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"THE PHILOSOPHY OF WHEN TO REPAIR"

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ABSTRACT

Repair strategy tends to be reactive rather than being based on the functional objectives of the structure and risks to them. The use of risk engineering techniques (ie systematic approaches) lead us to recognition that there is an economic and value engineering dimension to "the philosophy of when to repair". Such an approach is considered relevant to developing rational means of extending the Life of Welded Structures. The concepts and methods introduced are illustrated with case histories.
Keywords:
Risk, repair, safety, reliability, economics, inspections, information systems.

INTRODUCTION

The days of unlimited resources to repair and maintain structures in their original as built state are over. Yet it is considered impractical to adopt the "Roman approach to Durability Management" whereby the engineer responsible for the structure was required to carve his name in the stone and if the structure fell down within 40 years his head was cut off.

The approach set out here-in is based on total quality management principles whereby objectives are defined and performance is assessed against the objective. As the life of structures progresses so the objectives may change. If damage occurs then the functional objectives as well as the consequence of the damage should be reviewed to decide when to repair.

Normally some form of deterioration or damage will have occurred for there to be a need to consider repair. This may either be due to inadequate original design or construction, deterioration in use or some unplanned event which has caused damage. Abrasion, corrosion, fatigue damage are examples of deterioration in use; whereas, earthquake, collision, fire and explosion are examples of events which due to their occurrence or severity may lead to the need for repair.

It is seldom practical or necessary to restore a structure to its original condition. Indeed, attention is paid to ensuring that designs and specifications have a reasonable level of damage tolerance. For example, corrosion allowances and ductility requirements are often included in specifications for this purpose. As structures approach the end of their economic life so it is to be expected that some of these reserves will be consumed. Furthermore, attempts to restore structures to their original state can be both impracticable and counter productive.

Most structural designs are prepared on the basis that the structure will be inspected regularly or with the implicit assumption that warnings of failure will be exhibited sufficiently in advance of catastrophic conditions being reached for their consequences to be mitigated. If for no other reason, such prudence is justified by our lack of experience of whole life performance of our structures. It is recalled that the materials and fabrication techniques employed today are hardly a generation old and most civil engineering structures are, to a degree, unique. Whilst the designer will have attempted to anticipate reasonable levels of use or abuse, it is seldom in the economic interest of the owner to prescribe or anticipate all the possibilities, even if it were possible.

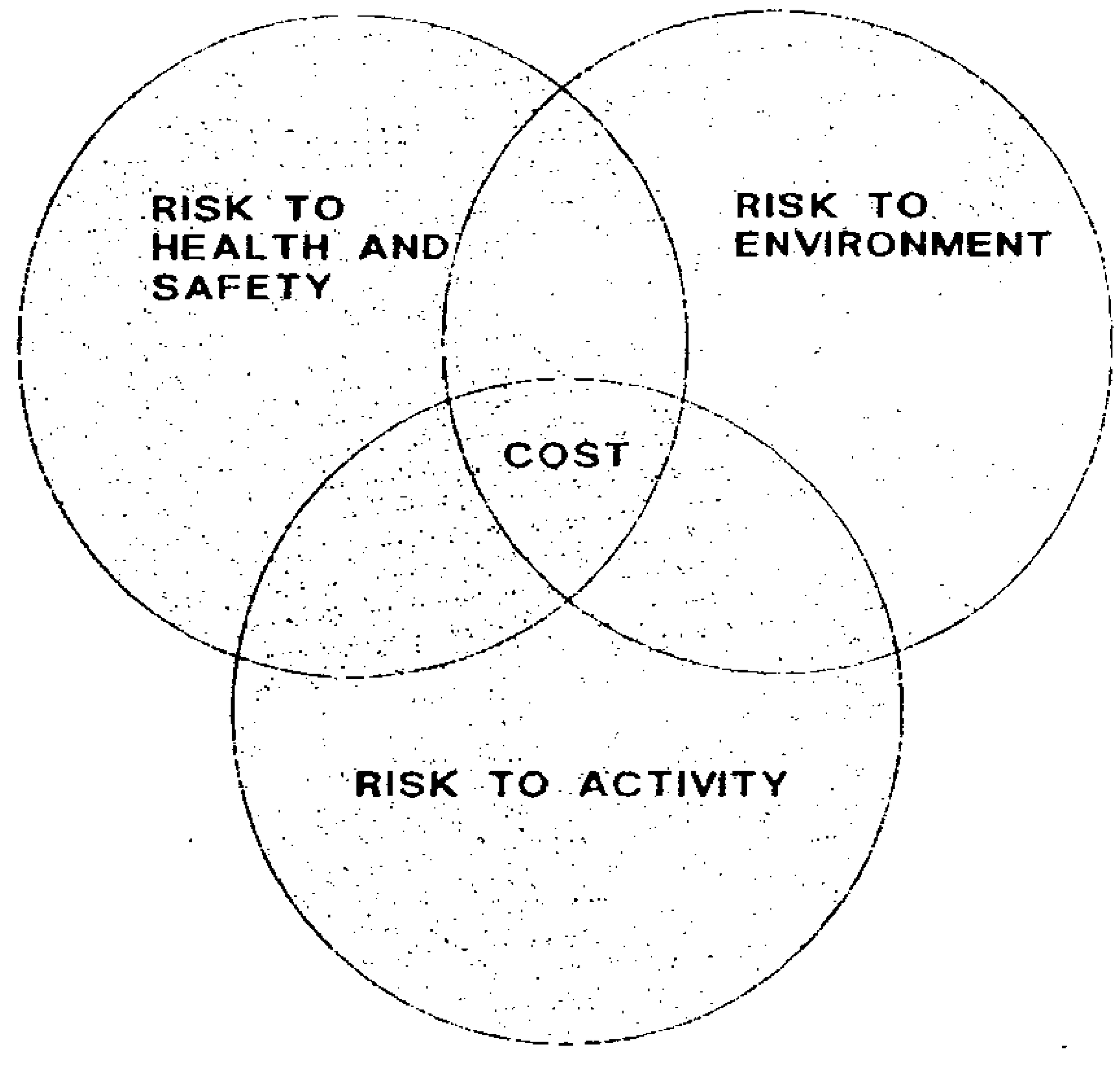

Figure 1
Engineering Risk Factors[2]

RISK ENGINEERING

It is useful at this point to address the issue on a holistic basis rather than consider the remedy for each element of damage piece-meal. The structure exists to serve functional objectives. Damage may have increased the risk to those objectives. Repair will be necessary when the risk to the objective is unacceptably high. It follows that Risk Engineering techniques can assist in addressing the issue: "When to repair".

Risk Engineering is:

A general term applied to systematic methods of hazard and risk identification, assessment, evaluation, creation of risk management strategies and performance monitoring.

As of 1st March 1993, it is mandatory for all United Kingdom Chartered Engineers to, "Take a systematic approach to risk issues"[1&2]. The risk issues referred to are illustrated in Figure 1 and the basic risk engineering cycle is shown in Figure 2.

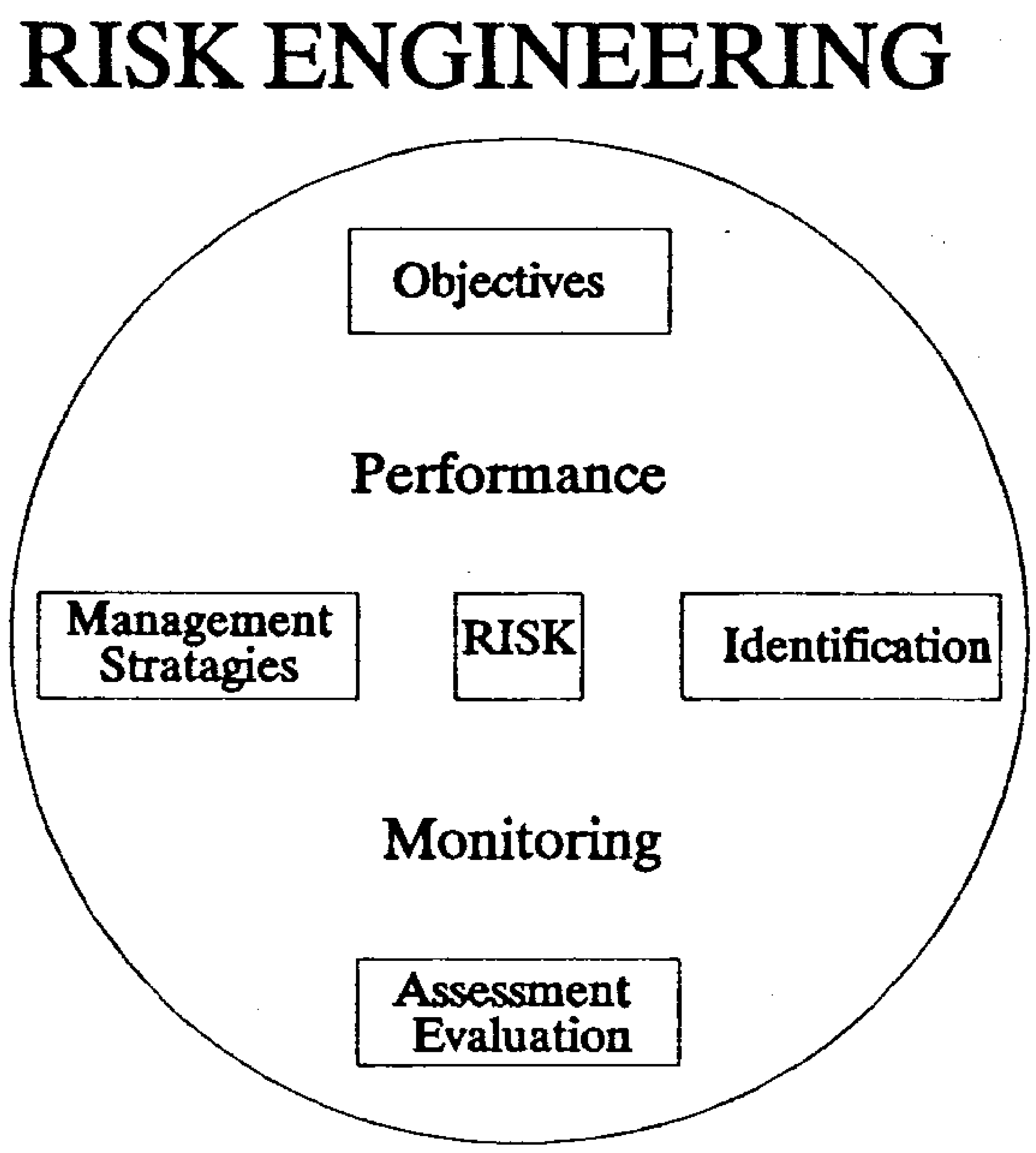

Figure 2
Risk Engineering Cycle

RISK ASSESSMENT OBJECTIVES

As emphasised by the UK Engineering Council, systematic risk assessment is an iterative process and useful results can often be achieved by simple analysis without resorting to more sophisticated techniques. All that is presented in this paper is predicated on the principle that "Professional judgement is by far the most important tool in risk management"[2]. Risk assessment is an aid to judgement not a substitute for it. The point is that there are limitations and uncertainties associated even with the most scientific methods and complete databases available. Not the least is the possibility that analysis of past data will not necessarily lead to good predictions of future performance but it may well help. This can be an issue of considerable importance because the damage itself may be an indication of a change or a limitation in the understanding of past performance. It follows that an objective of risk engineering should be to help to create foresight, an essential element in deciding when to repair.

The objective functions of risk assessment are summarised in Figure 3, they are: safety, environmental, performance, reliability, control of time scales (programme) and cost/earning capacity (budget). The alternatives in parenthesis apply to the special case of risks within project. Not surprisingly, objectives of risk engineering are also the objective functions of Total Quality Management and in the context of repair need to be considered on a whole life cycle basis. However in civil engineering the principles of "whole life" costing tends to be honoured as a concept more than as a practical basis for decision making. The whole topic is still ill defined and "information limited". The concept of service life is implicit in most structure design codes and standards but the value is seldom explicitly stated. There are exceptions, for example the UK bridge code BS 5400 specifies 120 years. This at least provides a basis on which to start to construct a durability plan by considering the risks to service life and identifying strategies to minimise them on a cost benefit basis.

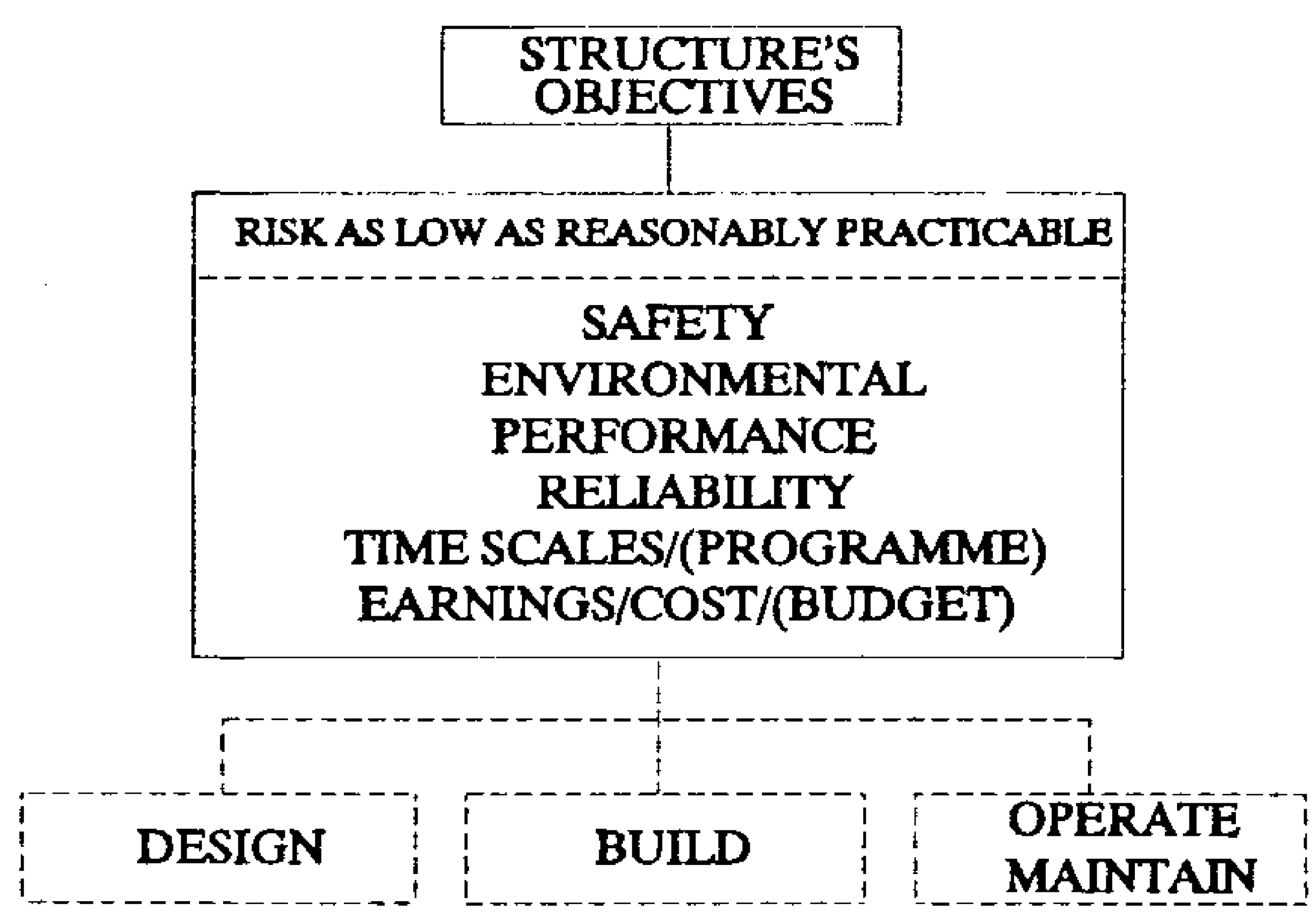

Figure 3
Objective Functions of Risk Engineering

Most of the risks faced in deciding when to repair are probabilistic in nature, but the probabilities are so uncertain that their calculation is of limited use as a sound decision making tool. Too frequently it is argued that the calculation of values will lead to decision takers being convinced even though the basis for those numbers is little more than a guess. The statistical validity of the data used is suspect and even the best trained professional's ability to guess probabilities from limited information is not good and may be over-optimistic[10]. For this reason Figure 2 has shown Assessment and Evaluation in the same box. It is recommended that assessment should precede evaluation, so that effort spent on evaluation is limited to those areas where it will make a valid contribution to sound decision making.

If we address any of the seven objective functions of risk assessment, Figure 3, in isolation we are likely to reach an unbalanced conclusion. For example, reliability isolated from safety, timescales and cost is practically meaningless. It is suggested that it is useful to use the headings as risk management starting points when considering any major engineering decision, not the least when to repair.

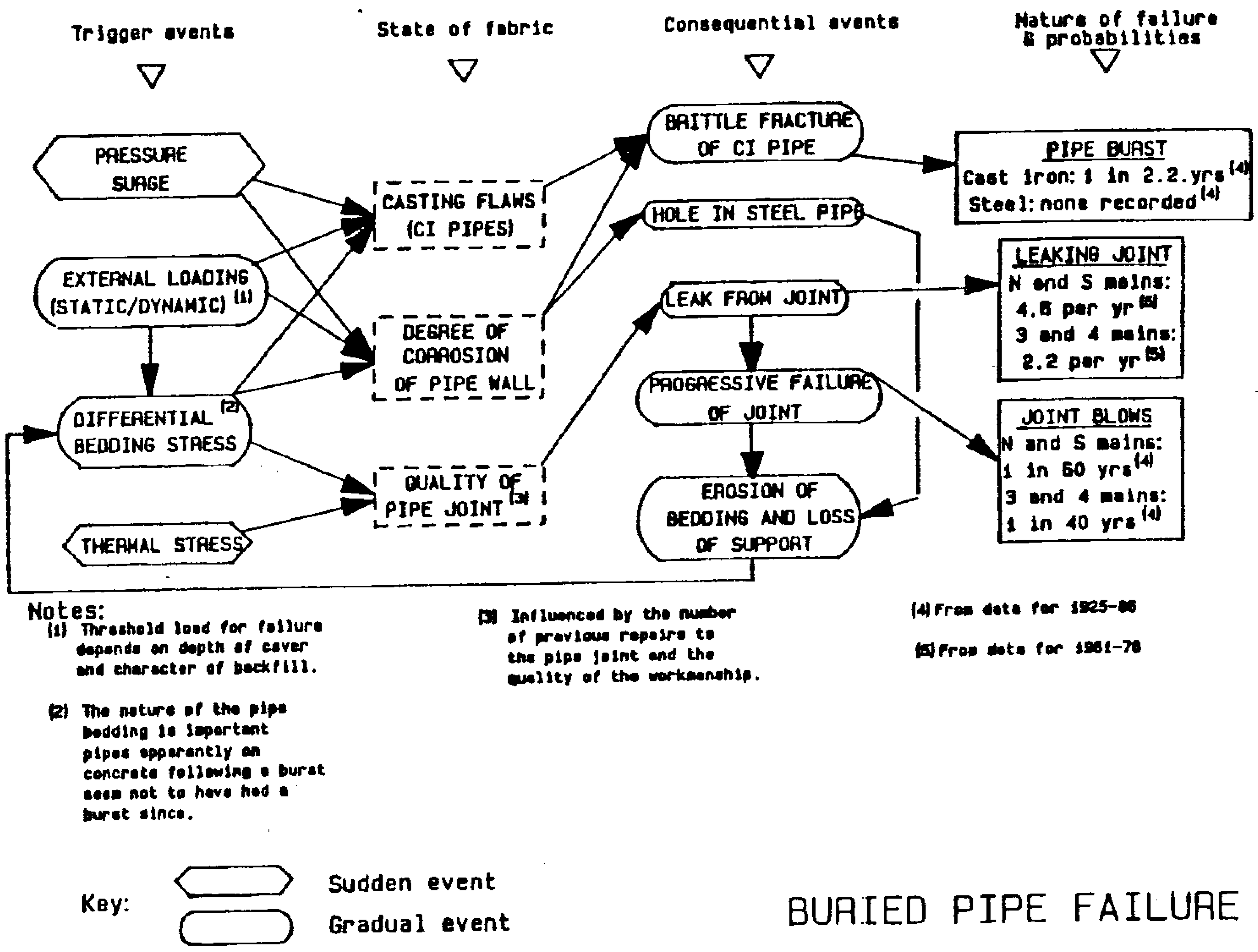

Figure 4
Interaction Diagram

Whilst risk engineering needs to be systematic and recorded as it is executed, it does not have to be sequential. Indeed interaction diagrams such as those prepared by Von Lani[3] Figure 4 are particularly important in civil engineering reliability assessments because the processes leading to failure are often interdependent. Consider the following scenario as an illustration: thermal stress, which is one of several possible trigger events associated with pipe-line failure could produce a leak from a pipe joint in poor condition. This unnoticed leakage could cause loss of support and induce differential settlement stresses which, in turn, depending on the degree of corrosion in the pipe wall, could lead to a pipe burst through brittle fracture. Failure has occurred as an indirect consequence of thermal stress via a feed-back loop initiated by leakage from the pipe joint.

