OBJ3/P6/A7-17

Transport and Works (Inquiries Procedure) Rules 2004

Proposed London Underground (Victoria Station Upgrade) Order

LAND SECURITIES PLC AND OTHERS (Objector No. 3)

APPENDICES 7 - 17 to PROOF OF EVIDENCE of TIM CHAPMAN OF ARUP

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APPENDIX 7

WS ATKINS REPORT – RISK TO THIRD PARTIES OF TUNNELLING IN SOFT GROUND

HSE
Health & Safety
Executive

The risk to third parties from bored tunnelling in soft ground

Prepared by **W S Atkins** for the
Health and Safety Executive 2006

RESEARCH REPORT 453

The risk to third parties from bored tunnelling in soft ground

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This study has looked at the hazards that are associated with soft ground tunnelling in urban areas. It has considered all of the types of construction methods that are used in soft ground tunnelling. Data on tunnel incidents has been taken from the published literature and from personal data bases and other sources of information. This has been used to understand and describe the range of possible primary and underlying causes of failures. Methodologies for the mitigation of risks are examined and conclusions drawn for the effective management of future urban tunnelling projects.

The authors would like to express their thanks to colleagues in UK and abroad for their assistance in compiling the data used in this report.

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EXECUTIVE SUMMARY

1. Tunnelling is increasingly being used world-wide to provide the infrastructure required for sustainable urban communities. The majority of these works are completed safely and satisfactorily. A number of recent emergency events in urban tunnel construction in UK, and elsewhere, have raised concerns regarding the risk to third parties from future work (Chapter 2).

2. Quantification of this risk cannot easily be undertaken because of a lack of sufficient and reliable data. There is no centralised world-wide data base on tunnelling that can be interrogated to obtain classified factual information on tunnel construction projects, or centralised records of construction failures. Such information that exists is piecemeal and rarely accompanied by documentation and studied analysis. The fragmented and incomplete nature of this data therefore makes formal statistical analysis difficult and potentially unreliable and misleading. (Chapter 3, paragraph 3.1).

3. Data on world-wide tunnel construction projects undertaken within the past six years was obtained from a range of sources including the internet, and was then classified. It indicates that the number of tunnels being constructed in the developed world is increasing year on year, while activity in the rest of the world remains variable and an order of magnitude below the developed countries (Chapter 3, Tables 3.1 and 3.2).

4. Data were also collected on the number of tunnel emergency events "recorded" in the last four decades. The information was obtained from a variety of sources, including the technical literature and the personal contacts of the authors. It indicates that both the total number of emergency events and the proportion of these events taking place in urban areas with soft ground tunnels, increased in the three decades since 1970. In the latter case this trend has continued into this part decade (Chapter 3, Tables 3.4, 3.5 and 3.6).

5. Analysis of the available data by the authors indicates that

   - With soft ground urban tunnels, NATM tunnels exhibit different failure characteristics from non-NATM tunnels.

   - The primary cause of failure in NATM tunnels is attributed to unpredicted ground conditions.

   - In non-NATM tunnels, the range of primary causes for incidents is wider than for NATM tunnels.

   - The underlying causes of emergency events are likely to be diverse. These could involve engineering, management, procurement, organisational, competence, resource or communications issues.

   In addition, there is evidence that incidents within soft ground urban tunnels result in surface craters forming above the tunnel face in NATM construction and behind the tunnel face in non-NATM construction (Chapter 3, paragraph 3.8)

6. Risks to third parties from underground construction can be classified in physical, economic and societal terms. Control of the physical risk is governed in UK by the health and safety legislation applicable to construction works. The economic risk is managed through the contractual arrangements between the constructor, the insurer and the affected infrastructure owners. Societal risk is more difficult to define and manage as it can include a wide range of issues including wide spread detriment and socio-political issues (Chapter 4).

7. Urban projects introduce additional risks to tunnelling work due to the density of the existing infrastructure and the spread of the population. Construction methodologies may need to be adjusted to suit local environmental restrictions and working space can be difficult to locate and safeguard. The close presence of possibly aging and unfavourably sited infrastructure can introduce hazards that are not met on rural sites (Chapter 5).

8. The statutory methods required to control tunnelling safety risk within the UK construction industry are set out in the health and safety legislation. Detailed guidance is available in British Standard 6164 Code of practice for safety in tunnelling in the construction industry. However one consequence of the recent number of construction incidents in UK and elsewhere has been the setting up of a number of committees to propose measures for better management of tunnelling projects in order to reduce and minimise the number of failures together with their attendant commercial losses. These include

   - The International Tunnelling Association. Guidelines for tunnelling risk management. (2004)

   - The Association of British Insurers (ABI) and the British Tunnelling Society (BTS) Joint Code of Practice for the procurement, design and construction of tunnels and associated underground structures in the UK.(2003)

   Both of these documents propose that risk based management techniques are embedded into the overall project management system for all stages of a project (Chapter 6, paragraphs 6.3 and 6.4).

9 To investigate the specific construction issues arising from closed face tunnel boring machine technology, the British Tunnelling Society has published a document titled "Closed face tunnelling machines and ground stability - A Guideline for best practice" (2005). The Guideline includes the results of targeted confidential research into the number of incidents occurring world-wide associated with this tunnelling methodology, and lists a number of identified hazards together with proposals for their mitigation. (Chapter 3, paragraph 3.6).

10. The manner in which human factors can undermine management systems and result in construction failures has been highlighted. Those elements of a positive safety culture that need to be integrated into effective safety management systems have been identified (Chapter 6, paragraph 6.5)

11. A number of individual risk management systems that have been effectively employed on recent major underground projects have been identified and referenced. Where applicable, these can be adopted as exemplars and developed for future projects by those parties associated with the schemes. (Chapter 6, paragraph 6.6).

12. Six factors have been identified by the authors as having a primary influence on the quality of the overall safety management systems for underground construction projects (Chapter 7). These are:

    1. Project management

    2. Organisational, procurement and contractual arrangements

    3. Engineering systems

    4. Health and safety systems

    5. Human factors

    6. The availability and use of "enforcement" action.

1. BRIEF FROM THE HSE

The authors were commissioned to undertake the following

1. Research and prepare a list of the incidences of internationally occurring "emergency events" in urban soft ground tunnelling construction. The term "emergency event" includes tunnel collapse, tunnel lining failure, fires, floods, excessive surface settlements (such as surface craters) and significant damage to third party surface and subsurface infrastructure.

2. This list should include incidents involving bored tunnel construction methods, but not open cut construction.

3. The focus of the research is third party risks during construction, not employee or worker Health and Safety risks.

4. The report should draw upon the information obtained to draw conclusions and recommendations on generic causes of such emergency events and identify contingency measures that might be adopted in mitigation.

The term "soft ground" means ground that requires a significant level of immediate support upon excavation. The term "bored tunnel construction methods" includes methods of excavation such as hand mining, machine mining and both open face and closed face Tunnel Boring Machine excavation. Lining methods could include sprayed concrete and pre-cast segmental linings.

2. INTRODUCTION

"Tunnelling is a form of civil engineering construction, carried out in an uncertain and often hostile environment, and relying on the application of special knowledge and resources." The above quotation is taken from the introduction to the 1978 CIRIA Report [1] on improved contract practices for tunnelling and provides a succinct description of the risks and resources associated with tunnelling that remains as relevant today despite revolutionary changes in methodology and specialist equipment.

The world wide expansion in the development of civil engineering infrastructure over the subsequent three decades has led to a major increase in the numbers and types of tunnels constructed for road, railways and water supply/sewerage schemes. Over this time there has been significant innovation in the development of tunnel linings, tunnelling machines, specialist equipment, ground investigation techniques, methods of procurement and in contractual relationships between parties. These developments have been driven by the knowledge-based experience of engineers and others. Clients now desire cost effective tunnel projects to be built in ground previously considered too difficult. - and all within defined budgets and time scales.

The evolutionary process of developing innovative design and construction often involves balancing the commercial risk of new forms of construction against proven methods while maintaining control over workplace and third party risks during construction. From this experience has evolved the necessity to consider risk-based management systems into the process to ensure that a balance between risk and rewards/benefits is achieved. However a number of identified tunnel failures have led to ground and sub-surface impacts involving loss of life and damage to property resulting in substantial underwriting losses. (For example, insured and uninsured losses following the Heathrow Express tunnel collapse in1994 are estimated [2] to have been as high as £400m.). Consequently, in the UK these events have resulted in an increased awareness of societal risk issues; prosecutions for

Health and Safety offences; and in a requirement for risk based management systems for all independently insured tunnelling projects.

The aims of this study, (see Section 1), were to establish a data base of emergency events that have occurred during recent world wide tunnel soft ground construction projects and to draw out conclusions from this information. Published data was collected for all soft ground tunnel types and construction methodologies, and, where sufficient information was available, the primary and underlying causes of emergency events were pin-pointed. This information was then used to consider what recommendations might usefully be made to the delivery processes of urban soft ground tunnelling projects in order to minimise the risk of third parties being exposed to risks arising from construction work (see Section 5). The scope of the study does not consider commercial risks or other financial risks.

The approach has been to, firstly update and expand existing published data on the numbers of emergency events arising during such tunnel construction that are in the public domain; to examine, where available, the technical and management details of these emergency events; and to assess the possible direct and underlying causes of the events. Secondly, within an understanding of construction methods and societal risk, to look at the technical issues and hazards associated with urban tunnelling and identify possible ways of prevention and mitigation.

Section 3 describes the background to the research into the data on tunnel failures; Section 4 describes the concept of risk as used in this document and the societal health and safety terms used; Section 5 lists the technical issues associated with urban tunnelling; Section 6 discusses the ways in which urban tunnelling risks might be avoided or mitigated; and Section 7 lists conclusions and recommendations about the contingency actions that can be taken to avoid emergency events.

3. BACKGROUND AND RESEARCH

3.1 Size of world and UK markets

There is no centralised international data base on the number of tunnels that are constructed around the world each year. A few countries regularly publish data regarding their own tunnelling statistics, but these are not presented in a common format which would aid the collation of all international data. The UK, for example, does not publish data on its tunnelling output. In order to establish reliable information on the size and rate of growth of the world market, a number of web sites listing current and future tunnel projects were interrogated (see Appendix B). The accuracy and completeness of the available information cannot be fully verified and has to be taken at face value.

From these sources a substantial data set has been abstracted, which lists the numbers of tunnel projects that have been commenced in each of over 100 countries for the last six years, together with the lengths of tunnel constructed per annum in each of these countries (See Appendix A). The data base has also been interrogated and re-classified to include the GNP (Gross National Product, per capita, using the World Bank definition) of the countries listed and the intended usage of the tunnel projects listed (see Tables 3.1, 3.2 and 3.3).

	1999	2000	2001	2002	2003	2004
GNP>$9386	174	201	241	244	283	363
$9386>GNP. $3036	8	33	19	35	11	19
$3036>GNP>$766	27	53	51	139	37	17
GNP<$766	11	13	16	8	11	3
Total	220	300	327	426	342	402

Table 3.1 Number of world-wide tunnel starts between 1999 and 2004 by GNP (US$ per capita).

	1999	2000	2001	2002	2003	2004
GNP>$9386	801	840	1039	1080	931	1208
$9386>GNP>$3036	34	144	62	89	86	81
$3036>GNP>$766	236	408	442	404	225	163
GNP< $766	97	123	125	47	76	9
Total	1168	1515	1668	1620	1318	1461

Table 3.2 Length of world-wide tunnels starts (km) between 1999 and 2004 by GNP (US$ per capita).

Both tables show consistent increases in both the number and lengths of world wide tunnel starts over the six year period shown. The figures in the $3036>GNP>$766 category are influenced by recent tunnelling projects in China

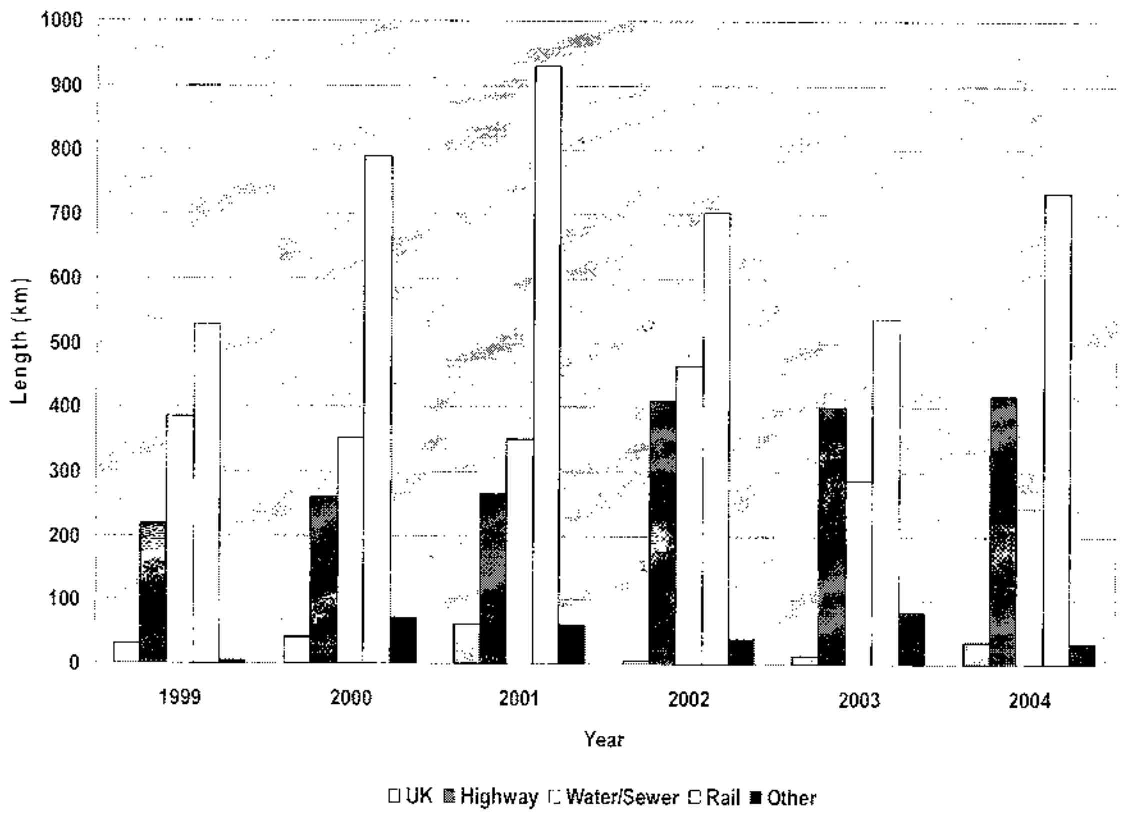

Table 3.3 World and UK tunnel construction 1999-2004. World construction divided by usage.

3.2 Soft ground urban tunnelling

The specific focus of this report is urban tunnelling and, given this objective, further research of the published literature was undertaken to identify and list the numbers of tunnel projects around the world that had experienced emergency events during such construction.

These data sets on historic projects have been divided into two main categories

- The New Austrian Tunnelling Methods (Appendix C),
- Other Tunnelling Methods (Non-NATM). (Appendix D)

Both sets of data were then interrogated to identify projects where emergency events had taken place in urban areas sited in soft ground.

3.3 Listings of recorded tunnel emergency events

The total numbers of incidents for all types of ground conditions, and environments in both NATM and non-NATM tunnels that have been obtained from the published data are shown in Table 3.4. The data has been classified into discrete decades in order to highlight trends.

	1970 - 1979	1980 - 1989	1990 - 1999	2000 – 2005(part)	Total
NATM	2	27	27	10	66
non-NATM	7	7	19	9	42
Total	9	34	46	19	108

Table 3.4 The total numbers of identified * tunnel emergency events per decade identified in this research, divided into NATM and non-NATM tunnels.

(* Note. This research does not purport to have identified all the tunnel emergency events that have actually occurred.)

From these total numbers of incidents it was possible to abstract the numbers of identified incidents relating to tunnels constructed in soft ground in urban areas. These are shown per decade in Table 3.5 and as a percentage of the total number of recorded incidents in Table 3.6.

	1970-1979	1980-1989	1990-1999	2000-2005 (part)	Total
NATM	0	9	12	4	25
non-NATM	2	2	9	6	19
Total	2	11	21	10	44

Table 3.5 The numbers of identified tunnel emergency events per decade identified in this research, occurring in soft ground urban environments, divided into NATM and non-NATM tunnels.

1970 - 1979	1980 - 1989	1990 - 1999	2000 – 2005 (part)
22%	32%	46%	53%

Table 3.6 Percentage of total tunnel emergency events occurring in soft ground urban tunnelling environments

Table 3.6 demonstrates a consistent rise per decade in the proportion of soft ground urban tunnels that have suffered emergency events when compared to the total of all tunnel emergency events world-wide.

3.4 NATM Tunnels

3.4.1 Listings of NATM Incidents

The HSE report on the Safety of NATM tunnels [3], published in 1996 following investigations after the Heathrow Express tunnel collapse, identified over 100 incidents recorded during NATM tunnelling. Table 1 of this document describes 39 significant NATM incidents and collapses throughout the world in both urban and rural environments. That information was obtained from an extensive literature search and was supplemented by other sources including private communication. Table 2 of that document classified the primary cause of each identified collapse. It was emphasised that, for a range of reasons, most incidents that occur are not reported in any "official" way and therefore the data must be considered as incomplete.

In addition to these listed incidents, the HSE report (see Para. 48 page 16) also identified that 71 incidents had been reported in 65 tunnels constructed in Japan between 1978 and 1991 at unspecified locations. These ground collapses ranged from the "quite small" through volumes of between 50 – 500m^3 of ground (15 Incidents.) to volumes of over 1000m^3 of ground (3 Incidents.).

Additional research undertaken since 1996, and for this study, has extended the HSE 1996 list of NATM incidents from 39 to 66 including two incidents that occurred in early 2005. In 1996 a document was published that uses the term" Sprayed Concrete Linings (SCL)" approach [4] but in this document the term NATM is used throughout.

3.4.2 Data from NATM Incidents

A feature of the data obtained for the 66 NATM tunnel incidents was the high number of failures that were recorded in the area between the tunnel face and the first completed ring of the sprayed concrete lining. These types of failure were classified as Type A in the 1996 HSE Report, and this classification has been repeated here.

Of the total number of 66 incidents, 44 have a reported primary cause, and from those 44, 39 are of the Type A.

Of the total number of 25 incidents in soft ground urban tunnelling, 22 have a reported cause of which 20 are of the Type A. Of the total number of 22 with a reported cause, 12 of those incidents also reported the incidence of a surface crater.

Other features of the 66 incidents include

- the overall ratio of urban incidents to rural incidents as was reported in Ref.3 (Para. 47) remains at 2:1, and

- The incidences of craters in urban areas also reported high level of consequences in terms of disruption to the construction work and to third parties.

3.5 non- NATM Tunnels

A list of identified incidents that have taken place since 1970 on Non-NATM tunnels in all types of ground and environments has been researched, and totals 42 emergency events (see Appendix D). As before, this list has been examined to extract details of those urban projects that were constructed in soft ground.

The 42 incidents demonstrate a wider range of primary causation and consequence than the NATM data. This is demonstrated in the following breakdown and analysis of the information from 32 of the most reliable data sets. The list of causes for all cases and for urban soft ground cases are as follows:

Cause	Number of Incidents All	Urban Soft ground
Unpredicted Ground	**20** (63%)	**10** (52%)
Fire	**3** (10%)	**2** (11%)
Compressed air explosion	**2** (6%)	**1** (5%)
Methane gas explosion	**1** (3%)	**1**(5%)
Defective Workmanship	**2** (6%)	**2** (11%)
Temporary works failure	**2** (6%)	**2** (11%)
Flood	**1** (3%)	**0**
Unknown, (despite investigation)	**1** (3%)	**1** (5%)
Total	**32**	**19**

Table 3.7 Causes of non-NATM incidents and numbers in soft ground.

Half of the above incidents have been recorded as being caused by unpredicted ground conditions. In general terms, unpredicted ground conditions may arise from poor or inadequate site or ground investigations. (See also Reference 3 Para. 57).

In these non-NATM emergency events there are few recorded instances of collapses at the tunnel face. Within the 19 incidents listed above there were 8 instances of surface craters.

A further feature of the data set of 32 incidents is that they are randomly distributed in terms of country of location.

3.6 British Tunnelling Society Working Group. Guideline Document (2005)

During construction of the Channel Tunnel Rail Link tunnels by closed face Tunnel Boring Machine in February 2003, an emergency event took place in Lavender Street in East London. [5] This incident followed two earlier events on UK tunnel projects at Portsmouth [6] and Hull [7], and reflected further incidents that had taken place in other countries (Shirlaw et al [8]) while using closed face tunnel boring machines. In some cases these incidents had resulted in craters appearing in the ground surface above the tunnel. In response, the British Tunnelling Society decided to set up a Working Group to research the details of similar incidents that could be identified and draw conclusions. Their final document, "Closed-Faced Tunnelling Machines and Ground Stability - A guideline for best practice", [9] was prepared and published in April 2005.

This Guideline (see Section 4- Incident data) brought together information on over 100 incidents which have taken place world wide during construction involving closed face tunnel boring machines. Included within their total of 100 incidents were data on 14 incidents associated with Slurry Type Machines, and 47 incidents relating to Earth Pressure Balance Machines. The BTS incidents are not specifically identified, but at least three of the incidents that are included in the BTS document (i.e. Hull, Portsmouth and Lavender Street) have also been included in the non-NATM list summarised in this report.

The Guideline identifies a number of hazards that are associated with the systems and technologies applied within these particular tunnelling methods, and propose ways to mitigate these hazards through additional works and the application of good practice (see Guideline, Section 10 "Conclusions and Recommendations").

3.7 Other relevant data sources

The world-wide Insurance industry has data on the numbers of tunnel projects that are insured and the quantum and causation of claims arising on those projects. This information is not in the public domain. However, two recently published papers (Hecke et al [2] and Blückert et al [10]) analysed the causes of tunnel construction claims. Hecke reports on a study by Munich Re of 107 important tunnel claims made between 1993 and 2003 which stated that the losses arose as follows:-

- 10% from fire
- 50 % from "natural events"
- 25% from "construction methods"
- 10% from "design defaults"

(5% were unassigned)

The paper by Blückert et al entitled "Possible Maximum Loss – Assessment of Civil Engineering Projects" provides further information under the headings "Excavation and Lining Sensitivity Factors" (See sections 6.1.6 and 6.1.7 of their paper). These are reproduced as Tables 3.8 and 3.9 below. Their definition of "Possible Maximum Loss" - used by property and fire insurers, is "the largest loss that may be expected from a single fire (or other peril when another peril may be the controlling factor) equal to any given risk when the most unfavourable circumstances are more or less exceptionally combined and when, as a consequence, the fire is unsatisfactorily fought against and therefore is only stopped by impassable obstacles or lack of sustenance." These factors provide an estimate of the sensitivity of each type of technical risk for the assumed most unfavourable hazards from which the maximum damage can be assessed. It is assumed that most projects will have enough in common with these Factors for them to taken as a tenable basis for estimates.

Excavation Method	Sensitivity Factors ★					
	Earthquake	Flood (external)	Flood (at face)	Fire	Explosion	Face collapse, inadequacies of method for unforeseen conditions
1. Conventional	3	2	3	2	2	3
2. Manual	3	1	3	2	2	3
3.Drill and Blast	3	1	3	2	1	3
4. Open cut	2	2	2	1	1	2
5. Open face shield	3	2	3	2	2	3
6.Closed face shield	3	2	2	2	2	2
7. Slurry TBM	3	2	1	2	3	2
8. EPBM	3	2	1	2	2	2
9. Full face hard rock TBM- No shield	3	2	3	2	2	3
10. Hybrid TBM	3	2	3	2	2	2
11. Micro Tunnelling	3	2	3	3	3	2
12. Submerged tube.	3	3		2	2	3

★<u>Severity Factors</u>
0 - Excavation method unaffected
1- Minor influence on method-drive method can be maintained once hazard relieved
2- Major failure of face or drive -may require alternative working method
3- Catastrophic failure of face, possible abandonment of drive.

Table 3.8 Excavation Sensitivity Factors (Source, Blűckert et al, Reference 10)

Tunnel Support & Lining Systems (fully installed)	Sensitivity Factors ★				
	Earthquake	Flood	Fire	Explosion	Inadequate design or method of execution: lining failure/collapse
1. Pre-cast segmental linings	2	2	1	2	3
2. Jacked concrete pipes	2	2	1	2	3
3. Steel ribs & Lattice arches (NATM)	2	1	1	2	3
4. Rock bolts (NATM)	2	0	0	0	3
5. Shotcrete, also NATM	3	1	2	2	3
6. In-situ concrete linings (inc. NATM)	3	1	1	2	3
7. Compressed air (face support ×)	2	2	3	3	3
8. Ribs and lagging	2	2	3	3	3
9. Contiguous piles (open cut)	2	0	0	0	2

★Severity Factors

0- Lining or support method unlikely to suffer damage

1- Minor damage (or localised) to lining or support system -can be repaired.

2- Significant damage to support systems or lining-may require alternative working method for repair

3- Catastrophic failure of tunnel, possible abandonment

× Compressed air only used for face support, in combination with other lining methods.

Table 3.9 Lining Sensitivity Factors (Source, Blűckert et al, Reference 10)

3.8 Analysis of Data

The Authors believe that the following conclusions may be drawn from a broad analysis of the data contained within this document.

1. The numbers of tunnels being constructed in the developed world is increasing year on year, while activity in the rest of the world remains variable and an order of magnitude below the developed countries.

2. Both the total number of tunnel emergency events, and the proportion of these incidents that took place in urban areas with soft ground tunnels have increased in the three decades since 1970. In the latter case this trend has also continued into this current part decade.

3. The incidents associated with soft ground urban tunnels show that NATM tunnels exhibit different failure characteristics from Non-NATM tunnels.

   - Approximately 90% of the NATM tunnel incidents occurred within the uncompleted structure close to the face. Of these approximately 55% result in a surface crater.

   - Just fewer than 50% of non-NATM incidents reported resulted in a surface crater, the majority being located behind the tunnel face. Of these incidents, approximately 50% took place in shielded/TBM drives and approx. 50% in hand drives.

   - There is evidence that tunnelling with closed face TBM's can result in surface craters above the face of the machine. A direct relationship between face incidents and the occurrence of surface craters is not clear.

4 The significant numbers of the identified NATM incidents recorded in Germany/Austria can perhaps be put down to-

- A desire to release technical information in order to disseminate lessons learned to assist in the development of this particular method.

- An approach to the design and construction of tunnels [11] that seek to refine the balance between risk and rewards systems[12].

5. The primary cause listed in the emergency events is attributed to unpredicted ground conditions. This matches a finding in the 1996 HSE Report.

6. In the non-NATM tunnels, the range of primary causes for incidents not attributed to ground conditions is wider than for NATM tunnels. This points to a more complex chain of causation.

7. The underlying causes of emergency events are likely to be diverse. They could involve engineering, management, procurement, organisational, competence, resource or communications issues.

4. THIRD PARTY RISK

4.1 General

4.1.1 Societal Risk.

A paper by Hambly and Hambly[13] asserted that the Fatality Accident Rate, FAR, (defined by them as the risk of death per 100 million hours exposure to the activity) is greater for construction work than for industrial work as a whole, which would confirm why the construction industry has a poor reputation for accidents. Their figures also show however that some other industries (oil and gas, offshore) have higher FAR (See their Table 1), and that the fatality accident rate for construction workers is lower than the rate for all men of working age. These figures apply to workers within industry rather than third parties affected by industry, and they also show that the FAR to third parties from a collapsing building is extremely small.

In addition, it is the view of insurers when evaluating Possible Maximum Loss [10] that "whilst every segment of construction has to contend with particular hazards and associated risks, there is none which compares to the range and exposures plaguing the tunnelling industry"

It is clear that from the point of view of personal accident statistics and commercial liability that some people may consider tunnelling to be "high risk", albeit to those working in this industry rather than as risk to the general public. There are exceptions however, in particular an incident on 24 April 1995 in Taegu South Korea [14] during which a gas explosion took place within the underground construction works of a new metro station. This one incident resulted in the death of more than one hundred members of the public including 50 children.

When assessing the magnitude and extent of risk it is important to clarify the definition of the terms being used. This is because the terminology used can be interpreted in various ways by different groups It might be argued that the consequence of tunnelling induced hazards impacting upon third parties is a societal risk, but one that is not necessarily restricted to accidents. The HSE in

"Generic terms and concepts in assessment and regulation of industrial risks" 1995 [15] defined societal risk as-

"The risk of widespread or large scale detriment from the realisation of a defined hazard, the implication being that the consequence would be on such a scale as to provide a socio/political response, and /or that the risk (i.e. the chance combined with the consequence) provokes public discussion and is effectively regulated by society as a whole through political processes and regulatory mechanisms."

"Societal Risks" by Ball et al [16] also characterises societal risks that derive from the public's normal activities being interrupted by an accident as "societal concerns" with the assessment criteria stated as "political judgement-possibly aided by multi-criteria techniques". This would imply that quantitative cost benefit analyses based on "value for a life" calculations are not appropriate. An "as low as reasonably practicable" (ALARP) approach allied to judgements on other relevant issues is likely to be the most practicable way of evaluating societal risk.

Within this document the terminology that is used to describe risk and related terms is defined below.

4.1.2 Definitions

Hazard * is something with the potential to causing harm

Risk * is the combination of the likelihood/probability that an identified hazard will occur and the consequence/severity if it did.

* see paragraph. 11(b) of Management of Health and Safety at Work Regulations 1999 Approved Code of Practice.[17]

4.2 Risk Evaluation

4.2.1 Risk Assessment.

Risk Assessment is a part of the overall process of Risk Management. Risk Management starts from initial hazard identification through to the effective control of risks during the use phase of the project. Risk assessment is that stage where residual risks are evaluated in terms of their likelihood and consequence for the purpose of setting priorities and in terms of what can be done to devise effective preventative and protective measures. Risk assessment is not an end in itself, but a means to the end objective of a safe and healthy working environment both during construction work and during the phases thereafter.

In order to identify, understand and control the risks to third parties arising from a tunnelling project, risk assessments should be undertaken to aid decision making on protection and preventative measures.

4.2.2 Acceptability criteria

A risk assessment can be undertaken using either qualitative judgements or quantitative values. As discussed above, it is impracticable to assess societal risk for acceptability against quantitative criteria for two reasons (a) because of the lack of reliable data, and (b) because of the diversity of each tunnel's characteristics (i.e. size, purpose, tunnelling mediums, design and construction details etc.), and in particular the excavation and lining techniques employed.

However, qualitative acceptability criteria are suggested in the ITA Guidelines for risk management [18] and these should be considered. These are as below;

Unacceptable	This risk shall be reduced at least to "Unwanted" regardless of the costs of mitigation.
Unwanted	Risk mitigation measures shall be identified. The measures shall be implemented as long as the costs of the measures are not disproportionate with the risk reduction obtained (ALARP principle)

Acceptable	The *risk** shall be managed throughout the project. Consideration of risk mitigation is not required.
Negligible	No further consideration of the *risk** is needed.

(* amended from the term "hazard" by the authors of this document).

4.3 Consequence Analysis

For urban tunnel projects, where damage and disruption on a local scale can result in high impacts, the consequences of emergency events can include-

- Death or Injury to third parties
- Damage or economic loss to third party property or services
- Harm to the environment such as release of toxic/inflammable/harmful materials
- Delay and consequent economic loss to third parties
- Loss of public goodwill which might lead to political action

Any significant incidents occurring in urban areas would be politically, economically and environmentally sensitive, and public opinion and actions could be expected to play a significant role. Loss of public goodwill, arising from emergency events affecting third parties, has the potential to result in adverse media coverage.

Third party injuries and damage arising from emergency events will be controlled by factors such as impact, location and chance. For example, craters opening in rural areas are likely to be less damaging than in urban areas. Similarly, time of day will affect the numbers of third parties exposed to an incident. This randomness, or chance, is an important factor in the consequence of any event.

Consequences arising from an emergency event have been quantified and graded in the ITA Guidelines as follows

Class°	Consequence				
	Injury to third parties	**Damage to third party**	**Harm to the Environment**	**Possible Delays**	
				Minimum	**Other**
	Numbers of fatalities/injuries	Loss in Million Euros	Guidelines for proportions of damage	Months per hazard	Months per hazard-
Disastrous	F>1,SI>10	>3	Permanent severe damage	>10	>24
Severe	1F,1<SI<10	0.3-3	Permanent minor damage	1-10	6-24
Serious	1SI,1,MI<10	0.03-0.3	Long-term effects	0.1-1	2-6
Considerable	1MI	0.003-0.03	Temporary severe damage	0.01-0.1	0.5-2
Insignificant	-	<0.003	Temporary minor damage	<0.01	<0.5

Table 4.1 Classification of consequence. Source ITA Guidelines, Ref. 18. Based on an underground project with a value of €1Bn. and duration of 5-7 years. (Where, F = Fatality, SI = Serious Injury, MI = Minor Injury.)

(° N.B. The data collected in this report is based upon "emergency events", which are defined as events that required contingency measures to be activated.)

In 1995 there was an incident on Hollywood Boulevard, Los Angeles when re-mining a completed tunnel caused a crater 70 foot wide and 60 foot deep which resulted in the complete closure of the famous Boulevard.[19] No one was hurt in the incident, but a $1bn.law suit was brought against the contractor.

The consequences of the three tunnel collapse at Heathrow airport in October 1994 [20] included the temporary closure of the underground Piccadilly Line, disruption to tens of thousands of travellers, road closures, extensive remedial works and the cost of lengthy investigations, redesign cost and legal costs involved in the subsequent prosecutions.

4.4 Frequency Analysis

The ITA has prepared the following classification of frequency from a review of statistics and from pooled expert judgement.

Frequency class	**Frequency Interval**	**Central value***	**Descriptive frequency class**
5	>0.3	1	Very likely
4	0.03 -0.3	0.1	Likely
3	0.003-0.03	0.01	Occasional
2	0.0003-0.003	0.001	Unlikely
1	<0.0003	0.0001	Very Unlikely

Table 4.2 Frequency of events in the construction period. (Source: Ref 18).

(NB. The central value represents the logarithmic mean value of the given interval)

The five frequency classes presented represent a way of identifying an event frequency which enables the number of emergency events to be described in terms of per year or per kilometre of tunnel construction. However the ITA recommendation is for such frequencies to be related to the number of events per project construction period.

4.5 Risk Matrix

Based on the above ITA definitions and criteria a risk assessment can be undertaken to produce a risk matrix for frequency and consequence.
An example of a risk matrix is given below.

	Frequency				
Consequ-ence	**Very Likely**	**Likely**	**Occasional**	**Unlikely**	**Very unlikely**
Disastrous	Unacceptable	Unacceptable	Unacceptable	Unwanted	Unwanted*
Severe	Unacceptable	Unacceptable	Unwanted	Unwanted	Acceptable
Serious	Unacceptable	Unwanted	Unwanted	Acceptable	Acceptable
Consider-able	Unwanted	Unwanted	Acceptable	Acceptable	Negligible
Insignificant	Unwanted	Acceptable	Acceptable	Negligible	Negligible

Table 4.3 Risk acceptance matrix (Source ITA Guidelines, Table 8)

* Special attention needs to be given to the case of a potentially low or very low probability events but which could have high consequences. In the above table this is labelled "unwanted", but, in the authors' opinion, the correct classification should be "unacceptable". The report on the collapses at Heathrow [20] and the publicity associated with the events [21] illustrates this issue.

5. URBAN TUNNELLING – THIRD PARTY IMPACTS

5.1 General

Modern cities depend on underground infrastructure to provide support for their commercial and domestic needs. The age, extent and nature of these works in a particular location are a function of local geology, economic growth and technological development. London, for example, with its advantage of a widely distributed geological material of good tunnelling properties- London clay-has a long modern tradition of tunnelling and infrastructure provision. Other cities, such as Hong Kong and Singapore, have developed their underground infrastructure more recently by taking advantage of technological advances in tunnelling equipment to overcome less advantageous ground conditions.

Construction and maintenance of such existing urban tunnels have provided a rich source of information, detailing both the problems and the solutions thrown up during the life cycle of these works. The problems arising from a variety of different hazards associated with urban conditions and constraints provide a set of data that must be fully understood by designers and constructors of new urban tunnelling schemes. A recent paper by Kovári and Ramoni [22] provides a useful summary of the tunnelling risks encountered in an urban environment.

Kovári and Romani cite some examples of the range of direct impacts that might affect third parties from urban tunnel construction, including:

- Failure of tunnel support leading to the formation of a surface crater
- Excessive settlement leading to damaged buildings
- Settlement movement damage to above and below utilities

Specific examples each of these types of impacts, and others, are provided in Section 5.4 below.

5.2 Constraints

5.2.1 Geology

The principle engineering determinant in the design and construction of urban tunnels, as in all tunnels, is the ground conditions and the construction methods that are required to realise the works. Tunnels built in sound rock, as in Stockholm, can be formed of large caverns with little or no added support, allowing spacious architectural designs for transportation systems. Cities founded on soft soils with high water tables require tunnels whose form and support are dependent upon excavation methodologies determined by the need for both immediate support and continuing support of the ground. In the latter case, variation of the anticipated ground provides the largest single risk to the tunnelling works together with the potential to affect third parties. The point is well summarised in CIRIA Report 79 [1], which states:

"The ground is the principal construction material, supplied by nature and seldom to specifications that engineers would choose. Methods of construction are highly dependent on the ground, and costs are a function of the rate of advance. An encounter with unforeseen ground conditions not only imposes large extra costs but may itself introduce additional hazards. Although similar circumstances may attend other forms of heavy construction, it is unusual for the difficulties to be so severe or the effects so acutely felt as in underground construction, where difficulties are unavoidably compounded by the limited and restricted access to working faces."

A large amount of international urban tunnelling is carried out in soft ground, which can be characterised as non uniform, infinitely variable, contain variable water levels and perhaps incorporating man-made features of varying age. For new tunnel schemes it is essential that the prevailing ground conditions including their possible variations are well understood at an early stage of planning, so that subsequent decisions on layout, design and construction methodology can be based on a thorough and detailed understanding of the potential hazards. This requires that early and comprehensive site and ground investigations are expertly planned, carried out and reported.

Interpretation of the factual ground investigations should also be undertaken and the findings set out in a separate document. Depending on procurement strategy, this document might, or might not be issued with the construction contract documentation but, in any event all information relevant to health and safety considerations will be gathered together and made available to designers and others. In view of the number and variety of risks associated with urban tunnelling, the authors believe that Interpretative reports should be made available at tender stage as either Ground Reference Conditions[1] or Geological Baseline Reports [23].

The quality of the information and the opinions expressed in these reports is crucial to those whose task is to make informed decisions on design and construction methodology issues. Site investigations should include geological studies, desk studies, historical and archaeological research, examination of aerial photographs to determine the extent of man made obstructions (see BS 5930) [24]. In addition archived records of wells, ground anchors, basements, records of previous excavations nearby, and even unexploded ordnance need to be researched.

It has been reported [1] that site investigation costs have been found to range between 0.1 to 7.5% of the capital tunnelling costs. Also reported [25] are the results of a 1984 study of 89 underground projects in USA that compared the cost input to site investigations and the out-turn costs of the works. The conclusions of the study were

- Overall, contractual claims averaged 29% of the engineer's original cost estimate for the works

- Low site investigation effort (i.e. cost) resulted in an increase in the level of claims

- When ground investigation effort exceeded 0.6metre length of borehole per metre of tunnel length, there was a pronounced reduction in the cost of contractual claims, and the contractual claims continued to reduce in cost as the level of exploration increased.

If the level of claims is equated to the level of unforeseen ground conditions, the relationship between level of ground investigation and level of unforeseen ground risk is identified. The limit of 0.6 metres of borehole length per metre of tunnel is roughly equivalent to an exploration cost of 1% to 1.5% of project cost depending on the relative amounts of cable percussion or rotary drilling included in the investigations. Site investigations and their costs are likely to be particularly project specific.

5.2.2 Structures on the ground surface

All bored soft ground tunnelling causes relaxation in the medium being excavated (described as ground loss) together with associated ground movements around the tunnel. In turn these below ground movements translate themselves into deformations at the ground surface that vary in magnitude with distance from the tunnel centre line. Infrastructure sited close to the line of the tunnel will be distorted by the resultant "settlement trough" and may be damaged (to varying degrees) by the differential movements placed upon it. In London clay for example, the degree of damage will be dependent on a range of factors and can be predicted with a high level of accuracy [26], thus enabling appropriate mitigation and monitoring to be installed in most cases.

Building damage due to settlements arising from remote tunnelling can, in general, also be predicted with some accuracy. As a consequence the effects of settlement can be determined and mitigated through either variation of vertical and horizontal tunnel alignment, or partly by site specific measures such as underpinning, compensation grouting etc.

Tunnelling induced ground movements will also affect other sub-surface structures such as piles, deep basements, other tunnels and underground structures. As with surface buildings the movements and reactions can be calculated and, where necessary, appropriate mitigation measures introduced. It is usual in these circumstances for tunnel project teams to liaise with engineers representing third parties to ensure due process in the analysis and design of mitigation of the applied forces and movements. Where the impacted tunnels are used by the public additional risk assessments are normally required.

There are recent examples of emergency events that have taken place at tunnel level that have led to the formation of surface craters (Barcelona[27] and Shanghai [28]) These resulted in severe damage to buildings and major disruption to surface activities including road closures and evacuation and demolition of buildings.

5.2.3 Shallow tunnels

Where tunnels have ground cover that is less than twice their diameter, particular difficulties can arise from impacts and clearances to physical obstructions such as foundations, basements, deep utilities etc. In addition, a shallow tunnel may result in a greater magnitude of surface settlement than a deep one, albeit over a narrower settlement trough. In urban areas such configurations can cause severe engineering problems in congested zones. An additional issue, that can cause cumulative problems to the above, is the uniformity of the ground cover. Surface layers can include irregular man-made ground and softer deposits that can result in local distortions to the settlement profile and *in extremis* loss of ground into the tunnel. The Athens Metro [29] construction provides an example of the many problems than can arise if inappropriate equipment is used. The recent tunnel collapse at Lausanne [30] appears also to be a similar case.

5.2.4 Utilities

All buried utilities, whether shallow or at depth, will be affected by settlements arising from tunnel construction. Culverts and large pipes, such as sewerage, water or gas mains, may be distorted by the ground movements and have additional loadings placed on their joints and barrels. Particular problems can arise at the connections between linear pipelines and fixed structures such as buildings, manholes etc. Careful analysis is required in all cases to ensure that the pipes, taking account of their material and physical condition in the ground, can sustain foreseeable imposed loadings and movements. Fractured gas and water mains have the potential to cause major disruption and serious or significant damage to third party interests. Other utilities such as telecommunications networks can be similarly damaged from settlements which can result in outage to important systems and cause major third party disruption and result in significant commercial claims. It is good practice safely to divert sensitive utilities prior to tunnel construction, as unexpected movements can endanger plant that might previously have been thought to be at low risk.

5.2.5 Working sites

Tunnel construction sites located in urban areas can suffer from restrictions arising from land availability; access limitations and environmental restraints imposed local Authorities. Their uses can include any of the following functions: permanent works construction, temporary works for access to the permanent works, materials handling and staff accommodation and welfare facilities. All of these activities will impact on third parties close to the site and for which environmental protection will be provided by agreed Site Environmental Construction Plans.

Risks to third parties from these uses can arise from instability of the temporary and permanent works, which by their nature, are close to existing infrastructure. Other risks include the staging of traffic management controls including diversions that can be required to release parts of the working site to enable phased construction.

Restricted sites can result in limitations in the provision of plant and materials which could result in an increased construction risk and which should be taken fully into account in the design, planning, permissioning and construction stages. Satellite or remote working sites such as for the provision of permanent works shafts and temporary works shafts for ground treatment are also likely to suffer from restricted access issues, and hence the possibility of increased third party risks.

5.3 Potential Constraints

The available references on all the incidents that have been identified in this study provide a plethora of hazards leading to the emergency events. In addition to these there are other potential hazards that might arise specifically from urban tunnelling, modern construction methods and materials and other sources that have not been ascribed to third party risks in the literature. Some of these potential hazards have been identified in the British Tunnelling Society Report on Closed Faced Tunnel Boring Machines and listed in that document for review – see Reference 9.

Other processes with the potential to introduce hazards to the construction of urban tunnels in respect of third parties might include the following:

5.3.1 Compensation grouting.

This is a method of injecting grout into the ground above an advancing tunnel face in order to compensate for anticipated associated ground loss and thus reduce settlement. The process requires the development of very high grouting pressures in the ground and has the potential, if improperly designed, controlled and monitored, to damage the excavation/support at tunnel level leading to local loss of ground and possibly significant surface settlements. There is also the potential to cause heave at the ground surface with consequential damage to utilities and buildings.

5.3.2 Ground water lowering.

This is a traditional technique that has been adopted on a number of recent projects to ease the progress of closed face tunnel boring machines by reducing general ground water levels and in particular the water pressure in the ground at the tunnel face. Its application over a wide area of urban landscape is capable of causing secondary effects such as irregular surface settlement and local piping of the ground.

5.3.3 Tunnel induced disturbance.

Incidents of unexpected near-surface settlement were recorded during construction of the Jubilee Line Extension.[31, 32] The movements took place in near surface alluvium and affected overlying railway structures. They have been interpreted to be caused by the tunnelling process introducing meta-stability (sensitivity to slight disturbance) of the ground above the tunnel. In similar ground conditions (which are extensive in East London) it is possible that tunnelling could cause settlement damage to buried utilities such as water or gas mains leading to loss of water/ground or explosions. Such a scenario has been accepted in a judgement given in recent Court proceedings[33].

5.3.4 Compressed Air.

A form of temporary works that is used to control the ingress of water and ground into the works during open face tunnelling, and also to limit ground movements during construction. Cases in Tokyo [34] and London [35] demonstrate the risk to third parties from a sudden loss of air pressure resulting in lining failure and the sudden formation of a surface crater. Advice on the design and control of compressed air installations for tunnels is provided in BS 6164 [36] and the Work in Compressed Air Regulations 1996 [37].

5.4 Problems/ Lessons from recent incidents

5.4.1 Open face working

NATM tunnel construction incidents in the city centres of Barcelona [27] and Lausanne [30] in early 2005 resulted in the appearance of large surface craters and consequent significant third party disruption. In Barcelona emergency evacuation of a number of buildings was ordered, with some being subsequently demolished. In both instances open face tunnelling methods were being employed when unforeseen ground conditions were met and miners and others were unable to prevent face collapse and the consequent ravelling of ground to the surface. These cases highlight the necessity for a good prior understanding of the ground and for thorough contingency and emergency pre-planning.

It is also possible for ground conditions to change rapidly over short tunnel advances and this can result in sudden unstable open tunnel faces. In the case of shallow tunnels, like Lausanne, this instability can last for a very short time-span and there may not be sufficient time available to install effective emergency ground support systems. With open-face tunnelling methods the safety-critical feature is the ability of the face to remain in a stable condition during the whole construction sequence. This has been the subject of research spanning 25 years (Lunardi 2000) and has led to the development of an engineering approach which is titled "Analysis of controlled deformations of rock and soil"[38]. In simple terms, analysis of this type of ground behaviour leads to the solution of stability problems by means of confinement of the existing void and/or the pre-confinement of the block of ground ahead of the tunnel face. Professor Lunardi states that this is a fully designed solution based on an engineering characterisation of the ground. It is not an "observational method" of tunnelling. The range of confinement and/or pre-confinement options are so extensive that the method of tunnel construction remains flexible, but the main principle is that the soil ahead of the tunnel face is stabilised before it can move and present a danger.

On the other hand, the Madrid City Authorities have rejected open face methods for tunnelling through soft ground, except under very carefully controlled conditions. (Melis *et al* 2000) [39]

5.4.2 Closed face working

Recent examples of third party impacts from closed faced working can be found in Singapore [40] and Stratford [5] in east London. In both cases surface craters above bored tunnel works resulted at the least in some social disruption, either to domestic householders or to road users. The consequence of these incidents to third parties and to the existing infrastructure was determined in some measure by route planning but also to chance. These cases are included in the data bases researched by the BTS closed face working group and outlined in their Guideline. Both cases demonstrate the need to fully understand the ground conditions both at the closed face and further ahead of the face. The TBM operator has to be guided by both direct and indirect monitoring systems in the quantification of spoil removed per ring advance and the avoidance of over-excavation, as the cutting process is enclosed and therefore unseen. Obstructions or voids in the ground can be driven through unknowingly if waste material is not visible and accurate spoil measurements are not readily available.

The recommendations in the BTS Guideline include:

- A holistic approach to safety issues.
- Wide dissemination of details of the expected ground conditions within the project team.
- Active management of TBM functions to control surface settlements.
- Maintenance of operational face pressures above the hydrostatic head, checked by continuous monitoring.
- The accurate monitoring of excavation quantities.
- Early grouting of the ring annulus and monitoring for pressure and volume.
- Sufficient, controlled and tested spoil conditioning.
- Setting and real time monitoring of the operating parameters of the TBM.
- Adoption of risk management systems for design, procurement and operation of TBMs.

- Extensive site investigations, including geophysical methods where applicable, to be undertaken at design and construction stages.

- Monitoring of surface and sub-surface structures.

- Good record keeping.

- Use of experienced staff.

5.4.3 Temporary Works

The design and detailing of temporary works can be more complex than for the permanent works themselves. The abnormal loading patterns involved, and the temporary nature of the supports, demand a high level of skill and competence in effecting suitable arrangements. The detailing of primary linings in NATM construction is a case in point.

An example of a temporary support failure in an urban environment is provided by the Los Angeles Metro[41] where a serious underground fire led to the destruction of the timber laggings between the steel ribs and resulted in a collapse of the lining.

Another incident, on the Docklands Light Railway, resulted from the application of internal compressed air pressure to a completed section of running tunnel in order to construct a breakout cross passage beneath the river Thames. In this case the tunnel lining failed following the application of a test pressure and loss of ground cover which resulted in an explosion and surface crater adjacent to a nearby school [35].

Ground Treatment to aid tunnel construction and reduce settlement has also provided examples of disruption and third party impacts. On the Heathrow Express site at the Central Terminal Area [20], jack grouting was employed to rectify building settlements during tunnel construction. In the event the grouting caused a failure in the primary tunnel lining which in turn led to the ultimate collapse of both the lining and the building. In another case, this time in Sweden, albeit in a rural location, grout was injected into the ground ahead of an advancing tunnel face in order to create a low permeability zone around the tunnel and reduce water ingress to the tunnel to an acceptable level. One of the chemical grouts used and which was most effective in reducing water flow, was subsequently found to be toxic, having initially affected miners and later some cows that drank water pumped out of the tunnel works [42]

5.5 Future Projects in London

A number of transportation and infrastructure projects are currently at an advanced stage of planning within the London area. These include railway tunnels in the central area and collector sewers beneath the Thames corridor. It is anticipated that a full range of urban engineering and environmental issues will be met by these works and that construction planning will need to be very detailed.

Appropriate construction methodologies within the deposits of the London basin have already been developed and successfully used over a number of past and recent projects. The technology is therefore available to enable safe and economical tunnelling to be undertaken within all the generally anticipated ground conditions. Planning must ensure that sufficient knowledge of local ground conditions and existing infrastructure is available to enable informed decisions to be made during the design and construction of the works.

There is no reason why these projects cannot be safely and successfully completed.

6. RISK REDUCTION FOR URBAN TUNNELLING

6.1 UK Legislation

The UK legislation that covers the general avoidance of risks to third parties arising out of any work activities is contained in Section 3 of the Health and Safety at Work Act 1974. [43] There is a general duty on employers and the self-employed to "conduct their undertaking in such a way as to ensure, so far as is reasonably practicable, that persons not in his employment are not exposed to risks to their health and safety". An "undertaking" includes not only employers undertaking the construction work but also those providing a service such as a design practice. Because the wording of this Section includes the phrase "so far as is reasonably practicable", the burden of proof is reversed in any court proceedings as a result of the effect of Section 40 of the 1974 Act.

Regulation 3 of the Management of Health and Safety at Work Regulations 1999 [44] requires every employer and every self-employed person to make a "suitable and sufficient assessment of the risks to the health and safety of persons not in his employment arising out of or in connection with the conduct of his undertaking". These risk assessments are "for the purpose of identifying the measures he needs to take to comply with the requirements and prohibitions imposed on him by or under the relevant statutory provisions". Regulation 4 adds that "where an employer implements any preventive and protective measures he shall do so on the basis of the principles specified in Schedule 1 to these Regulations" – the Risk Hierarchy.

Regulation 13(2) (a) (i) of the Construction (Design and Management) Regulations 1994 [45] places a duty on "designers" (as defined by these Regulations), to "give adequate regard to the need to avoid foreseeable risks to the health and safety of any person who may be affected by the (construction) work". At the time of writing the Health and Safety Executive has issued a Consultation Document with the proposed text of the Construction (Design and Management) Regulations 2006 [46]. This text significantly alters the 1994 designer's duties, and the duty to manage risks to third parties is contained in the new proposed Regulation 14(3). In the Consultative Document the text of this Regulation reads (in part) as follows –

Every designer shall in preparing or modifying a design which may be used in construction work in the United Kingdom avoid risks to the health and safety of any person liable to be affected by such construction work.

The Consultative Document includes a new proposed general duty on the Principal Contractor to "plan, manage and monitor the construction phase in a way which ensures that, so far as is reasonably practicable, it is carried out without risks to health and safety".

Within the Construction (Health, Safety and Welfare) Regulations 1996 [47] there are several duties placed on defined parties to construction work to prevent danger or harm to "any person" – which would include persons not party to the construction works. For example, see Regulations 9(1), 12(1), 12(2), 12(3), 28.

6.2 Controlling Tunnelling Risk

The number of prominent tunnel failures that took place in the 1990's led to pressures on the tunnelling industry from regulatory bodies and project Insurers to address project risk in a formal and uniform manner. Different client authorities developed their own methodologies for incorporating a risk based processes into the decision making systems serving their management teams, but these were ad hoc and did not rely on common methodologies and criteria. The International Tunnelling Association (ITA) therefore set up a working group to report on these systems and make proposals for a unified approach. This was achieved in 2004 with the publication of their "Guidelines for tunnelling risk management". [18]

At the same time the global insurance industry represented by the Association of British Insurers (ABI) and the British Tunnelling Society (BTS) began working together to prepare a Joint Code of practice [48] for risk management for tunnel works in the United Kingdom. Compliance with this code is required to secure project insurance for "Contractors All Risks" and "Third Party" Insurances. This document was issued in the UK in 2003 "for active implementation". The UK document is the forerunner for an International Code which is currently under preparation.

6.3 ITA Guidelines (2004)

These Guidelines were prepared by Working Group 2 of ITA "*to give guidance to all those who have the job of preparing the overall scheme for the identification and management of risks in tunnelling and underground projects. The guidelines provide owners and Consultants with what is modern-day industry practice for risk assessment, and describes the stages of risk management throughout the entire project implementation from concept to start of operation*".

The ITA has provided no statements with regard to compliance with their Guidelines.

The ITA Guidelines set out how tunnelling risk management systems can be used throughout three stages of a project

- Early Design
- Tendering and Contract negotiation
- Construction

The objectives of risk management are defined as the identification of project risks resulting from design and construction and these are to be achieved by establishing a construction risk policy. This policy would include scope, risk objectives and a management strategy.

It is stated that the scope of the risk policy might include the risks and consequences that are set out below

- Risk to the health and safety of workers, including personal injury and, in the extreme, loss of life
- Risk to the health and safety of third parties
- Risk to third party property, specifically existing buildings and structures, cultural heritage buildings and above and below ground infrastructure

- Risks to the environment including possible land, water or air pollution and damage to flora and fauna
- Risk to the owner in delay to the completion
- Risk to the owner in terms of financial losses and additional unplanned costs

Some of these risks and their consequences are set out in Table 4.1 above.

The stated objectives of risk management would be to allow for the identification of hazards, the identification of mitigation measures and the implementation of mitigation on the ALARP principal. The guidelines suggest that the policy might minimise overall risk by reducing the likelihood of hazards with high consequences (linked to political concerns) if it is considered that low probability high consequence events to be of more concern than high probability low consequence events. Policy should also allocate risks to those best able to control them.

It is suggested that the management strategy should include

- Definition of the responsibilities of the parties
- A list of the actions required to meet the objectives
- A live risk register to be cascaded to all parties
- Feed back loops
- Audits

It should also define its risk acceptance criteria which should be provided in either qualitative form (Table 4.3 above), or in quantitative form. Such quantitative limits, as mentioned before, are difficult to define for third party hazards as the data is not available and anchor points ("accident frequency just on the borderline of acceptability") [16] have not been established.

The Guidelines also set out the necessary components of a risk management system which are stated as being:

- Hazard identification
- Classifications

  Frequencies

  Consequences

  Risk classification and risk acceptance

A range of the risk management tools that are available are also listed in the Guidelines and these include the following:

- Fault tree analysis
- Event tree analysis
- Decision tree analysis
- Multirisk
- Monte Carlo simulation

6.4 ABI Code of Practice (2003)

Contact Insurers will require compliance with the Code on all projects in UK where the value of the Tunnel Works is £1.0 million or more. Section 2 of the Code also notes in respect of compliance that:

"*Compliance with the Code as it applies to construction projects involving tunnel works should minimise the risk of physical loss or damage and associated delays. It follows that Insurance contracts covering Tunnel Works should include provisions enabling Insurers to enforce the requirements of the Code, if necessary on pain of suspension or cancellation of the cover.*"

The Code divides the stages of a project into four

- Development Stage (feasibility studies, site investigations, optioneering and design studies)
- Design
- Construction Contract procurement (contract documentation, tendering process and tender assessment)
- Construction

It then requires that hazard identification and risk management is undertaken throughout each of these stages through the use of a formalised Risk Management System (RMS). This system is to be used as a means of formally documenting the identification, evaluation and allocation of risks, which then have to be managed and mitigated on the ALARP principal. Live risk registers are required in order to properly assign risks and also to cascade the risks to parties throughout the project.

A Client's roles and responsibilities are set out, together with a requirement for their technical and management competences to be demonstrated.

Apart from statutory duties, health and safety issues are only defined within the Code in terms of their input to H&S Plans, Method Statements and Management Plans.

The Code also sets out requirements for the scope and quality of input to the ground investigations, including the need for adequate budget and programme time for the task. It also requires that a set of Ground Reference Conditions [1] or Geotechnical Baseline Conditions [23] be prepared.

A systematic process of risk assessment and management is described in order to identify consequences associated with " *third parties and existing facilities including buildings, bridges, tunnels, roads, surface and subsurface railways, pavements, waterways, flood protection works, surface and subsurface utilities and all other structures/infrastructure which shall be affected by the carrying out of the works*". Design risk assessments are also required to consider the impact of the design and its implementation (including realistic variations in the design criteria and/or design values adopted) on third parties. Design checks that assess the level of risk and compliance with requirements to third parties are also a requirement.

6.5 Human Factors

Whichever of the above, or other, system is used to manage risks within a tunnel project's planning, design and construction cycle, it has been put forward that there are four fundamental processes that need to be employed. (Anderson and Lance [49])

- The design and implementation of engineering systems
- The design and development of health and safety systems
- The adoption of appropriate organisational and management systems
- The consideration of human factors issues.

(These views are further developed in Section 7).

Most engineers are familiar with the first three of these processes through training and experience, but are generally less familiar with the need to take account of human factors issues in project planning and implementation. At all stages of project development, people have to make complex judgements and can make errors. Such unintended errors can be made at senior levels of management as well as at the tunnel face, and they can lead to an increase in the exposure to risk to workers and third parties. In the construction industry particular factors that might lead to such errors include:

- The number of differing organisations that may be closely involved in any one tunnelling project, and whose work activities may overlap
- The sometimes "adversarial" nature of contractual arrangements between the parties

- The pressure of progress, cost and site productivity in displacing safety as the first priority
- Lack of effective communication and coordination
- The over-compartmentalisation of functions within a project

The role of effective proactive monitoring in picking up errors can not be overestimated. This can be achieved by

1. Validating the initial competence of staff at recruitment
2. Self monitoring by all project personnel
3. The application of peer review to the process of data interpretation and decision making.

Risk based management systems should be able to take account of human factors issues by acknowledging the range of the three types of decision maker identified by Rasmussen, [50] and further explored by Reason [51]

- The skill based practitioner who largely relies on his self acquired experience and practice gained on previous projects
- The rule based practitioner who follows the known rules and guidance set out for design and construction, and
- The knowledge based practitioner who relies on both technical expertise and experience and who may go beyond established "rules"

Urban tunnelling provides a wide range of technical, organisational and human factors challenges to project engineers and others, and these need to be met with a flexible approach to problem solving while ensuring health and safety at all times. Whereas rule based systems and practitioners are a necessity for a base culture, care must be taken to avoid a culture of unquestioning reliance on rules for problem solving. In his 2004 Harding lecture [52], Sir Alan Muir Wood stated that "in practice bureaucracy needs to be tempered by professional judgement. Only the experienced tunneller will detect the potential interactions between superficially diverse and unrelated type of risk." (See also Section 4.2 in 1996 HSE NATM Report)

The common elements of a positive safety culture that need to be integrated into an overall effective project management system in both design/planning and construction include

- Commitment from the top through an effective safety policy; through positive prioritisation of safety; through organising for safety including the allocation of adequate resources, and the setting of well defined individual responsibilities.
- Demonstration of this commitment through day to day management actions and the provision of resources where and when they are needed, including the provision of specialist support and training and education
- Open and effective communication within and between organisations including a continuous dialogue on safety and risk matters. Communications should maintain impartiality and, where appropriate, confidentiality isolated from a culture of blame and mistrust
- Encouragement and enthusiastic support should be given to good performance, innovations and ideas. This can encourage all those involved to feel fully "on board" and working as a team with the aim of continuous improvement.

- Ensuring that particular management positions are filled by people with the appropriate background, experience, competence and leadership skills.
- Building in appropriate and effective monitoring and review arrangements

6.6 Risk Management Systems

Management teams for some large tunnelling projects that have been delivered within the last decade, have developed and prescribed methodologies to integrate risk management systems into each stage of their project implementation process. These projects have been developed independently but the concepts derived have been reviewed and incorporated as necessary within the development of the ITA Guidelines.

Examples of projects that have successfully employed risk management systems (RMS) throughout the planning, design, procurement and construction cycle include

- Hamburg- 4 th. Elbe Tunnel [53]

- Copenhagen Metro [54, 55, 56, 57]

- Madrid Metro[39]; and very recently

- Heathrow Airport, Terminal 5 tunnels [58]

These projects all used risk assessments as a tool to inform and record each decision making process and to minimise unforeseen events and costs.

At Heathrow Terminal 5, for example, the means chosen to minimise risk in the engineering and commercial management organisation was to adopt a non-adversarial contractual approach based on relationships and behaviours rather than transactions tied into a cost reimbursement type contract. This was achieved by reimbursing suppliers' costs with a defined profit plus an incentivised target payment based on performance. The contract agreement focused on performance and success rather than failure, and on people as individuals in integrated teams not as external suppliers. The contractual risk of cost and programme variations was not shared with the suppliers and was held by the client, British Airports Authority (BAA). As BAA carried the commercial risk, risk management was undertaken by a fully-integrated, co-located team of BAA and its suppliers comprising client, designers, civil constructors and the fitting-out provider.

