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Evaluation of Experience

Leçons à tirer des expériences passées

Auswertung von Erfahrung

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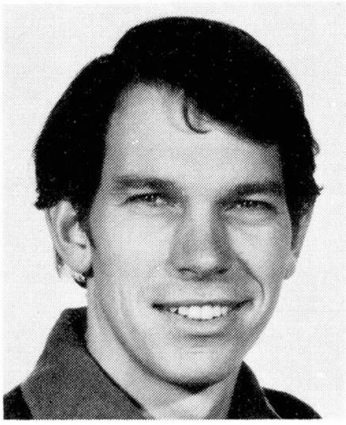

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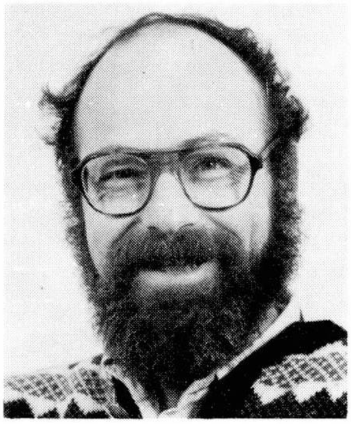

SUMMARY
This Introductory Note summarizes the experience gained from the study of structural failures and satisfactory construction and comments on the accuracy and completeness of reporting. Comparison is made of the findings of a number of investigators according to type of failure mode, structural elements affected, time of failure, prime causes of failure, reasons for their occurrence and their consequential cost. Most failures can be shown to occur because of gross human errors. The nature of these errors is discussed and the requirements for the evalutation of experience in the future is considered.

RESUME
L'article résume l'expérience acquise dans l'étude d'accidents structuraux par rapport à des comportements corrects. Des commentaires sont faits sur la précision et la méthodique des études. Divers aspects sont traités, sur la base de travaux de plusieurs chercheurs, tels que mode de rupture, éléments structuraux concernés, date de la rupture, causes premières et conséquences financières. Il apparaît que la plupart des accidents sont dus à une grossière erreur humaine. La nature de ces erreurs est passée en revue et les conditions requises pour tenir compte de ces expériences à l'avenir sont évoquées.

ZUSAMMENFASSUNG
Der Beitrag fasst die Erfahrungen zusammen, die aus Erfolg und Misserfolg im Bauwesen gezogen werden können und äussert sich zur Genauigkeit und Vollständigkeit entsprechender Berichte. Die Berichte einiger Autoren werden in bezug auf Versagensart, betroffene Bauwerkskomponenten, Zeitpunkt, Hauptursachen und deren Wurzeln sowie bezüglich Schadenkosten miteinander verglichen. Es zeigt sich, dass die meisten Schäden auf grobe Fehler zurückzuführen sind. Die Art dieser Fehler sowie die Anforderungen an eine geeignete Auswertung von Erfahrungen werden diskutiert.

1. INTRODUCTION

It is well known that as children we learned from our experiences; probably learning more quickly from our mistakes than from our victories but nevertheless learning. However, we did not all learn the same lessons. What we learned depended on our childhood environment and the prevailing level of technological achievement in that environment. Thus, today, many children have the opportunity to experience different cultures and environments as a result of the ease long-distance travel; whilst others learn how to operate and use microcomputers. In contrast, few children today learn at an early age that fire is dangerous and hot - central heating took care of that.

Just as in childhood the lessons learned from interaction with one's environment depend on that environment, so in engineering the lessons we as practising engineers learn from interacting with day-to-day engineering and with other engineers depend very largely on the environment in which we find ourselves. It is therefore relevant to ask some questions about what it is that we learn from our (collective) experiences, how relevant that experimental learning is to modern engineering and how such experimental learning can be best attained. It is to questions such as these that this introductory note is addressed.

In what follows, we shall attempt to review the types of past experience from which engineers have typicaly made evaluations, to review how such evaluations have been made, and to consider the validity or otherwise of the conclusions that have been drawn. We shall use these findings to make some suggestions concerning the Quality and Safety of Structures and the implications for Quality Assurance matters. In so doing we shall touch on the importance of human error and measures to reduce it, and consider organizational matters briefly. These topics will be taken up more fully in later Introductory Reports. Finally we propose a number of matters for consideration for future evaluations of engineering experience. Not only should we learn from past experience, but we should learn from past attempts at the evaluation of experience.

Summaries of a number of case histories of structural failures are given in an Appendix. These are not for consideration in their own right, but are intended as examples of circumstances which can arise in practice, and against which quality assurance measures must be designed to be effective.