INSPECTION AND STRUCTURAL DATA BASES

The decision to repair needs to be based on an understanding of the existing state of the structure and not just the original design assumptions. To a degree at the design stage, the uncertainties can often be managed by decisions that reduce risk at little or no additional cost. This luxury is seldom available when considering when and how to repair. Furthermore, the position considered may well be without precedence. To restore the balance, the repair engineer may have the benefit of in-service performance history, normally denied to the original structural engineer.

Clearly the quality of the in-service performance data and the results of inspection will be of great importance in making economic decisions on repair issues. Owners do not always appreciate the economic value of good data collection. Some owners may consider the use of longevity studies to both review the adequacy of their current policies and assess the cost benefit of changing them.

In-service inspection can be seen as part of a strategy of observational risk management. Until recently, at least, the approach to inspection has tended to be passive then reactive for welded structures. The inspection requirement has been to demonstrate compliance with specification which may have been set at as high a standard as is practically achievable during construction, rather than based on a value engineering approach. Routine visual observations are backed up by detective work under extreme pressure to establish: information and a strategy to control a problem that has progressed to such a point that it makes itself clearly evident either visually or by encroaching on serviceability limit states. Very occasionally, extreme limit states are reached and catastrophe follows. The availability and use of non destructive testing is helping to improve the position but the approach is still largely reactive and the data collected controlled by practicality and testing methods for existing structures rather than the need for information to take economic engineering decisions. The consequence has been that although a very great deal of experience of the use of welded structures in civil engineering has been accumulated, design and the decision to repair is still by and large a "data limited activity" relying on judgement and codes of practice, the basis for which is also fairly uncertain.

However active approaches to risk management are used successfully to save considerable cost in other branches of engineering. For example, geotechnical engineering is also a "data limited" activity where it is often impracticable to obtain sufficient data to make accurate predictions of behaviour. Either a large and, often costly, factor of safety has to be included or observations of the behaviour of the structure are used to trigger additional preplanned action. The observational approach is described by Muir Wood[4] as applied in tunnelling.

This method is referred to here because it presents a systematic approach to one of the primary constraints on making a decision to repair. The "data limitation" generally includes not knowing enough about what has happened, the properties of the materials and their current state. This leads to the conclusion that keeping adequate records and structural monitoring results can have considerable cost benefit in real terms. The emphasis on real terms is deliberate because there may have been a tendency to regard the cost of repair as an insurance item leading to a devaluation of the apparent benefit to the owner himself of record keeping. Well kept records, preferably on a Structural Information Managements System should be regarded as an asset in itself as well as a means of reducing the cost of future investment in structures.

Figure 5
Stonehenge

Where this is the case the question is not whether regular non destructive testing should be undertaken but how it can be most cost effectively executed. The world wide interest in smart structures indicates that the point is beginning to be appreciated.

FUNCTIONAL OBJECTIVE

The next step in determining when to repair, must be to decide what the future function of the structure will be. This may not necessarily be the same as its past functions. This opens up the need to consider the economics of investment in repair.

An extreme case illustrates the point. Stonehenge, Figure 5, stands as it has for more than two thousand years on Salisbury Plain in the South of England, a partially collapsed structure. To suggest that it be restored to its original structural form would create uproar because its function has changed. It is no longer a religious temple, except to a few. Instead it is a monument to our historic past and its partial collapse is part of its history. However should it be that we are faced with further imminent collapse or further damage leading to risk to the public or loss of the historic record, public opinion would prompt action to prevent the damage.

Stonehenge may not be the best example for this conference on extending the life of welded structures, but the recent restoration of Paddington Railway Station roof[5] demonstrates how heritage issues can have an impact on the decision making processes. Being a building, listed by the UK Government as being of historic importance, replacement of the structure was not an option. As far as I can ascertain the first welded structures were built in the 1920's but few if any substantial structures were built until after 1936 [6]. Perhaps we will soon be facing heritage incentives and constraints on the repair of welded structures.

More usually the change in function results from a change in customer demand. For example, Flint points out that "The two mile long structures of the Severn Crossing (between England and Wales) have been strengthened to provide adequate safety for increased vehicular loading following changes in road transport during the 1970's. The design traffic loading has been increased by up to three times that used for the original design to cater for changes in commercial traffic into 21st century."[7]

There is a natural tendency to regard repair as just a cost rather than an opportunity. This need not be the case and a more entrepreneurial approach may be considered. Benefit may be achieved by deliberately seeking increased functionality as part of the repair scheme. The strengthening of an oil production platform in the Arabian Gulf, Figure 6, combined an extension to the original production facilities with support from a new platform. By this means, the economics of the whole scheme were improved over that which would be achieved if either project had been undertaken independently. This was achieved by addressing the value of repair as well as its cost.

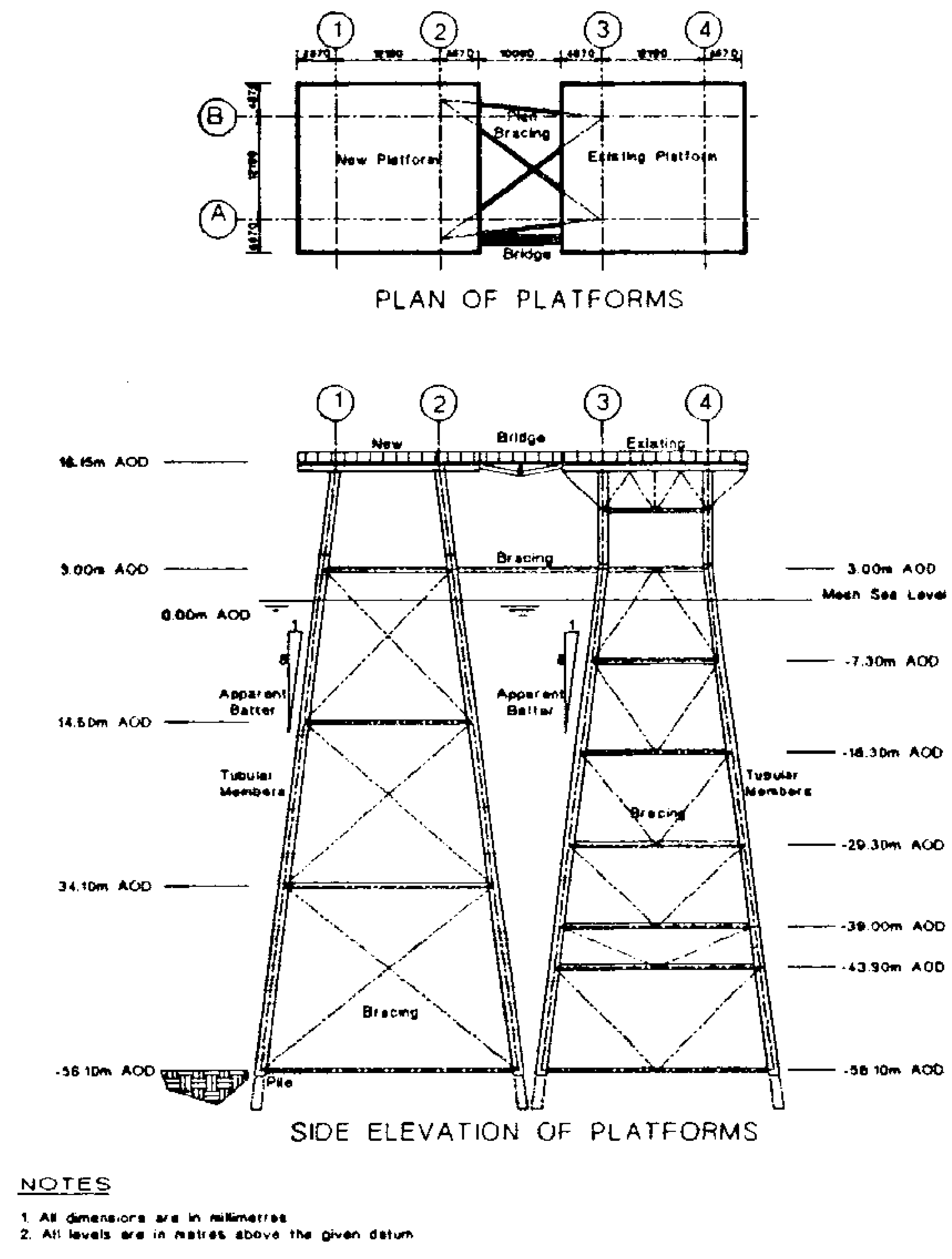

Figure 6
Two platforms joined

THE "DO NOTHING" BASE CASE

Faced with the need for such a fundamental approach it is often valuable to systematically examine a wide of range of options on a cost benefit basis. The original design may provide a reasonable base case for such an examination, often favoured if the objective is to demonstrate the maximum loss from the damage. However, it has been found that the "do nothing" option is usually a better basis for systematic decision making. For this case the objective is to discover whether and for how long the damaged structure will fulfil the current (or alternative) functional objectives.

This approach has four specific benefits:

- It automatically reveals a need to repair. Since the "do nothing option" is the lowest cost option to execute, an unsatisfactory outcome indicates that some form of repair or abandonment is required.
- It involves investigation of the existing condition and will therefore be the first that should be considered if there is need for early action. eg to prevent further early deterioration or collapse.
- If the option is not satisfactory then an understanding of the minimum improvement required is revealed.
- It can provide a realistic start point from which the addition of value to a facility can be combined with a need to repair.

EXAMPLE OF THE USE OF THE "DO NOTHING" BASE CASE

A northern North Sea platform had been hit, on its wooden fenders, just above mean sea level, by a supply vessel. The fendering was seriously damaged and had to be removed to allow inspection of a large dent in a main leg of the platform. The effect of the dent was initially assessed using loads from the structural analysis data base maintained for such purposes.

This is a case where there is, no practical means of restoring the structure to its original condition. Most of the dead weight taken by the now dented area had been transferred to the un-dented area simply because it provided the stiffer load path and was capable of taking the load applied. The approach was aimed at discovering the most cost effective method of ensuring that the structure could resist the extreme design loads and have a durability commensurate with the remaining operational life of the structure.

A first assessment was made using the AMTE method[8]. The platform was analyzed to identify the "at risk of undetected impact damage" members for detailed inspection. Some member distortion was identified in members remote from the dent. Buckling/ultimate strength analyses were undertaken to establish that although some loss of strength had occurred the factors of safety against collapse were still above the minimum required in original design. Structural strengthening was unnecessary.

However the long term safety and reliability of the platform was at increased risk due mainly to the vulnerability of the dented area to a second impact from a supply ship. Qualitative assessment is sufficient to show this to be a substantial risk and a major testing programme would have been required to assess the consequence, with the likely outcome that protection would be required in any case.

A repair was developed to reinstate the fendering using a split steel sleeve grouted to the damaged leg. Particular attention was paid to corrosion detailing and ease of installation in this splash zone location.
See Figure 7.

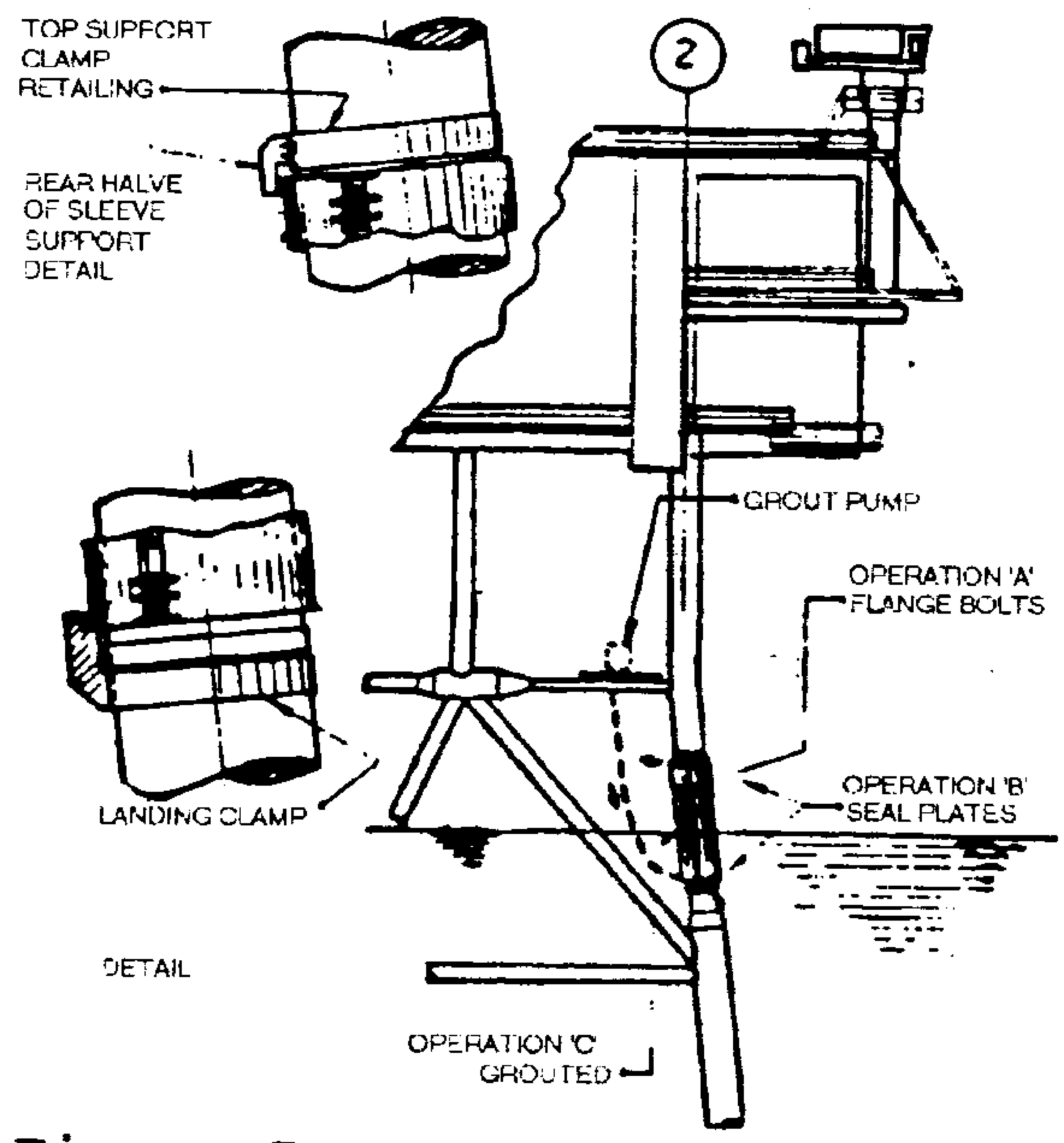

Figure 7
Repairs Following Impact Damage

STITCH IN TIME CASE

Many potential repair conditions arise as a result of progressive and often accelerating deterioration. Fatigue and corrosion damage are examples. Figure 8 expresses the issue temporally. It is useful to consider four stages in the development of the history of the repair

- The critical stage is reached, if and when the damage has progressed to a point where it contributes to a failure or at which the risk of failure is considered unacceptable. In either case the use of the structure is prevented by the damage and in the former there may have been a safety loss. This is referred to as the "Critical time"

- Prior to this there will usually be a level of damage at which it is most economic to undertake repair. This is referred to as the "Economic time"

- A finite time is required to design and install a repair. This is referred to as the "Repair duration"

- There is also a time before which the process of deterioration remains undetected. This is referred to as the "Detection time"

The first priority of a risk management strategy has to be to ensure that the Detection time occurs before the Critical time.

The objective of an economic strategy is to ensure that the Detection time precedes the Economic time by more than the Repair Duration.

The Repair Duration is not a fixed period in that the costs of repair increases rapidly when too short a period is dictated by circumstances.

Once repair becomes necessary the cost of repair tends to increase rapidly with increased deterioration. It is for this reason that the economic time precedes the critical time sometimes by a large margin. Of course the difference will be influenced by the cost of money but the real escalating cost of repair tends to dominate. The philosophy is contained in the old english proverb "a stitch in time saves nine".

