3. Demolition

Demolition Risk Management - Blasting of two high-rise buildings in Copenhagen, Denmark

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ABSTRACT

Risk Management proved to be a valuable approach for systematic analyses, assessment and control of risk related to demolition blasting of two high-rise buildings in the Copenhagen suburb Rødovre in 2012.

The paper presents part of the demolition method, risk assessment, requirements for the demolition work, and the monitoring and evaluation program.

The paper explains how the choice of demolition by blasting was based on a systematic analyses of three promising demolition methods; long range concrete crusher, deconstruction from the top by crane, and demolitionby blasting.

The analyses were carried out with respect to a number of carefully selected criteria covering occupational safety, third party damages, environmental impact, time and budget.

In the planning phase, the initial likelihood and consequences of hazards related to the chosen method of demolition by blasting were assessed using the semi-quantitative risk assessment method. Furthermore, mitigation measures were identified to control risks which initially were found unwanted or unacceptably high. By incorporation these mitigation measures, it was in the end possible to achieve an acceptable level of risk.

Findings of the risk assessment resulted in the definition of a set of requirements for the work carried out by the demolition contractor.

Throughout the project, a comprehensive monitoring program was conducted, which made it possible to document and evaluate the environmental impact caused by demolition by blasting.

1. INTRODUCTION

The two concrete buildings, 13and 15stories, 14,100 t and 16,500 t, were constructed in 1954 - 1956. Because of outdated design and high cost of renovation, the owner, AAB co-operative housing society, decided in 2006 to demolish the buildings and hired NIRAS for planning and management of the demolition work. The two buildings were sited among spread outlowresidential housing , a kindergarten/nursery, an after school center, main traffic and public transport routs.

After an initial selection the options for demolition came down to three possible methods, namely;
long range concrete crusher, deconstruction from the top lifting elements by crane and demolition by blasting.

Fig 1 The 13 story Ruskær Building. Picture taken from South East.

The three methods were evaluated with respect to occupational safety, third party damages, environmental impact, time and budget. When assessing the risk,the criteria for critical events weredefined as unwanted/unintended events which consequence would lead to damage on workers, third-party and third-party property.

On 13th May 2012 two tower blocks in Copenhagen were demolished byblasting,the demolition was the biggest building blasting job in Scandinavia until now.

2. ASSESSMENT OF SAFETY RISK, ENVIRONMENT PROTECTION, TIME AND COST

2.1 Safety risk

The risk wasassessed as the combined result of the likelihood of hazardous occurrence, the subsequent likelihood of occurrence of an accident , and the severity of consequence.
Evaluation of the risk of relevant critical occurrences for the three methods wasdone by the use of a semi quantitativemodel. Which includes the risk matrix containing the three elements; hazard occurrence, severity of consequences and level of risk, helping in classification of risks.
The methods have been evaluated by initial and residual risk according to the ALARP principle (as low as reasonably practicable). For a risk to be ALARP it shouldbe possible to demonstrate that the cost involved in reducing the risk further would be grossly disproportionate to the benefit gained.
The likelihood of hazard occurrence for a given critical event wascategorized by five levels, see table 1.
Table 1.Likelihood hazardoccurrence

Hazard occurrence	
Scale	**Level**
1	Improbable
2	Occasional
3	Probable
4	Frequent
5	Very frequent

The likelihood of hazards occurrence were held against the severity of the consequence of the accident, again in a scale of five levels, see table 2.

Table 2. Severity of Consequence

Severity of Consequence				
Scale	**Level**	**Occupational safety**	**Safety, third-party persons**	**Safety, third-party property**
A	Negligible	Short discomfort, no need for treatment	None	No / insignificant damage, no repair
B	Marginal	0-1 sick days	Short discomfort, no need for need for treatment	Cosmetic damage, easy to repair, no consequence for the function of the building
C	Serious	> 1 sick days	0-1 sick day	Moderate damages, repair possible, no or limited consequences for the function of the building
D	Critical	Permanent injuries	> 1 sick days	Comprehensive damages, repair possible, temporary consequence for the function of the building (e.g. habitation impossible during repair, rerouting needed during repair of road, loss of supply network)
E	Catastrophic	Death	Permanent injuries/ Death	Comprehensive damages, impossible to repair or/and which has consequence for the surroundings (Building completely damaged, lasting loss of supply network)

With the two components a given event in the three methods can be classified in the risk matrix, were the shade of gray, from dark to light, indicates whether the level of risk is; Not acceptable, undesirable, tolerable or acceptable, see table 3.

Table 3. Risk matrix

	Risk Level	**Likelihood of Hazard occurrence**				
		Improbable (1)	**Occasional (2)**	**Probable (3)**	**Frequent (4)**	**Very frequent (5)**
Severity of Consequence	**Negligible (A)**	1A	2A	3A	4A	5A
	Marginal (B)	1B	2B	3B	4B	5B
	Serious (C)	1C	2C	3C	4C	5C
	Critical (D)	1D	2D	3D	4D	5D
	Catastrophic (E)	1E	2E	3E	4E	5E

Based on the on the evaluation of possible scenarios some major risks were found to be linkedand grouped. This was used to compare the relative risks in the three different evaluated methods. The linked groupsare:

I. Risk of unwanted/unintended collapse and related damage to; occupation, third-party and third-party property.

II. Other major risks (as a result of other scenarios than the unwanted/unintended collapse)

II.1 Risk of occupational safety

II.2 Risk of damages on third-party

II.3 Risk of damages on third-party property

In table 4. the relative risk for the group I and II are listed based on the analysescompletedfor the possible scenarios.

Table 4 Relative Risk

Risk		**RELATIVE RISK** *X: lowest/best, XX: neutral, XXX: highest/worst*		
		Deconstruction	**Long range crane**	**Blasting**
I. Unwanted/Unintended Collapse		XX	XXX	X
II. Other Major Risks	**II.1. Occupational safety**	XXX	XX	X
	II.2. Damage on third-party	X	XX	XX
	II.3. Damage on third-party property	X	XX	XXX

Based on the risk assessment it was evaluated that the risk for a unwanted/unintended collapse was greatest during cutting with a long range crane crushing the concrete. The primary reason for this is due to the buildings relatively slender and weak bearings and slabs which had a very low bearing capacity. Therefore the slabs are very vulnerable to the falling objects during concrete crushing with the long range crane. Hereof follows the increased risk of an unwanted / unintended collapse of the building. The identified risk was present during the full period of working with the long range crane.

Focusing on other major risks disregarding the risk of unwanted/ unintended collapses; it was found that:

· The risk related tooccupational safety, were assessedto be greatest during deconstruction. This due to the complexity of the work operations and the many and long operations with the crane. On the other hand the method was the least risky in regards to damage on third-party and their property.

· The risk related to damages on third-party was assessed to be of the same level for long range crane and demolition by blasting. In both methods this in relation to risk of getting hit by flying rock outside the safety zone. The crucial difference between the two is the muchlonger time of exposure for the long crane method.

· Risk related to damages on third-party property wasassessed to be greatest during demolition by blasting. This primary due to fly rock and subsurface vibrations.

2.2 Environment protection

The discomfort experienced by third-party individual depends on the intensity and the duration of the exposure. It is different from person to person how the combination of the two factors effects the individual. Both intensive exposure in short time and moderate exposure over long time can lead to discomfort.

Leaving out the intermediate evaluations in this article; the final evaluation of the possible discomfortswere compared relativelyfor the three different methods. The overall picture is summarized in table 5.

Table 5 Relative discomfort

Discomfort	**RELATIVE DISCOMFORT** *X: lowest/best, XX: neutral, XXX: highest/worst*		
	Deconstruction	**Long range crane**	**Blasting**
Noise	XXX	XX	X
Dust	X	XXX (Low intensity over longer time)	XXX (Intensive over short time)
Vibrations	X	XXX	XX
Access	XXX (Longest limitation of time)	XX (Limitation over long period)	XX (Extensive limitations over short period)

2.3 Time and cost

Based on knowledge and experience on the three methods they individually were assessed and evaluated in regards to time and cost.

Leaving out the intermediate evaluations in this article; the individual cost and time were ranked along with an estimate on risk for time delay and overspending. The ranking is summarized in table 6.

Table 6 Relative time and cost

Time/Cost	RELATIVE TIME/COST *X: lowest/best, XX: neutral, XXX: highest/worst*		
	Deconstruction	**Long range crane**	**Blasting**
Time	XXX	XX	X
Risk of delay	XXX	XX	X
Price	XXX	XX	X
Risk of overspending	XXX	XX	X

2.4 Ranking of the methods based on the assessment

The completed assessment of safety risk, environmental protection, time and cost for the three demolition methods; long range concrete crusher, deconstruction from the top by crane, and demolitionby blasting was in a ranked based on the summarizing as visualized in table 4, 5 and 6.
Herby it was found that most favorable of the three methods was the demolition by blasting.

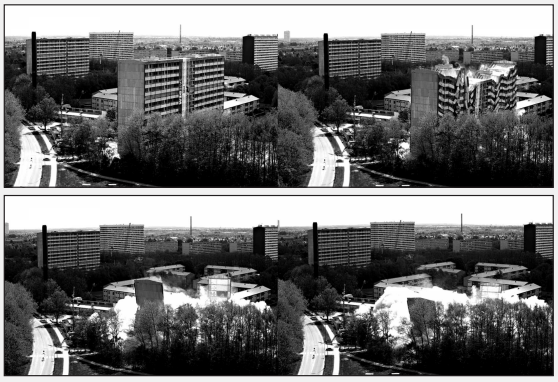

Fig 2. Blasting the first tower block.

3. RISK MANAGEMENT

Based on the risk assessment several possible hazardsoccurrences were found and linked mitigation measures were established to minimize the risk according to the ALARP principle.
In the following the possible hazardousoccurrences and mitigation measures for demolition by blasting, are listed in table 7. All the found mitigations measures for demolition by blasting was implemented in to the project.
Table 7. Scenarios and mitigation measures.

