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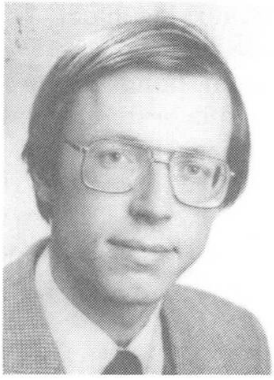
A "Best Practice" Approach to Structural Integrity Assessment

Une "meilleure procédure" pour l'évaluation de l'intégrité structurale

Ein "bestes Verfahren" zur Beurteilung von Gebäuden

John MAGUIRE
Civil/Structural Engineer
Lloyd's Register
Croydon, United Kingdom



John Maguire obtained his undergraduate and postgraduate degrees at Bristol University, UK. He has been involved with structural integrity assessment projects in the UK and overseas for nearly twenty years. Since 1989 he has been working for Lloyd's Register and currently heads their Civil & Structural Engineering Section.

SUMMARY

This paper briefly describes a number of UK government sponsored projects that have studied structural integrity assessment approaches across a wide range of industries. The aim of these projects has been to define and document "best practice" in complementary technology areas. One of these projects is described.

RÉSUMÉ

Ce document décrit brièvement un certain nombre de projets soutenus par le gouvernement britannique qui ont étudié les méthodes d'évaluation d'intégrité structurale à travers une gamme étendue d'industries. L'objet de ces projets a été de définir et de documenter 'la meilleure procédure' puisée dans des industries diverses. La méthode d'un projet est notamment décrite.

ZUSAMMENFASSUNG

Diese Arbeit gibt eine kurze Beschreibung von Projekten, die untersucht haben, wie in verschiedenen Industriezweigen Strukturintegritätsbeurteilung durchgeführt werden. Der Zweck dieser Projekte ist, die besten Verfahren zu beschreiben und zu dokumentieren. Ein Projekt wird mehr im Detail beschrieben.

1. INTRODUCTION

In the UK there are a variety of sector specific structural integrity assessment (SIA) approaches, for example for buildings and bridges, which are somewhat restricted in scope. Appendix 1 lists some of the major approaches.

In recent years a number of UK government sponsored projects such as the DTA (Dynamic Testing Agency), NAFEMS (National Agency for Finite Element Methods and Standards) and SAFESA (Safe Structural Analysis) have been studying SIA approaches across a wide range of industries (nuclear, defence, offshore, shipping, construction, automotive, aerospace, etc). The aim of these projects has been to define and document "best practice", drawn from a range of industries, in complementary technology areas. The DTA has addressed best practice in dynamic testing, NAFEMS best practice in finite element analysis (FEA) and SAFESA best practice in structural qualification, with particular reference to FEA.

The purpose of the paper is to briefly describe these three projects, and in particular to describe the DTA Primer and Handbooks, produced by the DTA, which provide a modern best practice approach to SIA for a variety of structures. The SIA approach is illustrated by application to a building example.

2. HISTORY AND STATUS OF THE PROJECTS

2.1 History

The chronological development of the three projects is shown in Figure 1. The aims of the three projects are given below.