2. PAST EXPERIENCE

2.1 General Remarks

It is readily evident that past experience comes in a continuum; the vast majority of experience is middle-of-the-road, mainly good, positive experience, generally reinforcing the status quo of accepted theory and practice, with minor excursions into problem areas. Seldom are major problems encountered; but when such negative experience does occur it seems to be largely in the form of individual events - there rarely appears to be much warning of a build-up to the problem experienced. Examples here include (i) Ronan Point, where problems of connectivity under blast loadings were not obvious before the event; (ii) Quebec Bridge, buckling of curved members; (iii) Westgate Bridge (and others at about the same time) with stiffened-plate buckling problems.

When thinking of these examples, it must be remembered that the interpretations noted are rather subjective, and are a function of the time at which the events occurred. The interpretation of experience will be considered in more detail in the next section.

It is important to note that both positive and negative reinforcement is required to defined experience properly; merely knowing that a particular technique has worked in the past under certain circumstances does not help very much in extrapolating to a new situation. Negative reinforcement, that is, also knowing when something did not work, can be extremely useful is setting bounds [1].

Unlike data gathered in a scientifically controlled experiment, the "data" represented by a continuum of experience, both "good" and "bad", is seldom truly comparable. The circumstances surrounding each structure are generally unique; whether this be in a contractual sense, a political sense, an industrial relations sense or an economic one. Similarly, the design concept and its realisation in design and construction is often quite different even for nominally similar structures. Hence, in no sense can one imagine that a structure is a sample from a real homogeneous population, but rather it is a special case of generically similar, but not identical structures. Comparison between such structures therefore requires rather more care than might otherwise be the case. Nevertheless, various attempts have been made to use the behaviour of structures as raw statistical data for rather formalised evaluation of structural performance. Such evaluations have generally focussed on "bad" experience, while unformalized, unstructured evaluation is the norm for evaluation of "good" experience.

2.2 Evaluation of past experience

As noted, the formalization of past structural engineering experience of a positive nature (good or satisfactory experiences) takes place in a relatively informal way. The successful completion and operation of a structure is the expected norm, at least in relatively modern times, and hence informal surveys by individuals or organizations of structures similar to that which is being proposed is usually taken to be an adequate procedure. This may be supplemented by formal reports of successful construction and operation, such as published in learned institution journals. Some aspects of the experience so gained may ultimately find its way into codes of good practice and text books, and so become available to future engineers.

The formalization of structural engineering experience of a negative nature (bad experience) takes place in a greater range of ways. The more important of these are summarised in Table 1 together with an estimate of the reliability of the evaluation and the possible effect of the results on the engineering profession. Except for the last item, the methods employed are arranged in order of increasing frequency, and also, incidentally in approximately the order of decreasing reliability of information on which evaluations are based. Probably the most outstanding misfit is that of newspaper reports - a rather unreliable source of information.

In all the methods of evaluating experience, the results of the evaluation must reflect the quality of the data used and the biases affecting the evaluations, conscious or otherwise. For this reason, it is to be expected that a formal enquiry, such as a Royal Commission, will yield a much more precisely detailed and qualified evaluation than would be expected from a more limited enquiry. However, such formal enquiries are usually only instituted in the case of grave accidents, where it is also politically expedient to do so. It is not unlikely, therefore, that evaluations based on such formal enquiries, while admirable in themselves, also introduce bias into attempts to obtain more generally based evaluations [2]. A simple example is that serviceability-type problems are extremely under-represented in formal enquiries, in "in-house" reports and newspaper reports, and even in technical papers, but constitute a considerable proportion, if not the major part, of negative experiences in structural engineering, as assessed by individual and generally unreported observations.

Several attempts have been made to improve the collection, and hence the possibility for evaluation of past experience. Probably the more ambitious of these is the EPIC (Information Centre on Structural Performance) program, which is meant to function as a system of university-based data banks. More than 100 case summaries have been used to test the system and many cases are in the process of being prepared for computer processing. A similar system has been proposed within the CEB.

Some early accounts of failure of modern engineering structures include Thompson's description of American Railway bridges [3], Lossier's study of concrete structures [4] and Pugsley's description of bridge failures [5]. Rather more systematic accounts have been published since then [6-14], including the well-known study by Matousek and Schneider [9] and the first results from a BRE/CIRIA survey [14]. Data from these studies will be used in the next section to review the principal findings.

3. OBSERVATIONS FROM PAST EXPERIENCE

3.1 Modes of Failure

It is not possible, within the confines of the present report, to present an exhaustive overview of the many observations which have been made as a result of analysing past experience. What can be done, however, is to present some selected material, with a view to indicating trends. It will also become evident in so doing that there are considerable differences in the data that have been presented and that this complicates simple analyses. But analyses of the present type must rely on diverse points of view in order to avoid the real possibility that observation in obscured by preconceived ideas.

Table 2 shows a simple breakdown of the types of failure observed by various analysts. Although agreement is quite astonishing, it is felt that "serviceability" type failures are distinctly under-represented in these statistics (see above) [14].

