There is too great a range in sensitivity of the possible items and structures we might encounter, and too great a range in the character of the vibrations. Appropriate limits could be as low as 0.01-0.001 in/s (0.25-0.025 mm/s) for some exceptionally sensitive items, for example during research experiments, yet damage might not occur at 600-1000 in/s (15,000-25,000 mm/s) for some rugged items at very close range and very high frequencies. Very small charges at very small distances have a completely different effect than large charges at large distances at the same particle velocities. Generalized rules have to be written in a conservative manner to cover an expectation of an occasional more sensitive condition and to gain acceptance by the public at large. For the many specific higher limitations established by the writer for certain buildings or certain field conditions, each was considered a special situation, and few were repeated exactly.

True vibration damage is quite rare, despite the public perception that it is common. In contrast, environmental damage exists in every building, and is often mistaken for vibration damage. If damage is project related, it is more likely to be from block motion, rupturing, cratering, venting, or associated physical effects that are the primary sources of close-in blasting damage, - much of which has been called "vibration" damage. Such block motion occurs out to the perimeter of every blast-ruptured zone. This effect is not prevented by establishing vibration limits, but by controlling the perimeter of the disturbed zone, the "crater zone."

Finally, for the proper implementation of close-in blasting, it becomes more important to consider certain blasting parameters that are less important in the far field, such as the spatial and time distribution of the explosives, confinement, coupling, and rock strength.

BLASTING UNDER AND NEAR UTILITIES AND HIGH-PRESSURE GAS PIPELINES

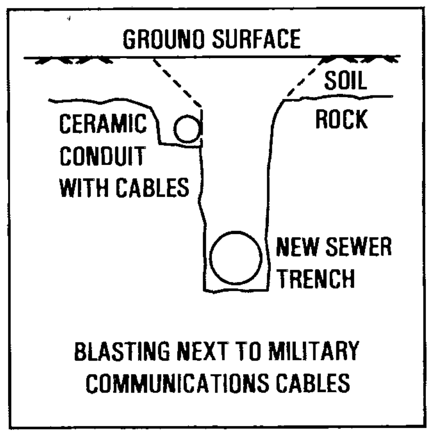
General. The experiences described above also apply to every manner of utility and pipeline. In Spokane, virtually every utility trench in rock that passed down a street with existing buildings had to intersect existing utilities, similar to the conditions seen in **Figure 9-2**. A typical utility trench would intersect up to a dozen or more utilities of various types in every city block underlain with rock. A single project might involve hundreds of such conditions. Many of the lines were amenable to visual and mechanical inspection where they were exposed at intersections with new trenches being blasted or because a new parallel trench was so close that it cut into an existing trench. It was a common experience to be blasting within 2 or 3 ft (0.6-0.9 m) of existing utility lines.

It is well known that buried pipelines and other utilities have been damaged during construction operations. The most common source of damage is that due to equipment operations. Damage usually occurs because the buried line's location is not accurately known and the equipment operator inadvertently digs into it. It is also well known that damage has occurred from ground failures, including landslides, earthquakes, even blasting operations. However, when damage has involved blasting operations, known cases of damage appear to have been the direct result of ground rupture or simply cratering into a pipe whose location was not accurately known, and an explosives charge was inadvertently placed adjacent to it. Such rupture damage is NOT vibration damage. For future planning and for damage prevention, it is important to be accurate in describing the true cause of damage.

Soil-Structure Interaction. On a number of occasions, observers have been surprised to discover the presence of some type of utility line immediately adjacent to trench blasting, being previously unaware of its existence or its exact location. In such cases, the lack of damage was a surprise, but also a very valuable learning experience. Why was there no damage at extremely high vibration levels?

The explanation lies in understanding the nature of the field conditions. The utility line in question receives its input from the ground surrounding it. It is not an above-ground structure. It does not amplify that motion, although a strong rigid pipeline can resist localized input motion being transmitted through a weak soil from adjacent blasting. Nor is there any point loading as long as block motion does not occur. It is actually strengthened, not weakened, by the surrounding ground. It becomes part of a composite structure. It can be thought of as a small lined tunnel, where the "lining" is the utility line. This interaction between the ground and the conduit is known by the general term "soil-structure interaction" or "rock-structure interaction" depending on the field conditions. The concept applies equally to all types of pipelines and utilities, as discussed later under the heading "Buried Pipeline Models." The more direct concern during adjacent blasting activities is block motion which can easily rupture a pipeline. There are important physical differences between elastic vibration and block motion, and differences in the methods to control each.