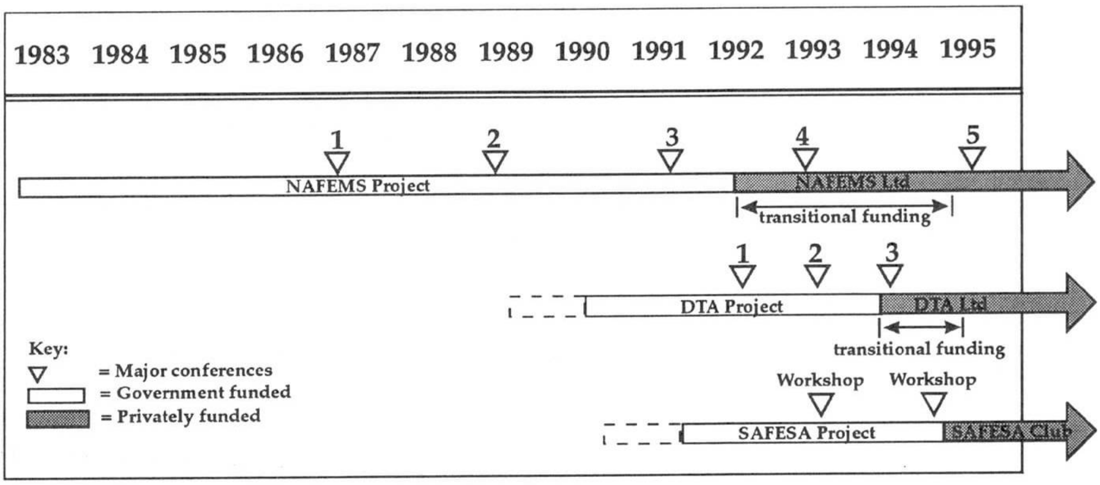


Fig. 1 Chronological development

2.2 NAFEMS Status

NAFEMS (now NAFEMS Ltd) was founded in 1983 as the UK National Agency for Finite Element Methods and Standards. Its aim is to promote the safe and reliable use of finite element analysis and allied technology. NAFEMS is a recognised "centre for quality" of international repute. Membership is open to all companies world-wide. NAFEMS activities include working groups, technology transfer, standards, information services and quality assurance. NAFEMS is owned and operated by the members for their mutual benefit, on a non-profit making basis. For further details see Appendices 2 & 3.

2.3 DTA Status

The Dynamic Testing Agency (DTA) was established in 1990 in order to draw together the many aspects of dynamic testing of structures. Its aim is to encourage best practice in all aspects of "standard" dynamic testing by providing documentation and guidance to supplement that already available. The DTA addresses the need for quality assurance standards in the analysis and measurement of dynamic data and provides practical and authoritative guidance on best practice. The DTA is demand led by industry and is non-bureaucratic. It is non-profit making and is controlled by and acts on behalf of its members. DTA activities include standards, co-ordination/promotion/support for technology, technology transfer, training and accreditation. For further details see Appendices 2 & 3.

2.4 SAFESA Status

SAFESA grew out of the NAFEMS Dynamics Working Group in 1990, leading to a project funded in its own right in 1991. It was organised as a collaborative R&D project, comprising 5 members:

- Assessment Services Limited (lead);
- Lloyd's Register;
- Cranfield University;
- Nuclear Electric plc;
- W S Atkins Science & Technology.

The principal aims were to enable structural qualification to be carried out reliably and accurately (with emphasis on the use of the finite element analysis method). The drivers for the project included:

- the trend to reduce physical testing by finite element analysis;
- the reduction of costs via reduced design cycle time;
- the improvement of quality/added value;
- the improved legal position.

For further details see Appendices 2& 3.

3. THE DTA SIA PRIMER AND HANDBOOKS

At the onset of the DTA programme it was recognized that dynamic testing is only one element in an overall structural integrity assessment process. Accordingly, the DTA commissioned the development of a suite of SIA documentation as follows:

- SIA Primer
- Handbook Volume 9
  - Item 90 - Overview of SIA
  - Item 91 - Detailed Methods for SIA
  - Item 92 - SIA Case Studies
  - Item 93 - SIA Management Procedures
  - Item 94 - Environmental Testing
  - Item 95 - Dynamic Loading
  - Item 96 - Partial Safety Factors

The SIA Primer gives an appreciation of the various facets (test, analysis and experience) to be regarded, and is of interest to project managers and technical managers alike. The SIA Handbooks explore technical subject matter in greater detail giving guidance on procedures, benchmarks, acceptability criteria, relevant codes of practice and databases.

4. EXAMPLE APPLICATION OF SIA PRIMER

4.1 Background

During 1991-93 a number of large panel system (LPS) tower blocks were assessed by Lloyd's Register (LR) for a client. For reasons of client confidentiality the tower blocks will be referred to as blocks A, B, C and D - typical construction is shown in Figure 2 and key details are given in Table 1. The assessment approach adopted by LR was that described in the DTA SIA Primer.