A separate example of formal hazard assessment procedures that were introduced into an ongoing project management system in order to control tunnelling risk is described in a paper on the Athens Metro [59]

Other project specific approaches have been described in the tunnelling literature, and include:

- Mala Kapela Tunnel, Croatia. Kolic [60]

- Hvalfjordur Tunnel, Iceland. Tengborg et al. [61]

One element of any risk management system which must not be overlooked is the quantity of paperwork that can be generated by the processes adopted. At the primary level a risk management system needs to be process driven in order to ensure that a comprehensive study of the issues is carried out. However, modern projects incorporate other processes such as Quality Assurance, Management Manuals, Project Plans, Safety Plans, design submission schedules etc. that provide a substantial demand for form filling and record keeping. Excessive quantities of paperwork can affect delivery of the product and it should be an aim of the risk management system to be effective above and beyond the process driven level.

7. THE STRATEGIC WAY FORWARD

"Fifty years ago, tunnelling was dominated by empirical methods in design, by traditional craft practices in construction. Today, design and construction of tunnels are based on a set of specialised technologies, with the success of each project dependent on their synthesis, on continuity between design and construction, and on appropriate means of project procurement. The art of tunnelling does not lend itself to inflexible rules or prescriptive codes of practice; engineering judgement remains the key factor." [62]

Incidents and 'emergency events' during underground construction are often the failures of the systems devised and put in place to prevent such happenings. It follows that effort should be put into devising, implementing and continuously improving the overall management systems tailored to fit the specific circumstances of any particular underground project. No safety management system, by itself, can eliminate all risk of danger and harm, and some hazards and risks have to be effectively managed on a day-to-day basis by competent persons and organisations. This needs commitment and input from the workforce to the higher management levels of key organisations within the specific project structure.

In underground construction work there is always a level of 'uncertainly', but this does not mean that this 'uncertainly' need translate itself into danger. The three key parties that have the power to set standards and procedures are the client or owner; the designers, planners and specifiers; and the main and other contractors and their suppliers of plant, equipment and materials.

From the study of the data presented in this report, and from the consideration of other information, it may be seen that the following six factors have an influence on the quality of the overall safety management system required within an underground construction project-

- *Project management*
- *Organisational, procurement and contractual arrangements*
- *Engineering systems*

- *Health and safety systems*
- *The consideration of human factors, and*
- *Availability and use of 'Enforcement' action*

These factors are briefly described in the following paragraphs.

7.1 Project management

Each of the parties needs to project manage effectively his particular input to the development of an integrated safety management system – starting with the client and his initial outline considerations for his project. Often project management needs to begin with the gathering of information; the undertaking of research; the assembly of sufficient and appropriate expertise and the provision of adequate time and other resources before the brainstorming of scenarios and ideas for possible alternative further courses of action. Consideration of the whole-project risk spectrum from the beginning is appropriate and there are advantages to be gained from, for example, integrated management teams, early contractor involvement; early supplier involvement and early operator involvement. Inadequate project management or even mis-management of some or of the whole process is possible at any stage and by any of the key parties.

7.2 Organisational, procurement and contractual arrangements

Organisational, procurement and contractual arrangements such as between the client and his chosen design teams can be crucial to the quality and effectiveness of the overall safety management system. Project organisational systems also need to relate to those third parties who may be affected by the works. The internal contractual arrangements provide an opportunity to emphasise the importance of cooperation, coordination and communication in the effective identification and control of hazards and risks throughout the project by all parties and the responsibility of all parties to expend the necessary competence and resource to ensure 'best practice' approaches. There is no other field of civil engineering where the integration of design and construction is more important or necessary. Poorly drafted procurement and contractual arrangements can conflict with statutory obligations; impose unfair and unreasonable conditions; create ambiguity, misunderstanding and doubt; and cloud areas of responsibility from the very beginning.

7.3 Engineering systems

Much of both design and construction on underground projects relies on the choice and implementation of the engineering systems that are commonly applied to this type of construction work. This could range from the overall design, specification, layout and detailed design of the tunnel lining to the specification and manufacture of complex tunnel boring machines to work effectively in the tunnelling medium of the particular project. The inappropriate choice of, for example, computer analysis software; persons to fill key safety-critical engineering management positions; open or closed face tunnel boring machine; the wrong type of ground stabilising technique; the tunnel lining/surrounding ground monitoring system or the depth of the construction work within the ground in urban situations can compromise the effectiveness of the overall safety management system.

7.4 Health and safety systems

The provision and effective implementation of specific systems for the purposes of health and safety are essential. These range from the provision of specialist advice on health and safety and risk management to those working in both design offices and construction sites and to the provision of persons with special expertise on project expert advisory panels. The prompt, thorough and timely investigation of any accidents, incidents and "near misses", together with the downloading of the information and conclusions obtained to all who need to know, is of the first importance. Also important is the auditing and review of the elements of the safety management systems with the objective of seeking ways of improvement. Health and safety standards never stand still, and there needs to be the flexibility within the system, coupled with appropriate leadership, to seek to apply higher health and safety standards where this is both practicable and appropriate whether or not required by law.

7.5 The consideration of human factors

In large projects it is almost inevitable that safety management systems will be complicated and diverse, but they will have to be implemented and managed by persons with a range of skills, experience and knowledge. Some persons will also be given the task of exercising judgements in both engineering and in other contexts – in other words aspects of the overall safety management system will be dependent on 'human factors'. The question then arises, when and where might 'human errors' of one kind or another be made, and, if so, what the possible consequences of these 'errors' might be. It is undoubtedly the case that some 'emergency events' have also been caused by persons intentionally violating work systems and procedures that have been correctly set out, and it is also the case that high level 'errors' can play a major part in creating the circumstances where others might make errors at the workface. The potential for human failure, error or misjudgement is almost everywhere within the system, and as such merits careful consideration by all parties.

7.6 Availability and use of 'enforcement' action

Where shortcomings in the safety management system have been pinpointed, there needs to be a mechanism by which, if appropriate, effective corrective action can be taken. In some cases it may be necessary to stop the construction work so that the necessary remedial measures can be given priority. This power exists within the statutory enforcement body for health and safety, but should also exist within the contractual arrangements. The sanction of ultimately stopping further construction work on health and safety grounds could be retained by the client – if necessary on being properly advised about the proposed course of action. Alternatively, the client could retain an organisation with the appropriate competence, skills and knowledge to exercise the role of construction supervision on the client's behalf. Such a function would have a clearly defined health and safety monitoring role with a contractual ability to stop further construction work if this was appropriate in the circumstances.

Because of the diversity of underground project work there is no 'one standard system fits all' situation. The authors believe that the factors listed and described above, together with existing published information and guidance, provide a framework of approach that can lead to a satisfactory safety management system for any underground construction project in any location.

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APPENDICES

A List of world tunnel projects 1999 to 2004.

B. List of Internet sites used to research world tunnelling market.

C List of NATM Emergency Events.

D List of Non-NATM Emergency Events

E. Glossary of terms.

F. Authors' background.

Appendix A - List of world tunnel starts 1999-2004

		1999		2000		2001		2002		2003		2004		Total	
COUNTRY	GNP per capita ($)	No of Tunnels	Length of tunnels (km)	No of tunnels	Length of tunnels (km)	No of tunnels	Length of tunnels (km)	No of tunnels	Length of tunnels (km)	No of tunnels	Length of tunnels (km)	No of tunnels	Length of tunnels (km)	No of Tunnels	Length of tunnels (km)
Afghanistan	[illegible]													0	0.0
Albania	[illegible]											1	5.0	1	5.0
Algeria	[illegible]						5.2	1	2.4	9	30.6			11	38.2
Andorra	$4,946							1	1.2	[illegible]	3.0			2	4.2
Argentina	$4,220			3	26.5	2	10.3					3	24.4	8	61.2
Armenia	[illegible]							1	3.0					1	3.0
Australia	$19,530	2	42.4	3	2.2	1	3.2	2	17.0	2	18.4	4	25.1	14	108.3
Austria	$23,860							2	3.1	3	1.4	9	17.4	16	27.9
Bangladesh	$380													0	0.0
Belgium	$22,940	2	17.9	1	2.7	4	0.4	[illegible]	12.0	2	4.1	2	9.6	12	50.7
Bhutan	[illegible]													0	0.0
Bolivia	[illegible]					1	1.3							1	1.3
Bosnia & Herzegovina	[illegible]							[illegible]	0.8					1	0.8
Brazil	[illegible]	1	1.0	4	55.5	1	20.2	1	2.2	2	38.9			9	117.7
Bulgaria	[illegible]					2	11.4	3	18.2					5	29.6
Canada	$22,390					3	11.4	3	21.9	[illegible]	9.4	[illegible]	19.0	6	61.7
Chile	$4,250					2	2.8	3	14.4	2	6.6	2	9.2	9	34.0
China	[illegible]	7	85.9	16	84.1	17	223.8	115	120.0	8	71.8	7	64.4	170	659.[illegible]
Colombia	[illegible]									[illegible]	4.1	2	8.8	3	12.9
Costa Rica	$4,070			12	61.0									12	61.0
Croatia	$4,540			2	7.8			3	12.6			1	4.1	6	20.5
Cyprus	$4,725													0	0.0
Czech Republic	$5,480	1	1.2	3	7.5	2	5.2	14	18.0	4	20.2	5	9.4	29	59.3
Denmark	$32,260			3	28.3					1	4.0			4	32.3
Dominican Republic	[illegible]					1	15.4							1	15.4
Ecuador	[illegible]			2	2.1	1	4.1	2	1.4	1	11.1			6	18.[illegible]
Egypt	[illegible]					2	5.0	2	16.5	1	4.3			5	25.8
El Salvador	[illegible]													0	0.0
Estonia	$4,190											1	1.0	1	1.0
Ethiopia	$100													0	0.0
Finland	$23,860	1	0.5	1	3.0	[illegible]	0.7	1	3.6	7	21.0	[illegible]	37.2	19	62.8
France	$22,240	1	9.5	7	22.7	5	[illegible]	8	20.1	14	29.4	18	86.7	53	179.6
Georgia	$650													0	0.0
Germany	$22,740	31	56.0	5	53.2	19	39.0	9	12.4	28	40.8	11	37.9	104	269.3
Greece	$11,660	1	24.7	9	40.8	1	1.6	[illegible]	4.7	6	10.0	9	12.2	26	93.9
Hungary	$5,290											1	7.3	1	7.3
Iceland	$11,752			[illegible]	10.2			3	70.9	6	31.2	2	4.6	12	116.9
India	$470	7	71.7	5	40.4	6	99.[illegible]	5	10.3	5	54.6	1	7.5	26	293.2
Indonesia	$710				14.5					1	10.0			2	24.5
Iran	[illegible]	2	36.4			1	23.0	2	8.5	3	15.5	[illegible]	65.5	9	148.4
Iraq	[illegible]							1	6.0					1	6.0
Ireland	$23,030	2	5.8	4	10.3	[illegible]	5.5	1	3.2	3	30.8	2	7.8	12	62.7
Israel	$16,020	1	1.0	1	1.7	[illegible]	6.5			[illegible]	9.0	[illegible]	7.5	5	25.7
Italy	$19,080	18	163.7	15	116.2	16	53.4	91	120.8	33	167.5	[illegible]	258.6	205	903.2
Japan	$34,010	18	30.7	4	37.5	[illegible]	1.1	1	0.0	3	60.6			27	130.0
Jordan	[illegible]				1.0						1.7			2	2.2
Kazakstan	[illegible]													0	0.0
Kenya	$360					1	5.3							[illegible]	[illegible]
Kuwait	$16,340													[illegible]	0.0
Kyrgyzstan	$290													0	0.0
Laos	$310	1	5.0											[illegible]	5.5
Latvia	$3,480													0	0.0
Lebanon	$3,350													0	0.0
Lesotho	$550		0.2				5.6							2	5.8
Liechtenstein	$23,000													0	0.0
Lithuania	$3,670													0	0.0
Luxembourg	$12,110				5.0			5	18.2			2	5.9	8	22.1
Macedonia	[illegible]													0	0.0
Malawi	$160													0	0.0
Malaysia	$3,540							2	5.0	1	25.0	4	19.7	7	59.2
Malta	$3,996					1	6.0							[illegible]	6.0
Mexico	$5,920	2	2.7					7	8.4			1	1.6	10	12.7
Monaco	$18,527													0	0.0
Morocco	[illegible]	1	4.2											1	4.2
Nepal	$250			5	38.3			1	27.0					6	65.3
Netherlands	$23,390			5	25.5	8	42.1	1	9.7	[illegible]	6.0	3	12.0	18	95.[illegible]
New Zealand	$13,260													0	0.0
Nigeria	$300													0	0.0
Norway	$38,730	9	35.8	[illegible]	2.6	49	51.3	22	94.2	15	[illegible]	24	53.2	121	284.5
Pakistan	$420	2	19.8	2	30.5	2	30.0		1.4	6	11.6			13	93.4
Panama	$4,020			2	2.4									2	2.4
Paraguay	[illegible]			1	3.0									1	3.3
Peru	[illegible]	1	7.5	3	26.3			[illegible]	25.0	[illegible]	14.0	[illegible]	7.3	[illegible]	77.4
Philippines	[illegible]			7	55.6	[illegible]	10.0	[illegible]	3.8			[illegible]	4.2	10	73.4
Poland	$4,570			1	1.6	[illegible]	3.5	1	4.1	2	9.4	[illegible]	3.8	[illegible]	27.0
Portugal	$10,720	23	32.5	15	27.3	35	49.[illegible]	9	22.0	39	35.[illegible]	[illegible]	90.1	[illegible]	[illegible]
Romania	[illegible]			3	16.4					1	1.4			4	17.5
Russia	[illegible]			4	17.5	2	22.[illegible]			2	4.0	[illegible]	1.2	[illegible]	45.5
Saudi Arabia	$8,530													[illegible]	0.0
Singapore	$20,600	4	46.4	4	23.[illegible]	[illegible]	33.2	2	8.6	2	17.0	3	25.7	[illegible]	[illegible]
Slovakia	$3,970	5	31.7			2	15.1			1	2.9			8	49.6
Slovenia	$10,370	8	69.0	2	4.0			4	6.5	3	8.0			[illegible]	87.9
South Africa	[illegible]							1	80.2					[illegible]	80.2
Spain	$14,580	8	20.5	64	189.[illegible]	14	266.5	74	378.6	88	281.3	136	409.[illegible]	404	1545.7
Sri Lanka	[illegible]	[illegible]	1.0	1	5.2									2	[illegible]
Sweden	$25,970	6	37.4	6	18.6	[illegible]	4.9	24	73.5	[illegible]	41.3	4	29.2	[illegible]	194.[illegible]
Switzerland	$36,170	5	19.0	19	61.4	[illegible]	87.6	14	14.2	3	12.2	2	95.1	[illegible]	330.5
Syria	[illegible]				4.5									1	[illegible]
Taiwan	[illegible]			4	73.1	[illegible]	58.4		4.8					[illegible]	136.2
Thailand	[illegible]	[illegible]	83.[illegible]	[illegible]	31.0				65.0	[illegible]	5.3		6.7	[illegible]	190.7
Tunisia	[illegible]													[illegible]	[illegible]
Turkey	[illegible]	3	10.6	[illegible]	31.4	11	37.3	3	22.3	6	23.0	2	3.8	25	137.1
Turkmenistan	[illegible]							2	15.0					[illegible]	15.0
Ukraine	[illegible]					[illegible]	9.8							[illegible]	[illegible]
United Arab Emirates	$10,000					[illegible]	1.7			1	1.5			[illegible]	2.7
United Kingdom	$25,510	10	31.5	6	43.9	4	31.9	3	4.9	8	12.2	3	35.8	[illegible]	190.2
United States	$35,400	24	161.3	[illegible]	51.0	25	180.[illegible]	[illegible]	149.[illegible]	5	[illegible]	9	41.[illegible]	105	642.5
Uruguay	$4,340							4	25.3					4	[illegible]
Venezuela	$4,080			10	[illegible]	5	6.8							15	43.9
Vietnam	$430					[illegible]	9.7	1	8.6			[illegible]	2.0	3	20.2
Yemen	$490						1.9							[illegible]	1.9
Yugoslavia	[illegible]													0	0.0
Zambia	$340					1	7.5							1	7.9
Zimbabwe	$310					[illegible]	4.3							1	4.3
		220	1168	300	1515	327	1698	426	1620	342	1319	402	1461	2017	8750

Appendix B

List of internet sites used to research world tunnelling market

www.miningandconstruction.com

www.fhwa.dot.gov/bridge/tunnel/index.htm - This website aims to produce data base of projects in0 the US.

www.tchas.c3/en/ps_sberaco3.htm

www.insituform.com/coperate/cer_affholder.html#

http://www.ita-aites.org/cms/329.html - Cairo metro lire information

www.aftes.asso.fr/revues_tos/2004/18/_res.htm – Biarritz sewer

www.nationmaster.com/encyclopedia/list-of-cities-in-france

www.rinbad.demon.co.uk/opening.htm - list of all opening metro lines

www.alpine.at/content.node2/alpine/de/nl/utb/tusptb/tu/2818.html - tunnel construction information

www.unece.org/trans/doc/2002/ac9/TRANS-AC9-1-info07e.doc - list of tunnels in different countries

http://home.no.net/lotsberg/data

www.showcaves.com/english/de/geology.html

www.roadtraffic_technology.com/project/egnatia/ - EGNATIA ODOS

http://tunnelling.metal.ritua.gr/GTS/projects/egnatia.htm - 49km tunnel; 4.5% cut &cover; half hard, half soft rock; 74 tunnels; NATM

www.mjconstruct.com/tunnel/archive/2003/jun/nordie%20tun%20%hard%20rock.pdf

www.mjconstruct.com/tunnel/arcjive/2003/sep/panorama.pdf-india

www.auaonline.org – American Underground Association www.britishtunnelling.org – British tunnelling society;

http://home.no.net/lotsberg - the worlds longest tunnel page

www.tunnel.no – Norwegian rock technology

www.ita-aites.org/cms/357.html - Norway -references to Norway storing statistical data since 1971 – 1998 best year; Germany – Printout; US $700-800 million dollars

www.library.tudelf.nl/dckc/keep-current - Delft Cluster Tunnelling – knowledge centre

www.geoscience.org.za/sancot/publ.httm# - South African Tunnel Database purchasable through SANCOT

www.cnplus.co.uk – construction news – pay only

www.tunnelcanada.ca

www.auca.org

www.naslt.org

*Searched German and Norwegian search engines

www.ita-aitet.org – international tunnelling association

www.britishtunnelling.org – British tunnelling society; newsletters –N/A; working groups

www.tunnelbuilder.com – comprehensive list

Geological Research

http://geologyabout.com/library/bl/maps/blecudermap.htm - geological maps

http://encarta.msn.com/encart/features/mapcentre/map.aspx - maps

www.worldatlas.com/atlas/world.htm - geological maps

GDP Research

www.worldbank.org/data/udi2004/table

www.nationmaster.com/country/an/economy

Appendix C - List of NATM Emergency Events

Research No	HSE Report No	Date	Location	Remarks	Reported Cause	Reference	Project	Environment	Ground	Third Party Consequences	With reported cause	A type Cause	A type Cause USG
NATM/01	HSE/1	Oct-71	Near Paris	Collapse	A6 and A8	Muller 1970	Rail	?	Soft		1	1	
NATM/02		1975?	Massenberg, Austria	Crown lining failure	A7	Muller, E & T 1990			Rock		1	1	
NATM/03	HSE/2	18-Dec-01	Sao Paulo	Face instability	A1	Kochen, 1997	Metro	Urban	Soft	Building Disruption	1	1	
NATM/04	HSE/3	1993	Santos, Brazil	Cave in		Private	Rail	Urban	?	5 houses demolished			
NATM/05		15/11/1983	Germany	Heading instability		Xudong Cui, 1991	Rail						
NATM/06		21-05/1984	Germany	Crater		Xudong Cui, 1991	Rail						
NATM/07	HSE/4	13-Nov-84	Landrucken, Germany	Collapse	A8, A13	Nussberger, 1987; Wogan, 1997	Rail	Rural	?		1	1	
NATM/08		16/10/1984	Germany			Xudong Cui, 1991	Rail						
NATM/09	HSE/5	1984	Bochum, Germany	Collapse	B1, A7, A9	Sagen 1987	Metro	Urban	Soft	Urban disruption	1	1	1
NATM/10	HSE/6	12-Jun-85	Richtof, Germany	Collapse	A1	Friedls, 1987	Rail	Rural	Rock		1	1	
NATM/11	HSE/7	1985	Bochum, Germany	Collapse	A1	Laue, 1987	Metro	Urban	Soft?	Urban disruption	1	1	1
NATM/12	HSE/8	Aug-85	Kaiserau, Germany	Collapse	A4, A5	Wallis, 1988, 1990	Rail	Rural			1	1	
NATM/13	HSE/9	07/02/1986	Krieberg, Germany	Collapses & Crater	A5, A10	Wallis, 1987	Rail	Rural	Soft	Surface damage	1	1	
NATM/14-18		1985-86	Hannover, Wurzburg	Collapses & Craters		T & T Summer 1990	Rail		Rock				
NATM/19	HSE/10	Pre-1987	Munich, Germany	Collapse	C2	Weber, 1987	Metro	Urban	Soft	Urban disruption	1		
NATM/20	HSE/11	Pre-1987	Munich, Germany	Collapse	A1, A12	Weber, 1987	Metro	Urban	Soft	Urban disruption	1	1	1
NATM/21	HSE/12	Pre-1987	Munich, Germany	Collapse	A1	Weber, 1987	Metro	Urban	Soft	Urban disruption	1	1	1
NATM/22	HSE/13	Pre-1987	Munich, Germany	Collapse	A1, A12	Weber, 1987	Metro	Urban	Soft	Urban disruption	1	1	1
NATM/23	HSE/14	Pre-1987	Munich, Germany	Collapse	A1, A12	Weber, 1987	Metro	Urban	Soft	Urban disruption	1	1	1
NATM/24	HSE/16	Pre-1987	Munich, Germany	Blow out	A1	Weber, 1987	Metro	Urban	Soft		1	1	1
NATM/25	HSE/15	Pre-1987	Wolfsburg, Germany	Cave in	A3	Schrewe, 1987	Rail		Soft		1		
NATM/26	HSE/17	1987	Karawanken, Austria	Large deformations		Martin, 1991	Road	Rural	Hard				
NATM/27	HSE/18	Pre-1998	Kerrenberg, Germany	Large settlements Tunnel props destroyed	(A3)	Wallis, 1995	Rail		Soft		1	1	
NATM/28	HSE/19	1988	Michaels, Germany	Tunnel collapse	(A10)	Private			Soft?		1	1	
NATM/29	HSE/20	08/01/1989	Karawanken, Austria	Collapse	A1, A21	March, 1993	Road	Rural	Rock		1	1	
NATM/30		1990	Channel Tunnel Crossover	Crown failure	B2	ICE Proc. Jan 93	Rail	Underseа	Soft		1		
NATM/31	HSE/21	27-Sep-91	Kwachon, Korea	Collapse & crater		Newspaper articles	Metro	Road	Soft				
NATM/32	HSE/22	17-Nov-91	Seoul, Korea	Collapse & Crater	A1	Park & Lee, 1993	Metro	Urban	Soft	Fractured gas main	1	1	1
NATM/33	HSE/23	27-Nov-91	Seoul, Korea	Collapse & Crater	A1	Park & Lee, 1993	Metro	Urban	Rock	Urban disruption	1	1	
NATM/34		Nov-91	Sao Paulo	Collapse & Flood		Press reports	Road	Urban	Soft				
NATM/35		1991	S Vitale, Italy	Large deformation		Lunardi, Tunnel 8/96	Rail	Rural	Soft				
NATM/36	HSE/24	1992	Funagata, Japan	Collapse	(C1)	Private	Road	Rural?	Soft		1		
NATM/37	HSE/25	12-Feb-92	Seoul Metro	Collapse	A2	Park & Lee, 1993	Metro	Urban	Soft	Utilities broken, Traffic Problem	1	1	
NATM/38	HSE/26	30-Jun-92	Lainbach, Austria	Collapse & Crater	A5	Vavrovsky, 1995	Rail				1		
NATM/39	HSE/27	27-Jan-93	Seoul, Korea	Collapse & crater	A1	Park & Lee, 1993	Metro	Urban	Soft	Road disruption	1	1	1
NATM/40	HSE/28	02-Feb-93	Seoul, Korea	Collapse & crater	A1	Park & Lee, 1993	Metro	Urban	Soft		1	1	1
NATM/41	HSE/29	Feb/March 93	Seoul, Korea	Collapse & crater	A3	Park & Lee, 1993	Metro	Urban	Soft		1	1	1
NATM/42	HSE/30	Feb/March 93	Seoul, Korea	Collapse & crater	A1	Park & Lee, 1993	Metro	Urban	Soft		1	1	1
NATM/43	HSE/31	Mar-93	Chungho, Taiwan	Collapse	A1	Private	Road	Rural			1	1	
NATM/44	HSE/32	Nov-93	Sao Paulo, Brazil	Collapse & Sink hole	A1, A3, A7	Private	Metro	Urban	Soft	Massive disruption	1	1	1
NATM/45	HSE/33	1993	Tuscany, Italy	Severe deformations	A3	Pelizza, 1994	Road	Rural?	Rock		1	1	
NATM/46	HSE/34	Apr-94	Cuadrilha Pinto Brazil	Re-inforce repairs	C1	Private	Road			Traffic Disruption			
NATM/47	HSE/35	30-Jul-94	Montemor, Portugal	Collapse & crater	(A3)	Wallis, 1995	Road	Urban	Soft		1	1	
NATM/48	HSE/36	01-Aug-94	Montemor, Portugal	Collapse & crater	(A1)	Wallis, 1995	Road	Urban	Soft		1	1	1
NATM/49	HSE/37	Aug-94	Gothenburg, Austria	Collapse	A1	Schubert, 1995				One Fatality	1	1	
NATM/50	HSE/38	20-Sep-94	Munich, Germany	Collapse & Crater	A1	Teltham, 1995	Metro	Urban	Soft	4 Fatalities 27 injured	1	1	
NATM/51	HSE/39	21-Oct-94	Heathrow Airport	Collapse & craters	A1, A3, A12	HSE, 2000	Rail	Urban	Soft		1	1	1
NATM/52		Mid 1995	Kozlu, Turkey	Collapses		Cuby, Tunnelling 97	Road	Rural	Soft				
NATM/53		Dec 1995	Vasto, Italy	Face collapse	A1	Lunardi, Tunnel 8/98		Rural	Soft				
NATM/54		18/06/1995	Eidsvoll, Norway	Collapse		Dolines 1996	Rail						
NATM/55		1999?	Bolu, Turkey	Partial collapse	A1	Menkiti, Mair & Miles: Underground Construction 2001	Road	Rural	Soft		1	1	
NATM/56		01/06/1999	Essen, Germany	Crown collapse		Arnold & Fugenschuh	Rail	Rural	Rock		1		
NATM/57		07/11/2000	Dulles Airport USA	Collapse	Not known	T & T Dec 2003	Walkway	Urban	Soft				
NATM/58		11/10/2001	Herzogberg, Austria	Collapse	A1	T & TI Jan 2003	Road	Rural	Rock				
NATM/59		Late 2001?	Herzogberg, Austria	Roof collapse	A1	T & TI Jun 2003	Road	Rural	Rock				
NATM/60		19/05/2001	Istanbul, Turkey	Collapse & Crater	A1	WT Dec 01	Metro	Urban	Soft	5 Fatalities	1	1	
NATM/61		21/02/2002	Gwensberg, CH	Collapse & Crater		Internet		Rural	Rock?				
NATM/62		Aug 2002	Hsinchu, Taiwan	Face collapse		T & TI Feb. 2003	Rail		Soft				
NATM/63		31/10/2002	Hsinchu, Taiwan	Face and lining collapse		T & TI Feb. 2003	Rail		Soft				
NATM/64		07/01/2003	Athens	Collapse	B2	Private	Metro	Urban	Soft		1	1	
NATM/65		Feb-05	Barcelona	Collapse		Press articles	Rail	Urban	Rock	Massive disruption			
NATM/66		Feb-05	Lausanne	Collapse & Crater		Press articles	Rail	Urban	Soft	Massive disruption			
											45	39	23

Total Number of incidents with a reported cause 44
Number with an A type cause 39

Total number of urban soft ground incidents with a reported cause 27
No with an A type cause 25

Appendix D - List of non-NATM Emergency Events

Appendix E

Glossary of terms

ABI	Association of British Insurers.
BTS	British Tunnelling Society.
CIRIA	Construction Industry Research and Information Association
HSE	Health and Safety Executive
ITA	International Tunnelling Association
NATM	The New Austrian Tunnelling Method
SCL	Sprayed Concrete Linings
TBM	Tunnel Boring Machine

Published by the Health and Safety Executive
08/06

Appendix F

Authors' background

Guy Lance graduated in civil engineering from King's College, London University and is currently a Technical Director at Atkins Tunnelling. He has over 30 years' experience in tunnel design and in the management of major transportation projects in both the UK and overseas. These projects include Jubilee Line, Singapore MRT, Channel Tunnel, Channel Tunnel Rail Link and London Underground PPP. He acted as technical advisor to HSE during the investigations following the Heathrow Express collapse.

John Anderson graduated in civil engineering from Glasgow University. After work in industry he joined the Health and Safety Executive as a Specialist Inspector. For over 10 years of his 26 years service in the HSE he specialised in underground construction occupational health and safety issues. He has contributed to the meetings and the on-going work of the ITA "Health and Safety in Works" Working Group since 1989, and has written international papers on tunnelling health and safety matters since 1991. He left the service of the HSE in 1997 and works as an independent consultant.

HSE BOOKS

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HSE website: www.hse.gov.uk

RR 453

OBJ3/P3/A8

Transport and Works (Inquiries Procedure) Rules 2004

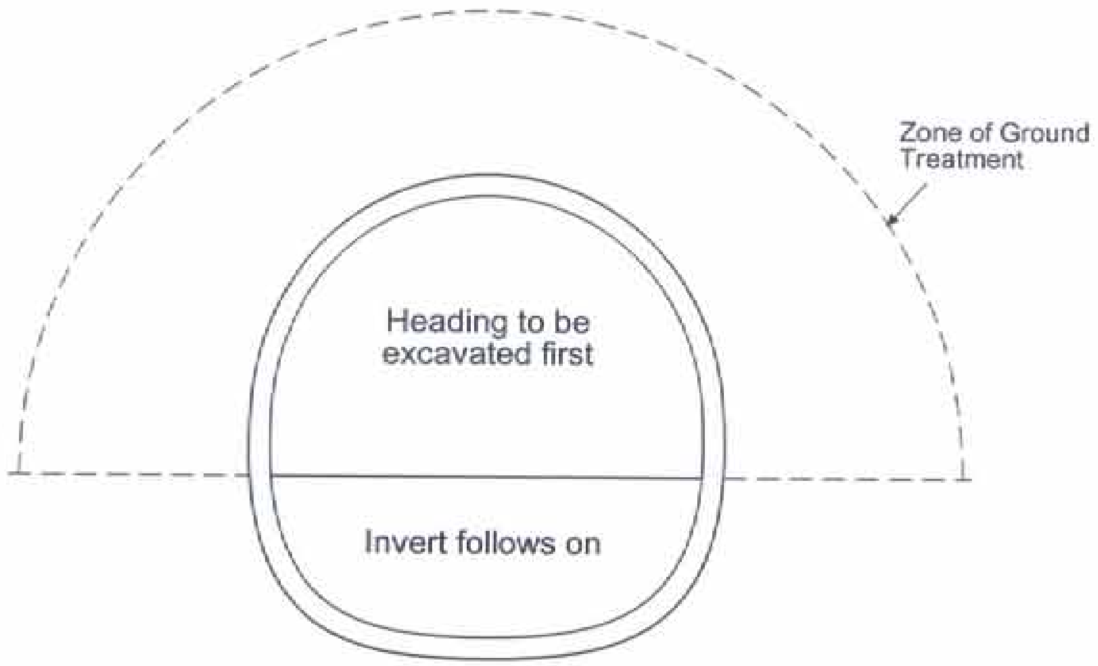
Proposed London Underground (Victoria Station Upgrade) Order

LAND SECURITIES PLC AND OTHERS (Objector No. 3)

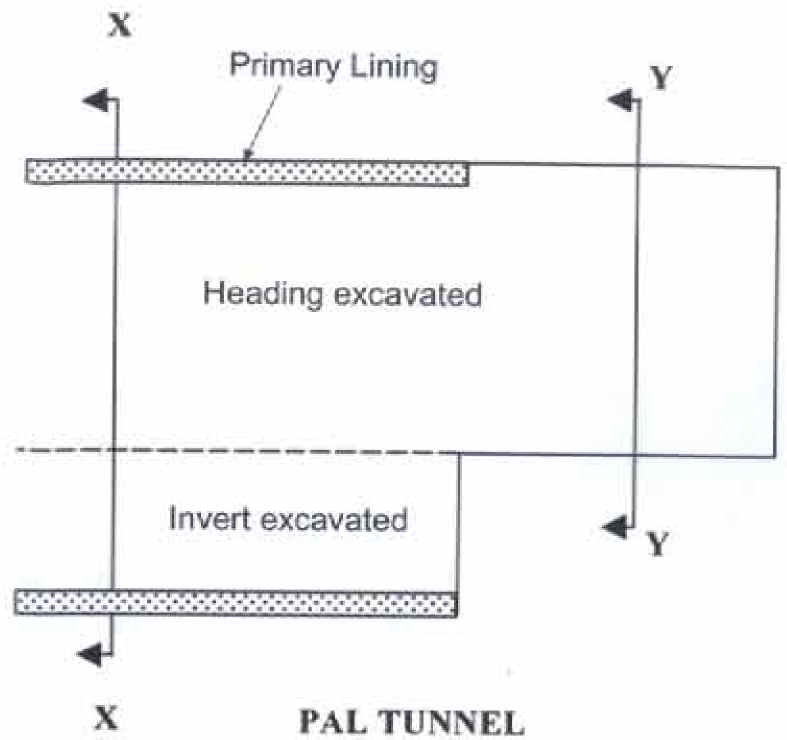
PROOF OF EVIDENCE of TIM CHAPMAN of ARUP

APPENDIX 8

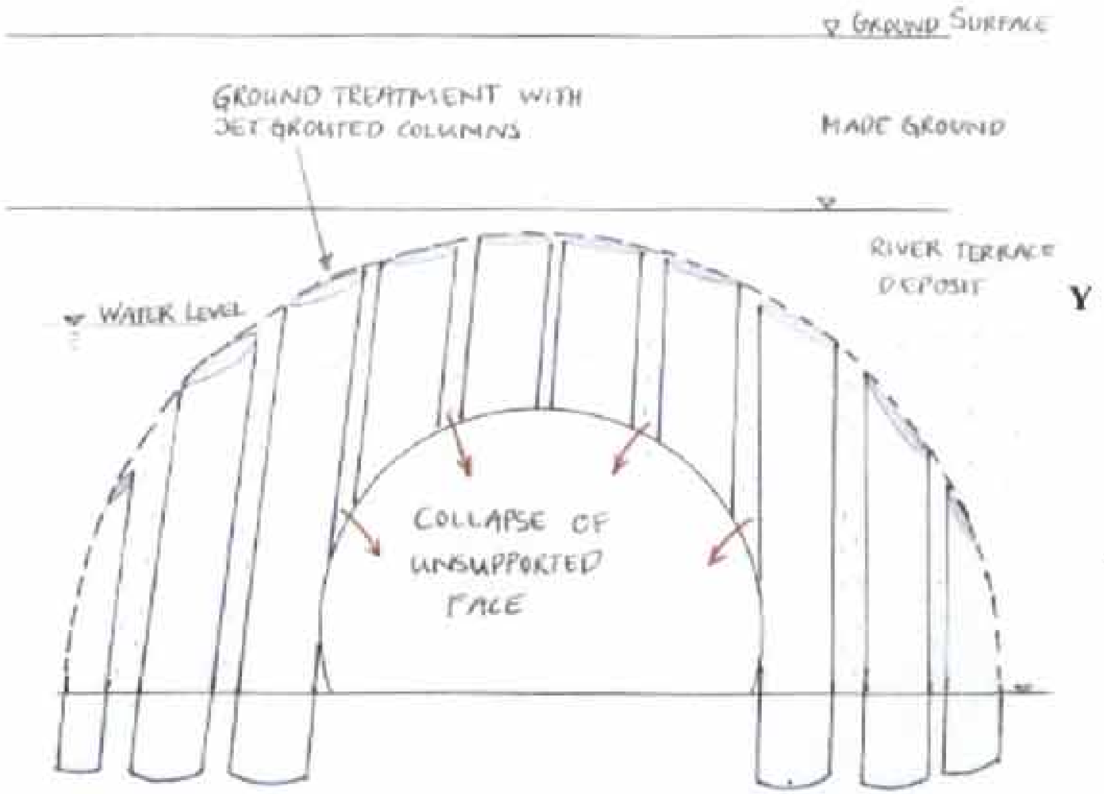
SCL EXCAVATION SEQUENCE / JET GROUTING PROPOSAL



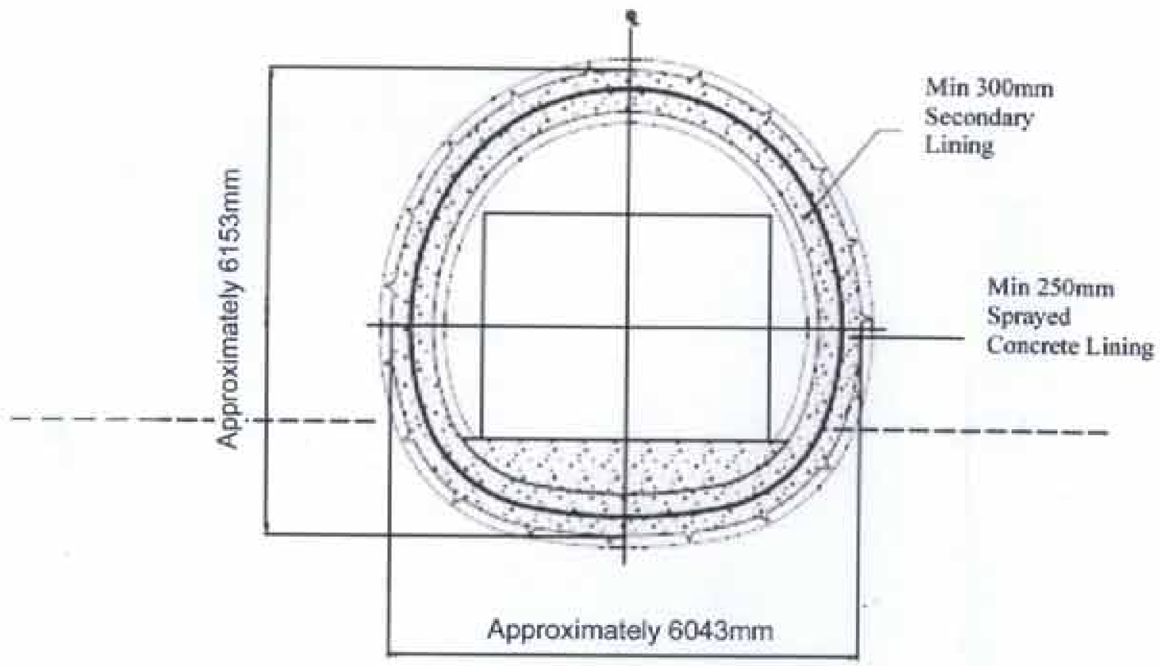
Section X - X



PAL TUNNEL



Section Y-Y



FINISHED PAL TUNNEL

EXHIBIT OBJ3/P3/A8
Evidence of Tim Chapman
Land Securities

SCL Excavation Sequence / Jet Grouting Proposal

OBJ3/P3/A9

Transport and Works (Inquiries Procedure) Rules 2004

Proposed London Underground (Victoria Station Upgrade) Order

LAND SECURITIES PLC AND OTHERS (Objector No. 3)

PROOF OF EVIDENCE of TIM CHAPMAN of ARUP

APPENDIX 9

ARTICLE FROM TUNNELS & TUNNELLING 2001

Shotcreting developments for German rail tunnels

Figure 11 (above): Installation of a grouted pipe umbrella (screen) in the Irlahüll tunnel

In the second part of his review, Prof Dr-Ing Bernhard Maidl of the Institute of Construction Technology, Tunnelling and Construction Management at the Ruhr University of Bochum discusses the role of screens in creating stability for excavation on major German rail links

As discussed last month, during construction of the Cologne-Rhine/Main and Nuremberg-Ingolstadt rail link, tunnels support had a particular significance.

In rock with insufficient stability for the required excavation cross-section, or when settlement was limited because of development, screens were called for, created by previously longitudinally installed pipes or by groutings.

At several places along the new German rail link, motorways were underpinned using this method, protected by umbrella screens. According to the conditions these were either pipe screen without injecting, injection pipe screen or high-pressure injection screen (Table 3).

Table 3

Method	Application	Range of application	Description	Equipment	System/Suppliers
Pipe screen		- non groutable loose and competent rock	- fan like/horizontal predrilling or thrusting of steel tubes 100mm dia, length up to 30m - concreting of the tube	- drill rig, jacks - bit, pipes - concrete pump	for example: - AT-Hüllrohrsystem (recoverable bit) Alwag-Techmo - System 'Odex' (recoverable bit) Atlas Copco
Injection pipe screen	- support measures of the roof zone in stratified rock structure - undercutting of objects - minimisation of settlements	- readily groutable competent rock with joints - readily groutable, non-cohesive loose rock	- fan like/horizontal predrilling or pressing of steel tubes 100mm dia, sometimes with length up to 30m - ground injection with cement-bentonite suspension through valve-holes in the tube	- drill rig, jacks - bit, pipes - grouting pump, injection-installations	for example: - AT-Hüllrohrsystem (recoverable bit) Alwag-Techmo - System 'Odex' (recoverable bit) Atlas Copco
High pressure injection screen		- mixed loose rock with variable grain size distribution - above groundwater level	- horizontal predrilling with simultaneous pre-injection to fill voids - jet grouting with simultaneous withdrawal of the drill pipes starting from the deepest borehole - construction of overlapping or touching high pressure injection zones 0.50m dia-1.00m, length up to 15m	- drill rig, drilling platform - jet grouting installations (pumps, mixers, dosage equipment etc.) - controlling and data aquisition installations	for example: - System 'Rodinjet'/Rodio & C. (Italy) - System 'Soilcrete'/ Keller - Bauer Spezialtiefau - Stump Spezialtiefau

Table 3 (left): The three methods of advance support with screens

Right: Core excavation in the Himmelburg tunnel

After the advance support, the cross-sections can be driven under cover of the screened vault. It is more cost effective if pipe screens can be produced by the heading team with the tunnel jumbo (new drilling installations are necessary though) rather than manufactured by special companies (Figure 11). The required niches for installing the pipe screen are no longer necessary.

Pipe screen without injecting

Examples of pipe screen without injecting include the Bodex-system and the AT-Hüllrohrsystem.

Production of the pipe screen uses an eccentric working drilling system where the drilling and the piping take place in one step. The pipe remains in the ground and is filled with concrete. For horizontal drilling, lengths of up to 30m are possible today where deviation depends closely on the predominant ground conditions. The space between the single pipes depends on the static aspects and is usually fractional.

Injection pipe screen

Injection pipe screens are similar in principle to pipe screens; there is a greater space between the single pipes as well as additional injecting of the *in situ* rock. The injection material is placed by special tubes which seals the drilling hole and prevents it getting wet. The method cannot be used in loose ground which requires a sleeve pipe to be installed using a cased bore.

Grout is placed in the surrounding rock through the openings in the sleeve pipe which are sealed by a rubber sleeve. At the Irlahull tunnel, a grouting pipe screen was used to underpin a motorway (Figure 12). Here the installation of the 15m long grouting pipe took place at an inclination of 7°, to within plus or minus 2%. Grout was placed through a valve in the pipe at a pressure of 10-15 bar. Finally, the pipe was filled with a cement suspension.

High-pressure injection screens

In high-pressure injection the soil is cut, held in suspension and mixed by a cutting jet of a cementitious slurry. The loose ground becomes the aggregate for the concrete. The column-like concrete structure is produced at the same time as grouting and pulling back the drill rod. In general, the high-pressure grouting column can be overlapping or tangential, with the horizontal length limited to around 15m.

The high-pressure jet grouting-method (HDI-Verfahren) was used in the 282m long underground driven section of the Frankfurter Kreuz tunnel. The advance support consisted of 14.5m long, overlapping horizontal jet grouted piles (Figure 13).

Ground freezing

Ground freezing, mentioned only for reasons of completeness, is a well known method in civil engineering. It affords a temporary seal and improvement for the subsurface ground in the form of an advance support for non-groutable, but water-bearing conditions. Basically, screen covers are created.

In contrast to the grouting methods, freezing affects the geological conditions only during the construction period.

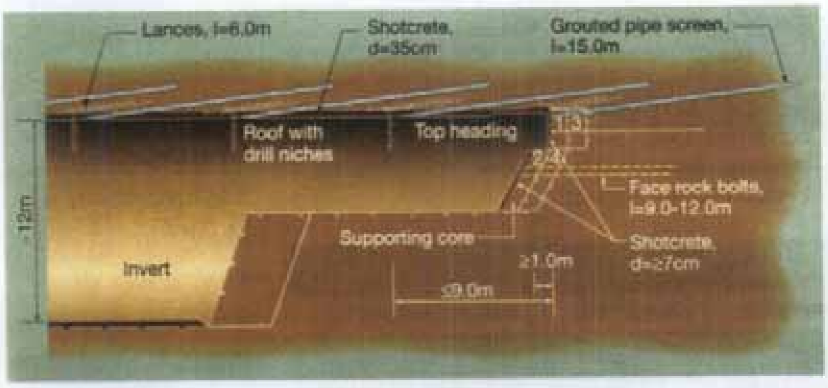


Figure 12, below: Section through the construction of the Irlahull tunnel under the protection of a grouted pipe umbrella (screen)

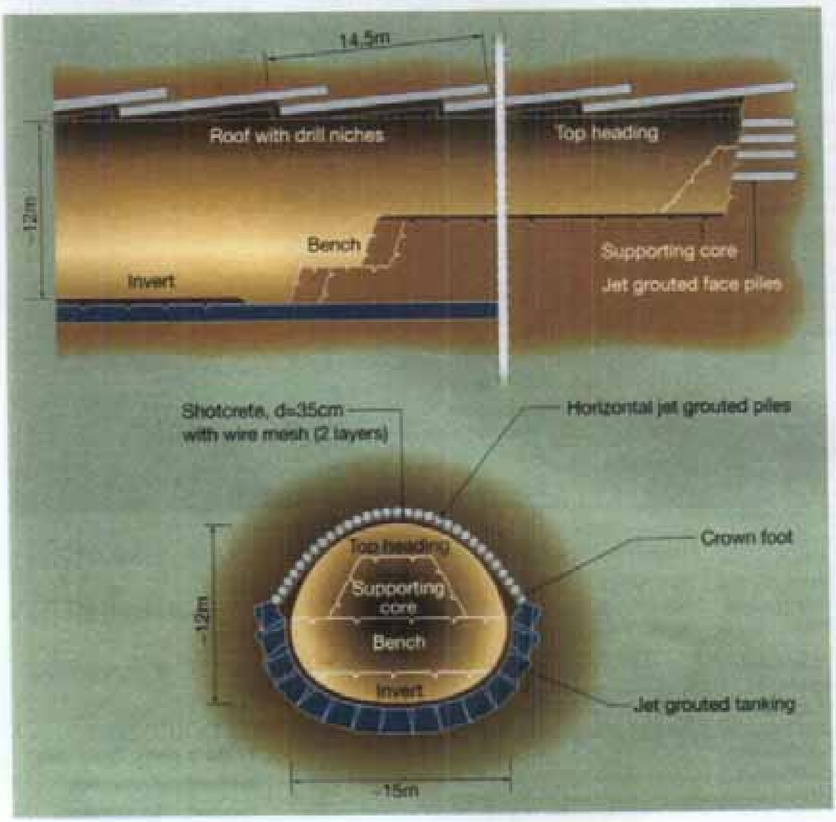


Figure 13, bottom: Section through the construction of the Franfurter Kreuz tunnel under the protection of a jet-grout shield umbrella

Left: Roof beam method being used in the Schulwald tunnel

Bottom: Installation of roof rods (lances) in the Euerwang tunnel

Summary

Using screens as advance support and the roof beam method, possibilities for the economical use of top heading methods are extended. These measures also contribute to increased safety at the face, which is of great importance today, because of the increasing shortage of experienced workers. Resorting to expensive side wall drifts will be confined to difficult conditions where there is high lateral pressure and special demands for settlement control. This way, it is possible to keep conventional tunnelling methods competitive with mechanised tunnelling. TT

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OBJ3/P3/A10

Transport and Works (Inquiries Procedure) Rules 2004

Proposed London Underground (Victoria Station Upgrade) Order

LAND SECURITIES PLC AND OTHERS (Objector No. 3)

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APPENDIX 10

ARTICLE FROM TUNNELS & TUNNELLING 2000

Tunnels for HS railways in Germany

Rupert Sternath, project director for tunnel construction with Deutsche Bahn (German national railways – DBProjekt), describes the context in which a large expansion of the high-performance network is under way. He also describes tunnelling work on one of the big projects, the Cologne-Rhine/Main high-speed link

Above: Compressed air-lock in use in the Siegaue tunnel

Far left: Side wall drift temporary linings indicating stepped construction of the Limburg tunnel

Length in tunnel (km)

1	Karlsruhe-Basel	9
2	Köln-Rhein/main (ohne FA)	41
3	Stuttgart-Augsburg	54
4	Mainz-Mannheim	1
5	Dortmund-Kassel	3
6	Nürnberg-München	25
7	Nürnberg-Erfurt	41
8	Erfurt-Halle/Leipzig	15
9	Hanau-Iphofen	7
10	Knoten Berlin	4
	Total	200

Fig 1, left: Important projects involving tunnelling within the Plan of Demand

DB-Netz, the operator of German railways (DB) runs about 38,500km of railway lines and their structures. The business is aiming for more rail traffic which can be achieved successfully in the long term only by realisation of increased performances and cost reductions[1]. Within its strategy, called Network 21, DB-Netz has set up a business-oriented investment programme to be implemented over 10 years. This includes all maintenance works and any new construction, as well as planned improvements. The programme includes many tunnels and other structures, some to be built under difficult conditions.

In order to ensure that these large projects can be realised without unexpected disruption, but within a given budget framework, there is a need to find new approaches, not only for technology, but also for tendering and contract awards, as well as project control. As a result there will be inevitable changes to the usual project procedures for tunnel construction.

New network

The railway network of DB-Netz is classified into priority, performance and regional lines.

- The priority network connects the great economic areas by two more or less parallel and efficient double-track lines. The fast and slow trains run on the priority lines on different tracks. DB's target is to achieve about 25% of the network in operation in this category by 2010.
- In the performance network the lines will be operated in mixed traffic mode. Bottlenecks will be eliminated and, by means of specific measures of improvement, additional traffic will be able to flow more continuously, thus providing better quality. The performance network will comprise about 25% of the total network by 2010.
- The regional network consists of a 50% share at present, and will also supplement the suburban traffic supply in future[2].

Legal basis

In Germany, the federal government finances 'investments into the railroads of the German federal railways in accordance with the law for the extension of the federal railroads' within the realms of available funds, and effectively for new construction, upgrades and for investment for replacement.

Furthermore, this law establishes the step-by-step extension of the network in accordance with a 'plan of demand', which is an addendum to this law. A component of this is the new construction of the upgrade of railway lines, junctions, new systems for combined railroad traffic, and new inter-modular connections for long-distance traffic to international airports. Aspects of this 'plan of demand' are currently being reviewed by the federal ministry of transportation. In accordance with the law, the financing of all projects must be agreed between DB and the federal government in financing agreements.

On the basis of the 'plan of demand', a 5-year programme will be established for project realisation by the federal ministry of transportation in close accordance with the proposals of DB. The last 'plan of demand' has been replaced with an investment programme for the years 1999-2002 (IP '99-'02). This programme outlines the measures which have priority up to the end of 2002[3].

Projects

DB-Netz is currently working on about 190,000 projects or part-projects, comprising an annual investment of about DM9bn ($4.3bn). The largest current construction projects are the extension of the railway infrastructure in Berlin, the rail-traffic projects called 'German Unity' (listed in the federal plan for traffic) and the new high-speed lines (NBS) of Nuremburg-Ingolstadt-Munich and Cologne-Rhine/Main.

Network 21 defines the order of these measures and so describes the position of DB-Netz for the new federal traffic plan. This is under revision at the federal ministry of transportation for the period up to 2015.

Project companies for the execution of large construction projects have been established as wholly-owned subsidiaries of DB-Netz. These are responsible for the preparation and control of the design, planning, execution and supervision of the construction works. Establishment of these companies will achieve a clear assignment of responsibilities, a lean management organisation with flat hierarchical structures and short internal decision making paths.

Taking DB-Projekt GmbH Cologne-Rhine/Main (PKRM), established in 1996, as an example, here is a brief survey of its project and performance.

NBS Cologne-Rhine/Main

There are DM10bn ($4.5bn) at PKRM's disposal to construct the new 219km-long railway line between Cologne and Rhine/Main (Fig 2). DB-Netz has signed an individual financing agreement with the federal government for the execution of the project which defines the extent of the investment, the sources of finance, the time frame, project descriptions and technical aspects. The project company acts within a project contract of the DB-Netz and handles the project exclusively with all the contractors involved[4]. Due to its central position the new line is one of the most important transport projects in Germany and Europe. It is to be completed by the end of 2001 and in operation in 2002.

The design, as a 300km/h, passenger transport line, makes possible a route with a grade of up to 40:1000. However, this runs closely parallel to the A3 motorway and needs several structures, due to crossing the low mountain ranges Siebengebirge, Westerwald and Taunus. There will be 30 tunnels with a total length of 47km and 18 valley bridges, total length 6km. Constructon of the tunnels can be costly as the geological conditions are relatively difficult.

The new line is a decisive step towards the integration of transport systems, with the two airports Frankfurt/Main and Cologne/Bonn connecting with DB's high-speed railway network.

The NBS Cologne-Rhine/Main will be connected to the Cologne/Bonn airport by a 15.2 km-long branch line (Fig 2). This line will be operated with a mix of high-speed (ICE) and local (S-Bahn) traffic. The federal government, the state of North Rhine-Westphalia and the Cologne/Bonn airport authority have signed an individual financing agreement with an additional DM1.04bn ($466M) and entrusted PKRM with execution of the project. The construction contract was signed in September last year by representatives of DB-Netz AG, DB-Station & Service, Airport Authority Cologne/Bonn GmbH and the Public Transport System Rhine/Sieg. Construction work has started to include about 5.5km of tunnels in open cut[5].

When the new line starts operating in 2002 many short-distance flights will become superfluous. The relevant agreements have already been signed by DB and German airline Lufthansa.

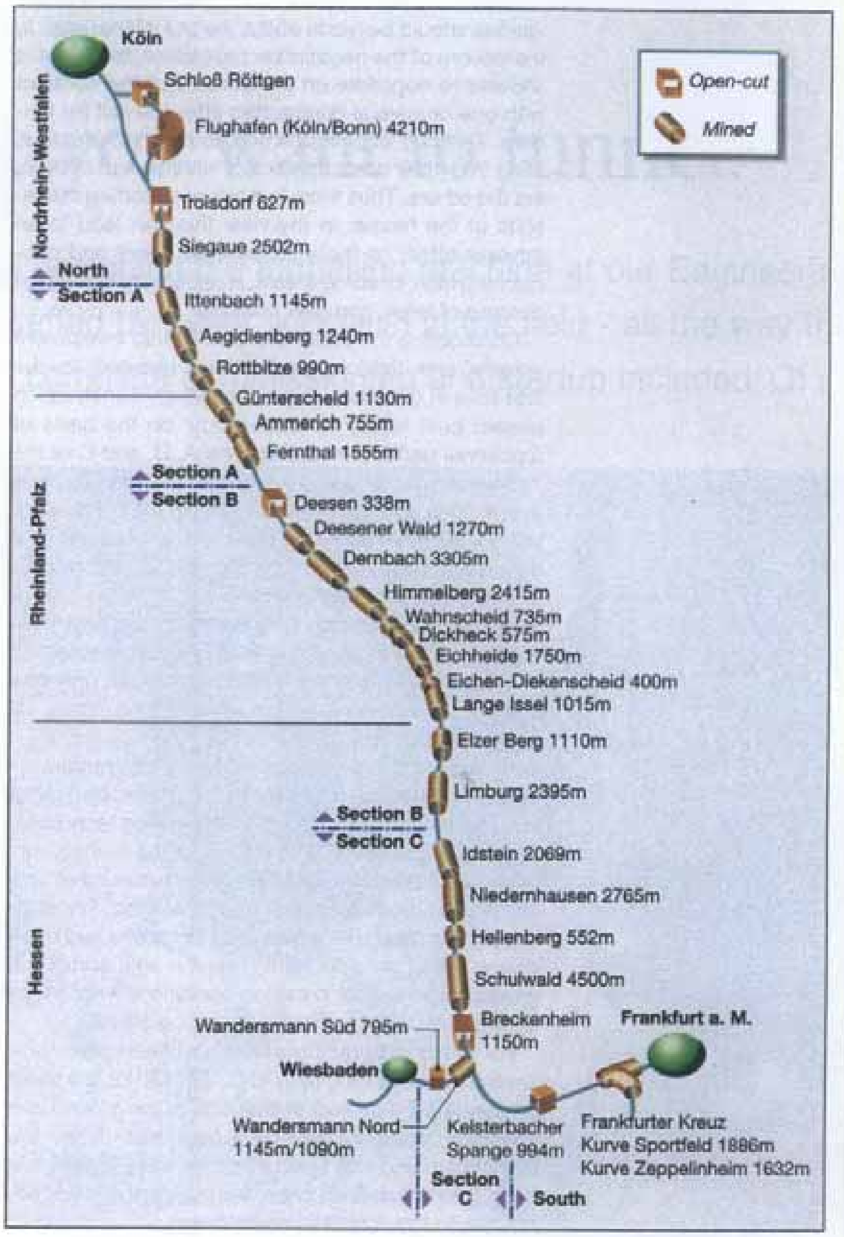


Fig 2. Tunnels on the new NBS Cologne-Rhine/Main route

Tendering procedure

There has been much discussion about tendering procedure, so I would like to limit this to explanations of two different terms:

- 'Performance description with performance programme' in accordance with German and European awarding rules, VOB, part A §9 (10, 11 and 12) – the real function of tendering, and
- award by the 'negotiation procedure' in accordance with the awarding rules section 4 § 3 (2.c) of the EU sector guideline[6,7].

Whilst the former has proved successful on the

Cologne-Rhine/Main project, in our view, some reservations should be made about the use of the latter. In the course of the negotiation procedure, the client is allowed to negotiate on the content of the contract with one or several contractors after the call for tenders. This can lead, in practice, to competitors deviating from their basis for pricing, with the aim of beating the others. Thus there is a risk of distorting the results of the tender. In my view this can lead to an adverse effect on the trust between client and contractor which, to some extent, is necessary in the processing of large, complex projects.

Considering the parameters of 'limited investment volume' and 'tight schedule' it was decided, for the first time in Germany, to have a railway construction project built 'ready for occupancy' on the basis of functional performance. Sections A, B, and C of the 135 km-long intermediate section between Siegburg in the north and the Main crossing in the south were put out to tender all over Europe with a functional tendering procedure. The advantages of this are obvious:

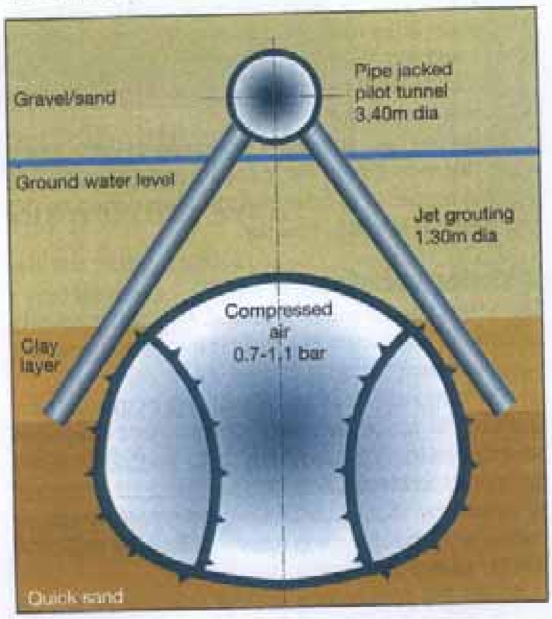
- acceleration of the project;
- easier bidding procedure, awarding and payment;
- use of successful companies' innovation potential;
- award to one general contractor, and so only one contact person;
- price guarantee;
- transfer of tasks of the client to the contractor.

Special attention was paid to the ground investigations for the tunnels relevant to the mining technique. The bidders received a prognosis of the tunnels' excavation classes with which support measures and excavation procedures have been defined. For each excavation class a standard price per metre had to be given. Also, an additional fee for any additional groundwater and for breaks in operations – not of the contractor's responsibility – had to be offered.

For a fair distribution of risk between client and contractor, according to VOB C, DIN 18312, the client guarantees the correct description of the ground and rock conditions, whereas the contractor takes the risk of planning and execution[8]. All work is paid in a lump sum at a defined price; the payment is made according to the construction progress.

Particular care was taken over the correct form of the tendering and awarding procedures on this project, so avoiding objections by competitors about discrimination. As known, the European contract laws are extremely complex, therefore competent legal advice is imperative.

Fig 3. Cross-section through the Siegaue tunnel showing crown ground consolidation by jet grouting



Working design

The double-track tunnels require a break-out profile of 150-160m² with the overburden being generally small to medium. Only the Tunnel Niedernhausen, which passes under the main ridge of the Taunus Hills, has an overburden of 100m. The existing Devonian rock formations of the Rhenish Slate Mountains are deeply weathered and this rock has undergone a later displacement at the surface. The drives therefore require immediate support after excavation.

The working design plans approved by the federal railway authority, Eisenbahn-Bundesamt - EBA, serve as a basis for the assessment of the excavation, and support classes by those responsible on site.

Execution of the working design has been awarded to the contractor, enabling him to respect the working procedures chosen by him in the planning. DB-Guideline No. 853: Railway Tunnel: Design, Constructing and Maintenance, has to be adhered to in the planning.

All the experience gained through construction of more than 200km of railway tunnels during the last 20 years are reflected in this guideline as well as observations of all tunnels operating within the total DB-network[9].