Ceramic Conduit. An example of one of the more delicate types of utility lines was a ceramic conduit carrying military communications cables in the city of Spokane. If the cables were disrupted, the contractor would be assessed $1000 per minute until the lines were repaired. The conduit rested on basalt and a new trench was being excavated parallel to the conduit and several feet below it, as shown in **Figure 9-12**. Charges were placed as close as 2 ft (0.6 m) below and 1 ft (0.3 m) to the side of the ceramic conduit. No damage occurred and no repairs were necessary. Vibration monitoring was done in the adjacent building because of the additional concern over sensitive switches and other facilities. Particle velocities in the rock next to the ceramic conduit could not be measured, but were estimated to be in the range up to about 50 in/s (1270 mm/s). However, carefully designed blasts prevented block motion damage. There was no block shifting to generate localized high strains in the conduit. Vibration was never considered to be the criterion for preventing damage to the conduit, although it was a criterion for the sensitive switches in the adjacent building.



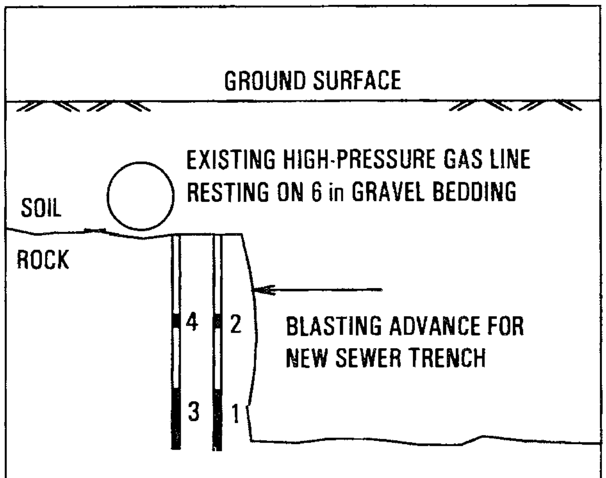
BLASTING NEXT TO MILITARY COMMUNICATIONS CABLES

Figure 9-12

Nearby Electrical Switches. Sensitive electrical switches were located in the communications building beside the blasting operations. An engineer from the facilities expressed alarm over the blasting in the street, so the writer accompanied him on an inspection tour of some of the switches thought to be the most sensitive. They were located on the sixth floor. The engineer had no criteria or test data, but regarded the switches as "extremely sensitive." After exhibiting the cabinet full of the most sensitive switches, the engineer closed the cabinet door with a relatively hard slam, with no comment, explanation or apology, apparently unaware of this strange discrepancy between his concern about blasting in the street and his own actions. The writer then attached a transducer to the switch mounts in the cabinet and asked the engineer to repeat his action. This time, he closed the cabinet door in a much more gentle manner, but the vibration was still far more intense than anything that could have been generated in the street. He was told that the blasting vibrations would remain well below that level, that he would be welcome to watch the monitoring, and that any further information or comment from him would be welcome. The work was completed without any further incident. (See Chapter 15 for more discussion of sensitive switches).

HIGH-PRESSURE NATURAL GAS PIPELINE, SPOKANE

The Project. A trench was blasted in basalt to provide utilities to a military installation near Geiger Field, Spokane. The trench intersected and passed directly under the high-pressure regional natural gas line from Seattle to Spokane, as shown in **Figure 9-13**. The existing line was resting directly on the surface of the rock through which the intersecting trench had to be excavated. The writer was asked to design the blasting plans for the intersection. Explosives charges were placed as close as 18-24 inches (0.46-0.61 m) from the operating, high-pressure line, and below it. **Figure 9-13** shows how this blasting was done. Special blasting techniques were employed. They had been tested many times before in similar situations. The basic concept was to use deck loading, with light burden to an open trench face. The bottom charge detonated first. When the upper deck detonated, there was easy relief to the trench face and downward, with no tendency to crater upward. Only two rows of holes were detonated for any blast, so as not to build up any muck burden in front of any holes, and muck was removed between shots.



BLASTING UNDER A HIGH-PRESSURE GAS LINE (see text)

Figure 9-13

No damage was done. At this extremely small distance, the particle velocities could not be measured, but were estimated roughly to be in the range up to about 50-75 in/s (1270-1905 mm/s). The primary concern, of course, was to avoid block motion, which could easily have sheared the main gas line. Similar utility intersections have occurred in many locations. Compare, also to the 850-mile pipeline discussed later in this chapter.

Corroded, Leaking Pipelines. It is easily recognized that old corroded and leaking pipelines do not have as high a strength as new, recently installed lines of the same type. That increased sensitivity would apply to large strains during earthquakes, as well as block motion from adjacent blasting that did not effectively limit perimeter breakage. As above, however, and for the reasons given, even these old lines are not as sensitive to high-frequency vibrations as usually supposed. An example is that of an old, corroded natural gas pipeline between Boise and Mountain Home, Idaho that was being looped (replaced). Parallel trenching would be taking place generally at a distance of 20 ft (6.1 m), and sometimes as close as 10 ft (3 m). Although the old line was to be replaced, it had to remain in operation until the new line was completed, a process that would take many months. Specifications were requested to protect the old line. Joint consultations were held with the Owner and the Contractor so that all parties would understand the importance of preventing extensive ground cracking and block motion. Recommendations included the stipulation that small test blasts were required if and when new geologic conditions were encountered, or when any significant variations in the blasting design concepts were proposed by the Contractor. If the test blasts were successful, work was allowed to proceed with an approved blasting plan. Rock depths were highly variable, up to 9-10 ft (3 m). An inspector for the Owner measured and recorded ground cracking on a continuing, daily basis. Vibrations were not measured. If this comes as a surprise to the reader, it should not. With an experienced full-time inspector on the job, there was no question about following approved plans. The range of vibration intensities was easily predicted from the blasting plans, if there was any interest in it, but it was not an item related to any decisions. All it would do would be to distract the Owner and the Contractor from the real concerns about ground rupture. As a worst-case precaution, there was radio contact with operators at valve locations who could quickly shut off the gas. It was not necessary. Occasional tests were conducted to determine if gas leaks increased or if new leaks occurred. No damage occurred at any time throughout the project.