Event	Possible occurrences	Possible causes	Mitigation measures
Unwanted/ Unintended collapse	The Building falls in an unintended direction. The Building "kicks back".	Error in blast design. Error in execution of drilling/charging.	Increased control of the contractors blast design and check of collapse mechanism. Increased control of contractor during drilling and charging. Support mat over roads and underground supply network. Use of double firing system. Setup and sustainment of safety zone in accordance with the assessed scenario.
House end falls out	The house end falls outwards in its full length.	Missing anchoring/attachment.	Anchoring /attaching of the house endse.g. on elevator shafts. Blasting of house wall on several floors/levels if necessary. Setup and sustainment of safety zone in accordance with the assessed scenario. The ban on the safety zone is only lifted when the blast managerhave checked that the blasting was successful and correct. This including the correct fall direction of the house end.
Building only falls partly	Part of the building is still standing after blasting.	Error in the drilling/charging/ ignition.Construction stronger than expected.	Increasedcontrol of the contractorsblast design and check of collapsemechanism. Increased control of contractor during drilling and charging. Use of double firing system. The ban on the area is only lifted when the blast manager have checked that the blasting was successful and correct.
Unauthorized persons	Intrusion by the audience	Curiosity. Unauthorized demonstration.	Guards in the buildingduringcharging. Information to the public. Control of unauthorized persons in the restricted zone. The ban on the area is only lifted when the blast manager have checked that the blasting was successful and correct.
Flying debris	Damage due to falling/flying/rebounding debris.	Inefficient covering of objects or/and securing of third-party property.Insufficient safety zone.	Covering of the building subject to blasting by the use of heavy and light blast mats. Increased control prior to blasting. Securing of exposed buildings with plywood plates, net and old containers. Rearranging of underground supply network. Support mat over roads and underground supply network. Picture registration of the area and its buildings prior to demolition. Setup and sustainment of safety zone. Setup of emergencyteam ready to deal with damage on infrastructure etc.
Subsurface vibrations	Vibrations originating from the building impact on the ground.		Support mats in the area of impact. Temporary closing of local supply stations, if this is found reasonable. Picture registration of the area and its buildings prior to demolition. Implementation of vibrations measurements for documentation of impact. Setup of emergency team ready to deal with damage on infrastructure etc.
Dust is spread to the surroundings	Cloud of dust upon collapse of the building.		Covering of the building subject to blasting by the use of heavy and light blast mats. Removal of sand in floor elements and general vacuum cleaning of buildings prior to blast. Watering of building and establishing of water cloud prior to blast. Sealing of doors, windows and ventilations. Securing of exposed buildings with plywood plates, net and old containers, if suitable. Inspection and dust protection of sensitive filters and installations, if applicable. Setup of team ready to deal with dust removal if ready. Blasting is postponed if wind is adverse.
Air blast	Short dynamic air blast		Setup and sustainment of safety zone . Nearest windows are covered against flying debris and air blast.
Misfire. Explosives and detonators in the debris.	Unintended blasting of misfires during clean-up.	Ignition error.	Increased control of the contractors blast design. Increased control of contractor during drilling and charging. Use of double firing system. Setup and sustainment of safety zone.Video control of selected charges. The ban on the safety zone is only lifted when theblast manager have checked that the blasting was successful and correct. Setup of fence around work area in protection of third-party. Thorough control of debris during clean-up.
Accidents during preparation for explosion.	Unintended explosion during handling of explosives etc.	Maloperationfire electromagnetic impulse. Mechanical influences of explosives etc. Drilling in charged holes.	Increased control of blasting contractors education and performance. Increased control of storage of explosives etc. Control of the contractors QA and QC. Use of EXEL detonators or similar.

4. OUTCOME

After evacuation of dwellersliving in the neighboringbuildings and establishment of the safety zone the trains and roads were stopped. Just after 12 hour theblow down of the first tower block took place successfully.
According to the contract with the insurance company the results of the vibration recordings and eventual damages on buildings should be checked before blasting of the second tower block. The recorded vibrations were lower than 20 mm/s (a threshold value of 20 mm/s was set for the vibration velocity, this in accordance with the German standard DIN 4150-3) and no damages were observed. The next blasting followed shortly after and was also successful.

Fig 3. Blasting of the second tower block

The total amount of explosives provided by Orica Mining Services A/S Denmark:

- 3500 m detonation cord
- 60 kg explosives, 30 kg per building
- 14000 m EXEL shock tubes, 7000 per building
- 1600 charges, 800 per building

Making status of the project after the demolition of the two towers it is listed that:

- The maximum recorded vibration velocity was 10.20 mm/s in a distance of 12 m and 4.55 mm/s in a distance of 22 m.
- Airblast/noise and dust was kept to a minimum level, below the predicted levels.
- No damage was reported or registered after the following inspection .
- No complaints were given by neighbours or other individuals or organizations.

By assessing the initial selected methods on safety risk, environment protection, time and cost of the it was possible to rank and select the best method for the demolition. Furthermore the process also includeden evaluation of hazardous scenarios and a thoroughly consideration of possible linked mitigation measures for the hazardous occurrences.
Risk assessment and management was clearly the right tool in selection of the best demolition method. At the same time it enabled the project management to identify and foresee hazardous occurrences and with the mitigation measures address and prevent the occurrences.

5. REFERENCES

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Explosives demolition of gasholder in Copenhagen, Denmark

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ABSTRACT: Supplying gastothe city of Copenhagen, Denmark, for domestic use and for heating, the gasholder of Køgevejen was situated in the Copenhagen suburb of Valby.The second highest building in Denmark, the gasholder was a significant a landmark of the Copenhagen skyline. The gasholder was commissioned in 1967 and was demolished in 2012.

This paper will summarize the demolition from its planning to site preparation, preweakening of the structure, charging of explosives, felling, and further on to the shearing of the steel and the finishing ground works. Also, brief summaries of the involved parties, both of the demolition contractor, the explosives expert and the engineering consultants, will be included.

As detailed descriptions of explosives demolitions will be supplied by other parties, this paper's main focus will be the planning and the preparatory works, leading to the successful demolition of the Copenhagen gasholder.

1. THE GASHOLDER

The gasholder, rising to some 108 meters and with a diameter of 54 meters, was of the waterless type. A permanent steel construction of plates, mounted between 24 vertical legs, originally contained a piston, rising and lowering inside the gasholder corresponding with gas stock. The weight of the piston produced the supply pressure for the gas to the consumer appliances. The gasholder covered an area of approximately 2,000 m2.

Heating of housings in Copenhagen over the years shifted to other sources of energy, decreasing the demand for gas. Since the type of gas supplied had also changed, the gasholder became redundant and consequently was taken out of use in 2007. As no other use for the construction was found relevant and profitable, the gasholder became the subject of demolition.

The piston having been removed earlier, the weight of the gasholder amounted to some 1,800 tonnes.Along with the removal of the piston, approximately 90 m3 sealing oil and 100 m3 of tar was cleaned out of the gasholder, and disposed of.

Fig. 1. The Copenhagen gasholder on the day of demolition

2. OBJECTIVE

The gasholder had to be prepared for demolition, felled, its steel contents cut up for recycling, the grounds cleaned of any accumulated pollution, and the area to be prepared for subsequent building or other, non-industrial use.The demolition and its contained processes were to consider occupational health issues as well as environmental issues, and neighbouring buildings and recreational areas were to be left unharmed.

To avoid the spreading of substances and chemicals, unwanted in the environment, constructions in Denmark are to be cleaned before demolished. In order to do so, thorough samplings and investigations are to establish their presences prior to the demolition.

Such substances could be lead and PCB, contained in paint, or the oil and tar used for the sealing of the gasholder. Hence, and important part of the turnover of Danish demolition contractors is created in environmentally safe cleaning jobs. Furthermore, as the gasholder was to be felled on to a football court, particles of glass and metal debris were highly unwanted. All these matters were to be taken into consideration when performing the demolition.

3. THE COMPANIES INVOLVED

The demolition of the Copenhagen gasholder was undertaken by the Danish demolition contractor G. Tscherning A/S as main contractor, and was carried out as an explosives demolition.

G. Tscherning A/S was founded in 1975 by Georg Tscherning. Providing services within demolition, general contracting and sewerage works, the company has managed some of the largest and most spectacular demolitions in Denmark, including the decommissioning of two nuclear test reactors. G. Tscherning A/S currently has some 200 employees, and is internationally certified according to ISO 9001, ISO 14001 and OHSAS 1800. Responsible for the gasholder demolition was Mr. Peter Hansen and Mr. Klaus Bodilsen, assisted by Mr. Tue Brix.

Calculations and the design of the explosives demolition, as well as the charging of explosives and the actual felling, was carried out by Precision Demolition Company Ltd. (PDC) of South Yorkshire, England. PDC was founded in 1994 by John M. Faulkner, previouslyan explosives engineer employed with other companies. With some 40 years of experience, John. M. Faulkner has felled some 1,7 million tonnes of structures, including high rise tower blocks, power plants, industrial chimneys and bridges. Work on the Danish site was carried out by Mr.John M. Faulkner and Mr.Robyn Rushford.

As advisory engineer, Niras Demex A/S of Allerød, Denmark, was chosen. Founded in 1978 by Erik K. Lauritzen, Niras Demex A/S has been involved in a wide range of demolitions with an international field of operation.Having been employed by private contractors, the Danish Foreign Ministry, United Nations and the European Commission, among others, Niras Demex is a market leader in Denmark regarding explosives demolitions advisory. Participating in the gasholder demolition was Mr. Erik K. Lauritzen and Mr. Johan Finsteen Gjødvad.

4. DEMOLITION PLANNING

4.1 Tendering and contracting

Demolition of the gasholder came out for tendering in May/June 2012. In order to forward a precise offer, the needed work and the relevant information had to be achieved and calculated already as part of the tendering process.

The tasks to be undertaken were methodically agreed upon and split between the parties. Exact borderlines were established, detailed listings of jobs were prepared, and calculations for the tender carried out. The calculations by the main contractor were done for the stages of ground preparation, shearing and handling of the steel, removal of polluted soil, and finishing works. The explosives charging, and the felling, was calculated by the explosives subcontractor, and costs for risk assessment, authorities handling and reviewing of felling calculations calculated by the advisor.

