Soil vibrations and damage criteria
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PREFACE

Blasting represents an important construction technique for loosening and removal of rock and soil materials. However, the vibrations emanating from the blast area can cause damage to nearby buildings and other structures. Criteria for safe blasting levels are therefore of great importance.

This report by the Swedish Council for Building Research presents an analysis of safe vibration levels applicable to blasting and similar construction vibrations. It also presents data from longterm observations of buildings in Sweden. This should prove valuable to contractors, blasting consultants, and those concerned with the formulation of standards for safe blasting.

The Division thanks P.J.E. Glynn who translated this report and J.H. Rainer of the DBR Noise and Vibration Section who checked the translation for technical accuracy.

Ottawa
December 1983

C.B. Crawford
Director, DBR/NRC

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SUMMARY

The present work is a compilation of a large number of projects in the field of soil vibrations where the vertical vibration velocity is viewed as a function of distance and charge weight. The compilation contains very heterogeneous measurement data since the results are based on various measuring techniques and instruments, and may also depend on individual interpretations of vibration values. Supplementary measurements have been made and one single relationship

$$V = K/\left(\frac{R}{\sqrt{Q}}\right)^{\alpha}$$

has been fit to all values.

An examination of damage in buildings as a function of the magnitude of the vibrations has been carried out and compared with similar non-Swedish investigations. This shows that damage can occur at very low vibration values but that the probability of this occurring is small. At 50 mm/s the probability is thus 40% that cosmetic damage may occur but the likelihood is only 3 - 4% for larger damage. It will be observed that the investigations were based on changes between records of pre- and post-event inspections of properties. In post-event inspections the pre-event inspection record is used as a reference; a post-event inspection indicates, therefore, never a smaller number of cracks. For this reason it is very likely that there will be a number of additional entries in the post-event inspection record without an actual increase in the number of instances of damage. New instances of damage caused by ageing of the building have not been considered either.

1. THE RELATIONSHIP BETWEEN VIBRATION, CHARGE AND DISTANCE

In an earlier BFR* report (1) a large number of projects were compiled where the vibrations were presented in the form of vertical velocity in mm/s as a function of $R/\sqrt{Q}$, where R is the distance in metres and Q is the instantaneous charge in kg. Fig. 1.1 shows this relationship. An attempt was

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* BFR: Byggforskningsradet - the Swedish Council for Building Research

also made to obtain a formula with a better agreement. The principle for this is shown in fig. 1.2. The normal procedure is to adapt straight line bounds or ceilings to the vibration values as function of R and Q. In the improved fit the bound is sloped so that the agreement is improved. This fit is, however, complicated and difficult to handle. It furthermore appears that the values at the extremes of the bounded region deviate more from the measured value than with the simpler fit. Because of this and a number of new values it is proposed that the simple fit mentioned above is used, i.e.

$$V = K/(\frac{R}{\sqrt{Q}})^{\alpha} \qquad (1.1)$$

where a certain value of $R/\sqrt{Q}$ indicates a line having the same "ceiling height" V.

Gosta Rundqvist (2) has measured 72 values on vibrations at Cementa AB in Skovde. The values were analyzed on SveDeFo's computer and are shown in fig. 1.3. The charge varied between 40 and 110 kg and the distance varied between 45 and 1,060 metres. This relationship

$$V = 724/(\frac{R}{\sqrt{Q}})^{1.52} \qquad (1.2)$$

is also incorporated in fig. 1.1.

Böttcher, Lüdeling and Würtenhagen (3) stipulate the relationship

$$V = 1226/(\frac{R}{Q^{0.426}})^{1.579} \qquad (1.3)$$

which is indicated in fig. 1.4 with the charge weight as parameter. In the case of Q = 1 kg, (1.3) may be written:

$$V = 1226/(\frac{R}{\sqrt{Q}})^{1.579} \qquad (1.4)$$

From fig. 1.4 the relationship can also be obtained for other charge weights, e.g. 1,000 kg, i.e.

$$V = 546/(\frac{R}{\sqrt{Q}})^{1.579} \qquad (1.5)$$

(1.4) and (1.5) are incorporated in fig. 1.1.

(1.4) is located significantly above other curves, particularly at short distances, while (1.5) agrees well with other relationships.

Medearis (4) states

$$V = 388/(\frac{R}{\sqrt{Q}})^{1.39} \qquad (1.6)$$

which also agrees well with the other values.