There is also some evidence [14] to suggest that isolated failure events and progressive failures are about equal in number (Table 5) and that for ultimate strength failure modes (Table 4) rupture of the critical section, with or without formation of a mechanism, predominates. Curiously the British BRE/CIRIA [14] study found that instability problems accounted for 20-30% of cases, yet the European (Matousek & Schneider) [9] study found this mode to be negligible, with loss of equilibrium being an important factor. This type of inconsistency may well reflect classification difficulties rather than being the result of regional structural engineering practices; there appears at present no other relevant data.

For serviceability failure, the figures shown in Table 3 must be taken as a rough guide only since categorization here differs considerably between the studies reported. Some further interpretation has been done to arrive at Table 3. Nevertheless, it is clear that cracking in concrete structures, and excessive deformation generally are the most severe problems encountered.

The types of structural elements for which failures have been reported depend, of course, on the type and form of construction. An overview is given in Table 6. It should be noted that there are significant differences in the columns of figures, with the European study [9] listing many combined cases, and excluding foundation failures. The figures should not, therefore, be compared horizontally. If foundation problems are excluded, it is seen that slabs and walls are the elements most commonly involved in failure with beams and columns about half as often. The reasons for this rather suprising observation are not clear from the available data. In view of the high inherent safety (due to redundancy) and and overload capacity of 2-way slabs, one might expect that they would not figure large in failures. Review of Allen's [11] list of errors in concrete structures (in Canada) suggest that construction errors feature at least as often as design errors. Slabs-in-ground are included in his figures and some slabs might, therefore, be considered under the foundation category.

The results for the time of occurrence of failure, given in Table 7 show some inconsistencies between data sources. The European study [9], which consisted of 52% office buildings, and less than 12% bridges, suggests that on aggregate of those structures that fail about 60% fail during construction, with a gradual reduction of failures with time after construction. These results are not significantly changed by isolating office buildings, but differ somewhat from the British study [14], which has rather fewer (31%) of buildings failing during construction, and another 30+% within 5 years of completion. This latter breakdown agrees better with the European figures [9] for factories alone. The greater failure rate for factories during service life is, of course, to be expected, since factories are more likely to be exposed to unforeseen loadings and misuse than office buildings.

If bridges are considered, the inconsistency in statistics is even greater. It is commonly held that because of the low live load/dead load ratio for bridges, the selfweight of the bridge provides a certain amount of proof loading, so that failure during construction should be relatively high. This accords with the small samples (75) of bridges in the European study (c. 70% failure during construction) but is in stark contrast to that report by Smith [8], who has only 16% failure during construction. Part of the reason for this difference is that Smith's study reports that about 50% of failures were caused by flooding (see Table 8). This included two cases of floods in which nearly one-third of his total sample failed. If these cases of simple bridges are removed, his figures fall more closely in line with the others.

3.2 Causes of Failure

The various reports on the reasons for structural failure are not easy to compare directly although the messages contained in the reports are much the same, even if not always spelt out. Smith's summary [8] of prime causes (Table 8) has already been mentioned. The British study [14] again lists prime causes (Table 9). "Inadequate appreciation of loading conditions or real behaviour of structures" stands out as the major problem area, with "grossly inadequate execution of erection procedure" as next in importance. Somewhat similar conclusions were reached in the European study [9], as is shown in Table 10, which lists the types of errors, assessed by the study group, to have been committed in the cases studied. This represents an attempt to move from prime causes to underlying causes, although it is self-evident that such a progression cannot be bounded. Nevertheless, it shows that human error is a principal problem and that "insufficient knowledge" together with "under-estimation, neglect, error and ignorance, thoughtlessness and negligence" constitute by far the most important components. The European study [9] went into considerable detail about the further breakdown of the figures in Table 10; however, this need not be of concern here. What is of interest here, and for the later Introductory Reports, is the estimation of how effective additional control measures in the building process might be in order to discover errors (Table 11). Although these results are subjective, they do have a certain commonality with the results from the ACI survey [12] (which includes the results of Allen's Canadian study [11]) and which indicate that of the errors which were detected, a significant proportion were detected by succeeding people in the design-construction sequence. It is also of interest in this context to note that in the British study [14], it was found that in about 50% of cases studied, it was felt that the "absence of a person with authority" or lack of an effective "project leader" was a major factor contributing to structural failure. Such a person would offer not only leadership but control as well. A related finding is given by Bentley [15] in his assessment of building site quality control.

3.3 Costs of Failure

The data on the costs involved in structural failure are not particularly compatible. Table 12 shows estimated total economic costs of failure relative to original costs; it is clear that generally this is about 10% with only very occasional greater costs. The European study [9] has a lot of data on cost, injuries and fatalities. However, it does not appear possible to present this information in the form of Table 12. It can be derived that in the majority of cases (>90%) there are no deaths or injuries, and that for about 95% of failures the failure costs are less than 5% of original construction costs.