More Cross-Country High-Pressure Pipelines. The writer has been involved with thousands of miles of rock trench which have been blasted close to existing high-pressure pipelines. Vibration intensities routinely have been very high for this work, quite often in the range of 10-20 in/s (250-500 mm/s), and sometimes as high as 50-75 in/s (1270-1905 mm/s) at intersections and crossings. There was no vibration damage in any of the cases where the writer was involved, although direct equipment damage has been a common occurrence, and block motion damage is known. Some damage cases have been reported in the literature, but it is questionable whether these were the result of vibration or the result of block motion. In cases where there was damage reported from low levels of high-frequency vibration, the writer would be reluctant to accept it as "vibration damage" without a review of the pipeline conditions and a personal firsthand field inspection. There is a lifetime of overwhelming evidence to the contrary, and an ample history of rupture or block motion damage. Where caution and controversy are expected to exist are those cases where pipeline ruptures have occurred in the absence of a proved, identifiable cause other than increased internal pressure. There are those who will search diligently even out to very remote distances for some outside triggering force to blame. The writer is aware of cases where inexperienced investigators blamed a pipe rupture on a small triggering force at very remote distance, offering explanations which were technically nonsensical because the true mechanism of damage was not recognized.

OTHER SOURCES OF STRAINS ON PIPELINES

Pipe sections are often subjected to very high strains during shipping, hauling, handling, installation and service. As successive pipe sections are welded to the completed portion of the pipeline, the pipe often undergoes very large bending strains before it is finally in place in the ground. The familiar "S" curve formed by the pipeline as it is lifted during installation is often seen to cover many feet of bending and deformation before the pipe is finally in place. When the pipeline route crosses deep water bodies, the bending strains can be especially large because of the large distance through which the pipe must be lowered from the barge surface to the underwater trench or bottom position. Other cases are known where operating pipelines have been supported on bridge hangers which have failed, allowing the pipe to drop into a deep catenary curve, without being damaged. Pipes have been lifted back into place and placed on new supports without ever being removed from full operations. These large strains are not desirable, but they serve as very valuable illustrations of the great strain tolerance of ductile materials. Such strains are far greater than those induced by any type of construction blasting.

Easily observed settlements, displacements and pipe bending are common within trench backfill and for pipelines which are suspended above the ground. Also, large temperature changes can cause surprisingly large deflections, thousands of times greater than that from typical construction blasting. One of the more dramatic examples was the TransAlaska Oil Pipeline which was forced completely out of the ground by thermal expansion when warm oil entered the cold pipe in frozen ground. That case also demonstrates that this welded steel pipeline was stronger than the surrounding ground.

It is common for backfill to settle with time, but the volume of soil in a rock trench is too small to permit enough settlement to be of concern when blasting in another rock trench nearby. There would have to be a more significant shearing mechanism involved. An example might be a pipeline going from a stable rock area through a geological discontinuity into a deep fill of unstable material which would liquefy or settle during an earthquake in large enough volume to generate a large shear stress at the discontinuity. Similar shear stresses could be generated by landslides, but not by settlement within the padding in rock trenches.

DEFINITION OF DAMAGE TO DUCTILE MATERIALS

An associated question is the definition of damage to a welded steel pipeline. When we see pipelines subjected to such extreme bending strains during handling, installation and service, we cannot help but wonder how much they can tolerate, and what is an appropriate definition of damage. How do we define threshold failure in a ductile material? We could probably agree that a loss of strength or a loss of performance would constitute damage. However, if there were a small deformation with no loss of strength and no loss of performance, two questions naturally arise. How would we detect minuscule deformation (from any cause) to a line in service? And, does that deformation constitute damage if no loss in performance could be ascertained?

It is well known that some steel structures have undergone great strains with particle velocities as high as 200 in/s (5000 mm/s) or more at low frequencies during earthquakes, and have continued to serve their design functions.

The question of fatigue is often raised. A structural steel member can undergo large deformations close to its yield strength a limited number of times where there is some loss of strength and the beginnings of metal fatigue. The number depends on the yield strength and how far beyond the yield strength the material was strained. Neither large deformations alone, nor a large number of repeated events at large deformation take place under the circumstances involved in routine construction blasting.