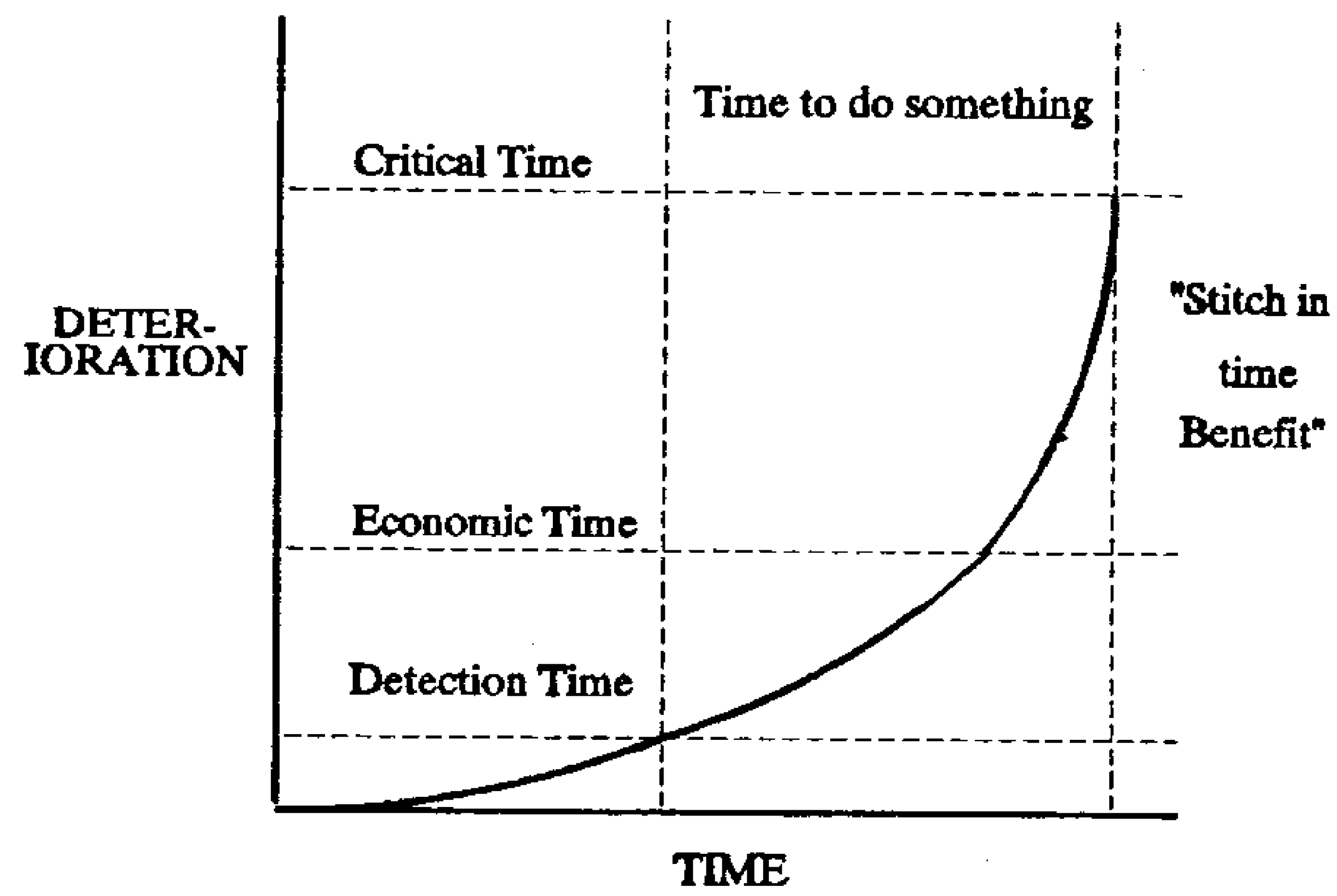

Figure 8
Deterioration Time Plot

In practice, the rates of deterioration can be difficult to predict to the sort of accuracy required to control the deterioration on a cost benefit basis due mainly to the data limitations. The performance of the structure itself can provide the best basis for prediction if the record of performance is well kept. This observation is particularly relevant to fatigue behaviour.

EXAMPLE OF STITCH IN TIME CASE

A North Sea platform like many others had fatigue cracking in some of the joints in its conductor frame. A fatigue analysis was undertaken to correlate with existing and future inspection results. The analysis correctly identified the known problem areas and identified, on a ranking basis, the likelihood of further damage, which on inspection was found to be consistent. The computer programmes used also provided diagnostics which demonstrated the source of the damage in terms of types of wave, and member types. The outcome was a realisation that the main source of damage was the vertical wave loading from relatively small but frequent waves on the frame itself and the very high stress concentrations in certain of the joints. The "do nothing" base case had revealed further deterioration which in view of the consistency of the earlier predictions seemed reliable. In effect the "critical time" was less than the planned service life. To improve the life of the joints to a normal design standard for the remaining service life would have cost more than the original structure. The fatigue damage would escalate from secondary to primary structure if left and had already started to do so. As often is the case the "detection time" had already exceeded the "economic time". So within the constraint of ensuring that an economic scheme was devised and implemented, the repair was needed as soon as practicable.

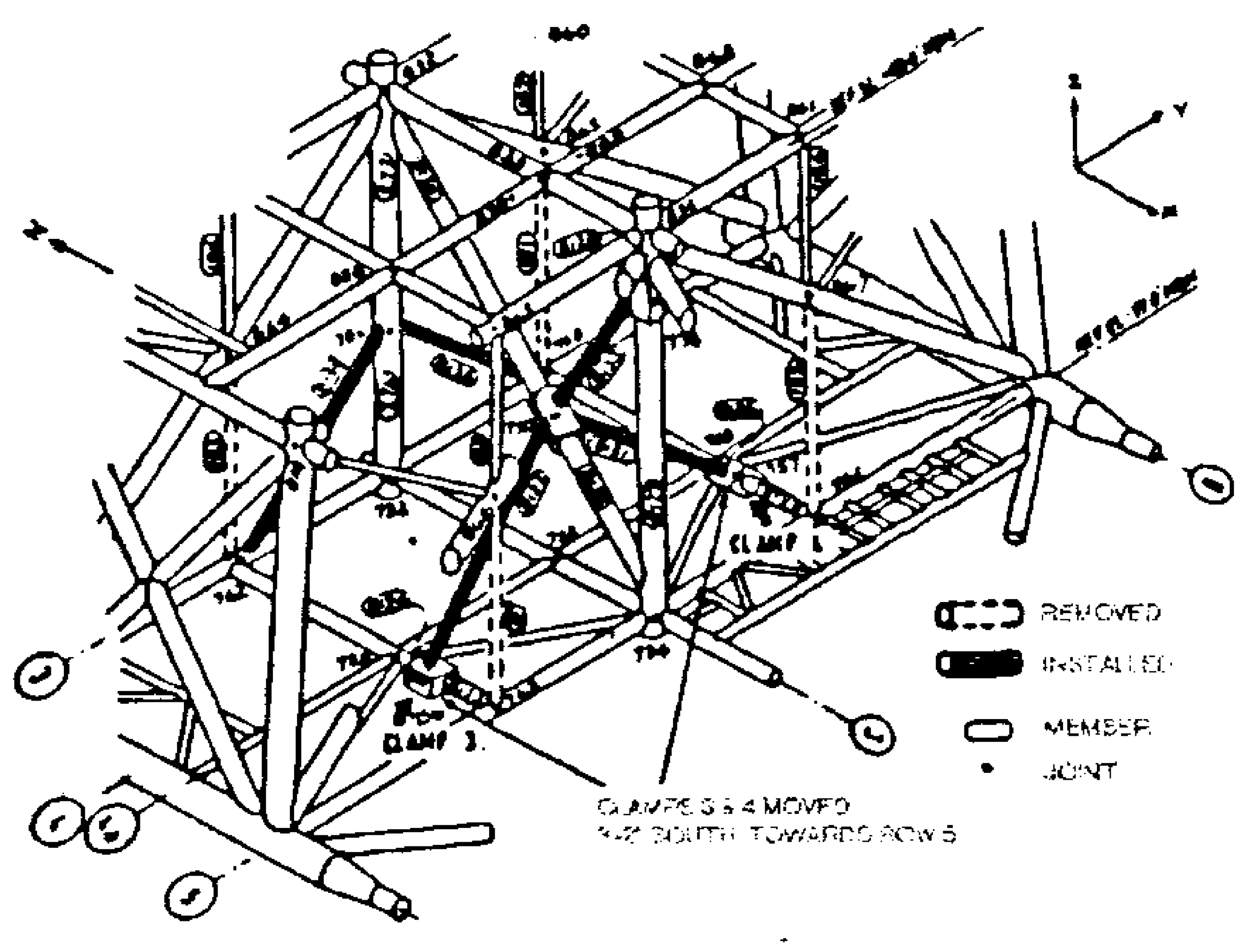

Figure 9
Stitch in Time Repair

The repair installed is shown diagrammatically in Figure 9. It involved the removal of four members, the installation of 5 new members and the strengthening of a joint using underwater habitat welding. The concept of the repair was to provide some overall life improvement but far more importantly it was intended to alter the load paths so that the consequence of future damage would be manageable should it occur. An inspection plan was devised to monitor performance and provide the necessary triggers for action inherent in this "observational" approach to risk management which had been adopted.

URGENT REPAIR

The nature of the damage may be such that there is no need to consider the "do nothing option" as a solution. This is often the case after accidents have occurred. The decision when to repair becomes a matter of priorities, balancing the degree of functional restoration against the time required to obtain information about the damage and organise the necessary resources to achieve the repair. In this case, a fundamental understanding of how structures work, detailed planning ability and an understanding of the risk to programme is particulary necessary.

EXAMPLE OF URGENT REPAIR

A platform located in 140m water depth Northern North Sea suffered almost complete loss of strength in one leg during installation. A pile section weighing 240 tonnes, dropped and severely damaged a corner leg of the jacket structure at a depth of 92 metres. The key economic requirement was to enable load out and installation of modules to continue in order to achieve production of first oil on programme. The method chosen from a number of rapidly considered alternatives was selected because it restored 70% of the original load carrying capacity of the leg by simply repairing the ring stiffener. Examples of risks and management strategy are provided in Table 1

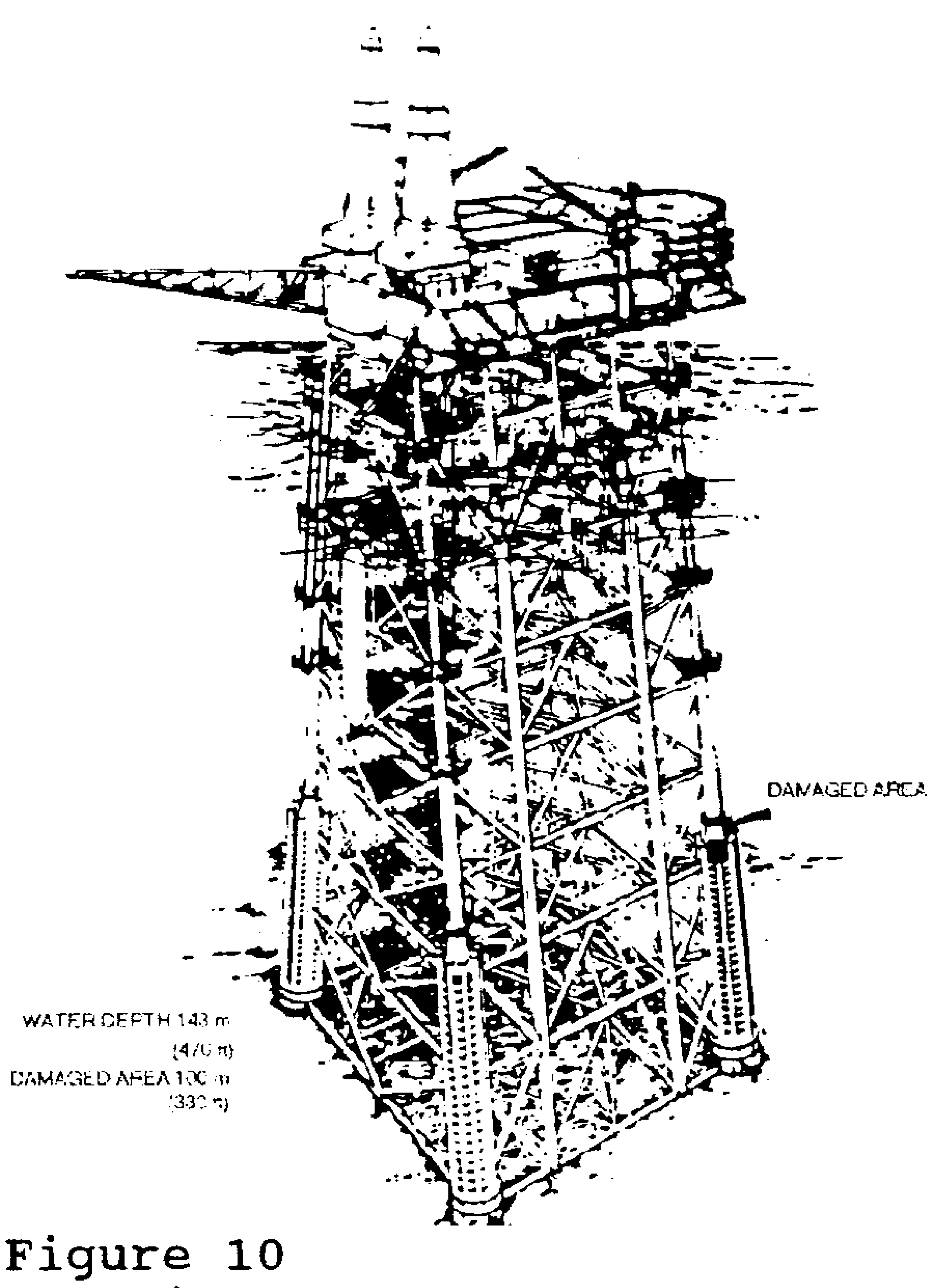

Figure 10
Location of damage

EXAMPLE OF RISK	MANAGEMENT STRATEGY
Lack of fit between stiffener and repair plate.	Design underwater bolts to force fit the plates (approach similar to that used when rivetting plates)
Inability of divers to undertake the repair at 92m water depth.	Deploy diving superintendent to design team at critical stages so that buildability built into design.
Repair not ultimately acceptable to Certifying Authority.	Continual liaison with a "listening to convince" approach.
Possibility of blame inhibiting flow of information	Design principles were adopted that avoided investigation of original design, which was not the issue. eg strength and durability of repair to be as good as the undamaged structure.

Table 1 Examples of Risk and Response.

CONCLUSIONS

1 Risk Engineering, preferably within a Total Quality Management culture can provide a valuable aid to deciding when to repair.

2 It is beneficial to take a holistic approach to the assembly of the risk engineering plan.

3 The early definition of functional and risk engineering objectives is fundamental to the approach.

4 Quality of the in-service performance data and the results of inspection will be of great importance in making economic decisions on repair issues. The incentive to collect and keep such data may have been reduced by transfer of risk to insurance. Where this is the case it is an example of risk transfer diverting attention from efficient risk management and thereby increasing overall costs.

5 The "do nothing" option provides a logical basis for deciding when to repair and is usually preferable to the original design for comparison with options for repair.

6 The need to repair should be regarded as an opportunity as well as a cost, by this means value may be enhanced.

7 Deterioration processes usually lead to exponential rates of repair cost increase with time. It follows that early repair can have high cost benefit. Lack of detection can reduce the benefit.

8 Risk engineering can assist in determining the timing and planning of emergency repairs.

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ADVANCED TECHNOLOGIES IN THE MAINTENANCE OF STEEL BRIDGES

Chitoshi Miki[1]

ABSTRACT : Recent developments in the field of maintenance technology on steel bridges have been listed and considerations for the maintenance plan, advanced diagnosis technologies, monitoring and retrofitting technologies have been discussed.

KEYWORDS: steel bridges, maintenance, diagnosis, monitoring, retrofitting

1. INTRODUCTION

Bridges play key roles in the road and rail networks that support our society and economic activities. Since elements of the social infrastructure are provided in order of demand, major road and rail lines were built first. In other words, the greater the importance of the line under bridges are built, the earlier the technology used to build it; moreover, the bridges are likely to be subject to greater loads. These are vital data from the point of view of bridge maintenance, and civil engineers involved in the maintenance of bridges must be aware of them.

Bridge maintenance technology has traditionally depended on the experience of the engineers performing the inspection. However, only a small number of such experts are available. In addition, the number of bridges that are approaching the end of their design life is increasing. Recently, we have started to face the problem that faults such as fatigue, which had not been experienced much in the past, are now starting to be reported, and it is becoming difficult for operating bodies to provide adequate maintenance.

Recent developments in information technology have raised the possibility of major changes in bridge maintenance practice. Advanced sensors and monitoring technologies not only hold the promise of replacing older techniques that depend heavily on the experience of the inspection engineers, but also make it possible to perform more accurate inspections. Even in the evaluation of deterioration, which was once the realm of experienced engineers, advanced information processing technology offers the prospect of new approaches. Here we report on the application of advanced technologies to the inspection, evaluation of the deterioration, repair and retrofitting of bridges.

2. MAINTENANCE PLAN

The development of a bridge maintenance plan starts with a decision on a maintenance policy. Recently, concepts such as the Life Cycle Cost (LCC) and asset management have been proposed as a method for bridge maintenance planning. The concept of asset management has high potentiality to be the beneficial tool of the management of existing bridges. These tools are now available as commercial software packages and are gradually being introduced into actual bridge management schemes(1). However, in order to establish practical tools, the implementation of several separate technologies will be required.

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The process of developing a maintenance plan for an individual bridge starts with predicting its state of repair and evaluating the damage that has occurred. **Figure 1** indicates the schematic performance curve of bridge structures. Normally, immediately after the completion of construction, the performance of a structure will much exceed the required level, according to the factor of safety and the degree of structural redundancy. The problem is to maintain structural performance at the required level throughout the period of public service, this is set at 100 years in current Japanese highway bridge design specifications. Maintenance, involving a combination of inspection, repair and reinforcement, must be performed so as to minimize the total cost. Every stage has an associated cost, so it is not a simple process. The present author has recently suggested that in addition to direct costs, it is necessary to consider social and broader economic costs such as those associated with closing traffic access or restricting passage across the bridge within LCC(2).

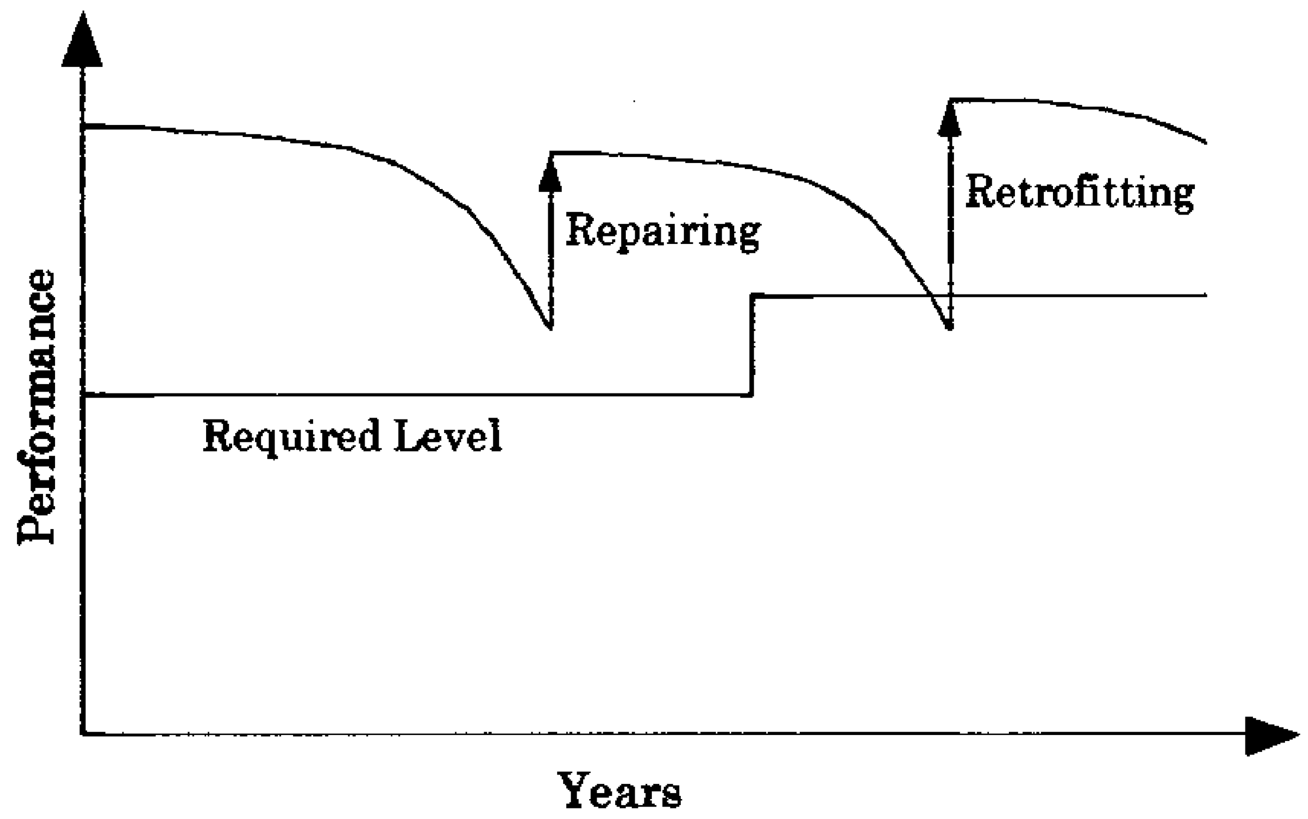


Figure 1. Performance Curve of Structures

The performance curve of a structure can be predicted to some extent if the damage mode can be estimated. It is also possible to correct the predicted curve by inspection and monitoring. Physical deterioration in steel bridges includes corrosion and fatigue of structural members, wear of the bearings, delayed fracture of the high-strength bolts and deterioration of concrete decks. Among these modes, fatigue is the most important maintenance items.