4. IMPLICATIONS FOR QUALITY ASSURANCE

The collected data and evaluations of the previous section lead to a number of topical conclusions. These are of interest in themselves. More generally, however, some overall implications can be drawn from the data and a number of tentative conclusions about their importance for quality assurance can be made. These will now be discussed.

4.1 Relationship between Reliability Theory and Quality Assurance

If it is accepted that the above data are reasonably representative, although probably biased towards major accidents rather than serviceability problems, it is reasonable to conclude that the majority of failures occur because of unexpected combinations of circumstances, and because of gross numerical or conceptual errors in the processes of design and construction. Failures rarely occur (in modes for which the structure has been designed) because of the chance combination of "low strengths" and "high loads"; but this is to be expected, since the deterministic design codes used in most countries have sufficiently high safety factors (or partial coefficients) to ensure that both the theoretical failure probabilities and the actual relative frequencies experienced in practice (in these modes) are extremely small.

Structural reliability theory has been used to a significant extent in the development of deterministic structural codes e.g. [16] and in the rationalization of partial coefficients (partial safety factors), and it should therefore be questioned whether these applications are meaningful if most failures occur as a result of circumstances which are not normally considered in any formal analysis.

Studies have shown, however, (e.g. see chapter 13 of [17]) that in many cases, the choice of partial coefficients, which in effect govern the amounts of materials used in a structure, can be made independently of the choice of the measures adopted, and the resources spent on, checking for the occurrence of gross (human) errors. Some other work in this direction by Allen [18] suggests a trade-off between control procedures and notional reliability. This has long been an accepted approach to the design of structures such as cranes under fatigue loading.

4.2 The Importance of Gross Human Errors

From the early, rather intuitive work of Pugsley, it was already evident that various aspects of human errors had played an important part in the occurrence of a number of quite spectacular engineering failures. Objective backing for this thesis has been provided by the work of Matousek and Schneider [9], Allen [11], Sibly and Walker [13], Walker [14] and others, and it is not generally accepted that gross human error plays a critical part in the malperformance or failure of structures. Similar conclusions [19] have been reached in other industries.

The term gross human error is used here as a substitute for the terms "human error" and "gross error" often found in the literature on this subject. The two latter terms are not entirely appropriate; "human error" because <u>all errors</u> are directly or indirectly caused by defective human behaviour, and "gross errors" because this implies that the errors are necessarily very large. By gross human error we mean an error of concept, of calculation, of design, of construction, or of maintenance which gives rise to a gross misunderstanding of how a structure will behave at some or all stages of its life, or how it would behave under hypothetical loads of different magnitudes.

However, although the acceptance of these ideas has been rapid and widespread, and despite the studies mentioned above, it is reasonable to conclude that there is still not very much objective information about the nature of gross human errors and their occurrence. A number of relevant questions are as follows:
(a) What percentage of errors committed in design and construction actually lead to failure of the structure (however failure is defined)?
(b) What percentage of errors are not detected and are built into the structure as a consequence? What effect does this have on the structure?
(c) What percentage of errors which are detected are then satisfactorily corrected?

Answers to questions such as these are essential if procedures to control gross human errors are to be established in the rational way; several suggestions for such procedures have been made [20-23]. However, all suffer from lack of data in defining critical parameters. Although this matter is discussed in more detail in Introductory Reports 4 and 5, it is worth noting here that although it has been suggested [22] that (i) education, (ii) personnel selection, (iii) task complexity reduction, (iv) quality control procedures, and (v) the legal framework, are all important in reducing human error, not all are equally effective. In fact, it is not unreasonable to state that there is virtually no objective data on the effectiveness of any of these activities or procedures, since, apart from the status quo in each country, there is an almost total lack of experience in changing any parameter.

About the only recent experience which exists is that relating to human error and control measures, as reflected in the Matousek-Schneider [9] study. This showed that lack of control can lead to structural failure, but the converse, that greater control reduced the incidence of failure, is not so easily deduced. For that type of evidence, one has to look back to the reasons for the introduction of material quality control, for the reasons for requiring design checking, etc. Interestingly, the motivation in each case can be found in ensuring a level of protection for society, rather than for individual designers or contractors or clients (c.f. Melchers [24]).

Similarly, the necessity for education, and in particular, continuing education, of engineers is usually accepted without question. In view of the studies by Sibly and Walker [13] for example, this is wholly reasonable. However, our experience does not yet lead us to be able to suggest how the continuing education of engineers might best be achieved in terms of obtaining more reliable structures. Greater emphasis on, or skill in, say, engineering mathematics may be quite counter-productive in this setting.

Finally, the experiences summarized in the previous section make little reference to quality assurance techniques in other industries, perhaps with different levels of technology. It is inconceivable that the human error problems in structural engineering are unique and further comparisons of experience in different industries in the future are almost certain to be of value.