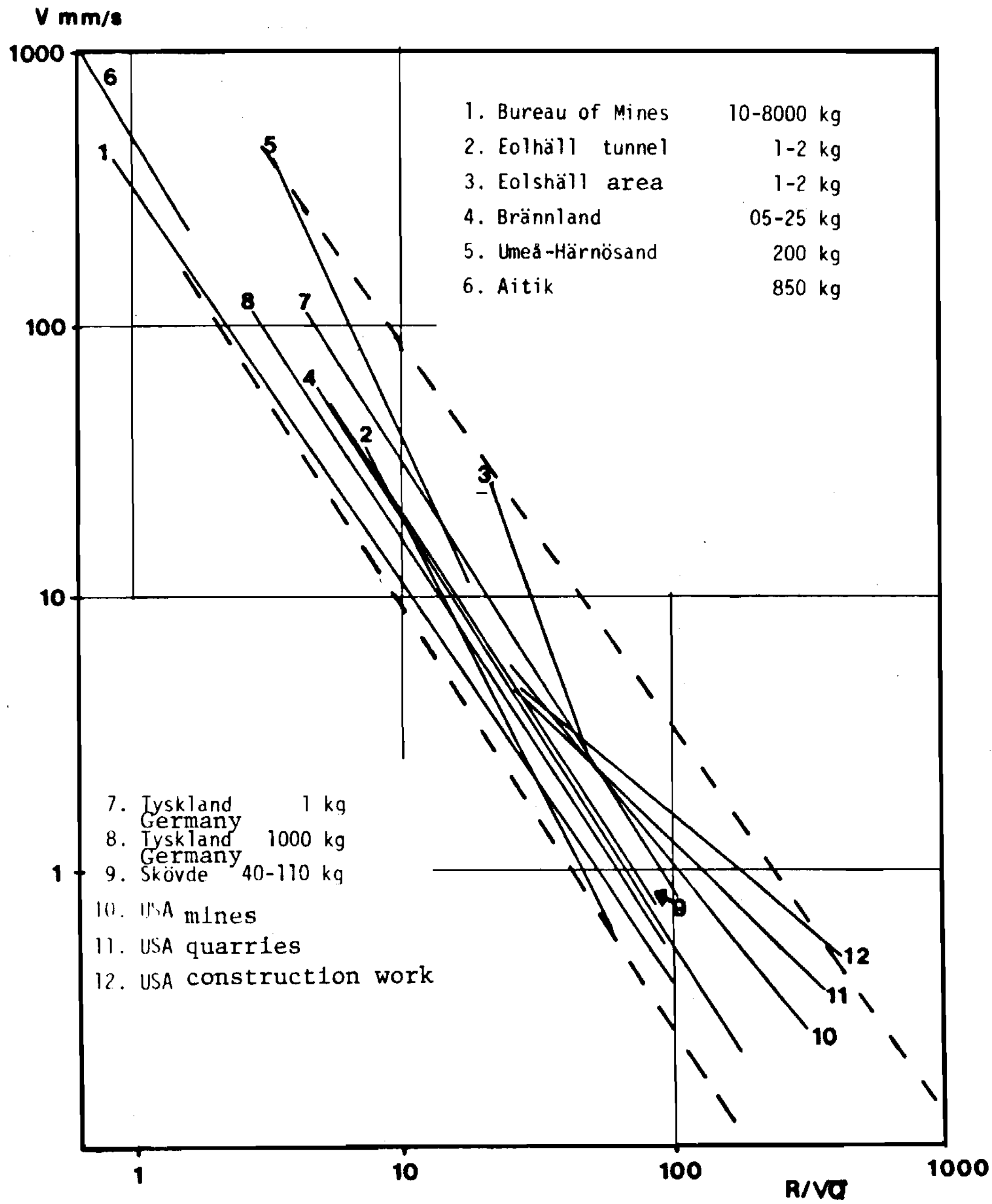


Fig. 1.1. Relationship between vibration, charge and distance at various locations. (Editor's note: "9" applies to the line with the arrow).

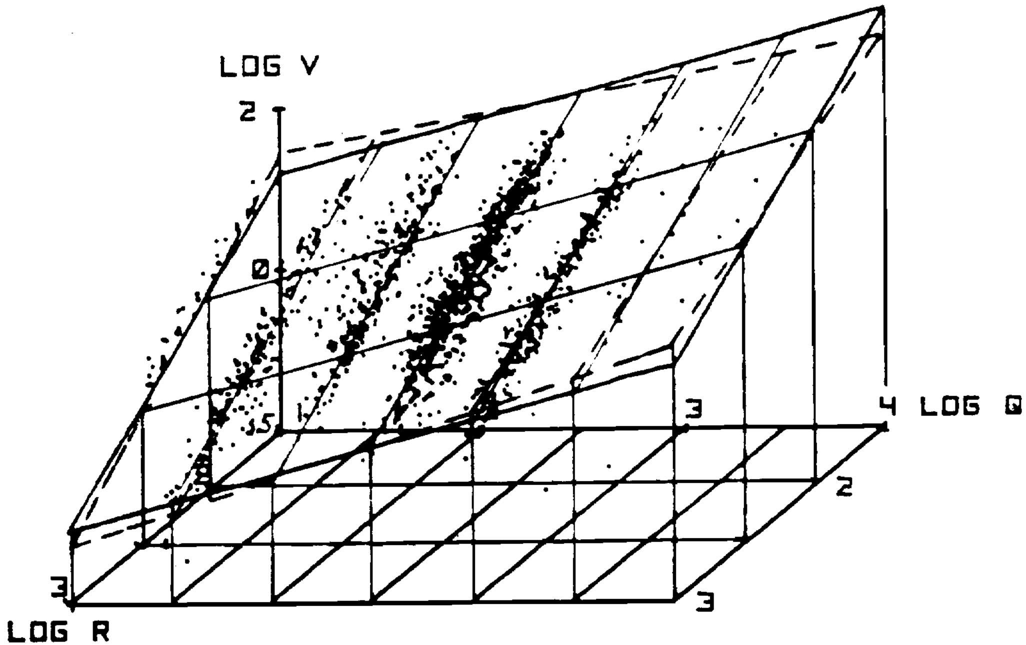


Fig. 1.2. Threedimensional representation of vibration velocity as a function of charge weight and distance.

Spencer and others (5) indicate various relationships for mines, quarries and factory installations. The mean value for these is

$$V = 135/(\frac{R}{\sqrt{Q}})^{0.98} \text{ mm/s} \tag{1.7}$$

with R stated in metres and Q in kg.

The following is stated for above mentioned working sites

$$V = 310/(\frac{R}{\sqrt{Q}})^{1.21} \text{ for mines} \tag{1.8}$$

$$V = 100/(\frac{R}{\sqrt{Q}})^{0.91} \text{ for quarries} \tag{1.9}$$

and $$V = 68/(\frac{R}{\sqrt{Q}})^{0.8} \text{ for construction work} \tag{1.10}$$

The relationships 1.8 - 1.10 are incorporated in fig. 1.1.

Upper and lower limits for the measured relationships have been incorporated in fig. 1.1. The correlation between them is significantly poorer than the spread of the individual values around their mean.

The mean value* from different locations, measured by various authors, is thus less than the scatter in individual measured values at one location when measured by the same person and with the same measuring equipment. (* the spread in the mean value? - the translator).

The relationships shown in fig. 1. can therefore be considered to provide a good prediction of the vibration which will develop in blasting operations with a certain number of kg (charge) and at a certain distance.

Since the scatter in individual values is greater than that indicated in the figure, it is ineffective to carry out one single trial blasting to obtain a more exact relationship. In locations with varying ground conditions there may, however, be large variations, and there it may be justified to carry out trial blastings. The (observed) scatter should be kept in mind, and a certain number of blastings should be made, using measuring points at various distances and soil types, to obtain an acceptable relationship. The magnitude of the spread decides then the size of the margin which must be used to obtain vibrations in subsequent production shots which be below the stipulated limits with some degree of certainty.

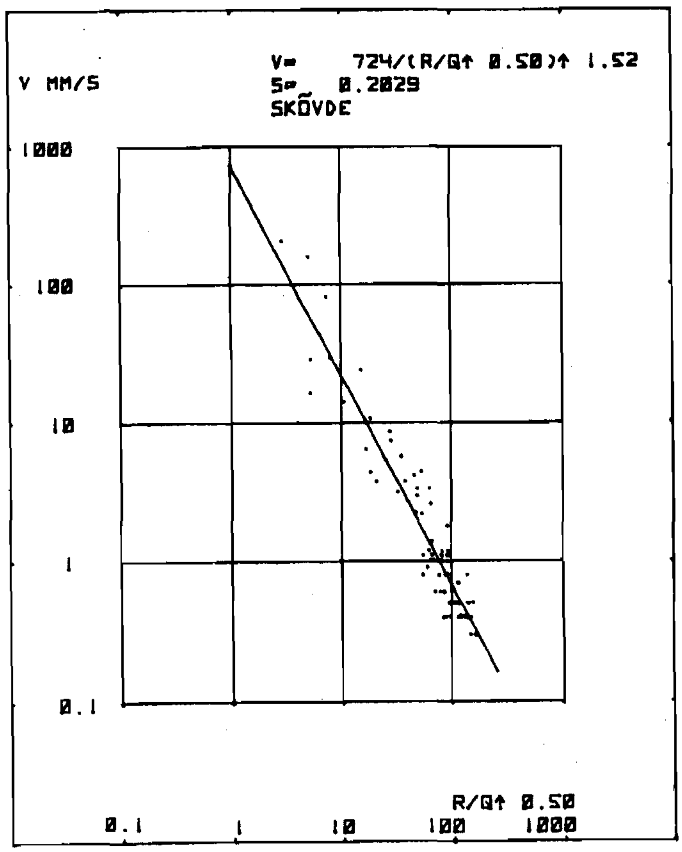


Fig. 1.3. The relationship between vibration, charge and distance at Cementa AB in Skövde.