	Block "A"	Block "B"	Block "C"	Block "D"
Number of Stories	9	20	13	20
Height (m)	23.9	50.8	33.2	50.8
Length (m)	21.7	22.6	20.0	21.2
Width (m)	15.7	22.0	15.5	14.4
Underlying soil shear modulus (MN/m^2)	100	100	100	4400
Measured first mode (Hz)	2.72	1.11	1.65	1.11
Estimated mass (kg)	$3.25x10^6$	$9.94x10^6$	$4.39x10^6$	$5.08x10^6$
Estimated density (kg/m^3)	400	395	428	328

Table 1 - LPS Tower Block Key Data

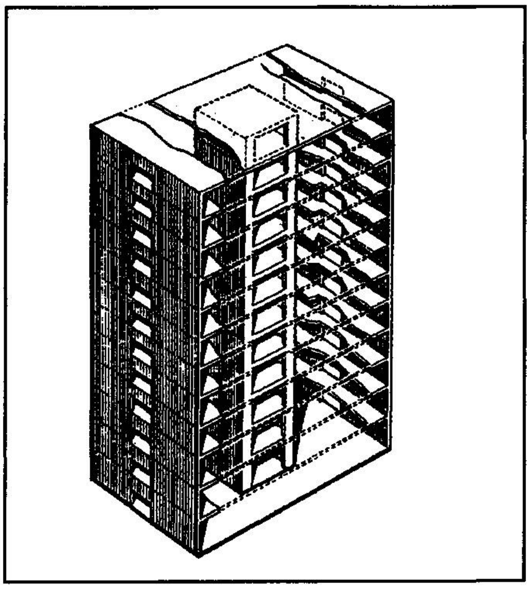


Fig. 2 Exploded Isometric View of Typical Tower Block (with acknowledgements to CIRIA)

4.2 General approach

The DTA SIA Primer assessment approach is summarised in Figure 3. It is a three stage approach, the precise nature of each stage being governed by the required acceptance criteria, level of criticality, etc. The DTA have also published a set of case studies (Handbook Volume 9 - Item 92) which illustrate the application of the SIA Primer to a number of industries. One of these case studies, known as "DTA Handbook Item 92.3", illustrates the approach applied to a tower block very similar to those studied in this paper.

4.3 Detailed approach

In view of the critical nature of the tower blocks assessed, a combined dynamic analysis, test, correlation and updating approach was adopted. The dynamic analysis was carried out via the finite element method. Dynamic testing (experimental modal analysis) was carried out using ambient (wind) vibration. Correlation and updating was carried out using both manual and computer-aided approaches. Stresses in the tower blocks were checked against acceptance criteria agreed in conjunction with the client. Full details cannot be given for client reasons.

