x direct. v = 90 mm/s y direct. v = 36 mm/s

During driving of the double poles it was revealed that the shock components acting in the z direction acted predominantly on the sturcture. High frequency vibrations of f = 160 Hz were measured in the sand area and up to f = 300 Hz in the blasted concrete area. These frequencies fall within the range of structure borne noises, in which vibrations pass through structures without having any influence on them. The pile driving in the vicinity of the blasted concrete spur yielded maximum values of v = 0.43 mm/s for the z direction and maximum values of v = 0.29 mm/s in the area of the outcropping sands downstream. Pile driving could thus be carried out without risk to the structure.

## EXAMPLE 3

In the course of the extension of the docks of the industrial harbour at Kehlheim on the Danube, built in 1977/78, it was planned to construct a sheet pile wall on the east quay to serve as a quay wall. During later extensions it would be possible to remove the sheet pile wall by pulling it out. Both the existing dock and the future docks are located in the valley plain to the south of the Danube between Kehlheim and Saal.

The geology of the substratum was as expected, as shown in the foundation drill hole in Figure 7. The section, seen from top to bottom, consists of a cohesive valley clay layer that is underlain by Quaternary gravels (sand, gravel, partially organic and clay-like). Beneath this, at a depth of 2.5 - 3 m the Kehlheim Limestone is found.

These limestones displayed extreme variations in hardness, joint development and thickness, in both the exploratory drill holes and in the subsequent excavations. In general the upper zone having a thickness of 1.5 m consists predominantly of laminated and foliated marley limestone. This is underlain by dark coloured and gray limestones in varying thicknesses, partially as thin and partially as thick layers. In one section these limestones were missing completely. The joints are mainly filled with calcite. Cavities found are formed by karst processes and are very often filled with soft Quarternary material.

The quay steel pile wall projects 3 m above the pile driving level as shown in Figure 7.

The top of the chosen Larssen III pile had to be at an approximate height of 342 m above sea level, after driving the 10.0 or 10.3 m long piles approximately 7.0 m into the ground.

The previously mentioned RSB technique was selected as a pile driving aid for the limestone layers. The drill holes necessary were initially spaced at a = 0.6 m and subsequently at a = 1.0 m. The holes were charged with 12 gr./m detonation cord Nitropenta and depending on the rock hardness with 7 - 20 cartridges of Seismo Gelit having a 125 gr. charge.

The drill hole length and charge were varied according to the hardness and thickness of the intersected rock layers. This was achieved very simply by continuous observation of the respective penetration speed. It was established that at roller bit penetration speeds of 1 m/min the softer layers were present, while through very hard layers penetration speeds of 0.07 m/min were measured. As cavities and soft joint fillings were occasionally present in the basal zone of the sheet pile wall, it was necessary to drive 5 of the 94 double piles a further 1.5 m deeper.

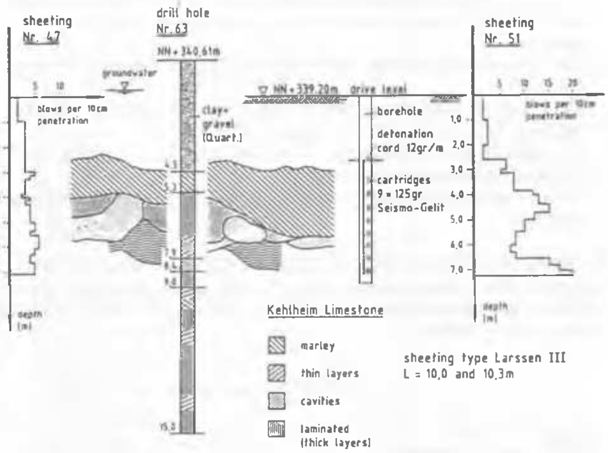

Figure 6: Cross section from subsoil and driving record

The advantages of the RSB technique, such as the flexibility and the adaptability to the changing hardness of the substrata could be fully utilized in the course of construction work, so that the sheet pile quay wall could be put in place without difficulties, in a short space of time and with corresponding low costs.

## EXAMPLE 4

In order to build a flood control reservoir to the north of Fulda, it was necessary to construct a 13.5 m high earth dam, sealed on the inside

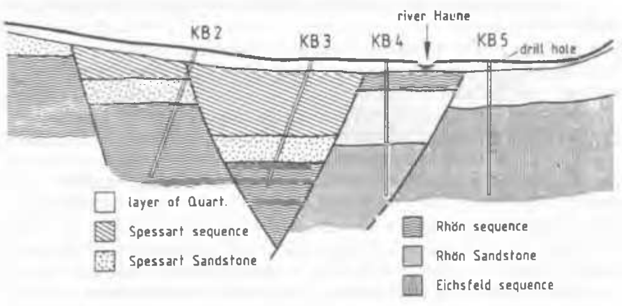

Figure 7: Transverse section (Stratigraphique)

(see Fig. 8 ). The dam site and the basin itself lie 100 m east of the Bebra-Frankfurt railway line, close to the town of Marbach on a tributary of the river Fulda. Geologically speaking, the building site is located in the Fulda Bergland which is predominately made up of rocks of the Bunter Sandstone and the Triassic Muschelkalk. The upper Permian Zechstein formation with clay and dolomite layers and up to 300 m thick Werra rock salt formation are found below. The middle Bunter sandstone is found at the dam site under a 4.0 m thick layer of Quaternary cover. Core drill holes and the attendant drill sections showed that in the area of the dam wall rocks of the Rhön sequence have been faulted against rocks of the Spessart sequence along a WNW-ESE fault (see Fig. 7 .).