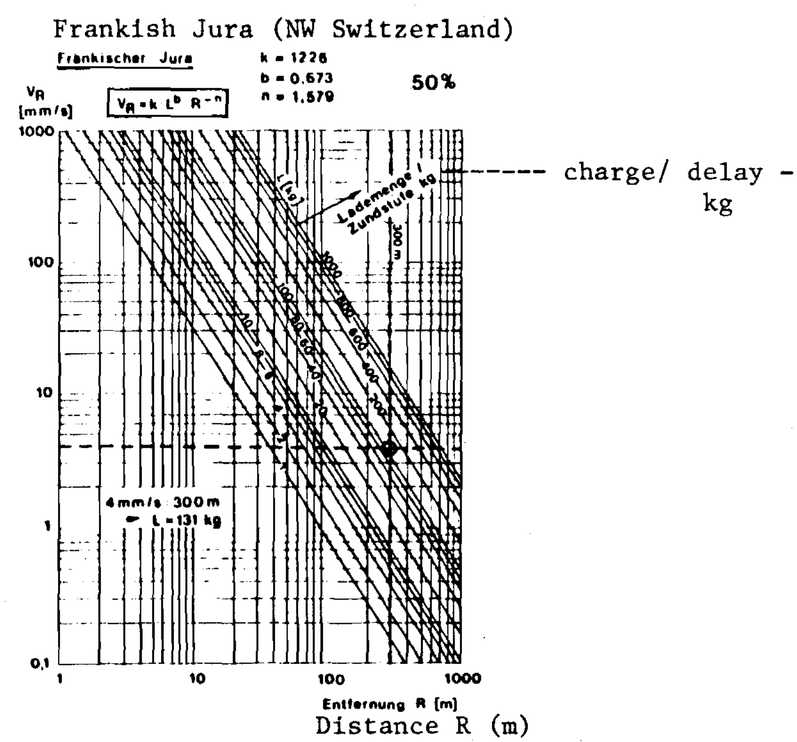


Fig. 1.4. Relationship between vibration, charge and distance according to Böttcher and others (3).

2. DAMAGE IN BUILDINGS

All existing buildings have different life spans or time periods until damage occurs. The life span depends on the stresses to which the building is exposed, as well as on the resistance of the construction materials to chemical and physical effects. Heat, moisture, loads, settling (subsidence) etc. all cause different movements in the building, and with an optimized selection of construction materials and designs, these movements should take place without a disturbing buildup of stress concentrations in the structural element of the building. If the building design does not achieve this, then cracks will always develop which indicate where these movement-absorbing joints should be installed or where reinforcements should be constructed. This is described in detail by Wilhelmsen and Larsson (15).

In the case of a building design which is not exposed to major external disturbances such as vibrations, it is a known fact that the number of new cracks can be expressed as a function of time. It is probably natural that a major number of cracks are formed per time unit directly after construction as is shown in fig. 2.1.

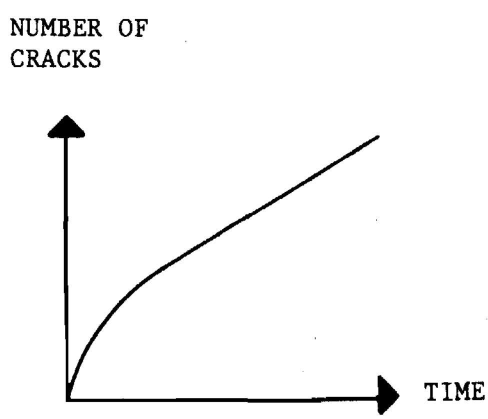


Fig. 2.1 Increase in cracks, as a function of time, in buildings which are not exposed to external disturbances.

If external disturbances are introduced, then stress concentrations increase, and a number of cracks are triggered instantaneously. Subsequently the natural crack increase per unit of time will probably stagnate until the stress concentrations become once more critical. The formation of cracks will then once more follow the earlier curve according to fig. 2.2.

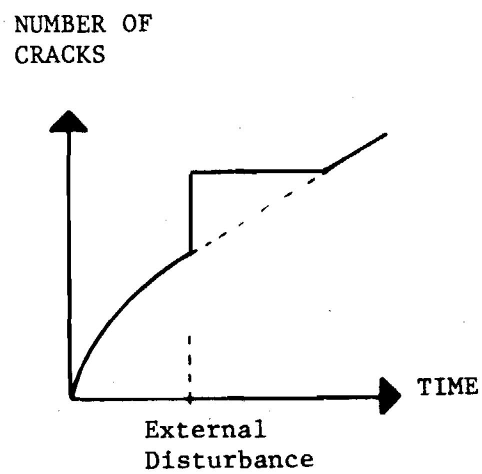


Fig. 2.2. Crack formation increases instantaneously when the building is exposed to vibrations.

The pre- and post-event inspections, which are compared, usually result in ascribing earlier undetected cracks to vibrations, and it is not taken into consideration that the damage situation would likely be the same after a given unit of time, independent of the introduction or absence of vibrations.

3. EXAMINATION OF REPORTED DAMAGE IN BUILDINGS AS A FUNCTION OF VIBRATION VELOCITY

Gosta Rundquist (6) has, by analysis of earlier pre- and post-event inspection records, established a relationship between recorded changes and measured vibration velocities.

Because of the relatively limited material (approximately 100 cases) he was only able to study three types of construction materials - wood, lightweight, and regular concrete. He has compared detached houses with apartment blocks and also various cladding materials. This analysis was based on Nitro Consult's archives.

3.1 Passage of Wave Fronts

Since vibrations caused by blasting, pile driving, etc. all contain the same types of waves, this section is of general validity. The surface waves of the Rayleigh type are dominant. This wave causes all the types of stresses which are described here. Since it is, furthermore, surface waves that have the lowest propagation velocity and the highest amplitude, they will expose buildings to the greatest stresses.

3.1.1 Tensile Stresses

When a P-wave is propagated in a material it is exposed to tensile and compression stresses. Since the stress limit for tensile stresses is much lower than for compression stresses, only the former are discussed in the following, although the formulae given apply to both types.

The developed stress $\sigma_{tension}$ is indicated by the relationship

$$\sigma_{drag} = \varepsilon \cdot E \qquad (3.1)$$

drag = tensile

where ε = strain in the material and E = the modulus of elasticity of the material;

but $\varepsilon = \frac{v}{c}$, therefore

$$\sigma_{drag} = \frac{v}{c} \cdot E \qquad (3.2)$$

drag = tensile

In fact, a similar situation exists in fig. 3.1, where a wave front passes the house foundation with a horizontal velocity component which is of the same magnitude as the propagation velocity c_b in the underlying rock.

When vibrations have been generated on or in the ground, then the direction of wave propagation in the ground becomes chiefly horizontal; therefore c_b and i_b may be exchanged for c_m and i_m in the respective formulae.