3. BRIDGE DIAGNOSIS TECHNOLOGY

Let us examine several recent technologies for the diagnosis of the state of repair of existing steel bridges. With the application of advanced information technology and progress in analysis methods, a number of technologies have developed rapidly.

(1) Measurement of vehicle weight

A major cause of fatigue is live loading and in particular, the increased load associated with the overloading of trucks. When investigating fatigue, it is of prime importance to determine the number of trucks that cross the bridge and the degree to which they are loaded. Methods for performing this task include embedding axial load scales and special lane sensors have been developed and are being applied. The weigh-in-motion concept, which was developed in the United States in the 1980s(3), uses the bridge beams themselves as balances; this method is both simple and economical and a number of methods for implementing this system have been proposed. The assessment of fatigue in bridges can be performed with information of the weights of passing vehicles and their frequency distributions and the frequency distribution of the stress fluctuation range, which will be discussed in the next section.

(2) Measurement of stress range frequency for fatigue damage assessment

Stress measurement is an effective method for estimating fatigue. Most fatigue cracks in bridge members are surface cracks that initiate from the toes of welds. If the stress in the vicinity of a weld toe is known, the severity of the fatigue can be judged immediately. The standardized method of such stress measurements and a evaluation of the relative degrees of fatigue has been used in Japan(see **Figure 2**)(4). Strain gauges should be installed at locations where there is a possibility of fatigue, and stress should be measured over a long period of time, usually one week, with actual loading under normal operating conditions. The measured stress variation records are processed by applying the Rain-flow method and the linear damage rule and the degree of fatigue damage is calculated.

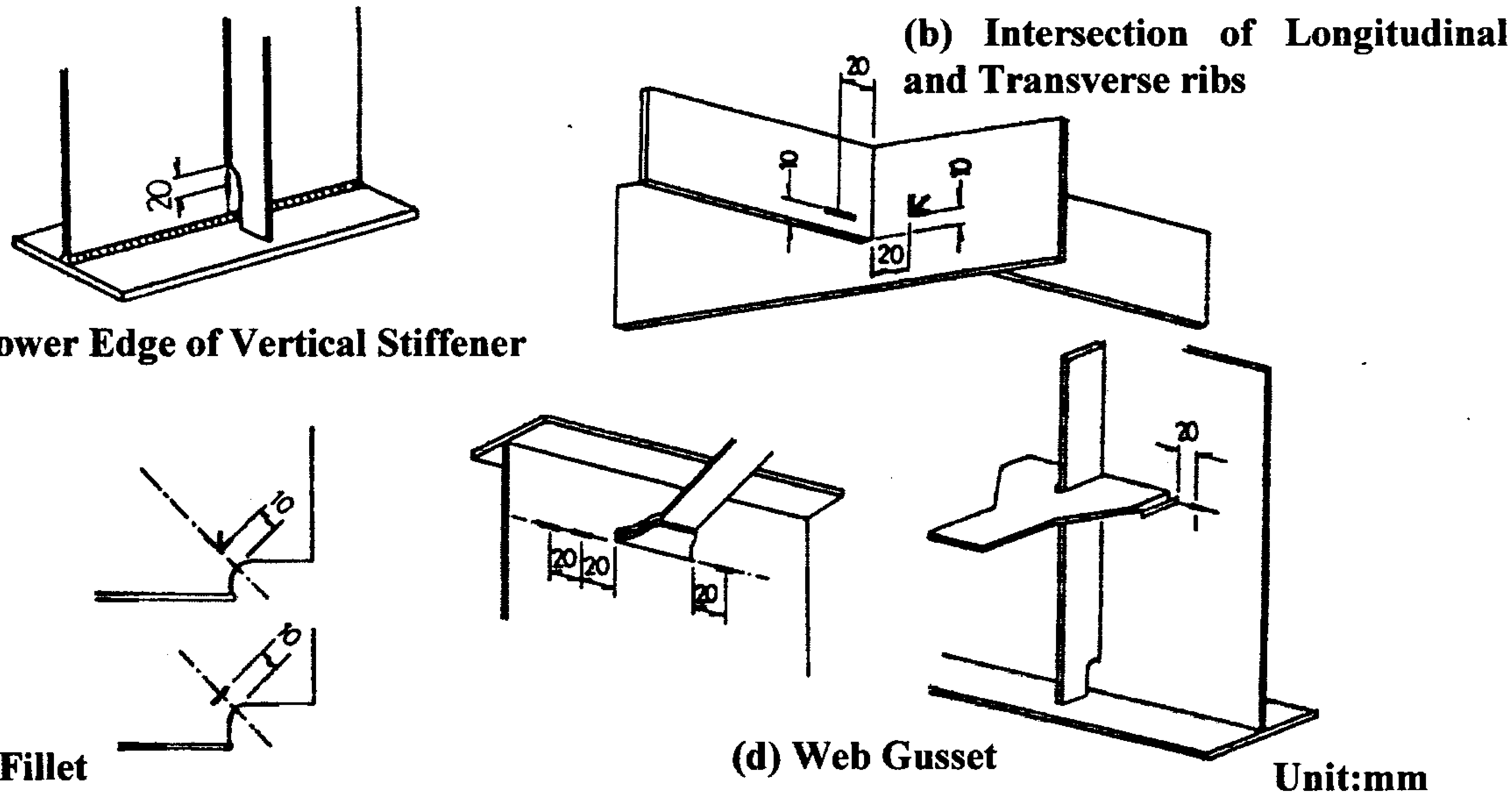


Figure 2. The locations of strain gauges for fatigue damage evaluation

(3) Determination of the displacement mode in which fatigue occurs

It is possible to determine the cause of fatigue cracks from inverse analysis of the fluctuation waveform stresses that accompany the passage of live loads(5). In order to decide the methods of repair and reinforcement, the accurate understanding about how trucks pass across the bridge and how the elements of the bridge move as a result is indispensable. The author has proposed a method for determining the displacement that governs fatigue by combining FEM analysis and inverse analysis, and has applied this method to the retrofitting of a number of bridges (**Figure 3**).

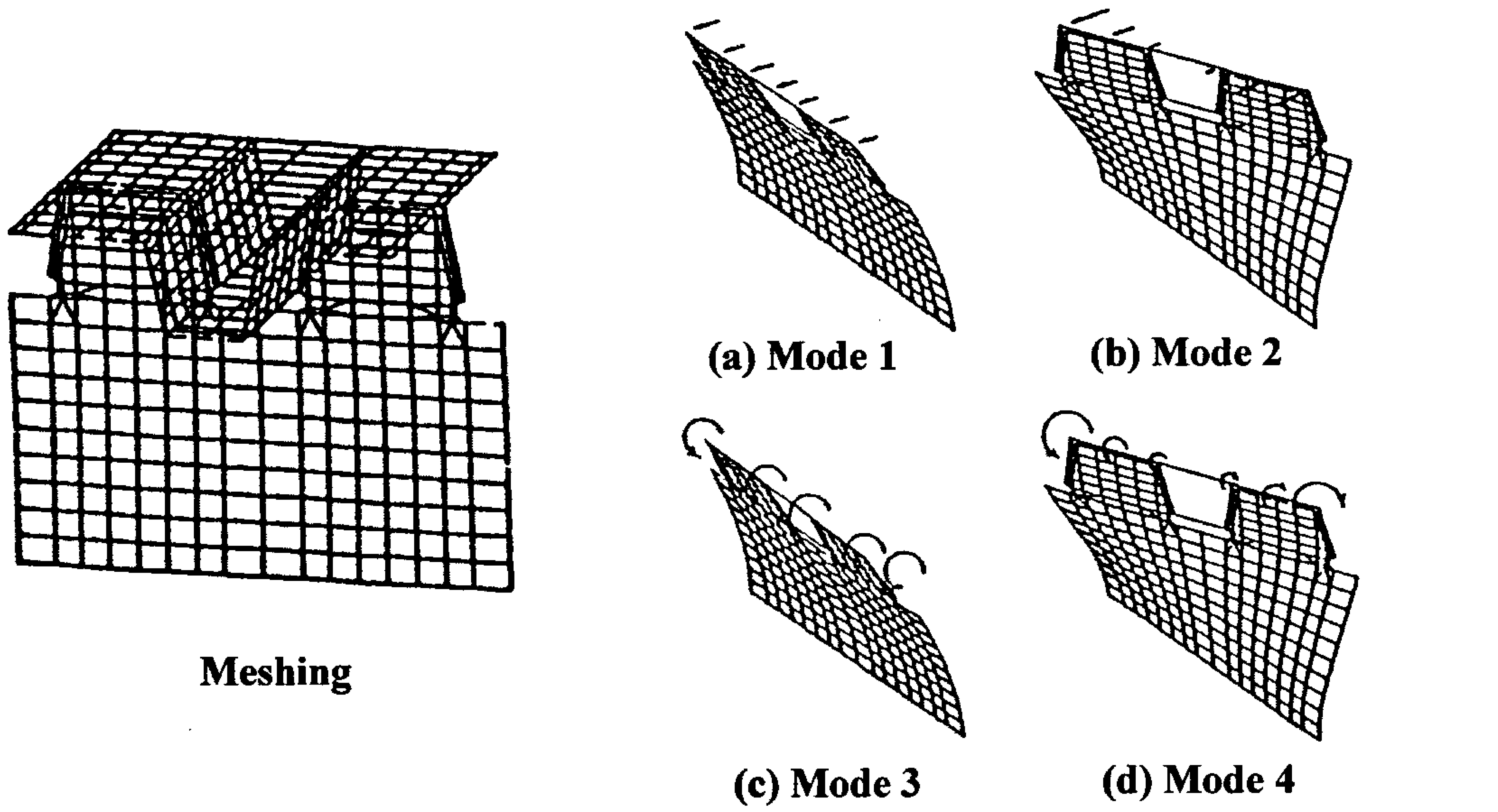


Figure 3. Modes of displacement at the connection of transverse and longitudinal ribs of orthotronic bridge deck

4. MONITORING

Monitoring of bridge girders has already been introduced in a number of forms such as the observation of the movement of long-span bridges, the counting of axles passing toll gates on toll roads and traffic censuses. However, long term monitoring aimed at the maintenance of bridge beams has not yet been performed. When long term monitoring for maintenance purposes, the data will be useless, unless it is clear what will be measured and for what purpose, and how that information will contribute to the evaluation of the deterioration of the bridge. Possible elements of the monitoring program include the following:

*Monitoring of external disturbances including live loads, earthquakes, wind loading, temperature effects and earth movements.

*Overall response characteristics of the structure including displacement of the structure and vibration characteristics.

*Damage to critical sections of the beams including the loss of function of supports and expansion joints.

*Local damage including the occurrence and growth of fatigue cracks, cross-sectional damage by progress and corrosion, and delayed destruction.

A monitoring plan should start by clarifying what sensors are to be used for the measurement of these quantities, how the measured data are to be retrieved and how the data are to be processed to evaluate the damage. Recently proposed sensors and processing equipment are listed in **Table 1** and **Figure 4**. The following issues regarding the sensors and equipment remain:

*The long term durability and stability of sensors and other units such as strain gauges and displacement meters. Maintenance of the monitoring equipment must be simpler than the maintenance of the bridge beams.

*The cost of the monitoring units. The cost of monitoring increases when increased durability is required.

* The application of optical fiber sensors from the point of view of system stability and durability, but the performance is inadequate, and their cost is high.

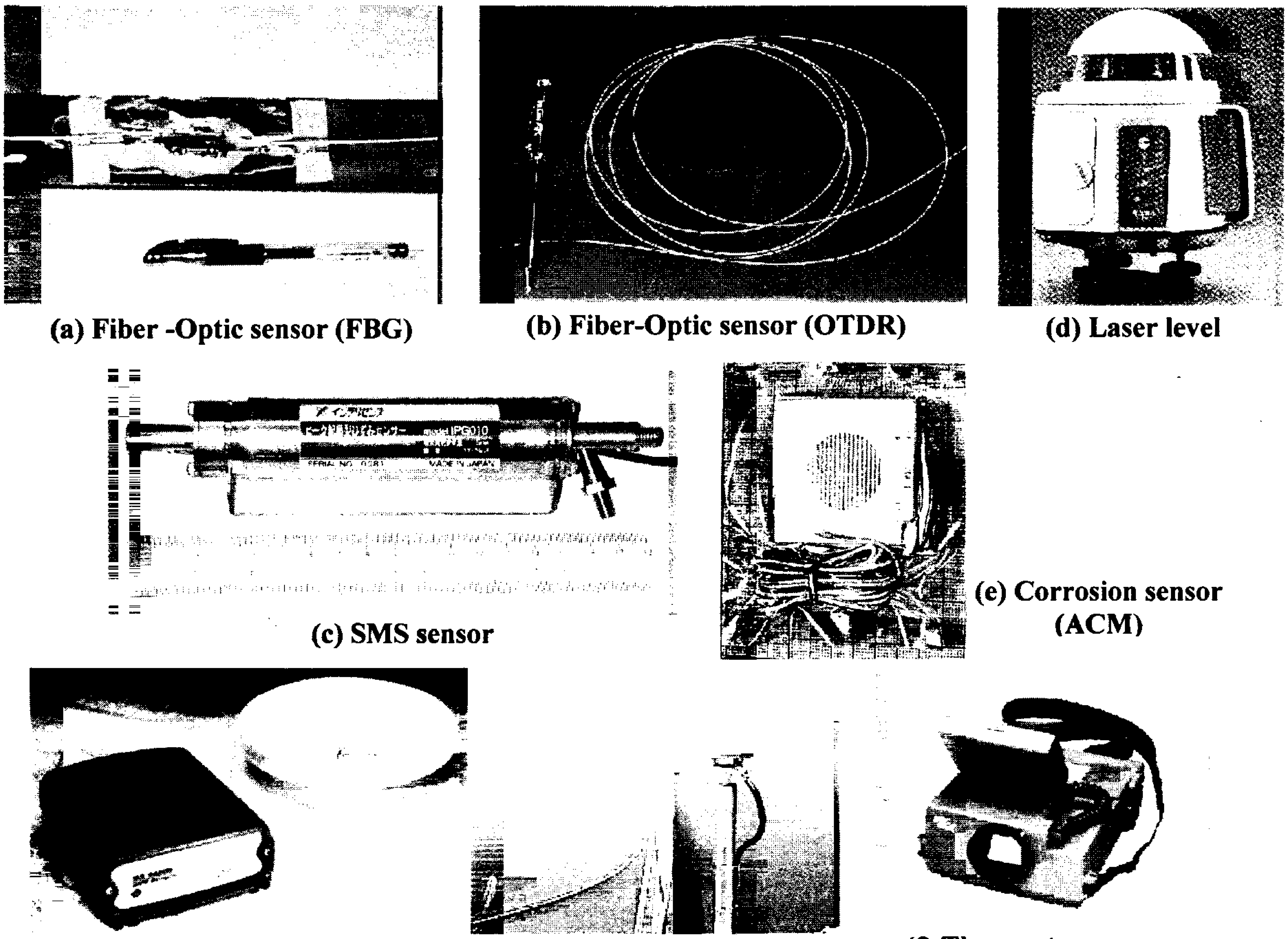
Figure 5 shows the plan of bridge remote monitoring by applying the fiber –optic communications net of Ministry of Land and Transportation of Japanese Government(6). Various newly developed sensors and instruments has been applied and evaluated in this pilot project.

Table 1. Recently proposed sensors and processing equipment

Sensor		Measurement / Detection target	Characteristic / Application
Optic fiber sensor	FBG	strain	Measurement of the strain at the point. 10 measurement points per one line.
	OTDR	displacement	Measurement of the strain distribution. (maximum length : 100km)
Trip sensor (SMS sensor)		peak strain peak displacement	Peak strain is recorded as magnetic properties of the TRIP alloy.
Laser level		displacement	One of the survey instrument. Measurement of the deflection of the long span bridge.
GPS		displacement	Measurement of the large deformation of the long suspension bridge.
Laser Doppler velocimeter		velocity frequency	Detection of the flaws by vibration measurement.
Corrosion sensor		corrosion progress	Observation of the progress of the corrosion by measuring the corrosion current.
Fatigue crack sensor		fatigue damage	Observation of the crack propagation of the dammy stainless steel.
Thermo tracer		temperature crack	Survelliance of the crack, exfoliation, void inside by sensing the difference of the surface temperature of the concrete.

5. RETROFITTING

This term has come to be used in a variety of situations, and will be used here in preference to repair and reinforcement. Retrofitting means adding to existing elements and leaving the structure in a better condition. Retrofitting could, for example, include attaching a plate, or adding or removing a member. Naturally, retrofitting in response to damage must start with determining the cause of the damage. However, the reality is that this is not happening. A common mistake is to simply attach a steel plate to thicken the cross section, thereby hoping to reduce the stress, without knowing what the real problem is.



(a) Fiber -Optic sensor (FBG) (b) Fiber-Optic sensor (OTDR) (d) Laser level

(c) SMS sensor (e) Corrosion sensor (ACM)

(d) GPS (f) Thermo tracer

Figure 4. Recently proposed sensors and processing equipment

Retrofitting methods to deal with fatigue damage that remove or alleviate the fundamental cause of the fatigue, and methods that increase the fatigue strength of damaged element are being developed. New advanced tools such as ultrasonic impact method(7) and new materials such as low temperature transformation welding electrode(8) which can introduce compressive residual stress in the vicinity of welds have been applied to improve fatigue performance of structural elements.

As one of the activities of IIW(International Institute of welding), we are distributing information about the causes of fatigue damage in bridges and retrofitting on the World Wide Web (http://iiw-wg5.cv.titech.ac.jp). We believe this to be the most complete source of publicly available information. We are also considering providing consultation software for maintenance engineers.

6. CLOSING REMARKS

Maintenance technology is a highly sophisticated discipline combining materials, joining methods, strength evaluation, structural response, destruction dynamics and non-destructive evaluation. Consequently, maintenance engineers must have knowledge of all of these subjects. One must consider the present condition of the bridge, the design concept, quality control including welding during construction, the mode in which the bridge has been used and how it has been maintained must all be considered when devising a maintenance plan.