Several other companies tendered for the job, but after careful scrutinizing of the offers, regarding price, offered methods, and handling of environmental and occupational health issues, demolition of the gasholder was handed to G. Tscherning A/S. Detailed planning began, based on the already established facts and contained processes, and preparation of the gasholder was commenced.

4.2 Planning, instructions and communication

In order to avoid the wet and windy winter season, and in accordance with the wishes of the gasholder owner, demolition was planned to take place in the late autumn. To minimize the disturbance of car traffic, passing on the near-by roads, and to trains of the train line just outside the exclusion zone of the demolition, a week-end day was to be preferred, and consequently the demolition date of Sunday, the 4. November 2012, was set.

The gasholder being a well-known structure in Copenhagen, and as the visual impact of the gasholder was substantial, a large amount of spectators to the demolition could be anticipated. Rather than closing off a large area, thus avoiding larger crowds, it was decided to establish organized facilities for the inhabitants of the region to witness the important event.

In accordance with Danish legislation, detailed written risk assessments for all included processes, containing measurements to avoid the discovered risks and obstacles, were prepared.

For the traditional works, riskssuch as employees falling from heights, being run into by heavy machinery, being exposed to toxic fumes when flame cutting the steel and exposed to chemical substances when working in the gasholder or when removing polluted soil, were duly noted. To these were added the risks for employees regarding the handling of the explosives and working with the pre weakening, also during the explosives-charged state.

Risks to the environment and surroundings, also including risks to the anticipated spectators and to neighbouring facilities including residents, were laid out, and careful steps to avoid all risks were planned.

Planning of site lay-out, work flow, time schedule, manning, machinery and resources was carried out, and all anticipated activities and needs were recorded. The agreed outcome was communicated to relevant employees, and instructions carried out accordingly.

Fig. 2. Site instructions

A series of meetings between both the involved contractors, and between the contractors and the authorities, were held in order to achieve precise planning and clearly defined tasks and duties.

Regarding traffic and infrastructure, results of the planning was communicated to train authorities, Copenhagen Airport, and to the local police. It was decided to block off one of the main roads, passing the demolition area, and to stop train traffic for a shorter period adjacent to the demolition.An exclusion zone and methods for evacuation of residents and public was planned, a number of sentries were assigned, and fixed guard post were recorded and communicated.

To facilitate spectator handling, police and volunteers from the Danish civil Home Hard were to be present at specific post on the demolition day. Police was responsible for the evacuation of residents and would also supply the needed forces to carryout traffic regulation.

Project information was authored and communicated to relevant parties by the means of posters, personal contact to board members of resident's associations, and contact to the press. Neighbours handling and information to the public was a crucial part of the planning and preparation, and both the main contractor and the gasholder owner allocated resources to inform the public and to make sure authorities handling was well under way.

All discovered risks and their handling, the structural and engineering calculations, the explosives placement (charging) of the structure, and the planning and means to perform the demolition, were taking in to an overall risk assessment by the advisor, and the outcome acted upon by theparties.

5. SITE PREPARATION AND PRE WEAKINING

Facilities for the personnel were established on site, and machinery taken in. The decided exclusion zone was established and fenced, and personnel instructed.The ground was prepared by the cutting of trees, and sand and iron plates were spread out to prevent damage from the heavy machinerytraffic.

In accordance with the calculations, the structure was prepared by the removal of all items, having an impact on the stability and foreseen behaviour of the gasholder or presenting a risk of polluting the ground or the surroundings. The steel plates of the structure were precisely cut, in order to obtain a wedge (mouth) providing the fall and the fall direction, and a back cut (heel) enabling the cylinder to rotate around the remaining side legs (hinge), thus coming to a controlled fall.

Fig. 3. Pre weakening of structure

6. CHARGING AND FELLING

In order to make sure of correct calculations and adequate explosives, a test blast was performed in the beginning of October 2012. One of the front legs (in the wedge) were chosen, and a combination of cutting charges and a kicking charge liberated a section of the leg and supplied the force needed to remove it from the structure. After an apparently successful test blast, the edges of the cuttings was thoroughly examined, and with the good result of the test,the calculations and the methods were found satisfactory.

Fig. 4. Preparing the test blasting

Explosives, amounting to a total of less than 50 Kg., were placed around the legs in the fall direction, following the mouth (wedge) shape, according to the calculations.Just prior to the actual demolition, detonators were programmed and placed. From the beginning of the charging, the site was guarded constantly until the area was reopened after careful examination by the blast leader.

Fig. 5. Preparing charges

The explosives demolition was initiated and carried out at the time planned, and in the presence of approximately 10,000 spectators. TV and newspapers were present, and the event was also recorded by many spectators and published in electronic, social media.

Fig. 6. The explosives demolition

7. SHEARING AND FINSHING GROUND WORKS

Immediately following the successful demolition and the reopening of the exclusion zone and the traffic, the fence was moved to establish the site for the finishing works.

Once felled and collapsed, the metal of the gasholder was cut into sections by flame cutting, being moved to a working area where the sections were rolled up and then cut with machine-mounted shearers to manoeuvrable sizes, decided by the steel works to receive the metal for recycling. The metal was loaded into containers and taken away every day, minimizing the daily stock on site and making it possible to decrease the needed area weekly in order to quickly re-establish the grounds and the plantations.

After the finished shredding, the remaining concrete slap and the foundations were removed, using hydraulic hammers, mounted on heavy machinery. The concrete was also taken to recycling, except some parts having been exposed to the oil or tar, sealing the gasholder cylinder.

Fig. 7. Shearing of metal

Once the concrete was removed, a series of ground samples were taken and analyzed, and the pollution levels concluded. Accordingly, some of the sand underneath the gasholder was taken to special treatment, and the main part used to fill up the ground on site. A layer of soil was laid out and trees replanted in the original areas.

8. CONCLUSIONS

The thorough fact finding, discovering of processes, and analyses, as well as the use of carefully instructed, skilled personnel,let to a well done demolition, free of mishaps and unwanted incidents. As each step was closely monitored, and its successful completion confirmed before beginning the step to follow, the demolition project could be carried out according to the plans and to thetime schedule.

The close cooperation between the contractors, and the early involvement of the authorities, established a good working environment with simple and effective lines of communications,ensuring an aware organisation and crew, and facilitating stress free decision making. Clear communicationto neighbours and to the public enabled the handling of the large number of spectators.

During the works, there were no occupational health accidents, all machinery was left without damages, and the area handed back to the gasholder owner for future use.

Fundamentals of Explosive Demolition

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ABSTRACT: Explosive demolition is a complex process that carry's high risks if it is not carried out in a very controlled manner. The purpose of this paper is to inform the reader of the basic approach to explosive demolitions, the materials used and to provide an insight to the potential problems that can be encountered if poor choices are made.

The topics included are pro's and con's of different types of initiation system including the necessity for backup systems. The preferred types of explosives and the alternatives that can be used in the event of unavailability of the first choice. Also discussed are the hazards encountered through the incorrect choice of explosives.

It is not possible within this paper to discuss all types of structures to be demolished but the latest methods that are used on common structures are to be discussed along with the consequences of not adopting best practice.

1. INTRODUCTION.

1.1 Explosive demolition has been carried out for many years in a traditional way. A lot of the methods used have remained largely unchanged as once a system has been demonstrated as being effective there seems to be a reluctance both within the industry and also by clients to accept the need for change. This has led to a situation where some explosive engineers have used methods 'because that is the way it has always been done' without understanding how that method works.

1.2 The problem the explosive demolition industry is now facing is that construction techniques have evolved that produce structures that upon initial inspection are similar to many others but in fact have subtle differences that mean that they do not behave in quite the same manner when demolished.

1.3 There have been incidents where structures have failed to collapse or have not collapsed in the expected manner and it is from studying these that we realized that each project must be taken as unique and a demolition mechanism that is fully understood developed in each case.

2. HOW DOES IT GO WRONG

2.1 There are many possible answers to this but in order to understand the problem properly we need to ask ourselves three further questions;

2.1.1 Did all of the explosives fire as intended?This leads us to review our initiation system.

2.1.2 Did all explosives do what was intended?This makes us review the type and size of charges.
2.1.3 Did the structure behave as expected?This causes us to consider the structural engineering.
2.2 Thus if we address these three points we can design a successful demolition.

3. INITIATION SYSTEMS
3.1 We basically have three types of initiation system available to us, these being non electric, electric and electronic. As may be expected each of these systems has benefits and weaknesses.

3.1.1. There are various different Non electric 'shock tube' systems on the market around the world but for the purpose of this paper we will not be looking at any individual system. Shock tube systems have the advantage of being quick to install, immune to radio frequency and easily available. The draw backs are that the range of delay intervals is restricted to standard times, complex systems cannot be easily checked other than visually and that the accuracy of the timings is not good, particularly when components have been in store for some time. This can be a major consideration in a complex system where there may be a large number of delays all being initiated by the previous device as the errors become compounded possibly to the point where the error in timing can be greater than the intended delay increment. The use of shock tube systems is therefore limited to simple circuits where the timing of the shots is not critical and the circuit can be readily inspected.

3.1.2 Electric systems again are available around the world from many manufacturers. There are variations from one supplier to the other regarding firing currents and delay intervals etc. The major drawback with electric detonators is their risk of premature initiation from either radio frequency transmissions, magnetic fields from high voltage cables, or from thunder storms. The range of delays that can be used is also restricted to those available from the supplier as it is not possible to form chains of delay with one detonator being initiated from the previous one in the chain. The main advantage of electric detonators is that they are cheap, that the timings are far more accurate than shock tube and that the integrity of the circuit can be tested from the firing point immediately prior to firing.