At low strains, the number of deformations becomes relatively unimportant for pipelines.

BURIED PIPELINE MODELS

Many decades of successful construction work demonstrate that we don't need to develop theoretical models of buried pipelines if our only concern is to conduct nearby blasting operations effectively and safely. That technology is well developed and has been practiced for the past 75 years. However, if we wish to understand the behavior in greater detail for other reasons, such as the study of the large ground strains that develop from earthquakes, a model is attractive. In a buried condition, the pipeline is a composite structure, of which the encapsulated steel pipe section is only a part. The composite entity is more like a small-diameter lined tunnel than it is a beam in free air. A further point to consider is that the seismic wave does not strike the pipe as an instantaneous point or planar front, as has been proposed by some researchers. Rather, at the time that the peak of the wave has arrived at one side of the pipe, the wave front has already passed well beyond the pipe and has encompassed the pipe and the surrounding soil. As a similar type of concept, it is well known that soils may form a strong arch over culverts and carry much of the load. In model studies to test the ability of buried structures to resist nuclear detonations, aluminum cylinders buried in sand and subjected to quasi-static loading have shown increases in strength up to 200-300 times more than the buckling strength in air. Results of transient loading have been variable and sometimes contradictory. It seems clear that part of the overpressure is borne by the surrounding soil. There is some indication that this may be independent of the overpressure, but there is also some indication that the backfill may behave as a viscous fluid, probably a function of loading time and stress level, adding importance to the stiffness of the buried pipe. Theories have not been able to predict very accurately the dynamic failure mode and load (hence dynamic strength) for buried cylinders.

The natural gas, pipeline, and blasting industries have been guided to very restrictive ground vibration limitations for pipelines, often based on inappropriate data or models, or simply by adopting residential-type limitations. Some of the first research for the pipeline industry (American Gas Association) developed a model considering ground vibrations to act as a point load on a pipeline. Later, this model was changed to that of a wave with a vertical planar front. Neither of these models is appropriate. Later, the final recommendations combined vibration data from blasting in soil and a low response with a prediction of very high stresses associated with the vibration. Using that recommended industry approach for the 850 mile pipeline discussed below would predict pipe stresses of about 200,000 psi (141,000 kg/sq m) and a failed pipeline. That did not take place.

At the present time, for construction purposes, the simple field solution is perimeter control, not highly restrictive vibration limits. For additional study, please see the references at the end of this chapter.

SOIL SETTLEMENT

It is evident that the soil backfill around a buried pipeline is weaker than the pipe and the rock trench, hence is more subject to various failure mechanisms. The pipe bedding and soil backfill may not always be compacted. Consequently, it is a common occurrence for soil to undergo static settlement over time even without any vibration input. The introduction of large amounts of rainfall, ground runoff, etc. may cause such things as erosion, percolation, piping, or hydrodensification, all resulting in some added strain to the buried pipeline. However, except for cases of extreme erosion for pipelines in soil, where the soil is actually carried away, the limited volume of soil in a rock trench does not allow enough pipe settlement to induce damaging strains. Thus, even though vibration induced settlements in fresh uncompacted soil backfill could take place during blasting operations, the added pipe strains are not known to be of significance in rock trenches. They are small compared to those commonly induced during installation. Similarly, larger scale static settlements in deep soil deposits are common, but without a geological discontinuity they are not usually a matter of concern.

BLASTING SPECIFICATIONS FOR AN 850 MILE PIPELINE

The Project. The above thoughts provide the background for the development of specifications for an 850 mile trenching operation running in very close parallel position to an existing high-pressure natural gas pipeline that would continue in full-time operation, and was a crucial supply line. There may be some value to the reader in learning about the Project Designer's thinking processes involved in finally arriving at a set of blasting specifications. These processes were very typical. The first question came in a brief generic form, "Would a vibration limit of 2.0 in/s (50 mm/s) prevent vibration damage to a high-pressure gas pipeline?" The answer was that of course it would, but was the right question being asked? It turned out that it was **not** the right question. Blasting would take place on the same right-of-way, often at a distance of 14 ft (4.3 m) between trenches, and would often require blasting to a depth of 9-10 ft (3 m) of rock. If a vibration limit of 2.0 in/s (50 mm/s) were imposed it would not be feasible to do such work in a single pass, even with deck loading. It might require several separate excavation lifts, depending on the rock conditions.