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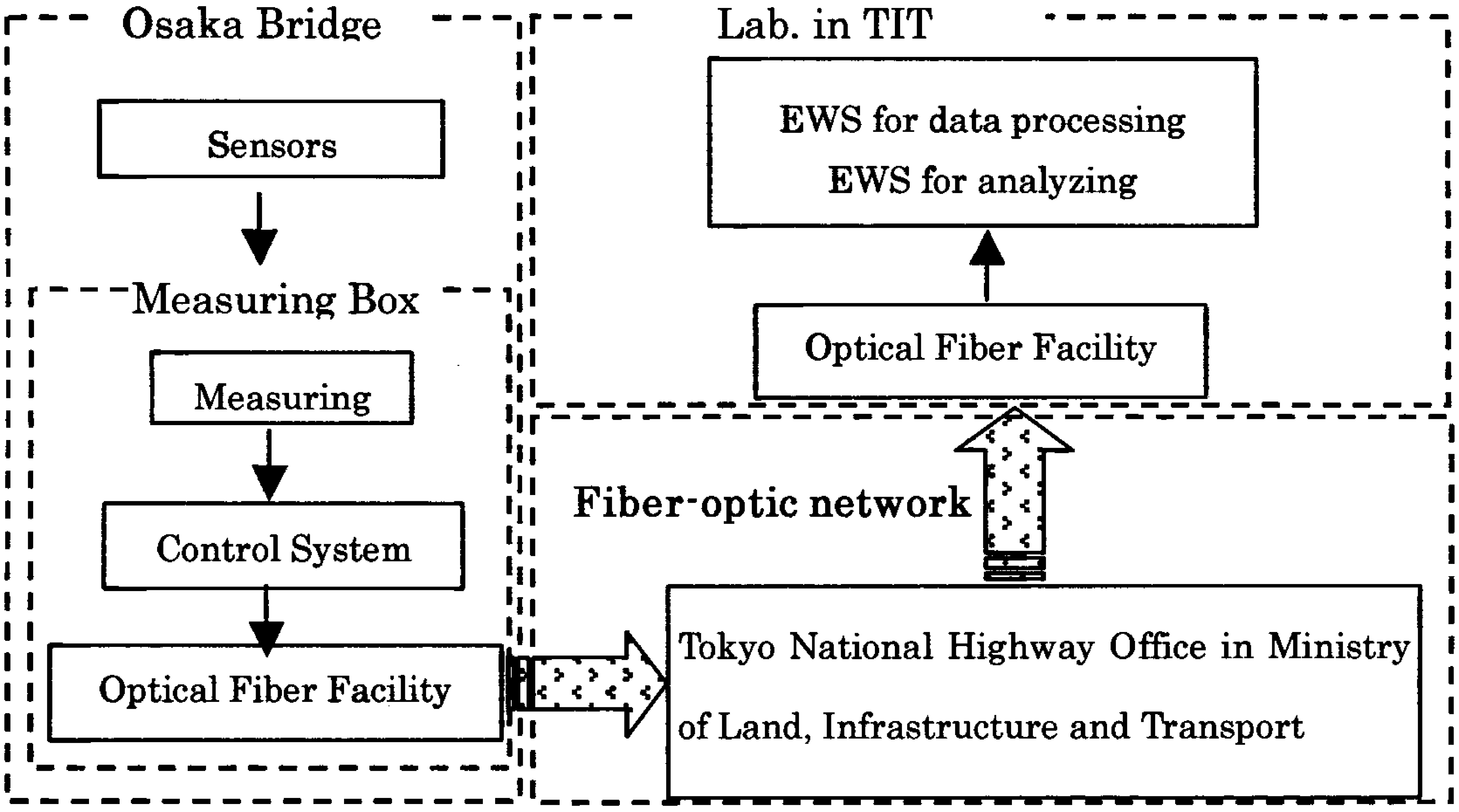
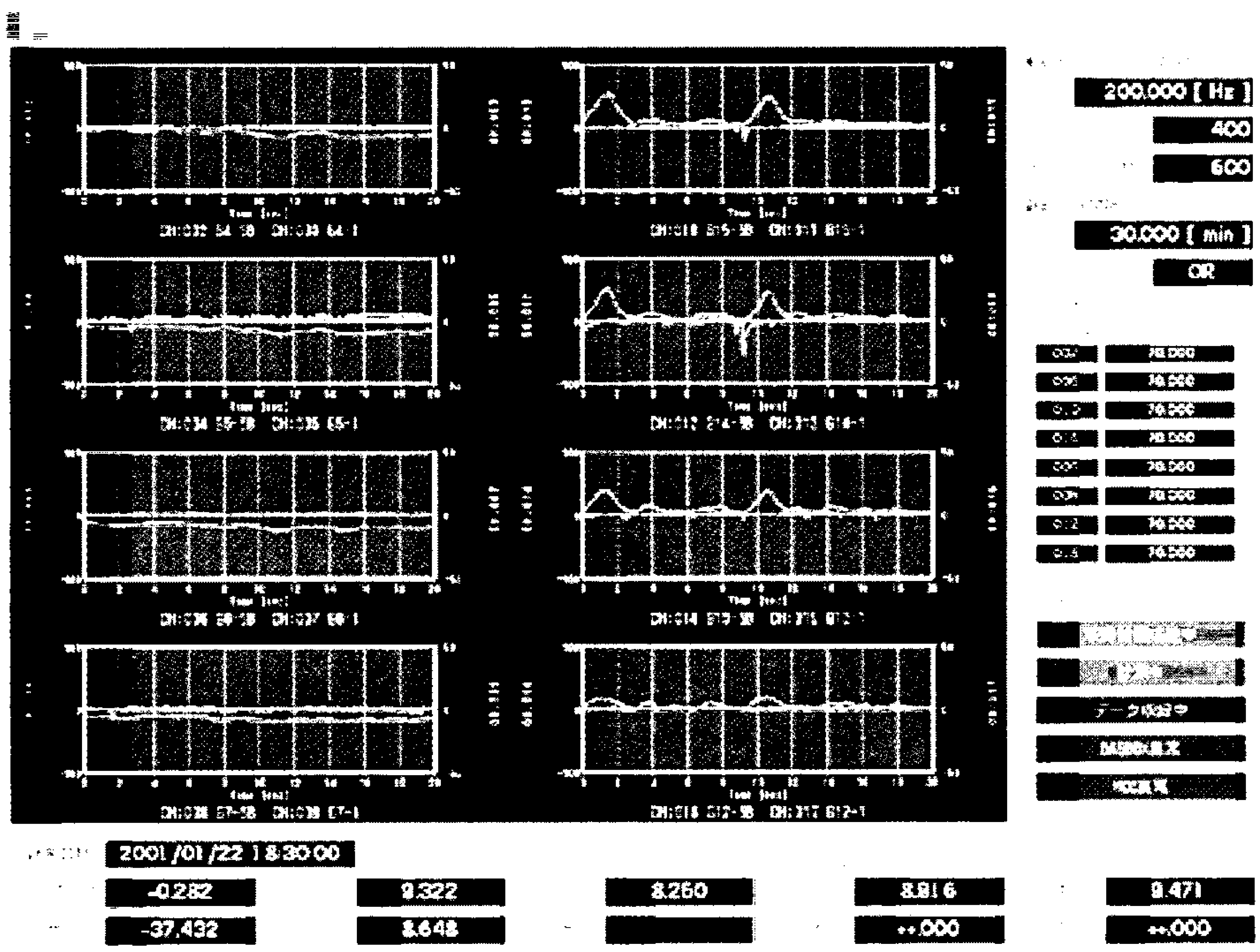


(a) System



(b) Monitoring Screen

Figure 5. Bridge monitoring system by applying advanced communication systems

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High Performance Steels and Their Use in Structures

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Abstract

Safety, reliability and economy of structures are governed by material, design, fabrication, inspection and maintenance. The most efficient and economical structures are based on optimized trade offs between these parameters. Thus, the use of the best material in combination with deficient design or fabrication or both and the use of the best designs in combination with deficient materials or fabrication or both do not produce safe, reliable or economical structures. This paper describes the effects of composition and thermomechanical processing on the properties of medium carbon "conventional" steels and on low-carbon "high-performance" steels. A general discussion of the attributes of high-performance steels and the advantages derived from their use in various structures are presented. The use of high-performance steels for bridges is discussed in more detail. This discussion shows that significant benefits may be derived by using high-performance steels in bridges of present design. However, a significant increase in benefits may be achieved by changes in present specifications and by using high-performance steels in new and innovative designs. More important, properties of high-performance steels should be tailored to the needs and requirements of the different structural types.

IMPROVED PERFORMANCE OF THE NATION'S INFRASTRUCTURE can result in significant savings. It is estimated that a one percent improvement in the performance and durability of roads and bridges alone could result in about $30 billion savings (1).

Steel with its outstanding strength, ductility, fracture toughness, repairability and cyclability continues to be a primary constructional material. Steelmaking technologies have been, and new ones are being, developed to improve further these and other steel attributes. Extensive research is under way in direct steelmaking, casting, rolling and process control.

The interrelationships among chemical elements and conventional processing such as hot rolling, controlled rolling, normalizing, and quenching and tempering and their effect on properties of steel products have been known for many years. Also, steelmaking and processing technologies have been in use to control chemical composition and to produce steels having low sulfur, sulfide shape control, low hydrogen content, fine grain microstructure and close dimensional tolerances. These steels are less susceptible to hydrogen cracking, exhibit less chemical segregation, have improved through thickness and heat-affected zone properties and exhibit enhanced fracture toughness.

Steel processing has undergone significant development in the past few years. In addition to the traditional hot rolling, controlled rolling, normalizing and quenching and tempering, various combinations of rolling practices and cooling rates have opened new opportunities to develop high strength with very attractive properties. Figure 1 is a schematic diagram of conventional and thermomechanical control processes. Controlled rolling has been used to produce steels with fine grain microstructure and improved mechanical properties, particularly steels with higher strength and enhanced fracture toughness. This practice has been widely used to produce line pipe steels with excellent fracture toughness. Further grain refinement is achieved by accelerated cooling which cools the deformed austenite rapidly thus increasing ferrite nucleation sites and nucleation rate. Structural steels that are presently produced using various processes are presented in Table I. This table demonstrates that processing options decrease as the desired strength of the steel increases from 50 ksi to 100 ksi. Also, processing options decrease as plate thickness increases.

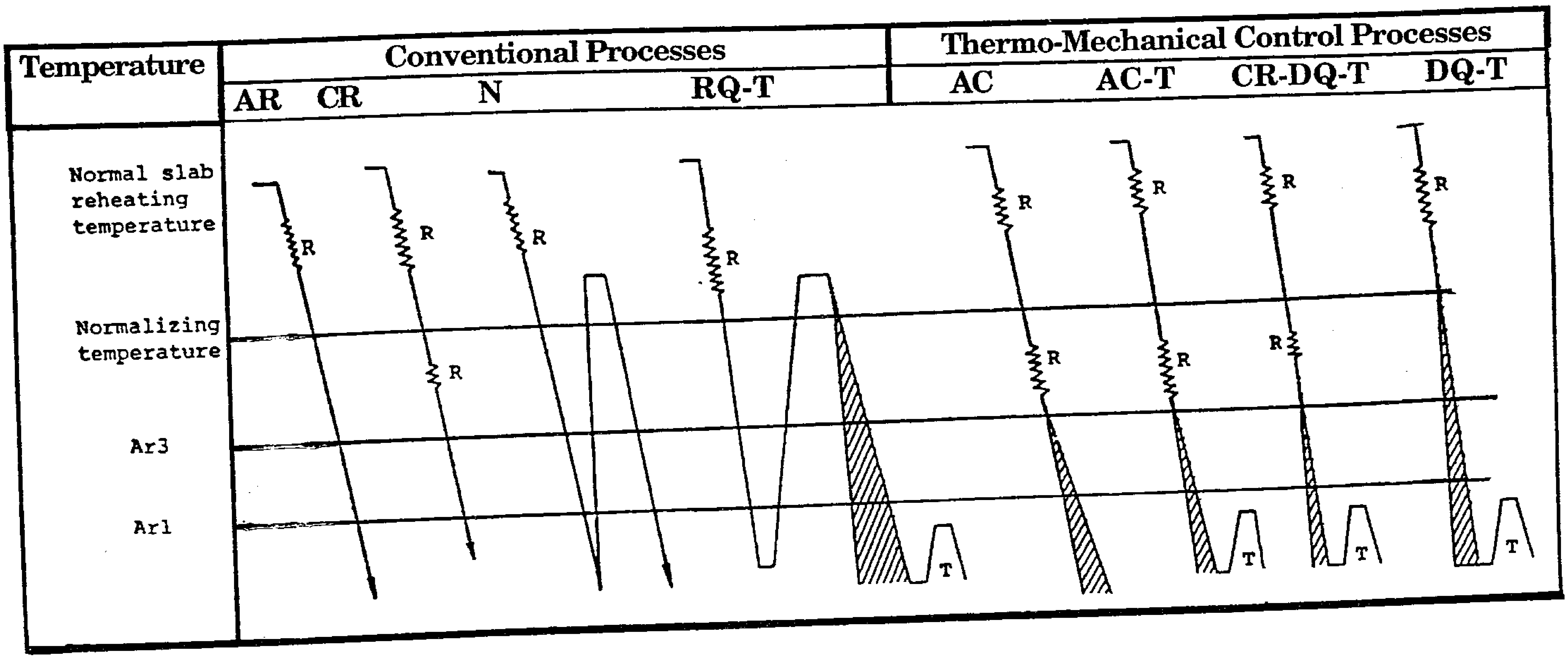

Fig. 1 - Schematic diagram of conventional and TMCP processes for manufacturing steel plates.

Table I. Structural (Non-Armor) Plate Grades and How They are Processed

Process	Examples
50-Ksi Minimum Yield Strength	
HR	A572, A588
N	A537, A572, A588, A633, API 2H
CR	A572, A588, A656, A808
QT	API 2Y-50
AC/DQT	API 2W-50
70-Ksi Minimum Yield Strength	
CR	A656, X-70
QT	A710, A852
AC/DQT	HT-60 (Japanese)
100-Ksi Minimum Yield Strength	
QT	A514/A517, A710 Mod.
DQT	HT-80 (Japanese)

HR = Hot Rolled
N = Normalized
CR = Controlled-Rolled and Air-Cooled
QT = Quenched and Tempered
AC = Interrupted Accelerated Cooled
DQT = Direct-Quenched and Tempered

High Performance Steels

Safety and reliability of structures is governed by material properties, design, fabrication, inspection, usage and maintenance. The best material properties alone do not ensure safety of a structure where one of the other parameters is deficient. Similarly, good fabrication and inspection procedures of materials that exhibit high fracture toughness and are not susceptible to cracking do not ensure reliability of a structure with design deficiencies. Consequently, performance of steels is determined in a large measure by the characteristics of the structure in which it resides. Furthermore, high performance implies that the steels possess attributes that are well beyond what is needed for fitness-for-service for a particular application. Any one attribute or a combination of a few of them may be improved by changing the composition or by using conventional or thermomechanical control processes.

Thermomechanical control processes (TMCP) employ rolling sequences within specific temperature ranges to promote fine microstructures. TMCP produce higher strength steels when compared with conventionally processed steels of similar composition. Similarly, TMCP produce a given strength steel having leaner composition than a steel conventionally processed. Thus, TMCP may be used to increase strength and decrease alloy content of steels, simultaneously. These benefits of TMCP decrease as steel strength and thickness increase. In spite of these limitations, the benefits of TMCP can profoundly improve steel properties, fabrication and structural performance. Some of these benefits are discussed in the following sections.

Steel Properties. Historically, carbon has been the principal source of strength for carbon-manganese steels. However, increased carbon content can have harmful effects on steel properties such as fracture toughness, Figure 2, (2) and weldability. Consequently, alloying additions within specified limits, rather than carbon, are used to increase the minimum yield strength of constructional steels from 36 ksi for A36 steel to 50 ksi for A572 Grade 50 steel, Table II. These alloy additions improve fracture toughness and do not seriously degrade weldability as compared with increasing carbon content.

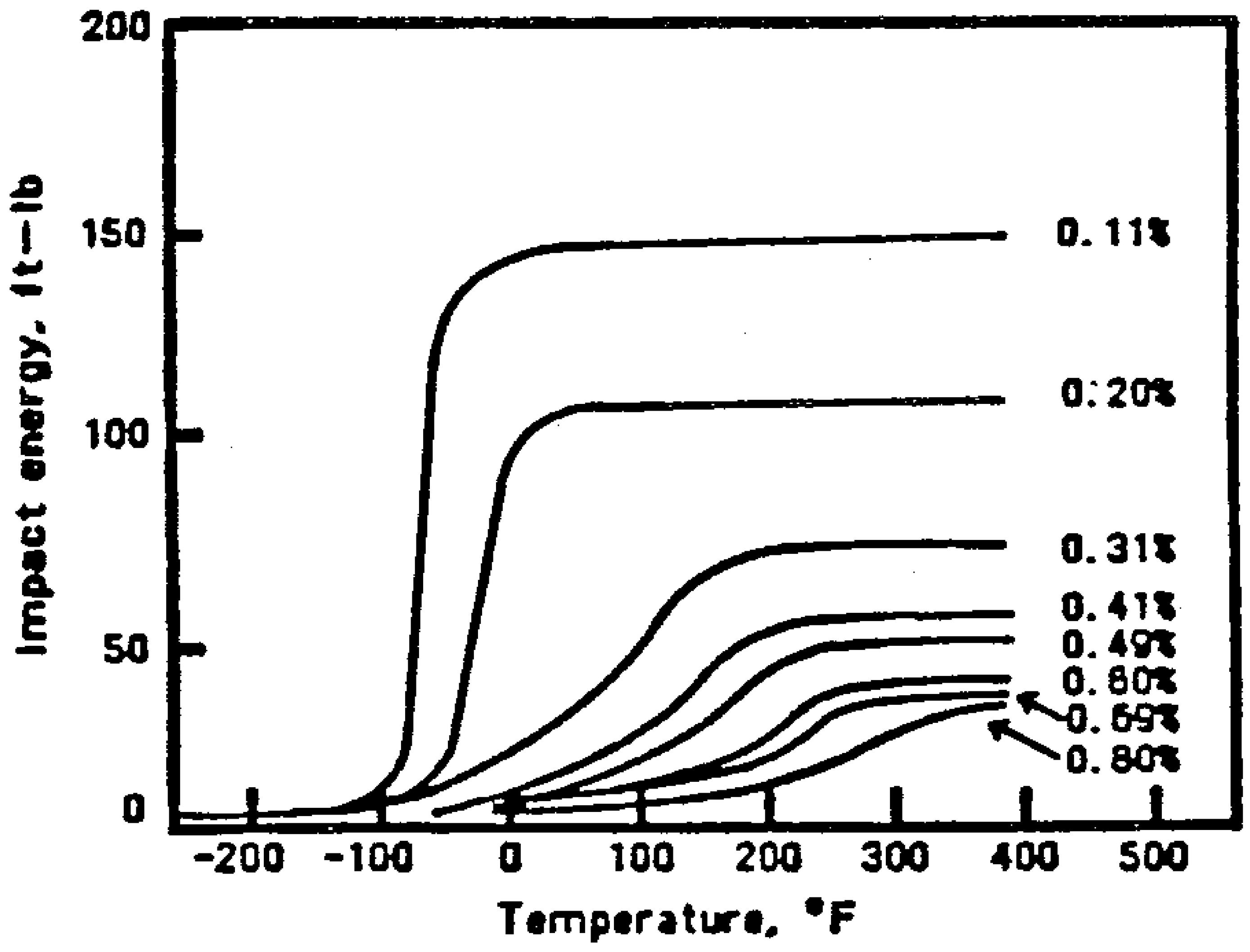

Fig. 2 - Effect of carbon content on Charpy V-Notch impact transition curves of normalized ferrite-pearlite steels.