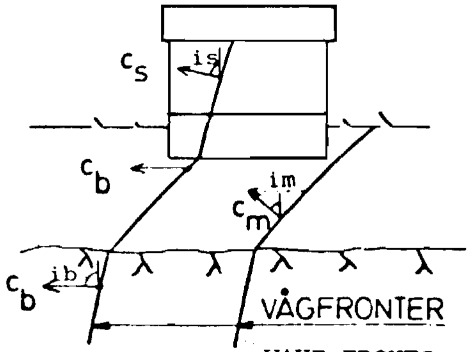


Fig. 3.1. Wave fronts for P- or S waves in the ground and in buildings.

Part of the wave energy will be refracted in the structure of the building by an angle i_s which is determined by the relationship

$$\sin i_s = \frac{c_s}{c_b} \cdot \sin i_b \qquad (3.3)$$

where c_b = propagation velocity in rock,

c_s = propagation velocity in the structure, independent of the intermediate layer.

The tensile stress in the building structure is then obtained from the relationship

$$\sigma_{drag} = \frac{V_{max}}{c_s} \, E_{structure} \qquad (3.4)$$

drag = tensile

where V and c are measured in the direction of the wave propagation in the structure.

Along the contact surface of the building and the substratum, a horizontal tensile stress σ_{hor} will develop, the magnitude of which will depend both on c_b and c_s.

$$\sigma_{hor} = \frac{c_s}{c_b} \cdot \sin i_b \cdot \frac{V_{hor}}{c_b} \cdot E_{structure} \qquad (3.5)$$

where v_{hor} = the horizontal component of vibration velocity (mm/s) in the direction of propagation in the structure.

c_b = the horizontal component (m/s) of the propagation velocity in the rock.

c_s = propagation velocity in the structure.

$E_{structure}$ = the modulus of elasticity (Pa) of the building structure.

$\sin i_b \approx 1$, in accordance with the assumptions.

When comparing (3.3) and (3.5) it is known that $V_{hor} < V_{max}$ generally applies and that $c_b > c_s$; therefore $\sigma_{tension}$ is usually greatest when calculated according to formula (3.4). When the wave front passes through the building it will be reflected, and at an incident angle of 90^o it will cause a doubling of the particle velocity at the moment of reflection while the tensile stress become zero.

3.1.2 Shear Stresses

The argument for shear stresses is entirely analogous to that above.

The particle velocity is measured perpendicular to the direction of propagation of the shear wave. Also in this case, two types of stresses are obtained: partly in the lower portion of the ground

$$\tau_{hor} = \frac{c_s}{c_b} \cdot \sin i_b \cdot \frac{V_{hor}}{c_b} \cdot G_{\substack{stomme \\ structure}}; \; y = \frac{V_{vert}}{c_b} \tag{3.6}$$

and partly in the structure

$$\tau_{max} = \frac{V_{max}}{c_s} \cdot G_{stomme}; \; y = \frac{V_{max}}{c_s} \tag{3.7}$$

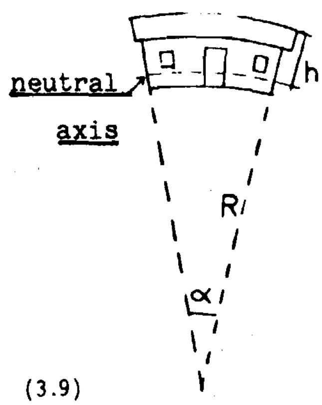
where V and c relate to the particle velocity of the shear wave and to the propagation velocity, respectively. $G_{structure}$ = shear modulus (Pa) of the building.

3.1.3 Bending (Flexural) Stresses

It is this type of stress which is presumably dominant in connection with traffic vibrations. It can, however, develop only in low structures. Flexural stresses develop when the building site consists of fairly loose soils, such as clay and sand, where the propagation velocity of the surface wave is low and the wavelength is of the same magnitude as the length of the building.

If it is assumed that the surface wave is similar to a sine function, then the smallest radius of curvature for a wave crest can be described thus:

$$R_{min} = \frac{c^2}{A\cdot\omega^2} = \frac{\lambda^2}{A\cdot 4\pi^2} \qquad (3.8)$$



where c = phase propagation velocity of wave crest

A = displacement amplitude

ω = angular frequency

λ = wavelength

The relative strain ε is determined by:

$$\varepsilon = \frac{(R+h)\alpha - R\cdot\alpha}{R\cdot\alpha} = \frac{h}{R} \qquad (3.9)$$

$$(3.8)+(3.9) \text{ qer gives } \varepsilon_{max} = \frac{h}{R} = \frac{h\cdot A\cdot\omega^2}{c^2} = \frac{h\cdot y_{max}}{c^2} = \frac{h\cdot A\cdot 4\pi^2}{\lambda^2} \qquad (3.10)$$

(3.10) shows that flexural stress is proportional to the displacement amplitude of the movement and the reduced height of the building and is inversely proportional to λ^2.

To the flexural stress, as derived above, the tensile and shear stresses must be added; these develop because of the particle motion in the surface wave.

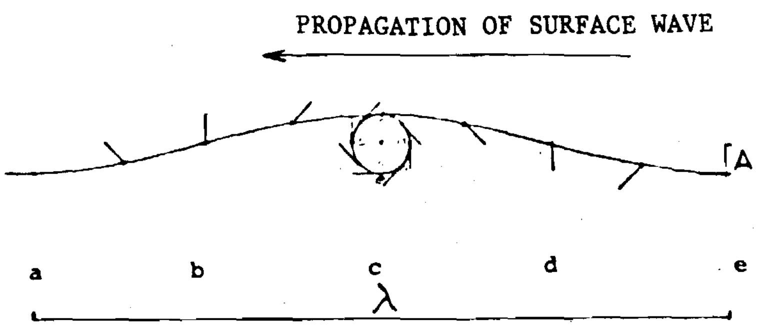


Fig. 3.2 Particle motion in a surface wave.

Fig. 3.2 shows one period of the surface wave. It has wavelength λ and displacement amplitude A. The figure shows also how a solitary particle moves during this period. It has been assumed that this takes place in the shape of a circle while it is in reality closer to an ellipse. The direction of the velocity vector is also shown for the various phases.