4.3 Organizational Effects

The importance of the organizational structure of a project is recognized as an important parameter in quality assurance of structures (c.f. Introductory Reports No. 1 and 5). However, it is equally true that there have been only very preliminary efforts to evaluate experience in this area (c.f. Melchers [25]).

Matters of importance for quality assurance in an organizational framework include the contractual and actual relationships between organizations, the managerial competence and managerial aspirations of each organization as well as professional jealousies, personalities and pride. Other matters of importance may include industrial relations matters and the availability of proprietary technologies to individual organizations.

To complicate matters, any investigation of organizational effects must take into account national systems of contract and legal liability. This may well invalidate comparisons between countries with Anglo-Saxon based legal systems and those with other systems.

4.4 Unimaginables

It is often suggested that ultimately the quality assurance of structures is hampered by co-called "unimaginables", events or oversights leading to disastrous consequences which cannot be imagined beforehand. Such events may include a previusly unforeseen loading condition, or an unrecognized mode of structural behaviour. There certainly appears to be evidence to support such a notion (e.g. Tay Bridge - wind loading; Tacoma Narrows Bridge - wind oscillations...). However, closer examination of cases such as these has revealed that in nearly all circumstances there were known antecedents for the observed loading or behaviour (e.g. Sibly & Walker [13]). Unfortunately, such antecedents were not always recognized by the designers, or were consciously set aside as being of insignificant importance. The available evidence suggests very strongly that so-called "unimaginables" constitute a negligible proportion of all structural failures. Hence, it would appear reasonable to ignore them in the first instance as factors in quality assurance considerations.

4.5 Costs of Structural Failure

Despite considerable preoccupation by structural engineers with the safety of their structures, it is evident from the figures now available that very few structures fail in an ultimate strength sense and that more, but still relatively few, show serviceability failures. It is tempting, therefore, to suggest that fear of failure for structural engineers is not so much based on monetary loss as on loss of prestige and livelihood, and that a case might be advanced that the profession is already achieving a sufficiently low level of structural damage cost taken in aggregate, without needing to be unduly worried about matters such as human error. Alternatively, the balance between quality control procedures and statistical variability could be altered so as to lower traditional safety factors and recover the concomitant increase in rsk of failure through increased control of structural quality (i.e. human error related problems). Techniques for developing such a new balance are becoming available (e.g. Allen [18]); the desirability of such a stance should be a matter for considerable discussion in the profession.

5. FUTURE EXPERIENCE EVALUATION

The evaluation of future experience, may be considered to consist of two parts: that of evaluating the experience itself and that concerned with assessment of the evaluation techniques. Both of these matters ought to be given attention by the profession.

Up to the present time, little attention has been given to the differences that are almost certain to exist between failure rates and types of failure in structures and structural components associated with different levels of technology. Benefits could be gained by studying past experience, both now and in the future according the level of technology. Three levels would probably be sufficient:

- high level technology e.g. nuclear plants
- medium level technology, e.g. bridges, offshore structures, important buildings, etc.

- low level technology, e.g. small structures, domestic housing, etc.

In the past, there have been different levels of control applied to each technology and we now need to know its separate effectiveness.

In addition to the above and the various detailed matters raised in this report, the additional requirements for future experience evaluations seem to be as follows:

(a) determination of appropriate procedures for analysing structural failures, whether this be at ultimate load level or at service load level; the EPIC program is one possible approach, reruns of studies of the type conducted by Matousek and Schneider is another. However, other approaches may exist and ought to be examined;

(b) determination of appropriate procedures for drawing together structural failure analyses such that soundly based conclusions can be derived; the present report is in part one such effort. However, a formalized procedure might be more appropriate once a regular flow of information is available;

(c) procedures for minimizing the influences of national legal systems and local legal constraints on the study of structural failures;

(d) similarly, procedures to isolate organizational differences in projects involving structural failure, and hence attempting to make various cases comparable;

(e) procedures which will allow the effectiveness or otherwise of particular quality assurance schemes to be objectively assessed; thus, for example, allowing the effectiveness of the French 10-year legal liability scheme for buildings, as is being studied for introduction into the U.K., to be objectively assessed against conventional U.K. procedures; and

(f) development of appropriate feedback systems and trend warnings mechanisms such that the profession can get sufficient and timely warning of unfavourable trends in structural engineering practice or theory.

Of the greatest importance, however, is the need for a means of assessing the effectiveness of changes in control on both the design and construction processes on the occurrence of gross human errors of the various types.

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APPENDIX: CASE HISTORIES

This appendix includes the summaries of four case histories of structural failures. Full details are not given but further information can be obtained from references [26-29]. The purpose of these summaries is to give examples of circumstances which can arise in practice and against which quality assurance measures must be designed to be effective. The case histories are of failures which have occurred in recent years and involve a variety of consequences, some simple and some complex.