Some time later, the question was repeated with reference to a limit of 5 in/s (127 mm/s). This appeared to be based on a review of the American Gas Association criteria (AGA, 1981). The Owner's technical personnel were familiar with these guidelines and had been following them. If the Owner wanted the Project Designer to place a vibration limit in the specifications, the work would be physically possible with a limit of 5.0 in/s (127 mm/s), but it would cost far more than the budget that was mentioned. Of course, the final choice would be the Owner's. In order to do the work in the manner the Designer was discussing and according to the budget he was discussing, the vibration limit would have to be raised to about 12 in/s (305 mm/s). Such a limit was acceptable if used with ground fracture control. Case histories were provided where the writer had recommended the limit of 12 in/s (305 mm/s) on other projects for the same reasons. For example, that limit was used for similar work running from Spokane, WA to Sandpoint, ID. Other projects were described also, as in the case histories discussed earlier in this chapter. In some of those cases, no vibration limits at all were imposed. It was also emphasized that the vibration limits were neither necessary nor sufficient in themselves. If the writer were to be asked to prepare the specifications, the primary element of protection would be ground fracture control and blast design control. Vibration limits would be only of secondary importance.

Some time later, a request came for a specific demonstration program to **prove** that 12 in/s (305 mm/s) would be safe for the project in question. In particular, the Project Owner wanted the program to take place in the area thought to be the worst along the route. There was virtually no soil cover in the area, and the rock would have to be blasted to a depth of about 10 ft (3 m). The rock was a vesicular basalt which had a very bad reputation among local contractors, being described as very difficult to break and requiring very large amounts of explosives (mentioned in Chapter 2). It had an appearance of being much stronger and more brittle than was actually the case. At first glance it looked like ordinary basalt, but a closer look revealed that it had many small vesicles (small holes from small gas bubbles in the lava). It was very difficult to break with a hammer, giving an added impression of strength. However, a perceptive person would notice that it could be crushed slightly under heavy hammer blows, rather than fracturing. Although the vesicles were small, they were very pervasive, and the rock was of lower density than ordinary basalts. Despite repeated blows with a 3-lb hammer, the writer was unable to break rock specimens, although the surfaces gradually wore away under the blows, producing rounded depressions. The reason the ground surface was not breaking during blasting was not because of unusually high strength but rather its weakness under compression combined with the fact that tensile strength was relatively high in comparison to the compressive strength. The local contractors said that the rock had a weak layer at the bottom and a strong layer at the top. That was not true. It was a single, homogeneous material, but it crushed around the explosives charges and did not fracture to the surface. The local practice of increasing the heavy charges to break to the surface eventually caused cratering and violent flyrock. The reaction of the rock to blasting was similar to that of permafrost, and the solutions were the same.

After the first test shot, there was no visible evidence that a blast had been detonated.

Electric cap lines were checked to prove that they had detonated. The stemming was easily removed, and a pole was used to probe the holes to prove that detonation of the charges had occurred. The holes were re-loaded with a slightly longer column of explosives, then detonated again, and the results were the same. The ground surface was not disturbed. By now it was eminently clear that the appearance of the material was deceptive and that it would have to be treated in the same way as permafrost. But, as in the case of permafrost, there was no need to use the large quantities of explosives that were "common knowledge" among the regional contractors. Optimum results were obtained when using only half the quantity of explosives used by the regional contractor who had installed the test section of pipe before these demonstration tests began.

The Tests. For the tests on the buried pipe, sixteen blasts were detonated in two trenches. One trench was 7 ft (2.1 m) from the pipe trench wall, the other 14 ft (4.3 m). Ground vibrations, pipe vibrations and pipe strains were measured for different charge sizes and distances. Blasting took place as close as 7.5 ft (2.3 m) from the pipe (7 ft or 2.1 m from the trench wall). For the final production project, the distance between trenches would be 14 ft (4.3 m) or greater. **Figure 9-14** shows a cross-section of the test area.

At the closest location of 7 ft (2.1 m) from the pipe trench, the charges were deliberately increased beyond an acceptable design to simulate a worst-case scenario, and were able to blow away the wall of rock between the trenches, but the pipe was not damaged. **Figure 9-15** shows pipe gages being tested after a shot was detonated in the right foreground.

After the tests on pipe durability were completed, it was considered desirable to demonstrate for future bidders that excellent trenching results could easily be obtained with a moderate powder factor and conventional procedures, without generating flyrock. **Figure 9-16** shows a nearly perfect blast, with excellent mounding for easy excavation, and virtually no lateral overbreak. A diamond or "five-spot" trench pattern was used, one hole per 17 ms delay and surface initiation, with a small deck charge in the center holes only (with no need for a separate delay on the top deck). The holes were string loaded with cartridges of water gel. The pattern was 6 ft x 4 ft (1.8 x 1.2 m) with a center hole. There was a charge of 10 lb (4.5 kg) per hole. The powder factor was only half that used by the contractor who excavated the test trench to install the test pipe.