If the wave has amplitude A, then the distance b - d = $\frac{\lambda}{2}$ + 2A, and there is thus an elongation in connection with the passage of the wave crest. A house which finds itself on a wave crest is thus moved by the particle movement on the contact surface a distance equivalent to (see the theoretical section - harmonic waves):

$$\varepsilon_{\substack{drag\\ tensile}} = \frac{2\pi \cdot A_H}{\lambda} = \frac{V_{hor}}{c_R} \qquad (3.11)$$

where A_H = horizontal displacement amplitude of surface wave

V_{hor} = horizontal vibration velocity of the surface wave

λ = wavelength

C_R = phase velocity of surface wave

(3.10) and (3.11) indicate that

$$\varepsilon_{resultant} = \frac{h \cdot A_V \cdot 4\pi^2}{\lambda^2} + \frac{2\pi \cdot A_H}{\lambda} \qquad (3.12)$$

If the building acts as a unit with the underlying soil (which is an assumption), then $\varepsilon_{tensile}$ must be added to $\varepsilon_{bending}$, and the measuring location to determine $\varepsilon_{tensile}$ must be located high in the building, right above the other (measuring point for $\varepsilon_{bending}$?), if the level of the neutral axis is $<\frac{h}{2}$*. From the point of view of measuring technique it may be tempting to measure the maximum acceleration for the surface wave and its propagation velocity (see (3.10)) but since both the P- and S-waves have a higher frequency than the R-wave, these waves will most probably produce higher acceleration levels than the R-wave. An analog recording instrument for wave identification is therefore required, as well as an opportunity to filter out the P- and S-waves before the acceleration level is determined.

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* Editors' note: It is presumed that h here refers to the building height and not to the h in connection with Eqs. 3.9 and 3.10.

3.1.4 Permissible Strains for various Building Materials and Examples of Vibration Strains

Table 1 below shows permissible strains before failure, using static loads. Since the strength of the material usually increases under dynamic stresses, this implies that the stated values contain a good safety margin for vibration stresses. For tensile stresses the values must be reduced by 75-90%.

Building Material	Strain (ε) 0/00
Clay brick and sand - limestone A	2.5
B	4.0
Concrete block - hollow, class A-B	3.5-4.9
Lightweight clinker	2.5
Aerated concrete, 0.65 kg/dm^3	2.0
Masonry of Aerated concrete	4.0
Glued aerated concrete	3.5
Stacked aerated concrete	4.5
Regular concrete	0.1
Gypsum sheet	3.5

Table 1. Permissible compressive strains for various building materials.

Some examples of the strains which may develop in various building materials with different soil- and wave types are shown in table 2 below. The height of the building structure is assumed to be four metres.

*HV indicates that the vibration level must be measured relative to the direction of wave propagation in the structure.

Table 2. Shear in building materials for various foundation soils and vibration levels.

Ground/ Structure	Wave type P_{ground}/ $S_{structure}$ (m/s)	Vibration level (mm/s)	Shear strain (0/00/ radians)
Rock concrete	$P_M = P_S = 4500$	70 V/HV *	0,016
	$S_M = P_S = 3000$	70 V/HV	0,023
Rock/light-weight concrete	$P_M = 4500$	30 H	0,007
	$P_S = 2000$	30 HV	0,015
	$S_S = 1500$	30 HV	0,020
Moraine/ concrete	$P_M = P_S = 4500$	50 H/HV	0,011
	$R_M = 1500$	50 V/H20 HZ	0,045
Clay/concrete	$P_M = P_S = 4500$	20 H/HV	0,004
	$R_M = 400$	20 V/H 10 HZ	0,082
Clay/light-weight concrete	$P_M = 4500$	10 H	0,002
	$P_S = 2000$	10 HV	0,005
	$R_M = 400$	10 V/H 10 HZ	0,041

The stated wave propagation velocities are very approximate, and there are in fact large variations. Especially the R-wave may have a very low propagation velocity which has been found to be lower than 50 m/s in some instances.

3.2 Comparison between Theoretical and Practical Applied Limits

The load figures in the preceding table agree well with the actual limit values for permissible vibration stresses. In practical risk assessments a more detailed classification of the foundation soil types, structural materials etc. is used however, and age - as well as the condition of the building - are considered. Unfortunately it was found difficult to obtain information on the shear- or tensile stresses tolerated by various building materials. The calculated shear stresses for various loads therefore lack test results for comparable materials. The tests deal generally with the compressive strength of the material, and in the case of concrete, also with the fracture limit for flexural tensile stress. Usually it can, however, be assumed that the tensile strength is 10-15% of the compressive strength. In dynamic processes it is not unlikely that a greater strain can be tolerated than in a static process. Table 2 shows that the given examples of loads produce strains which are 10 - 20 times lower than the tolerance of the material under static loading. Concrete is an exception; it has properties close to the limit. Concrete is, however, always reinforced so that it can absorb tensile stresses. As a result of the reinforcing the strain is uniformly distributed and permissible values are as provided. In projects (12) and (13), where blasting was continued until the building started to crack, very high values were recorded before the aerated concrete cracked. The vibration level was then approximately 350 mm/s. With a propagation velocity of the P-wave in aerated concrete of approximately 2,000 m/s the following result is obtained:

$$\varepsilon = \frac{350}{2000}\ {}^{0}/_{00} = 0{,}18\ {}^{0}/_{00} \qquad (3.13)$$

This has a strain limit of 0.4 0/00 in tension.

Age and possible accumulated stresses may explain the difference. Brick-constructed basement apartments are one of the main uses for aerated concrete. Backfilling against the wall is arranged after completion, and an earth pressure develops against the wall. This produces tensile stresses which are so large that tension cracks are often initiated on the inside of the wall. This has been partly corrected by reinforcing the joints. Nevertheless it may happen that only small additional stresses are needed to increase the total tensile stress to a level where cracks develop.

3.3. Relationship between the Number of Additional Entries in Post-Event Inspections and the Level of Vibration Measurements

It was shown in the preceding section that it is very difficult to determine theoretically a useful value for maximum permissible vibration velocities. The existence of accumulated stresses implies that there is always a potential risk of new cracks when the vibration stresses are larger than those that occur in normal use; to this the climatological factors must be added. The limits, which are used, must therefore always be a compromise where a certain risk is accepted that new cracks may develop. One way to estimate this risk is to attempt to outline a statistical relationship between the number of additional entries and the measured vibration level on the respective site, and by analyzing inspections and vibration measurements.- This examination has been made possible thanks to Nitro Consult's inspection procedure where a form is completed for each inspected site, containing soil type, foundation etc. There are, however, two primary sources of errors. To start with, it is almost impossible to pick up every crack in a pre-event inspection, and secondly some damage is always added in the course of time as a result of the "ageing" of the building.

3.4. Accuracy of the Pre-event Inspection

The publication "Appraisal - Crack Inspection of Buildings" (14) contains records of an inquiry dealing with attempts to estimate the proportion of existing damage which was missed in the pre-event inspection.

	Missed instances of damage, in percent					
	0-0.3	1-2	3-5	6-10	10-15	20
Number of consultations who provided above instances	2	3	4	2	3	2

The mean value is approximately 7%.

Table 3. Estimate of missed damage in percent.

In post-event inspections, the pre-event inspection records are used as a reference; thus there are never fewer number of cracks in a post-event inspection record. It is therefore very likely that there will be a number of additional entries in the post-event inspection records without an increase in the number of incidents of damage.

3.5. Ageing of the Building

With a view to investigating to what extent the number of cracks increases with time, the inspection records of two apartment blocks were examined; these had been inspected three times on various occasions since their construction in 1965. Their designs were identical, and they had similar foundations.

Fig. 3.3 shows that there is a continuous increase in the instances of damage. In the case of the premises marked (o) this happens very uniformly while the rate of increase in the number of cracks is more irregular for the other property (x). The mean increase in the number of cracks is, however, very similar and amounts to 12-13 instances per year.