A.1 Falsework collapse, Chicago, U.S.A. - April, 1982 [26]

On 15 April 1982, two spans of a partially completed post-tensioned concrete bridge, being constructed at Riley Road interchange in East Chicago, collapsed during the casting of the deck, killing 13 men. The bridge was of in situ pre-stressed concrete box-girder construction and was to an alternative design put forward by the contractor.

At the time of failure, three of the spans had been completed and post-tensioned, and the bottom and sides of the box-girder for the fourth span has been cast. Until the completion of the post-tensioning operations in each span, virtually all the loads (both permanent and temporary) were supported by the temporary falsework consisting of pairs of isolated high-capacity shoring towers close to the permanent concrete piers and at 1/3 points in each of the 54.9 m spans.

The failure occurred during the casting of the deck slab for the fourth span when about 100 m length of the partly finished bridge and its supporting false-work collapsed. On investigation it was found that the falsework as built was substantially different in several vital details from that envisaged in the design. The collapse was probably triggered by the excessive settlement of one of the temporary foundation pads of one of the shoring towers at the 1/3 span position. This caused an increase in the reactions provided by the other pads which were under-designed and thus cracked. The differential settlement of the foundations caused an estimated increase in the loads in the diagonal bracing members of the tower to about 40 kN which was grossly in excess of the average value of about 28 kN for the buckling strength of the tubes, determined from later tests. This partial tower failure induced a slight sway at the top of the tower causing the main cross-members supporting the bridge to be eccentrically loaded. The welds holding these in place fractured and one cross-beam fell away imposing an eccentric load on the tower which then buckled and collapsed, precipitating collapse of the two partially-completed spans.

On subsequent investigation it was found that:

- the temporary foundations pads for the towers had been constructed on top of about 3 m of compacted fill, but this overlay 300-600 mm deep pockets of highly compressible black organic silt.
- the temporary foundation pads were only 300 mm thick, whereas ACI Standard 318 would have required a thickenss of at least 530 mm.
- the external guys originally designed to prevent sway of the falsework towers had been replaced by internal X-braced cables.
- the main cross-beams at the top of the towers were initially eccentrically loaded because wedges had been omitted.
- some cracks in the foundation pads had been noted by the site surveyor a few days before the collapse, but their significance had not been appreciated.

A.2 Walkway collapse, Kansas City, U.S.A. - July 1981 [27]

On 17 July 1981, two suspended walkways in the Hyatt Regency Hotel, Kansas City, collapsed without warning killing 13 people and injuring 186 others attending a dance. The two walkways which collapsed were two of three which spanned the hotel foyer and linked the hotel's newly opened guest block to its convention centre. At the time of the collapse about 60 couples were dancing on the second floor walkway, about 40 on the one at the fourth floor level, and others in the foyer below. Each spanning 40 m, the 2.2 m wide steel/concrete composite walkways were hung from the ceiling by three pairs of 30 mm diameter asbestos clad suspension rods. The two walkways which collapsed (those at the second and fourth floor levels) were vertically above each other and were hung from the same suspension rods. The third floor walkway which did not collapse was suspended independently and to one side of the others.

Each walkway comprised a steel frame resembling a horizontal ladder with 460 mm x 305 mm longitudinal I beams and 200 mm x 100 mm cross beams at 2.5 m intervals, supporting a 75 mm thick concrete deck laid on permanent steel shuttering. At the three suspension positions on each walkway, the designer had provided a transverse box beam fabricated from two channels welded toe-to-toe. The suspension rods passed through holes drilled close to the ends of each box beam and the loads from the walkways were transmitted to the rods by the provision of washers and nuts threaded onto the rods.

It had been deduced that failure was initiated at mid-span on the fourth floor walkway by the suspension rods and nut pulling up through the box beam. The load it had been carrying was immediately redistributed to the five remaining suspension points which were then overloaded and failed in rapid succession.

On investigation it was found that the walkways had not been fabricated as originally designed. In particular, the suspension rods were made discontinuous at the fourth floor level instead of running continuously from the roof to the second floor. This changes meant that the loads from the second floor walkway were transmitted to the fourth floor walkway, effectively doubling the load at the fourth floor connections. Laboratory tests, however, showed that the connections were capable of withstanding only 27% of load which they should have been able to carry with the modified arrangement of suspension rods. Thus failure would have occurred at loads far less than the design loads, even if the walkways had been fabricated as originally designed. Other points of relevance are:

- the walkways' self weights were found to exceed the nominal self weight by about 8%.
- an estimated 63 people were on the walkways at the time of failure but the dynamic contribution to the loading was considered to be small.
- the connection failure mode was ductile involving upward rotation of the bottom flanges of the channel members of the box beams and extensive local yielding in the webs; nevertheless the overall failure was rapid with negligible warning.
- severe local yielding was found at the suspension points of the walkway which did not collapse.