The surface ground vibration data is shown in **Figure 9-17**. For comparison, the writer's prediction curves are superimposed on the plot of the data. It can be seen that the data cover a typical range of results. Funding limitations for test instrumentation

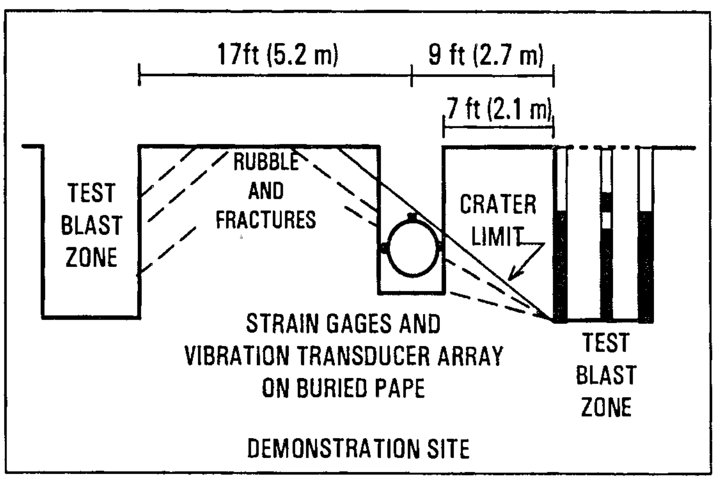


Figure 9-14

required measuring the ground vibrations at greater distance with conventional instruments and extrapolating to the ground surface above the pipe location, giving a maximum of 63.3 in/s (1583 mm/s), far above the level of 12 in/s that was being discussed. Motions measured directly on the pipe were less than half this amount on average, as seen in **Figure 9-18**, although the results were somewhat variable. Motions at depth are often only about half of those at the surface, but an additional factor here was thought to be the strength of the pipe itself in resisting the ground motion transferred to localized or limited portions through the loose backfill. For on-going repeated blasting on a project at a distance of only 7 ft (2.1 m), it would be appropriate to be more conservative and assume that this type of excessive blasting might generate particle velocities ranging up to as high as 50 in/s (1270 mm/s) on the pipe.

The highest strain at a gage location was 1494 microstrain. For that strain, the calculations for stresses showed a tensile hoop