For static's calculations a model has to be chosen for the integrated system, consisting of surrounding rock-mass and support, which reflects real behaviour in nature. The static's calculations gain more importance for tunnels close to the surface as here critical situations occur without warning, and every tunnel is situated in zones where public safety is concerned[10].

The most effective measure for face support in unstable ground conditions has proved to be the use of 8-15m-long face anchors in combination with drilled and grouted steel tube umbrellas. This was used under the Frankfurt-Cologne motorway at the southern end of the Fernthal tunnel. In cases where these measures are not sufficient, the tunnel cross section is divided further (eg side wall drifts, crown drifts).

The sprayed concrete shell is a temporary measure only, so that the steel reinforced inner lining can be placed under well defined conditions. Experience has shown that it is not possible or practical to establish the outer shell in a quality for a durable structure without adversely affecting the mining operation.

Sprayed concrete

More than one million cubic metres of sprayed concrete have been required for the mining operations of the 22 double-track tunnels constructed with shotcrete. A further single-track twin tunnel (Wandersmann Nord) is being constructed, using a shield of 11.5m o.d., whilst seven tunnels are being built by open cut. Best performances have been achieved using the wet-mix sprayed concrete method. Alkali-free accelerators, which can also cope with wet rock conditions, have been developed and tested on site.

In areas where a batch plant could not be kept in constant operation during the night and weekends, dry-mix shotcrete using special accelerated shotcrete mixes and naturally moist aggregates have been applied successfully.

The application of the sprayed concrete is performed regularly by means of semi-automatic hydraulic arms, which allow the operator to control the spraying process by electronic means, thus making sprayed concrete application considerably easier than using previous manual methods[11].

Emergency plan

As a consequence of recent tunnel fires, planning for emergencies has gained great importance, and all tunnels have been upgraded to the standard of the

EBA-guideline, published in July 1997: 'Requirements on fire and catastrophe prevention for the construction and operation of railway tunnels'[12].

On both sides of the tunnel there are escape routes with a minimum width of 1.2m, which lead to emergency exits, each placed a maximum of 1,000m apart. In this way the tunnels are suitable for both self-rescue and rescue from outside.

The tracks outside the tunnels are accessible for rescue teams at every 1,000m. These are described in rescue plans published to meet the requirements of German standard DIN 14 095.

The rescue teams and the railway personnel have to undergo a training programme before the new railway line is put into operation. These programmes have to be repeated every year so that everyone is prepared for any emergency that may occur.

Geology

The route of the new line Cologne-Rhine/Main in the section north and south runs in Quarternary strata, and in the intermediate section it crosses the low mountain ranges of the Rhenish Massif. The Devonian slates found within this structure have been greatly stressed tectonically.

During the Mesozoic period the rock surface was exposed to a long weathering in a subtropical climate. Volcanic activity in the early Tertiary disturbed the formation even more, and this has consequences for tunnel driving, eg the problem of close proximity of basalt pipes. In historic times, mining and other human intervention has loosened the ground further.

The water table is usually close to the surface.

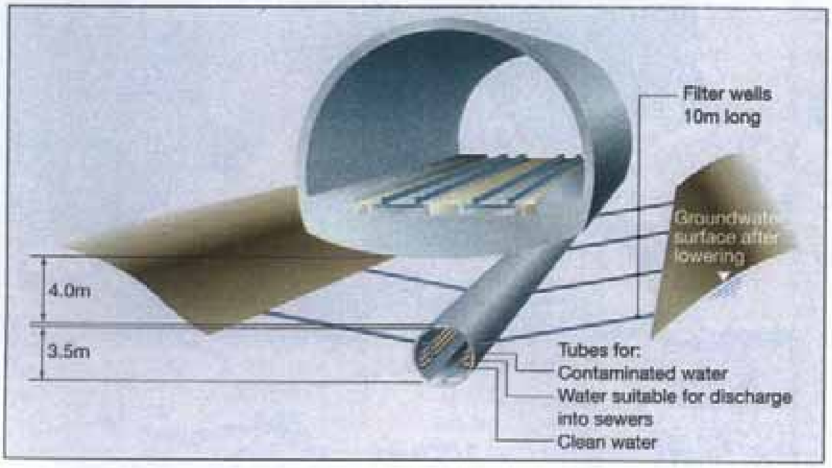


Fig 4. Groundwater collection and treatment system in the Fernthal tunnel

Tunnel construction

Some of the tunnel structures are particularly interesting, including tunnels with little cover, those in settlement-sensitive areas, and those which, due to their length, have to be qualified as critical in the time schedule[13].

The 1,555m-long Fernthal tunnel passes under a closed rubbish dump within its 400m-long middle section. In the course of hydrogeologic investigations slight contamination of the groundwater beneath the dump was discovered.

Prior to starting mining operations in this area, a groundwater cleaning device (Fig 4) had to be installed to purify the groundwater until permanent purification is proven. The surface of the dump is being sealed by a combined mineral and plastic cover. The polluted water from the dump is being pumped out thoroughly in advance.

The 2,502m-long Siegaue tunnel, in its 370m-long middle section, passes under a cultural community centre, leads closely past a Romanesque church and passes under a cemetery to the north.

Due to these local conditions the surface is inaccessible and so the tunnel must be built by mining in very adverse ground conditions. These comprise water-bearing loose Quarternary and Tertiary layers of gravel, quicksand, and clay, which connect into the nearby Sieg River. The 220m southern part of the tunnel was mined under protection of a groundwater lowering system consisting of deep bored wells, drilled out of a 3.4m o.d. pilot tunnel, driven by pipe jacking above the planned crown of the main tunnel.

In the last 150m of the northern section the water ingress was so strong that the quicksand and gravel layers became unstable. Therefore a jet-grouted roof was formed from the pilot tunnel with the aim of guarding the crown from water ingress from the over lying gravel (Fig 3). Mining continued after installation of compressed air equipment for pressures of 0.7-1.1bar. In this way, advance rates of 1m per day are being achieved, so that breakthrough can be expected this month (October).

Reinforced inner concrete linings

Reinforced concrete linings form the final structure, and have to meet highest demands in respect of water tightness and durability. The automatic control devices for the signalling alone do not allow wet conditions in the tunnels, all of which are situated below groundwater. This is allowed to be lowered temporarily only during construction, and must recover to its original level after completion. The inner linings are therefore designed for full groundwater pressures.

Water tightness can be achieved at:

- groundwater levels <30m above the lowest edge of the tunnel structure by application of watertight concrete, the reinforcement of which has to be designed for special crack minimising procedures. The maximum length of each single block in this case is limited to 10m;
- groundwater levels >30m by placing plastic sealing sheets. In this case the maximum block length must not exceed 12.5 m[14].

Currently no less than 22 steel travelling formwork structures are in operation along the whole route, concreting one block each day. Logistical problems encountered on the supply of material, equipment and qualified personnel are immense, but generally are being satisfactorily solved.

Trackwork

On a length of 192.8km, where the design speed exceeds 200km/h, slab track is being installed instead of conventional ballasted track.

This entails forming a monolithic block whereby the sleeper/rail grid is supported inside a concrete trough and subsequently cast *in situ*, thus creating a sound rail permanent way known as the Rheda system, named after the town where it was first installed some 30 years ago. Works for this operation started in April this year and have to be completed by the end of September 2001.

Experiences

Mining operations on this project are nearly finished. The motorway BAB A 3 Frankfurt-Cologne has been successfully crossed under 14 times in difficult conditions and without traffic disruption, using mining methods.

Besides this, many additional traffic routes, industrial plants, buildings for supermarkets and leisure centres have been passed under without disturbances to business or residential buildings in Ammerich, Aegidienberg and Ittenbach. All these challenges have been mastered brilliantly.

Hopefully, the experience gained on this project can be implemented on further large-size projects in the near future.

Conclusion

Construction methods in heavy civil engineering have reached an astonishingly high level of development. Today tunnels can be built under preconditions which could not be considered just 20-30 years ago.

This should not lead engineers to believe that natural circumstances can be ignored when choosing the alignment for a new infrastructure project. It is during this early stage that future delays and increases in costs can become, as it were, pre-programmed by the decisions made.

But the main problems with the realisation of new projects today lie, on the one hand, in the area of permission procedures – meaning the persuasion of all those concerned with the project – and, on the other, in keeping costs within the preliminary set budget.

At the beginning of railway construction in Germany the euphoria for this new technology was high. In 1835, the initial public share offer totalled 1.5M thaler (predecessor to the DM) for the first 110km of long-distance railway line from Leipzig to Dresden.

This was oversubscribed in just two days, but it soon became obvious that the actual profits were significantly below expect-ations.

This, coupled with the difficulties encountered during the planning and construction phases, and in the preparatory phase for the operation of other lines, led to a noted fall of enthusiasm to invest in this sector[15].

Unfortunately we have learnt little about this matter during the past 135 years. TnT

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APPENDIX 11

ARTICLE FROM TUNNELS & TUNNELLING 2002

Construction of Rio de Janeiro's Metro Line 1 extension has involved tunnelling under multi-storey buildings with minimal cover. Luis Sozio of the contract's detailed design team, Promon Engenharia, describes how a combination of horizontal and vertical jet grouting ensured minimal settlement along the tricky alignment

Jet setting in RIO

The Rio de Janeiro Metro system consists of two lines, totaling 35km, that transport some 400,000 passengers daily. Work is currently progressing at Line 1's south end, to extend it further south into the Copacabana District, one of the most densely populated areas in Brazil.

The extension's first stage, planned in the early 80's, was awarded in 1987 on a unit price basis to local contractor Construtora Andrade Gutierrez. At that time the Contractor was only authorised to build Arcoverde Station.

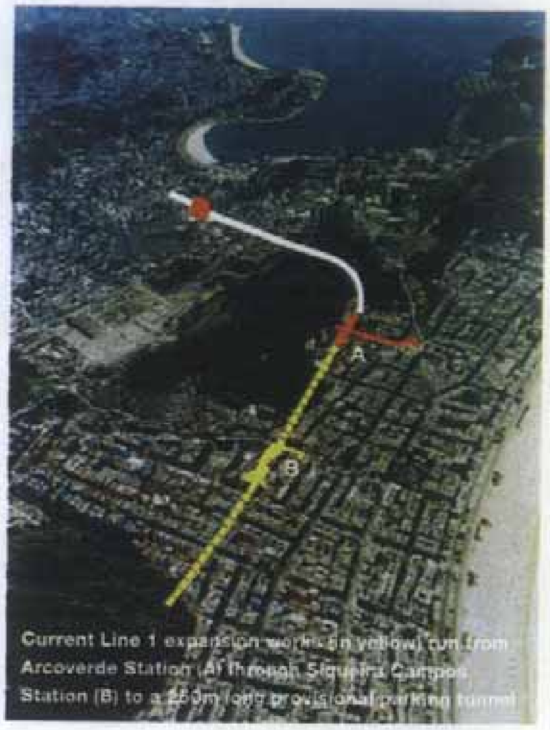
The estimated US$80M new phase, being built by the same contractor and discussed here, was authorised in December 1999 by the contract client, Rio Trilhos Metrô Do Rio De Janeiro, and includes the following main structures;

- The 150m long Siqueira Campos Station at the heart of the Copacabana district. The station will be one of the three busiest stations in Rio, and is expected to provide access to a further 150,000 passenger a day to the metro system. It is scheduled to start operation in December 2002;
- A 180m long, double section (each 6m diameter) tunnel, linking the operating Arcoverde Station to the new Siqueira Campos Station;
- A 250m long tunnel of the same dimensions, beyond Siqueira Campos Station, which will serve as a provisional train parking tunnel, until Phase 2, which will extend Line 1 into the Ipanema District. Construction access for this tunnel is through a temporary shaft, located at Garibaldi Street, at the middle length of this tunnel.

Geology

The existing Arcoverde Station is located in a sound Gneiss rock hill formation and was excavated by drill and blast. The 180m tunnel starts in this rock and progresses from a freshly weathered rock to a completely decomposed residual soil. This is a silty and clayey sand, which loses strength when exposed at an excavation face and when subjected to groundwater percolation. As it approaches Siqueira Campos, the tunnel traverses soft alluvium soils (mixed sand and marine organic clay layers). The groundwater level is always above the tunnel.

Some high rise buildings are found above the tunnel, two of them on shallow foundations in the alluvium, about 6m above the tunnel crown, with damage due to settlement from their own weight.



Current Line 1 expansion works (in yellow) run from Arcoverde Station (A) through Siqueira Campos Station (B) to a 250m long provisional parking tunnel

Below: View from Siqueira Campos' bottom slab at the tunnel portal

From Siqueira Campos through the 250m parking tunnel, the soil consists exclusively of alluvium, made of cohesionless marine sands and soft organic clay layers, again, below groundwater level.

At the southern end of Siqueira Campos Station part of the tunnel section is directly below a shallow foundation of a 12 storey building, with 5m cover to tunnel crown.

The surface topography changes from a hilly profile near Arcoverde Station, to a flat terrain near Siqueira Campos, and along the 250m parking tunnel. Ground cover above the tunnels varies from some 30m, near Arcoverde, in rock, to 6m, in alluvium, over the parking tunnel.

Construction methods

Siqueira Campos is 16m deep and is being excavated as a top down sequence (cover and cut), with 80cm thick diaphragm walls acting as retaining walls, braced by the top slab, mezzanine slab and bottom slab.

The Station inner columns are temporarily formed by steel profiles founded on diaphragm wall panels, below the bottom slab. These steel profiles are lined with concrete, after casting of the bottom slab.

Two 15m diameter shafts were excavated at both Station ends to allow tunnelling to start from two fronts without having to wait for Station excavation to reach the bottom slab, after casting of the top and mezzanine slabs. In fact, the Station excavation reached the bottom slab some 12 months after tunnelling began.

A short 20m stretch near Arcoverde Station was excavated entirely by drill and blast with support provided by rock anchors and sprayed concrete. This condition changed to a mixed face, where the rock and soil interface gradually dips towards the tunnel invert.

The design concept first adopted for the tunnels in soil was based on a closed face EPB or slurry shield. The difficulty in stabilising cohesionless sands and soft clays below groundwater level, such as in the alluvium, the lack of available space at surface level, and the need to excavate very close to building foundations reinforced this selection.

However, the client, Rio Trilhos Metrô Do Rio De Janeiro, abandoned the shield technique after a significant devaluation of the local currency in the first half of 1999 resulted in unacceptable cost increases as the equipment needed to be imported from abroad. Other factors also contributed to this decision: the time period between the equipment order and actual excavation start was too long to fit the schedule. Also the shield would have to excavate a mixed face with a high strength; very abrasive rock at the bottom and cohesionless soil at the top. That would not rule out a shield but would imply specifically designed equipment, adding cost and time compared to a conventional EPB Shield.

The alternate method chosen was a sprayed concrete lined tunnel in a previously treated soil by jet grouting. The tunnel was basically hand excavated, with the aid of portable percussion tools. Soil removal was via a backhoe excavator and small size dumper trucks.

The jet grouting concept was designed as a continuous application at the whole excavation perimeter (except where the bottom of excavation was in rock, near Arcoverde), and through the excavation face area, forming 2m thick bulkheads at intervals ranging from 7m to 10m, to prevent major face instabilities.

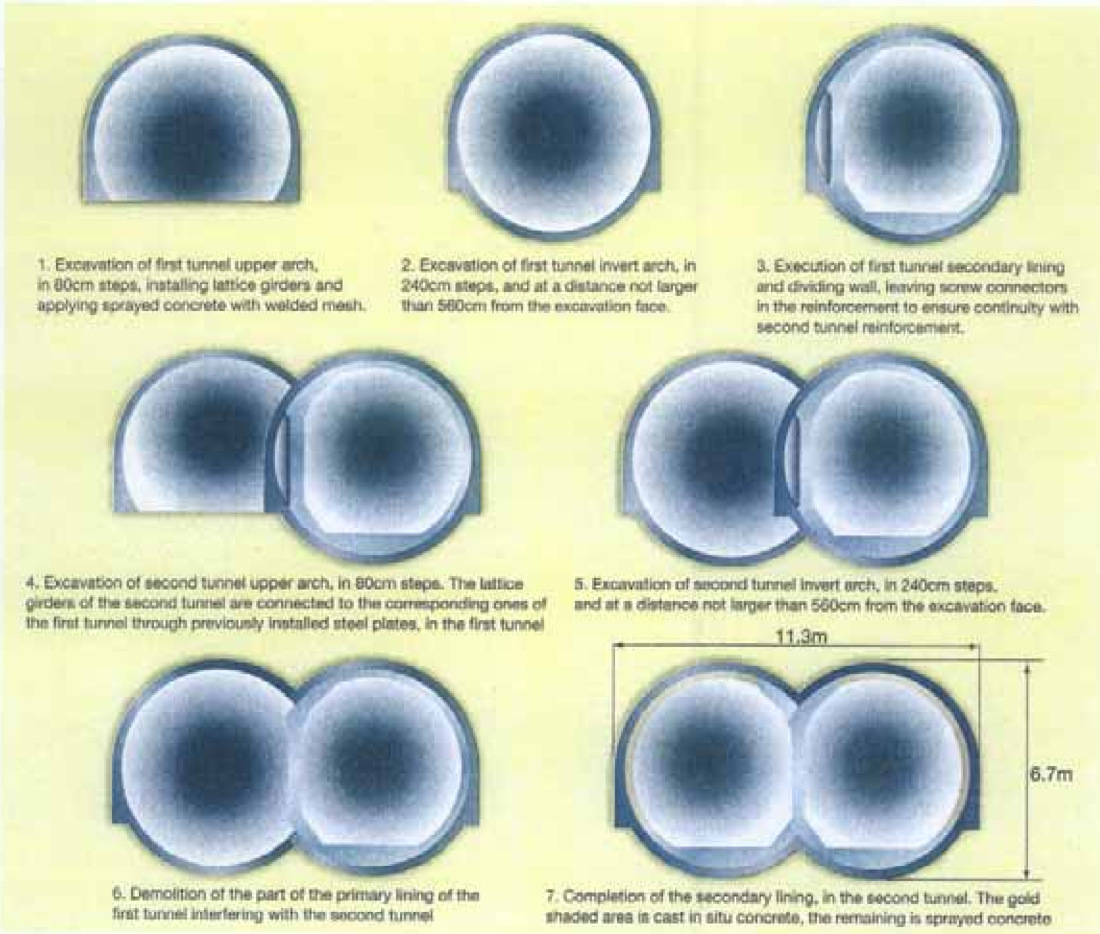
Since crossovers were not required, a twin section with dividing wall between tracks was found to be more economic, for soil treatment, than a conventional double track single section. Also, the double track, single section would be far too wide to permit excavation in one phase, requiring a side drift and enlargement, not dissimilar to a twin section but requiring a larger excavation area and perimeter.

Lattice girders and welded mesh were used with sprayed concrete to form the 25cm thick primary tunnel lining. Cast in situ and sprayed concrete were used to form the 25cm thick secondary lining.

Horizontal and vertical Jets

For tunnel stretches where access from the surface level was either impossible or unacceptable the horizontal single jet grouting system was selected; consisting of near horizontal columns, 30cm to 45cm in diameter, injected from the tunnel face at regular intervals. The total length of injected column, per advance, ranged from 1200m to 2000m (when including the length required to form the bulkhead).

The horizontal jet grouting technique was applied for the 180m tunnel between Arcoverde and Siqueira



1. Excavation of first tunnel upper arch, in 80cm steps, installing lattice girders and applying sprayed concrete with welded mesh.

2. Excavation of first tunnel invert arch, in 240cm steps, and at a distance not larger than 560cm from the excavation face.

3. Execution of first tunnel secondary lining and dividing wall, leaving screw connectors in the reinforcement to ensure continuity with second tunnel reinforcement.

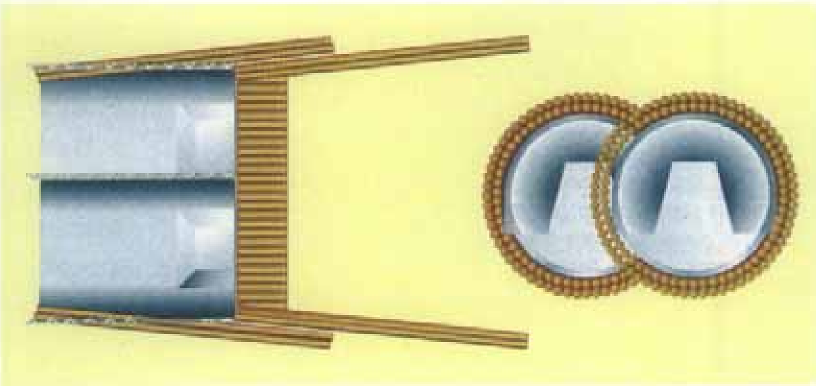
4. Excavation of second tunnel upper arch, in 80cm steps. The lattice girders of the second tunnel are connected to the corresponding ones of the first tunnel through previously installed steel plates, in the first tunnel

5. Excavation of second tunnel invert arch, in 240cm steps, and at a distance not larger than 560cm from the excavation face.

6. Demolition of the part of the primary lining of the first tunnel interfering with the second tunnel

7. Completion of the secondary lining, in the second tunnel. The gold shaded area is cast in situ concrete, the remaining is sprayed concrete

The tunnel construction sequence employed on Rio's Line 1 extension



Above: Plan and section of the horizontal jet grouting pattern

Right: Section through a vertical jet grout

Campos, and for a 30m stretch at the southern end of Siqueira Campos, to enable crossing under Figueiredo Magalhães street, which is heavily occupied by underground public services, and also below the 12 storey building mentioned earlier.

For most of the 250m parking tunnel, a solution based on vertical and near vertical jet grouting (double jet system; grout and compressed air) was defined. The Column diameter was in the range 120cm – 180cm with the grout pressure before the injecting nozzle being 40MPa, enveloped by a 0.8MPa air pressure. The basic vertical jet grouting array consists of 13 holes, totaling 130m of injected column per metre of double tunnel,

The advantage of the vertical jet grouting system is that it enables uninterrupted tunnel advance. The horizontal system requires a complete stop of tunnel excavation, at 7m to 10m intervals, slowing tunnel advancement rates considerably.

Tunnel starting fronts

Five tunnel excavation fronts were selected. The first was adjacent to Arcoverde Station, towards Siqueira Campos. Since there was no space available for a surface construction site, all the supply of equipment and materials as well as mucking out were done through the Line 1 track to a depot 12km away, during metro non-operating hours.

At Siqueira Campos two shafts were built enabling simultaneous tunnelling fronts towards Arcoverde and the parking tunnel. A shaft, offset from the tunnel axis, was created at Garibaldi Street where the 250m parking tunnel is roughly divided in two. A tunnel transverse to the track tunnels was excavated from this shaft, allowing two portals to be created, one in direction of Siqueira Campos, and the other towards the end of the parking tunnel.

Horizontal jet grouting injection of a bulkhead. The yellow pipe at the drilling mouth is connected to the 'preventer'

Difficulties and adjustements

Soil loss whilst drilling the horizontal jet grout columns was experienced due to high groundwater pressure at the drill hole. This was aggravated by a lack of cohesion in the sand, resulting in excess soil being flushed out during drilling at the first jet grouting trials, leading to a potentially unacceptable settlement at surface and on buildings.

The horizontal jet grouting sub contractor, Novatecna, implemented a pressure control valve, called a 'preventer', at the drill hole mouth, sealed by a sprayed concrete layer at the tunnel face, and used it to control soil flushing during drilling as well as grout return during injection.

The settings of the preventer were adjusted as a function of settlement readings taken by local sub contractor, Technosolo, at deep and surface settlement points, and at building columns. The preventer was gradually adjusted to shut or open positions, depending upon whether the readings indicated settlement or heave during the process.

The readings were based on high precision topographic leveling during drilling and injection of the uppermost columns, with key settlement points leveled at intervals not exceeding 5 minutes.

The double jet system, being carried out by Brasfond, consists of a simultaneous injection of grout and compressed air, through a concentric nozzle. The high potential energy of the grout fluid (at a pump pressure of 40MPa) is converted to kinetic energy after passing through the nozzle, so that this pressure is not transmitted to the ground.

This mechanism does not occur with a compressible fluid like air and in several instances air bubbles were observed to emerge at surface or through basement slabs, at distances as far as 30m from the column being injected. This was usually linked to heave of shallow foundation buildings by as much as 15mm. Deep foundation buildings were hardly affected by jet grouting heave.

At selected buildings, which were already in a precarious state, and whenever heave was recorded or air bubbling was observed, the jet grout column array was redesigned to substitute the double jet system columns for single jet columns (the compressed air was turned off), about 70 to 90cm in diameter (compared to 120cm to 180cm for the double system). This procedure was very effective in preventing the development of heave.

The jet grout was used firstly to provide a zone of high strength soil, protecting the tunnel excavation; the resulting unconfined compressive strength of the treated soil cores was in the range 2MPa for organic clay, to 5MPa for pure sand. This equaled or exceeded the minimum specified value of 2MPa.

The second main function of the treatment was to maintain dryness, since a defect in the treated zone could allow uncontrollable sand and water inflows into the tunnel (a phenomenon known as piping).

In order to create an impervious chamber, the jet grouting was designed to fully envelope the excavation and provide a bulkhead sealing across the tunnel section at selected intervals, which in practice varied from 7m to 10m.

This geometry was simple to implement in the vertical system, but for the horizontal system the numbers of columns necessary to create an envelope and a bulkhead was such that the treatment of a single chamber lasted for 3 to 4 weeks. Therefore, in the horizontal system the envelope and bulkhead concept was used strictly in alluvium soil, for about 60m in the Arcoverde to Siqueira Campos tunnel.

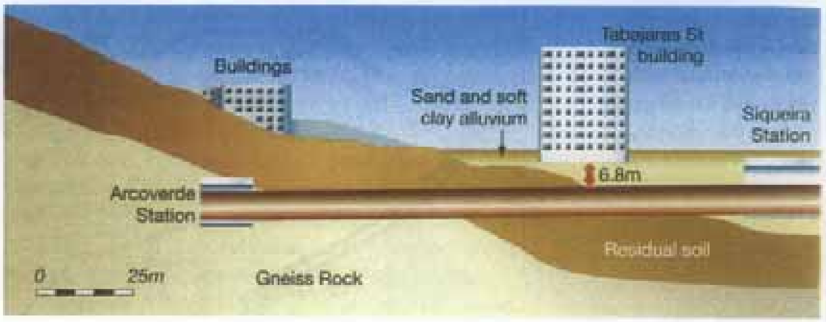
About 100m of this tunnel was in residual soil. Horizontal vacuum drains were employed to relieve groundwater pressure and simplify and reduce the horizontal jet grouting quantities to about 50% of a full treatment section. Being a stiff material, the residual soil responded well to the effective stress increase, contrary to a soft alluvium.

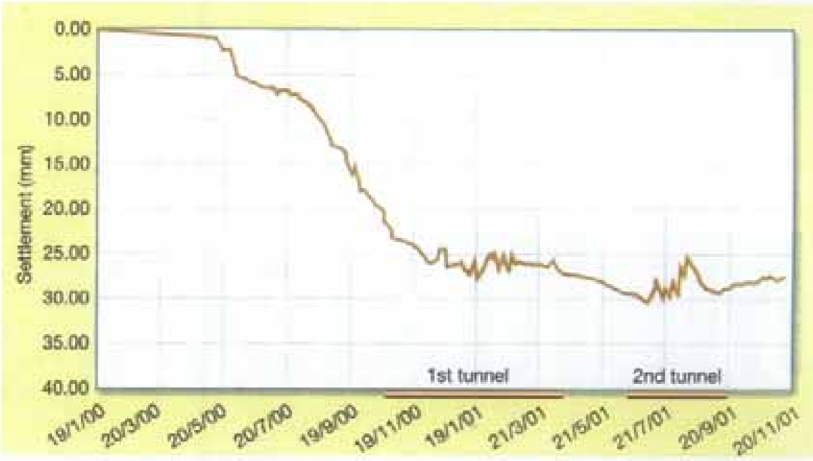
Experience in other tunnelling works in Brazil with jet grouting indicated that the requirement of a watertight tunnel excavation cannot statistically be achieved to an acceptable confidence level. Sealing a jet grouting treatment zone with such materials as silicates and resins was analysed but abandoned due to excessive costs and time consumption.

The design concept adopted was to install groundwater lowering wells from surface level, and operate if and when necessary. It was considered preferable to allow a moderate consolidation settlement than to risk a collapse. The wells were 60cm in diameter and sunk to 6m to 8m below the tunnel invert level. The wells were equipped with submersible pumps, being automatically switched on and off by electric sensors placed at 1m and 2m elevations above the pump. Their average pumping flow was in the range 1m/hr - 5m/hr.

Horizontal drains were drilled for dewatering each chamber before excavation. If flow continued, a leakage was thought likely through the treated zone. The groundwater wells were then turned on to avoid risk of piping, and excavation proceeded to that specific chamber. Some 60% of the installed wells were operated in that way and this allowed safe excavation in the partially "defect" chambers.

The resulting maximum consolidation settlement was 20mm or less (the soft clay was slightly pre consolidated due to ageing and possibly dry season water lowering). This consolidation settlement was evenly distributed (low distortion), thus inducing only minor damage on some building's brick walls.





Top: The tunnel alignment passes perilously close to the high rise bulding foundations

Above: The jet grouting proves successful as shown in the settlement figures

Excavation under buildings

Of the buildings that were directly above the tunnel, two cases should be highlighted. The buildings at 14 and 20 Tabajaras Street are 10 storeys high, built in the '50's, with shallow foundations in alluvium, each 6m above the tunnel crown. Both presented a damage record, due to the weak condition of their foundations.

Underpinning of these buildings was not possible since this would temporarily force the inhabitants out of the building, which was not acceptable.

These buildings were densely instrumented, and settlement monitoring was carried out at 12 hour intervals during excavation. Some selected points were read at 5 minutes intervals during drilling and injection of the upermost jet grout columns.

The experience gained through the advancing tunnel fronts and the control of the horizontal jet grouting operations in alluvium indicated that the passage under these buildings could be safely effected. In fact, by controlled use of the preventer, the jet grouting operations did not induce settlement at all. A slight heave of a few millimetres was observed during injection. The final settlement observed on these buildings was some 25mm, almost entirely due to consolidation settlement induced by a groundwater lowering well nearby.

Left: A completed section of the tunnel showing the dividing wall with arches

Progress to date

The Siqueira Campos Station top, mezzanine and bottom slabs are cast and the columns lined. The 180m tunnel linking Arcoverde Station to Siqueira Campos is complete, including secondary lining. Of the 250m parking tunnel, about 120m, from Siqueira Campos to the Garibaldi Shaft are completed.

Some 130m of tunnel remains to be excavated, from the Garibaldi shaft towards the end of the parking tunnel. This can be done independently from Siqueira Campos, which will allow an early operation date for the atation, scheduled for the end of 2002.

The remaining tunnel section is expected to be complete in June 2003.

OBJ3/P3/A12

Transport and Works (Inquiries Procedure) Rules 2004

Proposed London Underground (Victoria Station Upgrade) Order

LAND SECURITIES PLC AND OTHERS (Objector No. 3)

PROOF OF EVIDENCE of TIM CHAPMAN of ARUP

APPENDIX 12

INFERRED VSU/PAL PROGRAMME

VSU CONSTRUCTION PROGRAMME

(INFERRED FROM SES ANNEX V TABLE 1-2)

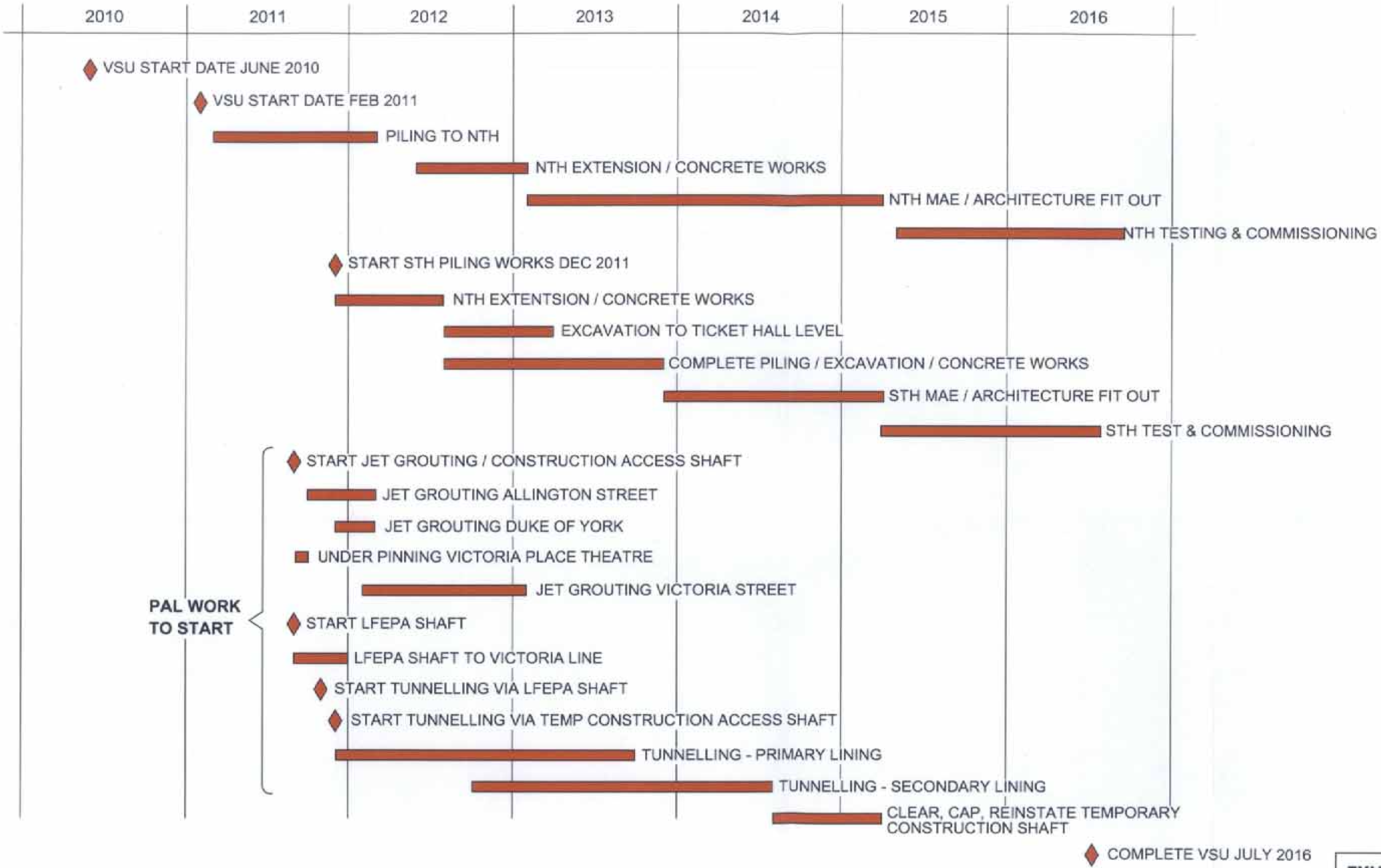


EXHIBIT OBJ3/P3/A12
Evidence of Tim Chapman
Land Securities

Inferred VSU/PAL Programme

OBJ3/P3/A13

Transport and Works (Inquiries Procedure) Rules 2004

Proposed London Underground (Victoria Station Upgrade) Order

LAND SECURITIES PLC AND OTHERS (Objector No. 3)

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APPENDIX 13

BS5228 PART 4 1992 – NOISE CONTROL ON CONSTRUCTION AND OPEN SITES

BRITISH STANDARD

BS 5228-4:
1992

Incorporating Amendment No.1

Noise control on construction and open sites —

Part 4: Code of practice for noise and vibration control applicable to piling operations

Committees responsible for this British Standard

The preparation of this British Standard was entrusted by the Basic Data and Performance Criteria for Civil Engineering and Building Structures Standards Policy Committee (BDB/-) to Technical Committee BDB/5, upon which the following bodies were represented:

Arboricultural Association
Association of District Councils
Building Employers Confederation
Department of the Environment (Property Services Agency)
Federation of Piling Specialists
Incorporated Association of Architects and Surveyors
Institute of Clerks of Works of Great Britain Incorporated
Institution of Environmental Health Officers
Landscape Institute
Royal Institute of British Architects
Scottish Office (Building Directorate)
Trades Union Congress
Association of County Councils
Association of Metropolitan Authorities
Construction Health and Safety Group
Federation of Civil Engineering Contractors
Health and Safety Executive
Institute of Building Control
Institution of Civil Engineers
Institution of Structural Engineers
National Council of Building Material Producers
Royal Institution of Chartered Surveyors
Society of Chief Architects of Local Authorities

The following bodies were also represented in the drafting of the standard, through subcommittees and panels:

Association of Consulting Engineers
British Coal Corporation
Concrete Society
Construction Plant (Hire Association)
Federation of Dredging Contractors
Sand and Gravel Association Limited
British Aggregate Construction Materials Industries
British Compressed Air Society
Department of the Environment (Building Research Establishment)
Institution of Highways and Transportation
Society of Motor Manufacturers and Traders Limited

This British Standard, having been prepared under the direction of the Basic Data and performance Criteria for Civil Engineering and Building Structures Standards Policy Committee, was published under the authority of the Standards Board and comes into effect on 1 May 1992

First published January 1986
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The following BSI references relate to the work on this standard:
Committee reference BDB/5
Draft for comment 90/14249 DC

ISBN 0 580 20381 6

Amendments issued since publication

Amd. No.	Date	Comments
7787	July 1993	Indicated by a sideline in the margin

Contents

Foreword

This part of BS 5228, Which has been prepared under the direction of the Basic Data and Performance Criteria for Civil Engineering and Building Structures Standards Policy Committee, covers the control of noise and vibration from piling sites, and is a revision of BS 5228-4:1986, which is withdrawn.

The standard refers to the need for the protection of persons living and working in the vicinity of such sites and those working on the sites from noise and vibration. It recommends procedures for noise and vibration control in respect of piling operations and aims to assist architects, contractors and site operatives, designers, developers, engineers, local authority environmental health officers and planners, regarding the control of noise and vibration.

Vibration can cause disturbance to processes and activities in neighbouring buildings, and in certain circumstances can cause or contribute to building damage.

Vibration can be the cause of serious disturbance and inconvenience to anyone exposed to it. The Control of Pollution Act 1974, the Environmental Protection Act 1990 and, in Northern Ireland, the Pollution Control and Local Government (Northern Ireland) Order 1978, which define "noise" as including "vibration" (Section 73(1) of the 1974 Act, Section 79(7) of the 1990 Act and Article 53(1) of the 1978 Order), contain provisions for the abatement of nuisances caused by noise and vibration.

It should be noted that BS 6472 covers the human response to vibration in structures and BS 7385-1 covers the measurement and evaluation of structural vibration. An item dealing with the vibratory loading of structures is being processed within ISO/TC 98/SC 2 "Safety of Structures". This is being monitored by BSI.

BS 5228 consists of the following Parts:

— *Part 1: Code of practice for basic information and procedures for noise control;*

— *Part 2: Guide to noise control legislation for construction and demolition, including road construction and maintenance;*

— *Part 3: Code of practice for noise control applicable to surface coal extraction by opencast methods;*

— *Part 4: Code of practice for noise and vibration control applicable to piling operations.*

BS 5228-1 is common to all the types of work covered by the other Parts of BS 5228, which should be read in conjunction with Part 1.

Other Parts will be published in due course as and when required by industry.

Attention is drawn to the Control of Pollution Act 1974 (Part III) (Noise), the Environmental Protection Act 1990 (Part III) (Statutory Nuisances and Clean Air), the Health and Safety at Work etc. Act 1974 (in Northern Ireland, the Pollution Control and Local Government (Northern Ireland) Order 1978 and the Health and Safety at Work (Northern Ireland) Order 1978), and to the Noise at Work Regulations 1989, Statutory Instrument 1989 No. 1790.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to iv, pages 1 to 62, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

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Section 1. General

0 Introduction

This Part of BS 5228 is concerned with all works associated with piling operations on sites where temporary or permanent foundation or ground stability requirements are to be met by the installation of piles by any of the recognized techniques (see **7.2**). In common with other mechanized construction activities, piling works pose different problems of noise and vibration control from those associated with most types of factory-based industry for the following reasons:

a) they are mainly carried out in the open;

b) they are of temporary duration, although they may cause great disturbance while they last;

c) the noise and vibration they cause arise from many different activities and kinds of plant, and their intensity and character may vary greatly at different phases of the work;

d) the sites cannot be excluded by planning control, as factories can, from areas that are sensitive to noise.

Increased mechanization has meant the use of more powerful and potentially noisier machines. It is now widely recognized that noise levels that can be generated are unacceptable in many instances and that reductions are desirable for the benefit of both the industry and the public. Piling works frequently form one of the noisier aspects of construction. The trend towards medium and high rise structures, particularly in urban areas, coupled with the necessity to develop land which was hitherto regarded as unfit to support structures, has led to increasing use of piled foundations. Piling is usually one of the first activities to be carried out on site, and special precautions should be taken to mitigate the disturbance created, particularly in sensitive areas.

If a site upon which construction or demolition work will be carried out involves an existing operational railway, special features that are significant in relation to noise and vibration control have to be taken into account. Advice should be sought in such cases from the appropriate railway authorities.

Because of the variable nature of vibration transmission characteristic of soils, rocks and structure, the prediction of vibration levels is a less precise science than the corresponding prediction of air-borne noise levels. Whilst data obtained from various sources are included for illustrative purposes, any predictions based thereon for specific circumstances should ideally be verified by appropriate field measurements.

1 Scope

This part of BS 5228 supplements the information given in BS 5228-1, with information especially relevant to piling works. It sets out recommendations for noise and vibration control measures which can be adopted to ensure good practice and enable piling to be carried out economically with as little disturbance to the community as is practicable.

Section 2 contains recommendations relating to noise control. Section 3 contains recommendations for the mitigating of the effects of ground-borne vibration.

NOTE 1 This Part of BS 5228 should be read in conjunction with BS 5228-1.

NOTE 2 The titles of the publications referred to in this standard are listed on the inside back cover.

2 Definitions

For the purposes of this Part of BS 5228, the definitions given in BS 5228-1 apply together with the following.

2.1
amplification factor

the motion measured at a given point (usually on the structure) divided, by the motion measured at a reference point (usually at the base of the structure or on the foundation)

2.2
peak particle velocity (p.p.v.)

the maximum value of particle velocity obtained during a given interval

2.3
piling

the installation of bored and driven piles and the effecting of ground treatments by vibratory, dynamic and other methods of ground stabilization

3 Legislative background

Attention is drawn to the following legislation, current at the date of publication of this Part of BS 5228.

a) Control of Noise (Appeals) (Scotland) Regulations 1983.

b) Control of Noise (Appeals) Regulations 1975.

c) Statutory Nuisance (Appeals) Regulations (as amended) 1990.

d) Control of Noise (Appeals) Regulations (Northern Ireland) 1978.

e) Control of Pollution Act 1974.

f) Environmental Protection Act 1990.

g) Health and Safety at Work etc. Act 1974.

h) Health and Safety at Work (Northern Ireland) Order 1978.

i) Land Compensation Act 1973.

j) Land Compensation (Scotland) Act 1973 (in Northern Ireland, the Land Acquisition and Compensation (Northern Ireland) Order, 1973).

k) Noise Insulation Regulations 1975 (in Scotland, the Noise Insulation (Scotland) Regulations 1975).

l) Pollution Control and Local Government (Northern Ireland) Order 1978.

m) Public Health Act 1961.

The Control of Pollution Act 1974, the Environmental Protection Act 1990 and, in Northern Ireland, the Pollution Control and Local Government (Northern Ireland) Order 1978 SI 1049, which define noise as including vibration (Section 73 (1) of the 1974 Act, Section 79(7) of the 1990 Act and Article 53(1) of the 1978 Order) contain provisions for the abatement or cessation of nuisances caused by noise and vibration.

4 Guidance notes on legislation

4.1 General

This information on procedures is given for guidance purposes only and attention is drawn to the relevant Acts.

4.2 The Control of Pollution Act 1974

The Control of Pollution Act 1974 gives local authorities powers for controlling noise and/or vibration from construction sites and other similar works. These powers may be exercised either before works start or after they have started. In Northern Ireland, similar provision is made in the Pollution Control and Local Government (Northern Ireland) Order 1978. Contractors, or persons arranging for works to be carried out, also have the opportunity to take the initiative and ask local authorities to make their noise and/or vibration requirements known. Because of an emphasis upon getting noise and/or vibration questions settled before work starts, implications exist for traditional tender and contract procedures (see **4.5**).

4.3 Notices under Section 60 of the Control of Pollution Act 1974

Section 60 enables a local authority, in whose area work is going to be carried out, or is being carried out, to serve a notice of its requirements for the control of site noise and/or vibration on the person who appears to the local authority to be carrying out the works and on such other persons appearing to the local authority to be responsible for, or to have control over, the carrying out of the works.

This notice can perform the following.

a) Specify the plant or machinery that is or is not to be used. However, before specifying any particular methods or plant or machinery a local authority has to consider the desirability, in the interests of the recipient of the notice in question, of specifying other methods or plant or machinery that will be substantially as effective in minimizing noise and/or vibration and that will be more acceptable to the recipient.

b) Specify the hours during which the construction work can be carried out.

c) Specify the level of noise and/or vibration that can be emitted from the premises in question or at any specified point on those premises or that can be emitted during the specified hours.

d) Provide for any change of circumstances. An example of such a provision might be that if ground conditions change and do not allow the present method of working to be continued then alternative methods of working should be discussed with the local authority.

In serving such a notice a local authority takes account of:

1) the relevant provisions of any code of practice issued and/or approved under Part III of the Control of Pollution Act 1974;

2) the need for ensuring that the best practicable means are employed to minimize noise and/or vibration;

3) other methods, plant or machinery that might be equally effective in minimizing noise and/or vibration, and be more acceptable to the recipient of the notice;

4) the need to protect people in the neighbourhood of the site from the effects of noise and/or vibration.

A person served with such a notice can appeal to a magistrates' court or in Scotland to the Sheriff or in Northern Ireland to a court of summary jurisdiction, within 21 days from the date of serving of the notice. Normally the notice is not suspended pending an appeal unless it requires some expenditure on works and/or the noise or vibration in question arises or would arise in the course of the performance of a duty imposed by law on the appellant. The regulations governing appeals (the Control of Noise (Appeals) Regulations 1975; in Northern Ireland, the Control of Noise (Appeals) Regulations (Northern Ireland) 1978; and in Scotland, the Control of Noise (Appeals) (Scotland) Regulations 1983) also give local authorities discretion not to suspend a notice even when one or other of these conditions is met, if the noise and/or vibration is injurious to health, or is of such limited duration that a suspension would render the notice of no practical effect; or if the expenditure necessary on works is trivial compared to the public benefit expected.

4.4 Consents under Section 61 of the Control of Pollution Act 1974

This subclause concerns the procedure adopted when a contractor (or developer) takes the initiative and approaches the local authority to ascertain its noise and/or vibration requirements before construction work starts (see also 4.3).

It is not mandatory for applications for consents to be made, but it will often be in the interest of a contractor or an employer or their agents to apply for a consent, because once a consent has been granted a local authority cannot take action under Section 58 or Section 60 of the Control of Pollution Act 1974 or Section 80 of the Environmental Protection Act 1990, so long as the consent remains in force and the contractor complies with its terms. Compliance with a consent does not, however, exempt the person holding that consent against action by a private individual under Section 59 of the 1974 Act, under Section 82 of the 1990 Act, or under common law.

It is essential that an application for a consent is made at the same time as, or later than, any request for approval under Building Regulations or for a warrant under Section 6 of the Building (Scotland) Act 1959, when this is relevant. Subject to this constraint, there are obvious advantages in making any application at the earliest possible date. There may be advantages in having informal discussions before formal applications are made.

It is essential that an applicant for a consent gives the local authority as much detail as possible about the construction work to which the application relates and about the method or methods by which the work is to be carried out. It is also essential that information be given about the steps that will be taken to minimize noise and/or vibration resulting from the construction work.

Provided that a local authority is satisfied that proposals (accompanying an application) for the minimizing of noise and/or vibration are adequate (and in deciding this it may have regard, among other things, to the provisions of this standard), it will give its consent to the application. It can however attach conditions to the consent, or limit or qualify the consent, to allow for any change in circumstances and to limit the duration of the consent. If a local authority fails to give its consent within 28 days of the lodging of an application, or if it attaches any conditions or qualification to the consent that are considered unnecessary or unreasonable, the applicant concerned can appeal to a magistrates' court or in Scotland to the Sheriff or in Northern Ireland to a court of summary jurisdiction, within 21 days from the end of that period.

When a consent has been given and the construction work is to be carried out by a person other than the applicant for the consent, it is essential that the applicant takes all reasonable steps to bring the terms of consent to the notice of that other person; failure to do so or failure to observe the terms of a consent are offences under the Act.

4.5 Contractual procedures

It is likely to be to the advantage of a developer or contractor, or an employer or his agent, who intends to carry out construction work, to take the initiative and apply to the local authority for consents under the Control of Pollution Act. This will have implications for traditional tender and contract procedures because the local authority's noise and/or vibration requirements may well affect both the tender and contract price. It is therefore preferable that the local authority's requirements are made known before tenders are submitted. The best way of achieving this is for the person for whom the work is to be carried out to make the application to the local authority for a consent, before inviting tenders. As much detailed information as possible should be given concerning the methods by which the construction work is to be carried out, and concerning also the proposed noise abatement and/or vibration control measures to enable the local authority to give a consent (see also 4.4).

When a person for whom construction work is to be carried out has sought and obtained consent from the local authority, the local authority's requirements should be incorporated in the tender documents so that tenderers do not base their tenders on the use of unacceptable work methods and plant.

As far as possible, a contractor should be allowed freedom of choice regarding plant and methods to be used but a local authority can, in consultation with the recipient of a consent, specify the type of plant or methods to be used with its consent. In addition to any approach made by a person responsible for construction work, a tenderer may also wish to apply to a local authority in order either to seek consent for the use of methods or plant in place of those specified in an earlier consent (or notice), or to satisfy himself that the detailed methods and plant that he had planned to use meet the conditions laid down.

4.6 Emergencies

In the event of any emergency or unforeseen circumstances arising that cause safety to be put at risk, it is important that every effort should be made to ensure that the work in question is completed as quickly and as quietly as possible and with minimum practical disturbance to people living or working nearby. The local authority should be informed as soon as possible, should it be found necessary to exceed permitted noise and/or vibration limits because of an emergency.

4.7 Flow diagram

The procedures available under the Control of Pollution Act 1974 for the control of construction noise and/or vibration are illustrated by the flow diagram shown in Figure 1.

4.8 Land Compensation Act 1973 (as amended), Highways Act 1980 and Land Compensation (Scotland) Act 1973

The Noise Insulation Regulations 1975 and Noise Insulation (Scotland) Regulation 1975, made under the powers contained respectively in the Land Compensation Act 1973 and the Land Compensation (Scotland) Act 1973, allow a highway authority to provide insulation for dwellings and other buildings used for residential purposes by means of double glazing and special ventilation when highway works are expected to cause serious noise effects for a substantial period of time.

The 1973 Acts also contain provisions that enable a highway authority to pay the reasonable expenses of residents who, with the agreement of the authority, have to find suitable alternative accommodation for the period during which construction work makes continued occupation of an adjacent dwelling impracticable.

The Highways Act 1980 and the Land Compensation (Scotland) Act 1973 enable highway authorities to acquire land by agreement when its enjoyment is seriously affected by works of highway construction or improvement. In addition, these Acts give the highway authority power to carry out works, for example the installation of noise barriers, to mitigate the adverse effects of works of construction or improvement on the surroundings of a highway.

5 Project supervision

5.1 Project programme

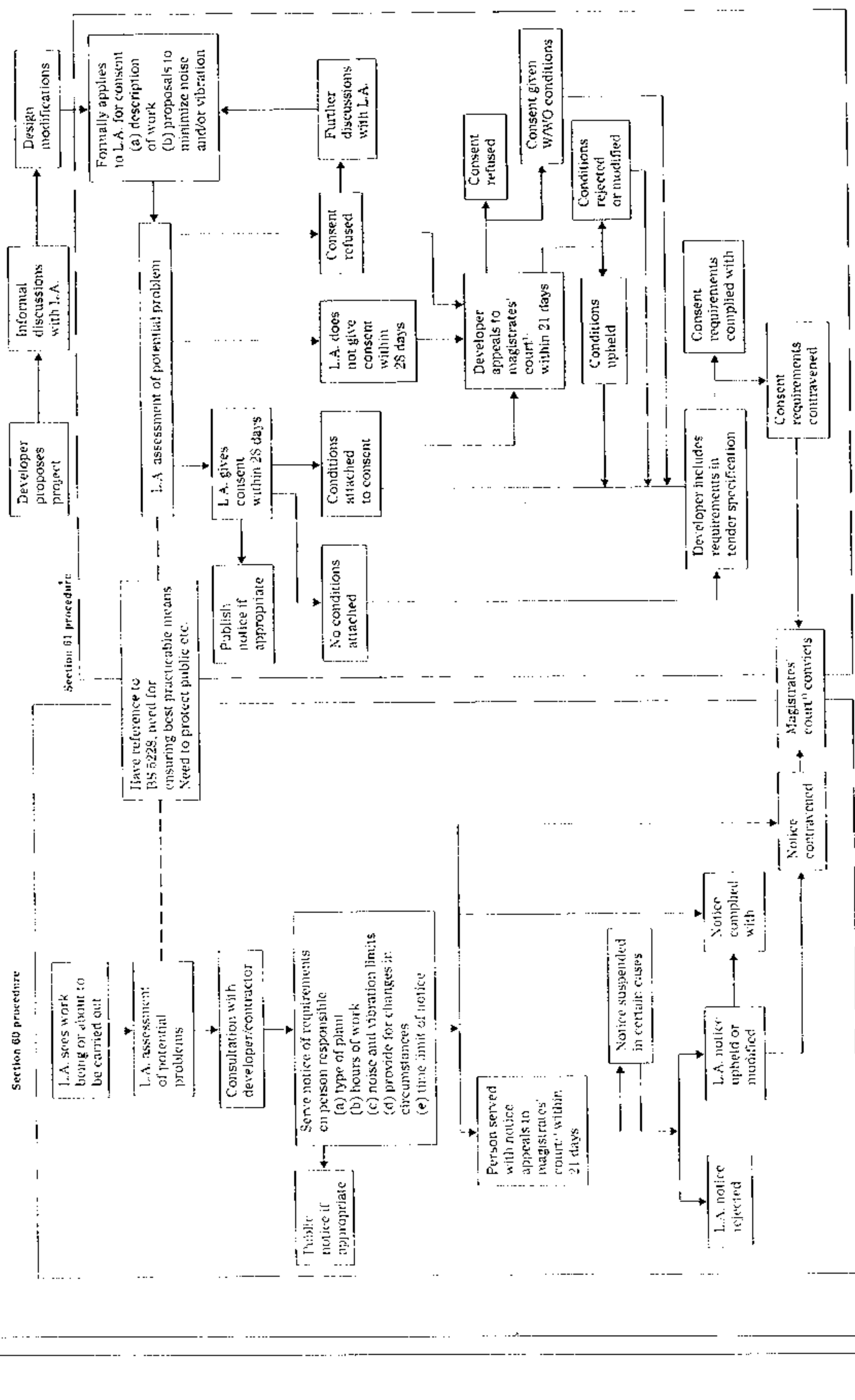
Piling programmes should be arranged so as to control the amount of disturbance in noise and vibration sensitive areas at times that are considered to be of greatest sensitivity. If piling works are in progress on a site at the same time as other works of construction and demolition that themselves may generate significant noise and vibration, the working programme should be phased so as to prevent unacceptable disturbance at any time.

5.2 Piling subcontracts: consents and notices

When piling works are to form a subcontract to the main construction and demolition works on a site, copies of noise and/or vibration consents and details of other noise and/or vibration restrictions should be included in the tender documents for the piling subcontract. Any such noise and/or vibration restrictions, limitations on hours of work, etc., may be at variance with conditions with which the piling tenderer may otherwise be expected to comply. Provision should therefore be made for further consultations with the local authority that could in turn lead to a special consent or variation in restrictions for the duration of the piling works.

During such a consultation the planner, developer, architect and engineer, as well as the local authority, should be made aware of the proposed method of working of the piling subcontractor, who in turn should have evaluated any practicable and more acceptable alternatives that would economically achieve, in the given ground conditions, equivalent structural results. Information relating to the mechanical equipment and plant to be used (see BS 5228-1) should be supplied in support of the proposed method of working. An indication of the intended programme of works should be given, but the piling subcontractor will wish to retain as much flexibility as possible in order to combat unexpected ground conditions or other problems, and it should be recognized that substantial deviations from a detailed programme of works could be made in practice. Due attention should be paid to safe working practices and to emergency procedures.

The developer, as the person ultimately responsible for a project, will need to instigate a check that the proposals suggested by those tendering for piling works are likely to be acceptable to the local authority.



[1] In Scotland, the Sheriff; in Northern Ireland, the court of summary jurisdiction.

NOTE The following abbreviations have been used:
L.A. local authority; W/WO with or without.

Figure 1 — Procedures to control construction noise and/or vibration under the Control of Pollution Act 1974

Section 2. Noise

6 Factors to be considered when setting noise control targets

6.1 Selection of piling method

6.1.1 The selection of a method to be used for the installation of piles will depend on many factors, some of which are outlined in **6.1.2** (see **7.2** for types of piling).

6.1.2 It should be remembered that a decision regarding the type of pile to be used on a site will normally be governed by such criteria as loads to be carried, strata to be penetrated and the economics of the system, for example the time it will take to complete the installation and other associated operations such as soil removal.

6.1.3 It may not be possible for technical reasons to replace a noisy process by one of the "quieter piling" alternatives. Even if it is possible, the adoption of a quieter method may prolong the piling operation; the net result being that the overall disturbance to the community, not only that caused by noise, will not necessarily be reduced.

6.1.4 Examples of typical noise levels associated with the different methods of piling are given in Table 1 which is an extension to the data given in Table 8 of BS 5228-1:1984.

6.2 Types of noise

On typical piling sites the major sources of noise are essentially mobile and the noise received at any control points will, therefore, vary from day to day as work proceeds.

The type of noise associated with piling works depends on the method of piling employed. For example, pile driving using a drop hammer results in a well defined, impulsive type of noise. Air and diesel hammers also produce impulsive noise although their striking rates can be much higher than with drop hammers. With auger-bored piling the impulsive characteristic is virtually absent. With bored or jacked piling methods the resultant noise is steady.

Highly impulsive noise is generally less acceptable than steady noise. However, other characteristics of the noise source play an important part in determining the acceptability of piling noise, e.g. cable slap, screeching of pulleys and guides and ringing of piles.

6.3 Duration of piling works

The duration of piling work is usually short in relation to the length of construction work as a whole, and the amount of time spent working near to noise-sensitive areas can represent only a part of the piling period.

6.4 Hours of working

When a local authority intends to control noise by imposing restrictions on working hours it should have regard to the specialized nature of some piling works, which may necessitate a longer working day.

A local authority should also bear in mind the acceptable hours for the residents and occupiers of a particular area.

6.5 Methods of monitoring and control

Whatever method is appropriate for the specifying of a noise target, there should be agreement between the piling contractor concerned and the controlling authority.

It is essential that a noise target is appropriate to the type of noise, and is practical and enforceable. It should adequately protect the community but allow work to proceed as near normally as possible.

Steady noise levels should normally be expressed in terms of the L_{Aeq} over a period of several hours or for a working day. Impulsive noise levels cannot always be controlled effectively using this measure alone. The specification of a higher short term limit is often found useful. This can be achieved by specifying a short period L_{Aeq} or the one percentile exceedance level L_{A01} over one driving cycle. Where L_{A01} is specified the F time weighting should be used and measurements should be made with a sampling rate of at least five samples per second. Noise limits should not be set in terms of $L_{pA,max}$ when the noise is impulsive.

The difference between limits set in terms of L_{A01} and L_{Aeq} will depend on the striking rate of the pile driver.

Those who wish to use the data for L_{Aeq} in Table 1 to estimate the corresponding value of L_{A01} should note the following approximate relationships [all measurements in dB(A)]:

a) $L_{A01} \approx L_{Aeq} + 11$	for pile drivers such as drop hammers with a slow striking rate; and
b) $L_{A01} \approx L_{Aeq} + 5$	for air hammers with a fast striking rate.

Table 1 — Sound level data on piling

Ref no.[a]	Pile		Method	Energy, power rating	Dolly	Sound power level L_{WA}	Soil	Cycle time	On-time	Activity equivalent continuous sound pressure level L_{Aeq} at 10 m (1 cycle)
	Depth	Width								
	m	m				dB			%	dB
	SHEET STEEL PILING									
50	12	0.4	Double acting diesel hammer	3 790 kgf m	Steel on fibrous material	135	—	—	100	107
51				16 500 kgf m	Not known	140		—	100	112
52	12	0.4	Double acting air hammer	560 kgf m	Steel on fibrous material	134	—	—	100	106
53	12	0.4	Hydraulic vibratory driver	20.7 kg m eccentric moment; 26 Hz	None	118	Sand and gravel	—	100	90
54	8	0.508	Air hammer	415 kgf m	None	131	Sandy clay overlying boulder clay	—	100	103
55	8	0.508		415 kgf m	None	134	Sandy clay overlying boulder clay	—	100	106
56	8	0.508	Drop hammer (hammer and pile enclosed acoustically)	3 t	150 mm greenheart timber plus rope	94	Sandy clay overlying boulder clay	—	100	66
57	8	0.508		3 t	150 mm greenheart timber plus rope	98	Sandy clay overlying boulder clay	—	100	70

[a] See reference numbers 1 to 19, in Table 8 of BS 5228-1:1981 for further information concerning sound level data on piling.