3.1.3 In recent years electronic detonators have become more readily available and new systems are being constantly developed. The draw backs of electronic detonating systems are that in addition to the higher cost of each detonator each system requires system specific ancillary equipment to program and fire the

detonators with the resultant high costs of capital purchase and training required. The benefits are that accurate timings are achieved over relatively long periods of time, the system can be checked and faults diagnosed from the firing point, the systems aresecure and immune to accidental initiation. Electronic systems are particularly useful when demolishing high rise multi story buildings as they allow control of the debris pile far in excess of any other method.

3.2 As can be seen each system has its own merits and draw backs and it is for this reason that in complex projects we often use hybrid systems comprising of electronics and non -electric.

3.3 A common problem in demolition where non-electric detonators are used to initiate multiple charges is that of leads being cut by flying debris before the leads have served their intended purpose. The way this problem can be avoided is for a sufficiently long delay period detonator to be used in each charged hole such that all of the shock tube and delay clips have fired before any demolition charges are fired. Fragments from delay clips can also cause cut leads so care must be taken to ensure that clips are positioned correctly and are then covered with small sand bags or other suitable means of preventing fragments from being liberated when they are fired.

3.4 Due to the inability of a non- electric detonation system to be checked for integrity from the firing point it is good practice for a backup system to be installed in parallel to the main firing system but with a small time offset.

4. TYPES OF EXPLOSIVE AND CHARGE DESIGN.

4.1 The type of explosive used and the charge design are dependent upon the target material against which they are to be used and the way in which the target has been prepared.

4.2 To be effective and to keep air over pressure to a minimum as is normally required the explosives used for concrete must be placed in shot holes drilled in to the target wall or column.

4.2.1 The geometry of the target will to some extent dictate which direction the shot holes can be drilled. When practical it is preferable to drill a concrete target in line with the long axis as this will give a more evenly distributed charge loading than a number of short holes drilled in line with the short axis. To demonstrate this lets consider a 150mm thick wall 2m long. If we drilled shot holes from the face in to the wall, even drilling down at a 45º angle the holes would be drilled 115mm deep to accommodate a nitroglycerine charge and allow for stemming. To remove a band of wall 600 high we would require three rows of holes with 5 or 6 holes in each row giving 17 charges to initiate and if the hole spacing is not optimized could result in holes being blasted in the wall with areas of concrete remaining intact and thus able to carry load and prevent the required collapse. If we now consider the same 2m long 150mm thick wall drilled along the long axis we would need to drill only 2 holes spaced 450mm apart. These holes would then be charged with PETN detonating cord. The cord would be initiated with only two detonators and due to the even distribution of the explosive along the full length of the wall would remove whole wall section without risk of any sections remaining able to carry load.

4.2.2 Care must be taken in the selection of the explosive used for concrete. As velocity of detonation is reduced the risk of throw of large pieces of unbroken concrete is greatly increased. The minimum velocity of detonation for explosive to be used on concrete demolition should be considered to be in the order of 4000meter/second.

4.3 Explosives used against steel can be used in two basic ways. Linear shaped cutting charges can be used to cut steel where pre-cutting cannot be undertaken on structural grounds and kicking charges can be used to displace legs that have been pre-cut during the preparation stages of the work.

4.3.1 Linear shaped cutting charges can be purchased either prefilled or for filling by the user at the point of use. Pre filled charges can be either rigid metal cased of flexible. Rigid charges are good for flat targets but some types can be difficult if not impossible to safely cut to size on site. Some prefilled charges are flexible which allows them to be easily cut to size and also to be used on curved surfaces. The only user filled linear shaped cutting charges that the author is aware of are rigid. These user filled charges have the benefit that they can easily be transported to the location at which they are to be used and then filled with locally available explosive material. Care must be taken during the filling of such charges as any inconsistency in the filling can affect the cutting performance as will the type of explosive used to fill them. Within the demolition industry it is common to carry out a trial blast to demonstrate that the explosives selected are correct and that debris containment is effective. This is essential in the case of user filled linear cutting charges filled with a previously unused type of explosive as only small changes in the characteristic of the explosive can have a significant effect on the cutting performance of the charge.

4.4 Kicking charges are used where steel sections can be flame cut and shims inserted to the cut prior to the demolition taking place. The explosives are placed immediately adjacent to the steel section so that when detonated the blast energy displaces the leg section. The preparation of the steel section needs to be given careful consideration so as to ensure that it can be readily displaced and will not become trapped as the building collapses. The type of explosive used and its placement are important at using an explosive with

too high a velocity of detonation will cause fragmentation of the target steel and positioning the explosive incorrectly will cause the blast energy to 'punch' through the steel rather than be transferred in to the steel and cause displacement as required. The timing of the detonation of kicking charges is critical as too long delay between detonations allows time for loads to transfer between adjacent legs in a row and too fast between detonations can cause the effect of the blast waves to become combined and cause air blast damage. It is always beneficial to pack the space between the flanges with sand bags on either side of the web to assist with transmitting the kicking force between the two flanges.

5. EXAMPLES OF STRUCTURAL BEHAVIOR

5.1 Reinforced Concrete chimneys

5.1.1 The first thing that must be understood about a concrete chimney is that it is not a gravity structure that is stabilised by its own self weight alone. When a strong wind blows on a concrete chimney the chimney wall on the downwind side will be put in to compression and the upwind side will go into tension. The steel reinforcement bars in the concrete are designed to take this tensile load.

5.1.2 Best practice arrangement for the preparation of a concrete chimney is as shown on figure 1.

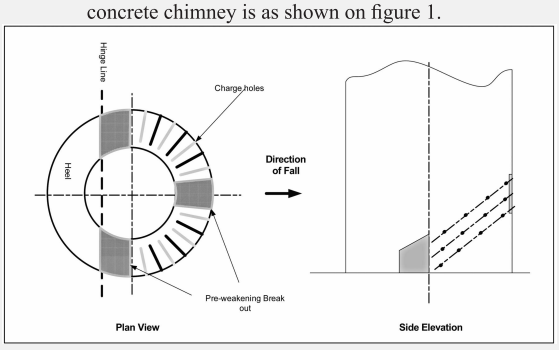


Figure 1

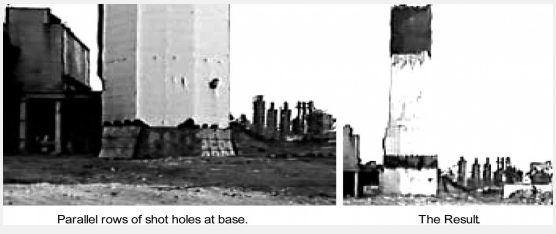
5.1.3 The rear edge of the two holes on either side of the chimney form the hinge line and dictate the direction in which the chimney will fall in the early stage of the collapse. Care must be taken here if an existing opening is to be used on one side of the chimney and a break out is to be formed on the other side as the existing opening may have additional steel reinforcing bars around it's perimeter to compensate for the loss of strength due to the existence of the opening.

5.1.4 In the early days of demolishing concrete chimneys by blasting the practice was to blow out a section of the front wall parallel to the ground to form a mouth with the intention that the chimney would fall in that direction in much the same way that a brick chimney would. Upon closer analysis we see that this is not the case. When the explosives are fired in a brick chimney they blow out the band described above which then allows the brick work above to drop as it has no tensile strength, thus forming a sloped opening in the front of the chimney as shown in figure 2.This does not happen in a concrete chimney as the steel reinforcing (and to a limited extent the concrete also) has tensile strength.

Figure 2

5.1.5 Having blown out a section of the chimney wall parallel to the ground transferring all the weight of the chimney into the hinge areas, which are behind the center of gravity of the vertical chimney. This eccentric support is intended to 'tip' the chimney is the required direction. As the top of the chimney moves very slightly forward and out of vertical the compressive stress rises rapidly in the hinge point whilst reducing rapidly in the heel. This mechanism can cause the hinge to become over loaded and fail compressively from the front edge back towards the heel very rapidly.

5.1.6 The result is that after the initial slight lean in the required direction of fall the chimney rapidly drops vertically crushing the hinges and heel, at this point the chimney will be free standing on a pile of hardcore with an approximately flat base, so could fall in literally any direction or may just remain standing vertically. Upon carefully examining video recordings of the demolition of concrete chimneys worldwide that were considered as successful the author was able to determine that a high proportion of these chimneys were very close.



How it happened.

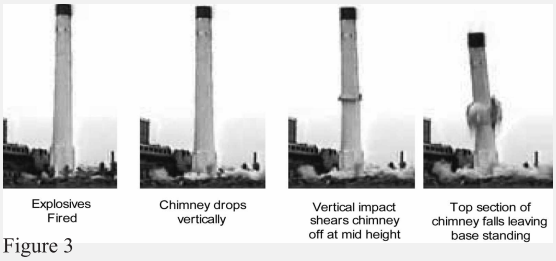


Figure 3

5.1.7 In order to prevent the foregoing situation occurring there are two steps that the prudent engineer can take. The first thing to do is to set out the pre-weakening and blast plan so that a flat line on the front of the chimney is not formed. This is done by raising the position of the opening broken in the front of the chimney and then blasting a sloping section out from the central front opening down to the side openings. The second thing to do is to reduce or even completely remove the tensile capability in the rear (heel) of the chimney by exposing and cutting the steel reinforcing bars.

5.1.8 Experience has demonstrated that if the practices outlined above are followed the direction of fall of a conventional reinforced concrete chimney can be controlled to within 5ºeither side of the required line.

6. STEEL FRAMED STRUCTURES

6.1 Tall steel framed structures are more often demolished by explosives than their concrete counterparts of similar height. The reason for this is that many super high reach demolition machines are not able to carry a sufficiently heavy tool to cut through large steel beams at high level. The risk of cut steel sections falling to ground level in an uncontrolled manner and striking the demolition machine must also not be over looked.

6.2 There are two common types of charge used during the explo sive demolition of steel structures. Shaped cutting charges and kicking charges which can be used either individually or in combination.