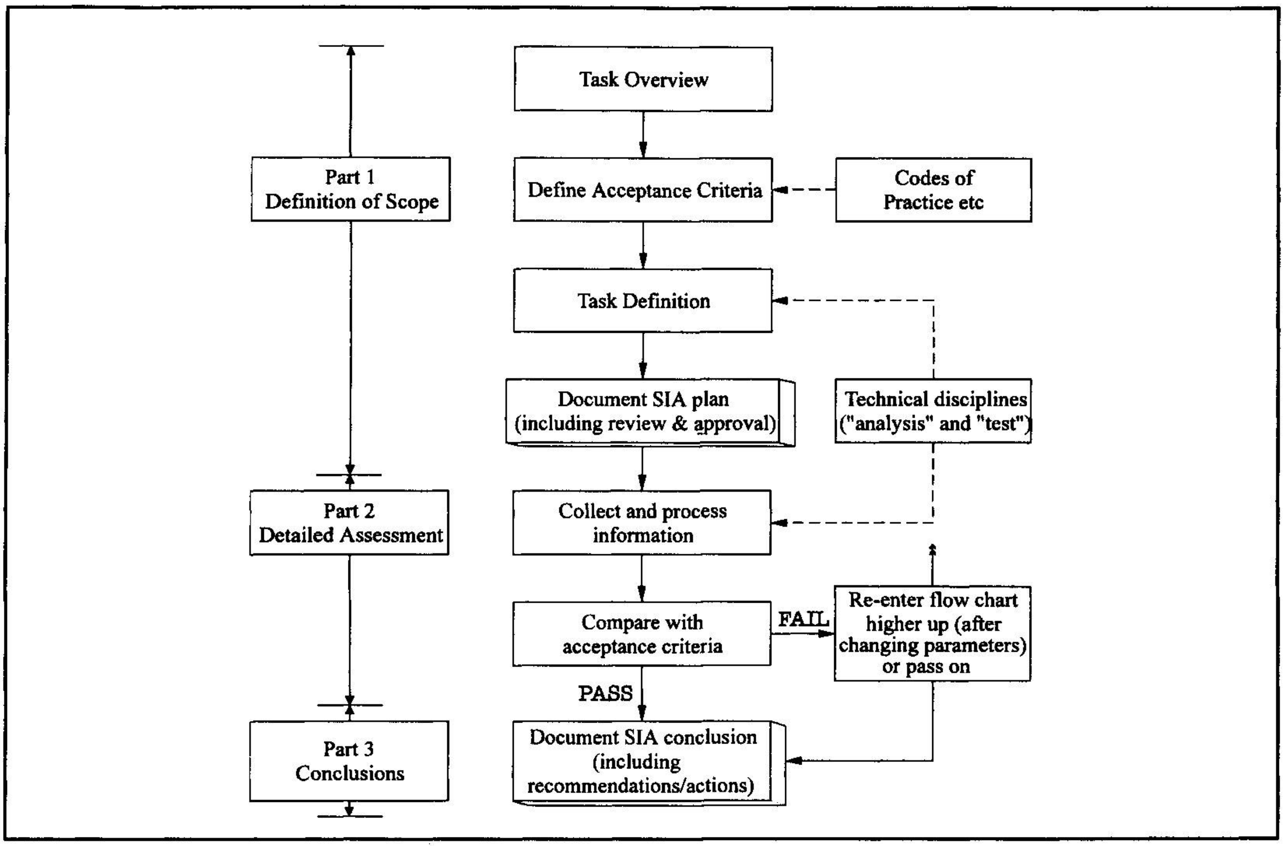


Fig. 3 DTA SIA Primer assessment approach

4.4 Assessment results

The four assessments led to the following conclusions:

a) the LPS blocks met the client acceptance criteria in that the measured and predicted stresses were within acceptable values;
b) the LPS blocks were significantly heavier than "normal" construction;
c) the LPS blocks were significantly stiffer than "normal" construction;
d) in view of (b) and (c) and also the significantly conservative original calculations most of the LPS blocks in fact have a significant margin of safety as regards structural integrity.

5. CONCLUSIONS

This paper has briefly described the DTA, NAFEMS, and SAFESA projects, in particular the DTA SIA Primer and associated handbooks, which provide a modern best practice approach to SIA for a variety of structures.

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APPENDIX 1 - LIST OF MAJOR UK ASSESSMENT APPROACHES (STANDARDS)

1. INSTITUTION OF STRUCTURAL ENGINEERS, Appraisal of Existing Structures, July 1980.
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9. INSTITUTION OF STRUCTURAL ENGINEERS, Appraisal of Sports Grounds, May 1991.
10. BRITISH STANDARD INSTITUTION, Guidance on methods for assessing the acceptability of flaws in fusion welding structures, BSI PD 6493:1991.
11. HMSO, Offshore Installations: Guidance on Design Construction, UK Dept. of Energy, 4th edition, 1990.

APPENDIX 2 - LIST OF PRINCIPAL PUBLICATIONS (AT DEC. 1994)

<u>NAFEMS</u> Details from:	Benchmark Magazine (quarterly), Major Reports, Summary Reports, Guidance Documents, Books, Conference Proceedings (5) NAFEMS Publications Limited, NEL Technology Park, East Kilbride, Glasgow, UK, Tel: +44 (0)3552 72639 - Fax: +44 (0)3552 72749
<u>DTA</u> Details from:	Primers (5), Handbook Volumes (9), Conference Proceedings (3) Dynamic Testing Agency, College of Aeronautics, Cranfield University, Cranfield, Bedford, MK43 0AL, UK Tel: +44 (0)234 751037 - Fax: +44 (0)234 750878
<u>SAFESA</u> Details from:	(draft) Quality Standard, Approach, Best Practice Manual, Examples NAFEMS Publications Limited