Table 1 — Sound level data on piling

Ref no.	Pile		Method	Energy, power rating	Dolly	Sound power level L_{WA}	Soil	Cycle time	On-time	Activity equivalent continuous sound pressure level L_{Aeq} at 10 m (1 cycle)	
	Depth	Width									
	m	m				dB			%	dB	
58	10 (4 m exposed)	0.96	Double acting air impulse hammer	15 kN m	Air cushion	111	—	—	100	83	
59	15 (5 m exposed)	1.05	Hydraulic hammer, enclosed acoustically	60 kN m	Steel on fibrous material	121	Gravel overlying stiff clay	—	100	93	
60	15	1.05	Hydraulic drop hammer, enclosed acoustically	60 kN m	Steel on fibrous material	113	Gravel overlying stiff clay	—	100	85	
	TUBULAR CASING										
61	23	1.07 dia.	Double acting diesel hammer	6 219 kgf m	Not known	122	Silt overlying chalk	—	100	94	
62	23	1.07 dia.		16 000 kgf m	Not known	132	Silt overlying chalk	—	100	104	
	TUBULAR STEEL CASING/PILE CAST IN PLACE										
63(a)	13	0.35 dia.	Drop hammer	3.3 t, 1.2 m drop	Resilient composite pad	130	Estuarial alluvia	20 min	20	95	97
63(b)	13	0.35 dia.		3.3 t, 1.2 m drop	Resilient composite pad	126	Estuarial alluvia	20 min	30	93	
63(c)	13	0.35 dia.	Drop hammer, extracting casing	3.3 t	Resilient composite pad	120	Estuarial alluvia	20 min	10	82	
64(a)	14	0.4 dia.	Drop hammer	4 t, 1.2 m drop	Resilient composite pad	132	Dense sand	45 min	40	100	100
64(b)	14	0.4 dia.		4 t, 1.2 m drop	Resilient composite pad	125	Dense sand	45 min	20	90	
64(c)	14	0.4 dia.	Drop hammer, extracting casing	4 t	Resilient composite pad	118	Dense sand	45 min	5	77	

Table 1 — Sound level data on piling

Ref no.	Pile		Method	Energy, power rating	Dolly	Sound power level L_{WA}	Soil	Cycle time	On-time	Activity equivalent continuous sound pressure level L_{Aeq} at 10 m (1 cycle)	
	Depth	Width									
	m	m				dB			%	dB	
65(a)	8	0.35 dia.	Drop hammer, partially enclosed acoustically	3.3 t, 1.2 m drop	Resilient composite pad	117	Silt/peat/shale/ sandstone	25 min	15	81	91
65(b)	8	0.35 dia.		3.3 t, 1.2 m drop	Resilient composite pad	122	Silt/peat/shale/ sandstone	25 min	35	89	
65(c)	8	0.35 dia.	Drop hammer, partially enclosed acoustically, extracting casing	3.3 t, 1.2 m drop	Resilient composite pad	121	Silt/peat/shale/ sandstone	25 min	8	82	
66(a)	8	0.4 dia.	Drop hammer, partially enclosed acoustically	4 t, 1.6 m drop	None	129	Stiff to hard sandy clay	30 min	35	96	97
66(b)	8	0.4 dia.		4 t, 1.6 m drop	None	125	Stiff to hard sandy clay	30 min	30	92	
67(a)	5	0.45 dia.	Internal drop hammer	3 t, 4 m drop	Dry mix aggregate plug	113	Made ground overlying clay	40 min	50	82	86
67(b)	5	0.45 dia.		3 t, 4 m drop	Dry mix aggregate plug	115	Made ground overlying clay	40 min	50	84	
68(a)	14	0.4 dia.		3 t, 4 m drop	Dry mix aggregate plug	111	Ballast	—	50	80	84
68(b)	14	0.4 dia.		3 t, 4 m drop	Dry mix aggregate plug	116	Ballast	—	25	82	

Table 1 — Sound level data on piling

Ref no.[a]	Pile Depth	Pile Width	Method	Energy, power rating	Dolly	Sound power level L_{WA}	Soil	Cycle time	On-time	Activity equivalent continuous sound pressure level L_{Aeq} at 10 m (1 cycle)	
	m	m				dB			%	dB	
	IMPACT BORED/PILE CAST IN PLACE										
69(a)	20	0.5 dia.	Tripod winch	20 kW	None	106	Fill/ballast/stiff clay	6 h	30	73	83
69(b)	20	0.5 dia.		20 kW	None	108	Fill/ballast/stiff clay	6 h	60	78	
69(c)	20	0.5 dia.	Tripod winch, driving casing	3/4 t, 1 m drop	Steel	118	Fill/ballast/stiff clay	6 h	2.5	74	
69(d)	20	0.5 dia.		3/4 t, 1 m drop	Steel	122	Fill/ballast/stiff clay	6 h	2.5	78	
70(a)	25	0.6 dia.	Tripod winch	20 kW	None	108	Fill/sand/ballast/ stiff clay	10 h	30	75	88
70(b)	25	0.6 dia.		20 kW	None	113	Fill/sand/ballast/ stiff clay	10 h	60	83	
70(c)	25	0.6 dia.	Tripod winch, driving casing	3/4 t, 1 m drop		127	Fill/sand/ballast/ stiff clay	10 h	2	82	
70(d)	25	0.6 dia.		3/4 t, 1 m drop	Steel	129	Fill/sand/ballast/ stiff clay	10 h	2	84	
	H SECTION STEEL PILING										
71	22.5	0.31 × 0.31 × 0.11	Double acting diesel hammer	3 703 kgf m	Steel on fibrous material	127	Sand and silt overlying stiff clay	—	100	99	
72	—	0.35 × 0.37 × 0.089	Diesel hammer	6 219 kgf m	Not known	122	Rock fill	—	100	94	
73	75	0.3 × 0.3	Hydraulic drop hammer, enclosed acoustically	36 kN m	Hardwood	113	Chalk	—	100	85	
74	75	0.3 × 0.3		36 kN m	Hardwood	116	Chalk	—	100	88	
75	75	0.3 × 0.3	Hydraulic drop hammer	84 kN m	Steel on fibrous material	124	Chalk	—	100	96	

[a] See reference numbers 1 to 49, in Table 8 of BS 5228-1:1984 for further information concerning sound level data on piling.

Table 1 — Sound level data on piling

Ref no.	Pile Depth	Pile Width	Method	Energy, power rating	Dolly	Sound power level L_{WA}	Soil	Cycle time	On-time	Activity equivalent continuous sound pressure level L_{Aeq} at 10 m (1 cycle)
	m	m				dB			%	dB
	PRECAST CONCRETE PILES									
76	—	—	Drop hammer	5 t, 0.75 m drop	Not known	114	Fill	—	100	86
77	50	0.29 × 0.29 square section modular (joined)	Hydraulic drop hammer, enclosed acoustically	60 kN m	Hardwood	107	Chalk	—	100	79
78	50			60 kN m	Hardwood	111	Chalk	—	100	83
79	20	0.275 × 0.275 square section modular (joined)	Hydraulic hammer	3 t, 0.3 m drop	Hardwood	111	Stiff clay overlying mudstone	—	100	83
80	20			3 t, 0.3 m drop	Hardwood	119	Stiff clay overlying mudstone	—	100	91

Table 1 — Sound level data on piling

Ref no.	Pile		Method	Energy, power rating	Dolly	Sound power level L_{WA}	Soil	Cycle time	On-time	Activity equivalent continuous sound pressure level L_{Aeq} at 10 m (1 cycle)
	Depth	Width								
	m	m				dB			%	dB
81	10	0.275 × 0.275 square section modular (joined)	Hydraulic hammer, partially enclosed acoustically	4 t, 0.3 m drop	Hardwood	109	Clay/gravel overlying mudstone	—	100	81
82	10			4 t, 0.3 m drop	Hardwood	106	Clay/gravel overlying mudstone	—	100	78
83	17	0.285 × 0.285 square section modular (joined)	Drop hammer	5 t, 1 m drop	Wood	114	Silt/sand/gravel	55 min	80	85
84	20	0.08 m^2 hexagonal section modular (joined)	Drop hammer, hanging leaders; soft driving	4 t, 0.6 m drop	Wood	114	Alluvium	—	100	86
85	20	0.08 m^2 hexagonal section modular (joined)	Drop hammer, hanging leaders; medium/hard driving	4 t, 0.75 m drop	Wood	121	Stiff clays and gravels	—	100	93

Table 1 — Sound level data on piling

Ref no.	Pile Depth	Pile Width	Method	Energy, power rating	Dolly	Sound power level L_{WA}	Soil	Cycle time	On-time	Activity equivalent continuous sound pressure level L_{eq} at 10 m (1 cycle)	
	m	m				dB			%	dB	
86	20	0.406 dia. modular shell	Drop hammer driving on mandrel/pile cast in place	5 t, 0.75 m drop	Wood/sisal	114	Fill overlying chalk	41 min	30	82	
87	28	0.444 dia. modular shell		6 t, 1 m drop	Wood	121	Sand/clay/chalk	57 min	30	89	
	BORED PILING/PILE CAST IN PLACE										
88	10	0.45 dia.	Crane-mounted auger: donkey engine in acoustic enclosure	65 kW	None	108	Fill overlying stiff clay	45 min	100	80	
89(a)	25	0.6 dia.		90 kW	None	110	Sand/gravel/stiff clay	90 min	85	81	85
89(b)	7	0.6 dia.	Driving temporary casing to support upper strata in prebored hole by drop hammer	2.5 t, 0.6 m drop	Steel	128	Sand/gravel/stiff clay	90 min	1.5	82	
90	15	0.45 dia.	Lorry-mounted auger: donkey engine in acoustic enclosure	90 kW	None	109	Sand/gravel/clay	55 min	100	81	
91	20	0.6 dia.		90 kW	None	113	Fill/clay	75 min	100	85	
92(a)	25	0.9 dia.	Crane-mounted auger	90 kW	None	114	Fill/clay	3 h	95	86	87
92(b)	25	0.9 dia.	Crane-mounted auger: kelly bar clanging	90 kW	None	122	Fill/clay	3 h	3	79	
93	30	1.05 dia.	Crane-mounted auger	120 kW	None	117	Ballast/clay	5 h	100	89	

Table 1 — Sound level data on piling

Ref no.	Pile		Method	Energy, power rating	Dolly	Sound power level L_{WA}	Soil	Cycle time	On-time	Activity equivalent continuous sound pressure level L_{Aeq} at 10 m (1 cycle)
	Depth	Width								
	m	m				dB			%	dB
94(a)	24	2.1 dia.	Crane-mounted auger and drilling bucket: pile bored under bentonite	110 kW	None	112	Alluvia/sands/clay	2 days	50	81 } 82
94(b)	24	2.1 dia.	Crane-mounted auger and drilling bucket: kelly bar clanging	110 kW	None	121	Alluvia/sands/clay	2 days	2	76 }
95	40	1.2 dia.	Crane-mounted auger and drilling bucket: pile bored under bentonite	120 kW	None	117	Sand/boulder clay/marl	2 days	50	86
96	20	0.9 dia.	Lorry-mounted auger	110 kW	None	115	Fill/sand/gravel/clay	3 h	100	87
97	20	1.2 dia.		110 kW	None	112	Fill/ballast/clay	6 h	100	84
	CONTINUOUS FLIGHT AUGER INJECTED PILING									
98	11	0.45 dia.	Crane-mounted leaders with continuous flight auger; cement grout injected through hollow stem of auger. Engine/power pack partially enclosed acoustically	90 kW	None	111	Alluvium	30 min	50	80
99	15	0.35 dia.		90 kW	None	108	Sand and silts	30 min	50	77
100	12	0.45 dia.	Crane-mounted continuous flight auger rig: concrete injected through hollow stem of auger. Engine/power pack partially enclosed acoustically	100 kW	None	109	Gravels overlying chalk	30 min	50	78

Table 1 — Sound level data on piling

Ref no.	Pile Depth	Pile Width	Method	Energy, power rating	Dolly	Sound power level L_{WA}	Soil	Cycle time	On-time	Activity equivalent continuous sound pressure level L_{Aeq} at 10 m (1 cycle)	
	m	m				dB			%	dB	
	DIAPHRAGM WALLING										
101	25	1.0 × 4.0	Crane-mounted hydraulically operated trenching grab guided by kelly bar	90 kW	None	114	Sands and gravels overlying chalk	12 h	100	86	
102	25	1.0 × 4.0	Crane-mounted hydraulically operated trenching grab guided by kelly bar	90 kW	None	116	Sands and gravels overlying chalk	12 h	100	86	
103	25	1.0 × 4.5	Crane-mounted rope operated trenching grab	8 t, 10 m drop	None	113	Sands and gravels overlying clay	10 h	80	84	
	VIBROREPLACEMENT/VIBRODISPLACEMENT										
104(a)	4	0.5 dia. approx.	Stone column formation by crane-mounted hydraulically powered vibrating poker. Compressed air flush: nose cone air jets exposed	90 kW	None	110	Miscellaneous fill	15 min	80	81	85
104(b)	4	0.5 dia. approx.	Stone column formation by crane-mounted hydraulically powered vibrating poker. Compressed air flush; nose cone air jets exposed	90 kW	None	117	Miscellaneous fill	15 min	20	82	

Table 1 — Sound level data on piling

Ref no.	Pile Depth	Pile Width	Method	Energy, power rating	Dolly	Sound power level L_{WA}	Soil	Cycle time	On-time	Activity equivalent continuous sound pressure level L_{Aeq} at 10 m (1 cycle)	
	m	m				dB			%	dB	
105(a)	—	2.4 × 2.4	Tamping weight raised by large crawler crane	120 kW	None	114	Made ground and fill	10 min	80	85	86
105(b)	—	2.4 × 2.4	Tamping weight released by crane: impact of weight	20 t, 20 m drop	None	125	Made ground and fill	1 drop per min	1.5	79	
106(a)	—	2.4 × 2.4	Tamping weight raised by large crawler crane	120 kW	None	110	Made ground and fill	10 min	80	81	82
106(b)	—	2.4 × 2.4	Tamping weight released by crane: impact of weight	20 t, 20 m drop	None	122	Made ground and fill	1 drop per min	1.5	76	
	INSTALLATION OF VERTICAL BAND DRAINS										
107(a)	7	0.1	Hydraulic vibratory lance starting up	50 kW	None	113	Sandy silty fill	5 min	1	65	
107(b)	7	0.1	Hydraulic vibratory lance installing band drain	50 kW	None	107	Sandy silty fill	5 min	70	76	80
107(c)	7	0.1	Hydraulic vibratory lance being extracted	50 kW	None	115	Sandy silty fill	5 min	15	79	

NOTE 1 Energy and power relationship: 1 kgf m = 9.81 joules (J).
NOTE 2 1 t dropped 1 m = 9.81 × 10^3 J = 9.81 kJ = 9.81 kN m; 1 kW = 10^3 J/s = 1 kJ/s.
NOTE 3 Depths, cycle times where quoted and on-times are typical for specific cases but can vary considerably according to ground and other conditions.

7 Practical measures to reduce site noise

7.1 Assessment of noise levels of mechanical equipment and plant

Those undertaking piling works should endeavour to ascertain the nature and levels of noise produced by the mechanical equipment and plant that will be used (see Table 8 and Appendix B of BS 5228-1:1984). They may then be able to take steps to reduce either the level or the annoying characteristics, or both, of the noise. Some guidance on noise control techniques is given in 7.3.

7.2 Types of piling

7.2.1 *General*

Piles can be divided into two main categories, bearing piles and retaining piles. It is possible in principle to install either category by driving, jacking or boring (see Figure 2). Ground or other site conditions can, however, prohibit the use of one or other of these techniques, that are described in more detail in 7.2.2 to 7.2.4.

There are other methods of forming medium to deep foundations under certain conditions. These include the installation of stone columns by vibroreplacement (see 7.2.5), deep compaction by dynamic consolidation (see 7.2.6), and the technique of diaphragm walling (see 7.2.7). Although the mechanical plant and equipment can differ in some ways from those used in conventional piling, the problems of protecting the neighbourhood from noise disturbance are similar.

7.2.2 *Driven piles*

In conventional driven piling, a hammer is used to strike the top of the pile via a helmet and/or a sacrificial dolly. High peak noise levels will arise as a result of the impact. The hammer can be a simple drop hammer or it can be actuated by steam, air, hydraulic or diesel propulsion. Displacement piles can be top driven, bottom driven or can be driven by means of a mandrel.

In certain ground conditions it may be possible to drive piles using a vibratory pile driver, in which cases high impact noise may not arise, but the continuous forced vibration together with structure-borne noise can give rise to some disturbance.

When piles are driven for temporary works further disturbance can occur at a later date when the piles are extracted.

7.2.3 *Jacked piles*

A method for installing either retaining or bearing steel piles without either hammering or vibratory driving is by jacking. One or a pair of piles is pushed into the ground using the reaction of a group of several more adjacent piles. The main source of noise is the engine driving the hydraulic power pack for the jacking system. Other sources of noise include cranes and ancillary equipment. The use of jacked piles is appropriate in most types of cohesive soil and silty sands, but specialist advice should be sought in such cases.

7.2.4 *Bored piles*

Bored piles can be constructed by means of a rotary piling rig or by impact boring. In the former case the major source of noise is the more or less steady noise of the donkey engine that supplies the power to perform the drilling. In certain types of soil it is necessary to insert casings for part of the depth. If the casings have to be driven in and/or extracted by hammering, high peak noise levels will result. Similar considerations apply to the impact boring technique. The noise characteristics may therefore be at a relatively steady and continuous level with intermittent high peaks superimposed upon it.

A method for boring piles that does not need a temporary casing is the use of a continuous flight auger and the injection of concrete or grout to form the piles. It is applicable only in certain ground conditions and the range of pile diameters is limited.

7.2.5 *Vibroflotation/vibrocompaction and vibroreplacement/vibrodisplacement*

A method for improving the bearing capacity of weak soils and fills is to use a large vibrating poker which can be mounted on a crane or an excavator base. In loose cohesionless soils the vibrations cause compaction to a denser state; this process is known as vibroflotation or vibrocompaction. In other weak soils a vibrating poker is used to form a hole which is then backfilled with graded stone and compacted by the poker; this process is known as vibroreplacement or vibrodisplacement. Water or compressed air can be used as a jetting and flushing medium.

Typically, vibrating pokers are actuated by electric or hydraulic motors. To reduce the noise of the operation, attention should be paid to the generator or power pack as appropriate. Other sources of noise could include pumps when using water flush, or air escaping from the poker when this is exposed.

7.2.6 *Deep compaction by dynamic consolidation*

An alternative method for improving the bearing capacity of weak soils and fills is to drop a large tamping weight from a height on to the ground at selected locations. Typically in the UK, tamping weights between 10 t and 20 t are used and are dropped from heights between 10 m and 25 m, although in some cases other weights and drop heights can be used. The tamping weight is normally raised by and dropped from a very large crawler crane and the noise characteristic contains both steady (crane engine) and impulsive (impact of weight on ground) components.

7.2.7 *Diaphragm walling*

When deep foundation elements with both retaining and bearing capabilities are needed, the technique of diaphragm walling may be applicable. The soil is excavated in a trench under a mud suspension (e.g. bentonite) in a series of panels, usually using a special clamshell grab; when the full depth has been reached a reinforcing cage is inserted and concrete is placed by tremie pipe, thus displacing the mud to the surface.

The grab is normally suspended from a crawler crane although a tracked excavator base may sometimes be used. It is operated either by gravity or hydraulically in which latter case it is guided by a kelly bar. Diaphragm walling sites frequently need much ancillary equipment including bentonite preparation and reclamation plant, reinforcing cage manufacturing plant, pumps and handling cranes. The layout of plant on the site is important for efficient operation and can exert considerable influence on noise control.

7.3 Noise reduction techniques

7.3.1 *Piling operations*

Noise can be reduced at source or, when this is not possible, the amount of noise reaching the neighbourhood can be reduced by various means.

Impact noise when piling is being driven can be reduced by introducing a non-metallic dolly between the hammer and the driving helmet. This will prevent direct metal-to-metal contact, but will also modify the stress wave transmitted to the pile, possibly affecting the driving efficiency. The energy absorbed by the dolly will appear as heat. Further noise reduction can be achieved by enclosing the driving system in an acoustic shroud. Several commercially available systems employ a partial enclosure arrangement around the hammer. It is also possible to use pile driving equipment that encloses the hammer and the complete length of pile being driven, within an acoustic enclosure.

For steady continuous noise, such as that caused by diesel engines, it may be possible to reduce the noise emitted by fitting a more effective exhaust silencer system or by designing an acoustic canopy to replace the normal engine cover. Any such project should be carried out in consultation with the original equipment manufacturer and with a specialist in noise reduction techniques. Caution should be exercised in order that the replacement canopy does not cause the engine to overheat and does not interfere excessively with routine maintenance operations.

It may be possible in certain circumstances to substitute electric motors for diesel engines, with consequent reduction in noise. On-site generators supplying electricity for electric motors should be suitably enclosed and appropriately located.

Screening by barriers and hoardings is less effective than total enclosure but can be a useful adjunct to other noise control measures. For maximum benefit, screens should be close either to the source of noise (as with stationary plant) or to the listener. It may be necessary for safety reasons to place a hoarding around the site, in which case it should be designed taking into consideration its potential use as a noise screen. Removal of a direct line of sight between source and listener can be advantageous both physically and psychologically.

Consideration should be given to the possible application of some of the alternative techniques of piling referred to in **7.2**. For convenience these are grouped together in Figure 2.

7.3.2 *Location and screening of stationary plant*

In certain types of piling works there will be ancillary mechanical plant and equipment that may be stationary, in which case care should be taken in location, having due regard also for access routes. Stationary or quasi-stationary plant might include, for example, bentonite preparation equipment, grout or concrete mixing and batching machinery, lighting generators, compressors, welding sets and pumps. When appropriate, screens or enclosures should be provided for such equipment.

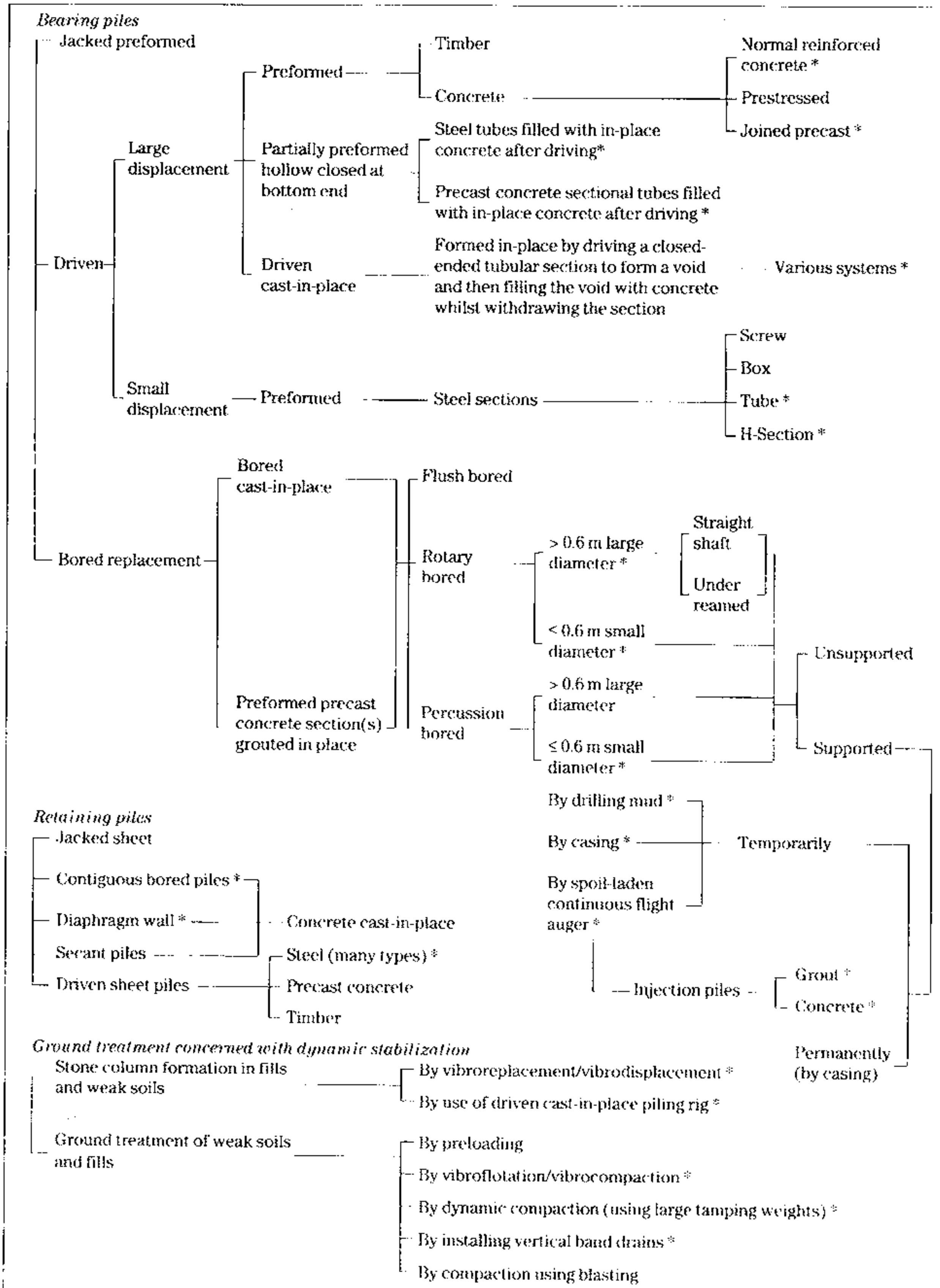
7.3.3 *Mobile ancillary equipment*

Contributions to the total site noise can also be anticipated from mobile ancillary equipment, such as handling cranes, dumpers, front end loaders, excavators, and concrete breakers. These machines may only have to work intermittently, and when safety permits, their engines should be switched off (or during short breaks from duty reduced to idling speed) when not in use.

7.3.4 *Maintenance and off-site traffic*

All mechanical equipment and plant should be well maintained throughout the duration of piling works.

When a site is in a residential environment, lorries should not arrive at or depart from the site at a time inconvenient to residents.



NOTE 1 It should always be remembered that the type of pile to be used on any site is normally governed by such criteria as loads to be carried, strata to be penetrated and the economics of the system.

NOTE 2 Where necessary, allowance should be made for the extraction of piles in addition to their installation.

NOTE 3 Sound level data for systems marked thus * are included in Table 1. Other data may be found in Table 8 of BS 5228-1:1984.

Figure 2 — Piling and kindred ground treatment systems

Section 3. Vibration

8 Factors to be considered when setting vibration control targets

8.1 General

The most common form of vibration associated with piling is the intermittent type derived from conventional driven piling. Each hammer blow transmits an impulse from the head to the toe of the pile and free vibrations are set up. Sensors at a remote receiving point would indicate a series of wave disturbances, each series corresponding to one blow. (See also Appendix A.)

When setting targets for maximum vibration levels (**8.2** to **8.6**) reference should be made to the existing ambient vibration levels, which should be measured prior to commencement of pile driving. This is particularly applicable on sites adjacent to roads carrying heavy commercial traffic, railway tracks and large industrial machinery. It is not uncommon for vibrations from such sources to mask vibrations from pile driving.

8.2 Vibration levels

The intensity of each vibration disturbance registered at the remote receiving point will normally be a function of many variables including:

a) energy per blow or cycle;

b) distance between source and receiver;

c) ground conditions at the site, e.g. soft or hard driving and location of water table;

d) soil-structure interaction, i.e.. nature of connection between soil and structure being monitored;

e) construction of structure and location of measuring points, e.g.:

1) soil surface;

2) building foundation;

3) internal structural element.

In soft driving conditions, where a significant proportion of the energy per blow is directly used in advancing the pile, the intensity of vibrations transmitted to the environment is generally less than under hard driving conditions, where so much of the energy per blow is devoted to overcoming resistance to penetration that relatively little is available to advance the pile.

When driving piles in soft soils the free vibrations set up are found usually to have a greater low frequency content than when driving into denser soils or rocks.

Various empirical formulae have been proposed relating the intensity of vibration measured at the remote receiving point, to the distance between it and the source and the energy of the source. The use of such formulae enables a rough estimate to be made as a check on the acceptability of the proposed process from a vibration standpoint, prior to the commencement of the piling works. This estimate could also assist, with applications under Section 61 of the Control of Pollution Act 1974 for prior consent (see 4.4). For guidance regarding the prediction of expected vibration levels see Appendix B.

NOTE 1 Appendix B is included for information only and does not form part of this standard.

NOTE 2 See Appendix C for examples of vibration levels measured under various conditions throughout the UK.

8.3 Human response to vibration

Human beings are known to be very sensitive to vibration, the threshold of perception being typically in the peak particle velocity range of 0.15 mm/s to 0.3 mm/s, at frequencies between 8 Hz and 80 Hz. Vibrations above these values can disturb, startle, cause annoyance or interfere with work activities. At higher levels they can be described as unpleasant or even painful. In residential accommodation vibrations can promote anxiety lest some structural mishap might occur. Guidance on the effects on physical health of vibration at sustained high levels is given in BS 6841, although such levels are unlikely to be encountered as a result of piling operations.

BS 6472 sets down vibration levels at which minimal adverse comment will be provoked from the occupants of the premises being subjected to vibration. It is not concerned primarily with short term health hazards or working efficiency. It points out that human response to vibration varies quantitatively according to the direction in which it is perceived. Thus, generally, vibrations in the foot-to-head mode are more perceptible than those in the back-to-chest or side-to-side modes although at very low frequencies this tendency is reversed.

Base curves in terms of both vibratory acceleration and peak particle velocity in the different coordinate directions are shown in BS 6472. These curves apply to continuous vibrations and there is a series of multiplying factors which can be applied according to the sensitivity of the location to vibrations. In addition, formulae are quoted which may be used to establish minimal adverse complaint levels where the vibrations are intermittent but overall of relatively short duration in comparison to the daytime or night-time period.

A kindred problem is that vibrations may cause structure-borne noise which can be an additional irritant to occupants of buildings. Loose fittings are prone to rattle and movement.

8.4 Structural response to vibration

8.4.1 *General*

Structural failure of sound buildings or building elements or components is not a phenomenon generally attributed to vibration from well controlled piling operations. Extensive studies carried out in this country and overseas have shown that documented proof of actual damage to structures or their finishes resulting solely from piling vibrations is rare. There are many other mechanisms which cause damage especially in decorative finishes and it is often incorrectly concluded that piling vibrations should be blamed.

In some circumstances, however, it is possible for the vibrations to be sufficiently intense to promote minor damage. Typically this damage could be described as cosmetic and would amount to the initiation or extension of cracks in plasterwork, etc., rather than the onset of structural distress. In more severe cases, falls of plaster or loose roof tiles or chimney pots may occur.

NOTE 1 It has been suggested that vibrations generally provide one trigger mechanism which could result in the propagation of an incipient "failure" of some component which hitherto had been in a metastable state.

NOTE 2 Vibration can increase the density of and cause settlement in loose, wet and cohesionless soils, which may put structures at risk.

The making of an assessment of the vulnerability or otherwise of building structures to vibration induced damage needs rather more detailed structural knowledge at the outset than is generally available. Among the points to bear in mind are the following:

a) the design of the structure;

b) the nature, condition and adequacy of the foundations and the properties of the ground supporting these;

c) the age of the structure;

d) the method and quality of construction, including finishes;

e) the general condition of the structure and its finishes;

f) a schedule of existing defects, especially cracks, supplemented where necessary by a photographic record;

g) any information pertaining to major alterations, such as extensions, or past repair work;

h) the location and level of the structure relative to the piling works;

i) the natural frequencies of structural elements and components;

j) the duration of piling operations.

8.4.2 *Response limits of structures*

It is recommended that, for soundly constructed residential property and similar structures which are in generally good repair, a conservative threshold for minor or cosmetic (i.e. non-structural) damage should be taken as a peak particle velocity (p.p.v.) of 10 mm/s for intermittent vibration and 5 mm/s for continuous vibrations. Below these vibration magnitudes, minor damage is unlikely to occur. Current experience suggests that these values may be reduced by up to 50 % where the preliminary survey reveals existing significant defects (such as a result of settlement) of a structural nature, the amount of the reduction being judged on the severity of such defects. The range of frequencies excited by piling operations in the soil conditions typical in the United Kingdom is between 10 Hz and 50 Hz. Acceptable values of p.p.v. may need adjustment for predominant frequencies outside this range.

NOTE 1 At low frequencies (below 10 Hz), large displacements and associated large strains necessitate lower p.p.v. values (50 % lower), whereas at high frequencies (above 50 Hz), much smaller strains allow the p.p.v. limits to be increased (100 % higher).

Buildings constructed for industrial and commercial use exhibit greater resistance to damage from vibrations than normal dwellings, and it is recommended that light and flexible structures (typically comprising a relatively light structural frame with infill panels and sheet cladding) should be assigned thresholds of 20 mm/s for intermittent vibrations and 10 mm/s for continuous vibrations, whereas heavy and stiff buildings should have higher thresholds of 30 mm/s for intermittent vibrations and 15 mm/s for continuous vibrations.

Where buildings appear not to conform precisely to one or other of the descriptions given in this subclause, the thresholds may be adjusted within those stated.

NOTE 2 Additional guidance on the relative sensitivities of various types of building to vibrations is given in BS 7385-1.

Special consideration should be given to ancient ruins and listed buildings[1].

The vibration levels given in this subclause refer to the maximum value on a load bearing part of the structure at ground or foundation level in the vertical, radial or tangential direction. See Figure 3.

[1] See also WATTS, G.R., *Case Studies of the Effects of Traffic Induced Vibrations on Heritage Buildings*. TRRL Research Report 156, 1988. Available from the Transport and Road Research Laboratory, Old Wokingham Road, Crowthorne, Berkshire RG11 6AU

In certain circumstances it may be necessary in addition to specify limits at other locations. For example, modern multi-storey buildings employing continuous construction methods exhibit little inherent damping. Significant amplification of incoming vibrations can, therefore, occur at the upper storeys, notably in the horizontal modes. Likewise, amplification of vibrations (mostly vertical) can occur in the middle of suspended floors. A vertical p.p.v. of up to 20 mm/s during driven piling may be tolerated at such positions. However special care may be needed for old plaster and lath ceilings beneath suspended floors.

NOTE 3 Amplification factors will vary according to individual circumstances, but factors of between 1.5 and 2.5 are typical.

8.5 Assessment of vulnerability of structures and services

8.5.1 *Retaining walls*

Unlike conventional buildings, which are tied together by crosswalls, intermediate floors and roofs, retaining walls may have little lateral restraint near their tops. This can result in substantial amplification of vibrations particularly in the horizontal mode normal to the plane of the wall. Amplification factors of between 3 and 5 are typical.

For slender and potentially sensitive masonry walls it is recommended that threshold limits for p.p.v. of 10 mm/s at the toe and 40 mm/s at the crest should generally be adopted. Propped or tied walls or mass gravity walls can be subject to values 50 % to 100 % greater than the above. Similar values could be applied to well supported steel pile and reinforced concrete retaining walls. Where walls are in poor condition the allowable values should be diminished and at the same time additional propping or other methods of support should be devised. For continuous vibrations all the above levels should be reduced by a factor of 1.5 to 2.5 according to individual circumstances.

8.5.2 *Slopes and temporary excavations*

When piling is to be installed close to slopes, vibration of any form may cause movement of the slope material.

The effect of ground borne vibrations on the stability of temporary earthworks such as modified soil slopes and open excavations should receive careful consideration in order to avoid risk to personnel and partially completed works from dislodged lumps of soil, local collapse of soil faces or even ground movement due to overloading and failure of temporary ground retention systems.

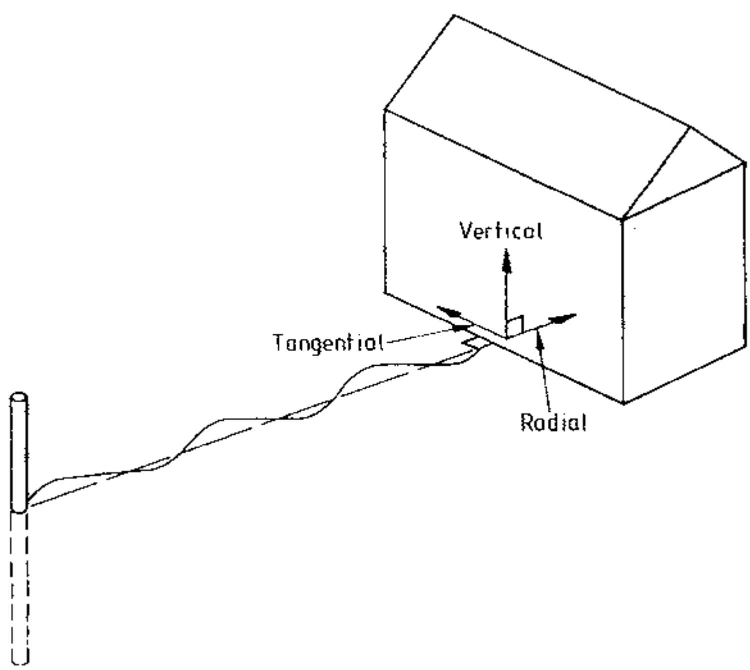


Figure 3 — Orientation of vibration transducers

The risk to stability is dependent on the extent to which the factor of safety under static loading is reduced by the vibrations, and hence on the intensity, characteristics and duration of the vibration and the soil response. The possibility that inherent weaknesses might exist in the soil due to the release of stress and subsequent surface weathering should be borne in mind.

When the pile type is chosen, care should be taken to avoid substituting the risk from vibration, pore pressure changes and soil displacement associated with driven piling and other systems which generate vibrations, by threats to stability resulting from uncontrolled soil removal or the release of ground water.

Consideration should be given to the use of controlled trials to establish a safe method of working, from observations of vibration intensity, of the onset of local distress to the soil face and of changes in line and level.

Where doubt about the loss of stability remains, action should be taken either to phase the work so that piling can be completed before earthworks are carried out, or to retain the soil effectively to allow piling to take place safely.

8.5.3 ***Underground services***

Some statutory undertakings have introduced criteria governing the maximum level of vibrations to which their services should be subjected. These vibrations are usually extremely conservative and it is recommended that the following limits be used:

a) maximum p.p.v. for intermittent or transient vibrations 30 mm/s;

b) maximum p.p.v. for continuous vibrations 15 mm/s.

Values should be applied at the crown unless the lateral dimension of the service is large in relation to the space between the service and the pile.

It should be noted that even a p.p.v. of 30 mm/s gives rise to a dynamic stress which is equivalent to approximately 5 % only of the allowable working stress in typical concrete and even less in iron or steel.

In the event of encountering elderly and dilapidated brickwork sewers the base data should be reduced by 20 % to 50 %. For most metal and reinforced concrete service pipes, however, the values in a) and b) should be quite tolerable. There is often some difficulty in assessing the true condition of underground pipes, culverts and sewers. Among the factors which could mean that such services are in a state of incipient failure are poorly formed joints, hard spots, badly prepared trench bases, distortion due to settlement or heave, or unstable surrounding ground caused by previous or existing leaks.

NOTE The extraction of temporary piling can also generate vibration.

8.6 Assessment of vulnerability of content of buildings

8.6.1 ***Computer installations***

Although modern computer installations incorporate solid state electronics, the disc drive units are considered to be vulnerable to excessive vibration or shock. These devices generate their own continuous internal vibrations from the spinning discs and associated machinery. Major manufacturers have set acceptable external vibration criteria for their equipment, in both operating and transit modes.

The criteria are often expressed in terms of limits on vibratory displacement up to a certain frequency and limits on vibratory acceleration at higher frequencies. A sinusoidal relationship is given between these parameters which can therefore be used to calculate the corresponding particle velocities. For continuous vibrations the allowable thresholds are set at about 40 % of the permitted levels of intermittent vibrations. An example from one major manufacturer quotes permitted levels for intermittent vibrations varying between 50 mm/s at 8 Hz and 10 mm/s at 40 Hz, a frequency range which covers much of that associated with piling in soils. These criteria are judged to apply to computer equipment correctly installed on the ground floor of a building.

Thus computers are not as fragile as is often believed and, with care, piling need not pose a threat to the continued safe use of a typical computer installation. Extra care may be needed if the installation is mounted on a suspended floor which might accentuate the level of transmitted vibrations.

8.6.2 ***Telephone exchanges***

In telephone exchanges where electro-mechanical methods of circuit selection are used, excessive vibrations of the appropriate frequencies may set up resonances in the contact arms leading to wrong lines or other malfunction. Research on one type of installation resulted in the adoption of a limiting p.p.v. of 5 mm/s for intermittent vibrations, as measured on the floor of the exchange room. With advances in telecommunication technology many different systems exist, some of which are less sensitive to vibration. Individual installations should be treated on their merits.

8.6.3 *Miscellaneous*

The sensitivity to vibrations of hospital operating theatres, especially those where microsurgery is undertaken, can well be imagined. Some scientific laboratories are similarly susceptible, whilst a range of other industrial processes ranging from optical typesetting to automatic letter sorting could be inconvenienced. In electrical power generation, turbine shafts are not able to accommodate large oscillatory displacements.

Where there is uncertainty concerning the level of transmitted vibration and its acceptability to the particular environment, it is advisable to investigate the actual conditions and requirements in detail. Preliminary trials and monitoring can then be designed to establish a suitable procedure for the work.

9 Practical measures to reduce vibration

9.1 General

Where the predictions indicate that a particular piling method could prove marginal in terms of critical vibration levels, further consideration should be given to the problem along the lines suggested in 8.4. Additionally, methods of alleviating the problem may be adopted as recommended in 9.2.

9.2 Reduction of transmitted vibration levels

9.2.1 *Use of alternative methods*

As with noise control methods it should be borne in mind that piling and ground engineering processes are primarily selected on the basis of the strata to be encountered, the loads to be supported and the economics of the system. After consideration of these constraints, however, it should be possible to select the process least likely to give rise to unacceptable vibrations in particular circumstances. Examples would include the use of continuous flight auger injected piles, jacked preformed piles, auger bored piles, or possibly impact bored piles in preference to driven piles. Some form of ground treatment might also be possible, depending on soil conditions and loading requirements.

There are sometimes cases in which the majority of a site is amenable to a particular form of ground treatment or foundation construction but where a limited area is too close to existing structures or services to permit unrestricted use of the process. For example, from Table 1 it may be deduced that dynamic compaction using large tamping weights should be kept a reasonable distance away from such features. If a small intervening area remains to be treated this may be done using one of the vibro processes of ground treatment. Similarly, the majority of a site may be piled using the driven cast-in-place process leaving a minority to be completed with continuous flight auger injected piling.

It should be noted that a change in method part of the way across the site might result in a mismatch in subsequent foundation behaviour. The engineering implications of any such changes should be considered carefully prior to construction on site.

9.2.2 *Removal of obstructions*

Obstructions constitute a hindrance to progress and exacerbate the transmission of environmental vibrations, especially where they occur at shallow depths. Obstructions known to exist, e.g. old basement floors, old foundations, timbers, etc., should be broken out at pile or stone column positions and the excavation backfilled. Where an unexpected obstruction is encountered it may be preferable that piling should be halted at that position until such time as the obstruction can be dealt with, rather than attempting prolonged hard driving.

9.2.3 *Provision of cut-off trenches*

A cut-off trench may be regarded as analogous to a noise screen, in that it interrupts the direct transmission path of vibrations between source and receiver. It should be noted that there are serious limitations to the efficacy of trenches. For maximum effect the trench should be as close to the source or to the receiver as possible. The trench should have adequate length and adequate depth. With normally available excavators on site, trench depths are seldom in excess of 4 m or 5 m. The length of the trench needed would be a function of the relevant plan dimensions of the piling site and the structure to be protected.

A trench may constitute a safety hazard. If the trench is not self-supporting, a flexible support mechanism, e.g. bentonite suspension may be needed. Care should be exercised in locating the trench to avoid any loss of support to the structure it is intended to protect or to the piles being installed. Care should also be taken to ensure that the stability of the piling equipment is not endangered by the presence of the trench. The wall of the trench closest to the piling operation may suffer progressive collapse during the course of the works. Provided that the safeguards in this clause are observed, such behaviour is acceptable as an energy releasing mechanism.

At the conclusion of the relevant piling operations the trench should be backfilled carefully to reinstate the site.

Specialist advice should be sought prior to embarking on cut-off trench construction. Trenches should not be regarded as the universal panacea for vibration problems.

9.2.4 ***Reduction of energy input per blow (or cycle)***

Consideration of the relationships 1) and 2) (see Appendix B) suggests that there is a dependence of the peak particle velocity on the energy input. For both relationships, the p.p.v. is seen to depend on the square root of the energy input. For example, halving the energy per blow (or cycle) would produce a p.p.v. of 71 % of its original value. It is sometimes found that reducing energy per blow has an appreciable effect at close quarters, but that at greater distances there is sufficient scatter in the results to indicate that modifications to the energy do not appear significantly to influence the p.p.v.

The penalty for adopting this method is that more blows at lower energy will be needed to drive the piles to a required depth. The trade-off will not necessarily be linear owing to other losses in energy in the system. The advent of modern hydraulic hammers, in particular, has permitted a greater degree of control, and flexibility in selection, of input energy and this may be used to advantage, in combination with appropriate monitoring, to minimize problems. For example, when driving piles close to buildings with shallow foundations or in the vicinity of shallow buried services, monitoring of the vibrations could enable an assessment to be made as to the appropriateness of starting the drive with low hammer drops, subsequently increasing the energy as the toe of the pile reaches the founding stratum at greater depth.

Although in general terms it is accepted that vibrations at any level may contribute to fatigue mechanisms in structures, the relative importance of vibration intensity and number of cycles at that intensity is not sufficiently understood. Under the appropriate circumstances, however, it may be more acceptable, or even preferable, to reduce the energy per blow, thus limiting the p.p.v. but sustaining a longer period of pile driving.

NOTE Special arrangements may be needed where piles are driven to a set. Driving to a set entails counting a number of blows from a standard height of drop (standard for the particular piling system) for a given (small) penetration, or by measuring the penetration obtained after a given number of blows from the standard height of drop. It should be borne in mind that set may not be achieved when using the lower drop height initially chosen to reduce vibration magnitude.

9.2.5 ***Reduction of resistance to penetration***

9.2.5.1 *Pre-boring for driven piles*

When piles are to be driven and there is the risk of excessive vibrations emanating especially from the upper strata, the problem can sometimes be reduced by pre-boring. This process removes some of the soil which would otherwise have to be displaced in the early stages of pile driving. There is some evidence to suggest that the final level of vibration during driving would not be reduced, although there would be a reduction in the number of blows needed to achieve the proper penetration.

A variant of this procedure which can be used with top driven cast in place piling is to commence by driving the tube open-ended. A plug of soil is formed within the tube, which is then withdrawn and the plug is removed. This may be repeated several times before the shoe is fitted and the tube driven closed-ended in the normal manner.

9.2.5.2 *Mudding in for rotary bored piles*

Whilst pre-boring is used in the construction of rotary bored piles in order to reduce the resistance of penetration of temporary casing, it is often coupled with mudding in to reduce the risk of collapse of the sides of the bore.

Following normal pre-boring a small quantity of bentonite slurry is added to the borehole and the auger is rotated rapidly in order to stir up the slurry and any collapsed material from the unlined sides. The casing is then offered into the hole, its penetration being assisted by the lubricating action of the mud slurry. Depending on conditions the final seating of the casing may be assisted either by use of a twister bar (the casing being spun in), or by tapping with a heavy casing dolly or by using a vibrator. The use of these latter two items should, however, be minimized.

9.2.5.3 *Adding water to the bore hole for impact bored piles*

The level of vibration from the impact bored piling method is generally considered acceptable and the method is frequently used on confined sites adjacent to existing structures. The level of vibration increases with the resistance to boring and particularly when the boring tool fails to make measurable progress, for example in dense dry gravel. Progress can be increased by adding water to the bore but great care is needed to ensure that the casing is advanced in pace with the boring tool and that excessive use of water is avoided to reduce overboring and the consequent risk of undermining adjacent structures.

9.2.6 ***Excavation under bentonite***

An alternative procedure for bored piles using very long casings where there are substantial depths of water bearing sands and silts, is to drill the piles under bentonite suspension.

It may then be possible to restrict casing to a relatively short length, thereby avoiding the need to resort to the use of either vibratory or percussive dollies for insertion or withdrawal.

9.2.7 ***Avoidance of shear leg contact with sensitive structures***

Tripod impact bored piling rigs can impart vibrations and shocks through the shear legs. Where, as is often the case, there is a confined working area for a tripod care should be taken in setting up the rig at any pile position, to avoid having one of the legs or its support in direct contact with any adjacent building which may be sensitive to vibrations.

9.2.8 ***Removal of the "plug" when using casing vibrators***

As explained in **A.3**, vibratory drivers have difficulty in penetrating dense cohesionless soils. Where such a machine is used to insert a casing into a stratum of medium dense to dense granular soil, a plug of this soil will accumulate inside the casing. The vibrator will now be confronted with additional resistance, thus slowing penetration and probably accentuating environmental vibration levels.

Provided the boring rig has a sufficiently high rotary table it should be used to drill out the plug at intervals between short periods of vibratory driving. This procedure should substantially reduce the total amount of time needed for use of the vibrator.

9.2.9 ***Bottom-driving***

Claims are made from time to time that bottom-driving results in lower vibration levels than top-driving. The method can be applied to some permanently cased piles and some specialized cast-in-place systems.

The process is certainly quieter than its top-driven counterpart; however any reduction in vibration intensity may be associated with the generally slower rate of production. Maintaining the same rate of pile penetration as top-driving may result in similar vibration levels.

10 Measurement

10.1 Monitoring

In order to ensure optimum control of vibration, monitoring should be regarded as an essential operation. In addition to vibration monitoring, static tell-tale measurements can also be useful. Precision tell-tales are capable of registering longer term trends and can provide early warning of impending structural problems.

It should be remembered that failures, sometimes catastrophic, can occur as a result of conditions not directly connected with the transmission of vibrations, e.g. the removal of supports from retaining structures to facilitate site access.

Where site activities other than pile driving may affect existing structures, a thorough engineering appraisal of the situation should be made at the planning stage.

10.2 Methods of measurement

10.2.1 ***General***

The method selected to characterize building vibration will depend upon the purpose of the measurement and the way in which the results are intended to be used. Although a measurement technique which records unfiltered time histories allows any desired value to be extracted at a later stage, it may not be strictly necessary for the purpose of routine monitoring.

10.2.2 ***Positions***

The number of measurement positions will also depend upon the size and complexity of the building.

When the purpose is to assess the possibility of structural damage, the preferred primary position is in the lowest storey of the building, either on the foundation of the outer wall, in the outer wall, or in recesses in the outer wall. For buildings having no basement, the point of measurement should be not more than 0.5 m above ground level. For buildings with more than one storey, the vibration may be amplified within the building. In the case of horizontal vibration, such amplification may be in proportion to the height of the building, whereas vertical vibration tends to increase away from walls, towards the mid-point of suspended floors.

It may therefore be necessary to carry out measurements (which should be simultaneous if a transfer function is required) at several other positions to record maximum vibration magnitudes. When the building is higher than four floors (approximately 12 m) additional measuring points should be added every four floors and at the top of the building. When the building is more than 10 m long, the measuring positions should be selected at a horizontal spacing not exceeding 10 m. Measurements should be made on the side of the building facing the source.

When the purpose is to evaluate human exposure to vibration in the building, or to assess the effect of vibration on sensitive equipment within the building, measurements should be taken on the structural surface supporting the human body or the sensitive equipment.

When ground vibration sources are being considered it is usual to orientate the transducers with respect to the radial direction, defined as the line joining the source to the transducer.

When studying structural response to ground vibration it is more usual to orientate transducers with respect to the major and minor axes of the building structure.

If it is not possible to make measurements at the foundation, transducers should be well coupled to the ground.

NOTE Information is given in BS 7385-1.

10.2.3 *Parameter to measure*

With an impulsive source of vibration it is usual to measure the peak value attained from the beginning to the end of a drive. It is also usual to measure in terms of peak particle velocity (p.p.v.) if the risk of damage to the building is the primary concern, and there is also an interest in human reaction. If the concern is purely for human tolerance, then acceleration is the preferred parameter. In the case of sensitive equipment, it is necessary to check the environmental vibration limit data supplied by the manufacturer and select accordingly.

Table 2 contains data that assist the selection of instrumentation.

In order to adopt an appropriate cost effective piling procedure, a survey of the sensitivity of the neighbourhood to vibration prior to issuing tender documents is desirable.

10.2.4 *Record sheets*

An important aspect of monitoring vibrations is the preparation and maintenance of records of salient details of the site observations. The format to be adopted will vary according to the circumstances appropriate to each investigation.

NOTE Appendix D contains examples of pro forma record sheets for site measurements and for vibration data summaries which have been devised for a multi-channel digital data acquisition system. Appendix D is included for information only and does not form part of this standard.

10.3 Trial measurements

The various formulae which have been developed empirically to predict vibration levels at a receiving point do not take into account variability of ground strata, the pile-soil interaction process, coupling between the ground and the foundations, etc. Hence these formulae can only provide a first assessment of whether or not the vibrations emanating from a site are likely to constitute a problem.

More accurate assessment can be achieved by the "calibration" of the site, i.e. the establishment of a site-specific formula. The data necessary for the derivation of the formula can be obtained from a trial drive using a piling rig, or by dropping a large weight (typically 1 t to 2 t) in the case of impact driving, on to the ground surface and recording the vibration levels successively at various distances from the point of impact. The preferred method is to cast a 1 m cube of concrete and to drop it from a height of 1.5 m. A range of heights can however be employed, varying between 0.5 m and 2 m. The point of impact should be well away from adjacent structures.

Vibration measurements may also be taken on structures to provide information on the coupling between the soil and the foundations and amplification effects within a building. A range of impact energies should be used to encompass the energy levels associated with the intended piling works.

Table 2 — Vibration effects on different subjects: the parameters to measure and the ranges of sensitivity of apparatus to use

Subject area	Examples	Measurement parameter and ranges of sensitivity
Equipment and processes	Laboratory facilities	Displacement between 0.25 μm and 1 μm in frequency range 0.1 Hz to 30 Hz Acceleration between 10^{-4} g and 5×10^{-3} g in frequency range 30 Hz to 200 Hz
	Microelectronics facilities	p.p.v. between 6 μm/s and 400 μm/s in frequency range 3 Hz to 100 Hz Acceleration between 0.5×10^{-3} g and 8×10^{-3} g in frequency range 5 Hz to 200 Hz
	Precision machine tools	Displacement between 0.1 μm and 1 μm
	Computer	Displacement between 35 μm and 250 μm Acceleration (r.m.s.) between 0.1 g and 0.25 g at frequencies up to 300 Hz
	Microprocessors	Acceleration between 0.1 g and 1 g
People	In dwellings or hospitals	Vertical acceleration (r.m.s.) from 5×10^{-4} g to 5×10^{-2} g in frequency range 4 Hz to 8 Hz Vertical p.p.v. from 0.15 mm/s to 15 mm/s in frequency range 8 Hz to 80 Hz Horizontal p.p.v. from 0.4 mm/s to 40 mm/s in frequency range 2 Hz to 80 Hz
	In offices	Vertical acceleration (r.m.s.) from 1×10^{-3} g to 1×10^{-1} g in frequency range 4 Hz to 8 Hz Vertical p.p.v. from 0.5 mm/s to 20 mm/s in frequency range 8 Hz to 80 Hz Horizontal p.p.v. from 1 mm/s to 52 mm/s in frequency range 2 Hz to 80 Hz
	In workshops	Vertical acceleration (r.m.s.) from 4×10^{-3} g to 6.5×10^{-1} g in frequency range 4 Hz to 8 Hz Vertical p.p.v. from 1 mm/s to 20 mm/s in frequency range 8 Hz to 80 Hz Horizontal p.p.v. from 3.2 mm/s to 52 mm/s in frequency range 2 Hz to 80 Hz
Buildings	Residential or commercial	p.p.v. from 1 mm/s to 50 mm/s
Underground services	Gas or water mains	Displacement from 10 μm to 400 μm p.p.v. from 1 mm/s to 50 mm/s

NOTE 1 Except where root mean square (r.m.s.) accelerations are quoted, all measurement ranges, whether displacement, velocity or accelerations, are in terms of zero-to-peak.
NOTE 2 The ranges given depend on the dominant frequency of vibration (see clause 8).
NOTE 3 Typical ranges from equipment and processes vary considerably, depending on the sensitivity of the equipment installed.
NOTE 4 g_n is acceleration due to gravity, i.e. 9.81 m/s^2.

Appendix A Description of vibration

A.1 Types of vibration

Vibrations may be categorized in several ways as follows:

a) continuous vibrations in which the cyclic variation in amplitude is repeated many times;

b) transient vibrations in which the cyclic variation in amplitude reaches a peak and then decays away towards zero relatively quickly;

c) intermittent vibrations in which a sequence (sometimes regular, sometimes irregular) of transient vibrations occurs but with sufficient intervals between successive events to permit the amplitude to diminish to an insignificant level in the interim periods.

Examples of these types of vibration within the piling field are:

1) continuous vibrations from a vibrating pile driver;

2) transient vibrations from an isolated hammer blow;

3) intermittent vibrations from a drop hammer pile driver.

NOTE Some air operated hammers have sufficiently rapid striking rates to prevent the amplitude of vibration diminishing to an insignificant level between successive events (or impacts). In spite of the impulsive nature of the wave form the resulting vibrations may be described as continuous.

The response of soil and structures to continuous vibrations is to vibrate in sympathy with the vibrating source, i.e. at the same frequency or harmonics thereof. The resulting vibrations are, therefore, known as forced vibrations. Impulsive shocks giving rise to transient vibrations, on the other hand, excite the natural frequencies of the soil-structure combination and thus the resulting vibrations are known as free vibrations.

A.2 Characteristics of vibration

Vibrations are physically characterized as wave phenomena. They may be transmitted in one or more wave types, the most common of which are compression, shear and Rayleigh (or surface) waves. Each type of wave travels at a velocity which is characteristic of the material properties of the medium through which it is propagated. The wave velocity determines the time lag between the event at the source, e.g. the pile position and the remote receiving point. It does not, however, determine the severity of the vibration response at the remote receiving point, although the material properties of the transmitting medium play a significant role in this.

As the wave passes through the receiving point the particles of matter undergo a vibratory or oscillatory motion. It is the intensity of these oscillatory particle motions which determine the vibration response at the receiving point.

The oscillatory motion can be characterized physically in terms of the following:

a) a displacement about the mean value A;

b) a particle velocity v;

c) an acceleration a;

d) frequency of the disturbance f.

In the case of sinusoidal wave propagation these parameters are simply related by the formulae:

$$v = 2\pi f A$$

$$a = 4\pi^2 f^2 A = 2\pi f v$$

where the symbols are each assigned their peak values.

It is not normally practicable to measure all four parameters simultaneously and indeed this is not generally necessary, since for the majority of frequencies of interest in piling operations the peak particle velocity (p.p.v.) is the best indicator of the vibratory response, especially when it is combined with the frequency content of the disturbance. Further guidance on human response to vibrations may be found in BS 6472.

A.3 Vibrations associated with specific operations

A.3.1 *Intermittent and transient vibrations*

A.3.1.1 *Single-acting pile hammers*

Intermittent vibrations are obtained with most single-acting pile hammers. A variety of mechanisms may be used to raise the hammer after each blow, e.g. winch rope, diesel, hydraulic, steam or compressed air. Some diesel and air hammers are double acting and have considerably more rapid striking (or repetition) rates than conventional free fall hammers. This may result in vibrations being set up in certain circumstances (see note to **A.1**).

A.3.1.2 *Impact bored piling*

Traditional impact bored piling gives rise to intermittent vibrations, both in the boring process when the boring tool is allowed to fall freely to form the borehole, and also when temporary casing is being driven or extracted.

A.3.1.3 *Rotary bored piling*

Although rotary bored piling tends to set up low level vibrations, transient vibrations may also occur when the auger strikes the base of the borehole. If it is necessary to insert an appreciable length of temporary casing to support the boring, a casing dolly may be used and, as with the impact bored piling method, this will give rise to intermittent vibrations. The use of special tools, such as chisels, will also result in intermittent vibrations.

A.3.1.4 *Clamshell grabs*

The construction of diaphragm walls and barrettes using clamshell grabs may also give rise to transient or intermittent vibrations. The grabs may be operated either hydraulically, or by rope, but in each case they impact (with open jaws) on the soil in the trench. Since the excavation is filled with a bentonite suspension for temporary support there will be a modest buoyancy factor.

A.3.1.5 *Free falling tamping weights*

Ground treatment by dynamic compaction using large free falling tamping weights results in intermittent vibrations. The process is generally carried out on large sites to improve the density of relatively loose soils or fill materials. The major frequency content of the free vibrations tends to be very low.

A.3.1.6 *Other operations causing intermittent vibrations*

The formation of stone columns using plant designed for driven cast-in-place piling is another source of intermittent vibrations.

A.3.2 *Continuous vibrations*

A.3.2.1 *General*

Continuous vibrations differ from intermittent or transient vibrations in that the vibratory stimulus is maintained through a sequence of cycles. If the frequency of the vibrations coincides with a natural frequency of, e.g. a structural element, then resonance can be induced. The resulting vibrations then exhibit substantially higher amplitudes than otherwise would be the case. This should be borne in mind if the criteria recommended in **8.4.2** are used for the setting of acceptable limits for vibrations at the remote receiving point.

NOTE For continuous vibrations the variables mentioned primarily in conjunction with intermittent vibrations are all significant (except that energy per blow is replaced by energy per cycle) in determining the intensity of vibration.

Continuous vibrations are associated primarily with vibratory pile drivers. They are used for installing or extracting steel sheet and H-section piles and temporary or permanent casings for bored piles. Small vibrators are used for inserting reinforcement cages in continuous flight auger injected piles, and during the extraction of the driving tube following the concreting of a driven cast-in-place pile. The vibration in this latter case assists in compacting the concrete in the pile shaft, and the technique is employed as an alternative to hammering the tube during its extraction.

A.3.2.2 *Vibratory pile drivers*

Vibratory pile drivers can be very effective in loose to medium, cohesionless or weakly cohesive soils. The continuous vibration of the pile member effectively fluidizes the immediately surrounding soil, removing contact friction during a fraction of each vibration cycle. The mechanism is thwarted in dense cohesionless soils and stiff cohesive soils, and a vibrator used at length under these circumstances merely succeeds, in increasing the level of environmental vibrations at the expense of very slow penetration, especially with displacement piles.

Most vibratory pile drivers derive their cyclic axial motion from one or more pairs of horizontally opposed contra-rotating eccentric weights which may be powered hydraulically or electrically. The design operating frequency of these vibrators is typically in the range 25 Hz to 30 Hz which is rather higher than natural frequencies associated with loose or medium loose soil sites. This can lead to a high and possibly dangerous (although short-lived) response at the remote receiving station whenever the vibrator is switched on or off, as it accelerates or decelerates through the range either of site frequencies or of the natural frequencies of floor slabs, etc.

NOTE 1 As a guide, whole building response for buildings up to four storeys in height, as opposed to building element response, generally occurs at frequencies between 5 Hz and 15 Hz. Buildings element response, e.g. slabs, may occur at frequencies between 5 Hz and 40 Hz. For buildings more than four storeys in height, the whole building response frequency is likely to be less than 5 Hz to 12 Hz.

NOTE 2 Care should be taken when using vibrators with frequencies less than 25 Hz.

A.3.2.3 *Resonant pile drivers*

A similar principle to that for vibratory pile drivers applies to very high frequency resonant pile drivers. In this case the vibrator is capable of oscillating at high frequencies (up to 135 Hz) and is designed to tune to one of the natural modes of vibration of the pile being driven, in order to obtain the benefits of pile resonance.

A.3.2.4 *Continuous flight auger injected piling and jacked piling*

The levels of vibration associated with continuous flight auger injected piling and jacked piling are minimal as the processes do not involve rapid acceleration or deceleration of tools in contact with the ground but rely to a large extent on steady motions. Continuous vibrations at a low level could be expected from the prime movers.

A.3.2.5 *Vibroflotation and vibroreplacement*

In ground treatment processes by vibroflotation or vibroreplacement, a rotating eccentric weight in the nose of the machine sets up a mainly horizontal vibration pattern. This is basically a much enlarged version of the familiar vibrating poker used for compacting concrete. Pokers for vibroflotation are generally energized by electric or hydraulic motors and typically operate at frequencies between 30 Hz and 50 Hz.