The outcropping Bunter sandstone are mainly medium grained, predominantly friable with argillaceous layers.

The WD tests carried out to determine the permeability of the substratum, showed that the Quarternary cover is very permeable and, in particular, water losses in the middle Bunter sandstone formation are very considerable. Consequently, the dam wall site and the dam floor itself would have to be sealed.

To prevent direct under seapage at the base of the dam wall a 0.65 m wide diaphragm wall was constructed using a single component process with a clay and cement suspension, to a depth of 15 m at an altitude of 255.0 m above sea level, according to the following procedure:-

- The slit wall was divided up into sections in the direction of the dam axis.
- The length of the sections was identical with the spacing between the large bore holes with a diameter of 0.65 m.
- The rock bridges between the 14 m deep large bore holes was intended to be removed by ripping.
- A clay-cement suspension was used as the filling and supporting medium.

By means of both the drilling of holes and ripping away of the rock bridges, it should be guaranteed that the rock substratum is disturbed as little as possible and that no additional seapage pathways by means of joints were created.

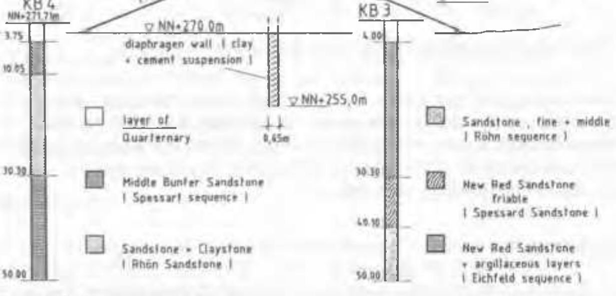

Figure 8: Transverse section

As it turned out that the rock excavation was more difficult than calculated, the contractor suggested loosening the rock in a limited area by controlled blasting. This was accomplished using the RSB technique. For this purpose, blast holes were drilled in the middle of the projected large drill holes. They were cased (diam. = 79 mm) according to the previously mentioned procedure and charged with detonation cord "Supercord 40" (40 gr. of explosive per meter) and additional explosives. A total charge of 1.4 kg per drill hole was distributed over a blasting length of 10 m.

After the drilling of the large drill holes, the rock bridges remaining between the holes were loosened by shock blasts, so that the rock could be excavated without difficulty down to a depth of 15 m using an excavating bucket.

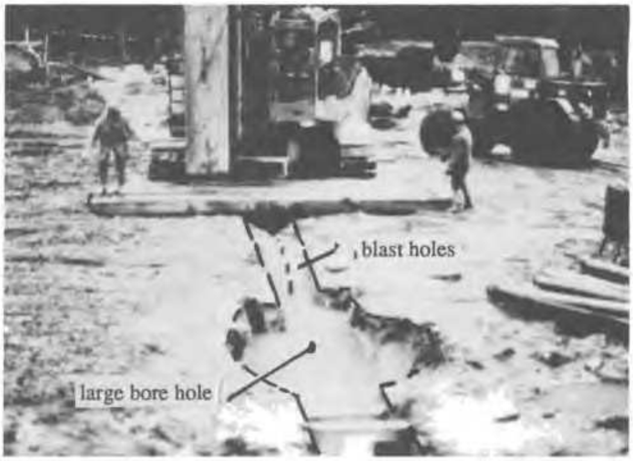

Figure 9: Diaphragm wall with large bore holes

With the aid of the ultrasonic impulse procedure applied by Chr. Eberstadt and B.K. Mayer 1983, the effects of the blasts in both their width and their depth could be checked. The two criteria applied, which were used to measure the change in porosity of the blasted rock are as follows:-

- The decrease in the velocity of sound in solid matter with direct irradiation of ultrasonic waves.
- The attenuation expressed as the relationship of the amplitude of an oscillation to the amplitude of the following oscillation.

The measurements showed the following:-

- Width of zone disturbed by the blasting was b = 0.5 m on both sides of the axis, giving a total width B = 2 x b = 1.0 m.
- Depth of the disturbed zone T = Z + 0.5 m Z = loading depth

The blasting continuously carried out before the excavation enabled the output of the excavation equipment to be more than doubled. The drilling capacity of the large drill holes (d = 0.65 m) could be increased from 2.0 - 2.5 m/hour without blasting to 8 m/hour after blasting.

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