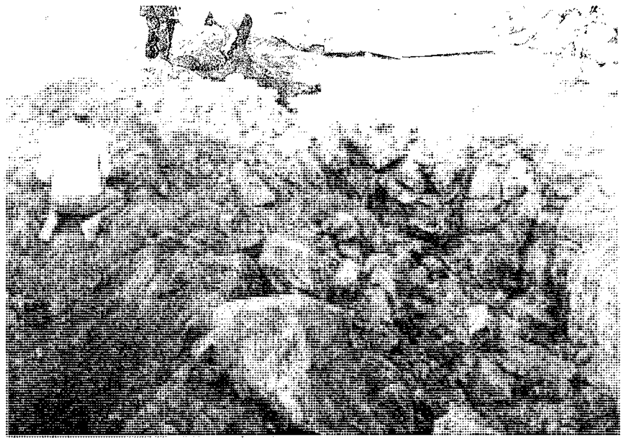


Figure 9-15

Figure 9-16

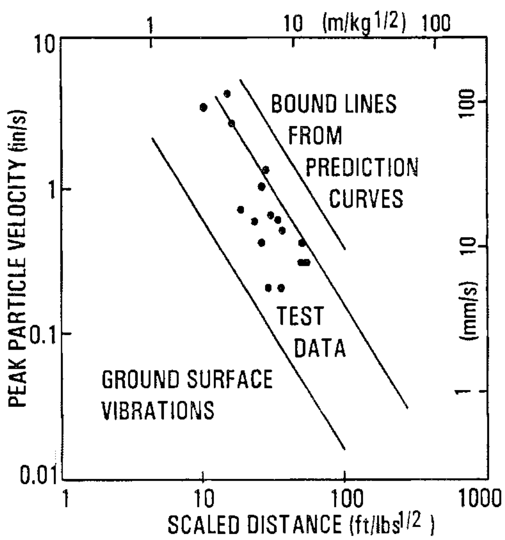


Figure 9-17

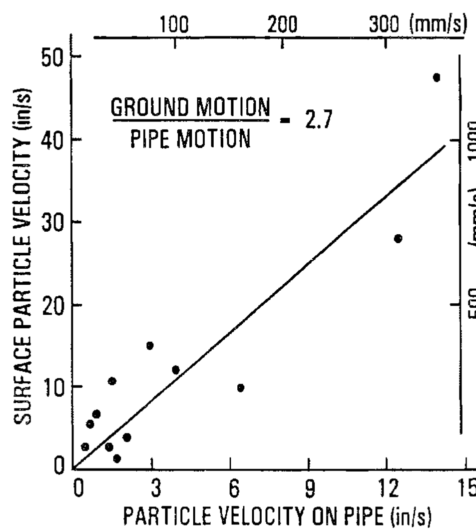


Figure 9-18

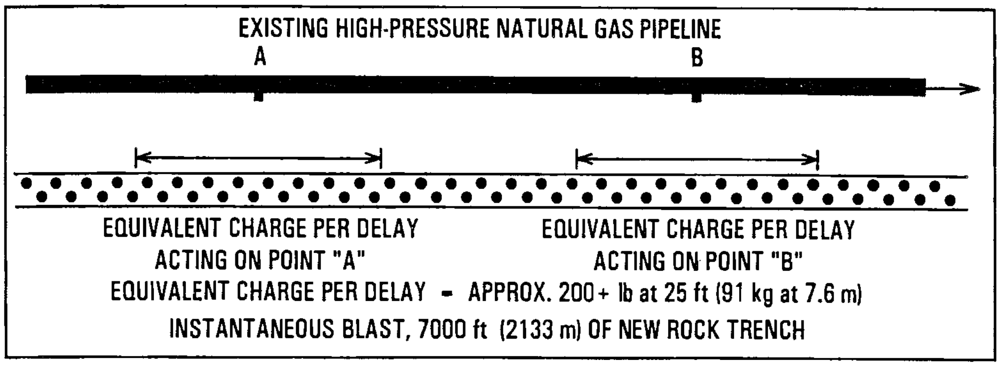


Figure 9-19

stress of about 36,000 psi (2531 kg/sq cm), and a longitudinal stress of about 55,000 psi (3866 kg/sq cm). Extrapolating to non-gage locations, it was calculated that the maximum particle velocity at the pipe side was about 30 in/s (762 mm/s), the strain was about 3000 microstrain, with a stress approaching 85,000 psi (5976 kg/sq cm). For the presumed operating conditions on this project, we expected an additional hoop stress of about 37,000 psi (2601 kg/sq cm) and an additional longitudinal stress of about 18,500 psi (1300 kg/sq m). It was assumed that there would be no reduction in static stress due to any mobilized passive resistance in the backfill.

Project Specifications. In the specifications prepared by this writer, the principal method of preventing damage to the existing pipeline was control of ground fracture, as was done for the corroded, leaking pipeline in Southern Idaho described previously. The specifications required that small test blasts would be required at the beginning of each new geological setting or each new type of blast design, and the work would only be allowed to proceed if the results were approved by the Project Manager. Ground fractures were to be examined after each blast. If they extended more than one half the distance to the existing pipeline, work would be stopped and a revised blasting plan would be developed. Vibration monitoring was a secondary matter, but it served a useful purpose in showing conformance to approved blasting plans. The peak particle velocity was not to exceed 12 in/s (305 mm/s), but that was not considered by this writer to be the factor which would or would not prevent damage.

Unexpected Final Proof Test. During full-scale production work, one of the contractors became frustrated with his inability to blast a suitable trench in the troublesome vesicular basalt and detonated four unscheduled blasts at a distance of about 25 ft (7.6 m) from the existing, operating pipeline. After the first trial blast of about 75 ft (22.9 m) in length, three additional blasts were detonated, consisting of about 7000 ft, 3000 ft, and 4000 ft of trench (2133 m, 914 m, and 1219 m), with all holes detonating simultaneously. No delay caps were used. No vibration monitoring was done. Project management personnel had monitored ground vibrations when blasting was taking place at distances under about 20 ft (6.1 m), but did not monitor at greater distances and did not monitor these blasts. We can estimate that particle velocities were in the range of 50 to 150 in/s (1270-3810 mm/s) along various parts of the route for more than 14,000 ft (4267 m). We can estimate the vibration intensity based on the quantity of explosives detonating simultaneously within a length of trench that is a little longer than the distance to the point of interest, as shown in **Figure 9-19**. Interviews with project personnel indicated that there was no surface evidence to suggest any ground fracturing or block motion as far as the existing pipeline, and the Construction Manager concluded that it was not necessary to uncover it for examination.

There was more likelihood of damaging the line by uncovering it than there was from the vibration.

Those four blasts would be fairly typical of decades of earlier cross-country pipeline trench blasting, although more controlled blasting is typical of work done in the last 30 years.

If the lack of damage comes as a surprise to the reader it should not. As long as the ground surrounding the pipeline remained undisturbed, there was insufficient strain to generate damage in the pipeline.

Canadian Section of the Line. Some time later, the writer received several calls from a Canadian firm that was going to be serving in a consulting capacity on a continuation of the line into Canada. The caller indicated that there was an intention to impose a vibration restriction of 2.0 in/s (50 mm/s) on the work. His firm was interested in knowing the basis for the 12 in/s (305 mm/s) limit that was used for the U.S. portion of the work. His comments suggested that his firm would have to justify to Canadian regulators any increase over 2.0 in/s (50 mm/s). The writer was happy to give him a history of the U.S. portion of the work, along with a description of the demonstration tests and a mention of the four large instantaneous blasts. It was obvious that the caller was extremely nervous about exceeding the standard limit of 2.0 in/s (50 mm/s). The writer wished him well, but never heard what was finally imposed on the Canadian section. As far as the writer knows, they imposed the limit of 2.0 in/s.