A.3.2.6 *Vibrating lances*

Another ground treatment process is the installation of vertical band drains. This may be achieved by using a vibrating lance. The vibrator is similar in concept to, but somewhat smaller than, vibrators used for pile driving.

A.3.2.7 *Other operations causing continuous vibrations*

Continuous vibrations, albeit at low intensities, may be experienced from diesel engines, for example from impact bored piling winches mounted on skids, crawler mounted base machines, and attendant plant.

Appendix B Prediction of vibration levels

Simple empirical formulae relating peak particle velocity with source energy and distance from the pile were deduced by Attewell and Farmer[2] from field measurements, and have been used for many years for prediction. More recent studies by Attewell and his co-workers have confirmed and refined their 1973 proposals, with a series of formulae characterizing different types of pile and piling hammer being derived. For the purpose of this appendix it is sufficient to note that a general relationship for hammer-driven piles is:

$$v = 0.75 \times \sqrt{\frac{W_0}{r}} \qquad (1)$$

and for vibratory-driven piles is:

$$v = 1.0 \times \sqrt{\frac{W_0}{r}} \qquad (2)$$

where:

v is the peak particle velocity (vertical component) (in mm/s);

[2] ATTEWELL, P.B., FARMER, I.W., Attenuation of ground vibrations from pile driving. *Ground Engineering*, 6(4), 26-29, 1973.

W_0 is the source energy per blow (or per cycle) (in J);

r is the radial distance between source and receiver (in m).

Use of either of these formulae will enable a prediction to be made of peak particle velocities (p.p.v.) which are unlikely to be exceeded significantly in the vast majority of cases. In fact in many cases the predicted values thus deduced will be found to over-estimate those which will occur in practice, for some or all of the following reasons.

a) Regression analysis of data from numerous case histories was performed on the highest peak particle velocities found in each data set rather than "average" values.

b) Although in driven piling the source of the vibrations is axially directed and therefore predominantly vertical, the three-dimensional nature of the resulting wave pattern ensures that some oscillatory movement will occur in the horizontal plane. Furthermore, horizontal components may well dominate at elevated locations on retained or retaining walls or on structures subject to vibrations from vibroflotation operations.

c) The constant 0.75 in equation (1) reconciles differences in units and averages soil conditions and driving efficiencies. Further commentary on the variations in vibration response depending on the nature of the soil may be found in other publications, e.g. Wiss (1967)[3] and Martin (1980)[4].

d) Where the plan distance between the source and the receiver exceeds the depth of the pile it may reasonably be substituted for the radial distance r. However, when piling close to a structure the r value would be very dependent on pile depth, and so an indication of the depth at which significant resistance to driving is likely to occur would be important in making an assessment. In Table 3, r is generally taken as plan (or horizontal radial) rather than radial distance.

e) Measurements made on the ground surface tend to yield levels which are greater than those made on adjacent load bearing structure. A variation of a factor of 2 is not uncommon (see for example Martin[4] and Greenwood and Kirsch[5]).

f) It can be seen from Table 3 to Table 13 that in many cases satisfactory levels can be achieved when the remote receiving point (see 8.1) is at relatively close quarters. In this nearfield situation it is not practicable to discriminate between the various wave types.

Appendix C Measured vibration levels

Information on measured vibration levels arising from various forms of piling and kindred operations has been summarized in Table 3 to Table 13. Data have been compiled from case histories recorded throughout the UK. Examination of the tabulated results will indicate the magnitude of scatter that can be anticipated.

Notes to Table 3 to Table 13

N/R	Not recorded or not reported
V	Vertical
H	Horizontal
p.p.v.	Where peak particle velocities are quoted the values will normally be resultant or substitute resultant values (i.e. vectorial sums of the three orthogonal components) unless indicated to the contrary.
*	Indicates that the p.p.v. shown has been calculated from measured displacement and frequency of vibration.
+	Indicates that the p.p.v. shown has been calculated from measured acceleration and frequency of vibration.
◆ §	Indicates that some annoyance (human perception of vibration) was reported.
91	See explanation in appropriate "Remarks" entry.

[3] WISS, J.F., *Damage effects of pile driving vibrations*. Highways Research Board USA No. 155. 14-20. 1967.

[4] MARTIN, D.J., *Ground vibrations from impact pile driving during road construction*. TRRL Supplementary Report 549, 1980. Transport and Road Research Laboratory, Crowthorne, Berkshire.

[5] GREENWOOD, D.A., and KIRSCH, K., Specialist ground treatment by vibratory and dynamic methods. *Proceedings of the 1983 Institution of Civil Engineers International Conference on advances in piling and ground treatment for foundations*, 17-45. London, Thomas Telford, 1984.

Ref No.	Where the reference is unprefixed, this represents a case history associated with an actual site. Where investigations yielded inadequate (or no) measurements, they have been omitted. Where the reference number is prefixed by "C", this represents a case history contributed to the CIRIA project RP299. The project report is CIRIA Technical Note 142 by J.M. Head and F.M. Jardine. Only case histories reporting measured vibration levels with relevant distances and some geographical information are included in the table. Where the reference number is prefixed by "M", this represents a case history which does not fall into either of the above two categories.
P	Penetration phase } for vibroflotation/vibroreplacement
C	Compaction of stone column phase

Table 3 — Summary of case history data on vibration levels measured during impact bored piling (tripod)

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
						m	mm/s	m	mm/s	m	mm/s	
1	1971 London EC2	Made ground/gravel/ London clay	Depth 12 m	Boring	N/R	0.9	3.9*	2.4	1.6*	3.7	1.1*	Measured on ground next to 17th century church
2	1972 London SW1	Made ground/soft clay/ballast/ London clay	500 mm ϕ depth N/R 600 mm ϕ depth N/R	Driving casing Base ramming gravel	N/R	2 1.5	3.3* 6.2*	6 3	1.8* 1.9*	6	0.5*	Horizontal radial measurements
3	1973 London EC2	Made ground/peat/ gravel/London clay	500 mm ϕ 20 m depth	Driving casing	N/R	2.5	2.8					Measured on 17th century church
4 ◆	1974 Dundalk (Louth)	Soft silts/gravels/ boulders	N/R	Driving casing	N/R	1.5	2.4					Cracking of adjacent property owing to loss of ground prior to piling
5 ◆	1980 Luton (Beds)	Ballast/chalk	600 mm ϕ 8.5 m depth	Initial boring	N/R	10	0.7					Shored retaining wall in poor condition
6	1980 York (N. Yorks)	Rubble with obstructions/soft silty clay/stiff clay	450 mm ϕ 10.5 m depth	Boring Driving casing Driving casing against obstruction	N/R N/R N/R	1 1.2 1.2	8 4 16	2.5	4	8	2	Adjacent structures elderly with existing cracks
7 ◆	1981 Berwick-upon-Tweed (Northumberland)	Tarmac/soft sandy Silty clay/sandstone bedrock	450 mm ϕ 4 m to 8 m depth	Boring through tarmac Boring obstruction (boulder)	N/R N/R	6 6	6.5 4.25	20	0.7			Vertical 4 mm/s at 6 m. Vertical component only measured
8	1982 Stockton-on-Tees (Cleveland)	Fill including timbers/sand/ boulder clay	450 mm ϕ 13 m to 18 m depth	Driving casing Boring through obstruction	N/R N/R	2.5 4	8 8	3.5 6.5	1 4	8 11	2 2	Old buildings (one listed) adjacent to site
9	1982 London SW1	Fill/sandy silt/wet ballast/London clay below 9 m	600 mm ϕ 12 m depth	Boring	N/R	1.5	2.2					Near to a telephone exchange Trial borings (pre-contract)

Table 3 — Summary of case history data on vibration levels measured during impact bored piling (tripod)

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
						m	mm/s	m	mm/s	m	mm/s	
10	1982 Bristol (Avon)	Soft silts overlying sandstone	500 mm ϕ and 600 mm ϕ 3 m to 12 m depth according to rockhead.	Boring	N/R	4.5	8	7	2.7	12	1.8	Medieval listed buildings adjacent to site
				Chiselling	N/R	4.5	12	10	7	12	3	
				Driving casing	N/R	4.5	4			12	2.5	
			1.5 m penetration rock sockets	Boring	N/R	4.5	2.6	7.5	2.1			After pre-drilling rock
				Chiselling	N/R	4.5	6.5	8	1.7			
11	1982 Halifax (W. Yorks)	Loose rock fill over weathered rock over rock	500 mm ϕ 15 m to 17 m depth	Boring	N/R	10	0.8	25	0.65	48	0.45	Sensitive industrial process in adjacent building
				Base ramming Rockfill	N/R	10	1.5	15	1.3	30	1.2	
12	1983 Swansea (W. Glamorgan)	Made ground/ dense sands and gravel with cobbles and boulders	500 mm ϕ 4.5 m depth	Driving casing	N/R	1	10	10	0.85			Measured on adjacent commercial building
				Boring	N/R	1	9.8	11	0.75			
				Driving casing	N/R	7	6.4	11	1.5			Measured on road surface above 19th century sewer
				Boring	N/R	7	6.6	14	1.4			
13	1983 Lincoln (Lincs)	Backfilled quarry-grouted stiff sandy clay and limestone block/lias clay below 6 m	500 mm ϕ 12 m to 15 m depth	Base ramming	N/R	4.5	22.2	20	1.6			
				Initial boring	N/R	4.5	12.4	20	0.73			
				Driving casing	N/R	4.5	3.3	20	0.41			
				Clay boring	N/R	4.5	0.75	20	0.16			
14	1983 London EC3	Backfilled sand/soft sandy soil/ballast becoming dense with stones/ London clay below 8.7 m	600 mm ϕ 23 m to 25 m depth	Boring (obstruction)	N/R	0.7	9.5	5	3.7			Measured on retained facades
				Boring (stones)	N/R	8	8.9					Different pile position
				Driving casing	N/R	0.7	11.5	5	4.5	8[91]	4.9[91]	

Table 3 — Summary of case history data on vibration levels measured during impact bored piling (tripod)

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
						m	mm/s	m	mm/s	m	mm/s	
15	1984 Guildford (Surrey)	Surface crust/very soft clay/sands and gravels/clay clay horizon between 5 m and 8.5 m	450 mm ϕ 12.5 m depth	Initial boring through crust	N/R	2.5	10.1	3.5	12.3	7	6.5	Sensitive equipment in adjacent building (protected by cut-off trench)
				Driving casing	N/R	2.5	5.5	3.5	5.3	7	3.6	
				Boring soft clay	N/R	3.5	1.1	7	0.8			
16	1984 London EC2	Made ground/dense ballast/London clay below 5.5m	600 mm ϕ 22 m depth	Driving casing	N/R	3	7.1	5.5	2.3	10§	0.9§	Measured on retained facade
				Boring casing	N/R	3	1.1	5.5	1.6	10§	0.86§	
				Shaking clay out of pump	N/R	3	7.5	5.5	0.75	10§	0.45§	
				Boring brick work obstruction	N/R	6	8.6	9	2.6	13§	1.5§	
17 ♦	1985 London EC3	Made ground/dense ballast/London clay below 6.5 m	500 mm ϕ 8 m depth	Driving casing 2 rigs (2nd at 10 m)	N/R	4	2.5					Trial borings Computer equipment beyond party wall

Table 4 — Summary of case history data on vibration levels measured during driven cast-in-place piling (drop hammer)

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
						m	mm/s	m	mm/s	m	mm/s	
18 ♦	1981 London SE1	Made ground/peat/ Thames ballast/ London clay below 10 m	500 mm ϕ 6 m depth with enlarged base	Driving tube Enlarging base	N/R	20 20	2.7 3.6	100 100	0.96 1.4			Bottom-driven
19 ♦	1982 London SW6	Fill/ballast/ London clay	500 mm ϕ 4 m to 7 m depth with enlarged base	Driving tube Expelling plug Enlarging base	N/R N/R N/R	30 30 30	2.3 2.6 2.3					Bottom-driven
20 ♦	1983 Aylesbury (Bucks)	Fill/soft material/clay becoming stiff	450 mm ϕ 10 m depth with enlarged base	Driving tube Expelling plug Enlarging base	N/R N/R N/R	4 4 4	8.4 6.1 4.0	20 20 20	5.0 4.8 4.1			Bottom-driven
21 ♦	1983 Aldershot (Hants)	Dense fine sand	450 mm ϕ approx 6 m depth	Driving tube	58.9 kJ	120	1.0					Tube driven open ended initially to remove some sand prior to driving with shoe top-driven
22 ♦	1983 Horsham (W. Sussex)	Peaty, silty alluvia over shale and sandstone	350 mm ϕ 7.5 m to 8 m depth	Driving tube Extracting tube	38.8 kJ	21 21	2.9 3.2	28 28	2.7 3.9	35 35	2.4 3.1	Top-driven
23 ♦	1983 Redhill (Surrey)	Dense fine sand with ironstone bands	450 mm ϕ 8 m depth (max) (6 m average)	Driving tube Expelling plug	N/R	22.5	3.1	43 43	1.1 1.25			Bottom-driven, computer etc. in adjacent building
24 ♦	1984 Weymouth (Dorset)	2 m to 3 m thick crust of sands and gravel over astuarial silty clay becoming firmer at greater depth	350 mm ϕ 15 m depth Some with enlarged base	Driving tube open ended Driving tube with shoe Extracting tube Enlarging base	47.1 kJ 47.1 kJ	8.5 8.5 8.5 25	6.1 8.3 2.9 2.2	13 13 25	3.6 4.4 2.1			Top-driven
25 ♦	1984 Cambridge (Cambs.)	4.75 m to 6.75 m loose fill over gault clay becoming stiffer with depth	350 mm ϕ 10 m to 11 m depth with enlarged base	Driving tube Enlarging base Extracting tube	47.1 kJ	13 13 13	5.6 4.9 4.6	22 22 22	3.1 1.9 2.5	34 34 34	2.6 1.1 1.6	Top-driven, sensitive equipment in adjacent building

Table 4 — Summary of case history data on vibration levels measured during driven case-in-place piling (drop hammer)

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances: Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	Remarks
						m	mm/s	m	mm/s	m	mm/s	
26 ♦	1984 London E14	Fill over Thames ballast	400 mm ⌀ 5 m depth	Driving tube	47.1 kJ	5.5	10.7	12	5.9	21	3.4	Top-driven, close to main service pipes
				Extracting tube		5.5	3.2	12	2.8	21	2.0	
27 ♦	1984 Isleworth (Greater London)	Clayey fill/London clay	350 mm ⌀ 10 m to 12 m depth Some with enlarged base	Driving tube	23.5 kJ	30	1.05	35	0.95	40	0.66	Top-driven, measured on suspended floor in a computer room
				Enlarging base		35	0.76					
				Extracting tube		30	0.55					
28 ♦	1984 Portsmouth (Hants)	Dense fine sand	400 mm ⌀ 4 m to 6.5 m depth	Driving tube	47.1 kJ	50	1.2	63	0.72			Top-driven
				Open ended driving tube with shoe		50	1.0	63	0.83			
				Extracting tube		50	0.37	63	0.31			
29	1984 London E1	Soft fill over dense Thames ballast below 4.5 m	400 mm ⌀ 5.5 m to 6 m depth with enlarged base	Driving tube (fill)	N/R	10	2.2					Bottom-driven, measured at base of riverside wall
				Driving tube (ballast)		10	7.7					
				Expelling plug		10	3.6					
				Enlarging base		10	6.9					
30 ♦	1985 Enfield (Greater London)	Fill/dense gravel/London clay below 5 m to 6 m	350 mm ⌀ 9 m to 11.5 m depth Some with enlarged base	Driving tube (gravel)	47.1 kJ	9.2	37.9	18.5	17.3			Top-driven, measured on earth retaining embankment
				Driving tube (clay)		9.2	10.3	18.5	2.4			
				Enlarging base		19.5	1.8	29.7	1.1			
31 ♦	1985 Littlehampton (W. Sussex)	Fill/very soft silty clay/thin layer of gravel/weathered chalk below 8 m to 9 m	350 mm ⌀ 10 m to 11 m depth with enlarged base	Driving tube	N/R	14	2.2	24	0.82	30	0.88	Bottom-driven
				Expelling plug		14	2.2	24	1.8	30	1.3	
				Enlarging base		14	2.3	24	0.88	30	1.0	
32 ♦	1985 Mitcham (Greater London)	Sub-surface crust of Hogging/London clay below 2 m to 3 m	350 mm ⌀ 9 m to 12 m depth Some with enlarged base	Driving tube	47.1 kJ	28	3.2	34	2.8	42	1.7	Top-driven (listed building)
				Enlarging base		37	1.2					
				Extracting tube		28	1.7	34	1.5	42	0.84	
33 ♦	1985 Uxbridge (Greater London)	Fill (including pockets of gravel) London clay below 3 m	350 mm ⌀ 5 m to 12.5 m depth Some with enlarged base	Driving tube	23.5 kJ to 35.3 kJ	10	4.2 (v)	14	2.2 (v)			Top-driven
				Driving tube after preboring		5.5	3.3	9	2.0	13	1.4	
				Enlarging base		5.5	2.8	9	3.5	13	2.8	
				Extracting tube		5.5	5.9	9	3.4	13	2.9	

Table 5 — Summary of case history data on vibration levels measured during dynamic consolidation

Ref. No.	Year and location	Soil conditions	Tamping weight	Mode	Measured peak particle velocity (p.p.v.) at various plan distances: Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	Remarks
			t			m	mm/s	m	mm/s	m	mm/s	
34	1973 Corby (Northants.)		9	Pass 1	up to 1.59 MJ	25	3.0*	225	0.16*			
				Pass 2	up to 1.59 MJ	25	4.7*	120	0.33*			
35	1973 Belfast (Antrim)	Clay fill	10		1.47 MJ	8	42	26	3.6	44	1.75	Dropping onto virgin ground
					1.96 MJ	14	12	25	3.2	49	1.35	Dropping on to fill
					981 kJ	14	10	25	2.9	49	1.4	
36	1974 Teesside (Cleveland)	Hydraulic fill of clean sand with some pebbles	17	Pass 1	2.50 MJ	5	240	12	53	20	15.5	
				Pass 2	2.50 MJ	5	177	12	67	20	20.3	
37 ♦	1975 Canterbury (Kent)	Sand fill containing much fine silt	N/R		20 m drop	12	16.5	20	5.8	32	2.7	
					15 m drop	10	20.5	20	6	32	3.3	
					10 m drop	12	15.5	20	4.5	28	2.2	
38 ♦	1975 Glasgow Govan (Strathclyde)	Old docks backfilled with well-graded permeable granular fill	15	Pass 1	2.94 MJ	15	22	30	13.5	50	9	Comparison between various tamping weights and drop heights
				Post-treatment	2.94 MJ	15	30	30	12	50	8.3	
				Post-treatment	2.21 MJ	15	27	30	10	50	8.5	
				Post-treatment	1.47 MJ	15	27	30	10	50	6.5	
			15 (small base)	Post-treatment	2.94 MJ	15	35	30	12	50	8.0	
			2 (ball)	Post-treatment	392.1 kJ	15	9	30	2.5	50	2.0	
39	1975 Cwmbran (Gwent)	Loose fill in old clay quarry; depth 7 m to 20 m	N/R		20 m drop	27	5.8					
40	1976 Port Talbot (W. Glamorgan)	Slag fill	15		2.94 MJ	75	2.1	250	0.16			Measured at ground level
					2.94 MJ	75	7.2	250	1.4			Measured at top of 30 m high silo
41 ♦	1978 London SE16	Old docks backfilled with various materials	10	Pass 1	981 kJ	24	8.9	40	4.6	70	2.0	
					1.96 MJ	24	13.5	40	11.2	70	2.0	
	1979	including	10	Later pass	1.96 MJ	10	52.3	22	8.9	65	2.2	
		cohesive clay soils with	15	Later pass	2.94 MJ	16	15					
	1980	substantial voidage; depth	15	Pass 1	2.94 MJ	20	11.6	27	6.5	34	5.1	
	1981	9 m to 11 m	15	Pass 1	3.24 MJ	150	1.6					

Table 5 — Summary of case history data on vibration levels measured during dynamic consolidation

Ref. No.	Year and location	Soil conditions	Tamping weight	Mode	Measured peak particle velocity (p.p.v.) at various plan distances: Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	Remarks
			t			m	mm/s	m	mm/s	m	mm/s	
42 ♦	1979 Walsall (W. Midlands)		15	Pass 1	2.21 MJ	60	4.1					
					1.47 MJ	60	3.5					
					735.8 MJ	60	3.1					
43 ♦	1982 Southampton (Hants)	Old refuse tip; depth 3 m to 5 m	8	Pass 1	1.37 MJ	10	15.9	16	11.0	27	6.2	Measured on pipeline
				Pass 1	1.37 MJ	25	9.0	35	6.9	49	4.7	Measured on house
44	1983 Glasgow Finnieston (Strathclyde)	Shaley fill: depth 10.5 m	15	Pass 1	3.09 MJ	75	5.2	100	2.8			
45 ♦	1984 Kingswinford (W. Midlands)	Old sand quarry backfilled with mainly granular material including foundry sand	15		2.65 MJ	32.5	8.9					Tamping on very shallow fill
					2.65 MJ	19	8.5	36	6.3	50	3.3	Tamping on deeper fill
					2.65 MJ	150	0.89					
46 ♦	1984 Dudley (W. Midlands)	Old opencast mine, filled with colliery shale in cohesive matrix	8	Pass 1	1.26 MJ	70	4.6	85	3.2			Measured on 300 year old building
				Pass 2	1.26 MJ	72.5	4.4	82.5	3.4			Measured on modern house
						65	3.7					
47 ♦	1984 Glasgow Kingston (Strathclyde)	Miscellaneous slightly cohesive fill; depth 6 m to 7 m	8	Pass 1	1.18 MJ	15	5.1	30	4.2	45	2.3	Deep cut-off trench between treatment area and monitoring position
				Pass 1	1.18 MJ	60	1.9	75	1.4	90	1.4	
	1985			Pass 1	1.18 MJ	15	12.7	30	5.4	70	3.0	Measured on metal rack 0.9 m above ground level
				Pass 1	1.18 MJ	15	24.3	30	9.7	70	5.5	Measured on metal rack 2.7 m above ground level
48 ♦	1985 Aberdeen (Grampian)	Demolition rubble, silty sands, peats, etc., overlying beach sand. Depth of fill up to 15 m	15	Pass 1	2.65 MJ	19	13.7	27	13.0	51	7.1	
				Pass 2	2.65 MJ	40	3.3					Very soft fill in this area
				Pass 2	2.65 MJ	55	6.1	70	5.1			

Table 5 — Summary of case history data on vibration levels measured during dynamic consolidation

Ref. No.	Year and location	Soil conditions	Tamping weight	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
			t			m	mm/s	m	mm/s	m	mm/s	
49 ♦	1985 Gravesend (Kent)	Old domestic fill including bottles overlying Thanet sands and chalk. Depth of fill 1.5 m to 6 m	8	Pass 1 Pass 2	1.26 MJ 1.26 MJ	50 50	2.8 2.6					
50 ♦	1985 Preston (Lancs)	Old brickworks clay pit backfilled with loose ash, bottles, etc. Depth of fill 4 m to 5.5 m	15	Pass 1 Pass 2	2.94 MJ 1.47 MJ	38 38	6.5 8.1					Fill very shallow
51 ♦	1985 Exeter (Devon)	Old quarry backfilled with rubble, clays and miscellaneous waste overlying hard shale. Depth of fill 4 m to 12 m	8	Pass 1	1.26 MJ	30	4.2					

Table 6 — Summary of case history data on vibration levels measured during vibroflotation/vibroreplacement

Ref. No.	Year and location	Soil conditions	Depth of treatment	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per cycle	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
			m		kJ	m	mm/s	m	mm/s	m	mm/s	
52	1973 Newport (Gwent)	Demolition rubble in old basements	N/R	N/R	3.0	3 3	7.9* 7.3*	6 6	4.5* 6.3*	12 12	2.7* 1.9*	Vertical Horizontal
53	1973 Manchester Central (Greater Manchester)	Unspecified fill	N/R	N/R	3.0	3.5	5.1*					Horizontal
54 ♦	1974 Worcester (Hereford and Worcester)	N/R	N/R	N/R	1.64	2.4	2.0					
55 ♦	1974 London E9	N/R	3	Airflush	3.0	6.5 13.0	12.7 10.5					Measured on ground surface Measured at mid height of 3 m high brick boundary wall
56 ♦	1974 Sandgate (Kent)	N/R	N/R	N/R	3.0	2	24.0	5	10.0	20	1.6	
57 ♦	1975 Hemel Hempstead (Herts)	Loose chalk fill	6	N/R	3.0	1 6.7	18.0 2.5	2 14.5	15.0 0.6	2.9	5.0	Vertical Vertical
58 ♦	1975 Oxford (Oxon)	Disused limestone quarry backfilled with rubble	3 to 4	N/R	3.0	12	2.6					
59	1975 Port Talbot (W. Glamorgan)	Soft alluvium with surface crust	9.2	Waterflush	3.0	8	3.2					Vertical
60	1976 Bradford (West Yorks)	N/R	N/R	N/R	3.0	0.6	19	1.2	8			
61 ♦	1976 Sutton Coldfield (W. Midlands)	Backfilled sand quarry	3 to 4	Airflush	3.0	25	1.4					
62 ♦	1976 Oxford (Oxon)	As for no. 58	3 to 4	N/R	3.0	15	1.9	20	1.1			

Table 6 — Summary of case history data on vibration levels measured during vibroflotation/vibroreplacement

Ref. No.	Year and location	Soil conditions	Depth of treatment	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per cycle	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
			m		kJ	m	mm/s	m	mm/s	m	mm/s	
63 ♦	1976 London SW11	Demolition rubble in old basements	2.5 to 4	N/R	3.0	4	10.1	6	6.7	10	2.1	
64	1976 Manchester Moston (Greater Manchester)	N/R	3	Airflush P	3.0	14	2.1	29	0.36	60	0.21	Cut-off trench
65 ♦	1978 Doncaster (S. Yorks)	Wet crushed limestone fill surrounding ground granular with high water table	5	Waterflush	3.0	22	0.98	57	0.18			32 Hz
						22	0.45	57	0.13			21 Hz
66 ♦	1979 York (N. Yorks)	Ash and clinker fill overlying clay	3 to 3.5	Airflush P	3.0	25	1.4					some alleged architectural damage
				C	3.0	25	1.3					
67 ♦	1980 Nottingham (Notts)	Demolition rubble in basements	3	Airflush	3.0	4.5	16.7	12	8.1	22	2.6	Ground surface measurement
68 ♦	1980 Stanstead Abbots (Herts)	Fill over soft silty clay over ballast	2 to 4	Airflush	3.0 3.0	17 17	1.6 0.82					First floor timber beam Ground floor house wall
69	1980 Rochdale (Greater Manchester)	Mixed fill of clayey consistency	2 to 5	Airflush P	3.0	2.5	17.8	4.5	5.8	6	5.7	Brief surge at end of penetration
				C	3.0	2	5.6	4.5	3.3			Shallow cut-off trench to protect service pipe
70 ♦	1980 Datchet (Berks)	Silty sand fill over chalk or sand and gravel	1.5 to 3	Airflush P	3.0	6	5.0	15	1.2			These holes partially prebored with 350 mm auger
				P C	3.0 3.0	26 26	1.9 2.4	40	0.95			Measured at first floor level
				P	3.0	23	1.4	38	0.65			Measured at ground level
				C	3.0	23	1.7					No pre-boring of holes

Table 6 — Summary of case history data on vibration levels measured during vibroflotation/vibroreplacement

Ref. No.	Year and location	Soil conditions	Depth of treatment	Mode		Measured peak particle velocity (p.p.v.) at various plan distances: Theoretical energy per cycle	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	Remarks
			m			kJ	m	mm/s	m	mm/s	m	mm/s	
71	1980 Belfast (Antrim)	Weak sandy clay	Up to 7	Airflush	P	3.0	5	2.9	8.3	1.9	8.3	1.5	
					C	3.0	3.5	5.0	5	2.4			
				Waterflush	P	3.0	3.8	1.4	6.6	0.78			
					C	3.0	3.8	1.1	6.6	0.81			
72	1981 Brigg (S. Humberside)	Fine silty sand	3	Waterflush	P	1.64	1.5	5.4	2.5	3.1	5	2.1	
					C	1.64	1.5	3.5	2.5	3.0	5	2.5	
73	1981 Huddersfield (W. Yorks)	Ash and brick rubble fill	3 to 3.5	Airflush	P	3.0	2.5	34.7	4.6	19.7	11.8	8.7	Ground surface measurements
					C	3.0	2.5	48.0	4.6	18.2	11.8	3.8	
					P	3.0	5.5	7.5	7.6	3.9			Measured on underground service pipe
					C	3.0	5.5	8.4	7.6	5.1			
74 ♦	1981 Cardiff (S. Glamorgan)	Backfilled railway cutting: slag fill	2 to 3	Airflush	P	3.0	6	3.5	20	0.57			
					C	3.0	6	3.3	20	0.78			
75	1982 Birmingham Hockley (W. Midlands)	Demolition rubble in collapsed basements	3	Airflush	P	3.0	5	2.6	8	1.6	11	1.1	Measured on old brick sewer
					C	3.0	5	3.5	8	1.8	11	0.98	
76 ♦	1983 Datchet (Berks)	Miscellaneous fill including dense fine sand and very loose sand	3	Airflush	P	3.0	8	4.9	12	3.8	20	1.3	Measurements on end terrace house with existing defects
					C	3.0	8	2.0	12	3.2	20	1.8	
77	1983 Rugeley (Staffs)	Demolition rubble fill to 3 m over sands and gravels	3	Airflush	P	3.0	6	16.1	10	8.6	22	2.0	Ground surface measurements
					C	3.0	6	8.6	10	5.8	22	1.9	
					P	3.0	4	35.2	7.5	4.5	16	1.4	Measured on top of retaining wall
					C	3.0	4	25.7	6.5	8.6	16	1.3	
78 ♦	1983 Tewkesbury (Glos)	Made ground including raised shingle	3	Airflush	P	3.0	6	12.5	15	2.9	27	0.87	
					C	3.0	6	9.1	15	3.1	27	0.87	
					P	3.0	3.5	22.3	10	15.5			Measurements on free-standing manhole surround
					C	3.0	3.5	25.7	10	11.6			
79 ♦	1983 Newcastle-upon-Tyne (Tyne and wear)	Ash and brick rubble fill	2.5 to 6	Airflush	P	3.0	5.5	2.5	7.5	2.0	15	1.5	Encountered buried obstruction
					P	3.0	11	2.6					

Table 6 — Summary of case history data on vibration levels measured during vibroflotation/vibroreplacement

Ref. No.	Year and location	Soil conditions	Depth of treatment	Mode	Measured peak particle velocity (p.p.v.) at various plan distances: Theoretical energy per cycle	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	Remarks
			m		kJ	m	mm/s	m	mm/s	m	mm/s	
80	1983 Oxford (Oxon)	Miscellaneous fill over weak cohesive soil over gravel	2.2	Airflush P	3.0	1.9	7.6	4	2.4	10.5	1.1	Cut-off trench
				C	3.0	1.9	6.9	4	2.3	10.5	0.55	
81	1983 London E1	Demolition rubble and other fill over gravel	1.5 to 2.5	Airflush P	3.0	18	0.75	26	0.44	32	0.15	Sensitive industrial processes nearby
				C	3.0	18	0.76	26	0.62	32	0.15	
82	1984 London SW6	Brick rubble fill over clayey sand and sands and gravels	2.5 to 3	Airflush P	3.0	3.5	12.6	5	10.7	18	1.6	Measured on service pipes
				C	3.0	3.5	16.5	5	10.3	18	1.7	
83 ♦	1984 Gravesend (Kent)	Ash, brick and demolition, rubble backfilled into old basements	2.5 to 3	Airflush P	3.0	8	2.4	14	1.2			
				C	3.0	8	2.1	14	0.9			
84	1985 Dudley (W. Midlands)	Granular fill over clay over black coal shale	2.5 to 4	Airflush P	3.0	3.5	7.4	6	5.4	15	1.4	Cut-off trench, measured on service pipe
				C	3.0	3.5	5.5	6	2.7			
85 ♦	1985 Birmingham Bordesley (W. Midlands)	Miscellaneous fill over stiff clay	2 to 2.5	Airflush P	3.0	3.5	7.7					Cut-off trench
				C	3.0	3.5	4.2					
86 ♦	1985 Hull (N. Humberside)	Miscellaneous fill over dense loamy sand	4	Airflush	3.0	12	8.1					
87 ♦	1985 Worcester (Hereford and Worcester)	Fill including sands, rubble and porcelain waste over dense gravel	3	Airflush P	3.0	9	5.5	13	3.3	26	1.2	Cut-off trench

Table 7 — Summary of case history data on vibration levels measured during the use of casing vibrators

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per cycle	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
					kJ	m	mm/s	m	mm/s	m	mm/s	
88	1973 Isle of Grain (Kent)	Hydraulically placed sandfill over estuarial silts over ballast over london clay	815 mm ϕ 24.4 m depth permanent liner	Driving liner	4.35 to 6.3	1	10	2	3.2	3	0.8	25 Hz
				Driving liner	6.9 to 8.5	8	4.1	11	2.2	16	1.5	12 Hz to 15 Hz
89 ♦	1974 London W6	Fill over ballast over London clay	750 mm to 1050 mm ϕ depth 2.5 m to 9 m	Driving casing	2.18 to 3.15	1.3	8.0	2	6.4	6.6	1.5	Vertical 25 Hz
				Extracting casing	2.18 to 3.15	2	5.0	6.6	3.2			Vertical 25 Hz Sensitive equipment in adjacent building
90 ♦	1976 London EC4	Fill over ballast over London clay	750 mm to 1050 mm ϕ	Driving casing	2.18 to 3.15	3	5.8					25 Hz
91 ♦	1976 London E1	Fill over ballast over London clay	N/R	Driving casing	2.18 to 3.15	10	1	25	1.5			25 Hz
92	1980 Newark-upon-Trent (Notts)	Alluvia/gravels/marl	750 mm ϕ 10 m depth	Driving casing	2.18 to 3.15	35	0.29	50	0.24	75	0.16	25 Hz
				Extracting casing	2.18 to 3.15	50	0.31	75	0.23			25 Hz Sensitive equipment in nearby building
93 ♦	1980 London E1	Fill/dry gravel/clay	900 mm ϕ 10 m depth	Extracting casing	4.35 to 6.3	40	1.3					17 Hz
94 ♦	1981 London SE1	Fill/gravels/clay	N/R	Driving casing	2.18 to 3.15	30	0.8					Vertical 25 Hz
95	1981 Reading (Berks)	Peat, silts and gravels/putty chalk with flints/firm chalk	600 mm to 1050 mm ϕ 10 m to 15 m depth	Driving casing	2.18 to 3.15	8	4.6	16	1.1	24	0.24	25 Hz
				Extracting casing	2.18 to 3.15	4.5	5.8	10.5	0.7			25 Hz
96 ♦	1981 London EC3	Fill/dense ballast/clay	750 mm to 1500 mm ϕ 9 m depth	Driving casing	2.18 to 3.15	30	0.88	73	0.19			25 Hz
				Extracting casing	2.18 to 3.15	25	1.5	65	0.11			25 Hz
97 ♦	1981 London SE1	Fill/ballast/clay	9 m depth	Extracting casing	2.18 to 3.15	25	1.5					25 Hz

Table 7 — Summary of case history data on vibration levels measured during the use of casing vibrators

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per cycle	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
					kJ	m	mm/s	m	mm/s	m	mm/s	
98	1984 Barrow-in-Furness (Cumbria)	Hydraulically placed sand fill/boulder clay marl	1 350 mm ⌀ 8 m depth concentric with 1 200 mm ⌀ 17.5 m depth permanent liner	Driving-outer casing	26.1	19	13.1					Warning up 10 Hz
					15.35	19	9.2					17 Hz
99 ♦	1985 Hatfield (Herts)	Clay over gravels	90 mm ⌀ 15 m depth anchor casing	Driving casing	1.25	11	0.8					Anchor casings driven at 30° to horizontal
				Extracting casing	1.25	8	1.3	11	0.8			

Table 8 — Summary of case history data on vibration levels measured during rotary bored piling (including casing dollies)

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
					kJ	m	mm/s	m	mm/s	m	mm/s	
100 ♦	1974 London W6	Fill/gravel/ London clay	N/R	Driving casing With 3 t dolly		7 7	3.2 1.0					Horizontal Vertical
101	1981 London EC3	Fill/dense ballast/London clay	1 050 mm ϕ	Augering		20	0.05					Listed building nearby
				Auger hitting base of hole		20	0.23					
102	1982 Cheltenham (Glos.)	Fill/wet sand/lias clay	900 mm ϕ	Augering		9	0.2					Listed building adjacent to site
				Hammering casing with Kelly bar		9	0.8					
103	1983 Romford (Greater London)	Fill clay	350 mm ϕ 14.5 m depth	Augering		10	0.38	20	0.3	30	0.03	
				Dollying casing	11.8	10	1.1	20	0.55			2 t dolly
				Auger hitting base of hole		10	0.96					
				Spinning off		10	0.57	20	0.44			
104	1985 London W1	Fill/sand/clay	500 mm ϕ	Augering		10	0.4	15	0.1	26	0.02	
				Auger hitting base of hole		14	0.3	26	0.1			
				Mudding in		10	0.3	14	0.2			
				Spinning off		10	0.3					
				Dollying casing	11.8	10	1.0	14	0.8			2 t dolly
105	1985 St. Albans (Herts)	Sands and gravels over chalk	600 mm ϕ 12 m depth	Augering		3.5	0.23	8	0.04			
				Auger hitting base of hole		3.5	2.4	8	1.7			
				Spinning off		6	0.08	8	0.06			
106	1985 Portland (Dorset)	6 m of soft ground over rock	600 mm ϕ 7 m depth	Augering		5	0.54					Sensitive equipment in adjacent building
				Surging casing		5	0.36					
				Twisting in casing		5	0.22					
				Spinning off		5	0.42					
				Boring with rock auger		5	0.43					
107	1985 Uxbridge (Greater London)	Fill including pockets of gravel over London clay	350 mm ϕ 7 m depth	Augering		5.5	0.13					Preboring for a driven pile

Table 9 — Summary of case history data on vibration levels measured during tripod bored piling

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
					kJ	m	mm/s	m	mm/s	m	mm/s	
C1 ♦	1971 London WC2	Overburden over London clay	N/R	Driving casing	N/R	1	12.5					
C2 ♦	1971 London SW1	Sand and gravel over London clay	500 mm ⌀ 17 m depth	N/R	N/R	11	2.6	42	0.31			
C3 ♦	Bury (Greater Manchester)	Sand and gravel/soft silty clay/hard glacial till	300 mm ⌀	N/R	N/R	15	4.0					

Table 10 — Summary of case history data on vibration levels measured during driven sheet steel piling

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
					kJ	m	mm/s	m	mm/s	m	mm/s	
C4 ♦	N/R Aldermaston (Berks)	3 m to 4 m sandy gravel over London clay	N/R	Air hammer driving sheets	15	12	0.05					Vertical
C5 ♦	N/R Bridlington (Humberside)	4 m to 5 m soft saturated sand over soft to firm clay	N/R N/R	Air hammer driving sheets Extracting sheets	6.4 7.6	6 6	1.1 0.14					225 Blows per min 150 Blows per min
C6 ♦	N/R Canvey Island (Essex)	Clay/soft silty clay/silty sand; high water table	Fordingham 3 X 8 m depth	Drop hammer driving sheets	4.5 t hammer drop N/R	35 35	3.0 0.5					Vertical Horizontal
C7 ♦	N/R Montrose (Tayside)	N/R	Larssen	Driving sheets	32 to 73	11.7	4					Vertical
C8 ♦	1971 London WC2	Overburden/London clay	N/R	Diesel hammer driving sheets Air hammer driving sheets	N/R N/R	1 1	20 10					
C9 ♦	1974 Lancashire	Fill/firm to stiff boulder clay/sandy stony clay/firm boulder clay	N/R	Driving sheets		33	0.89*					Horizontal
C10	1978 Crail (Fife)	Clay/rock	N/R	Drop hammer driving sheets	39.2	15 15	0.79* 0.48					Vertical, pile in clay Vertical, pile on rock
C11 ♦	N/R Hull (Humberside)	Fill/6 m alluvium/1 m to 6 m peat, clay, sand and silt/1.3 m sand and gravel/5 m stiff clay/9 m dense sand/hard chalk	Larssen no. 6 34 m depth Penetration 1 m into chalk; 27 m in total	Diesel hammer driving sheets	71.6 to 143.2	30 30 30	1.1 0.35 0.6	130 130 130	0.1 0.1 0.1	250 250 250	0.025 0.015 0.025	Horizontal radial Horizontal Transverse vertical

Table 10 — Summary of case history data on vibration levels measured during driven sheet steel piling

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
					kJ	m	mm/s	m	mm/s	m	mm/s	
C12 ♦	1978 Hazel Grove (Greater Manchester	Stiff clay/dense sand including clay bands	Frodingham 2 N	Drop hammer driving sheets	19.9	11	16	26	12.5	54	2.6	
C13 ♦	1978 Oldham (Greater Manchester)	N/R	N/R	Diesel hammer driving sheets	N/R	60	2.5 †					Vertical
C14	N/R Cambridge (Cambs)	Loose to medium sands over clay	N/R	Driving sheets	N/R	2 2	10 2					Vertical Horizontal
C15 ♦	1979 Molesey (Surrey)	Gravel over London clay	N/R	Diesel hammer driving sheets	N/R	5 5	13.5 40.4					on bungalow on ground surface
C16	1979 Rochdale (Greater Manchester)	N/R	N/R	Driving sheets	N/R	6	1.9					
C17	N/R Cambridge (Cambs)	Fill/sand and gravel/gault clay	Frodingham 1 B 6 m depth	Drop hammer driving sheets	13.5	1	9.1*					
C18	1980 Newton Heath (Greater Manchester)	N/R	N/R	Driving sheets	N/R	300	0.015					Vertical
C19	1981 Denton (Greater Manchester)	Firm sandy glacial till	14 m depth	Diesel hammer driving sheets	N/R	0.9	15					Vertical

Table 11 — Summary of case history data on vibration levels measured during driving of bearing piles

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
					kJ	m	mm/s	m	mm/s	m	mm/s	
C20	N/R Glasgow Cowcaddens (Strathclyde)	3 m fill, blaes, clay and boulders over 8 m soft to firm silty clay over sandstone	305 mm × 305 mm Steel H-pile	4 t drop hammer driving pile	N/R	13	0.19*					Vertical
C21	N/R Drax (N Yorks)	Granular fill, lacustrine deposits, sand, sandstone	Precast concrete 400 mm × 400 mm	Diesel and drop hammers driving piles	24.5 to 88.2	3	13					Vertical
C22	N/R Kinneil (Central)	N/R	N/R	Driving pile	N/R	6	5.2					
C23	N/R Leeds (W. Yorks)	4 m fill/2 m alluvial granular soils/rock	Driven cast-in-place dimensions N/R	Driving pile	N/R	12	5.1	23	1.4			When driven 1.5 m
C24	N/R Middlesbrough (Cleveland)	22 m firm becoming stiff boulder clay over marl	Driven cast-in-place dimensions N/R	Driving pile	N/R	12	11.6	30	4.7	45	1.45	
C25	N/R Ravenscraig (Strathclyde)	N/R	305 mm × 305 mm Steel H-pile	Diesel hammer driving pile	N/R	25	0.13					
C26	N/R Reading (Berks)	N/R	Driven cast-in-place dimensions N/R	Driving pile	N/R	60 90	0.07 0.12					Measured on fifth floor of office building
C27	1968 Wylfa (Gwynedd)	Rockfill and clay over mica schist	Steel H-pile	Diesel hammer driving pile	N/R	1	18					Vertical
C28	1969 Ince (Cheshire)	Alluvial peat and clay, boulder clay, sand, bunter sandstone	305 mm × 305 mm Steel H-pile	Diesel hammer driving pile	43.4	8	1.4					

Table 11 — Summary of case history data on vibration levels measured during driving of bearing piles

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
					kJ	m	mm/s	m	mm/s	m	mm/s	
C29 ♦	1972 Derby (Derbys)	N/R	400 mm to 450 mm ⌀ Driven cast-in-place	Driving pile tube	N/R	15	2.2					
C30 ♦	1972/3 Bristol (Avon)	Fill and alluvium over keuper marl	Simulation test for driven shell piling	Dropping test weight on ground	58.8	25	0.7					Vertical on ground
C31 ♦	1977 Southampton (Hants)	2 m to 3 m granular fill over bracklesham beds, very compact clayey fine sand	275 mm × 275 mm × 9 m depth pre-cast concrete piles	Drop hammer driving pile	N/R	25	2.15					Holes prebored to 3 m depth
C32 ♦	1977 Middlesbrough (Cleveland)	Made ground/9 m to 12 m firm to stiff laminated clay/4 m to 6 m glacial till/hard keuper marl	480 mm ⌀ Cast-in-place piling length N/R	Drop hammer driving pile tube	294.2	27	7.4 [a]	55	3.3 [a]			Horizontal on ground
C33 ♦	1977/78 Kings Lynn (Norfolk)	10.4 m soft clayey silt and peat/5 m stiff kimmeredge clay/hard laminated kimmeredge clay	406 mm ⌀ Driven cased pile, depth N/R	Drop hammer driving pile	36.8	14	0.3					Vertical
C34	1978 South Shields (Tyne and Wear)	Loose to medium sand and silt/soft to firm laminated clay/stiff boulder clay/medium to dense sand and gravel over mudstone at 21 m to 25 m depth	305 mm × 305 mm Steel H-pile, depth N/R	Diesel hammer driving pile	36.3	1.1	9.5					

Table 11 — Summary of case history data on vibration levels measured during driving of bearing piles

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
					kJ	m	mm/s	m	mm/s	m	mm/s	
C35 ♦	1978/9 Hull (Humber side)	N/R	Raking precast concrete piles, dimensions N/R	Drop hammer driving pile	N/R	20	0.51					
C36 ♦	1979 London SE8	N/R	Driven shell piles, dimensions N/R	Drop hammer driving pile	N/R	16.5	2.1	33	1.95	46	0.9	
C37	1980 Caernarvon (Gwynedd)	Fill/gravels and clayey silts/hard glacial till	Driven cast-in-place, dimensions N/R	Driving pile tube	N/R	2.5	18.6	5 to 10	5.5			Distances N/R precisely
C38 ♦	1980 Haxby (N.Yorks)	1.9 m to 3.5 m Clayey sandy fill over soft to firm laminated clay	Driven cast-in-place, depth 4 m to 5.5 m, ϕ N/R	Driving pile tube	N/R	3.8	25.0	5.5	22.0			
C39 ♦	1980 Leatherhead (Surrey)	N/R	Type and dimensions N/R	Driving pile	N/R	50 50	1.25 2.5					Measured on ground floor Measured in middle of 1st floor
C40 ♦	1980 Middlesbrough (Cleveland)	N/R	Driven cast-in-place, dimensions N/R	Driving pile tube	N/R	11	28.9	18	13.8	48	3.1	
C41	1981 Grangemouth (Central)	Soft alluvium	Driven shell piles, 450 mm × 36 m depth	Drop hammer driving pile	29.4	4.5	2.1	9.5	1.2			
C42 ♦	1981 London W6	4 m fill/2 m ballast/London clay	Driven cast-in-place, dimensions N/R	Driving pile tube	N/R	12	6.7					
C43	1981 Winchester (Hants)	4 m to 5 m made ground/gravel/chalk	Bottom driven cased pile 10.5 m depth	Driving pile	N/R	2 to 3	3 to 4					Occasional peaks up to 30 mm/s

Table 12 — Summary of case history data on vibration levels measured during use of vibratory pile drivers

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
					kJ	m	mm/s	m	mm/s	m	mm/s	
C44 ◆	N/R Bridlington (Humberside)	4 m to 5 m soft saturated sand over soft to firm clay	Sheet steel piling, dimensions N/R	Driving or extracting	N/R	6	2.6	8	2.2			27.5 Hz
C45 ◆	N/R Glasgow Cowcaddens (Strathclyde)	3 m fill, blaes, clay and boulders over 8 m soft to firm silty clay over sandstone	150 mm ⌀ casing, depth N/R	Driving casing	2.18 to 3.15	13	1.4*					25 Hz
C46 ◆	N/R New Haw (Surrey)	1 m fill/8 m to 12 m dense fine and medium sand with silty clay lenses (Bagshot), Claygate beds, London clay	Casing dimensions N/R	Driving casing	N/R	7	44	10	23.5	17.5	18.5	25 Hz
				Extracting casing	N/R	7	53	15	27	25	2.9	25 Hz
C47	1968 Drax (N. Yorks)	N/R	N/R	Warming up to drive pile (Resonant pile driver)	N/R	2	10 to 15					70 Hz to 80 Hz
C48 ◆	1968 Hastings (E. Sussex)	4 m clay/8 m peat/2.5 m clay/1 m sandy silt with gravel/6 m stiff clay (Hastings beds)/mudstone and siltstone	N/R	Resonant pile driver	N/R	6	2.5					70 Hz to 80 Hz
C49 ◆	1972 London EC4	Sand and gravel over London clay	N/R	Driving pile	2.18 to 3.15	10	0.55					25 Hz
C50 ◆	1975 Milngavie (Strathclyde)	N/R	casings, dimensions N/R	Driving casing	N/R	5	2.5					27.5 Hz
				Extracting casing	N/R	5	2.0					

Table 12 — Summary of case history data on vibration levels measured during use of vibratory pile drivers

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
					kJ	m	mm/s	m	mm/s	m	mm/s	
C51 ♦	1976 Glasgow (Strathclyde)	N/R	Sheet steel piling, dimensions N/R	Driving pile	N/R	10	11.0					25 Hz
C52	1979 Egham (Surrey)	N/R	Casings, dimensions N/R	Driving casing	N/R	1.6	18.9	3.2	16.3	4.8	11.2	25 Hz
C53 ♦	1979 Molesey (Surrey)	Gravel over London clay	Sheet steel piling, dimensions N/R	Driving sheets	2.18 to 3.15	5	4.3					25 Hz
C54 ♦	1980 London N1	Gravel over London clay	Casings	Driving casing Extracting casing	2.18 to 3.15	40 75	2.0 0.3					25 Hz
C55	1981 Rhondda Valley (Mid Glamorgan)	Glacial till/ gravelly sandy silt mixture with occasional cobbles	Sheet steel piling, Frodingham 3 N 12 m depth	Driving sheets	4.89	10	2.4	20	2.2	40	0.8	Vertical 26.6 Hz
C56 ♦	1979 Bromley (Greater London)	Gravel	Sheet steel piling	Driving sheets	N/R	3	12	9	3.8	25	0.95	Variable frequency up to 23.5 Hz

Table 13 — Summary of miscellaneous case history data on vibration levels measured during piling and kindred operations

Ref. No.	Year and location	Soil conditions	Pile dimensions	Mode	Measured peak particle velocity (p.p.v.) at various plan distances							Remarks
					Theoretical energy per blow	Plan distance	p.p.v.	Plan distance	p.p.v.	Plan distance	p.p.v.	
					kJ	m	mm/s	m	mm/s	m	mm/s	
M1	c1970 London WC2	0.3 m fill/0.8 m clay and gravel/3.6 m dense sand and gravel/stiff London clay including clay stones	Impact bored (tripod) pile dimensions N/R	Driving casing	1.25	2.7	3.1*					Measured at footings adjacent to old listed timber framed building
				Boring gravel	4.25	2.7	1.0*	1.3	0.6			
M2	1971 Bristol (Avon)	Soft clays over sandstone/marl at 10 m to 11 m depth	Driven steel H-piles 305 mm × 305 mm × 12 m depth	Drop hammer driving piles	35.7	1.5	68.1*	3	50.2*	4.6	37.7*	4 t hammer 0.9 m drop.
					35.7	1.5	48.8*	3	39.4*	4.6	20.6*	3 t hammer 1.2 m drop, all ground surface measurements
M3	1971 Stevenage (Herts)	Medium dense sands and gravels	Bottom driven cast-in-place piling	Drop hammer driving pile tube	127	3	116*	6.1	30.3*	9.1	25.1*	Ground surface measurements
M4	1986 Reading (Berks)	5 m granular fill and medium dense sands and gravels over chalk	Open ended casing 610 mm O.D. 10 m depth	Hydraulic vibrator PTC 25H2 (27.5 Hz)	7.08 per cycle	8	5.8	11.5	3.8	16	2.9	On sewer 6.5 m below ground level Ground surface measurements
						8	7.2	11.5	5.6	16	3.0	
M5	1982 Edinburgh (Lothian)	Fill and clay over sands and gravels	Driven precast concrete piles 15 m to 21 m depth	Drop hammer driving piles	26.5 to 44.1	8	23.7	16	7.4	32	2.7	Ground surface measurements
M6	1982 Linlithgow (Lothian)	Softish ground unspecified	Driven precast concrete piles 12 m depth	Drop hammer driving piles	15.5 to 30.9	8	13.4	16	4.4	32	1.5	Ground surface measurements
M7	1982 Ulceby (Humberside)	1.5 m crushed and rolled limestone over cohesive soils over limestone or chalk	Driven precast concrete piles 18 m depth	Drop hammer driving piles	26.5 to 44.1	8	18.6	16	6.6	32	1.3	Ground surface measurements

Appendix D Examples of record sheets

This appendix does not form part of this British Standard.

Investigators of piling vibrations may find the example pro forma record sheets in Figure 4 and Figure 5 helpful in formulating their own site record sheets. Figure 4 and Figure 5 are based on models extensively used by the University of Durham whose permission to publish them in this appendix is duly acknowledged.

Date	Time	Location	Disc	File
Ground conditions				
Ground surface		Subsurface		
Pile				
Type	Size			Length
Hammer				
Weight	Model			Energy
Geophones stand-off distances				
A	B	C	D	E
Additional observations				
File	Depth	Comments		
1				
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				

Figure 4 — Site measurements sheet

Disc no	Date					File name	
Pile							
Type		Sizes				Length	
Tubular		steel, 740 mm diameter and 7 mm thickness				20 m	
Hammer							
Frequency		Model				Energy	
27.5 Hz		Vibrodriver				10.7 kJ/cycle	
Peak particle velocity measurements mm.s^{-1}							
File no.	Depth m	Geophone-set Stand-off	A 2.8 m	B 4.0 m	C 8.0 m	D 10.0 m	E 15.0 m
H		Radial	14.6	6.3	0.73	3.5	1.4
O		Transverse	6.5	16.8	1.1	3.5	1.6
W	7.0	Vertical	12.2	13.1	2.1	3.6	1.5
8		Resultant	16.3	17.4	2.5	3.6	2.3
H		Radial	6.5	9.8	1.7	2.6	1.1
O		Transverse	6.4	14.0	1.3	3.0	2.0
W	9.0	Vertical	9.1	9.0	1.2	2.1	1.4
9		Resultant	11.3	17.4	2.1	3.6	2.3
H		Radial	14.3	9.8	4.0	4.1	0.9
O		Transverse	6.0	13.3	1.5	2.2	1.2
W	11.0	Vertical	10.2	10.9	4.9	5.0	1.9
10		Resultant	15.2	13.9	4.9	5.6	3.1
H		Radial	12.2	11.5	3.1	6.2	2.2
O		Transverse	13.8	18.7	2.6	5.1	1.6
W	12.5	Vertical	12.5	11.1	0.9	5.1	1.5
11		Resultant	18.6	21.9	3.2	7.1	2.5
H		Radial	15.3	11.5	4.5	6.0	1.7
O		Transverse	6.7	18.7	2.7	4.6	1.4
W	13.0	Vertical	15.5	13.2	5.2	3.3	1.6
12		Resultant	17.5	23.2	7.0	6.4	2.2

Figure 5 — Vibration data summary sheet

Appendix E Bibliography

NOTE See also "Publications referred to".

E.1 Publications relating to section 2

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HEAD, J.M. and JARDINE, F.M., *Ground-borne vibrations arising from piling*. CIRIA Technical Note 142:1992. Available from the Construction Industry Research and Information Association, 6 Storey's Gate, Westminster, London SW1P 3AV.

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APPENDIX 14

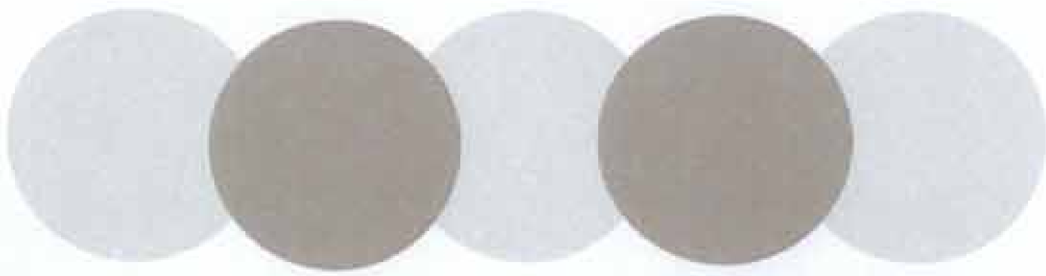
RETAINING WALL CONSTRUCTION SEQUENCE

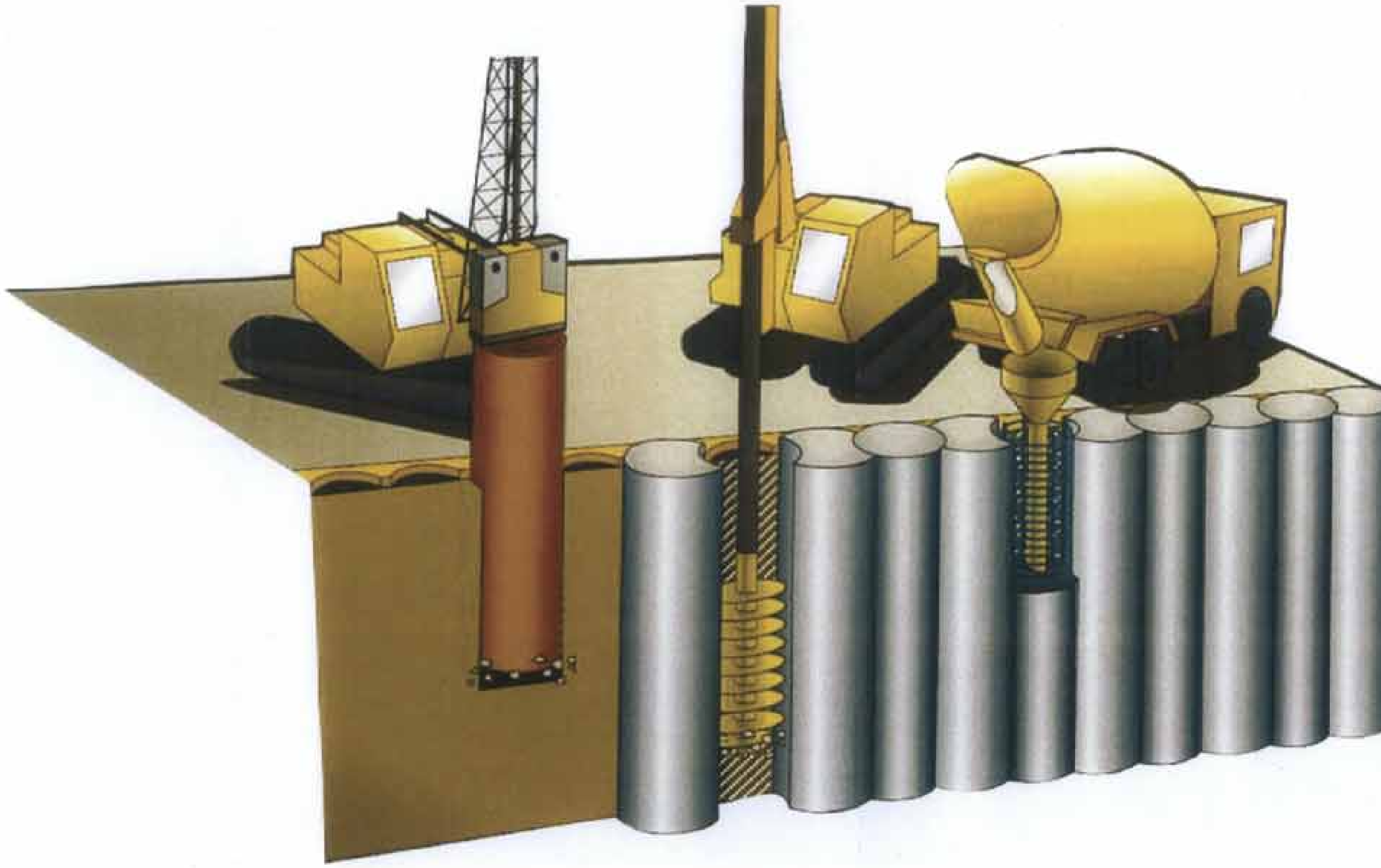
RETAINING WALLS CONSTRUCTION SEQUENCE

1. INSTALLATION OF PRIMARY PILE

2. AUGERING OF SECONDARY BOREHOLE

3. INSTALLATION OF SECONDARY PILE





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Evidence of Tim Chapman
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Retaining Walls Construction Sequence

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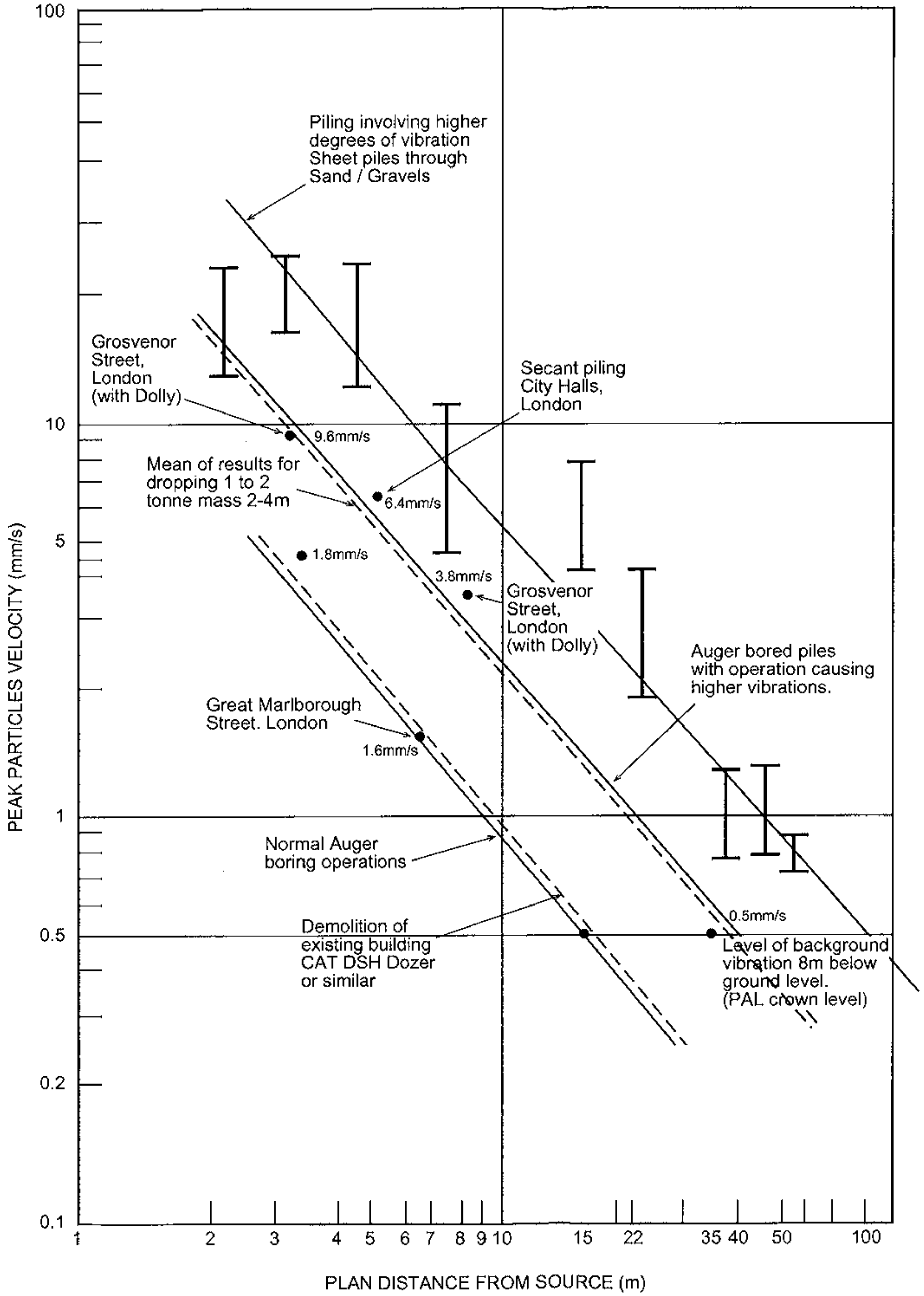
APPENDIX 15

VIBRATION DISTANCES



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Vibration distances

OBJ3/P3/A16

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APPENDIX 16

CIRIA TN 142 GROUNDBOURNE VIBRATIONS ARISING FROM PILING

Ground-borne vibrations arising from piling

CIRIA TECHNICAL NOTE 142

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Technical Note 142 1992

Ground-borne vibrations arising from piling

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Summary

Vibrations, generated by impact and vibratory methods and transmitted through the ground, affect people and, sometimes, property. This Project Report introduces the subject of ground vibrations and how they are propagated. It describes requirements and controls that may be imposed on piling or similar work. The potential effects of different intensities of vibration, as perceived by people nearby and as experienced by structures, are described with reference to available criteria and standards. A number of case records made available to CIRIA are examined in relation to the reported consequences, of annoyance or not to the public and of the effects on structures. Comparisons are made between the case-record vibrations from different piling systems and at different distances, and the results are expressed in ways which will allow engineers to make a preliminary appraisal of the vibrations which might be expected on a projected piling or ground treatment site.