APPENDIX 3 - MEMBERSHIP PROFILE

<u>NAFEMS</u>	300 members in 27 different countries, 60% in UK, 20% elsewhere in Europe, 20% outside Europe, members mainly industrial companies and consultants (80%) but also academics (13%) and software vendors (7%)
<u>DTA</u>	40 members in 8 different countries, 80% in UK, 15% elsewhere in Europe, 5% outside Europe, members mainly industrial companies (85%) but also academics/individuals (15%)
<u>SAFESA</u>	5 members (all UK), 4 industrial, 1 academic

Repair and/or Strengthening in Function of the Diagnosis on the Structural Failure

Réparation et/ou renforcement de structures en fonction de la prévision de ruine

Reparatur und/oder Verstärkung von Stahlbetonbauwerken aufgrund der Ursache der strukturellen Mängel.

Leon A. GRILL
Consulting Engineer
Sydney, Australia

Born in Vienna, Austria. Graduated cum Laude 1948 Politechnicum School of Bucharest. Experience in the design of civil and structural projects in Uruguay, Argentina and Brazil. 1971-1986 Head of the Structural and Civil checking section of Rankine & Hill in Australia. Since 1986 independent consultant.

SUMMARY

Five case studies of structural failures are presented where the successful result of the repairs, in terms of a satisfactory behaviour and an acceptable performance, was based on the correct assessment of the cause of the distress, and on the appropriate decision with regards to relating the kind of repair or structural modification to the actual cause of the structural failure.

RÉSUMÉ

Cinq cas de ruines de structure montrent que le succès des réparations, afin d'obtenir un comportement satisfaisant et un fonctionnement acceptable, est fondé sur une évaluation correcte de la cause du dommage et sur une bonne décision quant au rapport entre le genre de réparation ou modification structurale, et la cause des désordres structuraux.

ZUSAMMENFASSUNG

Fünf Fallstudien von strukturellen Schwächen werden diskutiert, bei denen das erfolgreiche Resultat der Reparatur - zufriedenstellendes Verhalten und akzeptable Brauchbarkeit - auf der Voraussetzung basiert, dass die Ursache korrekt erkannt werden kann und dass eine adäquate Entscheidung gefällt werden kann bezüglich der Art des Reparatur oder der strukturellen Modifikation aufgrund der Ursache der strukturellen Mängel.

1. INTRODUCTION

In our work as professional engineers, we are not always in the enviable position of conceiving and designing entirely new structures. For this latter task we are all able to apply our academic training and we can also make use of computers as tools and structural Codes as general guides.

The problems presented by structural failures are different. In order to diagnose correctly, to be able to identify the cause or causes of structural distress and to propose adequate repairs, (or most often substantial modifications when the causes of failure are conceptual), we can only resort to experience and informed engineering judgement.

2. THE CASE STUDIES

2.1 Stair-Treads Failure

2.1.1 Description of the Structure and Mode of Cracking

In a high-rise building a sizeable number of the precast stair-treads (Fig. 2.1.1) developed flexural cracks in the underside even before the completion of the building. The cracks appeared near mid span: one single major crack on some treads, or two closely spaced cracks on other treads. The width of most cracks exceeded the acceptable upper limit of 0.3mm.

The investigation concluded that no abnormal loading had been applied to the staircase.

2.1.2 Causes of Cracking: The Probable or Real Cause as Opposed to the Assumed One

At first glance it appeared that there was an underestimation in the calculations of the service loads, and by increasing the reinforcement cross-section we could cater for a greater live load or increase the impact resistance.

However it was found that either for simplicity or for practical reasons, the cross-section of the reinforcement was in fact in excess of that required by analysis, while the compressive stress of the concrete as checked by calculations was within acceptable limits.

The sizing and detailing of the treads based on the theory of elasticity and limit state design seemed to be perfect, but unfortunately, only on paper.

We also discarded another possible cause of the failure being as a result of a small misplacement of the welded fabric within acceptable tolerances. In our opinion there are two other aspects which deserve special consideration:

i) The dimensions of the precast treads are such that the reinforcement provided is near or at the neutral axis of the section and is therefore unutilized when the first loading occurs.