It must, however, be emphasized that these figures apply only to this particular case, and that it is very likely that other buildings age quite differently and at a different rate. It is quite obvious that, generally speaking, changes occur with time.

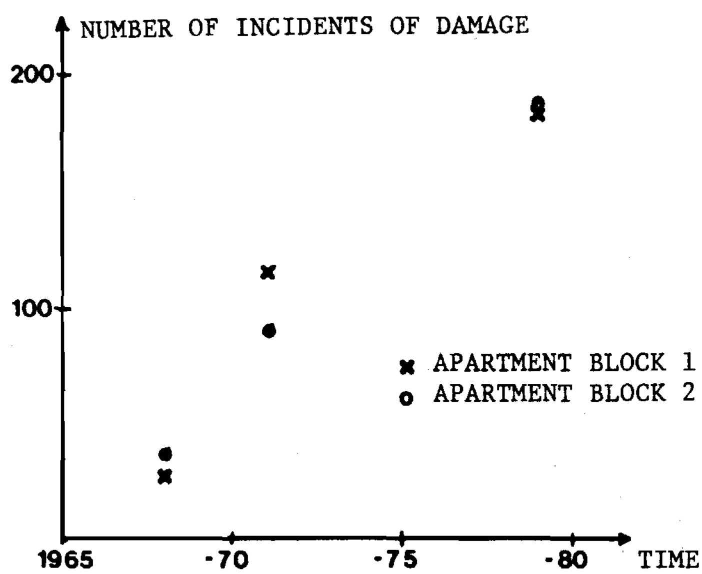


Fig. 3.3. Relationship between ageing of premises and the number of instances of damage.

3.6. Results of the Investigation

3.6.1. Detached Houses - Apartment Blocks

The majority of the examined properties are constructed on rock. Only a small number of detached houses are built on sand - clay. Figures 3.4 show the total number of newly detected instances of damage in apartment blocks constructed on rock and in detached houses built on rock and clay, respectively. It is seen that there is a considerable scatter of the values but in the case of the detached houses built on clay it is noted that there are no additional entries where the vibration level was below 10 mm/s. The total number of properties is, however, on the low side to permit reliable conclusions.

There appears to be a clear trend in the case of detached houses and apartment blocks that the number of additional entries increases with the level of vibrations. In both categories there is a large proportion of properties with 0-1 additional entries where the levels of vibration are under 20 mm/s. Most properties show additional entries at higher levels of vibration.

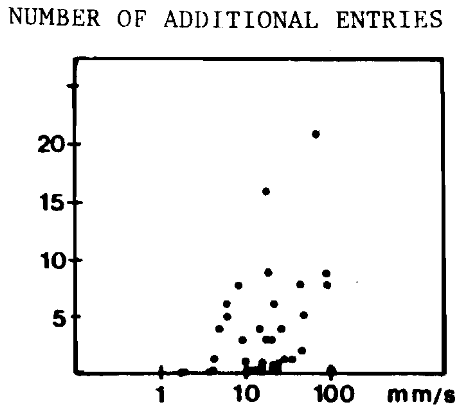
3.6.2. Various Construction Materials

In the case of properties built on rock the significance of the construction material was also examined. Figures 3.5 show results for structures of wood, lightweight concrete, and regular concrete, respectively. Surprisingly, it appears that lightweight concrete is associated with the lowest number of additional entries for vibration levels below 100 mm/s while wooden structures seem to involve a risk of additional entries above 5-10 mm/s.

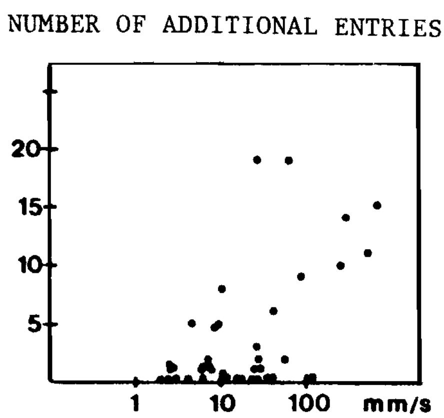
3.6.3. Various Types of Facades

Various types of facades were also examined without taking into account foundations. Three different cladding materials were reported in fig. 3.6, i.e. plaster, sand-limestone and brick. In most instances there were no additional entries reported for cladding materials.

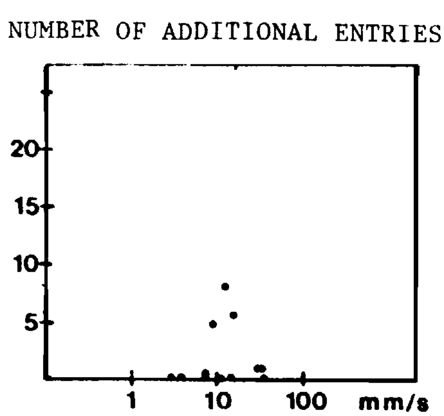
The analysis shows that the greatest number of newly detected instances of damage occurs in the wall-covering material inside the premises and not in the facades.



a. Apartment block on rock

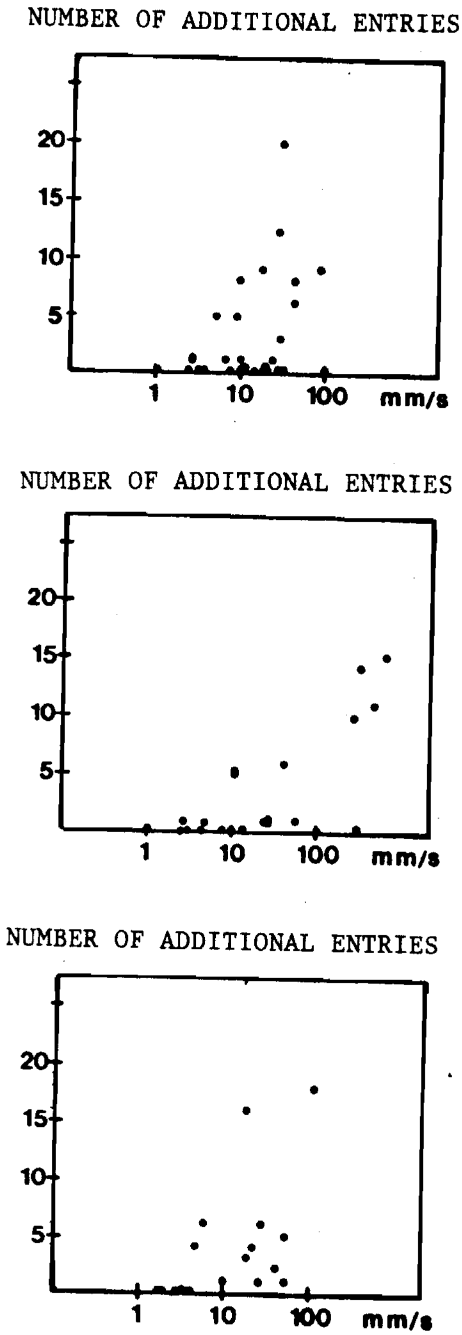


b. Detached houses on rock



c. Detached houses on clay

Figures 3.4 a,b,c. Relationship between newly detected damage in post-event inspections and the measured levels of vibration.