Recommended procedures are summarised for minimising nuisance, for controlling and monitoring the vibrations and for carrying out and reporting studies of ground-borne vibrations from piling. In addition to references cited in the text some additional relevant texts are listed.

J M Head and F M Jardine
Ground-borne vibrations arising from piling
Construction Industry Research and Information Association
TN142, 1992

Keywords:
Piling, piling vibrations, attenuation data, vibration criteria, building damage

Reader Interest:
Of general interest to construction professionals, local authorities, piling specialists

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CLASSIFICATION	
AVAILABILITY	Unrestricted
CONTENT	Subject area review
STATUS	Committee-guided
USER	Construction professionals, local authorities, piling specialists

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Foreword

This report is an introduction to the nature and effects of ground-borne vibrations caused by piling and similar works. It contains guidance about the nature of these vibrations and about how they are propagated. References relevant to the assessment of the risks of annoyance and damage are given as are recommended procedures for making these assessments.

The report is the outcome of CIRIA Research Project RP299. The first stages of this project were the collection and analysis of case histories of projects involving vibration measurements associated with bearing and sheet piles and a limited number of ground improvement works. The case records were collected in the early 1980s and relate largely to UK projects completed between 1970 and 1980.

In the time taken to finalise this report, the case histories have dated. It is likely that many, more thorough, studies have since been completed. Nevertheless, few are in the public domain and it is probable that, as with the records collected by CIRIA, many may be incomplete or inadequate. In the absence of a British or international standard giving acceptable vibration levels relating to damage, it is hoped that this report will provide helpful information for engineers faced with assessing the effects of piling and similar works.

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Notation

a	=	particle acceleration (mm/s^2)
a	=	empirical attenuation factor
A	=	displacement amplitude (mm)
A	=	cross-sectional area of pile
b	=	empirical attenuation factor
E	=	nominal energy delivered to pile per blow (kJ/blow)
f	=	frequency (Hz)
G	=	modulus of shear deformation
I	=	impedance of pile
k	=	empirical attenuation factor
K	=	Dieckmann perception intensity parameter
KB	=	modified Dieckmann perception intensity parameter
ppv	=	peak particle velocity (mm/s)
r	=	surface (plan) distance from pile (m)
t	=	time (s)
v	=	particle velocity (mm/s)
v_i	=	transient maximum single-component particle velocity (mm/s)
v_m	=	longitudinal wave velocity in pile material
$v_{max(t)}$	=	true instantaneous peak particle velocity
v_p	=	velocity of (compressive) P-wave
v_s	=	velocity of (shear) S-wave
$v_{(x,\ y\ or\ z)}$	=	particle velocity in direction x, y or z
W	=	power (of vibro-driver) (kVA = kJ/s)
x	=	empirical attenuation factor
z	=	displacement (mm)
γ	=	unit weight
ν	=	Poisson's ratio
ρ	=	density
ϕ	=	phase angle
ω	=	circular frequency = $2\pi f$ (rad/s)

1 Introduction

Many construction activities, including piling, generate both air-borne and ground-borne vibrations. Regulating authorities are now empowered to set limits to control noise and vibration: designers and constructors are thereby required to restrict these side effects of the construction works.

Ground-borne vibrations from piling can be disturbing to neighbouring people and may cause damage to nearby property. At present there are no explicit criteria against which these effects can be measured nor maximum levels and tolerances unequivocally defined. There are, however, subjective criteria based on human perception of vibration, and 'rule of thumb' guidelines for vibration levels which might cause property damage. Because these criteria are not specific to piling vibrations, they are limited in their applicability. It is difficult, therefore, for regulating authorities to set practical limits for the control of the effects of piling operations or other similar works such as ground improvement processes. Equally, designers and constructors need guidance to be able to meet their responsibilities to the community.

This report introduces these issues in order to give preliminary guidance on the effects of ground-borne vibrations arising from piling and similar works. The report results from two separate exercises. First, a survey was made of British practice, collating and interpreting over 150 case studies of piling and similar activities. Secondly, a desk study assessment was made of available knowledge of perception and damage criteria and of various international proposals relating to these criteria.

The available criteria were then compared with the empirical results. From the advice of specialists and engineers experienced in these aspects of piling and ground improvement, a practical approach is put forward which should help to mitigate complaints and damage (and unjustified claims of damage).

The generation and transmission of vibrations are explained in terms of types of plant, pile construction, ground conditions and structural response. Levels of vibration propagation are related to their consequence to people and property. The available criteria and case history records are compared. A procedure for monitoring operations and mitigating risk is proposed with guidance on specific problems. The recommended procedure draws particular attention to the need for good public relations.

The generation, transmission and effects of piling vibrations are complex with many variables and related uncertainties. Although vibration magnitudes can be measured accurately, prediction of their effects is uncertain. Some of the reasons for this are shown in Figure 1. Each piling operation is a unique combination of process and ground conditions. Each site (and its surroundings) is unique; and each of the nearby structures has its own special characteristics.

Broadly based guidelines, essentially in terms of the probabilities of risk, are the current approach to prediction, but expert judgement is needed for specific assessments. The intention of this Report is to provide an introduction to the subject so that engineers may recognise situations when experts should be brought in.

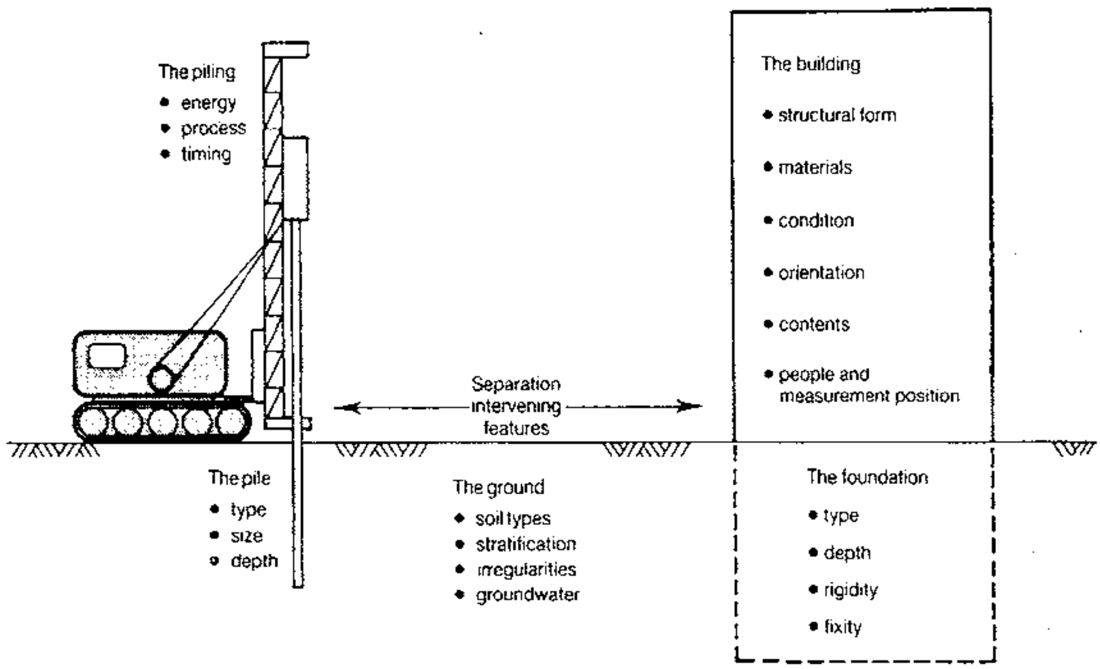


Figure 1 *Variables of ground-borne vibrations from piling*

2 Propagation of piling vibrations

How people and property are affected by piling-induced vibrations largely depends on how the vibrations are transmitted through the ground. This Section outlines the nature and propagation of ground-borne vibrations and of how they are generated by piling.

The vibration characteristics, and therefore their effects, depend on the form of piling, the type and geometry of the ground conditions, the distance to the affected property, the structural form and condition of buildings, and the type of foundation (Figure 1). An appreciation of vibration characteristics and propagation will help in identifying the potential risks of damage to adjacent structures and property and of physical discomfort and annoyance to local people. The explanation of the propagation of piling vibrations which follows is given in terms of classical wave mechanics.

2.1 VIBRATION OF ELEMENTARY SYSTEMS

The motion of a discrete element of a vibrating material (which may be a mass of soil or part of a structure) has its simplest form when it is sinusoidal (harmonic). Vibratory pile drivers can often generate practically steady-state sinusoidal vibration in the ground. More often, with other types of piling, the vibrations are transient, periodic or random. The diagrams of Figure 2 show examples of the displacement – time relations of these four types of vibration. Note that the displacement is the movement of one element or particle of the mass that is vibrating. Formal definitions of vibration can be found in ISO 2041 – Vibration and Shock – Vocabulary.

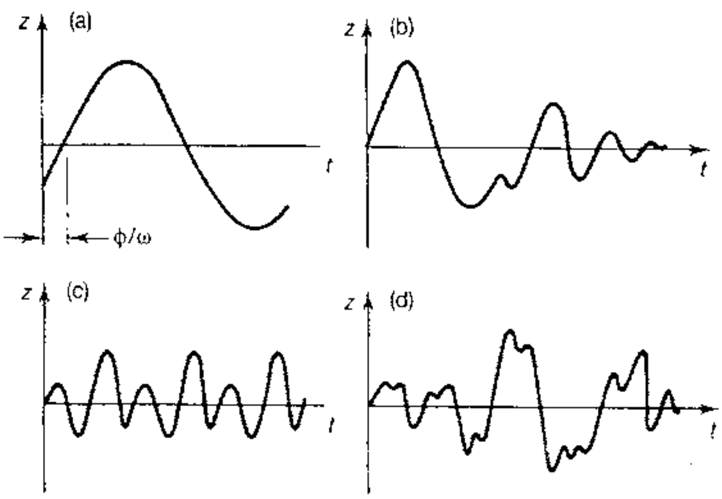


Figure 2 *Types of vibration: (a) sinusoidal (b) transient (c) periodic (d) random*

For sinusoidal motion, the displacement-time relation is:

$$z = A\sin(\omega t - \phi)$$

$$\begin{aligned} \textit{where } z &= \textit{displacement (mm)} \\ A &= \textit{displacement amplitude (mm)} \\ \omega &= \textit{circular frequency} = 2\pi f \textit{ (rads/s)} \\ f &= \textit{frequency (Hz)} \\ t &= \textit{time (s)} \\ \phi &= \textit{phase angle (rads)} \end{aligned} \qquad \text{... (2.1)}$$

Differentiation of the displacement equation gives the maximum velocity of the particle and, on further differentiation, its maximum acceleration i.e.

$$v = \frac{dz}{dt} = 2\pi Af \qquad \text{... (2.2)}$$

$$a = \frac{d^2z}{dt^2} = 4\pi^2 Af \qquad \text{... (2.3)}$$

Similarly, for other types of vibration the quantities (displacement, velocity and acceleration of the particle) can be derived by differentiation or integration, provided that the relation of one of these with time is known. Section 7.6 summarises the ways these quantities are measured.

2.2 VIBRATION PROPAGATION

As individual particles vibrate they transmit the vibration to adjacent ones. In an infinite elastic medium, the vibration motions are transmitted through the body of the material as dilational and distortional waves. The dilational wave is referred to as the *primary*, compressive wave or ***P***-wave. Its velocity through the medium, i.e. the wave propagation velocity, is v_p. The secondary wave is the distortional *shear* wave or *S*-wave: its propagation velocity is v_s. These wave propagation velocities are related to the elastic properties of the material through which they pass by:

$$v_p = \frac{\sqrt{\lambda + 2G}}{\rho}$$

$$\textit{and } v_s = \sqrt{\frac{G}{\rho}} \qquad \text{... (2.4)}$$

$$\begin{aligned} \textit{where } G &= \textit{modulus of shear deformation} \\ \rho &= \textit{density} \\ \lambda &= \frac{2\nu G}{(1 - 2\nu)} \\ \textit{and } \nu &= \textit{Poisson's ratio} \end{aligned}$$

The units of v_p and v_s are usually given in m/s. Typical examples of soil and rock physical properties and values of the wave propagation velocities are given in Table 1.

Table 1 Typical properties of some soils and rocks

Material		In-situ velocities (low strain) (m/s) v_p Saturated (not saturated)	v_s	Poisson's ratio ν Saturated (not saturated)	Density ρ	Mass shear modulus G (MN/m^2)
Sand (near surface)	Loose	1450-1550 (185-450)	100-250	0.48-0.50 (0.3-0.35)	1.5-1.8	15-110
	Medium	1500-1750 (325-650)	200-350	0.47-0.49 (0.2-0.3)	1.7-2.1	70-250
	Dense	1700-2000 (550-1300)	350-700	0.45-0.48 (0.15-0.3)	1.9-2.2	230-1000
Clay	Soft	1450-1550	80-180	0.47-0.5	1.6-2.0	10-65
	Firm	1500-1700	180-300	0.47-0.5	1.7-2.1	55-190
	Stiff	1600-1900	300-500	0.47-0.5	1.8-2.3	160-450
Unweathered		1400-4000	800-2000	0.25-0.35	2.0-2.4	1300-9500
Sandstone and shale Limestone (not chalk)		2100-6000	1200-3000	0.25-0.35	1.8-2.5	2600-20000
Unweathered Igneous or metamorphic rock		3500-7000	2000-3500	0.25-0.35	2.2-2.6	8500-32000

Compressive waves induce longitudinal particle motions which travel on a hemispherical wave front, as shown on Figure 3. The relative displacement amplitude associated with these particle motions decreases towards the ground surface. The proportion of total energy propagated in this way is very low (less than 10%) and rapidly decays with distance.

Propagated shear waves induce particle motions in a transverse direction on a hemispherical wave front. Relative component displacement amplitudes in vertical and horizontal directions increase towards the ground surface. Their propagation velocity is lower than that of primary waves (see Figure 3); the proportion of total energy transmitted is higher (about a quarter), the energy decaying more rapidly with distance.

The energy density of both compressive and shear waves attenuates rapidly with distance from a localised source such as piling, primarily because of the volumetric increase in material encountered as the hemispherical wave front expands (see Figure 4). This decrease in energy per unit volume is called *geometrical damping.* A secondary decrease in energy with distance is the *material damping*, i.e. it depends on the material through which the waves travel.

In an infinite elastic half-space, there is a third wave which is confined to a zone near its boundary (ground level). The surface wave propagation motion produced is commonly known as the *Rayleigh wave*. Because propagation is planar, rather than hemispherical, the decay of energy is much slower; the proportion of total energy transmitted in this way is about two-thirds. However, this wave has the slowest propagation velocity and its influence decreases rapidly with depth. Induced particle motions from piling operations are predominantly vertical in the immediate vicinity of the source, but rapidly generate a horizontal component with distance.

Although there is a relation between the Rayleigh (surface) wave velocity and the shear wave velocity, in practice the latter is used: it is usually within 10% of the Rayleigh wave velocity. Furthermore, it is not clear whether true Rayleigh waves can be discretely developed within the short distances relevant to civil engineering works. It is suspected that a similar but partly developed wave is generated at the surface which again approximates to the shear wave. A summary of the characteristics of the various waves is given in Table 2.

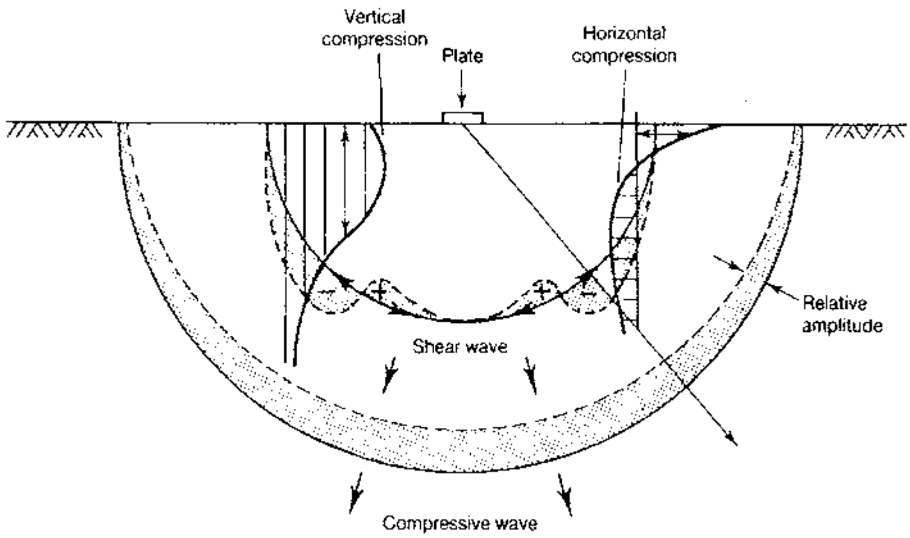


Figure 3 *Distribution of displacement waves from dynamic loading of a surface plate on a homogeneous, isotropic, elastic half-space*

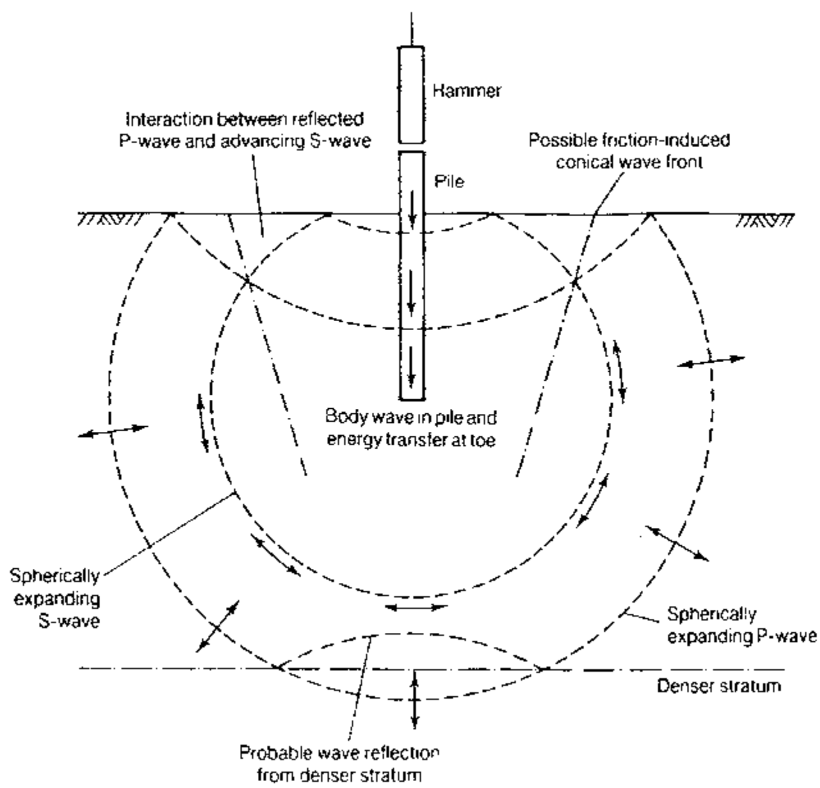


Figure 4 *Wave propagation near to a driven pile*

Table 2 Summary of characteristics of displacement propagation waves arising from a loading on a half-space

Compressive wave	**Shear wave**	**Rayleigh wave**
Highest propagation velocity	Intermediate propagation velocity	Lowest propagation velocity
Longitudinal oscillation	Transverse oscillation	Vertical oscillation, but rapidly develops horizontal component with distance
Propagation velocity increased below groundwater level	Propagation velocity decreased by groundwater	Propagation velocity unaffected by groundwater but generally lower in moist soil
Propagation velocity increases with material stiffness	Propagation velocity increases with material stiffness	Propagation velocity increases with material stiffness and is independent of frequency in homogeneous material
Energy proportion propagated is low	Energy proportion propagated is intermediate	Energy proportion propagated is high
Displacement amplitude proportional to $\frac{1}{\text{distance}}$	Displacement amplitude proportional to $\frac{1}{\text{distance}}$ except along the ground surface when amplitude proportional to $\frac{1}{(\text{distance})^2}$	Displacement amplitude proportional to $\frac{1}{(\text{distance})^{1/2}}$

2.3 EFFECT OF MULTIPLE LAYERS

In layered ground, some of the energy is refracted through the base of the first layer to the lower material, and some is reflected. The velocities of the reflected or refracted propagation waves can be greater than that of the incident wave. The amplitude and direction of the resultant propagation waves depend on:

1. The angle of incidence at the boundary.
2. The ratio of densities, hence velocities, of each material.

Figure 5 shows the action of multiple wave reflection and refraction in a layered half-space. Multiple total reflections can generate a subsidiary surface wave propagation known as the *Love* wave. Also, multiple layers and the surface can convert one wave type to one or several other wave types; thus a surface wave is generated when a primary wave is reflected from the surface.

2.4 GENERATION OF VIBRATIONS DURING PILE DRIVING

As a pile penetrates the ground, the driving hammer generates a body wave within the pile which travels down the shaft to its base in contact with the soil. A proportion of the wave energy is reflected within the pile but most is transmitted to the soil. Unless the pile is predominantly frictional, tapered or stepped, the proportion of energy transmitted from the shaft to the soil alongside is low. Body waves through the soil are generated at the pile base as shown on Figure 4 and at a critical distance, roughly equivalent to the pile depth, the shear wave reaches ground surface. The surface transmission of the shear wave is termed a *head wave*. Reflections at ground surface, possibly interacting with low energy wavefronts associated with overcoming pile and shaft friction, produce a complex array of particle motions. Any adjacent structures will experience incidental reflected compressive propagation waves.

The amount of ground energy generated depends on the characteristics of the hammer or vibrator, the deformation of any helmet packing, the elastic distortion of the pile, and the rate of penetration of the pile or casing. The proportion of impact energy transmitted to the ground will also depend on the proportion directed towards advancing the pile. In a soft clay most of the energy directly advances the pile whereas in stiff clays, dense gravels and weak rocks, a proportion is transmitted to the ground. At refusal, all the energy is used in vibrating the soil.

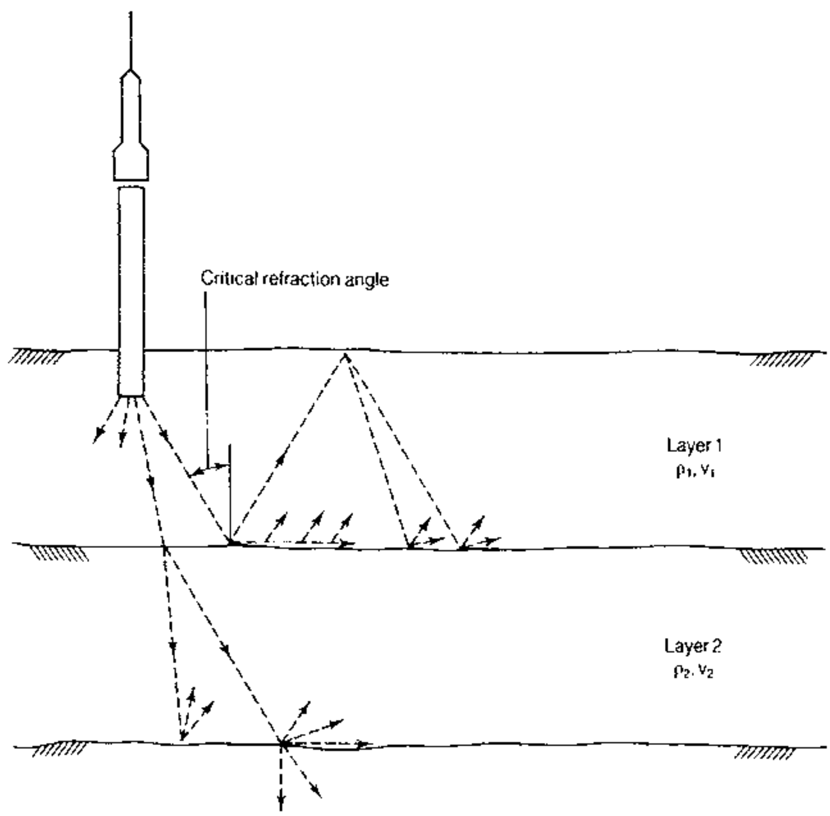


Figure 5 *Action of multiple wave reflection and refraction in a layered half-space*

Where the energy produced by impact piling effectively dies away before the next blow then the vibration is termed transient or intermittent (note that the term 'intermittent', although used, for example, in BS 6472 does not have a formal definition). A large proportion of driven piles fall within this category. Periodic and continuous vibrations are produced when piling with a vibratory driver and by vibro-compaction operations. Essentially this mode of vibration produces a forced particle motion through continuous sinusoidal wave propagation.

Where two sinusoidal motions at slightly different frequencies are superimposed, a non-sinusoidal motion is created with amplified characteristics. This phenomenon can be experienced on site when two or more piling rigs are operating at the same time.

Random vibrations are caused by traffic, site works, trains and other extraneous sources. These vibrations can combine with piling vibrations to provide an amplified effect at particular frequencies.

2.5 TYPES OF PILE VIBRATIONS

2.5.1 Transient vibration

Transient vibrations are produced by rapid-impact driven piling (e.g. precast piles, sheet piles or cast-in-place piles which are top or bottom driven). The displacement amplitude varies with weight of the drop hammer, drop height, penetration per blow, distance from pile, and soil type. Single-acting pile hammers, for example, produce vibrations which may be regarded as transient.

2.5.2 Continuous vibration

Vibratory driven piles produce a steady-state vibration when vibro-drivers, vibroflots or resonant drivers are employed. The ground particles are forced to vibrate in a predetermined mode, irrespective of any preferred frequency which the ground may have. The forced vibration may be made up of several component frequencies, but the predominant frequency is always that of the driver itself. A substantial increase in ground vibration always occurs when the frequency coincides with the characteristic pile/ground frequency.

A less regular but continuous form of vibration can also result from rapid impact piling where the vibrations do not fully die away between blows; for example, with some double-acting air or diesel hammers whose stroke rates are considerably more rapid than conventional gravitational-fall hammers. Augered piling tends to generate essentially continuous vibrations, but at low energy levels.

The distinction as to whether the vibrations from piling and similar works should be classified as intermittent or continuous is not necessarily clear cut; often it will be a matter of judgement.

2.5.3 Occasional vibration

Bored and augered piling can cause occasional vibrations from the driving of casing or chiselling through obstructions.

2.6 ATTENUATION OF GROUND VIBRATION

The rate at which vibrations pass through the ground as waves of energy is termed the *wave propagation velocity* (Section 2.2). The excitation of individual ground particles, caused by the energy waves, is an oscillatory motion (Section 2.1), which is described by the varying acceleration, velocity and amplitude of the displacement of the particle itself. *Particle velocity* is thus completely different from wave propagation velocity.

A soil particle experiences a combination of effects from the different energy waves, producing a particle motion which rapidly reaches a peak value before reducing as the waves pass. The techniques of wave propagation mechanics enable the superimposition of the different waves to be filtered, separated and analysed.

Attenuation of the generated wave motion, and therefore the peak particle motion, takes place through the geometric enlargement of the various wave fronts as they move away from the source and by material damping. Attenuation with distance from a source should not be confused with decay with time at a point.

The particle-motion parameters of displacement amplitude, velocity and acceleration are identified and measured by their components in the orthogonal directions x, y and z. Particle velocity can therefore be resolved vectorially as shown overleaf:

$$v = \sqrt{v_x^2 + v_y^2 + v_z^2} \qquad \text{... (2.5)}$$

where components, v_x, v_y and v_z, are those at the same instant of time. It is possible to identify maximum values of the resultant, v, and of the component in appropriate directions.

A convenient indicator for the characterisation of piling vibrations is the *simulated resultant peak particle velocity*. This is the vector sum of the peak velocities in the three mutually perpendicular directions, irrespective of the time at which these three peak values occurred. In other words, the simulated resultant treats the three maximum components as though they occurred at the same instant. The simulated resultant is sometimes referred to as *SRSS* (square root, sum of squares): it is a robust parameter for use in developing attenuation – distance relations, but for the purpose of establishing criteria on vibration levels, it is tending to be displaced by the vectorially-resolved maximum and components in appropriate directions (as in DIN 4150, 1986).

This report, unless otherwise stated, uses the simulated result when referring to peak particle velocity (*ppv*). Where reference is made to other texts, the definition of *ppv* used in them is specifically identified.

There is a consensus that measurement of particle velocity provides the best single parameter for assessing human and structural responses to piling vibrations. When considering human perception of vibration on its own, national and international standards tend to use root-mean-square acceleration as the most useful indicator. Particle velocity has been used in this Report, however, because of its direct application to both human and structural response.

A number of empirical relations have been developed to express attenuation in terms of *ppv* and distance. Theoretically, compression and shear waves attenuate at a rate inversely proportional to the distance from source, whereas surface and Love propagation waves attenuate at a rate inversely proportional to the square root of surface distance (in some cases attenuation is directly proportional to distance). However, surface and, less commonly, Love waves, are more likely to dominate particle motion at large distances from piling operations. At closer distances, compressive and shear propagation waves attenuate through complex processes which are frequency dependent: higher frequencies being attenuated more than lower. Buried objects, such as adjacent foundations, are likely to modify the characteristics and form of the vibration.

It is important to recognise that as the zone of interest becomes nearer to the piling operations, the ground response to the vibration and the effect of (or on) a structure in this zone are not well understood, nor are methods of prediction well developed. The extent of this near field is uncertain and depends on the type and size of the piling operations and on the influence of the nearby structure. Typically, but not invariably, this zone will be of the order of metres. This Report does not address the problem of structures in this zone and the general guidance given in the Report may not apply to these circumstances.

Wave propagation damping is affected by the characteristics of the soil or rock and by the amplitude of motions (the cyclic strain magnitude). The magnitude of ground motion (expressed in terms of particle velocity, acceleration or displacement) at a given distance from a vibration source principally depends on the magnitude of the source and the attenuation characteristics of the ground.

Because so many variables are involved, no explicit relations exist which allow accurate predictions of magnitude to be made for any given source and ground conditions. Approximate empirical relations have been developed based on limited case studies, and these concentrate on simulated resultant *ppv*, v, at ground surface. The dominant wave component being measured is the surface wave.

For a surface wave generalised relations between resultant *ppv* and distance have been proposed by Wiss (1967):

$$\textit{for clays: } v \propto \sqrt{E}/r^{1.5}$$
$$\textit{for silts and sands: } v \propto \sqrt{E}/r \qquad \text{... (2.6a and b)}$$

where E = pile energy input/blow (kJ)
r = surface distance (m)
v = peak particle velocity (mm/s)

Other authors have proposed similar empirical expressions (Attewell and Farmer, 1973; Morris 1950). The general form of these is:

$$v = k\sqrt{E}/r^{x} \qquad \text{... (2.7)}$$

where the empirical factors, k and x, have a wide range of possible values:
k varies from 0.1 to 1.5 for E in Joules depending on ground profile, and
x varies from 0.8 to 1.5 depending on soil characteristics.

The following conservative relation has been put forward by Attewell and Farmer (1973):

$$v = 1.5\sqrt{E}/r \qquad \text{... (2.8)}$$

This can be used for preliminary evaluation of an upper-bound resultant *ppv*, but it is more satisfactory to develop site specific correlations based on the general form of Equations 2.7 or 2.9. It is usual to measure at points on the ground surface identified by their plan (horizontal) distance from the source.

Empirical relations should only be applied to situations similar to those for which they were developed and within a similar range of distances. In particular, they may be unreliable at closer distances to the vibration source or where a structure intervenes.

Although individual soils and rocks do not possess a natural frequency as such, there are characteristic frequencies at which they transmit vibration more readily. Typical general ranges of characteristic frequencies noted during piling operations are:

Very soft silts and clays	5 to 20 Hz
Soft clays and loose sands	10 to 25 Hz
Compact sands and gravels and stiff clays	15 to 40 Hz
Weak rocks	30 to 80 Hz
Strong rocks	> 50 Hz

In practice, actual frequency characteristics depend on a combination of factors.

Equations 2.7 and 2.8 may be used to estimate *ppv* at ground level at selected distances from piling operations, but their value is limited to this two-dimensional assessment. In order to identify the more important and critical aspects for a specific site, the vibration study should take the three-dimensional effects on the propagation and attenuation of the vibrations into account.

Recent, further work at the University of Durham has included the analysis of 130 studies of vibrations caused by piling. This work is reported by Attewell *et al.* (1990), Attewell *et al.* (1991), and Oliver and Selby (1991). By setting up the records in an appropriate database, they have designed a knowledge-based system which may be interrogated in order to make broad predictions of vibration levels for a new site with different choices of pile and hammer.

Their work again confirms the applicability of relations of the general form:

$$v = b\left(\frac{\sqrt{E}}{r}\right)^x \qquad \text{... (2.9)}$$

In particular, however, they point out the following:

1. This type of relation should only be used for plan distances greater than about 10m.

2. Values of *ppv* at closer distances are often lower such that a cubical polynomial curve-fitting is more appropriate.

3. For impact hammers, they suggest vibration velocity estimation can be based on the following parameters:
   $b = 1.33$ and $x = 0.73$.
   For vibrodrivers, the parameters for estimation purposes are:
   $b = 1.18$ and $x = 0.98$.

3 Requirements and controls

Piling vibrations can affect nearby structures and inconvenience the occupants or users. Both aspects are important, either separately or together, because the level of disturbance might not only be a nuisance, but it could also damage buildings and services. Restrictions, if imposed, might involve suspending, modifying or even terminating the piling, thus affecting the progress of the construction project.

It is therefore important from all points of view that there should be early recognition of the potential for annoyance, nuisance or for damage. An investigative study at a sufficiently early stage will usually lead to steps which minimise risks without untoward consequences for the works.

For control purposes, vibration has been grouped with noise, and guidance or stipulative documents often do not differentiate between these two quite different phenomena.

Vibrations are generated, *inter alia*, by traffic, machinery, explosions and construction activities. Control of their level and effects is achieved through statutory and other requirements such as construction contract conditions. Piling vibrations are a special form of vibration, being both temporary and intermittent by nature. Vibrations in general, and piling vibrations in particular, are neither explicitly controlled by legislation, nor is their control always defined in construction contracts, (although it may be implied by indirect requirements such as the avoidance of 'inconvenience' or 'disturbance'). Because many assessments of nuisance or disturbance are subjective, it is difficult to establish comparative precedents to set acceptable levels.

3.1 LEGAL BACKGROUND

Legislative control of the nuisance element of vibrations used to lie within the provisions of the Noise Abatement Act 1960 (now repealed). This particular aspect of nuisance control, including vibration, is now covered under the Environmental Protection Act 1990 and also the Control of Pollution Act 1974. In certain severe cases, contraventions of the Health and Safety at Work Act 1974 may occur, particularly if disruption of mechanical or electrical systems result from these vibrations, which in turn present difficulties to operators of that equipment.

Restrictions can be imposed by local authorities at any time on methods employed, hours of working and consequent effects (see Section 3.2). Maximum vibration levels can be specified. If recorded or assessed levels of actual operations are deemed to be sufficiently disruptive to the community, or damaging to the property, then the vibrations may be designated a Public Nuisance.

Legal action can also be taken by individuals, under common law, to restrict nuisance or to seek compensation. Such cases are dealt with under the classification of Private Nuisance.

The legal position is complicated and the application of Common Law cannot be regarded as settled. The Final Report of the Wilson Committee on the Problem of Noise (HMSO, 1963) comments as follows:

> Noise can be the subject of a civil action for nuisance at Common Law. Nuisance has been defined as the wrong done to a man by unlawfully disturbing him in the enjoyment of his property. The disturbance may take the form of injury to property or interference with personal comfort. The wrong does not involve a direct physical interference like trespass; in relation to personal comfort it must not merely cause transitory disturbance but also substantially interfere with health, comfort or convenience. The following quotation from the judgement of Mr Justice Luxmoore in the case of Vanderpant v. the Mayfair Hotel Company Limited (1930) expands this definition with particular reference to nuisance:

'Apart from any right which may have been acquired against him by contract, grant or prescription, every person is entitled as against his neighbour to the comfortable and healthful enjoyment of the premises occupied by him, and in deciding whether, in any particular case, his right has been interfered with and a nuisance thereby caused, it is necessary to determine whether the act complained of is an inconvenience materially interfering with the ordinary physical comfort of human existence, not merely according to elegant or dainty modes and habits of living but according in plain and sober and simple notions obtaining among English people'.

In England and Wales the remedy sought in an action for nuisance at Common Law is an injunction restraining the defendant from continuing the nuisance. Sometimes damages are sought in addition. If proceedings are brought in a County Court it is necessary for the plaintiff to claim damages as well as an injunction since otherwise the court has no jurisdiction. In Scotland the corresponding remedy is an action for interdict or for interdict and damages: such actions may be brought in the Sherriff Court or in the Court of Session and it is not necessary in either case to claim damages as well as interdict. There have been many decisions by the courts granting injunctions or interdicts restraining various types of noise including the ringing of church bells, singing, holding noisy entertainments and bringing together disorderly crowds, using a steam organ in connection with a merry-go-round and making an excessive noise in carrying on a trade.

3.1.1 Public nuisance

Lord Justice Denning (1957) defined public nuisance as:

Nuisance which is so widespread in its range or so indiscriminate in its effect that it would not be reasonable to expect one person to take proceedings on his own responsibility to put a stop to it, but that it should be taken on the responsibility of the community at large.

A public nuisance can be a crime, a misdemeanour at common law and as such the subject of indictment. Criminal prosecution for public nuisance can be commenced by the Attorney General or, for example, by the Environmental Health Department of the local authority. Civil proceedings can be commenced by the Attorney General either alone or at the instigation of the Environmental Health Department of the local authority or a private individual. A private individual cannot commence civil proceedings unless it can be shown that the nuisance is the cause of special damage to that person, over and above that sustained by the public at large.

This branch of the law is rarely invoked and is largely superseded in practice by statutes such as the Control of Pollution Act, 1974.

3.1.2 Private nuisance

According to Kerse (1975), in private nuisance 'damage traditionally consists of actual physical damage to the plaintiff's property or some unreasonable interference with the plaintiff's use and enjoyment of his property. The cases involving noise and vibrations from pile driving are a good example of both these forms of damage; actual physical damage to the property caused by the vibrations together with loss of sleep, interference with communication ...'.

An isolated incident will generally not amount to a private nuisance unless it is associated with sudden and serious damage or inconvenience. The character of the locality is also important.

An important distinction of private nuisance is that an action is not necessarily based on compliance with conditions restricting noise or vibration detailed in a planning permission, or stipulations of any noise or vibration levels under the provisions of the Control of Pollution Act, 1974. However, it has been made clear that the law of nuisance is not to be used to prevent building works being carried out (Andrea v. Selfridge, 1938, Ch 1). The Court of Appeal head note states that 'no cause of action arises in respect of operations such as demolition and building, if they are reasonably carried on and all reasonable and proper steps taken to ensure that no undue inconvenience is caused to neighbours'.

An injunction and damages are the principal remedies for private nuisance, although, in practice an injunction is usually the only remedy sought. An injunction will generally not be given where damages would be an adequate remedy. It is a standard condition of granting an injunction that the plaintiff gives a cross-indemnity in the event that it is held the injunction should not have been granted. In most cases an application is made for an interlocutory injunction before the matter has been tried. The plaintiff also takes the risk of having to pay if it is decided that an injunction was wrongly obtained.

Another case of importance is Hoare v. MacAlpine (1923) 1 Ch. 167. This case concerned an old building in London which had to be demolished as a result of damage from vibrations. There appeared to be a strong argument that the defendants were not liable because of the frail state of the building. On page 174 of that record, authority is cited for the proposition that, in nuisance, persons are not to be allowed to enforce rights which limit the uses by others of property unless the facts relied on as constituting a nuisance are such as to interfere with the ordinary rights which according to the ordinary notions of mankind they are entitled to exercise in relation to one another and in relation to their property (Kine v. Jolly (1905) 1 Ch. 480, 489). The Court accordingly adopted the view that the defendant was under absolute liability for the escape of vibrations following Rylands v. Fletcher LR. 3HL 330. This was a leading nineteenth century case which established a species of absolute liability where a defendant gathered upon his land some unnatural and potentially dangerous material of force. In Rylands v. Fletcher this was a reservoir, which was held to put the defendant under absolute liability should water escape from it and damage neighbouring land. Hoare v. MacAlpine is therefore some authority that the same principle may be applied to vibrations. But the authority of the case has been doubted. The judgement concludes by seeking to justify the imposition of absolute liability on the basis this was the only way that owners of old buildings in London could be protected. Since then, however, the London Building Acts have been passed.

3.2 POWERS OF LOCAL AUTHORITIES

Part 3 of the Control of Pollution Act embodies the statutory provisions which control emissions of noise and vibration from construction sites. The Act covers noise extensively and defines noise as including vibration (Section 73). This statement confirms full application of the provisions of the Act. The Northern Ireland Local Government Order (1978 SI 1049, SI 19) defines 'noise' as including 'vibration' and contains provisions for the abatement of such nuisances.

Much of the complaint of residents often stems from the temporary and intermittent nature of piling works – they represent a change in the usual conditions in the neighbourhood and, to outsiders, the operations appear to be in bursts of intense activity and noise with irregular pauses between. Because of their statutory responsibilities and because they have to handle complaints, the local authorities will require prior notification of works of this kind which are likely to give rise to problems for residents. Prior notification is not mandatory, but failure to notify the relevant bodies may result in action which can bring into operation the statutory powers invested in these bodies.

Section 60 – Control of Noise and Vibration on Construction Sites. Section 60 of the Act gives local authorities powers to enforce their requirements for the control of noise and vibration. The Section applies to construction works, including pile driving, and enables a local authority to stipulate and impose their requirements prior to and during operations. Where it appears to a local authority that works of pile driving or other forms of construction are being, or are likely to be, carried out within the district, the local authority may serve on a contractor a statutory notice, listing the requirements. These can include methods of working, types of machines and hours of working. Maximum levels of vibration and a time for compliance can also be specified. As with other sections of this Act, the recipient of this notice has a right to appeal to the magistrates' court.

Section 61 – Prior Consent for Works on Construction Sites. Insofar as the notification of a particular piling scheme is concerned Section 61 lays down a prior consent procedure.

Any person intending to carry out construction work, such as piling, which could be subject to noise and vibration regulations may anticipate the requirements of the local authority and submit to them a request for Prior Consent. The submission must contain details of the work to be done, the method by which it is to be carried out and the steps which will be taken to control noise and vibration emitted. The application for prior consent can be presented at the same time as for approvals under other enactments but not to precede application for Building Regulations Approval. The local authority may, if it is satisfied with the proposals, give consent for the work to proceed. Alternatively, it may grant a qualified consent, stating additional conditions with which the contractor must comply. The authority must inform the applicant of its decision with 28 days of the request for Prior Consent.

This procedure enables local circumstances to be discussed between the local authority and the contractor, and methods of operation can be agreed at that stage. Any such consent agreement between the local authority and the contractor, while it may be grounds for defence against proceedings under Section 61, does not constitute any ground of defence against any proceedings instituted under Section 59 of this Act (see below).

In its deliberations, the local authority will bear in mind the provisions of any codes of practice which are issued in respect of the part of the Act and that the 'best practicable means' are used in execution of the work. 'Best practicable means' is an important phrase as it indicates that a local authority recognises that certain site operations are, by their very nature, noisy and cause vibrations, and consequently, with currently available equipment, not capable of achieving significantly reduced noise and vibration levels.

Irrespective of whatever limits are set for the execution of the work, the Contractor may carry out emergency work by other means, where there is a temporary danger to life or property.

The basic aim of this part of the Act is to preserve the quality of life, as reasonably as possible, for those people who live or work in an area where civil engineering work is in progress.

Any restriction under the Act is normally related to the usual circumstances in any particular locality and to a corresponding period of time, e.g. day or night, weekday or weekend. The local authority which intends to set maximum noise and vibration levels should therefore ensure that it knows what normal ambient noise and vibration levels exist before the work commences.

3.3 PROCEEDINGS AGAINST NUISANCE

Summary proceedings by local authorities. Notwithstanding the specific use of powers under Section 60 of the Control of Pollution Act, the local authority can still use its nuisance powers under Section 80 of the Environmental Protection Act 1990 to deal with a variety of noise and vibration nuisances.

Local authorities can serve statutory notices prior to or during operations to control nuisances, reinforced, if necessary, by injunction proceedings in the High Court.

Proceedings by aggrieved residents. Irrespective of agreements, consent or other arrangements between a local authority and a contractor, a resident or residents aggrieved by piling operations can make a complaint to a magistrates' court. The magistrates' court shall, if it is satisfied a nuisance exists, make an order for the abatement of the nuisance or prohibit a recurrence of that nuisance (Section 82 of the Environmental Protection Act 1990).

3.4 CODE OF PRACTICE RECOMMENDATIONS AND CONTRACTUAL REQUIREMENTS

BS 5228: Part 4: 1986, *Noise Control on Construction and Open Sites*, which deals with piling operations, does not cover vibrations but notes that serious disturbance and inconvenience can result to anyone exposed to them. The recent revision to Part 4 (BSI, 1992) deals with piling vibrations. Types of vibrations generated, their effects, prediction and reduction of levels, are covered.

General conditions of contract (e.g. I.C.E. 5th Edition, 1973) usually require that work be carried out without unreasonable noise and disturbance or that reasonable precautions are to be taken to prevent nuisance or inconvenience. These broad requirements would, therefore, include the effects of piling vibrations.

4 Human perception of vibration

The threshold of human perception of vibration is very low. Expressed in terms of peak particle velocity, the threshold of perception corresponds to a velocity of about 0.15 to 0.3 mm/s at frequencies between 8 and 80 Hz. As their intensity increases vibrations become irritating, annoying and can even be frightening at intense levels.

The degree of annoyance primarily depends on the following factors:

- Physical and mental condition, age and attitude of a person
- Characteristics of vibration
- Accompanying noise
- Duration and time of day
- Current activity of a person
- Proximity of source
- Quality of pre-existing environment.

If people have been informed about the nature and timing of the disruption and are confident that reasonable care will be taken by those responsible for the work, their annoyance is likely to be less. Although such a public relations exercise, to give prior warning and assurance, is important, it is only one of a series of necessary actions.

Acceptability depends on factors like those listed above for annoyance, e.g. whether the occupants are at work or in residence, their state of health, whether it is day or night, and whether the vibrations are continual, repeated or sporadic. Figure 6 indicates how the typically acceptable vibration levels vary for situations of differing sensitivity. At night, only people working in industry are likely to tolerate vibrations.

It is not possible to define the levels of vibration intensity which will be unacceptable in any particular circumstances. What may be ultimately tolerated, although with complaint in one situation might, in another, lead to injunction proceedings. The underlying truth of Lord Justice Thursiger's direction of 1879 still remains: what would be a nuisance in Belgrave Square would not necessarily be so in Bermondsey.

Most of the vibration problems from piling relate to human tolerance, and because this aspect is subjective there can be no rigid definition of what constitutes 'annoying' or a 'public nuisance'. Reflecting this difficulty, the tendency now is not to differentiate between different degrees of acceptability or annoyance, but to use the expression 'adverse comment'.

Nevertheless, past experience gives broad guidelines of acceptability and case law the precedents which proved unacceptable. But both change.

4.1 PERCEPTION CRITERIA

Several scales of perception criteria have been developed (see Steffens, 1974). BS 6472: 1984 provides guidance to evaluation of human exposure to vibration in buildings (1 to 80 Hz). The draft DIN 4150: Part 2: 1986 provides a detailed, scientific method for determining the effects on persons in buildings.

Many of the scales were developed for permanent steady-state vibrations or transient events such as earthquakes or wind loading on buildings.

The three parameters used to define scales of discomfort are displacement amplitude; particle velocity; and acceleration. Lack of consensus on a preferred parameter that suitably covers various types of vibration has encouraged the production of an array of criteria, many of which have little in common with the effects of piling vibrations. Some of the more relevant perception scales are summarised below (for more details see Steffens, 1974).

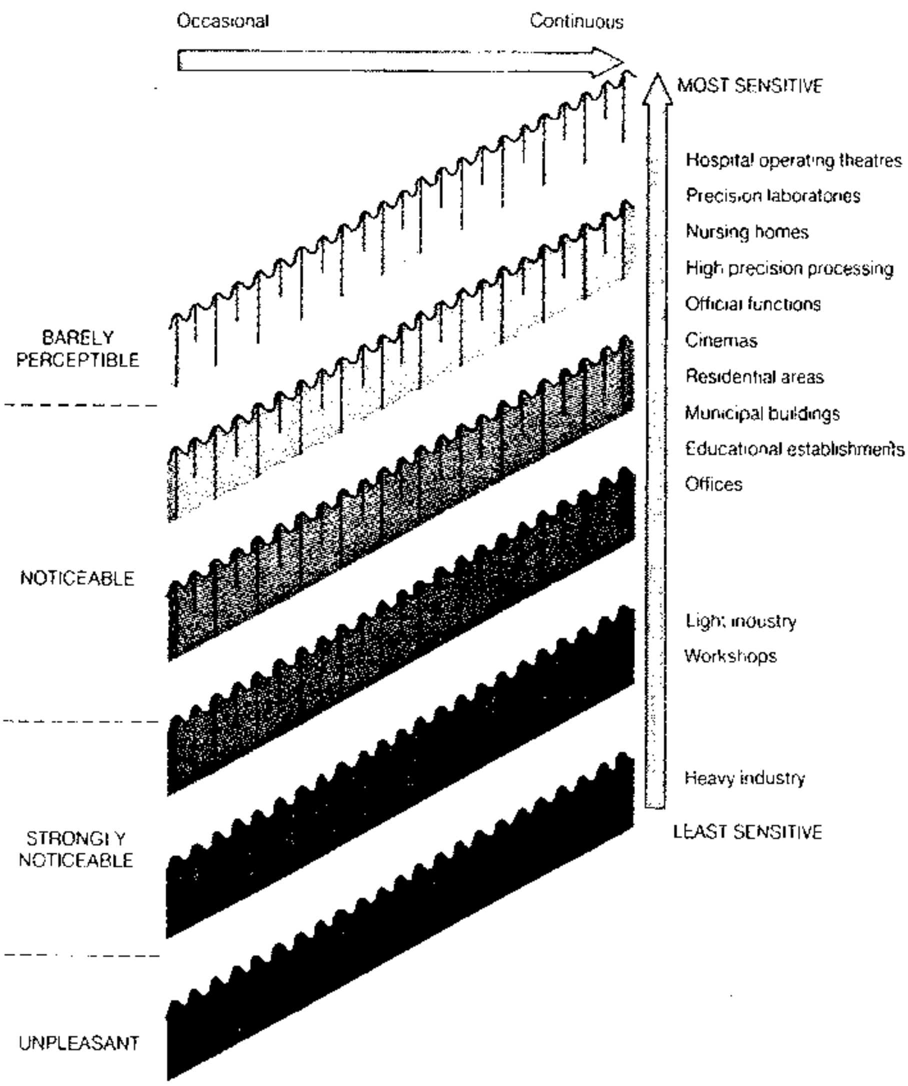


Figure 6 *Sensitivity and tolerance of different types of vibration*

Reiher and Meister related perception and discomfort to displacement amplitude and frequency for exposure to *continuous steady-state vibration*. They demonstrated that although a particular displacement amplitude may be just perceptible at low frequencies it may be unpleasant or even painful at high frequencies.

Dieckmann derived a perception intensity parameter, K, which can be related to the expected effects on people. K-values are derived from acceleration measurements at low frequencies, from velocities in middle ranges, and from displacement amplitudes at higher frequencies. DIN 4025 (German Standards Institute, 1958) employed K-values to classify and to predict the effects on workers. An amended version of this criterion is also adopted in subsequent DIN standards (see Section 4.2.1 about DIN 4150).

Zeller developed a unit termed a Vibrar which is based on common relationships between displacement amplitude, acceleration and frequency. The Zeller scale is related to the Mercalli-Cancani earthquake rating, and a prediction of the effect on both humans and property is derived. The scale is based on events of relatively short duration and therefore has limited application to piling.

The Pal Scale was developed by the German Standards Institute (DIN 4150, 1939) as a measure of the strength of vibrations in terms of root mean square peak velocities. The strength of vibrations can be related to various threshold levels of perception and discomfort.

4.2 AVAILABLE STANDARDS

4.2.1 DIN 4150 (1970 draft)

A much used source in vibration studies in the UK is the German Standard DIN 4150 (German Standards Institute, 1939, 1970 (draft), 1975 (provisional), and 1986 (draft)). The 1970 and 1986 drafts are available in translation from the British Standards Institution. BRE Digest No. 278 (HMSO, 1983), Vibrations: Building and Human Response, based its recommendations on the 1970 draft. For piling purposes the 1970 draft and provisional 1975 versions are considered adequate. The 1986 draft is highly technical and would only be applied in special circumstances.

The DIN specifications apply to vibrations transmitted through solid surfaces, such as floors, and apply particularly to sinusoidal vibrations. The method involves the calculation of the perception intensity (degree of perception) from the acceleration, velocity or displacement, and the frequency of the vibration. Comparison is then made with guide values given in tabular form for various locations in differing environments, having regard to duration and frequency of the vibrations and the time of day at which they occur.

In the 1970 draft, the degree of perception, K, is an amended version of that proposed by Dieckmann. Some guide values are given in Table 3. Suggested levels of vibrations acceptable in different situations are given in Table 4. It is thought that where piling in less sensitive situations is continued for only a few days, the acceptable vibration levels might be higher (2 to 3 times) than those suggested in Table 4.

4.2.2 DIN 4150 (1975 provisional standard)

In the 1975 provisional standard of DIN 4150 there are curves of equal perception intensity (in terms of another parameter, KB) for a frequency range 1 to 100 Hz: the curves are related to displacement, particle velocity and acceleration. The perception intensity parameter, KB, is derived from the peak values of these three characteristics as follows:

from acceleration: $KB = 20.2a/D$

from particle velocity: $KB = 0.13vf/D$

from displacement: $KB = 0.8Af^2/D$

where a = *acceleration* (m/s^2)
v = *velocity* (mm/s)
f = *displacement amplitude* (mm)

$$\text{and } D = \sqrt{1 + \left(\frac{f}{f_o}\right)^2}$$

for f_o = 5.6 *Hz* (*a reference frequency*).

Table 3 *K*-values and human perception of vibration (from DIN 4150: 1970 draft)

K value	Degree of Perception
below 0.1	not felt
0.1	threshold of perception
0.25	barely noticeable
0.63	noticeable
1.6	easily noticeable
4	strongly noticeable
10	very strongly detectable

Notes:
1. Applies to vertical and horizontal vibrations
2. Frequency range 0.5 to 80 Hz
3. Perception of persons sitting or standing
4. *K*-values calculated from:

$$K = 0.005Af^2/C$$
$$\text{or } K = 8vf/C$$
$$\text{or } K = 0.125a/C$$
$$\text{where } A = \text{displacement amplitude (mm)}$$
$$f = \text{frequency (Hz)}$$
$$v = \text{particle velocity (mm/s)}$$
$$a = \text{particle acceleration (mm}^2\text{/s)}$$
$$\text{and } C = \sqrt{1 + f^2}$$

Different values are given for the numerical constants if root mean square values of the vibration characteristics are used rather than peak values.

Figure 7 is a simplification of the peak particle velocity – frequency curves of equal perception intensity (*KB*) given in the 1975 provisional standard. A table of guide values (in terms of *KB*) is also given in the provisional standard for different situations. Table 5 presents those parts of that table most relevant to piling projects. Note that it refers to vibration in dwellings or comparable rooms.

Table 4 Suggested vibration levels acceptable in different situations (after DIN 4150: 1970 draft).

Situation		Degree of perception (*K*-values) Continuous vibrations[1]	Repeated vibrations[2]	Occasional vibrations[3]
Hospitals and nursing homes	day	0.1	0.1	2.1
	night			0.1
Residential areas	day	0.1	0.2	4
	night		0.1	0.1
Town residential and business areas	day	0.3	0.63	8
	night	0.1	0.1	0.1
Industrial areas	day	0.63	0.8	12
	night	0.63	0.8	12

Notes:
1. Continuous vibrations are defined as lasting more than 2h.
2. Repeated vibrations, here, are either continuous vibrations occurring occasionally or shocks that occur at intervals.
3. Occasional vibrations are transient vibrations lasting for a short time only e.g. shocks from blasting, which occur once to three times a day.

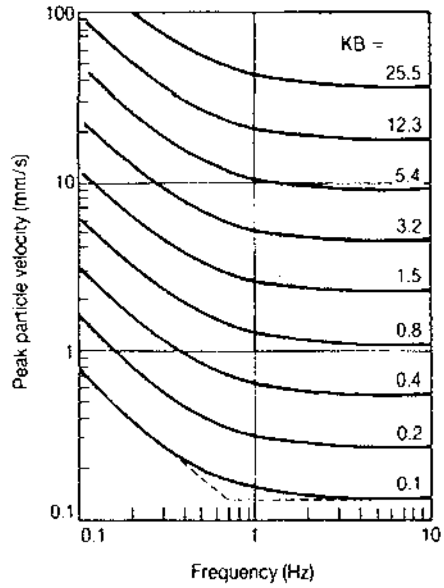


Figure 7 *Curves of equal perception intensity (KB) in terms of peak particle velocity and frequency*

4.2.3 DIN 4150: 1986 Part 2

The evaluation procedure presented in DIN 4150: 1986: Part 2, *Vibrations in Buildings, Effects on Persons in Buildings*, is a scientific approach based on a frequency-weighting method. Various formulae are used to derive time-weighted, root mean square vibration signals, which are pulse stretched to smooth, time-decay signals. Duration effects and rest periods are taken into account as are secondary audible effects (e.g. door and window rattling).

The oscillation amplitudes in both vertical and horizontal directions have to be measured for floor centres, structural members, walls, doors and window bays. At each point, measurements are taken in perpendicular directions.

Where measurements are of short duration, statistical approaches are recommended, except when constant signals are recorded or where occurrences are single or continuous.

In order to evaluate vibration emissions, the calculated time-weighted or root mean square vibration signals are compared with reference values, presented in a table, for both vertical and horizontal components. The table gives reference values mainly for commercial plants and housing (day and night). An approximate procedure is given in the draft standard with examples of worksheets and explanatory notes.

The approach, which this draft standard outlines, is not easily applied to piling vibrations acting over comparatively short periods. The vibration study needed to ensure compliance would be extensive and costly, and there would be a delay after measurement before the findings could be analysed. It seems probable, therefore, that this approach would only be useful in exceptional circumstances, such as where delicate sensitive work operations are necessary or where the vibrations can be expected over a substantial period of time.

4.2.4 ISO 2631 and BS 6472

ISO standard 2631: 1978: *Guide for the Evaluation of Human Exposure to Whole Body Vibration and Shock*, applies primarily to vibrations transmitted to the human body as a whole through the supporting surface in buildings, vehicles and by working machinery. It proposes three limits for preserving comfort, working efficiency, safety or health as follows:

(a) reduced comfort boundary
(b) fatigue – decreased proficiency boundary
(c) exposure limit.

Relationships are given for assessing perception in terms of (b), with suggested multiplying factors for assessing perception in terms of (a) or (c).

Of more direct relevance to this piling study is a draft addendum to ISO 2631, *Guide to the Evaluation of Human Exposure to Vibration and Shock in Buildings* (Document 81/78512 DC, 1981 – which has been translated into BS 6472: 1984, *Guide to evaluation of human exposure to vibrations in buildings (1 Hz to 80 Hz)*. This gives levels of vibrations in terms of satisfactory magnitudes of particle accelerations or velocities for people in buildings.

The draft addendum takes into account the following factors:

- type of excitation : steady-state vibrations, intermittent vibration and impulsive shock
- type of building : workshop, office, residential, hospital operating theatre or other critical area
- time of day
- position of persons affected : standing, sitting or lying, or combination of the three.

Impact pile driving is defined as producing intermittent vibrations and vibro- or resonant piling is classed as continuous. No differentiation is made between different types of dwelling.