   At this stage the treads will behave as precast mass concrete units, where the concrete alone has to withstand the tensile stresses in the underside. As soon as the modulus of rupture is exceeded, cracks appear. In our opinion the assumption that the initial cracks will slowly propagate under repeated loading till they reach the reinforcement, and from then on, the section will behave as a normal cracked section in bending (as shown on calculations), is nothing else but wishful thinking.

It seems that once small cracks appear under the first passing load, they develop with great speed under subsequent loads, causing a sudden excessive permanent deflection or even a complete "V" shape collapse of the tread associated with crushing of the concrete in compression.

<u>It becomes an energy absorption capacity problem rather than one of elastic design</u>.

ii) The mass of the slab is small compared to that of a person. This increases the effect of any sudden load application. It would appear unwise to apply an impact factor appropriate to a much higher dead load to applied load ratio.

2.1.3 Remedial Measures

The existing failed treads were discarded. Being confident of the validity of the diagnosis described in i) and ii) the new precast treads were increased in depth to 60mm in order to improve, even if very little, the ratio between the concrete mass and the live load mass, and also to permit a more rational location of the reinforcement. (Fig. 2.1.3)

Also, in order to have the reinforcement as far as possible from the neutral axis, the mesh was located at only 10mm from the bottom of the precast plank, disregarding the Code recommendation for cover. Being in sheltered conditions we considered that the risk of corrosion was negligible.

The cross section area of the reinforcement was <u>not</u> increased. After more than seven years in service, the new treads, continue to perform satisfactorily.

2.2 <u>The Failure of a Control Joint</u>

2.2.1 Description of the Structure and the Problem

The structure is an extended flat plate for a Shopping Centre.

In view of the size and the irregular plan shape of the structure, a control joint had been wisely introduced in order to minimise the adverse effects of shrinkage and temperature.

But unfortunately all other basic rules for the proper detailing of a control joint were ignored.

Figures 2.2.1 and 2.2.1 (a), show how the joint was originally detailed and the extent of the subsequent damage.

It can be appreciated that the insufficient width of seating, the erroneous detailing of the reinforcement and the total lack of any kind of sliding strip to reduce friction, led inevitably to the result shown on Figure 2.2.1 (a).

2.2.2 Remedial Works

Since to improve the detailing of the reinforcement would have required too much cutting back of the concrete slab, attention was turned to two other aspects: sufficient width of slab seating and much better sliding capability.

As shown on Fig. 2.2.2 the new support was an angle 152 x 152 x 12 with vertical legs of the same size welded to it at 1.0m centres; all, hot dip galvanized, including masonry anchors. This new support was wide enough to allow a good end development length of the slab bottom bars, making unnecessary to worry about improving the detailing of the existing reinforcement.

It was decided to add the vertical legs for two reasons:

i) It was considered unwise for the new support to be bolted to the beam only through the previously damaged area. The additional bolts through the vertical angles were clearly anchored in a perfectly healthy area of the reinforced concrete beam.

ii) With only the horizontal angle providing support, the bolts would be subjected (in addition to shear), to a pull-out action due to the eccentricity of the load. With the vertical legs fixed with two bolts each, this effect should be substantially reduced.

All bolts were long enough to reach the core of the support beam. Before fixing the support to the reinforced concrete beam, the spalled area was duly cleaned of dust and debris and made good with a suitable epoxy.

To ensure adequate stiffness of the supporting flange of the angle, 10mm plate fins were welded at regular centres. To facilitate handling and fixing, the new support was made up in units 2m in length.

A stainless steel on teflon sliding strip was installed between the angle and the slab for the full length of the joint.

From the first inspection of the deteriorated area, propping was recommended in order to prevent more severe damage or even possible total failure, while devising the repair system.

A line of propping to the slab was located close to and parallel to the joint, but at sufficient distance to allow repairs to be carried out.

After more than three years in service the solution appears to be safe, and at the same time permits the relative movement of the two sides of the structure.

2.3 The Column Failure Case

2.3.1 Description of the Structure and the Problem

The structure was a single storey institutional building with an extensive waffle slab.

A control joint had been introduced in a suitable location to reduce shrinkage and temperature effects as well as to prevent a stress concentration where, in plan, the slab has a sudden change in shape.