6.2.1 Explosive cutting charges

6.2.1.1 If explosive cutting charges were to be used exclusively without any pre-weakening the quantity of explosives that it would be necessary to use would be both expensive and also very likely produce an unacceptable amount of air over pressure. To reduce these problems we carry out pre-weakening of the structure by flame cutting in advance of placing the charges, leaving only the necessary size sections to be explosively cut. In order to be able to design the pre-cutting we must carry out a structural assessment and consider the loads that the steel members we are intending to precut are carrying and then calculate how much metal needs to remain uncut for the building to remain stable even under storm conditions.

6.2.1.2 When cutting charges are used in close proximity to each other on the same member careful considerat ion must be given to the initiation system to be used to ensure that they are fired absolutely simultaneously. If the charges are not fired simultaneously there is a risk that leads could be cut, the blast could displace adjacent charges and also vibration in the target steel could greatly reduce the cutting capacity of a given size charge.

6.2.2 Kicking Charges

6.2.2.1 Where steel members are only carrying compressive load and there is no way that the compressive load may be turned tensile by say storm winds, these members may be flame cut right through during pre-weakening and displaced by using a kicking charge.

6.2.2.2 When columns / Legs in a building are to be flame cut for later removal by kicking charges shims are generally inserted in to the cuts in the flanges so as to prevent the cuts closing. If the cut were to be allowed to close this could cause two possible problems. The first is that the structure would drop down by the width of the cut which may cause stability issues and the second is that a flame cut is not smooth so when the top face and bottom face close up there is a tendency for them grip together and become more difficult to displace sideways.

6.3 The mechanical characteristics of steel means that steel structures tend to undergo less break up whilst collapsing than an equivalent concrete structure. There are exceptions to this such as buildings that are designed to withstand earth quakes and very heavily reinforced concrete structures like silo's, When demolishing a steel framed structure an assessment needs to be made as to how much break up can be relied upon due to the self-weight of the structure and how much work needs to be done by explosives. The simplest mechanism of collapseis to topple a structure about its narrow axis by displacing a sufficient length of leg from one side to allow the building to roll forward to a point at which the center of gravity moves in front of the point of support. Heavy steel framed buildings and industrial structures often have sufficient self-weight to ensure that tension cannot occur in the legs so permit the sole use of kicking c harges. Lighter structures require that some of the leg section remain to ensure stability in which case cutting charges are required.

7. CONCRETE FRAMED BUILDINGS

7.1 Concrete framed buildings such as tower blocks are generally either precast, in-situ cast or a combination of both methods of construction. There are variations on these designs such as the use of 'no fines' concrete which has very little if any load carry capacity being used to form walls between load carrying walls. To the casual observer no fines concrete looks the same as the rest of the walls. Another method of construction used in where a steel framed structure is encased in concrete. This can look very much like a reinforced concrete structure. It is important that the construction

of the building is properly understood before the collapse mechanism and pre weakening can be designed.

7.2 Once all of the non-load bearing walls etc. have been removed the building will be standing on either legs/columns or on concrete walls.

7.3 The space available to accommodate the debris and the need to not damage adjacent structures are also factors that must be taken into consideration when designing for the collapse of a building. The position of the debris pile can be controlled by the use of time delays between charges across blast floor levels and also between floor levels within a building.

7.4 The pre-weakening break out is necessary to leave only panels that are of a suitable size to permit drilling and blasting. The break outs need to be designed so as to allow sufficient back space to accommodate standard length drill rods, typically up to a maximum of 2.4m long. The design should also allow for as much of the structural walls as possible to remain intact as far as practical from the center of the structure. This has the effect of maintaining as much of the original building stability as possible when the building is in its pre-weakened condition. A typical example floor plan is shown in fig 4.

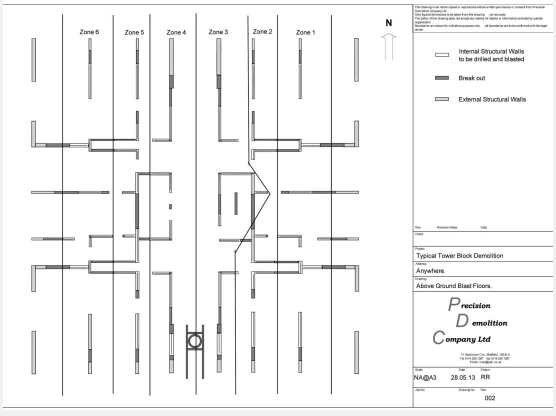


Figure 4

7.5 In the case of the example shown in figure 4 it was necessary for the building to fall as far as possible to the east with as little debris as possible to the west. For this reason the detonation was split into six delay zones. The time delay between each zone reduces as the building accelerates but are in the order of 100's of milli seconds.

7.6 Using this method it was possible to move the center of the debris pile approximately 18m from the original center of the building.

7.7 In order to ensure effective breakup of the building as it collapses it is necessary to blast ground and first floor levels and then every 5th of 6th floor working up the building. Depending upon the construction of the building and the required mechanism of collapse it may also be necessary to carry out limited pre-weakening and possibly drilling and blasting on intermediate levels between the main blast floors.

7.8 In order to prevent explosively propelled fragments of concrete causing damage or injury it is necessary to ensure that these are all contained within the building. This is achieved by two means referred to as primary and secondary protection. The primary protection generally used for tower block demolition is a combination of chainlink fencing and geotextile sheeting. The specification for each of these materials and the number of layers used depends upon the thickness of the concrete walls and the explosive loading used. A typical arrangement is shown in figure 5.

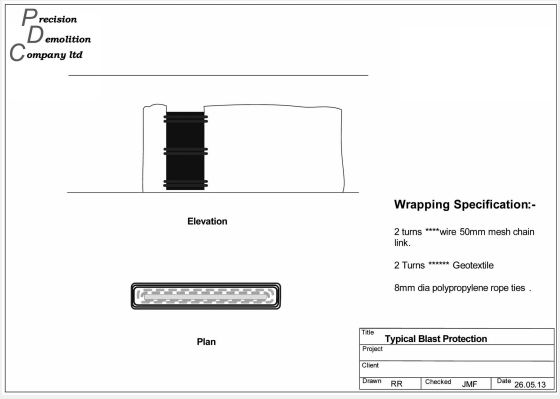


Figure 5

7.9 Secondary protection is usually in the form of addition layers of geotextile sheeting covering the windows at each blast floor level. Where perimeter walls are removed on the blast floors as part of the pre-weakening the secondary protection will normally also require the use of chainlink fencing.

Blast Folding of Reinforced Concrete Chimneys

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Design Office of Structure Demolition, Dresden, Germany

ABSTRACT: Multiple blast or "folding blast" of industrial smokestacks constructed in reinforced concrete is an excellent solution to collapse these ones in a small bordered area. A normal single tilted smokestack needs more length for its soil impact as its height. If you create a folding blast with a zigzag-collapse figure you get an impact length in only a fraction of the smokestack height. There are lesser problems with deviations from planned tilting direction because the chimney executes more a vertical collapse then a tilting and that's why its behaviour is more according the principle of minimal energy. Vibrations of soil impact will be reduced too because of the energy divergence by self-destroying and because of the longer time to input the energy into the soil. Since 1987 in Germany has been developed a blast demolition method named "Blast Folding" for concrete chimneys. Several blast mouths in a chimney make a collapse in a zigzag figure. Nevertheless 36 concrete chimneys already are blasted in this new method in Germany. Also thereby were vive chimneys with heights of 300 m.

1. COMMONS

In case of a normal and easy tilt blast of a chimney the impact length needs a dimension of about 110 % till 130 % of its height. In industrial areas this size of impact length often lacks.
But also the safety of realising the planned fall direction is not ever high in this manner of easy tilt blast.
Nevertheless the method "Blast Folding" is a good other possibility. In different heights arranged and alternatively opened blast mouths are detonating in different times and create very different collapse figures of high chimneys.

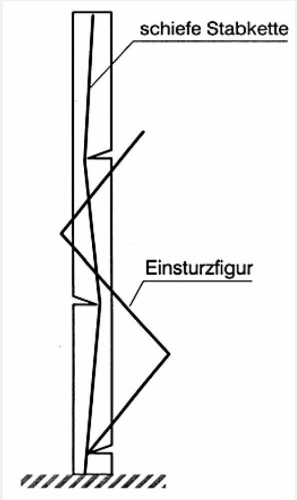

Figure 1. Folding collapse figure

In the blast folding the chimney parts mutual shifts (or helps) itself in the right direction. As the collapse is more vertical as lesser energy is needed for. Therefore the safety of realisation of the planned fall direction increases by folding.
I am able to calculate such collapse figures to create an optimal demolition in the given area. The maximum number of folding parts in my calculation is 10. The structural model of folding a chimney to a zigzag figure (here: three-mass-system with eccentric ductile hinges) is shown with follow designations:

m_3, J_3 ...Mass / Mass rotation momentum 3. Part
a_3 ...Mass Centre of the 3. Part
c_3 ...Eccentricity of the 3. Hinge
MF_3 ...Ductile momentum in the 3. Hinge
x_3 z_3...Displacements of the 3. Mass
φ_3 ...Rotation of the 3. Mass
t_3 ...Blast Ignition Time of the 3. Part (no hinge before blast, ductile hinge after blast)

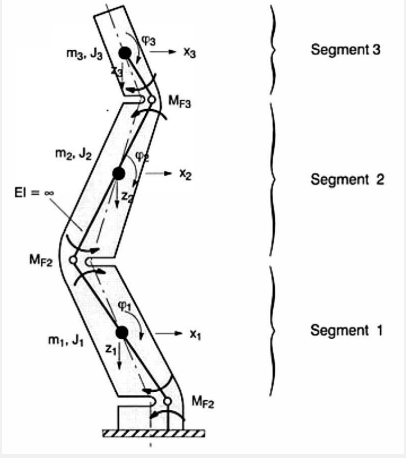

Figure 2. Fold collapse model

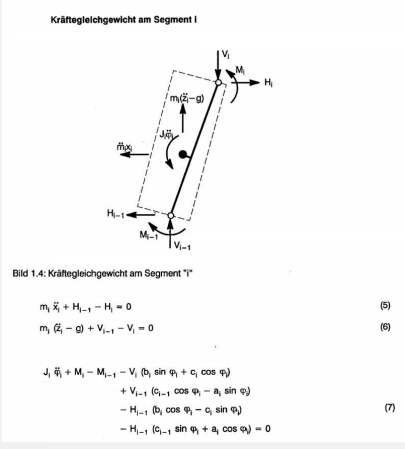

Figure 3. Math. equations for one segment

By the way, the multi-pointed soil impact and the high level of self-destruction of the folded chimney results lower soil vibrations.