Table 5 Guide values for the assessment of vibration in dwellings or comparable rooms (after DIN 4150: 1975 provisional standard)

Situation[1, 2]	**Guide values for continuous vibrations and vibrations occurring repeatedly with interruptions**[3]		
		KB	***v* (mm/s)**
Quiet residential areas	day	0.1 (0.15)	0.3 (0.2)
	night	0.15	0.2
Town residential and commercial areas	day	0.3 (0.2)	0.4 (0.3)
	night	0.2	0.3
Commercial areas including offices	day	0.4	0.55
	night	0.3	0.4
Industrial areas	day	0.6	0.85
	night	0.4	0.55

Notes:
1. Lower values may be appropriate in certain special areas.
2. Day (0600 to 2200 h): Night (2200 to 0600 h).
3. Continuous vibrations and vibrations occurring repeatedly with interruptions are those which occur for a period longer than 2h.
4. The guide values in brackets should not be exceeded when structures are subjected to horizontal vibrations of 5 Hz and below.
5. With vibrations confined to daylight hours and over a few days, such as those from piling, guide values up to double those given above might be suitable provided damage thresholds are not exceeded.

BS 6472: 1984, *Guide to evaluation of human exposure to vibration and shock in buildings*, provides guidance to human response to building vibration within a frequency range 1 to 80 Hz with weighting curves applying to different situations. It recognises that perception depends on human body orientation and therefore produces different curves for *z*-axis vibration (head to foot) and *x*- and *y*-axis vibration (back to chest and right to left side). These curves are given as Figures 8(a) and (b). The situations to which they apply are presented as Table 6.

Table 6 Multiplying factors applying to satisfactory magnitudes of building vibration for human response (after BS 6472: 1984)

Situation		Multiplying factors[1] Continuous vibrations	Multiplying factors[1] Intermittent vibrations
Critical working areas	day	1	1
	night	1	1
Residential areas	day	2 to 4	60 to 90
	night	1.4	20
Offices	day	4	128
	night	4	128
Workshops	day	8	128
	night	8	128

Note 1. Refer to curves in Figures 8(a) and (b).

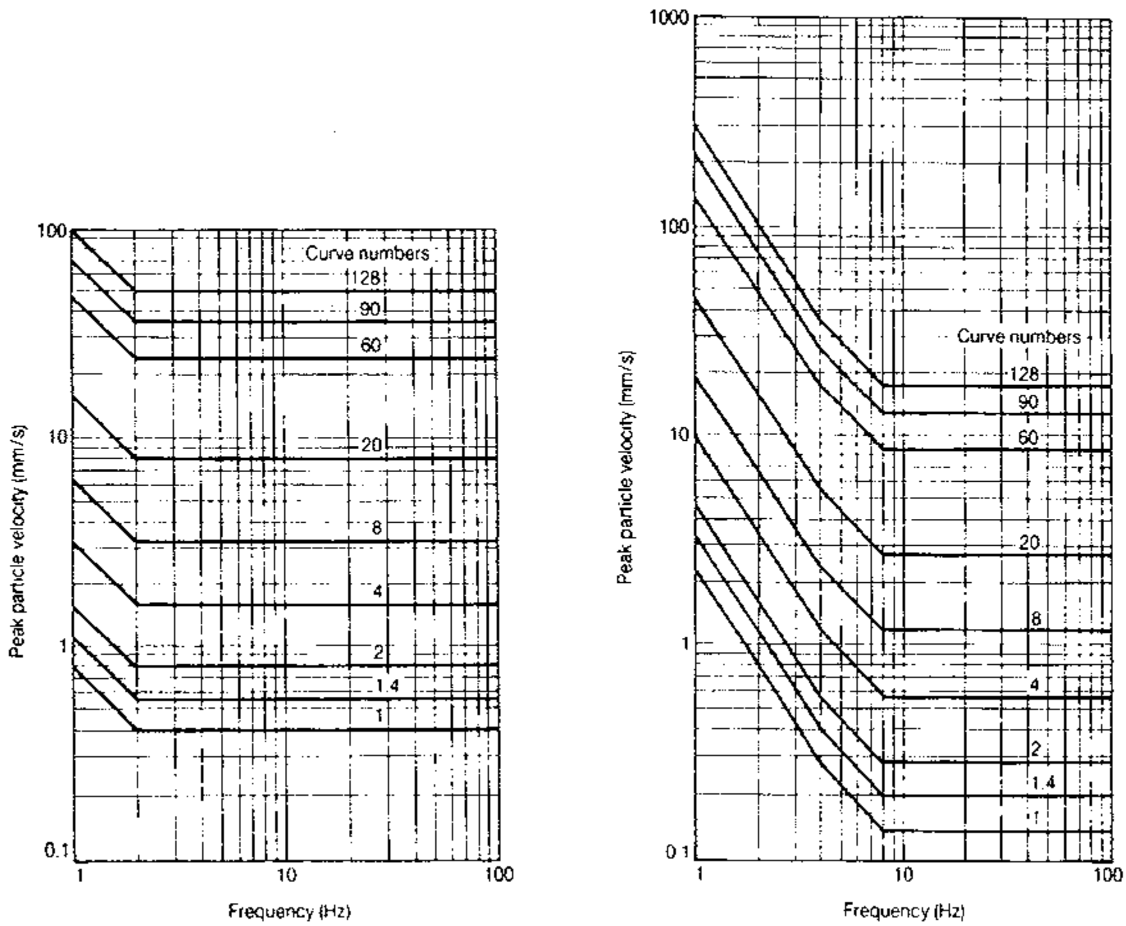


Figure 8 *Curves relating typical human response to vibrations in buildings (a) x- and y-axis vibration (b) z-axis vibration*

5 Effects on buildings

The response of buildings to ground-borne vibrations is governed by the following factors:

- Relationship between natural frequencies of the building and its elements and the characteristic frequency of the ground vibration
- Magnitude of vibration
- Stiffness of building and elements
- Damping characteristics of the building
- Other factors such as height and dimensions of the building and the types of materials used in its construction.

Specialist advice is required for a full assessment of stiffness, damping and interaction characteristics: in normal circumstances this level of sophisticated analysis would not be justified.

There is a distinction to be made between structural, serviceability and aesthetic damage. Structural damage impairs safety and may result in some form of failure, whereas aesthetic damage is mainly an impairment of the appearance of surface finishes and fittings. Cracking of plaster and rendering is annoying to building owners and could affect durability. Serviceability failures would include loss of weatherproofing, sticking of doors and windows, sloping floors and wall movements. A classification of damage is presented in Table 7 which is based on ease of repair of visible damage to walls. This classification does not consider the subjective judgements that need to be made on acceptability of cracking, nor should crack width be used as the only measure of consequences of damage.

Cases where vibration has caused serviceability damage to buildings are rare and those reported note that levels of vibration become unpleasant and even painful to the occupier before such damage results. When investigated, aesthetic damage, such as plaster cracking or loosening of roofing tiles, which was initially attributed to vibrations, is often found to have been produced by other causes. In many cases, such defects have only been noticed as a result of inspection through concern produced by the onset of vibrations – but were there previously. In other cases it is possible that there were incipient defects made apparent by the vibrations.

It is also important to recognise other possible causes of building damage, i.e. not necessarily associated with vibrations of the structure or its components. Piling can cause ground settlement or heave which could distort a structure; other associated construction operations may give rise to ground movements; or there may be several other quite separate reasons for building damage.

5.1 OTHER CAUSES OF DAMAGE TO STRUCTURES

A number of mechanisms can lead to damage to structures. These include:

1. Differential settlement of foundations.
2. Thermal expansion and contraction of structural elements.
3. Shrinkage and swelling of clay soils (causing differential settlement or heave).
4. Shrinkage of construction materials in new buildings.
5. Deterioration of construction materials.
6. Loss of ground support.
7. Groundwater lowering (causing settlement or loss of ground).
8. Frost heave.

Table 7 Classification of visible damage to walls with particular reference to ease of repair of plaster, brickwork or masonry (adapted from BRE Digest 251)

	Category of damage[1]	Description of typical damage	Approximate crack width (mm)
Aesthetic	0	Hairline cracks of less than about 0.1 mm width are classed as negligible	Up to 0.1[2]
	1	Fine cracks are evident which can easily be treated during normal decoration, perhaps with isolated slight fracturing in building. Cracks are rarely visible in external brickwork	Up to 1[2]
	2	Cracks easily filled. Redecoration probably required. Recurrent cracks can be masked by suitable linings. Cracks not necessarily visible externally. Some external repointing may be required to ensure weathertightness. Doors and windows may stick slightly	Up to 5[2]
Functionable serviceability	3	Cracks should be broken out and repaired by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired	5 to 15[2] (or a number of cracks up to 3)
	4	Extensive repair work necessary involving breaking-out and replacing sections of walls, particularly over doors and windows. Windows and door frames distorted. Floors sloping noticeably[3]. Walls leaning[3] or bulging noticeably. Some loss of bearing in beams. Service pipes disrupted	15 to 25[2] but also depends on number of cracks
Structural	5	Requires major repairs involving partial or complete rebuilding. Beams lose bearing. Walls lean badly and require shoring. Windows broken with distortion. Danger of instability	Usually greater than 25[2] but depends on number of cracks

Notes:

1. In assessing the degree of damage, account must be taken of the location in the building or structure where it occurs, also of the function of the building or structure. The categorisation is purely related to ease of repair, and is not a general objective statement of the acceptability of cracking in a building. Other factors, such that might, for example, affect the market value of the property, must be taken into account even when the cause of damage is not progressive or a threat to the stability or serviceability of the structure.

2. Crack width is one factor in assessing category of damage and it should not be used on its own as direct measure of it.

3. Local deviation of slope from the horizontal or vertical of more than 1/100 is normally clearly visible. Overall deviations in excess of 1/150 are undesirable.

In any enquiry into reported damage to buildings all other possible causes, as well as vibrations, should be carefully considered.

An important indirect cause of building damage through piling results from dynamic (vibration-induced) ground settlements in granular soils. Structural damage to varying degrees can result, but it is most pronounced where shallow foundations overlie loose or medium dense deposits. Dynamic settlement predominantly affects uniform fine sands and silty sands below the groundwater level, and produces movements that are differential in nature. The degree of induced movement will depend on the initial density of the soil, level of vibration, and distance from source. Various authors (NAVFAC 1982; Heckman and Hagerty, 1978; and D'Appolonia, 1971) suggest settlements can be expected for a distance of up to 10 pile diameters and occasionally as much as 10 to 15 m distant from driven piles.

In exceptional circumstances movements can be induced at greater distances. For example where a loose sand or fill overlies a dense gravel (or rock) and piles are driven to end-bear on dense material, adjacent buildings founded on shallow foundations can be affected, because the vibrations can be transmitted from the pile base, along the dense layer, and cause dynamic settlement directly beneath shallow foundations at adjacent structures. Ground settlements of up to 300 mm are also possible from driving pile groups in uniform granular soils. A further cause of building damage results where the bearing capacity of adjacent foundations approaches unity and piling vibrations trigger collapse or partial collapse, or where foundation stability has been reduced through temporary excavations.

The effect on granular soils from vibro-drivers and vibroflots is more marked as the characteristic frequency of the ground is approached. This is particularly the case on the run-up and run-down from the operating frequencies which are usually about 30 Hz.

5.2 DAMAGE CRITERIA FOR BUILDINGS

As with human perception of vibrations, there is no single set of criteria for predicting damage to buildings from vibrations. Various proposals have been made but they differ in the parameters used to characterise the vibrations, in the levels of risk, in the damage thresholds and in the categorisation of both damage and the buildings.

Much of the published guidance, including national codes and international standards, deals with the measurement and effects of vibrations, machinery, traffic, blasting, wind and earthquakes etc. Although construction activities may be included and piling may be mentioned, the guidance is not specifically about piling-induced vibrations.

It is not possible to define what constitutes a damaging vibration with any degree of confidence. There are no universally accepted criteria which can be used to predict the effects on all buildings and structures. Various criteria have been proposed in America and Europe, based on blasting, earthquake and similar evidence, which attempt to predict the likelihood of damage in terms of measured frequency, amplitude, velocity or acceleration. But there are also many other factors, such as the increased stresses generated by vibrations, fatigue properties, resonance and sensitivity, which are unquantifiable.

Early research in the USA, based on the effects of blasting vibration, provided guidelines in terms of maximum amplitude before onset of damage to wood-framed and brick buildings. Similar tests carried out in Sweden and Britain were also based on amplitude. For piling vibrations, however, measurement in terms of peak particle velocity (*ppv*) is considered more appropriate for several reasons. Displacement and acceleration measurements are more sensitive to low and high frequency components respectively. The measurement of velocity can be made directly (without having to integrate or differentiate) from geophones, which are robust and cheap, in the relevant frequency range (5 to 200 Hz). The response of people as well as structures can also be conveniently related in terms of velocity.

5.2.1 Criteria from blasting technology

The ground vibrations from blasting are of short duration from sporadic or even rare events. Blasting is almost invariably in hard rock and although piles may be taken to bear on rock at depth the reason for piling is that the ground is relatively weak. Much of the North America data is for timber structures. Therefore it is to be expected that blasting vibration criteria permit higher intensities (whether described in terms of amplitude or velocity) than are given for other types of operation and more sensitive buildings.

American velocity limits for damage from blasting are based on measurements in one component direction on the ground. Early reporters suggested that slight damage could occur at maximum (non-simulated) peak particle velocities of 50 mm/s and moderate damage at over 100 mm/s (Crandell 1949, Duvall and Fogelson, 1962). Steffens (1974) reports the Dieckmann *K*-value of about 60 (extremely unpleasant), equivalent to 75 mm/s, for damaging vertical vibrations of 5 to 40 Hz.

Theissen and Wood (1982) summarise various proposals of damage thresholds, which are primarily based on blasting technology, for four classes of structure affected. A summary of the criteria assessed is presented as Figure 9, which illustrates the divergence of opinion. They categorise vibration damage to structures as follows:

- Damage to building content: including stock, equipment and computers
- Structure deterioration: including aggravation effects to structural and architectural damage
- Architectural damage: including broken windows, cracked plaster, non-bearing wall and frame damage
- Structural damage: including effects on structural components or integrity.

In the past structural and architectural damage has received most attention, but it is expected that building contents will receive more attention in future because of the proliferation of vibration-sensitive computer equipment.

Siskind *et al.* (US Bureau of Mines, 1980) reassessed structural response to blasting vibrations based on measurements in 76 homes. They concluded that the previously recommended damage threshold *ppv* of 50 mm/s is not appropriate when frequencies are below 40 Hz. Within this range of frequencies (which is appropriate to piling vibrations) they proposed damage threshold values of *ppv* of 18 mm/s for modern homes with drywall interiors and 12 mm/s for older homes with plaster and wood lath construction. But, it is important to note that these values are for single events, measured only on one direction on the ground and, moreover, no account is taken to reduce thresholds for continuous or intermittent vibrations from operations such as piling. New (1990) discusses the ground vibrations caused by blasting and other construction works.

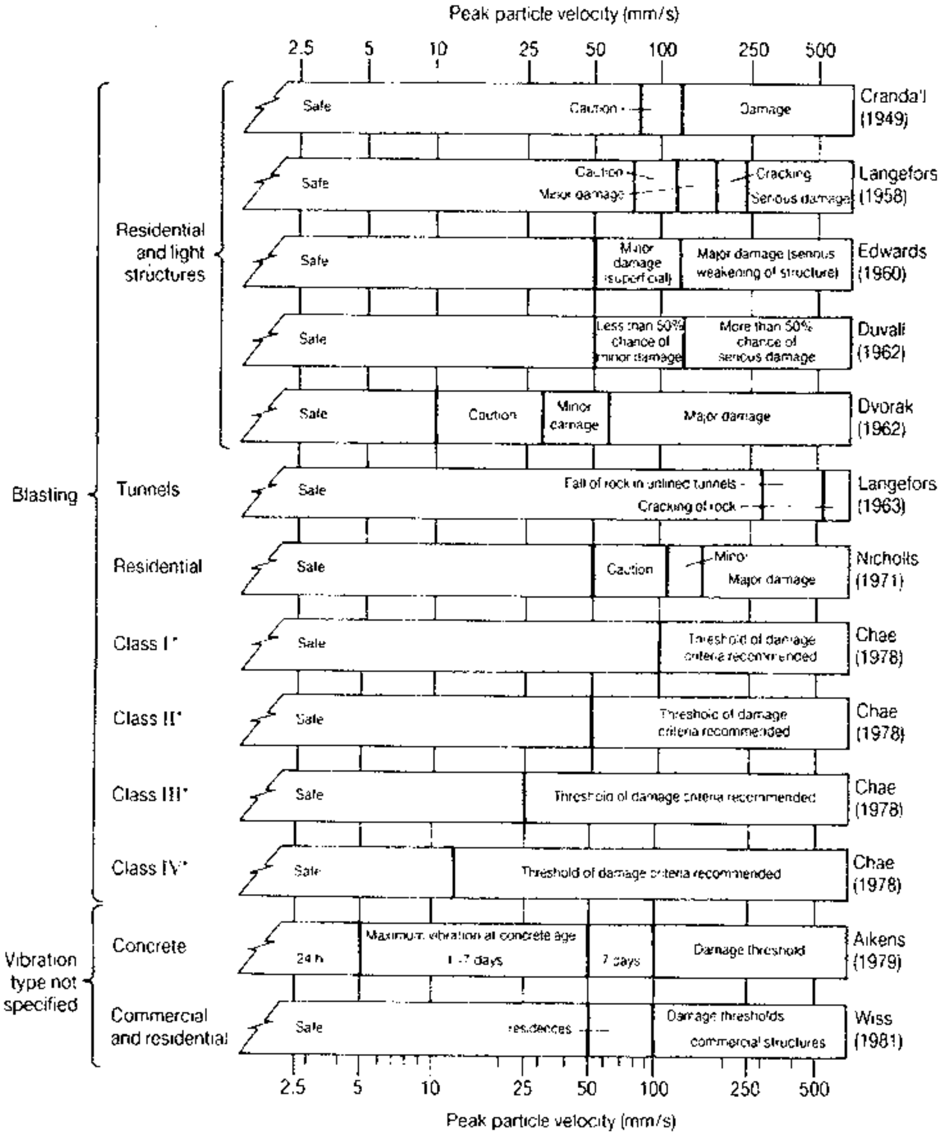
5.2.2 Published guidance about the effects of vibration on buildings

BRE Digest 353 is the most recent guidance note from an authoritative UK source on this subject. It points out that there is no current UK code or standard and that it will be some years before one could be developed. In referring to DIN 4150, a Swiss standard and a Swedish Code, it notes the different approaches these adopt and that the data on which they are based do not necessarily apply to UK conditions and structures.

In the absence of definitive guidelines for the UK, this Section presents some of the approaches that have been considered in other countries as well as in the UK. As with human perception criteria, much of the key developments have been in Germany.

Koch (1953) and Soir (1961) developed damage criteria based on the Zeller Scale of Vibrars (see Section 4.1), but it has since been widely acknowledged that peak particle velocity is the most appropriate single description for damage criteria prediction for piling vibrations.

DIN 4150 (1970 draft) recommended the measurement of the resultant peak particle velocity, although its recommendations are principally directed towards blasting technology. The recommended criteria are given in Table 8 for foundation vibrations (in the frequency range 8 to 80 Hz) caused by blasting once or twice each day, but the type of damage which these criteria are designed to prevent is not given. It may be noted that no damage is to be expected when the peak resultant velocity is less than 2 mm/s. Although the guide values would not apply to vibro-diving or to impact hammer piling where the vibrations do not have time to dissipate between blows, they may not be inappropriate where the blows are well spaced.



* Structures are classified as follows (Chae, 1978)

Class I : structures of substantial construction
Class II : relatively new residential structures in sound condition
Class III : relatively old residential structures in poor condition
Class IV : old residential structures in very poor condition

(note: if structure is subjected to repeated blasting or if blasting is done without instrumentation, lower class by one)

* Structures are classified as follows (Chae, 1978):

Class I: structures of substantial construction
Class II: relatively new residential structures in sound condition
Class III: relatively old residential structures in poor condition
Class IV: old residential structures in very poor condition
(Note: if structure is subject to repeated blasting or if blasting is done without instrumentation, lower class by one)

Figure 9 *Observed or anticipated damage thresholds*

Table 8 Representative values of peak particle velocity for assessing vibrations from sudden shocks (after DIN 4150: 1970 draft)

Class of Building	Nature of Building	Permissible *ppv* (mm/s)
I	Ruins and damaged buildings, protected as monuments	2
II	Buildings with visible defects, cracks in masonry	4
III	Undamaged buildings in technically good condition (apart from cracks in plastering)	8
IV	Well stiffened buildings (e.g. industrial buildings)	10 – 40

Notes:

1. Guide values are inappropriate if there is the possibility of ground compaction because of the vibrations.

2. If blasting is more frequent than twice in a working day, for example, the guide values should be reduced to two thirds of values shown.

3. One axis of measurement should be towards the source of the vibrations or parallel to one of the side walls of the building.

4. In large buildings, the vibrations should be measured in several places simultaneously, including the foundations, the ceilings and upper storeys.

5. For vibrations in ceilings, values of up to 20 mm/s in the vertical direction (v_z) may be permissible.

Part 3 of DIN 4150: 1986 deals with transient and continuous vibrations in building structures. For transient vibrations the velocity is defined as the maximum of the three single-components of vibration velocity. It should be measured on the foundation of the outer wall of the building or in the outer wall. A further important position is defined as the ceiling level of the last full floor. Table 9 presents the guide values given for various types of buildings and lists special features for their use.

For continuous vibration, including vibratory pile drivers and vibrators, *ppv* values of up to 5 mm/s (measured at the top floor) for types 1 and 2 should not result in damage or superficial cracking. Data for type 3 are not available. Similarly, it is noted that if values are exceeded, it is not inevitable that damage will occur. Where a building is excited as a harmonic it is recommended that measurements are made simultaneously on several storeys. The Standard suggests that a rough estimate of the lowest natural frequency of horizontal free vibrations for multi-storey buildings is:

$$f = \frac{10}{\textit{number of storeys}} \text{ (Hz)} \qquad \text{... (5.1)}$$

The Swiss Standard SN 640 312: 1978, which is also quoted in BRE Digest 353, is entitled *Effects of vibrations on structures*. It also relates limiting peak particle velocities (true peak resultant values) to various types of construction for differing frequencies.

Table 10 summarises the guide values provided for vibration sources *including machines, traffic* and constructional equipment. No allowance is made between continuous and intermittent vibrations, but it is implied that the quoted values apply more directly to semi-continuous vibrations. The measurements should be made at the foundation of the structure.

Table 9 Guideline values of vibration velocity, v_i, for evaluating the effects of short-term vibration (after DIN 4150 Part 3: 1986 and as given in BRE Digest 353)

Type of structure	Vibration velocity, v_i, in mm/s			
	Foundation			Plane of floor of uppermost full storey
	At a frequency (Hz) of			Frequency mixture
	< 10	10 – 50	50 – 100*	
1. Commercial and industrial buildings	20	20 to 40	40 to 50	40
2. Dwellings and buildings of similar design and/or use	5	5 to 15	15 to 20	15
3. Structures that, because of their particular sensitivity to vibration, do not correspond to those listed above and are of great intrinsic value (e.g. buildings that are under a preservation order)	3	3 to 8	8 to 10	8

* for frequencies above 100 Hz, at least the values specified in this column should be applied.

Notes:

1. If the maximum measured value at foundation level for buildings over two storeys is higher than 0.7 times the values given then it is essential to carry out measurement at the top floor.

2. Effects of piling vibrations are covered and relevant provided that the time period between driving blows is sufficient for the effect due to one blow to die away in buildings before the next is applied.

3. Damage referred to in the first two categories also includes fine plaster cracks.

4. Values quoted are applicable to direct damage resulting from vibration and will not occur as a result of effects of former vibrations.

5. If guide values are exceeded it is not inevitable that damage will occur, but continued observation is essential.

In France, the Ministry of Environment and Transport *Regulations for mechanical vibrations affecting buildings* (1986) adopt the same criteria and tables as defined by the ISO/DIS 4866: 1980 described below. They contain guidance for both intermittent and continuous vibrations, and provide a methodology for measurement and assessment for various structures and ground conditions. These are considered for both direct and indirect effects. Figures 10 and 11 provide limiting guide values for intermittent and continuous vibrations and are related directly to Table 11, Classes of building, and Table 12, Categories of structure. If guide values are exceeded it is not inevitable that damage will occur, but it is recommended that in such circumstances continuous monitoring is carried out.

The draft international standard ISO/DIS 4866: 1986 deals with measurement and evaluation of vibration effects on buildings; and is intended to be used as an International Standard. It considers source-related and building-related factors and outlines methods of vibration measurement and methods of evaluating data. Annex A to the Draft defines categories of vibration resistance.

Different building types are set into categories of structure (see Table 11) and different classes of building are shown in Table 12 depending on their structure category and their foundation type and ground conditions (Table 13). Thus a church of historical importance (building group 1) would be a structure category 8. If it has large stone footings (Class C foundation) on sand and gravel (Type e ground conditions) it would then be considered as a Class 14 building.

It may be noted that this special category (8) is not included in the French Regulations (Figures 10 and 11).

Table 10 Structural types and guide values of vibration velocity (after SN 640 312: 1978)

Structural type	**Guide values of true peak resultant particle velocity v_{max} (mm/s) for frequency ranges of:**	
	10 to 30 Hz	**30 to 60 Hz**
I Reinforced concrete and steel construction (without plaster) such as industrial and commercial buildings; retaining walls; bridges; towers; above ground pipelines. Underground structures such as caverns, tunnels, galleries with or without concrete lining.	12	12 to 18*
II Buildings with foundation walls and floors in concrete with walls in masonry or concrete. Retaining walls of ashlar construction. Underground structures as in I but with masonry lining. Pipelines buried in soft ground.	8	8 to 12*
III Buildings with foundations and basement floors of concrete construction with wooden beam construction in upper floors; brickwork walls.	5	5 to 8*
IV Buildings which are especially sensitive or worthy of protection.	3	3 to 5*

* The lower value applies at 30 Hz, the upper one at 60 Hz. Interpolate for intermediate frequencies.

5.2.3 Review of criteria

From these published criteria for building damage some broad, tentative guidelines can be derived.

It would appear that damage is unlikely (even improbable) at true peak particle velocities of less than 2 mm/s measured on structural foundations. Exceptions could be previously damaged or dilapidated buildings and monuments. The effects of vibrations also depend, however, on the predominant frequency; for a given value of *ppv*, the risk decreases for higher frequencies and increases for lower frequencies.

Table 11 Categorisation of structures according to group of building (after Draft ISO/DIS 4866, 1986)

Category of structure		Group of building 1	Group of building 2
Resistance to vibration decreasing	1	Heavy industrial multi-storey buildings, five to seven storeys high, including earthquake-resistant forms. Heavy structure, including bridges, fortresses, ramparts	Two- and three-storey industrial, heavy-frame buildings of reinforced concrete or structural steel, clad with sheeting and/or infilling panels of blockwork, brickwork, or precast units, and with steel, precast or *in-situ* concrete floors. Composite, structural steel and reinforced concrete heavy industrial buildings
	2	Timber frame, heavy, public buildings, including earthquake-resistant forms	Five- to nine-storey (and more) blocks of flats, offices, hospitals, light-frame industrial buildings of reinforced concrete or structural steel, with infilling panels of blockwork, brickwork, or precast units, not designed to resist earthquakes
	3	Timber-frame, single- and two-storey houses and buildings of associated uses, with infilling and/or cladding, including "log cabin" kinds, including earthquake-resistant forms	Single-storey moderately lightweight, open-type industrial buildings, braced by internal cross walls, of steel or aluminium or timber, or concrete-frame, with light, sheet cladding, and light panel infilling, including earthquake-resistant types
	4	Fairly heavy multi-storey buildings, used for medium warehousing or as living accommodation varying from five to seven storeys or more	Two-storey, domestic houses and buildings of associated uses, constructed of reinforced blockwork, brickwork or precast units, and with reinforced floor and roof construction, or wholly of reinforced concrete or similar, all of earthquake-resistant types
	5	Four- to six-storey houses, and buildings of associated urban uses, made with blockwork or brickwork, load-bearing walls of heavier construction, including "stately homes" and small palace-style buildings	Four- to ten-storey domestic and similar buildings, constructed mainly of lightweight load-bearing blockwork and brickwork, calculated or uncalculated, braced mostly by internal walls of similar material, and by reinforced concrete, preformed or *in-situ* floors at least on every other storey
	6	Two-storey houses and buildings of associated uses, made of blockwork or brickwork with timber floors and roof Stone or brick-built towers, including earthquake-resistant forms	Two-storey domestic houses and buildings of associated uses, including offices, constructed with walls of blockwork, brickwork, precast units, and with timber or precast or *in-situ* floors and roof structures
	7	Lofty church, hall and similar stone- or brick-built, arched or "articulated" type structures, with or without vaulting, including arched smaller churches and similar buildings Low heavily constructed, "open" (i.e. non-cross-braced) frame church and barn type buildings including stables, garages, low industrial buildings, town halls, temples, mosques, and similar buildings with fairly heavy timber roofs and floors	Single- and two-storey houses and buildings of associated uses, made of lighter construction, using light-weight materials, pre-fabricated or *in-situ* separately or mixed
	8	Ruins and near-ruins and other buildings, all in a delicate state All class 7 constructions of historical importance	

Table 12 Classification of buildings according to their resistance to vibration and the tolerance that can be accepted for vibrational effects (Draft ISO/DIS 4866, 1986)

Class of Building		Category of structure (see Table 11): 1	2	3	4	5	6	7	8
		Categories of foundations (capital letters) and types of soil (lower case letter)							
	1	A a							
	2	A b	A a	A a	A a				
	3		A b B a	A b B a	A b	A a A b			
	4		A c B d	B b	A c	A c B a B b			
	5		B c	A c		B c	B a		
Level of acceptable vibration decreasing	6		A f		A d	B d	B b C a	B a	
	7			A f	A e	B e	B c C b	B b C a	
	8						B e C c	B c C b	
	9		B f				C d	B d C c	A a
	10			B f			C e	B e C d	A b
	11				C f	C f		C e	B a
	12						C f		B c C a
	13							C f	B d C b C c
	14								C d C e C f

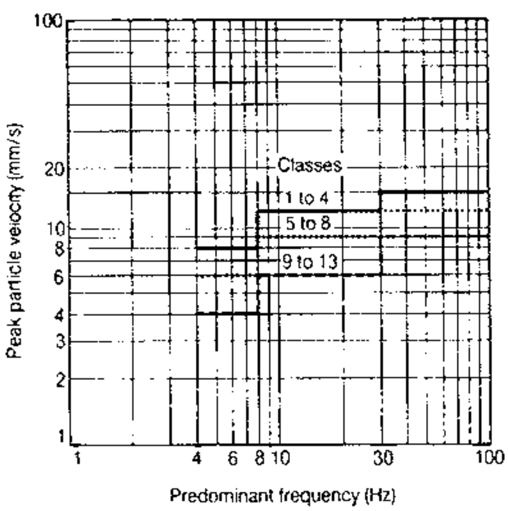


Figure 10 *Limiting vibrations from an intermittent source affecting various classes of building*

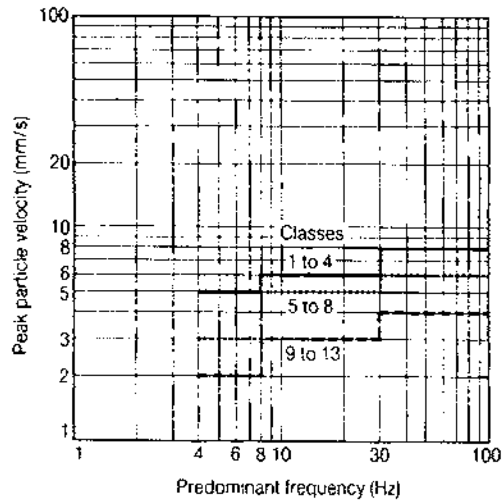


Figure 11 *Limiting vibrations from a continuous source affecting various classes of building*

Table 13 Types of foundation and ground conditions (from ISO/DIS 4866: 1986)

Type of foundation	Class
Linked reinforced concrete and steel piles Stiff reinforced concrete raft Linked timber piles Heavy retaining wall	A
Non-tied reinforced concrete piles Spread wall footing Timber piles and rafts	B
Light retaining walls Large-stone footing No foundations (wall direct on to ground)	C
Ground conditions	**Type**
Unfissured rocks or moderately strong rocks, slightly fissured; cemented sands	a
Compact, horizontally bedded soils	b
Poorly compacted, horizontally bedded soils	c
Sloping surfaces with potential slip planes	d
Open granular soils, sands, gravels (non cohesive) and cohesive saturated clays	e
Fill	f

For *ppv* values of between 2 and 10 mm/s generated by impact piling there is the increasing possibility of plaster cracking, of the enlargement of existing (visible) defects and of cracking in finishes. Undamaged buildings of modern construction to a high standard, and which have been well maintained, should not be affected by intermittent vibrations. At *ppv* values between 10 and 50 mm/s, damage is an increasingly high risk, but again much depends on the geometry (structural form), the materials and the construction quality of the building e.g. well stiffened industrial buildings are less likely to be seriously damaged. Consideration was being given in the draft addendum to Part 4 of BS 5228 to limits of 20 mm/s for light industrial structures and of 30 mm/s for heavy industrial structures.

Above 50 mm/s some form of damage is likely, which might include adverse effects on load-bearing units.

These guide values should be reduced to about 50% for continuous vibrations and further reduced if the buildings are dilapidated or have been damaged beforehand.

Particular care is necessary to anticipate the response of old buildings. For example, whereas modern plasterboard ceilings are unlikely to be affected by moderately intense vibration (apart from cracking at the joints), old lath and plaster ceilings can collapse when subjected to comparatively minor levels. Ancient structures also require special attention because dynamic settlements can be induced and walls are often out of line or inadequately propped by buttresses, anchors or underpinning. Any consequent damage will be costly to repair.

Retaining walls and retained facades also require careful assessment. They may have little lateral restraint near their top and can experience vibration amplification by factors of between 3 to 5.

Threshold values of 10 mm/s at the toes of masonry and earthwork walls and 40 mm/s at their crests have been suggested (BS 5228: Part 4: draft addendum) with increases of 50 to 100% for well-propped or massively constructed walls. Selby (1991) reports the results of a field test measuring ground vibrations and dynamic strains in 1.5 m high brickwork walls when an H-pile was driven by drop hammer and vibratory driver at different distances from them. He reports the high tolerance of the brickwork to dynamic strain, which was found to be proportional to the incident radial components of vibration.

The variety of situations and wide range of structures and their possible responses to piling vibrations are such that expert advice should be sought wherever a preliminary assessment indicates unacceptable risk.

5.3 EFFECTS ON BUILDING CONTENTS

Sensitive plant, computers, machines and laboratory equipment housed in adjacent structures can be damaged, even by low levels of vibration. When assessing potential effects to such equipment, it is recommended that advice is obtained from either the manufacturers or operators on acceptable vibration levels. It is usual to specify tolerances in terms of peak displacement but limits in terms of velocity or acceleration can also be expected. Experience suggests that the tolerances set are usually conservative, but each case should be assessed individually. Manufacturers' criteria for computers are usually expressed in terms of displacement amplitude at particular frequencies. Corresponding particle velocities can be evaluated using wave mechanics assuming sinusoidal motion. Allowable thresholds for continuous vibration should be reduced to about 40%.

ISO 8569 gives provisional guidance about measuring and evaluating shock and vibration in relation to sensitive electronic equipment, but there is no explicit guidance on acceptable vibration levels.

Martin (1980) notes that there is an amplification factor between ground vibrations and floor vibrations. For buildings with concrete floors amplification factors (*ppv* floor/*ppv* ground) of between 0.32 and 0.92 are quoted, whereas those for wooden floors are between 1.55 and 1.76. From an assessment of the case records described in Section 6, particle velocity values measured on foundations are lower (about half) than those measured on the adjacent ground surface (see also Greenwood and Kirsch, 1983).

5.4 DAMAGE TO FRESH CONCRETE

There is insufficient information on the deleterious effects of vibrations on fresh and young concrete. Tests on partially hardened concrete (Akins and Dixon, 1974) suggest that vibration during the initial 2 to 4 h can actually increase strength by up to 35%. Suggested safe levels of vibrations based on this research are:

Concrete age (days)	*Permissible ppv* (mm/s)
less than 1	5
1 to 7	50

The critical period for placed concrete is after the concrete has taken its initial set and before it has acquired appreciable strength. It is therefore expected that piles may be driven close to newly cast-in-place piles up to say 4 h or after a few days, but not in between.

6 Case records

The initial stage of this CIRIA project was to obtain case histories of piling projects for which there had been thorough studies of the induced vibrations and their effects. Public and private organisations and universities provided information for about 150 projects (including a few ground improvement works). Most of these projects were completed in UK between 1970 and 1980, some were from abroad. It had been hoped that this information, when analysed, would provide evidence or insights for the assessment of the potential for annoyance or damage from piling vibrations. Another purpose was to compile the records in a form that could be referred to by others and which would be the start of a UK database.

Neither of these intentions has been fully achieved. Many of the records provided to CIRIA lack important information. A sixth of them were entirely without measurements of vibrations, so that the assessments given appear to be based on subjective opinions or purely qualitative records. In only a few of the received records is it clear that a comprehensive assessment and monitoring programme had been completed. The following list shows the information typically missing:

- details of building
- foundation type
- ground conditions
- pile type, dimensions and depth
- type of piling rig, details of driving and hammer
- distances of pile construction from affected structure
- frequency of vibrations at measuring point
- ambient (background) vibration levels
- measured values of displacement, velocity or acceleration
- details of how or where measurements were taken.

It is understood that, over recent years, BSI has been conducting a survey of cases of ground-borne vibration damage. BRE Digest 353 (July 1990), referring to this survey notes that 'the data for many of the cases are neither documented fully nor verified ...'. It is partly for this reason that CIRIA has not published an analysis of the 150 case records received in the early 1980s. The existence of these records was made known to BSI, but it is not clear if they included them in their survey. Since 1980, many more studies have been made of ground-borne vibrations not only for piling works but of other civil engineering operations and of the effects of traffic on buildings (see New, 1986 and 1990; Watts, 1990). Walton (1990) reports a database of structural drainage cases related to ground-borne vibrations.

It is also probable that with the increased attention to environmental issues over recent years, and the care taken by piling contractors, engineers and local authorities to minimise nuisance, records from the 1970s would give a misleading picture. In any event they were even then not representative of piling works as a whole. It is thought that more than 20,000 piling projects were completed in the UK during the period covered by the 150 case studies. While it is probable that other vibration studies were carried out in that time, but not made available to CIRIA, the proportion of piling projects warranting vibration studies would still have been very small. By their very nature, most of the 150 records were in situations where adverse comments or damage had been expected.

It would be inappropriate and potentially misleading to give other than general descriptions from the information of these case records. It remains the situation that there is no UK Standard or Code of Practice about acceptable limits for ground-borne vibrations in relation to building damage.

Nevertheless, the case records do provide useful information about the magnitude of pile-induced vibrations and about their attenuation with distance from the source.

6.1 TYPES OF PROJECT AND STRUCTURE

As might be expected, the vibration measurements were made for relatively close structures. Figure 12 is a cumulative frequency distribution of the distances of the monitored structures from the piling or ground improvement operations. 70% of the structures were within 20 m of the piling; and 47% were within 10 m.

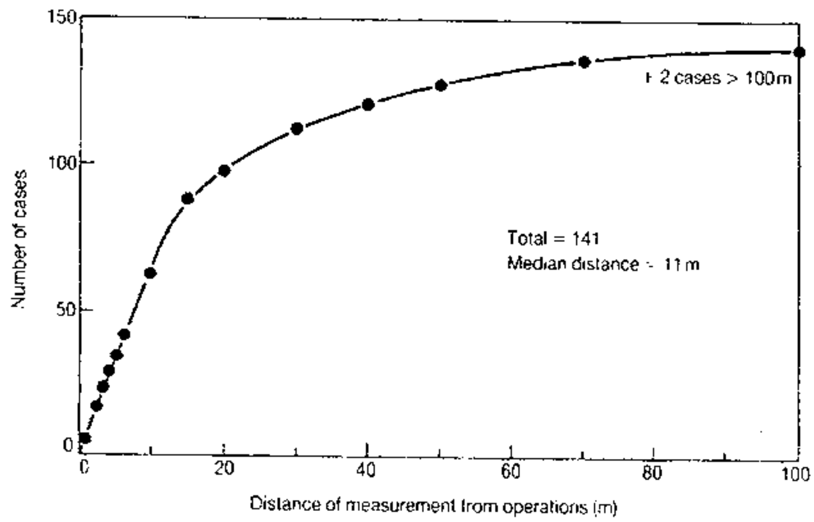


Figure 12 *Distribution of distances between piling operations and monitored structures*

Scant information was given about the structures being monitored, insufficient even to classify the structure type by the Swiss Standard (SN 640312:1978) given in Table 10 and certainly not in the detail needed for the ISO DIS 4866:1986 given in Tables 11 and 12.

Using the categories of DIN 4150: Part 3 1986, the German Standard, the case records can be grouped broadly as given in Table 14.

While the category (B) of dwellings and similar buildings is clear, it is not so obvious if office premises should be classified as commercial buildings. It is of interest that 18 of the 123 monitored structures were not buildings; as well as underground services, gas pipes etc., the other properties considered to be at risk included a railway line, an earth embankment, an elevated walkway and a steelworks chimney. A sixth of the monitored buildings were described in terms that would suggest they would be classified in Group C (e.g. churches, 'old' buildings in the sense of intrinsic value etc.). Nevertheless it could well be that while certain of the industrial or even public buildings (which for Table 14 have been considered as Class A) would be considered as having 'particular sensitivity to vibration' e.g. refinery buildings, town hall etc., the distinction between a structure and its use (occupants, contents etc.) may not always be obvious.

The ground engineering works represented in the case records are summarised in Table 15. Sixty projects involved driving bearing piles; 15 were bored piling works. 36 sheet-pile operations were monitored; this figure includes sheet piling installed or extracted with vibro-drivers. The other records are for ground improvement works of vibro-compaction (10 No.) and dynamic compaction (1 No.).

Table 14 Structures monitored in the case records (groups A, B and C are those used in DIN:4150 Part 3:1986)

	Category of Structure	Number
A	Buildings used for commercial purposes, industrial buildings and buildings of similar design	42
B	Dwellings and buildings of similar design and/or use	36
C	Structures that, because of their particular sensitivity to vibration do not correspond to those listed above and are of great intrinsic value (e.g. buildings that are under a preservation order)	18
D	Public utility services	10
E	Other types of structure	8
F	Unspecified structures	9
	Total	123

Table 15 Types of work reported in the case records

Operation	Number
Bearing piles – driven	44
– bored	15
Vibro-driving of pre-cast piles or casings	16
Sheet piling	36
Vibro-compaction	10
Dynamic Compaction	1
Total	122

In 24 cases, there are reports of measuring the ambient or background vibration levels at the structure and some measurements of particular but everyday events were also taken. Three quarters of these background levels were reported as being peak particle velocities of less than 1 mm/s – and more than half were less than 0.5 mm/s. Higher *ppv* values were associated with events such as the passage of trains (*ppv* up to 7 mm/s) and traffic (6 mm/s), i.e. routine occurrences for that situation. Specific *ppv* measurements were also made at or near perceived sensitive locations: examples, given here for illustrative purposes only, were:

- stamping on a computer room floor, 2 mm/s
- footsteps a few metres away from a home, 0.5 mm/s
- footfalls across a wooden floor, 3 mm/s
- bellringing (measured in the belfry), 2.2 mm/s.

These normal-situation *ppv* values are often of the same order of magnitude as those induced by piling operations, perhaps even greater. Even though the vibrations caused by piling works are additional, both in the sense that they are not normally present and that they may augment ambient vibrations, demonstrating that they may be of comparable intensity may help in public relations efforts. Measurements of ambient levels should always be part of the vibration study of a site.

6.2 VIBRATION MEASUREMENTS REPORTED IN THE CASE RECORDS

One of the difficulties in comparing vibration study data is that different types of measurement are made. In these records, nearly all quote *ppv* values (which can be derived from direct measurements of amplitude or displacement) or acceleration. But the values quoted are a mixture of the different possible vectors: some are horizontal, some are vertical, and some are resultants (either simulated or instantaneous). For those given as horizontal or vertical values on their own it is not clear whether the measurements were only taken in that direction.

Figure 13 shows the distribution of the reported *ppv* values (regardless of their direction). In 50% of the cases, the reported *ppv* was less than 4 mm/s; 90% were less than 20 mm/s.

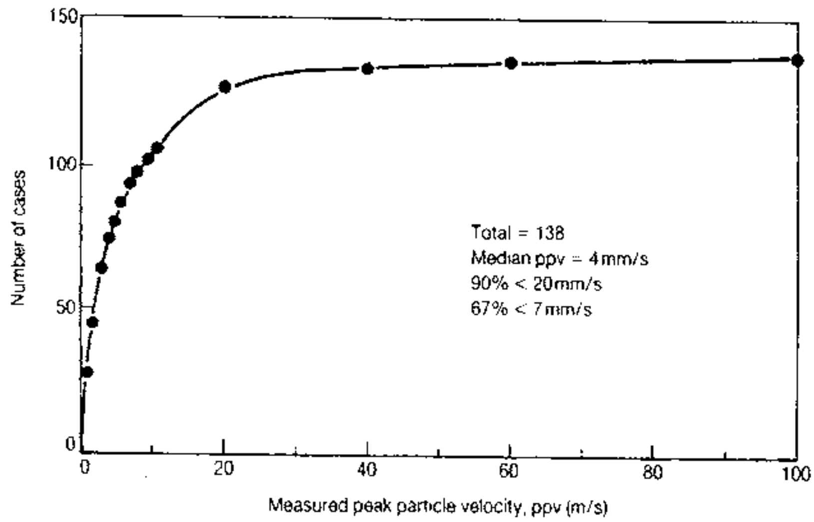


Figure 13 *Distribution of reported ppv values*

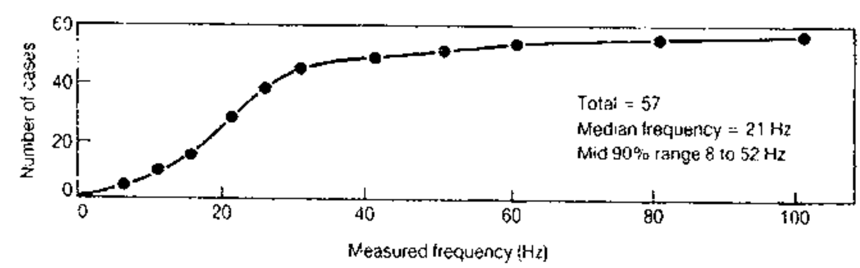


Figure 14 *Distribution of reported frequences at measurement points*

Only 57 of the case records reported the vibration frequency at the measuring point. The distribution of these values is shown in Figure 14. The median frequency is 20 Hz and over 70% of the results are in the range 5 to 30 Hz.

There is also wide variety in the positions chosen for measurement, and it is not always clear from the case records if measurements on a structure are at or near ground level and in direct contact with the structure's foundation.

6.3 PUBLIC RESPONSES

Over 80% of the case records note that the piling or ground improvement works were annoying to the public i.e. adverse comments were received. As 12% of all the sites were rural, there can be few projects in built-up areas which would not occasion some form of disruption or annoyance.

In many cases, the complaints were dealt with through explanations and by assurances. In others, modifications of plant and piling method were made to improve matters.

The case records do not provide information by which to judge relative degrees of annoyance. In the context of traffic noise and vibration, Watts (1990) presents some analyses of people's responses, a method which could be used for ground engineering projects. A major difference, however, is that in most cases traffic is a usual part of the environment: piling represents changed conditions and is the advent of the longer disruption which the construction works may bring.

The case records overwhelmingly demonstrate that the toleration of the public to piling-induced vibrations is markedly increased where good public relations have been fostered by the developers. Notification in advance, information procedures and practical assurances to those potentially affected are worthwhile. These have to be planned and co-ordinated with careful surveys of pre-existing conditions and preparations for appropriate preventative measures.

6.4 REPORTS OF DAMAGE

Assessing the reports in the case records about the effects on structures presented several difficulties. Table 16 gives the distribution of reports about damage.

Table 16 Incidence of reports about damage

Report	Number	(%)
No damage	84	(66)
Damage probably caused by vibration	20	(16)
Damage thought to be from other causes	5	(4)
Alleged damage	16	(13)
Totals	125	(100)

For those cases where damage was known to have happened, it was not always possible to be sure that it was the consequence of the piling vibrations i.e. there might have been some other cause, perhaps but not necessarily, associated with the piling (see Section 5.1).

In other cases, it was only possible to classify the report as being of alleged damage i.e. not only was there no proof of a direct link with the piling operations, but also most of the relevant vibration levels were low and suggest that they would have been unlikely to cause the damage. The uncertainties surrounding these cases of alleged damage are such that it is not worthwhile to examine them further. It is noteworthy, however, that about one sixth of the cases are in this category and vibration measurements were taken in half of these. Even if unfounded or unprovable, allegations of damage are a burden on those with responsibility for the piling works who have to deal with these claims. The cases of alleged damage and those thought to be from causes other than piling-induced vibration of the structure were as common as those where vibrations were the probable cause of the damage (20%).

It is possible to be more positive about the vibrations experienced in the 66% of all the cases where no damage resulted (see Section 6.4.1) and the 6% where damage could probably be attributed to the vibrations from the works (see Section 6.4.2). Table 17 shows the numbers of cases of different operations and whether or not damage resulted. Overall, the ratio of cases (damage: no damage) was about a quarter, the highest proportion being for vibro-driving (9:19) and the least for bored piling.

Table 17 Different types of work and reports about damage

Type of operation	Numbers of cases reporting		
	No damage	Damage	Totals
Driven bearing piles	31	5	36
Sheet piling (impact driving)	17	4	21
Bored piling	10	0	10
Vibro-driving and resonant piling	19	9	28
Vibro-compaction	7	2	9
Totals	84	20	104

6.4.1 No damage to structures

No damage was reported in 83 (66%) of the case records, 75 of which gave vibration measurements. Table 18 groups the ranges of recorded *ppv* values by pile type or operation. 77% of the *ppv* values were less than 10 mm/s and 36% less than 2 mm/s. For the structures to which these refer, the vibration levels would not be expected to cause damage (e.g. by the guide levels of DIN 4150: Pt 3: 1986).

Table 18 Peak particle velocities recorded in operations resulting in no damage to structures

Type of operation	Numbers of cases with reported ppv values of:			
	<2 mm/s	2 to 10 mm/s	>10 mm/s	Totals
Driven bearing piles	10	13	7	30
Sheet piling	6	6	5	17
Bored (tripod) piles	1	5	2	8
Vibro-driving	8	3	2	13
Vibro-compaction	2	4	1	7
Totals	27	31	17	75

Of the 23% with *ppv* measurements of more than 10 mm/s, about half of these relate to relatively stiff structures, for which problems would not be expected. The other (nine) cases are examples which underline the point made in published guidelines that damage is not inevitable if limits are exceeded.

6.4.2 Damage to structures

Vibration measurements were made in 15 of the 21 cases of reported damage, but they are a mixture of ground and structure measurements, single component (horizontal or vertical) and resultant peak particle velocities. The structures were of different types and conditions. It is also not possible to categorise the resulting damage from the descriptions given either in terms of its severity or in relation to the type of structure.

These uncertainties are such that it would be potentially misleading to detail individual records. It is certainly not possible to draw general conclusions from such scant data. Nevertheless, because these cases are of particular interest, it is worth summarising certain observations.

1. In only one case was there structural cracking (of the walls of a bungalow and of its drains). High-modulus sheet piling was being driven by a diesel hammer some 5 m from the building. Resultant *ppv* values were 40 and 14 mm/s on the ground and on the building respectively.

2. In eleven buildings cracking was observed, from hairline cracks to more open ones in finishes, plaster and blockwork. These would probably all be in Categories 0 to 2 (Aesthetic damage) of BRE Digest 251 (Table 7).

3. Three of the cases related to burst water and gas mains and two were embankments which were disturbed.

4. In three other cases, settlement was reported. As these structures were built on granular soils, the piling may have densified the ground causing the settlement. Strictly, therefore, these cases would not then represent damage because of vibrations in the structure. On the other hand, if the piling vibrations caused the settlement, it is not inappropriate to include these causes here (as distinct from the five cases (Table 16) where the damage was attributed to lateral ground movements).

Another cause of damage to buildings that could have occurred in the record cases (and cannot be eliminated as a possibility) is localised loss of ground support at excavations, whether or not associated with piling or building operations. Piling vibrations can trigger further ground movements or cause instability of buildings on sloping ground where safety factors are low.

In the context of the extensive search for case records (and the small number obtained) there was a very low incidence of serious damage and relatively little aesthetic damage to buildings. It is probable that even more care has been taken in recent years to prevent damage occurring.

6.5 ATTENUATION OF PILING VIBRATIONS

A number of the case records included results of vibration measurements at differing distances from the piling operations. These are plotted in Figures A.1 to A.15 of Appendix A as peak particle velocities against plan distance, both to log scales. In these examples the *ppv* values are simulated resultants. The measurements were made on the ground in ten cases and on structures for the other four. Where known, details are given in each figure of pile type, driving system, frequency of driver or hammer, and the type of ground.

In most cases there appear to be reasonably linear relations between *ppv*, v, and plan distance, r, of the form:

$$v = \frac{a}{r^x} \qquad \text{... (6.1)}$$

Lines through the data points have been drawn by eye and the values of the parameters, a and x, are given in the tables of Appendix 1. Where the driving system is known i.e. mass and fall of a drop hammer, make and model of a diesel or air hammer or vibro-driver, the nominal driving energy per blow, E has been used to derive the parameter, k, in the expression

$$v = k\frac{\sqrt{E}}{r^x} \qquad \text{... (6.2)}$$

where v is in mm/s
E is in Joules
and r is in m.

These values of E and k are also given in Appendix 1. See Section 6.7 for comments about energy values.

An alternative representation of these attenuation plots is that used by Wiss (1967) and, in relation to vibro-compaction and dynamic compaction, by Greenwood and Kirsch (1983). Peak particle velocity is plotted in Figure 15 against $(E)^{1/2}/r$ for twelve sets of results using the lines drawn on the individual log v – log r plots of Figures A.1 to A.15. Transformation of Equation 6.2 gives:

$$v = b\left(\sqrt{E}/r\right)^x \quad \text{where } b = \frac{k}{E^{(x-1)/2}} \qquad \text{... (6.3)}$$

For those case records where sufficient information is given, individual measurements of peak particle velocity are plotted against scaled distance ($E^{1/2}/r$) in Figure 16. It should be noted that many of the *ppv* values in this plot are peak values in one direction (i.e. horizontal or vertical) rather than simulated resultants. Distinction is made between measurements on the ground and those on the structure – usually at its base. Where the measurement point was described as being on a wall, a column, a facade or similar, the plotted points are identified separately.

Comparison of Figures 15 and 16 shows that the ground measurements tend to be higher than those on the structure base. Greenwood and Kirsch (1983) note this in their comparison of vibration – scaled distance measurements associated with vibro-compaction. Superimposed on these two figures are the upper-bound envelopes (A, B and C) of the data points on the Greenwood and Kirsch plots. It can be seen that:

1. Both the attenuation relations (Figure 15) and the single-point measurements of the piling records lie below envelope A (that for vibro-compaction: ground measurements).
2. All the measurements taken on structures, except the more distant measurements on case 133, lie below envelope B (that for vibro-compaction: structure measurements).
3. The Greenwood and Kirsch envelope for dynamic compaction (envelope C) lies through the single-point piling measurements and below most of the attenuation relations. One case record of dynamic compaction, not plotted here, gave measurements of *ppv* and scaled distance very close to envelope C.
4. The results from the vibro-driving of piles and sheet piles lie below envelope B apart from one low *ppv* value and two points, where the measurements were noted as being on a metal desk and on a column.

There are insufficient data for distinctions between different pile types, different driving systems or different ground conditions. In deriving scaled distances, the notional driving energy is used. In practice, driving energy varies during the drive and also depends on the efficiency of the system, its state of maintenance and the ground's response (see, for example, Mallard and Bastow, 1979).

For piling, whether by vibro-driver, drop or diesel hammer, in the range of scaled distances, $E^{1/2}/r$, from 1 to about 30 $J^{1/2}$/m, the following preliminary guidelines for attenuation with distance could be suggested:

(a) Conservative estimate for ground measurements in terms of simulated peak particle velocity, $v = 1.5\ (E)^{1/2}/r$. Attewell and Farmer (1973) proposed guideline values for peak *vertical* components of particle velocity of $v = 1.5\ (E)^{1/2}/r$.

(b) Conservative estimate for measurements at the base of foundation of a structure, $v = 0.2\ (E^{1/2}/r)^{1.54}$ i.e. envelope B.

These guidelines do not apply for scaled distances closer than about $E^{1/2}/r = 30\ J^{1/2}$/m, which for piling operations usually implies less than about 5m. At these close plan distances the wave propagation pattern is more complex than further afield, and the plan (horizontal) distance, r, may be an inappropriate parameter as the pile is driven to depth. The seismic distance from say the pile toe to the measurement point may be more suitable. Attewell, Selby and Uromeihy (1991) suggest that the simple attenuation relations should not be used at standoff (i.e. plan) distances of less than 10m. At closer distances, amplitudes and particle velocities are often lower, but increase to peak at about 10m because of the overlap of surface waves from pile movement at the surface with waves emanating from the pile toe.

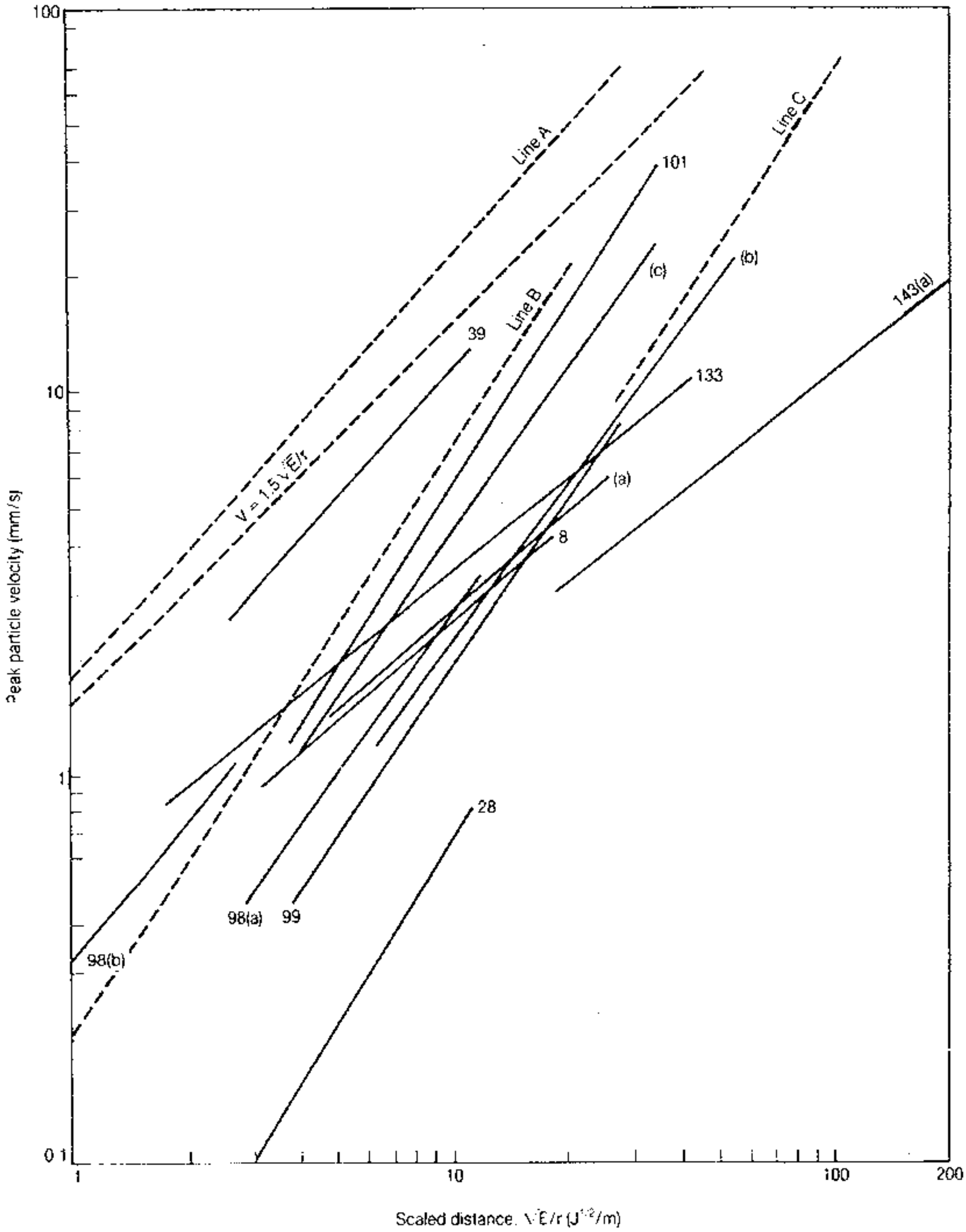
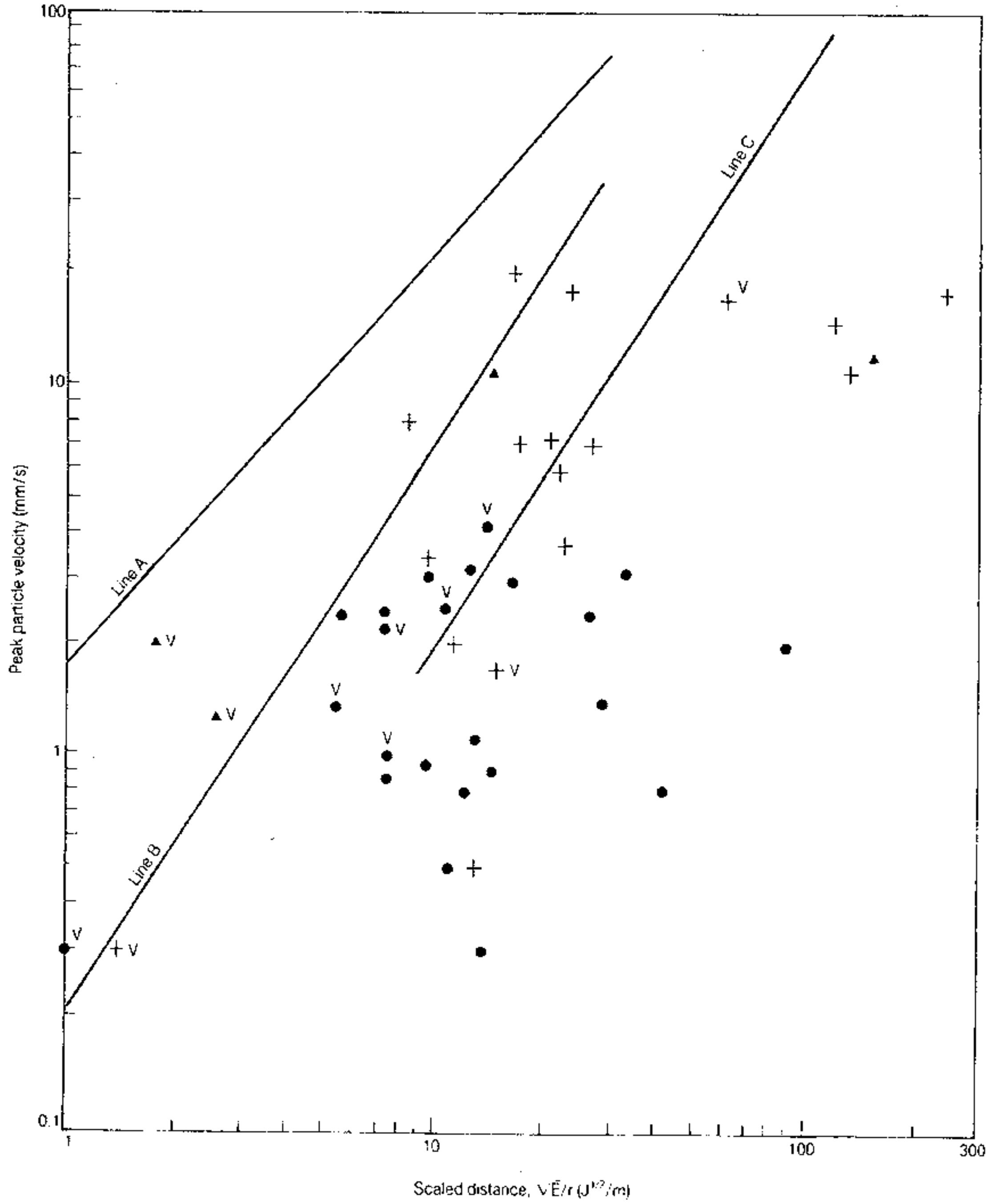


Figure 15 *Peak particle velocity and scaled distance: attenuation studies*



Key
+ measurements on ground
• measurements on base of structure
▲ measurements on columns, walls or elsewhere above ground
v vibrating hammers or vibro-driving
Line A envelope for vibro-compaction - ground measurements
Line B envelope for vibro-compaction - structure measurements
Line C envelope for dynamic compaction
Lines A, B and C taken from data of Greenwood and Kirsch (1983)

Figure 16 *Peak particle velocity and scaled distance individual points*

6.6 THE EFFECT OF PILING ENERGY

The case records provide insufficient evidence to distinguish between different types of driving system or the amount of energy applied. In Section 6.5, the notional energy is used as a means of normalising the data in the scaled distance term.