A.3. Complete collapse of apartment block, Cocoa Beach, Florida, U.S.A. [28]

On 27 March 1981, a five-storey apartment block collapsed to the ground in Cocoa Beach, Florida killing 11 workers and injuring 23 others, during the casting of the roof slab. The building was a five-storey flat-plate concrete structure supported on 254 mm x 457 mm internal columns and 254 mm x 305 mm columns at each end of the 75 m long building. Many of the columns were left standing after the collapse indicating that the 203 mm thick slabs failed in punching shear around the columns. The vertical stacking of the slabs indicated that no sidesway occurred.

It was concluded that the collapse was intiated by the punching shear failure of the slab at an internal column, on the fifth floor. This propagated to other slab/column connections on the fifth floor which then fell, causing failure of all the floors below.

The low punching shear capacity at the slab/column connections was the result of two factors:

- the punching shear requirements of ACI Standard 318 which would have controlled the slab thickness were not considered in the design of the building. This was the code applicable under the local building regulations. In consequence the slabs were 76 mm less thick than they should have been (203 mm instead of 279 mm).
- the two-way top reinforcement in the slabs in the column strips was placed (or ended up) about 25 mm lower on average than specified in the drawings. This reduced the effective depth and the corresponding punching shear capacity of the slabs.

It is considered that both errors contributed about equally in bringing about the collapse and if either had not been made, the failure would probably not have occurred.

A.4 Partial collapse of Kongresshalle, Berlin - May 1980 [29]

In May 1980, a large section of the prestressed shell roof of the Kongresshalle in West Berlin collapsed 23 years after completion. Although the building was crowded at the time only one man was killed. The roof which was elliptical in plan, failed between its inner ring beam and the southern arch edge beam.

Although the structure was designed and built in accordance with standards which applied at the time, a number of faults were discovered which in combination led to the failure:

- the structure was sensitive to the action of extreme temperatures which led to large cyclic displacements and weakening of the concrete and prestressing steel.
- due to cracking of the concrete, the tendon stresses had in places increased by up to 100%, and so increased the risks of stress corrosion.
- eight prestressing tendons at the junction between the roof and the ring beam had not been grouted over a 200 mm length and had been misplaced by about 20 mm in position.
- bad detailing at the roof's junction with the ring beam aided the access of rainwater to the unprotected tendons. The decomposition of the water led to hydrogen embrittlement of the high strength steel.
- chlorides in the grout and zinc paint on the tendons hastened the above effects.
- the nature of the structure was such that even regular inspection of the hall would probably not have led to a prediction of the impending collapse.

Prime Source	Evaluators	Estimated Reliability of Evaluation	Effect on Profession
Formal Reports (e.g. Royal Commission)	Engineers & Lawyers	very high	high
"In-house" Reports not published widely) (e.g. for insurance purposes)	Engineers	high	medium
Newspaper reports	Non-engineers	unreliable	very low
Individual observation (Formally reported)	Engineer/ non-engineer	medium	medium
Individual observation (Not formally reported)	Engineers	medium	sporadic uneven
Formalized Data Banks (e.g. EPIC program)	Engineers (with non-engineers	medium-high	very low as yet, potentially high

Table 1 Evaluation of Negative Past Experience

Type of Failure	%	%	%
Collapse	25		20
(Ultimate)		63	
Loss of Safety (distress)	35		40
Loss of Serviceability	40*	37	40
References	[14]	[9]	[11]**

* considered to be under-represented
** concrete structures only

Table 2 Type of Failure

Failure Mode	%	%	%
Excessive deformation	26	52	30
Cracking	50	38	60
Local Damage (Clearance, gap holes)	7	5	5
Oscillation & Vibration	15	-	-
Water Penetration, Deterioration	2	5	5
References	[14]	[11]+	[9]*

+ concrete structures only (small sample)
* approximate values

Table 3 Serviceability Failure Modes

Failure Mode	Sudden Collapse %	Ductile Collapse %	 %
Loss of equilibrium	18	10	-
Instability	1	30	20
Rupture & Mechanism	42	15	-
Rupture (no collapse)	15	45	80
Other	24	-	-
References	[9]	[14]	[14]

Table 4 Ultimate Failure Modes

Failure Mode	Ultimate %	Serviceability %
Isolated Failure	42	53
Progressive Collapse	40	40
Load Shedding	18	7
References	[14]	[14]