Table II. ASTM Compositions of 2-Inch-Thick Plate

	A36	A572*	A588-A	A514-F
Yield Strength Min. (Ksi)	36	50	50	100
Processing**	AR	AR	AR	Q&T
Carbon	0.26 max	0.23 max	0.19 max	0.10-0.20
Manganese	0.80-1.20	1.35 max	0.80-1.25	0.60-1.00
Phosphorus	0.04 max	0.04 max	0.04 max	0.035 max
Sulfur	0.05 max.	0.05 max	0.05 max	0.035 max
Silicon	0.15-0.40	0.15-0.40	0.30-0.65	0.15-0.35
Nickel	-	-	0.40 max	0.70-1.00
Chromium	-	-	0.40-0.65	0.40-0.65
Molybdenum	-	-	-	0.40-0.60
Vanadium	-	0.01-0.15	0.02-0.10	0.03-0.08
Titanium	-	-	-	-
Copper	-	-	0.25-0.40	0.15-0.50
Boron	-	-	-	0.0005-0.006

* Type 2

** AR = As-rolled.
Q&T = Quenched and tempered.

Increasing strength of hot rolled constructional steels to a 70 ksi minimum yield strength by changing steel composition alone without adversely affecting ductility, fracture toughness and weldability is extremely difficult, especially in thick plates. Therefore, quenching and tempering has been used to achieve this objective. For example, weathering steel A852 with 70 ksi minimum yield strength is the quenched and tempered version of hot rolled weathering steel A588 with 50 ksi minimum yield strength. These steels have identical composition, Table III, are available in thicknesses up to 4 inches and are weldable using identical welding parameters; however, A852 steel has higher strength and better fracture toughness than A588 steel. Further increase in yield strength to 90 ksi minimum yield strength and above, as for quenched and tempered A514 steel, Table II, requires increased alloy content and a corresponding increased care in fabrication, especially in welding.

Table III. Compositions of Weathering Steels

	A588	A852
Yield Strength Min. (Ksi)	50	70
Processing*	AR	Q&T
Carbon	0.19 max	0.19 max
Manganese	0.80-1.25	0.80-1.35
Phosphorus	0.04 max	0.035 max
Sulfur	0.05 max	0.04 max
Silicon	0.30-0.65	0.20-0.65
Nickel	0.40 max	0.50 max
Chromium	0.40-0.65	0.40-0.70
Vanadium	0.02-0.10	0.02-0.10
Copper	0.25-0.40	0.20-0.40

* AR = As-rolled.
Q&T = Quenched and tempered.

TMCP steels exhibit higher yield and tensile strengths than conventionally processed steels. Figure 3 demonstrates this effect on tensile strength for steels with different carbon equivalent values. The data show that TMCP may be used to increase strength at a given carbon equivalent value, to maintain a given strength with a lower carbon equivalent value, or to increase strength and decrease carbon equivalent simultaneously. However, decreasing carbon content alone decreases the strength of the base metal. The addition of microalloying elements along with accelerated cooling are often used to compensate for the decrease in strength.

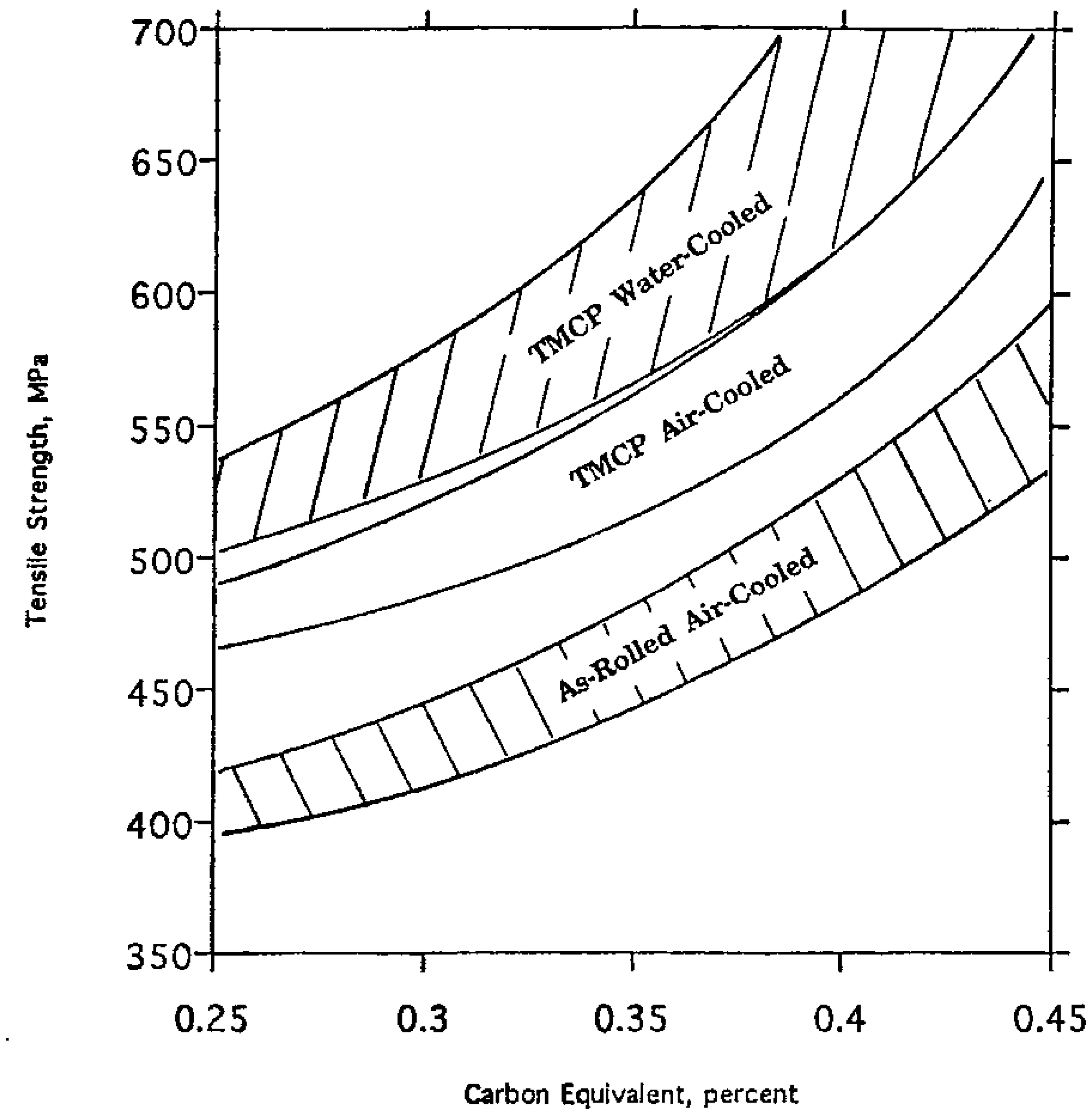

Fig. 3 - Relationship between carbon equivalent and tensile strength.

The fine-grain microstructure of TMCP steels increases strength and improves fracture toughness. Also, reduced carbon improves fracture toughness of the base metal and heat-affected zone. Fracture toughness of the heat-affected zone decreases from that of the base metal due to the formation of coarse-grain microstructure and martensite-austenite constituents. The difference between base metal and heat-affected zone fracture toughness decreases with reduced carbon equivalent of the steel. Thus, steels with lower carbon content and carbon equivalent values exhibit less degradation of heat-affected zone fracture toughness. A corollary to this observation is that steels, especially microalloyed steels, with lower carbon content and lower carbon equivalent values may be welded by using high heat input processes without significant degradation in heat-affected zone properties, Figure 4 (3).

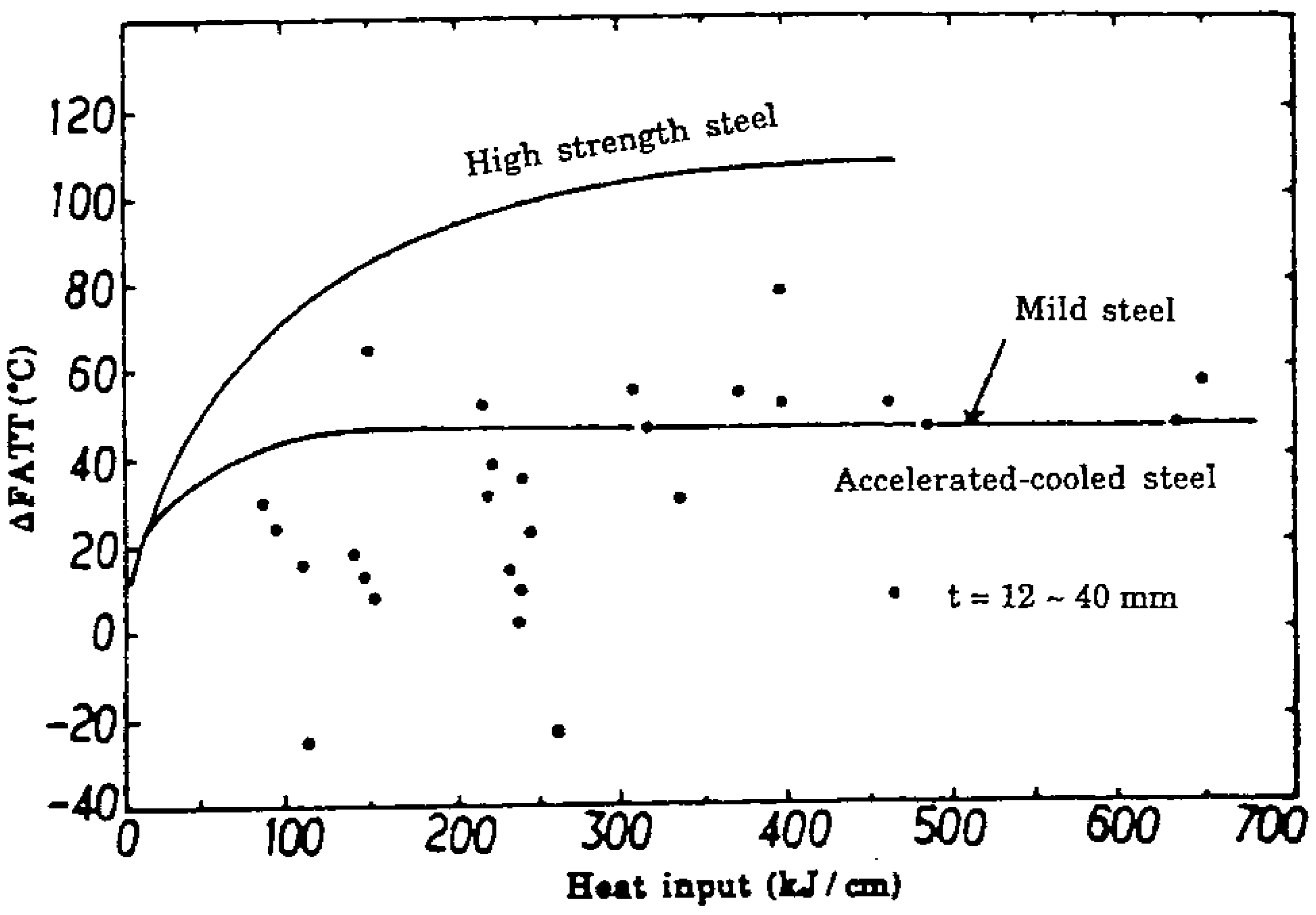

Fig. 4 - Deterioration of fracture toughness at the heat-affected zone of TMCP steels as a function of heat input.

Carbon equivalent is directly related to heat-affected zone hardenability which in turn is directly related to preheat requirements to avoid cold cracking. This aspect of weldability is one of the topics under investigation in a Federal Highway Administration (FHWA) project to develop high-performance steels for bridges. This project is administered by the Carderock Division of the Naval Surface Warfare Center. To ensure steel industry participation, the American Iron and Steel Institute (AISI) has been subcontracted to implement a three-year development program to tailor candidate high-performance steels to bridge requirements. As part of an initial six-month laboratory screening study, U. S. Steel (4) produced and evaluated 2-inch-thick plates of three 70 ksi yield strength weathering (70W) compositions and two 100 ksi yield strength weathering (100W) compositions that were processed by several different rolling and heat-treatment practices. A hot-rolled and quenched-and-tempered 0.08 percent carbon A852 steel exhibited a very good combination of strength and fracture toughness and appears to be an excellent candidate for a 70W bridge steel. Also, a quenched-and-tempered A514 grade F steel with about 0.10 percent carbon appears to be a good candidate for a 100W bridge steel.

The HAZ Calculator (5) software, Figure 5, was used to compare preheat requirements for the lower carbon high-performance 70W and 100W steels developed by U. S. Steel with those for presently produced higher carbon versions of these steels. The HAZ Calculator incorporates most available information and relationships for transformation temperatures, thermal cycles, heat-affected zone hardness, preheat, postheat, and cracking susceptibility into a single predictive tool. The HAZ Calculator is a valuable comparative tool to predict relative differences rather than to calculate absolute values. Figures 6 and 7 predict that the lower carbon high-performance 70W and 100W steels require no preheat for thin plates and less preheat when needed for thick plates.

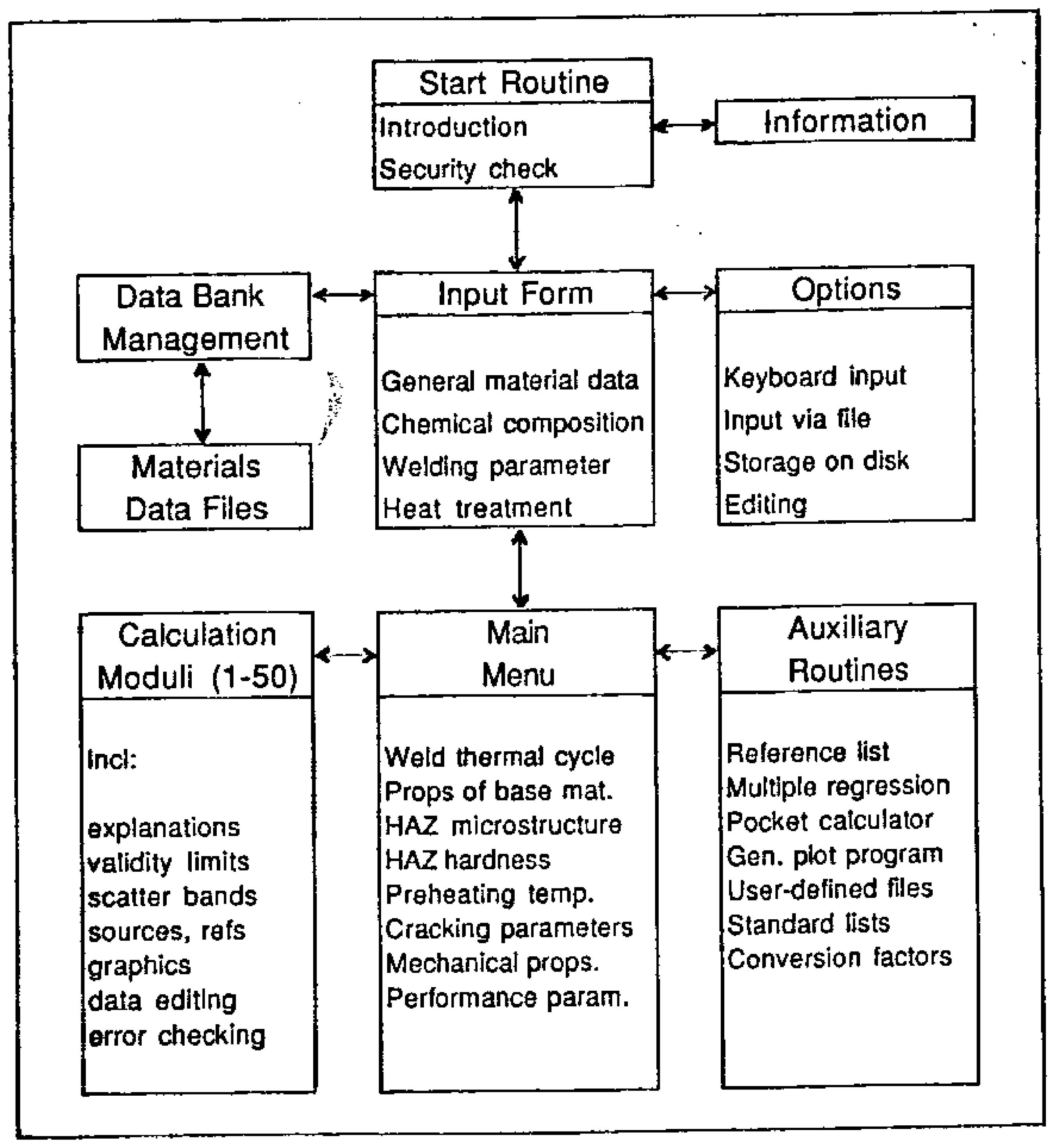

Fig. 5 - HAZ Calculator program structure.

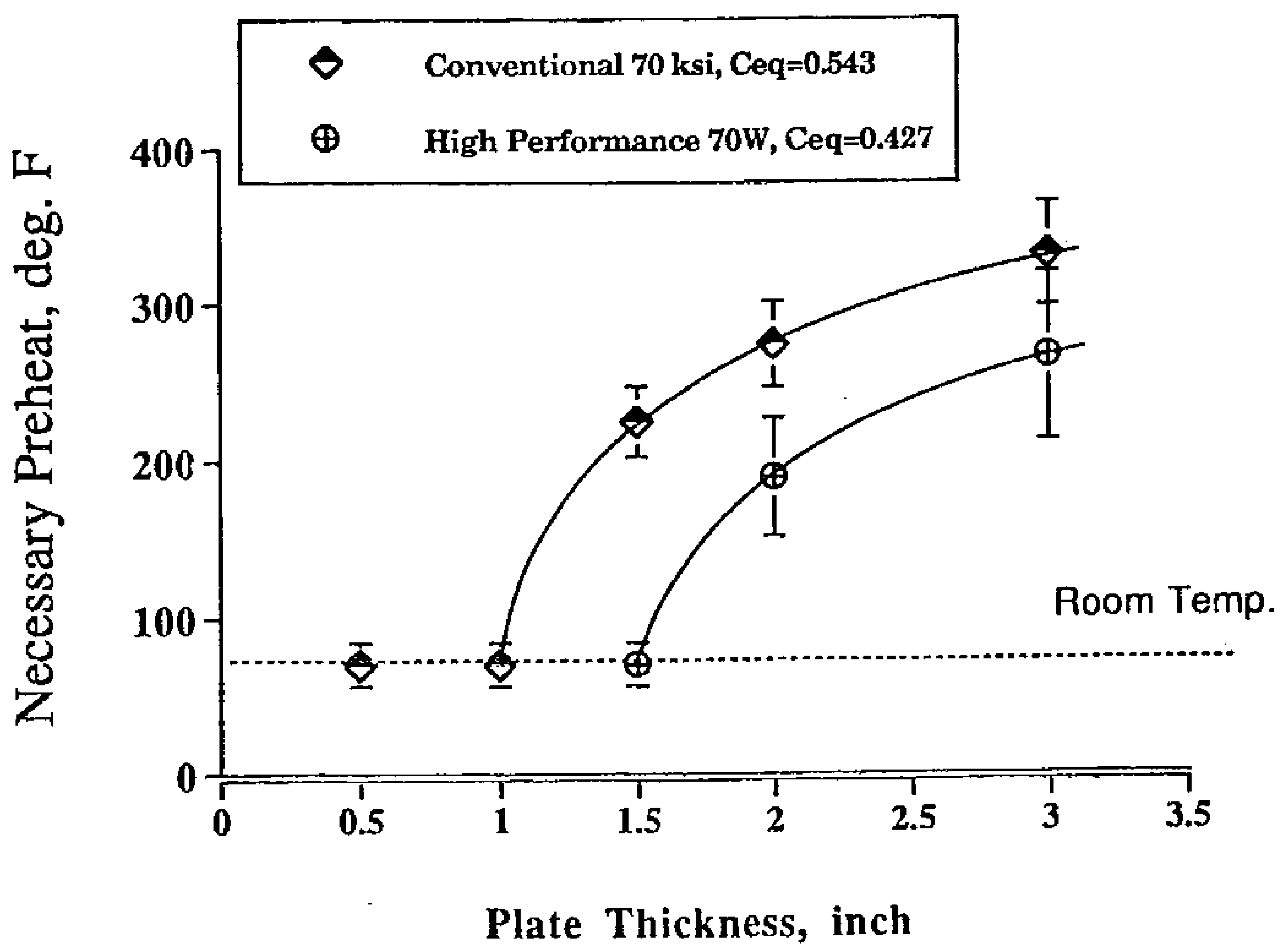

Fig. 6 - Predicted preheat temperatures for butt joints of 70-ksi plates at 50 kJ/in.