At one end of the control joint, both sides of the structure were supports by the same massive column, 2500 x 450 in section. (See Fig. 2.3.1)

The problem was that an approximately vertical crack developed in the column. It started at the top of the column in the vicinity of the slab control joint, and extended down for about 2m.

We were commissioned to assess the cause and seriousness of the fault and to devise repairs if considered necessary.

2.3.2 Remedial Works

In this case again the structure was clearly indicating the cause of distress.

The design was inherently faulty: the initially correct introduction of a Control Joint was not carried through to its logical conclusion. It is true that the slabs were not poured monolithically with the column, but no sliding pads were used to allow for movement.

The substantial vertical load combined with the high friction co-efficient concrete to concrete produced an unacceptable restraint to shrinkage.

To make things worse, starter bars connected the column to both slabs, contradicting still more the concept of a control joint.

It was recommended the demolition of part of the column back to the line of the slab joint.

The edge beam supported by the column section to be demolished was propped by two UC's adequate to support the load.

The demolished section was rebuilt as an independent column, with a well formed vertical joint in the plan of the existing slab control joint, down to the top of the footing. (See Fig. 2.3.2.)

Each separate column was checked for the load of the corresponding slab area and results were satisfactory.

Before forming up the new part-column the rough face of the undemolished side of the column was made good and a 10mm polystyrene strip was applied to it to form a permanent joint.

All that had to be done was to provide a well-formed joint where the structure tried to forcibly create one by cracking the column as a protest against the contempt and misunderstanding shown in the design for the natural behaviour of the structure.

3. CONCLUSIONS

The first case illustrates that even apparently very simple structures should not be hastily approached with routine design and uncritical compliance with the code.

Mathematical analysis and computers are of no help either for appraising the causes of failure or for providing the solution for repairs.

The last two cases show that bad detailing, lack of consistency in pursuing a design concept and/or a lack of understanding of structural behaviour inevitably lead to failure.

All cases demonstrate that repairing most often does not mean only patching or strengthening a faulty structure.

We have to resort to experience and informed engineering judgement to find out the real cause of the failure and to devise not merely repairs but total modifications to the structure in order to correct built-in errors of concept.

I should also emphasise that although this field is a somewhat restricted aspect of structural and civil engineering, it is a stimulating and challenging one because there are not all those standards and regulations which have so often impaired clear thinking and reduced many of our engineer colleagues to "book keepers of reinforcement" as somebody once put it.

4. REFERENCES

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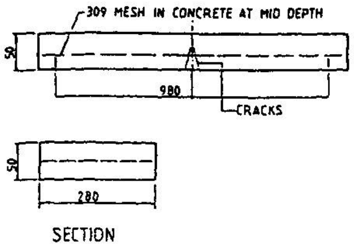

FIGURE 2·1·1

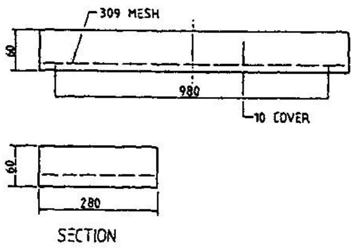

FIGURE 2·1·3

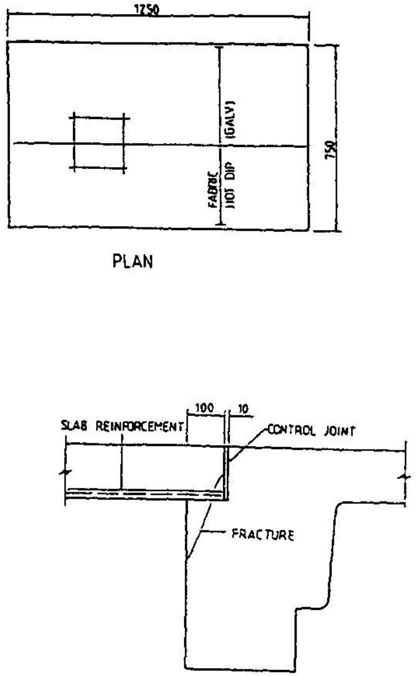

FIGURE 2.2.1

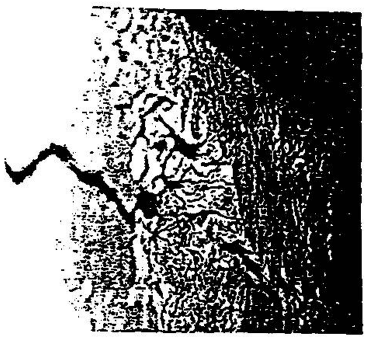

FIGURE 2.2.1(a)