Table 5 Isolated and Progressive Failure Modes

<table>
<tr><th>Types of Element</th><th>%</th><th>%</th><th>%</th></tr>
<tr><td>Foundations</td><td>16</td><td>-</td><td>7</td></tr>
<tr><td>Columns</td><td>7</td><td rowspan="2">3</td><td rowspan="2">14</td></tr>
<tr><td>Slender Strut</td><td><1</td></tr>
<tr><td>Wall</td><td>16</td><td>10</td><td>17</td></tr>
<tr><td>Bracing</td><td>1</td><td>-</td><td>-</td></tr>
<tr><td>Other</td><td>4</td><td>-</td><td>-</td></tr>
<tr><td>Beams</td><td>4</td><td rowspan="2">8</td><td rowspan="2">12</td></tr>
<tr><td>Roof Beams</td><td>7</td></tr>
<tr><td>Trusses</td><td>6</td><td>-</td><td>-</td></tr>
<tr><td>Seating</td><td>5</td><td>-</td><td rowspan="2">15</td></tr>
<tr><td>Brackets</td><td>4</td><td><1</td></tr>
<tr><td>Flat Slabs</td><td>1</td><td rowspan="2">17</td><td rowspan="2">27</td></tr>
<tr><td>Other Slabs</td><td>8</td></tr>
<tr><td>Arches</td><td>2</td><td>6</td><td>-</td></tr>
<tr><td>Other</td><td>8</td><td>54++</td><td>6</td></tr>
<tr><td>References</td><td>[14]*</td><td>[9]</td><td>[11]**</td></tr>
</table>

* Buildings Only - (derived) ** Concrete Structures only
++ include combinations

Table 6 Types of Failed Structural Elements

Building Phase	%	%	%	%	%	%
Preparations Foundations			4	6	4	8
Substructure	31	16	16 (69)	3 (57)	1 (37)	7 (58)
Superstructure			48	39	28	37
During handover			2	9	3	6
Year 1-2		5	5	13	10	11
	32					
Year 3-5			0	4	11	4
After year 5			9	11	24	12
	37					
Year 1 $\rightarrow\infty$		79	16	11	17	12
During demolition or renovation	-		1	3	1	3
References	[14]*	[8]**	[9]**	[9]+	[9]¶	[9]

* Buildings only

** Bridges only

+ Office Buildings only

¶ Factories only

Table 7 Time of Failure

Cause of Failure	%
Inadequate or unsuitable temporary works or erection procedures	8
Inadequate design in permanent material	3
Unsuitable or defective permanent material or workmanship	15
Wind	3
Earthquake	8
Flood and foundation movement	49
Fatigue	3
Corrosion	1
Overload or accident	10

Table 8 Prime Causes (Bridges) [8]

PRIME CAUSES OF WHICH SAFETY AND SERVICEABILITY DESIGN FACTORS DO NOT RELATE (Gross errors which could be reduced by checking and supervision)	Weighted %
Grossly inadequate appreciation of loading conditions or real behaviour of structure	36
Grossly inadequate appreciation of loading conditions or real behaviour of connections	7
Grossly excessive reliance on construction accuracy	2
Serious mistakes in calculations or drawings	7
Grossly inadequate information in contract documents and instruction	4
Gross contravention of requirements of contract documents and instructions	9
Grossly inadequate execution of erection procedure	13
Gross, but unforeseeable, misuse, abuse and/or sabotage, natural catastrophe, deterioration	7
Others	5
Subtotal	90
PRIME CAUSES TO WHICH SAFETY AND SERVICEABILITY DESIGN FACTORS DO RELATE (Stochastic variations which, singly, should not lead to failure but of which a combination of two or more may form an unfavourable situation leading to failure)	
Unfavourable load variation or combination (foreseeable, relating to γ_{S1} γ_{S2})	0
Inaccuracies in design assumptions of support conditions hinges etc., neglect or environmental effects (relating to γ_{S3})	3
Deficiencies in materials (γ_{m1}-related)	1
Deficiencies in workmanship (γ_{m2}-related)	3
Unforeseen, but foreseeable deterioration	3
Others	0
Subtotal	10

Table 9 Prime Causes [14]

Type of Error	% in the 212 cases with engineer involved	% in the 261 cases with contractor involved
Insufficient knowledge	36	14
Unclear definitions of competencies, error in information path	1	3
Reliance on others	9	5
Choice of poor quality for economical reasons	1	2
Underestimation of influence	16	11
Neglect, error	13	4
Ignorance, thoughtlessness, negligence	14	54
Objectively unknown situations	7	3
Other reasons	3	4

Table 10 Causes [9]

Possibilities of discovery	
Discovery probable with additional checking in phase of:	
Planning	33
Construction	17
Occupation	5
Discovery impossible	13
Discovery probable without any additional checking	32

Table 11 Possiblity of Error Discovery [9]

Economic Consequences as Percentage of Original Cost	Collapse (%)	Loss of Safety or Serviceability (%)
0 - 10%	48	60
11 - 20%	2	8
21 - 30%	8	8
31 - 40%	2	-
41 - 50%	8	4
51 - 100%	8	13
101 - 150%	8	4
151 - 500%	12	3
501 - 1000%	4	-
Total	100	100

Table 12 Cost of Failure [14]