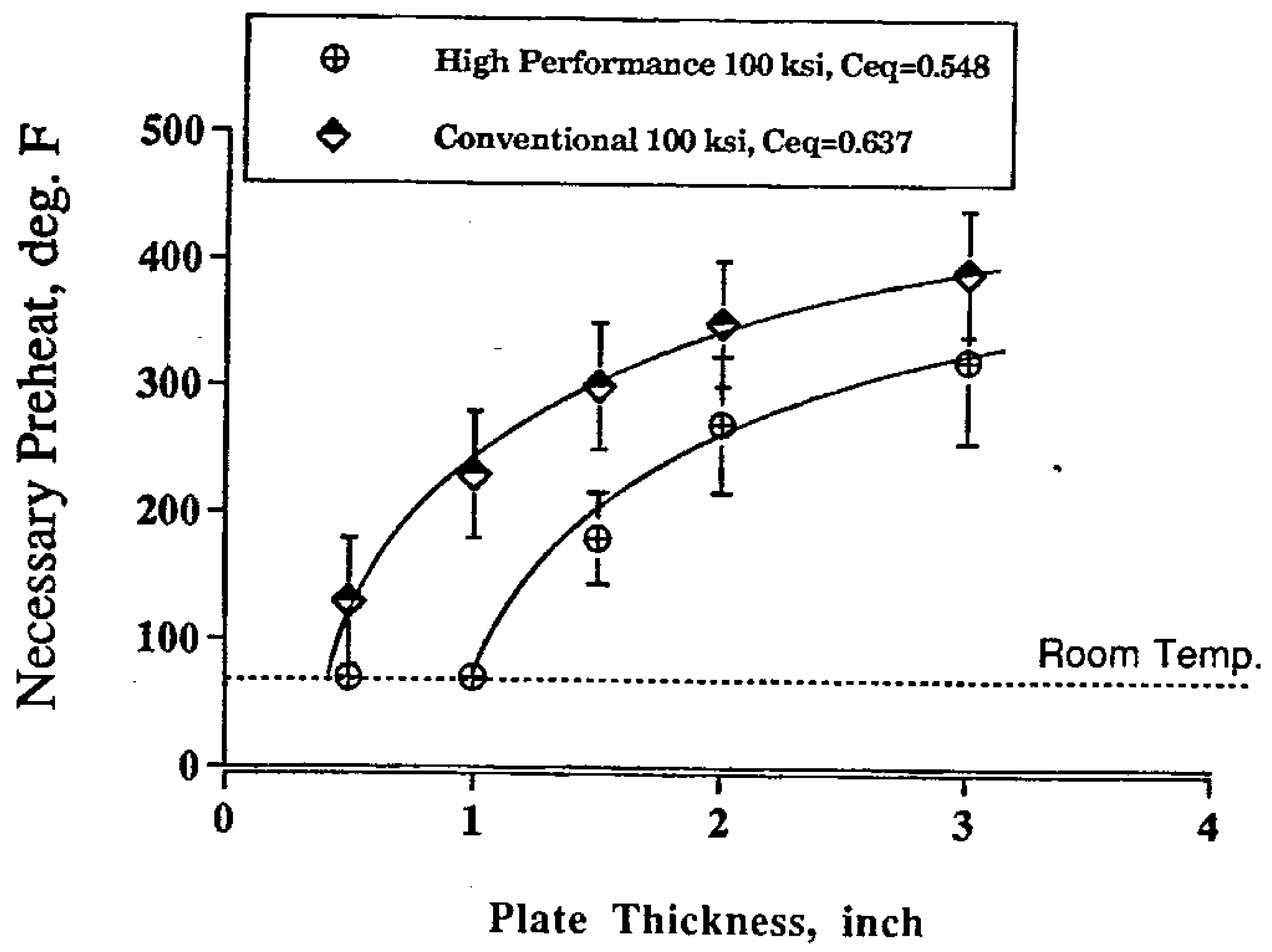

Fig. 7 - Predicted preheat temperatures for butt joints of 100-ksi plates at 50 kJ/in.

Hydrogen induced cold cracking has been observed in the heat-affected zone of conventional steel weldments. The higher the yield strength of these steels, the higher is alloy content and the propensity of the heat-affected zone to hydrogen cracking. The risk of hydrogen cracking in the heat-affected zone of low-carbon high-performance steels is much lower than for conventional steels. The need for higher strength weld metal to develop full joint efficiency of high-strength high-performance steel weldments shifts the hydrogen induced cracking site from the heat-affected zone to the weld metal. (6). Consequently, more attention should be focused on development of high-strength high-performance weld metals for joining high-performance steels.

The preceding discussion shows that high-performance steels, when compared with conventional steels, exhibit higher strength with lower carbon and carbon equivalent values, higher fracture toughness for both base metal and heat-affected zone, lower heat-affected zone hardness and higher resistance to cold cracking. These material characteristics have significant benefits for fabrication, erection and structural performance.

Fabrication. Increasing the strength of conventionally processed steels to 70 ksi yield strength requires quenching and tempering treatments. The length of quenched and tempered plates is limited to about 50 feet by the length of the heating and quenching facilities. This length limitation may be eliminated for 2-inch or less thick plates by using TMCP to produce long plates with 70 ksi strength. This may be achieved by controlled rolling followed by accelerated cooling. Long plates decrease the number of splice butt welds to fabricate long span girders thus decreasing fabrication and inspection costs.

High-rise buildings, long-span atriums and bridges as well as some pressure vessels require very thick steel plate to preserve strength and load-carrying capacity. Usually, to maintain strength as plate thickness increases, carbon and alloy content are increased within allowable compositional limits. The increased carbon equivalent along with faster cooling rates of welds tend to degrade the weldability of thick plates. Consequently, the use of high-performance high-strength steels can provide immediate savings from the use of thinner sections. Thinner sections require smaller weld volume. Figure 8 shows the significant decrease in weld volume per unit length when higher strength can be used to decrease plate thickness. The curve in this figure was calculated for full-penetration double-V groove butt weld with 60-, 52- and 45-degree included angles for the 4-, 3- and 2-inch thick plates, respectively. Also, the data was based on the assumptions that the reference design required 4-inch-thick plate having 50 ksi minimum yield strength and that the maximum design stress was a fixed percentage of the minimum yield strength of the steel. This decrease in filler metal volume decreases welding time, cost of fabrication and inspection, and distortion and restraint of welded assemblies. Thinner plates require less time and energy when postweld stress relief treatment is specified, fewer weld passes when repairs are needed, less time and cost for nondestructive testing, and more efficient handling in the shop. These benefits may be realized for structures that are not fabricated from very thick plate. Also, thinner plates increase fabrication efficiency, decrease energy consumption and increase cargo capacity (payload) of ships, off-highway vehicles and construction equipment.

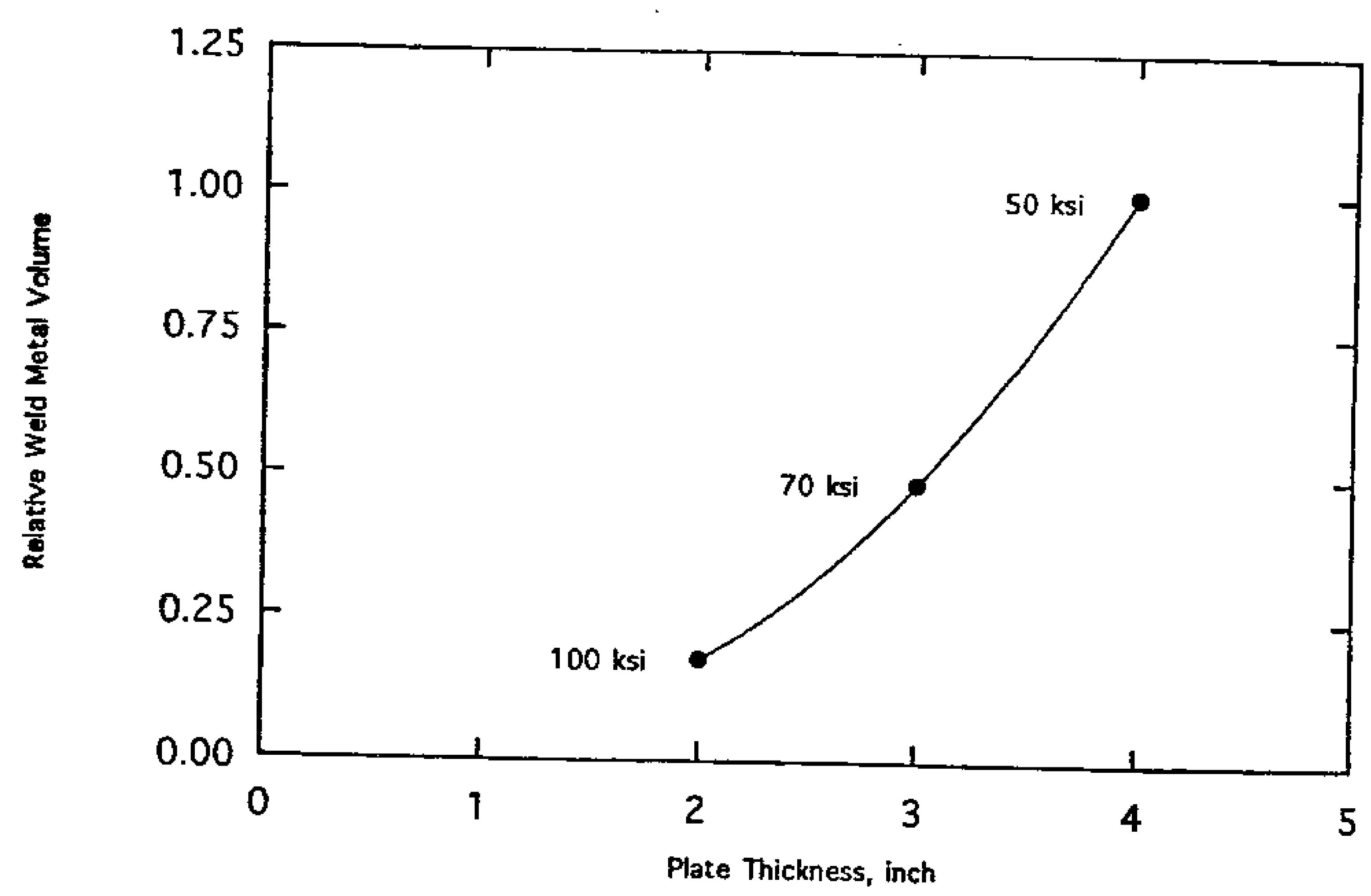

Fig. 8 - Volume of weld metal for full-penetration double-V groove-weld butt joint in tension as a function of thickness and strength.

Heat-affected zone hardness may be reduced by preheat and post-heat treatments or by decreasing the carbon content of the steel. Consequently, preheat and post-heat treatments for low-carbon high-performance steels may be eliminated for some plate thicknesses and reduced for thick plates. Heat-affected zone hardness decreases more rapidly when carbon content decreases. Therefore, preheat may not be necessary except possibly for very thick plates or very high strength constructional steels.

Conventionally processed low carbon steels and TMCP steels (3) exhibit less degradation in fracture toughness between the fusion zone and the base metal than higher carbon conventionally produced steels. Thus, low-carbon high-performance steels, especially those with microalloys that resist grain coarsening, may be welded by using high deposition rate processes without affecting the performance of the structural component. This characteristic along with decreased heat-affected zone hardness and susceptibility to cracking make low-carbon high-performance steels ideally suited for field welding. Field welding would eliminate costly bolted connections and produce more aesthetic structures. Also, these characteristics permit repair and strengthening of existing structures.

The preceding discussion shows that low-carbon high-performance steels would increase productivity, decrease cost of fabrication, and result in safe, reliable and aesthetically pleasing welded structures.

Design. The important properties of steels used in various structures are strength, ductility, fatigue, fracture toughness and fabricability, where fabricability encompasses forming, welding and weldment properties. The importance of any or a combination of these properties and their contribution to safety of a structure depend on the design of the structure and the loading throughout its useful life. For example, fatigue is rarely considered for statically loaded structures, whereas it commands primary consideration in the design of bridges. The need and minimum acceptable value for any of these properties should be incorporated in the specifications that govern the particular structure and must be investigated early in the design stage.

Statically loaded structures such as storage tanks, welded water pipe (7) and other similar structures are fabricated from thin, usually less than 0.75-inch-thick plate or sheet steels with yield strength equal to or less than 50 ksi. The use of thinner, higher strength steels than presently used may cause unacceptable distortion and buckling. This would be particularly true for buried water pipe. Consequently, the higher strength attribute of high-performance steels cannot be used effectively for most of these structures.

Fracture toughness of steels having yield strengths less than about 120 ksi is affected by rate of loading to fracture. The lower the yield strength of the steel the more pronounced is the effect of loading rate. Also, the higher is the loading rate the larger is the shift (separation along the temperature axis) between static and dynamic fracture toughness. Fracture-toughness transition under static loading occurs at lower temperatures than under dynamic or impact loads. Figure 9 (8) is a schematic showing the relationship between slow and dynamic fracture toughness and fracture appearance. At very low temperatures, steels exhibit low fracture toughness and brittle initiation and propagation behavior under any loading rate. At high temperatures, steels exhibit high fracture toughness and ductile crack initiation and propagation behavior under any loading rate. At intermediate temperatures, static fracture toughness is higher than dynamic fracture toughness. Also, in this region, initiation and propagation exhibit increased ductility as the temperature increases.

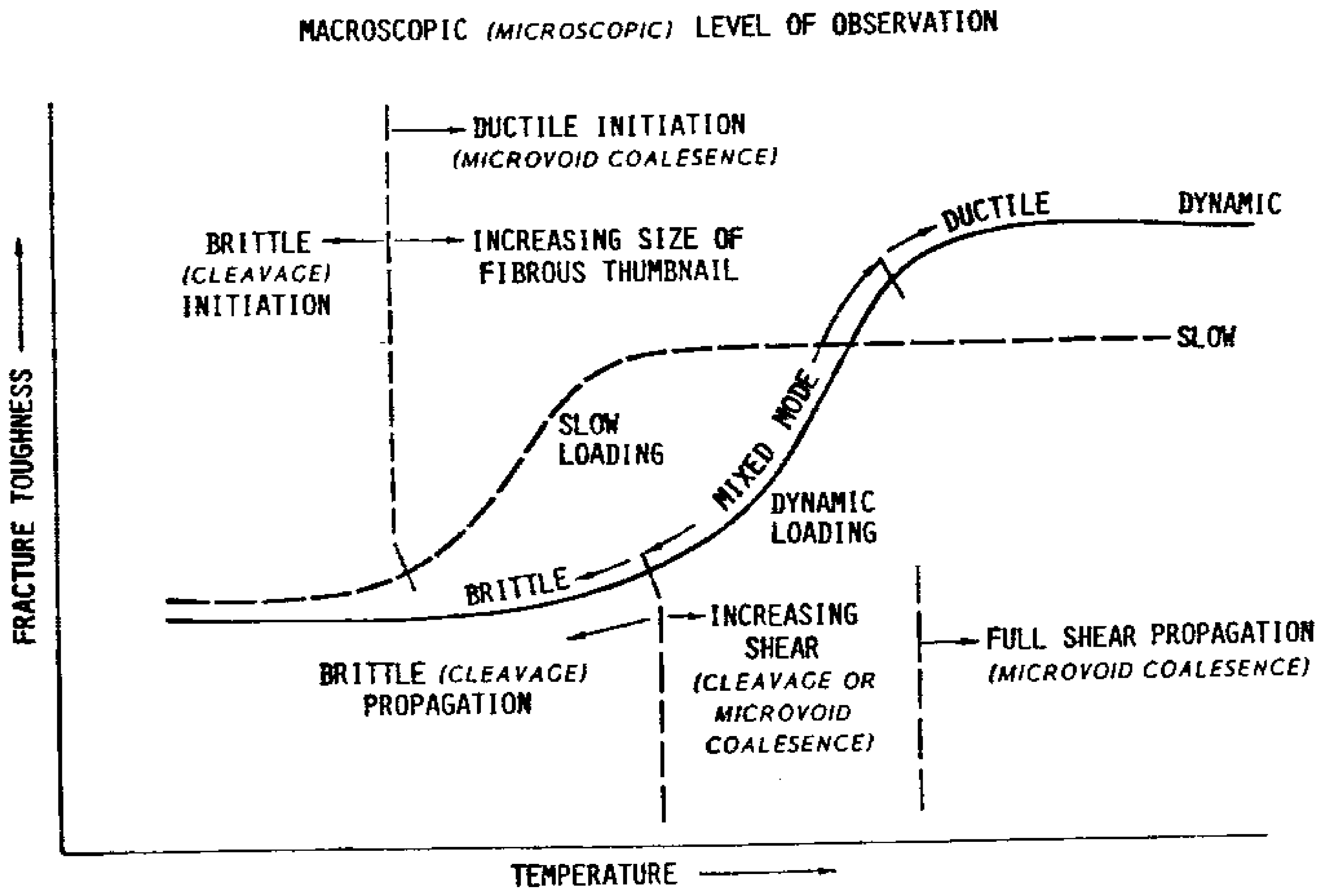

Fig. 9 - Schematic showing relation between slow and dynamic fracture toughness and fracture appearance.

Figure 10 (8) demonstrates that the temperature shift between static and impact fracture toughness curves for A36 steel is about 150°F. Figure 11 (8) shows that the temperature shift between static and impact fracture toughness is directly related to the yield strength of the steel and decreases in magnitude as the yield strength increases. The strong dependence of fracture toughness for low strength steels on loading rate, and the higher fracture toughness of thin plates compared to thicker ones ensure the adequacy of presently used steels for storage tanks, welded water pipe and other similar structures.