As an *aide-memoire* because energy values are quoted in different units, the calculation of values in Joules is given here for different types of driving systems

(1) Drop hammers

Mass of hammer	=	m tonnes
Fall	=	d metres
Energy per blow E	=	$(1000m)\ 9.807d$ kN-m
	=	$9807md$ Joules

(2) Diesel hammers

Rated energy per blow	=	(R.E.) kg-m
Energy per blow	=	(R.E.) x 9.807 Joules

(3) Vibro-drivers

Power supply	=	W kVA (= kilowatt = kiloJoules/s)
Rated frequency	=	f Hertz (cycles/s)
Energy per cycle	=	$1000W/f$ Joules.

6.7 EFFECTS OF GROUND CONDITIONS

6.7.1 Level of vibration

The ground conditions affect the propagation of the piling-induced vibrations in several ways (see Section 2). Stiff soils and intact rocks transmit vibrations more readily than softer, more compressible materials. Thus the presence of harder layers, through or into which the piling penetrates, can give rise to the highest vibration intensities. Often, of course, the piles are taken down to bear on a harder layer and the foundations of nearby structures may either also be piled to the same layer or, if shallower, they may rest on a higher stiff layer. Thus the vibrations transmitted through the ground and measured on the structure may be influenced by the stratigraphy of the site. There are relatively few places in U.K. where there are deep, uniform deposits. The case records and the attenuation data (at sites where more is known about the ground conditions) include various combinations of soft ground and weak rocks. On some sites the piling itself alters the stiffness of the ground e.g. driven large-displacement piles.

Many of the particle velocities measured in the case records reflect the effect of stiffer materials at depth or of adjacent structures founded on these stiffer materials. Figure 17 represents *ppv* values measured either at the ground surface or on structures plotted at varying distances, for softer and stiffer ground conditions. These data, extracted from the case studies, suggest that generally higher particle velocities can be expected in stiff or compact ground. The wide scatter demonstrates, however, that ground stiffness is not the sole determining factor.

6.7.2 Foundation settlements and movements

Localised movements in adjacent properties are frequently suggested in the case records. The cause of these movements is not ascribed with certainty but it is suspected that ground settlement (or other movement) could be a contributory factor. Usually there is little or no knowledge of the ground conditions beneath property adjacent to piling operations. It is possible that, in a number of cases, granular fills or natural soils settled differentially as a result of dynamic effects from piling, thus causing damage. The lack of substantiating information means that it was not possible to examine this further.

6.8 EFFECT OF PILING METHOD

The piling method includes the type of pile and how it is driven. Sheet piles, for example, may be driven by drop or diesel hammer or by vibro-drivers. The bearing piles of the case records were mainly driven by drop or diesel hammers, but included pre-cast piles, steel H- and tubular piles, shell piles and Franki-type (expanded base) driven, cast-in-place piles. Vibro-drivers and extractors were used for casing as well as for sheet piling.

Heckman and Hagerty (1978) suggest for different types and sizes of pile that the vibrations induced can be related to the pile impedance. The impedance, I, of the pile (Poulos and Davis, 1980) is:

$$I = \rho v_m A$$

where ρ = *mass density of the pile material* ... (6.4)
v_m = *longitudinal wave velocity in the pile material*
A = *cross-sectional area of the pile.*

It should be noted that the dimensions of impedance are mass-time/length and follow from $\rho = \gamma/g$ where γ is the unit weight.

For higher impedance piles, the wave energy transmission to the ground is less and Heckman and Hagerty suggest that the proportionality factor, k, reduces as impedance increases. Here k is given by

$$v = \frac{k\sqrt{E}}{r}$$

... (6.5)
where v = *peak particle velocity (mm/s)*
E = *driving energy per blow (Joules/blow)*
and r = *distance (m).*

Relatively few of the case records give sufficient information to calculate impedance values of the piles, but for those in which the pile type is known, values were estimated assuming typical values of the longitudinal wave velocity, v_m, i.e. for steel, 5000 m/s and for concrete 3000 m/s.

The k-values of these few case records generally lie well below the curve given by Heckman and Hagerty – those above the curve were from the same site in which the piling was driven into mudstone. Amongst other factors, the wide range in ground conditions and uncertainties about energy levels and impedance make it impossible to draw any general conclusions.

6.8.1 Driven bearing piles

When assessing the effects of driven bearing piles, a distinction is required on the basis of method of installation. For example, small-displacement piles such as H sections are likely to create maximum values of *ppv* when they encounter hard ground, including rock. Large-displacement piles may generate more uniform particle velocities throughout driving. Bottom-driven piles give rise to less noise than their top-driven counterparts but the position with regard to vibrations is not clearly defined.

In broad terms higher *ppv* values are apparent from the cases of driven piles. This fact is attributable to the high source energy required for installation and the need to drive to a competent stiff founding layer in a large proportion of cases.

The effect of pile penetration on *ppv* generated is likely to depend on the type of soil profile and the continuity of that profile. Two of the cases of cast-in-place piles report greatest vibrations during formation of the bulb whereas one of the H-pile records reports that the highest vibration intensities were at shallow depth.

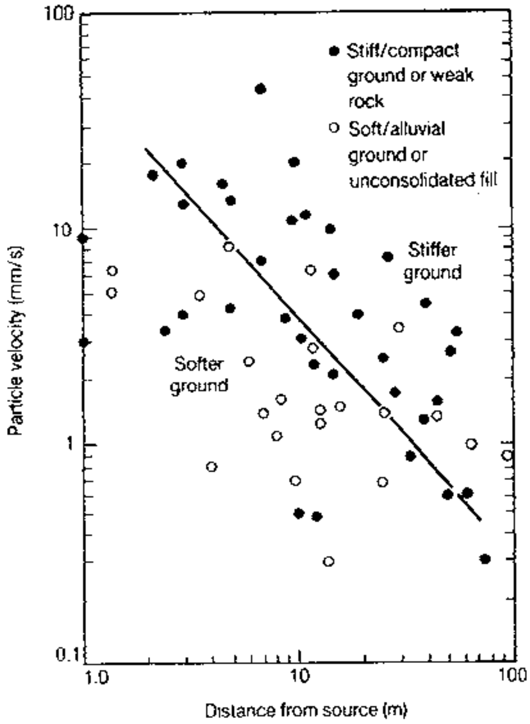


Figure 17 *Influence of ground conditions on measured peak particle velocities*

6.8.2 Sheet piles

Installation of sheet piles produces relatively small soil displacements and might be expected to require less driving energy than displacement piles. Most sheet piling is for support works often in urban areas and particularly close to existing structures. In addition, sheet piles are likely to be driven to a depth dictated by stability criteria, not by bearing characteristics, and may often be driven through stiff or dense ground.

Martin (1980) noted that vertical components of particle velocity can be much greater than other components. In some cases it will be necessary to assess separately the effect of vertical particle velocities.

6.8.3 Percussion bored piles

As might be expected the relatively few case records confirm, in general, that bored piling generates less intense particle velocities than driven piling.

Depending on the method of boring and how carefully it is controlled there can be a loss of ground into the borehole excavation; ground movements may be generated in addition to any effects of vibrations.

Significant vibrations can be produced whilst driving or extracting casings, during chiselling through obstructions or rock, and by tamping of concrete. Larger diameter bored pile operations may use casing oscillators to advance the casing (see Section 6.8.5).

6.8.4 Augered bored piles

The lack of reported annoyance or damage suggests that auger techniques are unlikely to generate significant vibrations unless associated with installation of casing or with chiselling operations. The recent more extensive use of continuous flight auger piles, which do not need separate casing,is partly due to their being relatively quiet and vibration-free (but see the comment in Section 6.8.3 about possible loss of ground).

6.8.5 Vibro-driving and resonant piling

Vibro-drivers are clamped to sheet piles or casing and a substantial driving (or extracting) force is created via two contra-rotating eccentric weights. Their operating frequency is typically 25 to 40 Hz. They are only effective in soils, such as loose to medium dense cohesionless materials, soft alluvium and weathered chalk, which can be displaced.

The resonant driver is used to vibrate an H pile or casing at its resonant 'end-to-end' frequency which generates large linear oscillations of the pile at frequencies of 70 to 125 Hz. These large vertical vibrations temporarily over-ride the frictional resistance of the soil surrounding the pile; thus allowing penetration by self-weight.

A high proportion of the cases reported damage and annoyance. The reason is probably due to the forced nature of the frequency of the driver which is, by necessity, close to the characteristic frequency at which soils are likely to vibrate intensely and to the resonant frequencies of structures and floors. Resonances are likely during run-up to and run-down from operating frequency but are possible at operating frequencies. Table 19 summarises typical values of maximum ground surface response for various soils.

Table 19 Typical frequencies of maximum ground surface response to impact and vibratory pile driving

Type of ground	Typical frequency (Hz)	Rayleigh Wave propagation velocity (m/s)
Very soft saturated silts and clays	5 to 20	75 to 100
Soft clays and loose sands	10 to 25	90 to 250
Compact sand, gravel and stiff clays	15 to 40	> 200

Because vibration is continuous, human tolerance to levels generated by this activity is low. Steady-state vibrations are more likely to produce a resonant response in building components, particularly panels. Wiss (1967) suggests that the safe level for continuous vibration might be between one half and one fifth of those produced by transient excitation.

6.8.6 Vibro-compaction

Vibro-compaction is used to improve the properties of poor ground. A vibroflot is introduced into the ground and is used to compact non-cohesive materials or to displace cohesive soils which are then replaced by stone columns. An eccentric rotor produces essentially *horizontal* vibrations to a poker-shaped probe which penetrates the ground under self-weight. Operating frequencies are usually about 30 Hz.

Greenwood and Kirsch (1983) give measured *ppv* values for various scaled energy/distance measurements recorded on the ground and on the foundations of structures (see Section 6.5).

Although two cases of damage were reported (and seven without damage) these could have been situations where vibration-induced settlement was a contributory factor.

6.9 MITIGATING VIBRATION LEVELS

Where piling vibrations are judged to be unacceptably annoying or the risk of damage is considered to be too high, there are several options available to reduce the effects.

Alternative foundation. If the problem is identified sufficiently early in the design stage, then various possible alternatives can be assessed and the most appropriate adopted. Solutions include modifications to foundations and structures to spread loads or selection of ground improvement processes. To make radical changes to the foundation design after piling has started would be extremely costly and delaying: every effort should be made to avoid this being necessary.

Alternative pile type or process. Section 6.8 outlines the main differences between pile methods with respect to vibration. The aim should be to reduce the particle vibration magnitude to an acceptable level, as it will not usually be possible to eliminate vibration. In many cases a compromise will be necessary between the suitability of piles for the ground conditions and limiting vibrations.

Reduction in pile cross-section will increase ease of installation and allow a reduced driving energy, but this may increase total cost and time on site if more or longer piles are used.

As explained in Section 6.8, Heckman and Hagerty (1978) relate the pile attenuation factor (k in Equation 6.5) to the pile impedance, suggesting that k reduces as impedance increases i.e. more energy is absorbed by the pile. Site-specific correlations are likely to be needed using different pile sizes or types in carefully controlled and measured trials.

Reduction of driving energy. If *ppv* is proportional to the square root of energy and the reciprocal of the distance from source, a lower driving energy is only likely to have a significant effect on *ppv* at closer distances. Figure A.11 (case 98) illustrates the small difference in *ppv* values between a lighter air hammer and a more powerful diesel hammer. It may be noted that the ratio of *ppv* values is less than 1.5 but the ratio of hammer energies is about 6 (or about 2.5 in terms of the square root of energy). Adjustments to the frequency of blows may be beneficial as would a reduction in the number of rigs operating simultaneously nearby. Another alternative might be to use a heavier hammer and smaller drop. It may be that reducing energy per blow would be more worthwhile at close distances but not further away.

Adjustment of piling method. Where high *ppv* values would be expected (e.g. when penetrating a particularly stiff layer) preboring is one method to avoid them. Several case histories report this option to be successful in limiting vibrations in the early part of the drive, particularly where a stiff surface crust is present. It is also reported that preboring before vibro-compaction reduces vibrations.

The dangers of using operating frequencies close to the frequencies that are likely to cause the ground to vibrate at its maximum level have been mentioned previously (Sections 2.6 and 6.8). O'Neill (1971) suggests that higher penetration rates, and thus lower vibration levels, might be obtained at driving frequencies which are 50 percent above the level at which the ground characteristically vibrates. This is produced because vibrations from the soil and pile are not in phase. In practice, frequencies would have to be continuously varied to produce optimum conditions.

Bottom-driving of piles, although less efficient than top driving, is considered quieter and may produce less severe vibrations. This, to some extent, remains a matter of opinion. The method is employed in the BSP permanently cased pile and in some types of driven cast-in-place piles. One case record reports bottom-driven permanently-cased piles successfully installed without damage in a very sensitive location, next to old buildings. Jetting in granular soils may also be helpful in assisting penetration of driven piles.

Isolation of vibrations. Several of the case records and various authors suggest that vibrations can be isolated from adjacent structures by trench excavations. In addition to reducing the general level of particle velocities, the use of trenches has been reported as being particularly useful to sever a stratum where preferential transmission of vibrations occurs. The location and depth of a trench are important in order to optimise its value. It should be close either to the source or the receiver, but care is necessary to ensure that static stability or safety will not be jeopardised by introducing a trench. In general, this technique has a limited scope of application, because the trench usually has to be several metres deep.

Mudding in. Temporary casing can be more easily inserted if a small quantity of bentonite is introduced (e.g. by auger) into the hole beforehand; alternatively, excavation could proceed under bentonite where deeper casing is needed.

Continuous monitoring is considered to be the most important single action that will help to reduce the instances of extensive damage from piling. Often just as important is that it should provide conclusive evidence of actual vibration effects. Physical measurements and photographic records, before, during and subsequent to piling can be used to demonstrate whether the effects are damaging and the extent of that damage. More critically, monitoring provides an early warning that the *ppv* values generated are at potentially damaging levels or, if damage occurs, it can identify levels that are not to be exceeded. Alternatively, a predetermined limit can be specified and operations monitored accordingly.

A further benefit of monitoring is in terms of good public relations. By demonstration to local occupants that professional care is being exercised, levels of concern and, therefore, complaint may be reduced. To be of greatest value continuous monitoring is best carried out by an independent vibration consultant.

7 Recommended procedures

Careful assessment and planning by developers, regulators and engineers will allow sites with potential problems to be identified and the risks of annoyance or damage to be avoided or minimised. Sections 2 to 6 provide information on how vibrations are transmitted and how particle velocities attenuate away from their source. Guidance is given on assessment of potential for annoyance and damage, and information is provided which allows judgements to be made on likely levels of peak particle velocity. Ways of mitigating the effects of vibrations are also identified. This Section draws together this experience and recommends sequences of procedures that can form the basis of most site assessments relating to vibrations associated with piling and allied processes.

7.1 CONTRACTUAL ARRANGEMENTS

Local authorities have legal powers to set predetermined limits of acceptable vibration levels. To do so requires a detailed knowledge of the ground and of specific structural responses to different types of pile and pile installation. In many situations, therefore, there will be insufficient data from past experience to be certain about vibration levels and their effects. Ideally, the limits for vibration (and noise) should be chosen after full consultation with the promoter's engineer and contractor's piling representative and taking into account the specific circumstances of the site.

Usually in civil engineering, the foundation piling and sometimes the sheet-piling is sub-let to a specialist sub-contractor by the main contractor. The piling specialist is unlikely to be party to the initial planning and may not be able to influence major policy decisions already taken. The Engineer should ensure these decisions have the benefit of advice from piling-vibration experts in order that a realistic balance is achieved between construction methods and limits.

For building works, the piling sub-contractor may be nominated by the Architect on the advice of the structural engineer. If so, the piling specialist should be brought in to the consultation process of setting appropriate vibration limits.

7.2 SITE INVESTIGATIONS AND APPRAISAL

The ground investigation of the project site identifies the need for piled foundations, for ground support such as sheet-piling and for ground improvement, together with other alternatives thought practical. These recommendations have to be set in the context of the site itself, its surroundings and the constraints these impose on both detailed design and construction operations. Too often in the past, selection of piles has been based on factors associated solely with ground conditions and structural performance requirements.

The site investigation studies should identify potential construction restrictions and include an appraisal of the risks of damage to adjacent property. A desk study of available records should be carried out to locate utility services and the types of structures close to the site (preferably with foundation details). A site reconnaissance is essential to supplement and clarify information about the buildings, their function and general conditions and to select those where further structural surveying is desirable.

An assessment of the reported ground conditions should be made to identify ground likely to transmit particle velocities preferentially to adjacent services, foundations and structures (Sections 2.2, 2.3, 2.4).

Situations where vibrations have to be assessed very carefully include sites close to:

- hospitals
- nursing homes
- museums
- laboratories
- precision machine workshops
- sensitive plant or equipment
- historical monuments and ancient buildings
- housing and other buildings in poor condition
- tall buildings or other thin or tall structures
- brittle and ancient underground services
- residential estates.

In addition, the ground conditions relevant to the nearby structures have to be assessed for the susceptibility to:

- collapse or large settlement, where stiff or brittle materials are to be penetrated by driven piling
- vibration-induced settlement
- loss of ground into nearby excavations or borings.

7.3 ASSESSING POTENTIAL FOR ANNOYANCE AND DAMAGE

The assessment of the potential for annoyance or damage may be made either on the basis of risks associated with a chosen pile type or plant or by selecting pile types and installation methods which satisfy the imposed vibration limits.

A preliminary estimate can be obtained from the empirical relations given in Section 6.6.

By inspection of the attenuation plots from individual case histories in Figures A.1 to A.15 and Tables A.1 to A.3 it may be possible to choose a comparable site. Alternatively, Figure 17 may be used, depending on the general estimate of the strength of the ground.

Checks on particular pile types and hammers can be made by comparison with the predictions using Equation 2.8 with Figure 15 or using Equation 6.3 with Tables A.1 to A.3.

In general, the *ppv* values measured on building foundations are lower than those measured on adjacent ground. The case history data suggest that they might be about half (see also Figures 15 and 16). On the other hand, the vibrations of structural components, particularly of a tall building or of a suspended floor, can be magnified.

The degree of annoyance produced by vibrations depends on the activity and condition of persons affected, as well as vibration intensity. Section 4 discusses vibration intensities in terms of *ppv* and frequency. The likelihood of adverse comment heavily depends on the situation and usage of the building. 83% of the case records reported that complaints of annoyance had been received. It is likely that local residents will complain about any perceived piling vibrations as soon as they become easily noticeable, the more so if they have not been forewarned.

There is no UK standard or code giving guidelines for assessing the risks to buildings from ground-borne vibrations. BRE Digest 353, although referring to German, Swiss, Swedish standards and the draft ISO standard, points out that UK conditions and structures may differ from those on which these other guidelines are based. At present therefore the best guidance which can be given is to consider each situation separately. If it appears that a piling or ground engineering project is likely to generate vibrations approaching the intensities considered as a risk for adjacent structures (using the general limits given in those international documents or local knowledge), then a detailed study should be undertaken by specialists.

In most cases, by careful choice of method and control systems it should be possible to keep the vibration intensities to levels at which the risks of nuisance are considered acceptable. This can best be confirmed by field trials either in the initial stage of the piling contract or by a pre-contract investigation, for example combining it with trial piling.

Attenuation relationships can be developed for various energy inputs and, in some cases, variations in pile type or geometry can also be tested. Vibration levels can be measured in adjacent structures to determine safe distances and the structures monitored, particularly any existing cracks, to prevent damage. However, a careful structural assessment and crack survey will be required beforehand, and closely controlled monitoring during the piling or ground improvement operations may also be necessary.

Where there is insufficient time or finance to carry out separate field tests the initial production piles can be tested in the same manner.

7.4 PUBLIC RELATIONS

Fostering good public relations should be a pre-requisite for any piling or similar ground engineering project which could annoy or interfere with local occupants. The attitude of residents or occupiers of affected property is largely determined by their concern about possible damage to property, effect on their health and well-being, including psychological effects, and uncertainty about proper compensation for disturbance and damage. Alleviation of that concern can be brought about by a planned programme of advice and guidance on the duration, magnitude and effect of expected vibrations. The effort will benefit the developers and contractors if the public have a better understanding of the nature of the work and are thereby more prepared to tolerate occasional or minor discomfort.

Such a strategy has to be honest and sincere. Misleading or over-optimistic forecasts are likely to be counterproductive. Local residents should be informed of the expected length and nature of operations and their possible effects, and given assurance of the care being taken to reduce disturbance to a minimum, e.g. that operations and their effects will be professionally monitored and improved where necessary. Realistic notice should be given; in some cases it would be appropriate to hold a public meeting in order to explain what is being proposed and to answer questions. A simple and effective procedure for dealing with complaints should also be established.

Vibration measurements coupled with crack monitoring at the affected structures are usually helpful and tend to alleviate fears. Confidence will be increased if the monitoring is carried out by an independent vibration consultant. In some cases it can be demonstrated that the slamming of a door and other domestic incidents or a passing lorry causes a higher vibration intensity than the piling works.

Comments on the advantages of keeping the public informed, directly to individuals and through local committees, were received in 40 of the case histories. Of these, 31 considered that the information process was beneficial, 4 that it did not appear to help; but 5 took the view that it had detrimental effects.

7.5 MEASUREMENTS OF EFFECTS ON STRUCTURES

Before starting the construction operations, it is necessary to survey the relevant structures. The survey should include a detailed record of:

- existing cracks and their widths
- level and plumb survey, including damp-proof course
- measurements of tilting walls or bulges
- other existing damage including loose or broken tiles, pipes, gulleys, or plaster.

Photographic records and measurable tell-tale devices are also helpful to confirm alleged or actual damage.

7.6 MEASUREMENT OF VIBRATIONS

Guidance on measurement and evaluation of vibrations is given by Broch (1980), by ISO/DIS 4866 (Draft 1986) and by Skipp (1978).

Site measurements should be carried out by competent and experienced personnel so that representative and meaningful results are reported.

When considering the type and method of monitoring vibrations it is essential to evaluate the site conditions and piling effects three-dimensionally. Points of measurement should be identified as part of a preplanned strategy and should be located so as to provide information on ground attenuation and building response characteristics. The proper characterisation of building response may require simultaneous measurement at several locations. Measurements should be evaluated on site so that the measurement strategy can be improved and the peak vibration levels identified during the piling programme.

The three components representing vibration characteristics that are commonly measured or calculated are displacement amplitude, *ppv* and acceleration. These are measured as peak values in three orthogonal directions to produce the resultant vector sum (see Section 7.6.4). Conversions between the three parameters are made by electronic instruments using an integration process.

Guidance on methods and details of measurement and on the evaluation of data is provided in ISO/DIS 4866 (1986 Draft).

7.6.1 Measurement devices

In the broad field of mechanical vibration and shock measurement it is common to use accelerometers to record vibrations. For general use they are smaller than velocity transducers (geophones), and their frequency and dynamic ranges are wider. However, in the range of frequencies normally under consideration for piling, geophones will give more consistent direct results; and velocity measurement provides the best single-parameter response of both people and structures.

Geophones operate on the principle that a magnet and coil will be relatively displaced and the voltage generated will be proportional to the relative instantaneous velocity. Alternatively, voltage is generated in a coil wound on a magnet which is proportional to the change in flux produced as a gap varies between the magnet and an adjacent movable metallic object. Devices with a natural frequency of about 5Hz are favoured (Skipp, 1978).

Accelerometers operate on the principle that vibrations load a small mass and a preloaded ring or spring. The force on an adjacent piezoelectric element is proportional to the acceleration. Accelerometers are usually constructed to operate in either compression or shear. Measurements are confined to frequencies up to one third of their natural frequency.

Moving-coil accelerometers developed for oil exploration geophysics provide a robust sensor in the frequency range 2 to 200 Hz; some numerical integration has to be undertaken to gain velocity information.

7.6.2 Monitoring and calibration (see ISO 5348)

The method of mounting the measuring devices is critical. Poor mounting will result in unreliable and misleading results. Transducers should normally be fixed to record in three orthogonal axes. Their mass should not be greater than 10 percent of the element to which it is fixed. For measurements on buildings or foundations they can be fixed by adhesives or by magnets. Brackets should be avoided. It is preferable to fix three uniaxial sensors on three faces of a metal cube rigidly mounted by means of studs or quick-setting, high modulus resin (ISO/DIS 4866, 1986 draft). It is acceptable to glue or use magnetic attraction for transducer fixing in special circumstances. For internal measurements on horizontal surfaces use of double-sided tape should be avoided where possible. For ground measurements, geophones can be positioned using an integral spike or fixed to a plate with spikes at each corner. Alternatively they can be buried. The following environmental effects on transducers and leads should be considered:

- humidity
- temperature
- electromagnetic interference
- acoustic noise
- corrosive substances.

Calibration and system performance checks should be carried out regularly.

7.6.3 Measuring systems (see ISO 4865)

Measuring systems range from simple portable meters to dedicated sophisticated devices using filters, recorders, amplifiers and analysers. Filters are often incorporated into the measuring system so that separate vibration intensities can be obtained at various frequencies. These take the form of mechanical filters to limit frequency range, and thereby eliminate unwanted signals, or tunable sweep filters. They are mounted between an accelerometer and point of measurement. Vibration signals can be recorded on magnetic tape for analysis later or can be analysed on site using hard copy produced by chart level-recorders provided the frequency response is suitable. Analysers can assess various frequency bands simultaneously and provide instant graphical display of frequency spectra. Real-time analysers are particularly suited for analysis of short duration signals, such as transient vibrations. Analysed spectra can be stored or produced on hard copy.

Many local authorities and other organisations have sound-level meters which are used for measuring noise. While they are not ideal general purpose vibration meters, they may be economically attractive to those who have only occasional need for such measurements. Many of the meters used may be adapted for vibration recording by substituting an accelerometer for the microphone. The further addition of an integrator allows measurements of velocity and displacement to be made. Frequency analysis can be carried out by fitting a band-pass filter. A portable level-recorder will enable hard copy to be produced, otherwise a peak-hold facility is required. Limitations on the frequency and dynamic ranges exist and many sound level meters are not sensitive below 10-25 Hz. Modification of such meters is often unsuccessful, however, and leads to inaccurate measurements. The use of sound meters should be confined to steady-state vibration: their use for impulsive piling is open to question and the results can be unreliable.

7.6.4 Velocity measurement

There are several possible definitions of peak particle velocity and the criteria and guidelines about the effects of vibration often involve different definitions. Thus DIN 4150: Pt 3: 1986 refers to the maximum of any of the *x*, *y* or *z* axes; the Swiss Standard SN640312:1978 uses the instantaneous peak resultant (which BRE Digest 353 refers to as the true peak resultant velocity); other guidelines may stipulate only the vertical component. In this Report and in other studies, particularly of scaled distance attenuation, it has been convenient to calculate a simulated peak resultant – the vector sum of three orthogonal maxima which are not necessarily coincident in time. The other term for the simulated resultant is SRSS (square root, sum of squares).

BRE Digest 353 recommends that the true peak resultant *ppv* should be determined, i.e. that all three components are measured simultaneously. Time histories of each response should be recorded so that the SRSS resultant can also be calculated. In this way *ppv* values can be determined by each definition.

Definitions of peak particle velocity (ppv)

$$\textit{SRSS (simulated resultant)} = v_{SRSS} = \sqrt{v_{x(max)}^2 + v_{y(max)}^2 + v_{z(max)}^2}$$

$$\textit{Uni-directional peak} = v_{max(x,y,\ or\ z)} = v_{x(max)}\ or\ v_{y(max)}\ or\ v_{z(max)}$$

$$\textit{Instantaneous (true) resultant} = v_{max(t)} = \sqrt{v_{x(t)}^2 + v_{y(t)}^2 + v_{z(t)}^2}$$

In many of the case records, the *ppv* reported was not defined so it is not known if they were uni-directional, true peak or simulated resultant values.

In particular circumstances e.g. on a structural component, it may be appropriate to measure uni-directional velocities. Usually this will be for supplementary parts of a vibration study.

7.6.5 Measurement procedure

Measurements are usually taken either to define ground attenuation characteristics or to identify building response. Where the purpose is to monitor buildings the preferred location is at the foundation, typically being at a point low on a main load-bearing external wall on the ground floor. In order to identify any amplification within the building it may be necessary to carry out simultaneous measurements at several points within the building. ISO/DIS 4866 (1986) recommends that where a building is higher than four storeys (approximately 12 m) additional measurements should be taken every four floors and at the top of the building. It also specifies that where a building is more than 10 m long measuring points should be installed at approximately 10 m horizontal intervals.

In summary, the recommended procedure for measurement is:

1. Carefully select measurement points.
2. Estimate the type and level of vibrations to be recorded.
3. Choose appropriate vibration transducer.
4. Choose suitable amplification, recording and analyser equipment.
5. Check and calibrate system.
6. Record the instrumentation and layout chosen.
7. Select appropriate mounting method and record the installation procedure.
8. Record instrument settings and interference from extraneous 'noise' and other ambient or background vibrations.
9. Record the disturbance.
10. Recalibrate after recording.

7.7 MITIGATION AND CONTINGENCY MEASURES

Following the assessment of expected vibration levels and the survey of potentially affected structures, procedures should be established in case the levels are higher than expected or are more harmful than assessed (see Section 6.9 for some of the matters which may have to be considered).

7.8 REPORTING AND CHECK LIST

Many of the case records were incomplete in various ways, perhaps because those who submitted them did not have all the necessary information or did not appreciate the importance of aspects outside their speciality. The reporting organisations included 17 specialist vibration consultants, 6 academic consultants, 5 civil and structural consulting engineers, 5 government research establishments, 5 local authorities and 2 miscellaneous – 40 in all. A particular feature of the reports from the specialist vibration consultants was that the information was almost exclusively on the vibration technology i.e. without details of pile or plant operations. Many case records lacked information on ground conditions, location and distance of measuring stations, frequency and energy of source, and details of either the plant or the measuring system. These gaps made the case records less useful.

Reports of vibration studies, like any specialist task, are often directed to the solution of specific problems. For the report to contain all the information that subsequent researchers would desire would go beyond the specialist's expertise and, usually, the brief for the study. Nevertheless, in order to enhance both the local knowledge and the national body of experience, it is important that as much information is given about the site and the works.

BRE Digest 353 presents a check list of matters to be included in vibration studies for structures. This list together with aspects which should be specifically reported for piling and ground engineering works are given in Table 20.

Table 20 Check list for reports of piling vibration studies

General matters

- Nature of the study, its purpose and its location
- Report reference/title and for whom it was prepared

Matters to be reported in building vibration studies[1]

- name, affiliation and professional standing of person taking measurement
- dates of measurement
- information on source of excitation including any technical details
- type of soil, any measured soil parameters (especially wave velocity)
- distance of source from structure
- description of structure, room sizes, layout and site location
- building construction type and floor plan
- general structural condition, including list of defects
- transducers, operating ranges and calibration factors
- amplifiers, recorders, analysers and calibrating equipment
- calibration procedures and results
- measurement positions and axes
- individual recorded time histories, maximum calculated *ppv* values and method by which they were calculated
- predominant frequencies in time histories
- background noise levels and normal vibration levels
- records of any damage, including photographs

Matters to be reported in studies about piling projects[2]

- Details of the ground engineering contract (client, engineer, main contractor, specialist subcontractor etc.)
- Location and plan of the site
- Ground conditions (locations and logs of exploratory holes from the ground investigation or a statement describing the known ground conditions and reference to the investigation report)
- Groundwater conditions
- Pile type, materials and dimensions (diameter or cross section, length and penetration)
- Piling rig and pile driving system (make and model of hammer and rig, weight of hammer and drop, energy per blow, frequency of blows, packing, dolly and recorded sets)
- Vibratory plant (make and model of vibro-driver or frequency of vibrator)
- Locations of piling and distances from relevant structures, services or plant for each vibration measurement
- Assessment of other possible risk factors such as loss of ground in pile boring, ground displacement caused by driven piling, lateral yield of excavations, settlements caused by groundwater lowering or compaction of loose granular soils, flooding, frost and drought etc.

Notes:

1. from BRE Digest 353
2. for ground improvement works such as vibro-compaction, vibro-replacement and dynamic compaction, the equivalent matters should be reported e.g. stone column nominal size, location, vibroflot etc.

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Appendix 1 Attenuation measurements in the case records

A number of the case records included studies of the attenuation of piling-induced vibrations with distance from the pile. These are summarised in Figures A.1 to A.15 as graphs of measured *ppv* against plan distance. Noted on the figures are pile type, driving method and energy and ground conditions, where known, and whether the measurements were on the ground or structure. Attenuation coefficients have been calculated for these cases in terms of the various equations put forward in the text. Table A.1 is for drop hammers, Table A.2 for diesel hammers, and Table A.3 for vibratory driving.

Table A.1 Attenuation data: drop hammers

Case	Hammer: Mass (t)	drop (m)	Energy E (kJ)	$x^{(1)}$	$a^{(1)}$	$k^{(2)}$	$b^{(3)}$	Figure
Martin (1980)a	5	0.6	29.4	0.90	34	63	74	A.1
Martin (1980)b	8	0.6	47.1	1.42	300	43.8	19.5	A.1
10	3	10	294	1.09	170	9.9	7.7	A.4
39	3	0.8	22.1	1.10	255	54.2	46.5	A.9
101	6	1	58.8	1.50	525	68.5	24.7	A.13
133	6	0.5	29.4	0.80	34	63	8.8	A.14

(1) Equation 6.1 $v = a/r^x$
(2) Equation 6.2 $v = K\sqrt{E}/r^x$
(3) Equation 6.3 (2.9) $v = b\,(\sqrt{E}/r)^x$

Table A.2 Attenuation data: diesel hammers

Case	Hammer	Energy E (kJ)	$x^{(1)}$	$a^{(1)}$	$k^{(2)}$	$b^{(3)}$	Figure
Martin (1980)c	–	40	1.38	165	26.1	12.9	A.1
28	Delmag D46	117.7	1.64	210	19.3	4.6	A.7
98	BSP B15	35.6	1.37	150	25.1	13.0	A.11
99	Kobe 13	36.3	1.49	156	25.9	10.7	A.12
101	Kobe 35	103.0	1.48	500	49.3	16.2	A.13

(1) (2) (3) See notes to Table A.1

Table A.3 Attenuation data: vibratory drivers

Case	Vibrodriver	Energy E (kJ)	$x^{(1)}$	$a^{(1)}$	$k^{(2)}$	$b^{(3)}$	Figure
7	ICE 81	-	1.16	80	-	-	A.2
8	Muller MS26	5.12	0.86	14.6	6.45	7.2	A.3
15	PTC 20H6	4.36	1.73	1030	490	290	A.5
59	Foster 4000	-	1.29	66	-	-	A.10

(1) (2) (3) see notes to Table A.1

SHEET PILES through silts over clay

MEASUREMENT on ground

• 5t drop hammer (30 kJ)
○ 8t drop hammer (48 kJ)
Diesel hammer (40 kJ)

SHEET PILES through sands/gravels

MEASUREMENT on ground

Vibratory (hydraulic) hammer
Driver frequency 26 Hz

SHEET PILES through gravel into London clay

MEASUREMENT on structure

Vibratory hammer (5.1 kJ/blow)
Driver frequency 25 Hz

PRECAST PILES through desiccated clay

MEASUREMENT on ground

3t drop hammer (294 kJ)

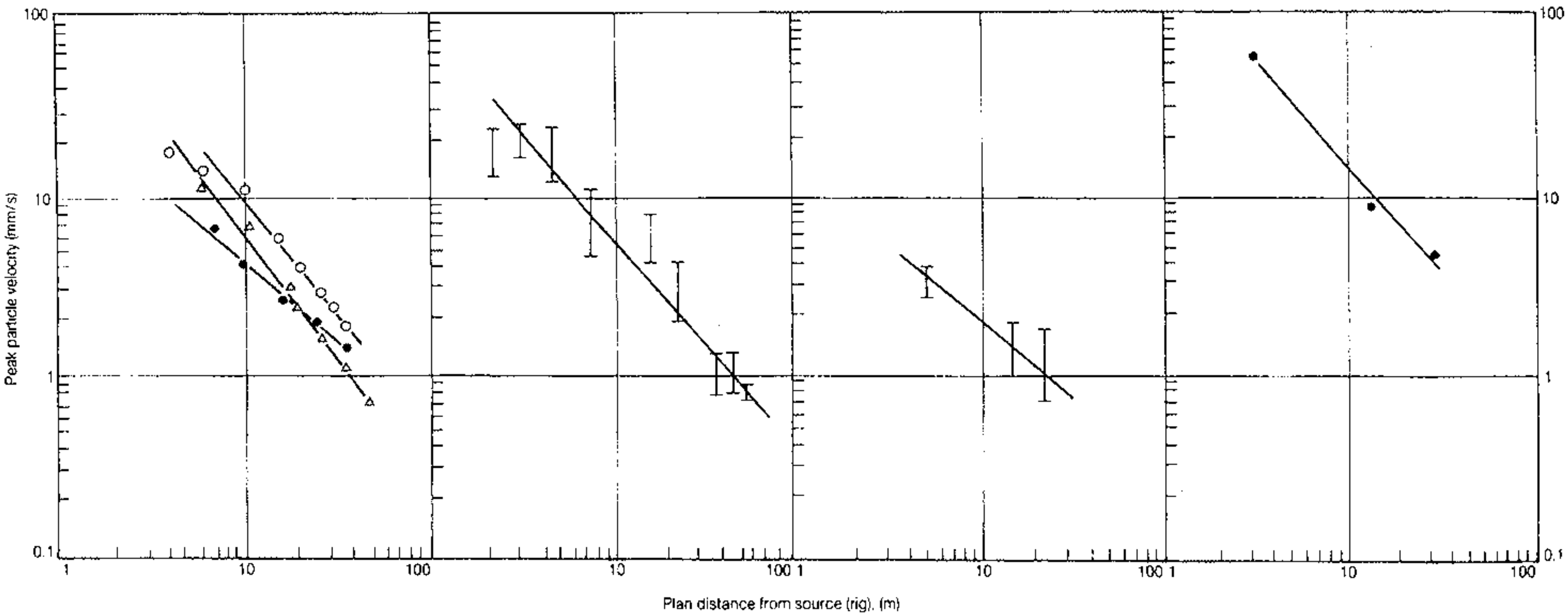

Figure A.1 *Martin (1980)*

Figure A.2 *Case record 7*

Figure A.3 *Case record 8*

Figure A.4 *Case record 10*

VIBRO-DRIVING of casing for auger-bored piles through Bagshot and Claygate Beds over London Clay

MEASUREMENTS on ground

Vibro-driver (4.36 kJ/blow)
Driver frequency 25Hz

DRIVEN CAST-IN-PLACE (WITH BULB) through stiff till over hard marl

MEASUREMENT on structure

Driving energy not known

SHEET PILES (Larssen No. 6) driven through alluvium, sand/gravel, and stiff clay over chalk

MEASUREMENTS on ground

Delmag D46 (118 kJ)
50 blows/min

DRIVEN CAST-IN-PLACE through fill, granular alluvium over rock

MEASUREMENTS on ground

Driving energy not known

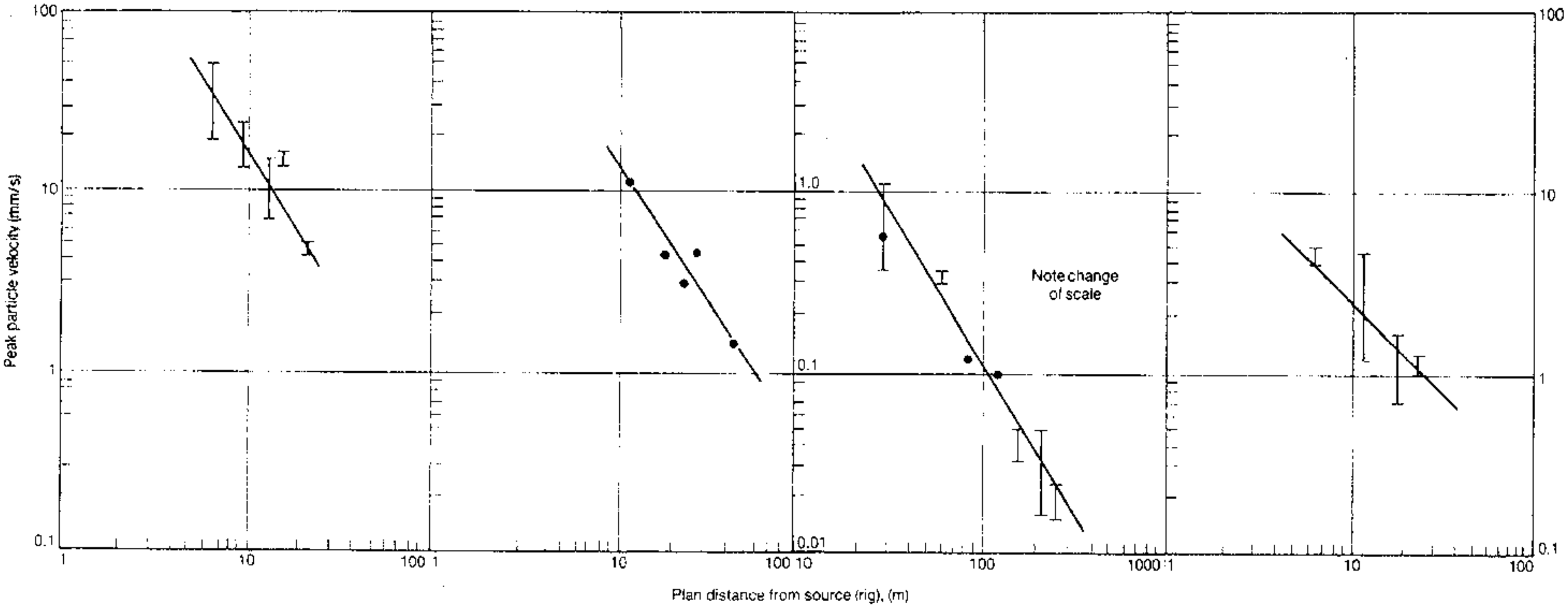

Figure A.5 *Case record 15*

Figure A.6 *Case record 20*

Figure A.7 *Case record 28*

Figure A.8 *Case record 33*

SHEET PILES through stiff clay
over compact sand

MEASUREMENTS on ground

Hush driver (2kJ0

SHEET PILES through gravel
over London Clay

MEASUREMENTS on structure

Vibro driver Foster 4000

SHEET PILES in sands and gravels

MEASUREMENTS on ground

• Diesel hammer BSP B15 (35.6 kJ)
∘ Air hammer BSP 600N (0.65 kJ)

H-PILES through sand, gravel clay to mudstone

MEASUREMENTS on structure

Diesel hammer Kobe K13
(3.7 kJ)

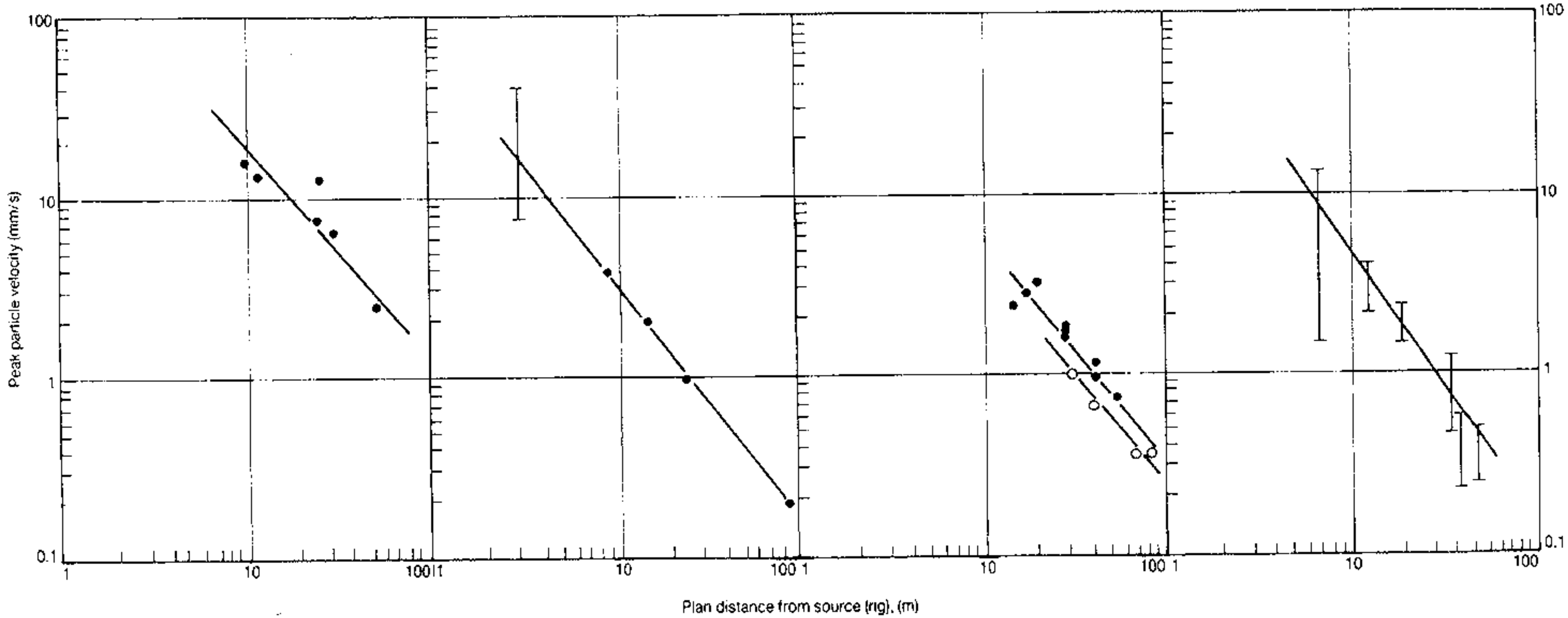

Figure A.9 *Case record 39*

Figure A.10 *Case record 59*

Figure A.11 *Case record 98*

Figure A.12 *Case record 99*

PRECAST CONCRETE PILES
through granular fill and clays
to sandstone

MEASUREMENTS on ground

► Delmag diesel hammer
● Kobe 35 diesel hammer (10.5 kJ)
• 6t drop hammer

PRECAST SHELL PILES through
soft alluvium

MEASUREMENTS on structure

6t drop hammer (3kJ)

PIPE PILES driven through sands
and gravel with limestone and shale
bedrock at 30m

MEASUREMENTS on ground

Bordine resonant driver and
Linkbelt 520 diesel hammer (36 kJ)

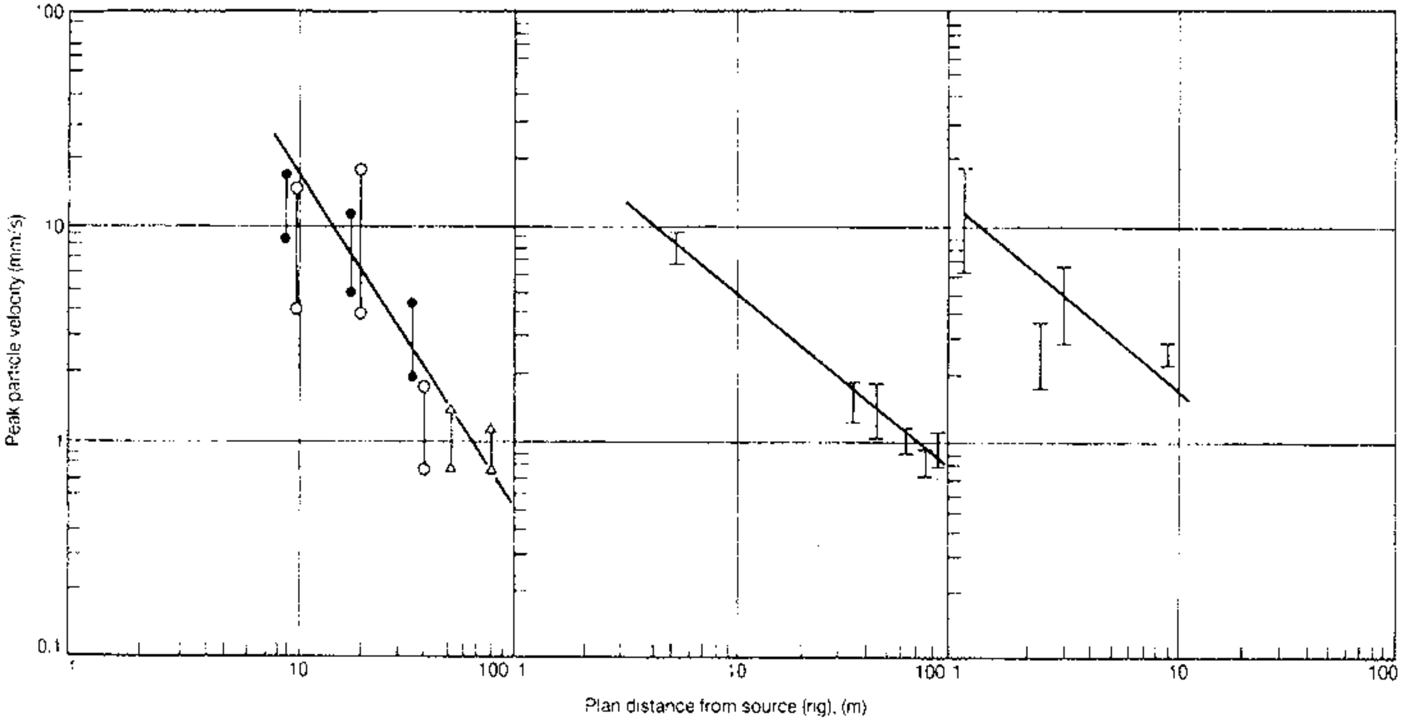

Figure A.13 *Case record 101*

Figure A.14 *Case record 133*

Figure A.15 *Case record 143A*

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(October 1992)

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Proposed London Underground (Victoria Station Upgrade) Order

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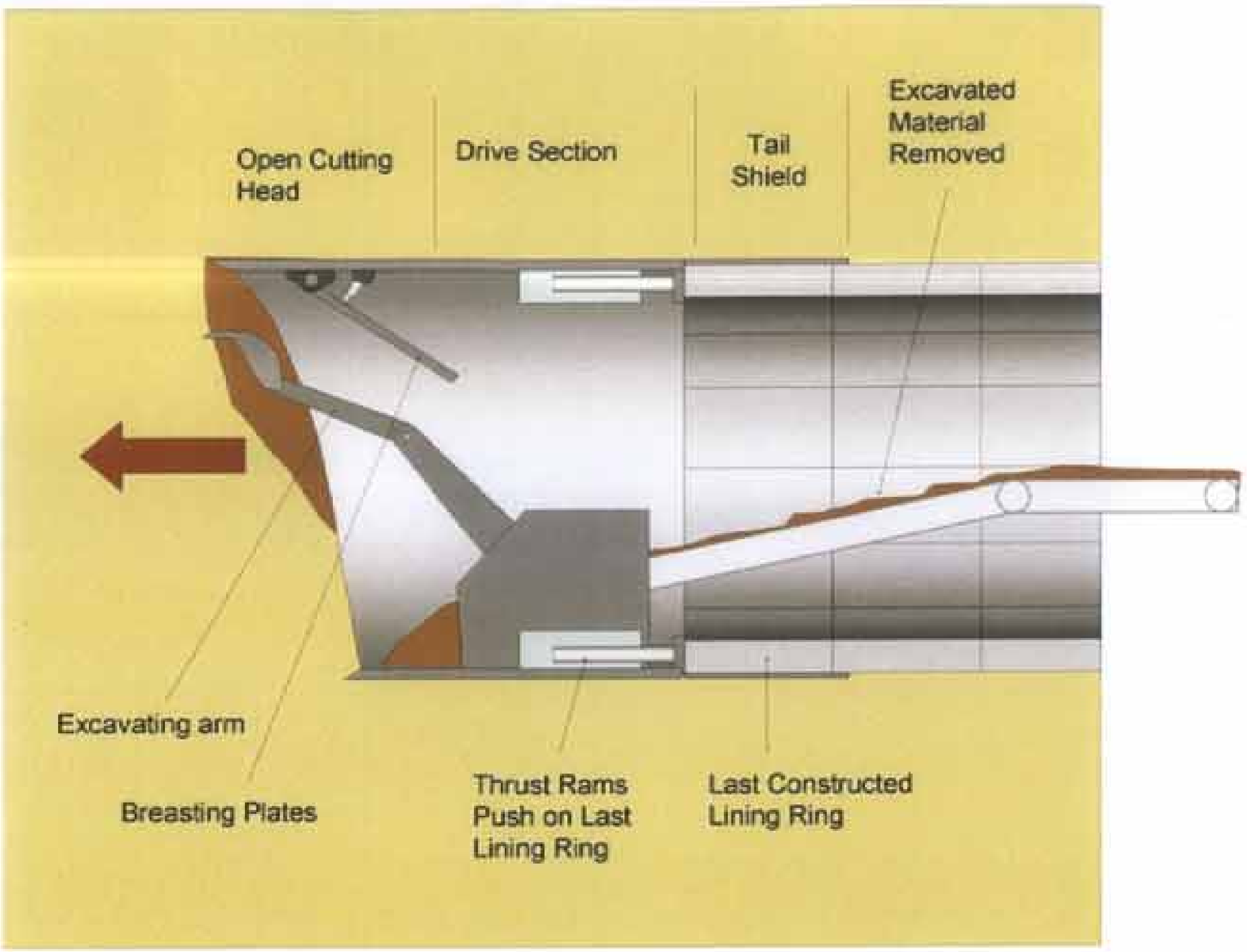
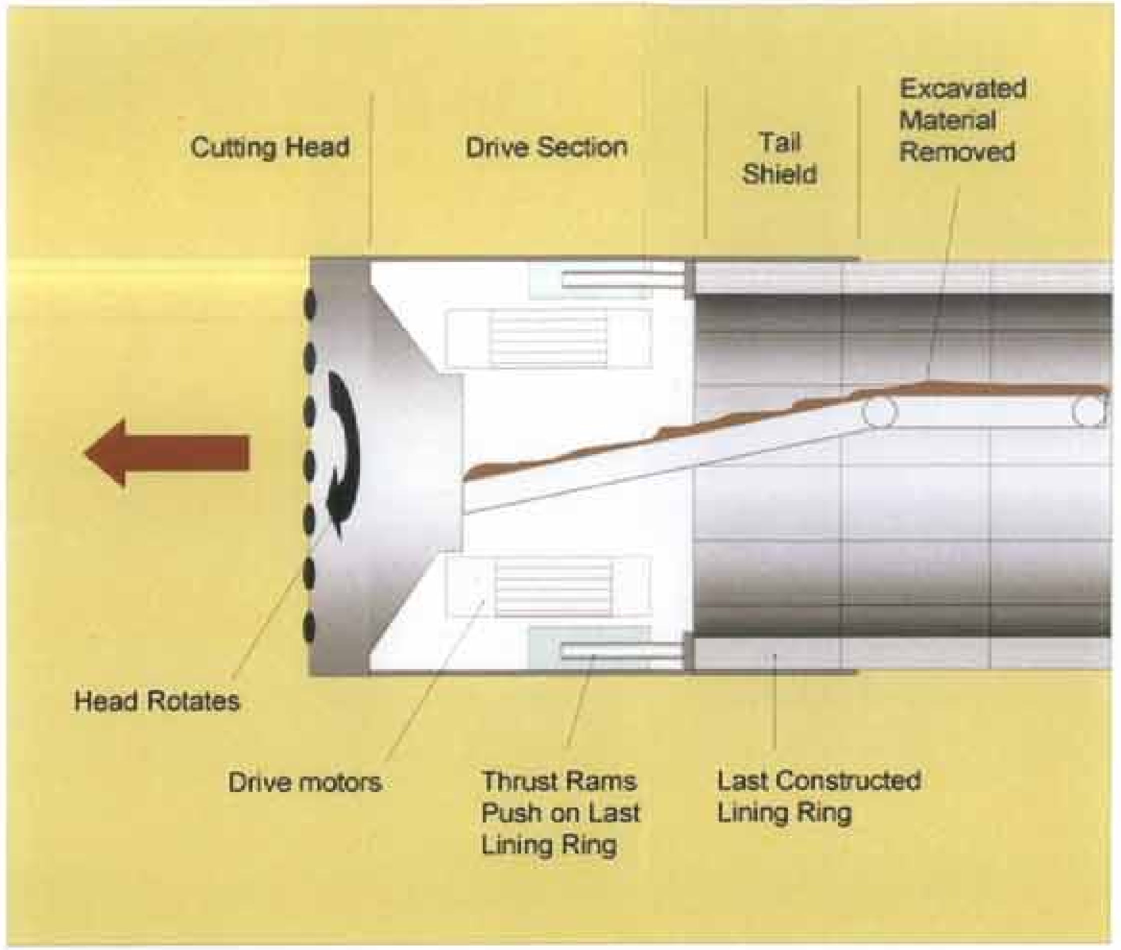
APPENDIX 17

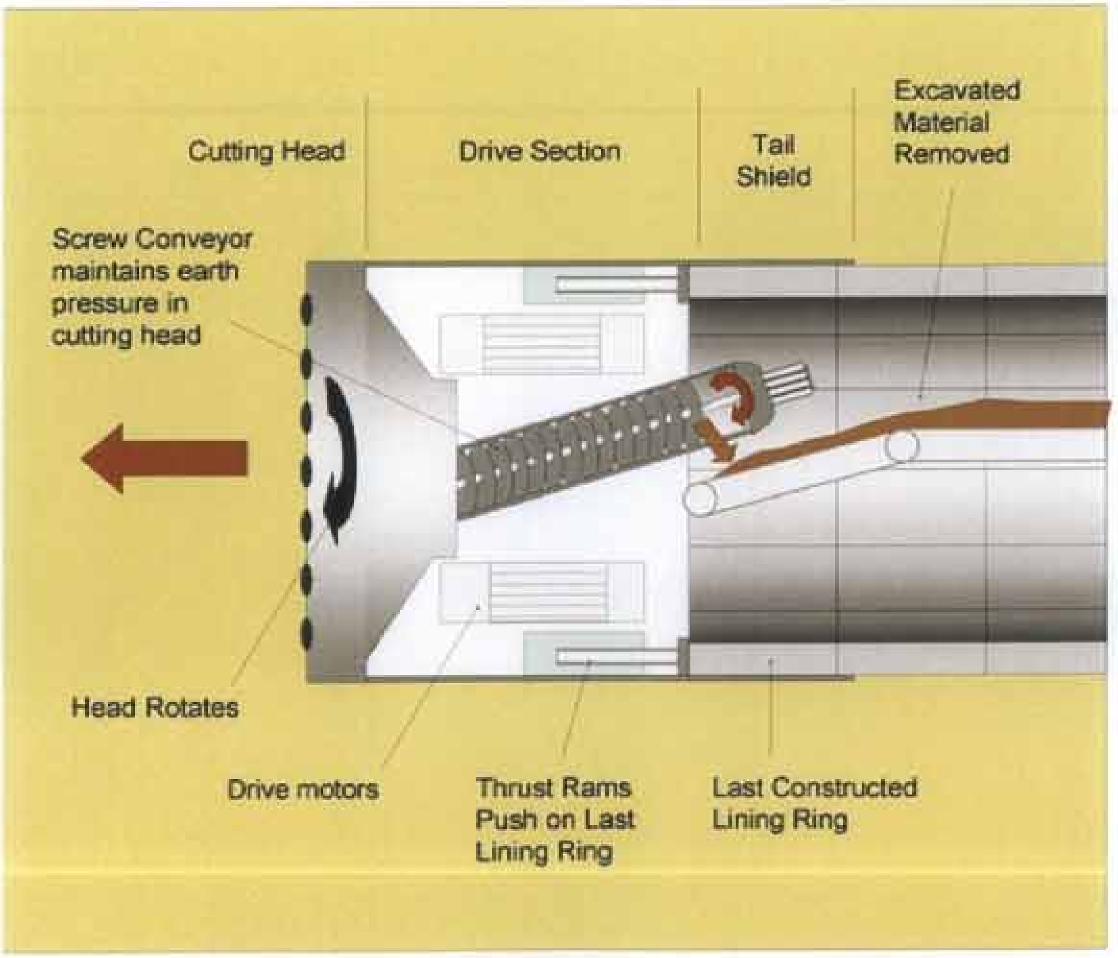
TUNNEL SHIELD MACHINES



OPEN FACE SHIELD MACHINE WITH BACKHOE EXCAVATOR



OPEN FACE SHIELD MACHINE WITH ROTARY CUTTER HEAD



CLOSED FACE EARTH PRESSURE BALANCE (EPB) MACHINE

EXHIBIT OBJ3/P3/A17
Evidence of Tim Chapman
Land Securities

Tunnel Shield Machines