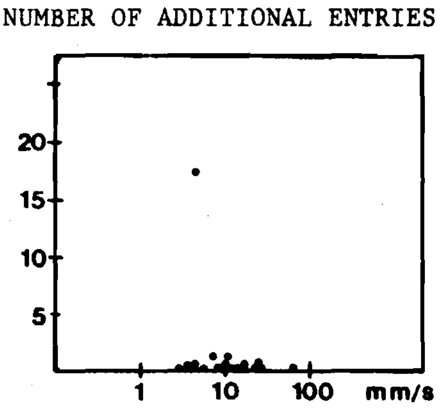


a. Rock - wood

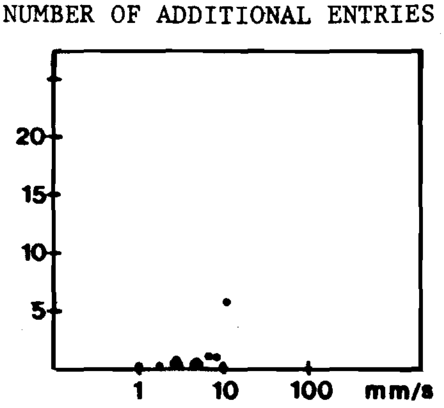
b. Rock - lightweight concrete

c. Rock - regular concrete

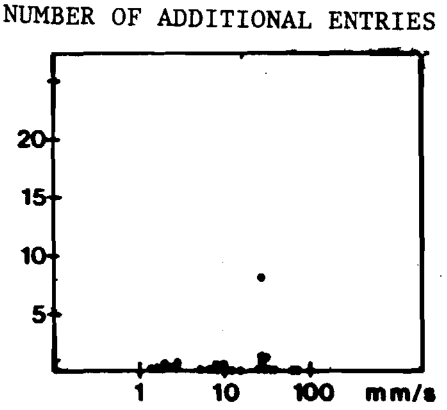
Fig. 3.5. a,b,c. Relationship between newly detected damage in post-event inspections and the measured levels of vibration.



a. Plaster facade



b. Limestone facade



c. Brick facade

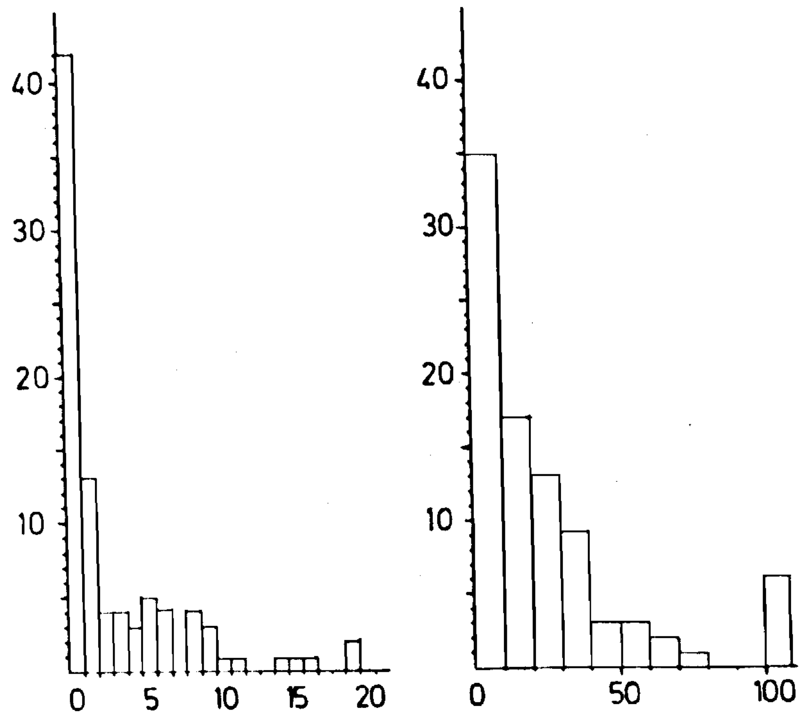
Fig. 3.6. a,b,c. Relationship between the number of newly detected instances of damage at post-event inspections and measured levels of vibration.

3.7. Statistical Analysis

With a view to establishing whether it is common that certain vibration levels and a certain number of additional entries are obtained on adjacent properties, the two histograms shown below were prepared - see fig. 3.7.

There is a lack of extreme values, both as far as the highest number of entries and measured vibration levels are concerned. A total of 91 properties were examined.

NUMBER OF PREMISES / FREQUENCY INTERVALS



a. Number of newly detected incidents of damage

b. Vibration level (mm/s)

Fig. 3.7. Histograms showing the distribution of the number of properties which received a certain number of additional entries at post-event inspections and the distribution of measured vibration levels.

The figure shows that for over half of the post-event inspected properties at most one instance of newly detected damage was reported. Approximately the same proportion of properties were exposed to vibration levels lower than 20 mm/s. From the point of view of this project it would have been better if the examined properties had been distributed more evenly between the frequency intervals, i.e. if the results of the investigation had produced (histogram) blocks of an equal magnitude.

Fig. 3.7 shows that during the time period that had elapsed between a pre-event and a post-event inspection - when work on the ground is in progress - approximately one instance of newly detected damage was recorded on the average. Fig. 3.3 shows that for two properties which were investigated over a fairly long time, the number of newly detected instances of damage is 12-13 per annum. It is thus evident that a post-event inspection ususally results in a number of newly detected instances of damage which are most often associated with external disturbances and not with the ageing of the property that would have been expected (verbatim: which should have been the case).

4. DISCUSSION

It was somewhat unexpected that there is so little difference between various building materials in the structure and also between detached houses and apartment blocks. All properties were therefore included in one combined comparison in fig. 4.1. Some properties were added which had not been reported earlier because of incomplete information concerning foundation and construction. Two lines have been drawn into the figure: The left one indicates the greatest number of additional entries which may be expected on a property when it is exposed to a certain vibration level. The right line shows the lowest number of entries which may be expected for a given vibration level.

The material under investigation has been insufficient to obtain absolute limits. The results give, however, a good indication that it is likely that damage must be expected with vibration disturbances.

Siskind (7) carried out a similar analysis in an American investigation. Because of more comprehensive material it was possible to determine the relationship between the likelihood of damage and vibration velocity more accurately - see fig. 4.2. Similar work was carried out by Medearis (4) who likewise compared vibration velocity with the response spectrum of the vibration in an analysis of the most suitable criteria. His results agree well with Siskind's (7).

From an economic point of view a balance must always be found between the costs associated with keeping below a certain maximum vibration level on the one hand & the expenses which must be expected for repairs of future damage on the other hand. A limiting value must therefore be determined which ensures that a reasonable distribution between these costs is obtained.