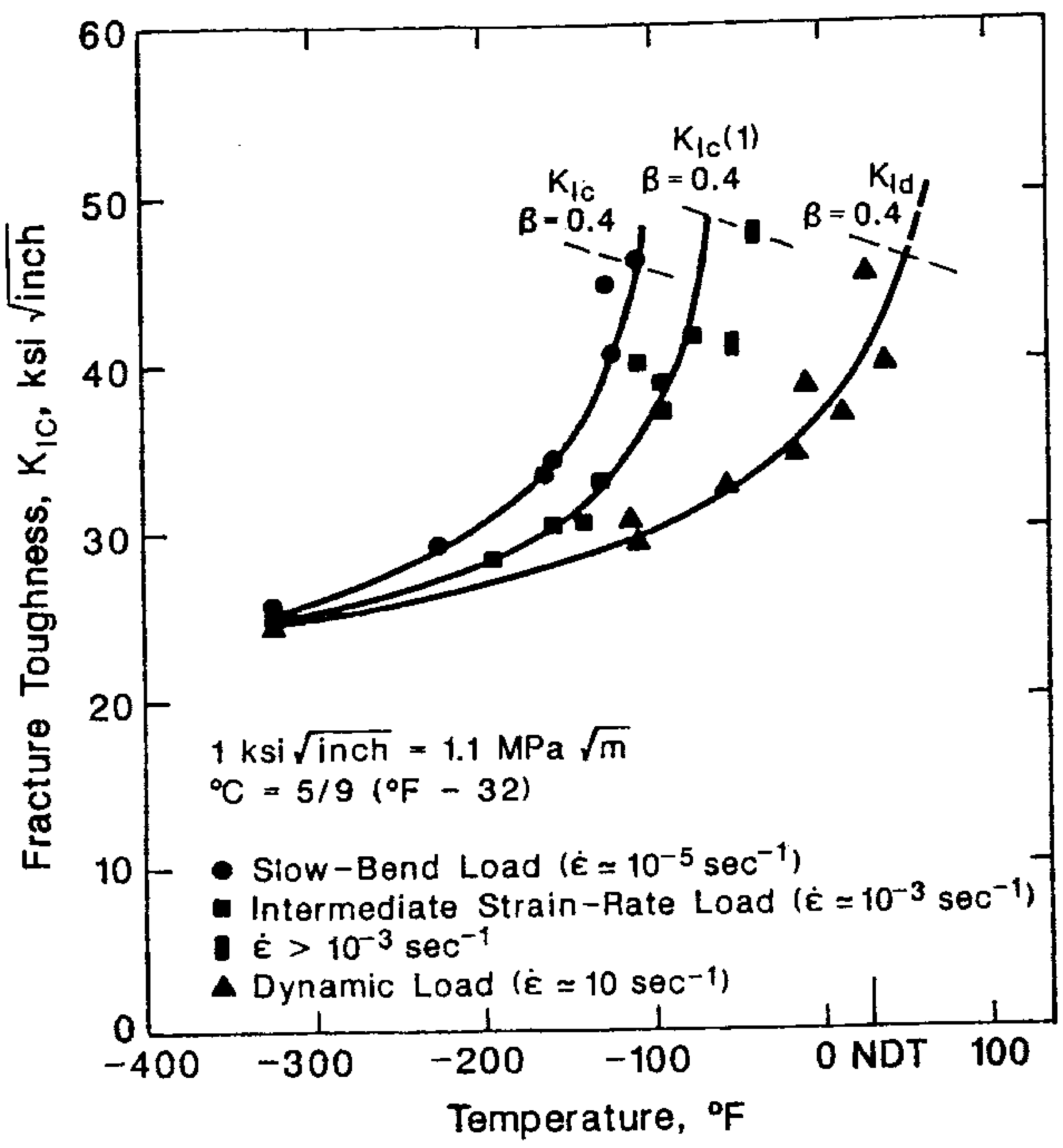

Fig. 10 - Effect of temperature and strain rate on plane-strain fracture-toughness behavior of A36 steel.

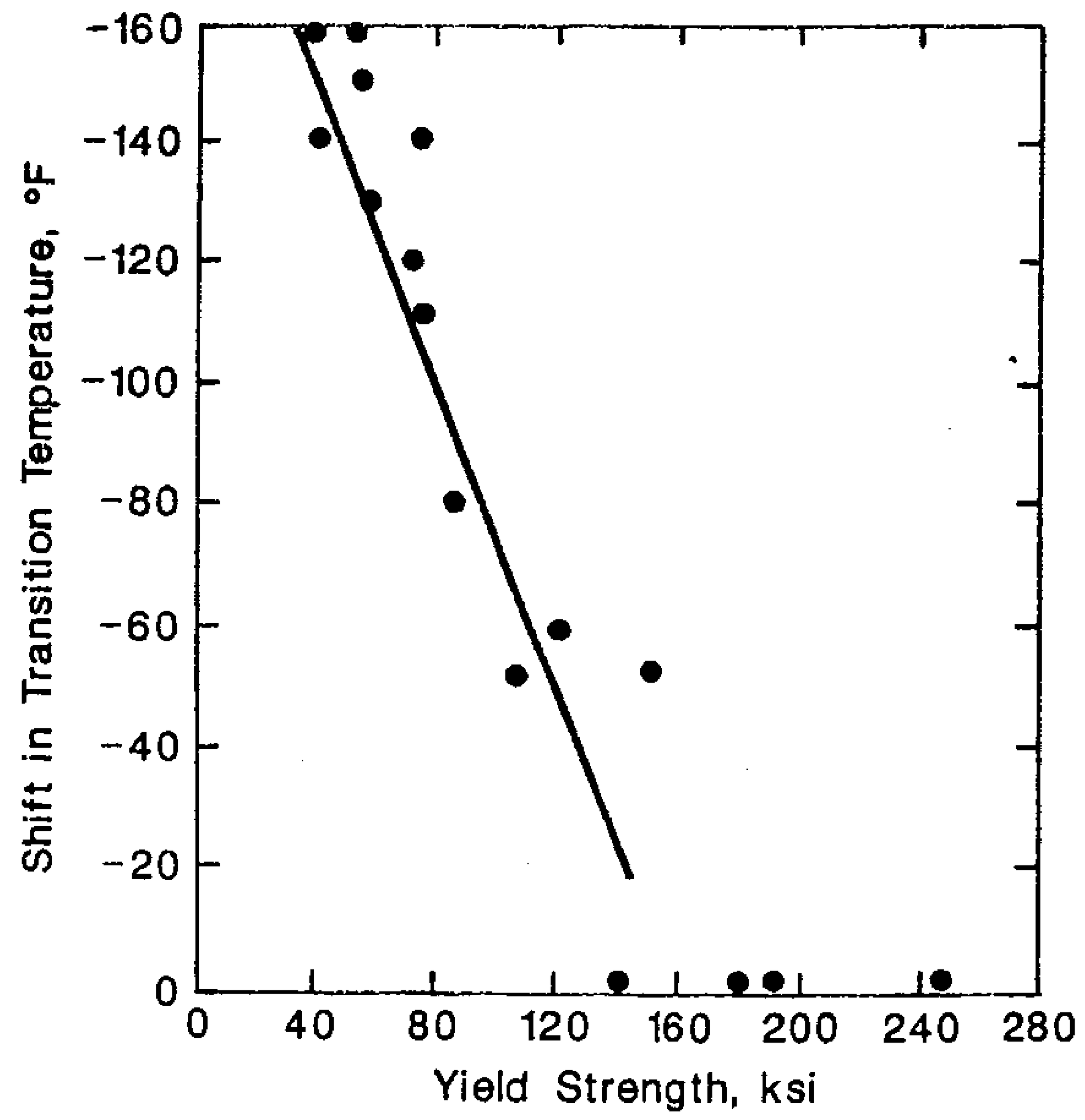

Fig. 11 - Effect of yield strength on shift in transition temperature between impact and static plane-strain fracture toughness curves.

Large storage tanks are field-welded usually without preheat, and welded pipe sections are joined together in the field usually without preheat to form a pipeline. Although heat-affected zone properties have not caused problems in these applications, the excellent heat-affected zone properties of low-carbon high-performance steels may increase the margin of safety for these structures. Also, cracks have propagated at corners of improperly designed and unstrengthened large openings in storage tank walls. The high fracture toughness of low-carbon high-performance steels may tolerate some deviation from proper design and fabrication but cannot prevent damage or fracture caused by serious error. Low-carbon conventionally processed steels with high fracture toughness have been available in thin sections for use in these and other applications. Thus, high-performance steels may not be necessary for use in these applications.

Large diameter gas and oil transmission pipelines are statically loaded structures fabricated from thin plates (less than about 1-inch thick) of medium and low-strength steels ($\leq$ 70 ksi). These are critical structures whose failure may cause loss of life and severe property and environmental damage. Therefore, steels used for this application have very high fracture toughness, corresponding to shear crack propagation behavior at minimum operating temperature. Welded sections of pipe are girth-welded in the field to form a pipeline. Consequently, low-carbon high-performance steels have been used extensively for this application. The use of controlled rolling followed by accelerated cooling in very low carbon to produce low-carbon-equivalent and high fracture toughness steels is highly desirable for this application.

High-rise buildings are statically loaded structures. However, unlike storage tanks, water pipe, gas and oil transmission lines, they are fabricated from very thick plate to maintain strength. Usually, high-rise buildings are fabricated from steels having 50 ksi or lower strength. The use of high-strength steels can result in immediate savings from the use of thinner sections. In addition to the previously enumerated benefits derived from welding, inspecting and handling thin sections, their use decreases the weight of buildings, which reduces the cost of the foundation, and increases the usable space in buildings.

In general, fracture toughness is not a primary consideration in the design of high-rise buildings. The high deposition rate electroslag welding process is used routinely for welding thick plates for buildings. This process would not be suitable for welding high-strength quenched-and-tempered conventional steels. Therefore, the development of low-carbon high-strength thick plates that are weldable by using electroslag and electrogas welding processes is very desirable.

Bridges are subjected to fluctuating loads superimposed on the static load of the structure. Fluctuating loads may initiate and propagate fatigue cracks from weld imperfections or at stress concentrations such as occurs at weld terminations and weld toes (9). The fatigue crack propagates from subcritical dimensions to a critical size that causes fracture. The critical size of a crack is determined by the applied stress and by the fracture toughness of the material. Thus, fatigue and fracture toughness properties of the steel and weldment are very important for bridges.

The fatigue properties of welded components are governed by stress fluctuation, by the size and shape of weld imperfections, and by the geometry of the welded component. Extensive fatigue tests on welded components show that steel properties do not affect the fatigue life (9). The use of high-performance steels would not have a measurable effect on the fatigue performance of steel bridges.

Presently, bridges may be fabricated from plates up to 4 inches thick having 36-, 50-, 70- or 90/100 ksi minimum yield strength. Because fracture toughness is an important factor for the safety and reliability of steel bridges, the use of steels with high fracture toughness is desirable. High levels of fracture toughness may be achieved by using different processes depending on strength and thickness of the plate. Hot rolling, controlled rolling and controlled rolling followed by accelerated cooling can produce high fracture toughness thin plates of low-carbon low-strength steel. On the other hand, quenching and tempering or direct quenching and tempering are required to produce high-strength thick plates with high fracture toughness.

About 80 percent of the steels used for bridges are of 50 ksi minimum yield strength. The use of the most recently added A852 steel with 70 ksi yield strength has been increasing steadily. Many factors have limited the use of the 90/100 ksi yield strength steels. One of these factors is the special care required in welding to avoid hydrogen cracking. However, the use of a 90/100 ksi steel and, to a lesser extent, a 70 ksi steel will continue to be limited with present bridge design and configuration even when the steels possess excellent welding properties. Presently, the use of high-strength steels for bridges is limited by live load deflection, vibration, buckling strength and fatigue resistance, which are independent of steel properties. Deflection, vibration and buckling strength are adversely affected by use of slender high-strength components, especially for the I-shaped girder configuration prevalent throughout the United States.

Maximum allowable design stress may be increased in proportion with increased strength to take advantage of high-strength steels. However, fatigue strength of welded components is determined by stress range and is independent of steel properties. Consequently, stress range cannot be increased with increased strength and high-strength high-performance steels would not measurable improve the fatigue performance of welded bridge components.

Design of steel structures is based in part on the assumption that components and connections have the capacity to deform plastically prior to fracture or collapse. Usually, plastic deformation is defined by the ability of a structural component to rotate and deflect inelastically under a specified load. For example, LRFD specifications by AASHTO and by AISC provide for a rotation capacity of compact section flextural members that are three times that required to reach the yield moment. On the other hand, the AISC seismic design of these members provide for a rotation capacity of about seven times that required to reach the yield moment. This example demonstrates that the required plastic deformation capacity is different for different applications. However, plastic deformation capacity of steel members and connections depends on many other factors. These factors include steel properties, weldment properties, component and connection geometry, and stress and strain triaxiality (constraint) at the connection. Thus, steel properties are one of several factors that define plastic deformation capacity of components and connections and in most instances is the least significant. Steel properties include yield strength, tensile strength, stress-strain behavior, ductility and fracture toughness. Stress-strain curves define yield strength, tensile strength, ductility and energy absorption capacity measured by the area under the stress-strain curve.

The shape of a stress-strain curve is different for different steels. Steels may have the same yield-tensile ratio yet exhibit different ductilities, strain hardening exponent, especially when they have different yield plateau lengths, energy absorption and fracture toughness. In spite of these differences, yield-tensile ratio has been used to compare different steels. This over simplification is complicated further by imposing a maximum limit on yield-tensile ratio to ensure that structural connections would have the capacity to plastically deform, rotate and deflect prior to fracture or collapse (10). Steels with yield-tensile ratios up to 0.95 have been used successfully in bridges and buildings and cold-formed members with yield-tensile ratios up to 1.0 have been used for limited applications (11).

High-strength high-performance steels may exhibit high yield-tensile ratio, Figure 12 (12). These steels exhibit higher elongation and therefore higher ductility and energy absorption than conventional steels. Consequently, the significance of yield-tensile ratio on the behavior of structures fabricated from high-performance steels should be reevaluated. This is especially true for seismically loaded structures where cyclic softening and cyclic hardening characteristics of the steel may be more significant than yield-tensile ratio.

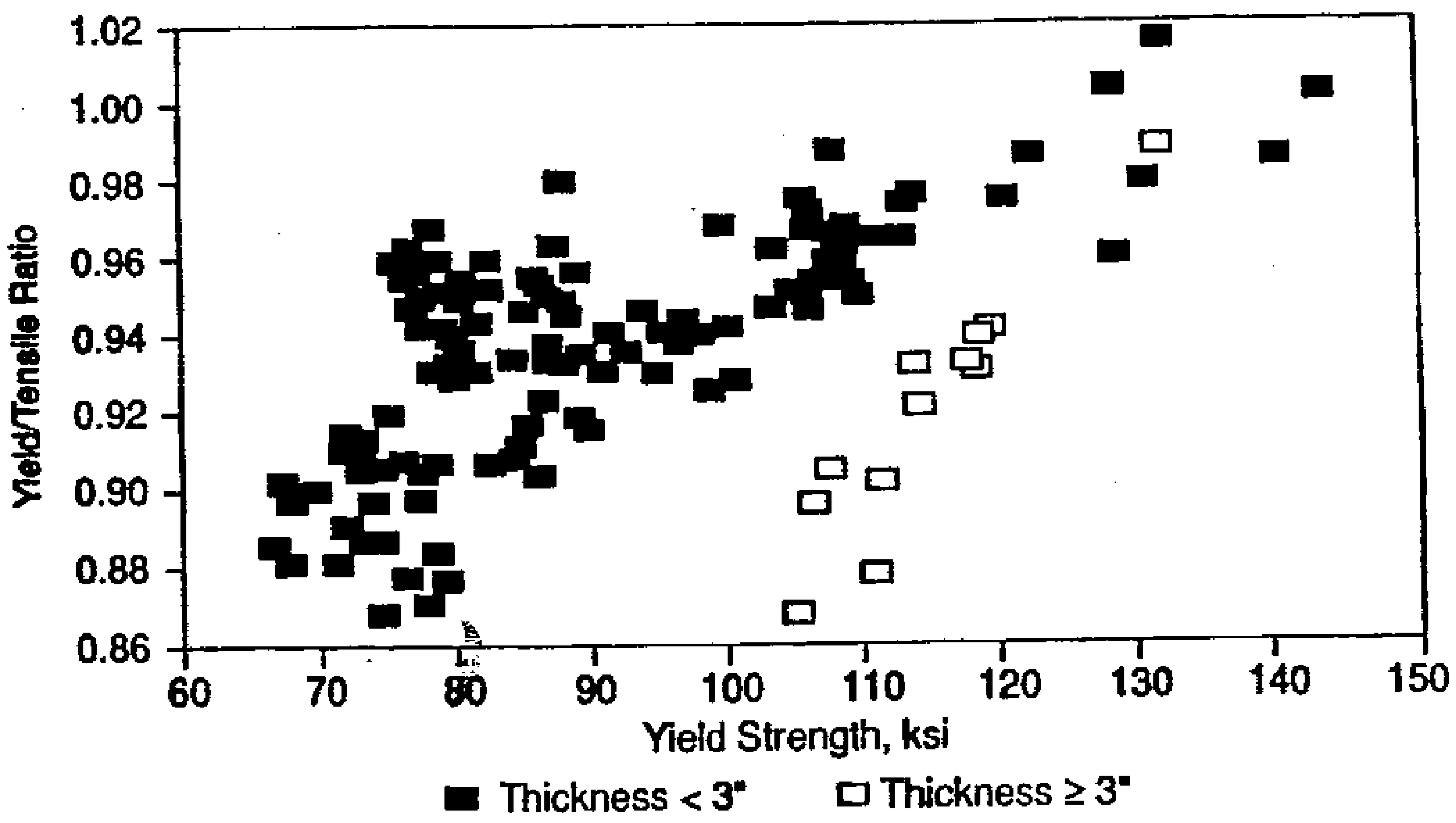

Fig. 12 - Yield-tensile ratios for Grades 65, 80 and 100 high-performance steels.

All the benefits that can be derived from improved weldability and high fracture toughness of low-carbon high-performance steels would have immediate benefits to steel bridges. Elimination of preheat, high resistance to hydrogen cracking, use of high deposition rate welding processes, ease of weld repair, field welding and high fracture toughness are very beneficial even to present design and configuration of steel bridges. An additional benefit would be to relax the stringent requirements imposed on fabricating fracture-critical members and to eliminate differences between fracture-critical and non-fracture-critical members. Such a change would streamline fabrication, improve weldability and handling and result in economical bridges with improved safety. However, changes in present AASHTO specifications and new innovative bridge designs are essential to utilize fully the attributes of high-strength high-performance steels. These are two objectives of an FHWA project on "Innovative Bridge Designs Using Enhanced Performance Steels." (13)

The preceding discussion demonstrates that the significance of a given material property to the performance of structures is dictated by the characteristics of the particular structural type. Consequently, properties of high-performance steels should be tailored to the needs and requirements of the structures. This in turn would define composition and processing to obtain the desired steel properties.

Low-carbon high-performance steels especially those having high strength present an opportunity for steel producers, fabricators and designers to change existing specifications and conventional procedures and to develop new innovative designs that are safe, reliable and economical. Research will continue to provide new knowledge and better understanding. However, the opportunities that high-performance steels present should be pursued now by incorporating present knowledge in specifications and in actual or demonstration structures.

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