Summary Comment. Unlike above-ground structures, buried pipelines are not able to amplify the ground motion. Normally, a buried pipeline can not move more than the surrounding ground (from external vibrations). When the source of motion is some distance away, with relatively long wave lengths, the pipe is expected to move entirely with the ground. The situation may be different when blasting takes place at very small distances, as discussed in some of these case histories. For shots at very small distances, there is a localized pressure over a small radius being transferred from the rock to the bedding sand to the pipe. In some of the tests where block motion occurred and ground cracks extended into the pipe trench, the bedding material flowed around the pipe and prevented damage to the pipe. In a few tests, even rock backfill and blasting muck moved around the pipe, causing scratching or scarring of the pipe coating but no measurable distortion of the pipe itself. It is emphasized that these effects occurred during blasting tests at extremely small distances. It would not be appropriate to apply these conclusions to distant blasting where long lengths of a pipe are involved in a single wave length of the ground motion.

There are discrepancies in some of the models which predict damage to buried steel pipelines at various low particle velocities. The recommended models or regulated levels are sometimes as low as 2.0 in/s (50 mm/s), sometimes in the range of 5 or 6 in/s (127-152 mm/s), compared to the ever increasing number of observations where no damage occurred at particle velocities in the range of 50-150 in/s (1270-3810 mm/s) for close-in blasting. These actual documented experiences serve to verify the estimates of such vibration intensities during the previous 75 years when vibrations were not usually recorded for pipeline trenching. This disparity is too great and too consistent to regard as random chance.

The reader should understand that these comments are not intended as recommendations for particle velocities of 50-150 in/s (1270-3810 mm/s). To the contrary, such intensities should be allowed only under very carefully controlled circumstances at small distances. Rather, the emphasis should be placed on dealing with the actual causes of damage, namely ground rupture, block motion, and the like. Ground control procedures may or may not include vibration restrictions. Vibration monitoring can be a convenient method of determining conformance to approved blasting plans. **High particle velocities are not recommended for more distant blasting and are not needed for the benefit of the work.**

Corps of Engineers Tests. These observations are in good agreement with other case histories where similar high vibration intensities were reported not to cause damage. As one example, the Corps of Engineers conducted similar tests where a very large blast generated a large crater that exposed one end of a pressurized pipeline that projected into the crater zone. It was subjected to a particle velocity of 168 in/s (4267 mm/s). Although the line was displaced by the blast crater, no visible damage or leaks were developed, contrary to theory. (Bart, 1979).

A Final Word of Caution. This writer has witnessed cases of ground fractures extending from the blasting zone out to distances where the ground vibration intensities were less than 2.0 in/s (50 mm/s). If there had been a buried pipeline in that zone, it could have been ruptured, but not from vibration.

When it is reported that pipeline damage occurred at a vibration intensity of 2.0 in/s (50 mm/s), the response should be to question the report to find out the actual cause of the damage and offer more explanation. If it occurred from block motion at a location where the vibration intensity was only 2.0 in/s (50 mm/s), the mechanism should be described and explained. Elastic vibration is not the cause of ground fracturing and block motion. Elastic vibration begins where the craters and ground fractures end.

It is not unusual for reports of vibration damage to pipelines to be made by construction personnel in the field. An investigation is often required to determine the facts. If blasting operations took place nearby, they should be included in the investigation, but the investigation should begin with a review of equipment operations, the most common source of pipeline damage.

At the same time, the reader should not let **this writer or anyone else choose criteria that make him uncomfortable**. The writer recommends setting up field tests that will verify the proposed criteria and/or blasting procedures. These can start with cautious testing, then continue with incremental increases while conducting careful measurements and observations.

TUNNELING IN RESIDENTIAL AREAS - PORTLAND TRI-MET

Out of the writer's assignments on several hundred tunnels, a significant number have been located in metropolitan areas, and many of those have passed through residential areas as well as commercial or industrial areas. A few are mentioned in this book and are selected individually to illustrate certain technical questions or public relations issues. The Portland Tri-Met project shares the usual public relations issues with all such urban blasting projects, but also included questions about landslides (covered in Chapter 5), complications

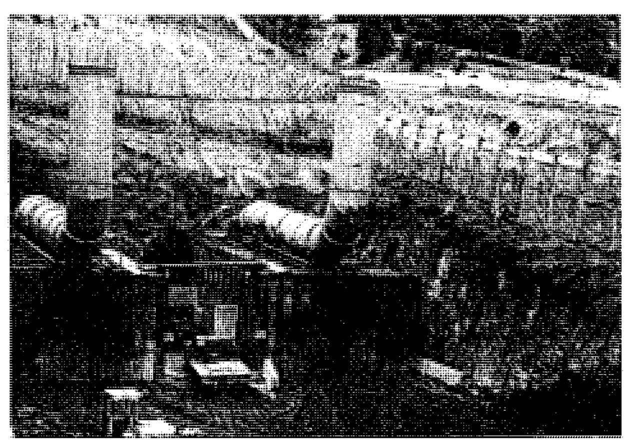


Figure 9-20