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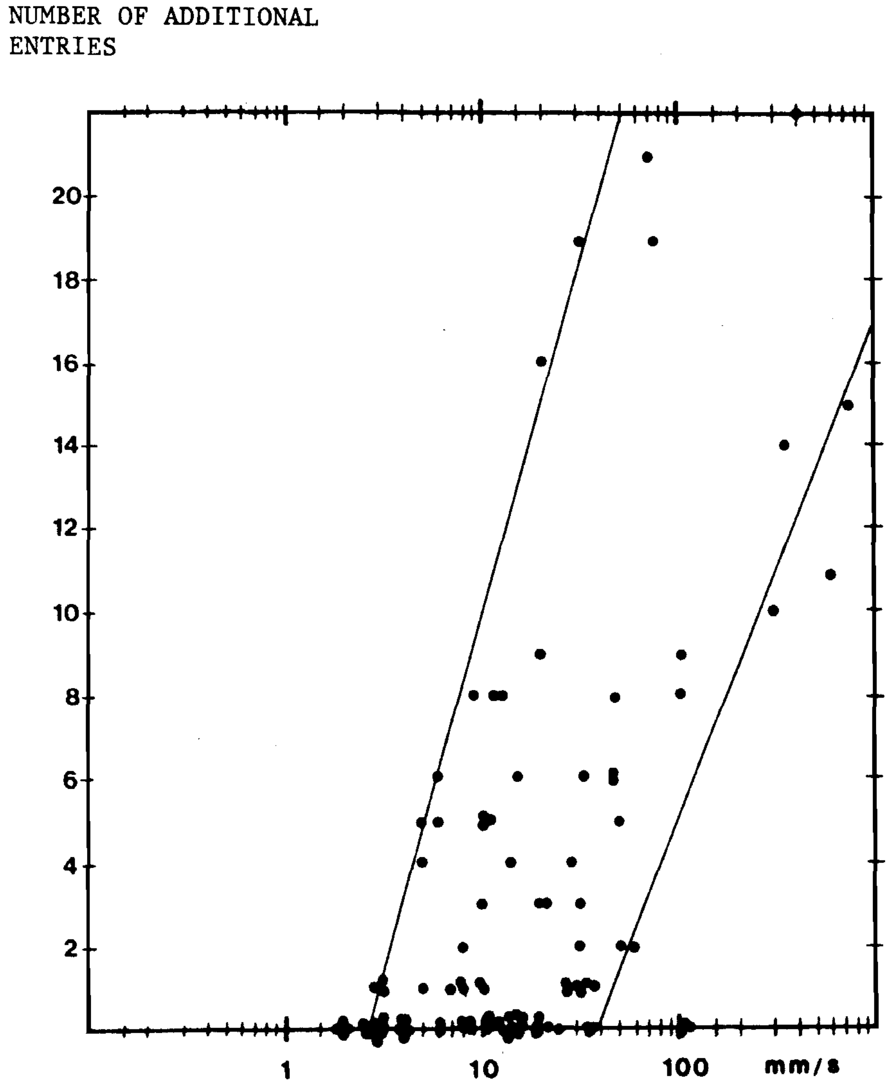

Fig. 4.1. Relationship between the number of additional instances of damage at post-event inspections and the measured vibration level.

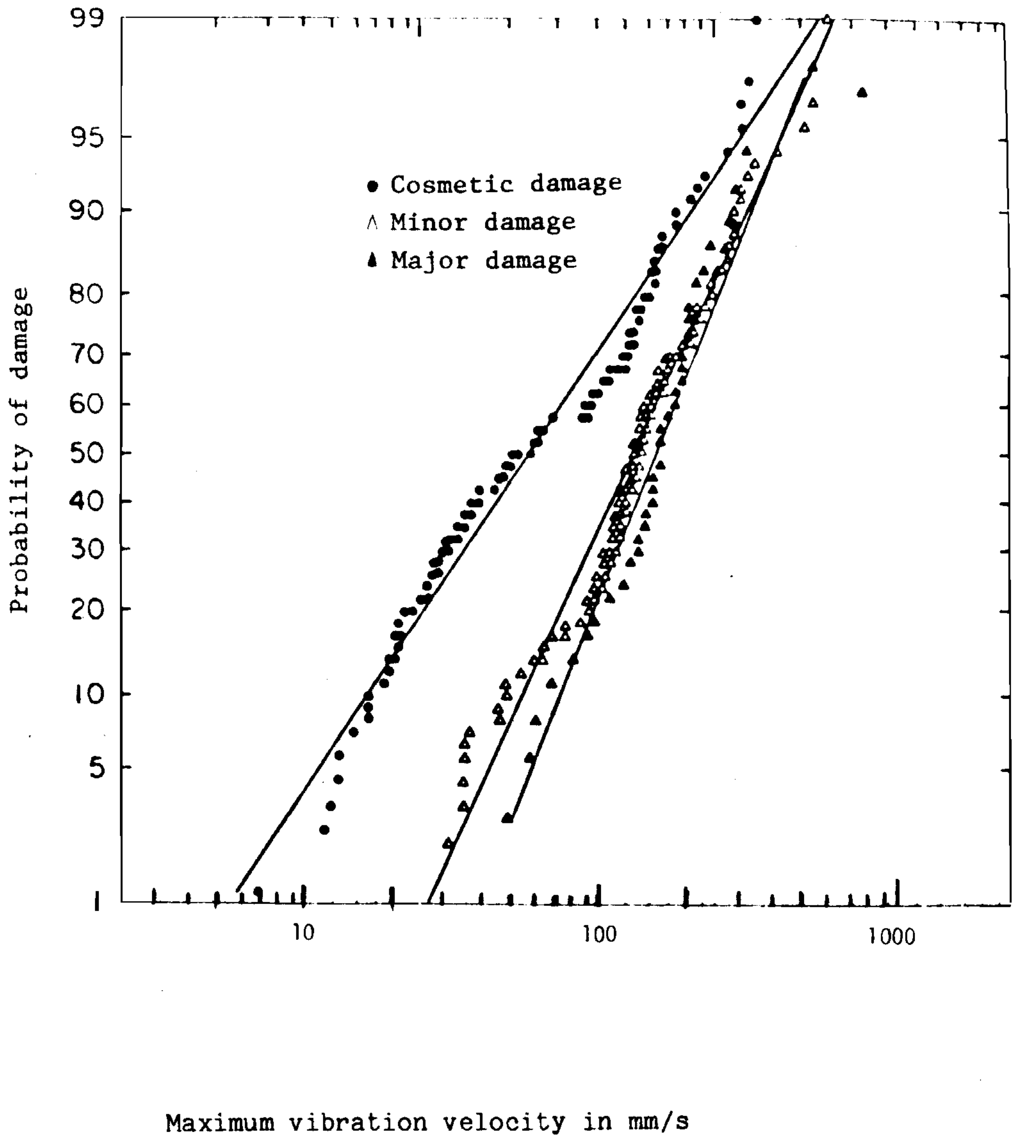

Fig. 4.2. Diagram for the likelihood that damage will develop.

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Editor's Note: The term "vibration velocity" corresponds also to "particle velocity".