In Germany have been successfully blast-folded nevertheless a number of 36 reinforced concrete chimneys with heights between 80 m and 300 m. But also brickwork-chimneys are able to be blast-folded. Other high buildings like towers, bridge pillars or sky scrapers can be collapsing in a zigzag figure too.

2. EXAMPLES OF BLAST FOLDING

2.1 160 m-Chimney in the power plant Rostock-Bramow

A normal tilt blasting of this 160 m chimney would need an impact length of about 200 m what not existed in the area of the power plant lying on the river Warnow. Between the river Warnow and an industrial zone for the chimney existed allowed lengths of only 40 m in one side and 100 meters in the other side. The smokestack in 1976 erected had a reinforced concrete shaft with a 5 m high brickwork head. The full inner brickwork liner has been released before. The resulted smokestack mass has got 3021 t. Under the given relations raised a two parted blast folding as optimal. The under blast mouth was defined on the ground. It was opened in the main fall direction and detonated at zero. The upper blast mouth was defined in a height of 75 m. It was opened against the main fall direction and detonated at time 4.5 seconds. In fig. 4 the computer simulation of the collapse figure at time = 4.0 s; t = 7.0 s and t = 8.5 s is shown here.

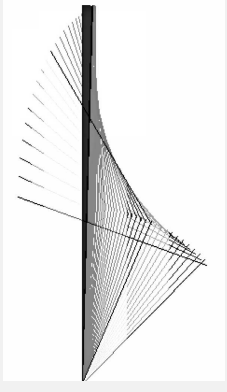

Figure 4. Computer simulation of the collapse

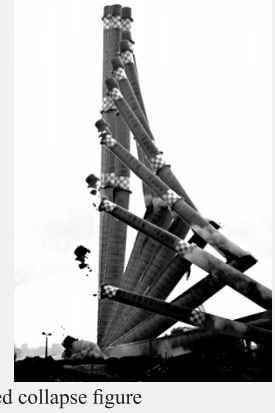

Figure 5. Realised collapse figure

The blast folding of the 160 m-chimney in Rostock at 1998, July 1st showed a good conformity between calculating and realising. The realised impact lengths were only 70 m to the right direction and 10 m to the left, so it was the half of the original height. The demolition contractor was the company “TVF-Altwert”.

*2.2 300 m chimneys in the power plant Boxber*g

In the power plant Boxberg had to be demolished three very large chimneys with heights of 300 m. The demolition contractor was the company “TVF-Altwert” too. Only about 100 m of both sides of a smokestack were free for an impact.

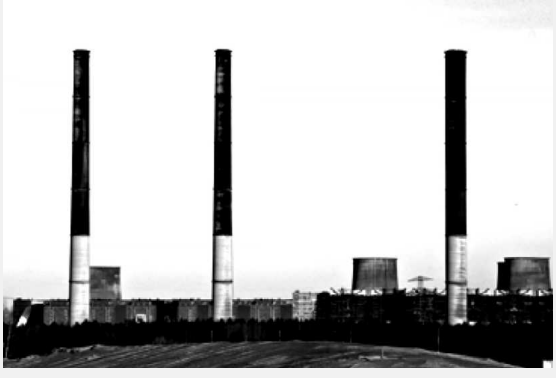

Figure 6. Boxberg, 300 m-Chimneys Nr. 1 - 3

In this case the strategic design was a three times folding. The under blast mouths lied over the conic foot shell in heights of 34 m (No. 1 and 2) respective 52 m (Nr. 3). The middle and the upper mouths divided the chimney in three about equal parts. The blast time of all mouths was zero.

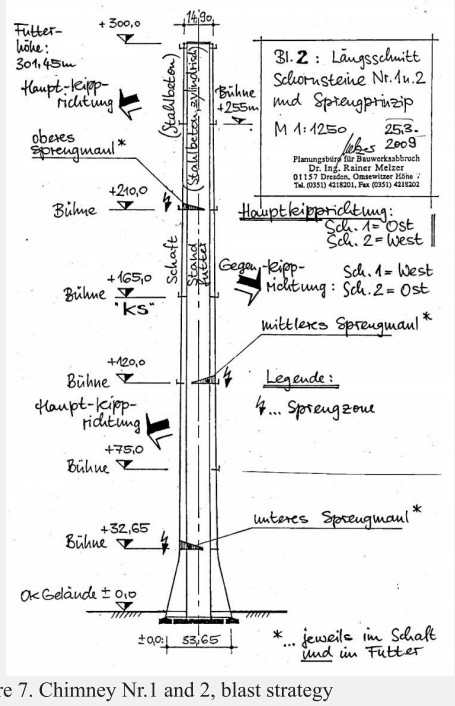

Figure 7. Chimney Nr.1 and 2, blast strategy

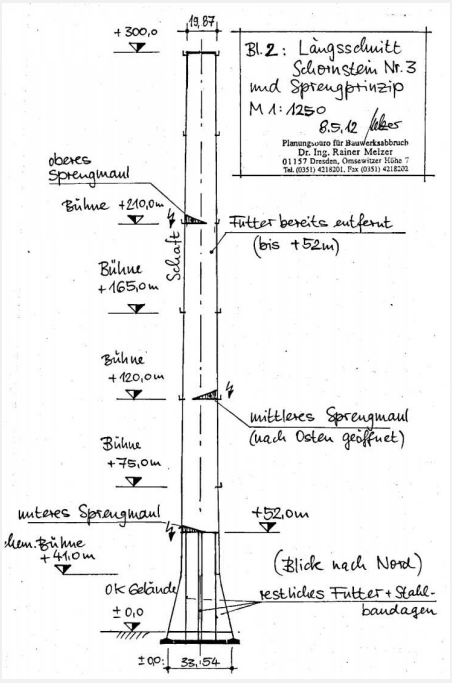

Figure 8. Chimney 3, blast strategy

All chimneys had an outer and an inner tube made in reinforced concrete. Only the inner line of chimney Nr. 3 was demolished before blast. The mass over the under mouth was about 19,100 t (Nr. 1 and equal Nr. 2) respective 10,900 t (Nr. 3).
The chimneys Nr. 1 and 2 were both blasted in 2009 May 9th. The figure 9 shows overlay photos of the zigzag collapse.

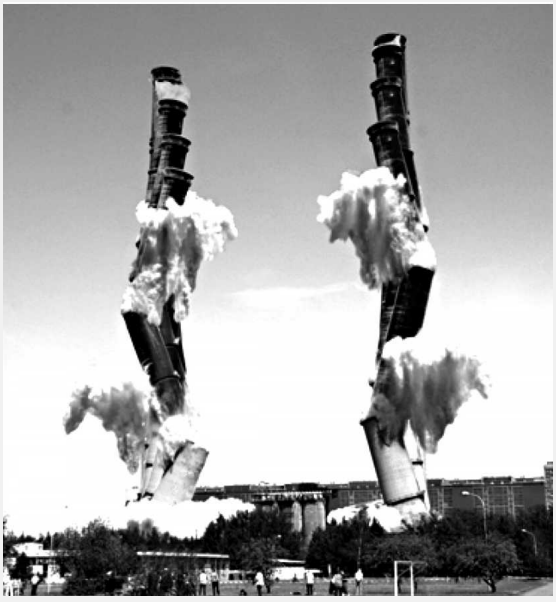

Figure 9. Nr. 1 and 2, colapse

The no blasted foot shells were full destroyed too. The waste heap of Nr. 1 and 2 (sum total almost 50,000 tons!) was very small. Only about 60 m in the fall directions were needed (see fig. 10).

Figure 10. Residual of Nr. 1 and 2

The chimney Nr. 3 was blasted in 2012 October 6th. The figure 11 shows the zigzag collapse.

Figure 11. Nr. 3, zigzag collapse

The waste heap of Nr. 3 was very small. Only 45 m to west and 60 m to east were needed.

Figure 12. Residual of No. 3

Because of the good reinforced shell and liners the foot of the chimney Nr. 3 was intact after the blast (see fig. 12).
The impact vibrations were very small in fact of the large masses and the high energies of the chimneys. No damages were at all. It was a big work but a big success.

CONCLUSIONS: Blast folding of high structures is a hard work but an excellent solution to collapse a structure in a small bordered area. Several blast mouths in a structure make a collapse in a zigzag figure. There are reduced problems with deviations from planned fall direction. Vibrations of soil impact will be reduced. Nevertheless 36 concrete chimneys are blasted already with this method in Germany. Blast folding of other structures like towers, bridge pillars and skyscrapers is possible and has been realised too.

Demolition blasting of newly constructed bridges due to engineering error.

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ABSTRACT: Failure to properly investigate ground stability prior to construction of seven highway bridges on a new alignment through the coastal mountain range in the US state of Oregon resulted in an expensive catastrophe. Four of the newly constructed bridges, one complete and three in various stages of completion, were in a landslide area that was moving, making the bridges unsafe. After two years of study and failed retrofit attempts the bridges were then demolished by blasting, the subject of this paper.A total of 12 free-standing bents (bridge supports), measuring 2-2.5 meters diameter by 20-35 meters tall were blasted down on the 3 uncompleted bridges, and 3 bents of similar size plus 200 meters of roadway deck came down on the fully completed bridge. The use of electronic detonators allowed the bents and bridge to be felled into designated locations away from watercourses and into areas for easier reduction and removal.
The cost to construct the bridges prior to demolition was approximately US$17 million. Additional costs going forward for the new alignment due to the original engineering error is approximately US$400 million.

1. PROJECT BACKGROUND

The 84km stretch of roadway US-20 between Corvallis and Newport, Oregon is one of the deadliest roads in Oregon, with an accident rate 60% higher than the state average. The road twists and turns through the mountains of the Oregon Coast Range, following rivers for most of its length. These tight curves forbid longer commercial vehicles from using the road, necessitating longer routes through other passes adding hundreds of kilometers to round trips. Passing is permitted in only a few areas across the entire stretch of highway and speed limits are low, frustrating drivers and contributing to traffic accidents. The designation of the road as a Safety Corridor in 1996, bringing more patrols and higher fines for traffic infractions, did little to decrease the rate of accidents.

County government officials, residents and local businesses lobbied for over a decade, demanding that something be done to address the deadly road. Finally in 2001 the 'Oregon Transportation Investment Act' was passed, allocating some $400M (€310M) to infrastructure projects. This abundance of local funding, along with a temporary high in Federal Fund matching, prompted state officials to seriously consider the project for the first time.

Due to a lack of in-house engineering expertise at the Oregon Department of Transportation (ODOT) and a short time window to apply for Federal funds, the state issued a request for Design-Build bids. Under the bidding specifications, the winning contractor would design as well as construct the project, subject to a number of requirements by ODOT including the stipulation that a number of sensitive streams be crossed by bridge rather than routing the stream through culverts. Initial cost estimates of $40M (€30M) quickly grew in excess of $100M (€77M). After an accelerated bid process the project was awarded to a winning bid of $130M (€100M). It was to be ODOT's most expensive piece of infrastructure work since the completion of the Interstate Highways decades prior.

The project would entail replacing 13km of winding two lane road with a 4 lane highway running a straightened course than reduced the length to 8km while crossing several sensitive waterways that had previously been untouched. Along with the bridges there was a substantial amount of cut and fill work to be done.

2. SITE BACKGROUND

These sensitive waterways are chiefly the Yaquina River, which runs alongside US-20 for several miles and eventually empties into the Pacific Ocean through the Yaquina Bay at Newport and numerous creeks and streams which feed the Yaquina. A number of salmon species spawn in the Yaquina and its associated tributaries while fishing and seafood processing are a sizable portion of the economy in the lower Yaquina basin. Yaquina Bay is maintained as a deep water port by the US Army Corps of Engineers.

The geology of the surrounding area is that of bedrock and mountains formed by volcanic basalt flows topped with clay shale and fine grained sandstone. The shale layers are primarily composed of bentonite clays, particularly known for their water absorption. With a high water content the friction between grains of bentonite is reduced significantly. With such slippery soil the Coast Range is host to a number of ancient landslide zones. The rugged terrain is also heavily forested and otherwise vegetated, making surveying very difficult.

3. PRIOR WORK

Early in the project the contractor cleared the vegetation from 64,000m3 of embankment without ODOT approval, prompting ODOT to halt the project and ask for the contractor to replace the project manager. While these discussions were ongoing, a heavy rainstorm washed an immense amount of material from the bare ground into a number of tributary streams of the Yaquina River. The state Department of Environmental Quality (DEQ) cited ODOT for 61 violations and levied the largest fine in ODOT history. DEQ would later fine the contractor as well.
Work was halted shortly in 2007 as problems with landslides above road cuts were addressed.

4. GROUND MOVEMENT & ATTEMPTS TO STABALIZE

In 2010 work was again halted when the contractor informed ODOT that ground movement, similar to the landslides thought dealt with, were again causing problems. Three bridges had been completed with another three bridges in progress when it was noticed that the ground was moving. Beyond simple landslides, the movement of underlying soil layers was forcing one completed bridge's deck up out of its abutment and at several other crossing sites, partially completed supporting bents (two

concrete legs with a concrete cap joining them at the top) were leaning substantially out of vertical. The leaning bents and bridge were given a short-term fix by jacking them back upright as the contractor and ODOT scrambled to address the underlying problem. It was determined that a large section in the middle of the project area was situated on a number of ancient landslide zones.

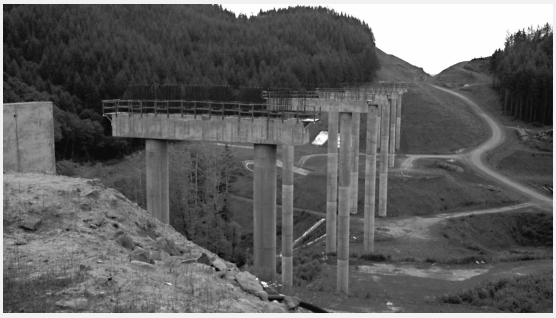

Figure 1. The largest incomplete bridge site, consisting of six bents.

Average annual precipitation in the area is in the lower range of the entire Pacific Coast Range. However, the area still averages a significant 170cm of precipitation a year. The resulting saturated ground in historic landslide areas was proving to be more of a problem than initially anticipated. These slide zones had been identified before the project began and were addressed in the contractor's plans. Those plans proved inadequate. While the bridge bents were anchored into the ground at a considerable depth, the ground was moving for a majority of that depth and simply pushing the bents over. Given the height of the bents (the tallest being 38m above the ground), a minor displacement at ground level produced a pronounced lean at the top.

Laser reflectors were affixed to the bents and bridge to monitor their position and many instruments were placed around the project site to better gauge the movement of the ground. A number of horizontal drain holes were drilled deep into the hillsides in an attempt to drain water from the soil and hopefully stabilize it by increasing the coefficient of friction.

While monitoring and abatement were ongoing, fingers of blame were being pointed back and forth between the contractor and ODOT. ODOT claimed that the contractor was liable for all costs accrued to address the slides as this was a preexisting condition at the site that should have been accommodated in the original bid. The contractor claimed that the ground movement was so much more than anticipated that it created a changed site condition, releasing them from liabilities and requiring a change order if ODOT wished the contractor to remediate the bridges.

5. DECISION TO DEMOLISH

After two years of monitoring and mitigation attempts, ODOT dismissed the contractor from the project and collected a relatively small amount of money from the contractor's bond. ODOT chose to demolish the failing bridge and bents as there was simply no economically feasible way to shore up the ground enough to safely support the bridges. The project would essentially start over, already years behind schedule and significantly over budget. The removed bridge and bents would then be replaced with culverts and fill, requiring upwards of 760,000m^3 of fill.

Rhine Demolition of Tacoma Washington was hired as the demolition contractor. Rhine hired Wallace Technical Blasting of Woodland Washington to blast the standing bridge and bents as they were too tall to safely demolish solely by mechanical means.

6. DRILLING OPERATIONS AND CHALLENGES

The structures to be demolished included one completed bridge and free-standing bents at 3 additional crossing sites. The completed bridge was 200m long, supported by 3 bents as tall as 27m above ground. Each of the 3 uncompleted bridge sites had between 3 and 6 bents, for a total of 12 more, free-standing bents to be removed. Each bent consisted of two concrete legs either 1.8m or 2.4m in diameter, between 24m and 38m tall with a supporting crosspiece at the top joining the two legs.

Initial design for drilling called for eleven 89mm horizontal holes to be drilled in each leg of a bent. Five holes were to be placed along the center line, spaced 0.6m apart vertically, drilled to a depth of 0.6m from the back wall of the leg. For a 1.8m diameter leg, each blast hole would be 1.2m in depth. For a 2.4m leg, a 1.8m hole was called for. There would be 3 groups of 2 holes drilled horizontally, spaced evenly between the centerline holes but off-set to the sides 0.3m from the centerline, giving an inter-hole spacing of 0.6m. These holes would be slightly shallower to maintain setback from the rear wall of the leg. The drill to be used was a typical construction/quarry type self-contained, tracked drill.

As drilling progressed, the rebar reinforcing the legs was found to be a serious problem. All legs were banded by many rings of closely-spaced, small diameter rebar along their entire height. The 1.8m bents were encircled by very hard 50mm vertical rebar unevenly spaced around the entire circumference of the leg just under the surface. Rebar joints were locked together with large couplers 76mm in diameter by 360mm long. The 2.4m bents had 76mm rebar for approximately the first 2.4m of above ground leg, after which a large step-down coupler linked it to 50mm rebar for the rest of the leg's height. Drilling through the rebar was slow and very damaging to drill bits, but possible. Drilling through a coupler was not possible.

Thanks to the rebar bits were being consumed at an unacceptable rate. Usually only 3 or 4 holes could be drilled before a bit was destroyed. To allow drilling to progress without running out of bits the outlying holes were eliminated. This dropped the number of holes per leg from 11 to 5, greatly reducing the number of bits used in a day.

Despite these adjustments, drilling continued slowly and required more bit replacement than normal. To address these issues, we hired a specialty concrete coring contractor to create a slightly larger starter hole by coring through the rebar and/or couplers, allowing our rock drill to move in later and quickly finish the hole to the required depth. Though the coring process was slow, it allowed for the economical use of time for our rock drill and saved numerous bits.

Figure 2. A 76-to-50mm rebar coupler, cored through. Drilling such hard steel with our rock drill and a conventional bit would have been impossible.

The drill design for the completed bridge deck consisted of single rows of 13 holes, evenly spaced across the 18.2m width of the deck. A 13-hole row was drilled vertically (relative to the curve and lean of the bridge deck) in the middle of each of the 3 bent top caps, where the sections of bridge deck and monolithic pre-tensioned concrete beams met and were joined with a large amount of rebar and concrete patch. These holes were 4m deep, again bringing them to a 0.4m setback from the bottom of the bent caps. As there was a tangle of rebar tying the bent cap, beams and bridge deck together the rebar again proved problematic. As luck would have it, several holes were found to contain vertical rebar for their entire depth. Not only did this cause slow drilling, but a chunk of this rebar became lodged in the central air blow-hole in 2 different bits. As with the bents before coring, a bit would survive drilling 3 or 4 holes before being destroyed.

7. BLASTING DESIGN AND RESULTS

The goal of blasting was not to totally demolish the bents or bridge, but to bring them to the ground where they could be safely demolished by mechanical means. Thus blasting only needed to remove enough material and strength to bring a bent off center; the design enlisted gravity to do the rest. With few, short holes available to load the powder factor would necessarily be very low. As such the blasting design took cues from a paper (Kamburova 2009) presented at the EFEE 2009 Blasting Conference in Budapest regarding the use of water in blasting operations. It was decided to take advantage of the mechanics of water to best use the limited explosives we could load in each hole. As the bent holes were horizontal they couldn't be pumped full of water. Instead a 200ml bottle of water was placed in the back end of each hole. Then half of a 50mm by 400mm cartridge of detonator sensitive emulsion was tamped in around the water bottle, followed by a 0.34kg cast booster primed with an electronic detonator. Finally a full 50x400mm cartridge was loaded, then a hole plug and 0.4m of stemming gravel. In the most extreme, this resulted in a powder factor of 0.03kg/ton in a 38m tall bent with 2.4m legs.

Figure 3. The designed loading of one hole. Water bottle, 1 ½ cartridges of detonator sensitive emulsion and a primed 0.34kg cast booster.

Electronic detonators were chosen for their programmability, testability and accuracy. With the goal in mind of destabilizing a bent and causing it to fall in a certain direction, the very precise timing allowed by electronics was required. By firing the holes in a designed order with carefully calculated timing the hope was to shift the center of gravity enough to cause the bent to fall in the preferred direction. The preferred direction of fall was chosen in terms of the ground level terrain which would provide the easiest access by mechanical reduction and debris removal equipment and the proximity of waterways and fragile monitoring equipment.

Finally, a 35mm cable was attached to the cap of the bent, run through a pulley and then to the bucket of an excavator. This cable was pulled taut prior to blasting to provide a slight pull in the intended direction of fall. Ideally this would influence the bent to fall in the correct direction, if not the cable would simply snap from the immense weight and the bent would fall in whichever direction it chose.

In practice, the blasting design was found to work extremely well. The water in the end of the hole was driven out the back side of the leg, throwing all of the small diameter ring rebar a considerable distance. All concrete in the loaded zone as shattered and thrown clear of the rebar cage in the leg. Additionally, the shock of the detonation broke the rebar free of the concrete for a considerable distance up and down the leg from the loaded zone. This resulted in a roughly 3m post-blast section of leg where only the vertical rebar remained. With no supporting concrete or ring bar the immense weight of the bent buckled this freestanding rebar and compressed almost directly down, then the bent leaned and fell to the ground.

The tensioned cable system to influence the direction of fall worked reasonably well and the majority of bents fell in the designed direction. There were two primary failures observed with this method. First, excessive tension may be applied to the cable to the point where it breaks before the bent has sufficiently leaned in the correct direction. The rebounding effect imparted by the sudden loss of tension caused the bent to fall the reverse direction in two instances.

Secondly, the inherent lean in the bent resulting from ground movement was, while slight, too great to overcome due to the immense weight of the bent. Once the shot was fired and all strength removed from the bent, it simply fell in the direction it had already been leaning. This was observed in three instances.

While the initial design called for five centerline holes and indeed the initial blasts were loaded to the design, the blasts were later stepped down to three centerline holes in each leg. While roughly halving the blasted zone from 3m to 1.8m this was still sufficient to destabilize the bents and bring them down.

Generally the blast timing was quite simple. For a bent designed to fall directly forwards or backwards, all holes were fired simultaneously. The precise timing of the electronic detonators was felt to be essential in this regard as any pyrotechnic scatter in standard non-electric detonators may have resulted in uneven firing, perhaps allowing slightly later holes to have more relief than intended and causing unwanted shifts in the bent's center of gravity.

There were two other desired directions of fall, for which timing was changed. One was to 'swing' a bent, that is to cause it to fall a few degrees off of straight forward or backward. To achieve this one leg was fired 15 milliseconds before the other to allow the center of gravity to shift more towards one side before the second leg fired and the entire bent began to fall.

Secondly was to fall a bent sideways, essentially directly over one leg. This was achieved by loading five holes in the leg on the side the bent was intended to fall towards, while loading three in the offside leg. The five holes were fired 35 milliseconds before the offside legs, again allowing a greater shift in center of gravity in the designed direction. This, coupled with the differential length of blasted zone in each leg caused the bent to fall directly to the side as designed, with one leg lying on top of the other.

Figure 4. Bents laid down on the ground in three different directions. The farthest away swung slightly to the left, the middle laid downhill and the closest bent laid sideways.

Thanks to the precise timing allowed by the electronic detonators we were able to demonstrate that we generally could influence a free-standing bent to fall in whichever direction we chose. This would be crucial in the blast design for the completed bridge.

The bridge was located in a delicate area. It crossed an ecologically sensitive waterway that opened into a fairly wide stream bed downhill from the bridge. To fall the bridge downhill would follow gravity; ground movement was already pushing it in that direction. While this would be the easiest method of blasting, it would drop the bridge right into the sensitive stream. Thus, the bridge had to fall uphill.

To achieve this, the sideways blasting method was applied to the bent legs. The uphill legs would have as many holes drilled into them as possible. Due to height reach limitations with the coring drill, and depression and ground clearance limitations with the rock drill's boom this would be six holes. The downhill legs would again have 3 holes, maximizing the blasted zone differential between the legs. The holes would be loaded to the standard design, with the detonators programmed to fire the uphill legs first followed by the downhill legs 64 milliseconds later.

The deck and bent caps, with their 4m holes would be loaded in three decks. The bottom deck consisted of a primed cast booster a 50x400mm cartridge of detonator sensitive emulsion, further decks consisted of a single cast booster. All boosters and thus decks were linked by threading 50gr detonating cord through the boosters, only the bottom deck contained a detonator. Decks were separated by hole plugs inserted to the desired depth.

The high level of political visibility of the project had resulted in the need to establish viewing areas for officials, press, and spectators that would be occupied during the blast on the completed bridge. Hanging blasting mats or other safety devices over the side of the bridge and around the bent caps was not feasible. It was decided to err on the side of safety by reducing the load factor, as the selected viewing areas afforded spectacular views of the bridge but might also be in the direct line of fire in the unlikely event of flying debris.

The blast holes in the bridge deck were horizontal holes in sound concrete, so they would be pumped full of water to just short of the collar, where a hole plug and stemming gravel would be placed. The deck holes would fire 156 milliseconds after the downhill legs, breaking the connections between the individual bridge components and damaging its rigidity such that it would retain enough structural integrity to fall only partway into the canyon and bind up.

After consultation with the demolition contractor, it was decided that no holes would be drilled in the east and west abutments of the bridge for two reasons. First, the bridge was just sitting on top of these concrete abutments as evidenced by the ground movement pushing the east end of the bridge up and out of the abutment. Secondly, the expense of this extra drilling was deemed to be unnecessary.

When the blast was fired, the legs fired as designed. Where before the bridge was banked, with the uphill side of the deck higher than the downhill side, it was now roughly level and leaning uphill as the deck holes fired and it began to fall. Unfortunately it dropped down a short distance and stopped. The shot bottoms of the legs had driven into the ground and the bridge deck retained too much strength to give way and continue falling.

Figure 5.The bridge after the blast, taken from the east abutment.

What happened? A number of factors combined to throw a wrench into our carefully laid plans. First, the bridge was, indeed, only sitting on the east abutment. However, the bridge deck and pre-tensioned beams in reality were tied into the west abutment with rebar and a subsequent concrete pour, much like the joints between deck sections on top of the bent caps. The plans the demolition contractor had consulted when deciding not to drill and blast the west abutment were in error. Second, in the intervening years between the project shutting down and the cleanup phase beginning, the eastern abutment had sunk into the ground somewhat, angling itself in such a way that it was trapping the east end of the bridge rather than simply holding it. Third, our design for the deck and bent caps was insufficient. The lightened powder load had not destroyed the joints to the anticipated degree.

These factors combined to keep the bridge standing. The deck, being tied into the west abutment, didn't fall as far as anticipated, causing the east abutment to retain its hold on the deck while the partially intact joints on top of the bent caps retained enough strength to keep the bridge together.

While the blasting crew observed the results and discussed what to do the demolition contractor moved several machines to the west abutment and began hammering and cutting. Their belief was that, freed from the west abutment it would resume its fall and end up on the ground. After some work with no progress the blasting crew made the suggestion to hammer away at the east abutment. The bridge was not secured to the abutment in any way; it was simply sitting on it rather precariously. With quite a bit less work, the east end of the bridge should come free which, combined with the direction that the legs were leaning, should bring it the rest of the way down. The demolition contractor moved a machine to the east end and began hammering at the section supporting the bridge deck. The deck soon came free and began moving down the slope, tipping farther uphill until the legs gave out and it collapsed all the way to the ground. With the added time to move machines around and hammer away at the structures, the bridge fell just short of 3 hours after the time of blast.

Figure 6. The bridge safely on the ground, photo taken the next morning.

8. POST MORTEM AND CONCLUSIONS

In the face of high political visibility, the designed powder factor for the bridge was reduced to increase the safety factor rather than moving spectators to a less favorable but more protected location. In the future, safety should continue being the paramount concern. At the same time, work involving explosives should not take a subservient position to political concerns. On a project running years behind and massively over budget, the responsible government agency wished to show that it was turning over a new leaf. The relics of the failed stage of the project were being torn down and progress could finally be made. In retrospect, the spectators should have been moved to a further location, even if it offered worse camera angles, and the powder factor maintained at original design levels.

The decision not to drill and shoot the west bridge abutment was based on inaccurate as-built documentation. Even if it added expense and time, drilling in the west abutment may have revealed that the bridge deck was in fact locked into the abutment and the blasting design could have been changed to account for this reality. Experience on other projects has shown that inaccuracy in design and as-built documents is not an isolated circumstance. When the success of demolition blasting operations hinges on such structurally-based decisions, the blasting crew must make a persuasive case to the general contractor or owner to conduct whatever field investigation is needed to confirm actual on-site conditions as a part of the blast design process.

While extra mechanical work and a few more hours were required to bring the bridge down it ended up exactly where intended and no material fell into the creek. For a project several years and hundreds of millions of dollars over budget this 3 hour delay was a slight, easily resolved bump in the road.

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