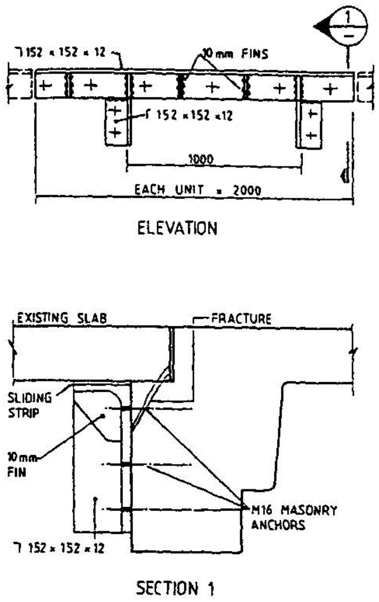

FIGURE 2.2.2

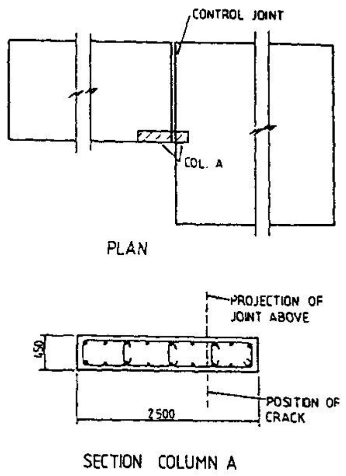

FIGURE 2.3.1

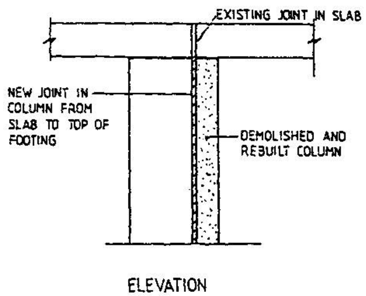

FIGURE 2.3.2

Global and Local Approaches in Structural Identification

Méthodes globales et locales d'identification structurale

Globale und lokale Methoden für die Gebäudeaufnahme

Andres W.C. ORETA
Associate Professor
De La Salle University
Manila, Philippines

Tada-aki TANABE
Professor
Nagoya University
Nagoya, Japan

SUMMARY

Two approaches in structural identification using the Kalman filter algorithm for the purpose of damage detection and evaluation of the condition of structures are presented. These two approaches, which are the global and local identification, are first described and then their application to concrete structures is investigated. Vibration data for this study are obtained from shaking table tests conducted on a concrete space frame.

RÉSUMÉ

L'article présente deux méthodes d'identification structurale à l'aide de l'algorithme du filtre de Kalman ayant pour but la détection des dommages et l'évaluation de la condition des structures. Les deux méthodes d'identification globale et locale sont décrits, et leurs applications aux structures en béton sont étudiées. Les données sur les vibrations sont obtenues par des essais de cadres en béton sur des tables vibrantes.

ZUSAMMENFASSUNG

Vorgestellt werden zwei Methoden für die Gebäudeaufnahme, die den Kalmans Filteralgorithmus für die Schadenermittlung und Festsetzung des Bauzustandes verwenden. Diese zwei Methoden, die globale und die lokale Identifizierung, werden zuerst beschrieben. Anschliessend wird die Anwendung auf Betonkonstruktionen gezeigt. Zur Bestimmung von Schwingungsdaten wurden für diese Untersuchung Versuche an einem Betonrahmen auf einem Rütteltisch durchgeführt.

1. INTRODUCTION

Civil Engineering structures, after many years of service under severe environmental conditions, are damaged and their strength deteriorate. This is especially true for concrete structures which deteriorate due to various factors such as creep, shrinkage, steel corrosion, spalling of concrete, microcracks, etc. Since the primary concern of the civil engineer is to assure the safety of these damaged structures, the civil engineer must be concerned with the problem of evaluating the existing condition of structures. This is important in connection with damage assessment and rehabilitation of structures.

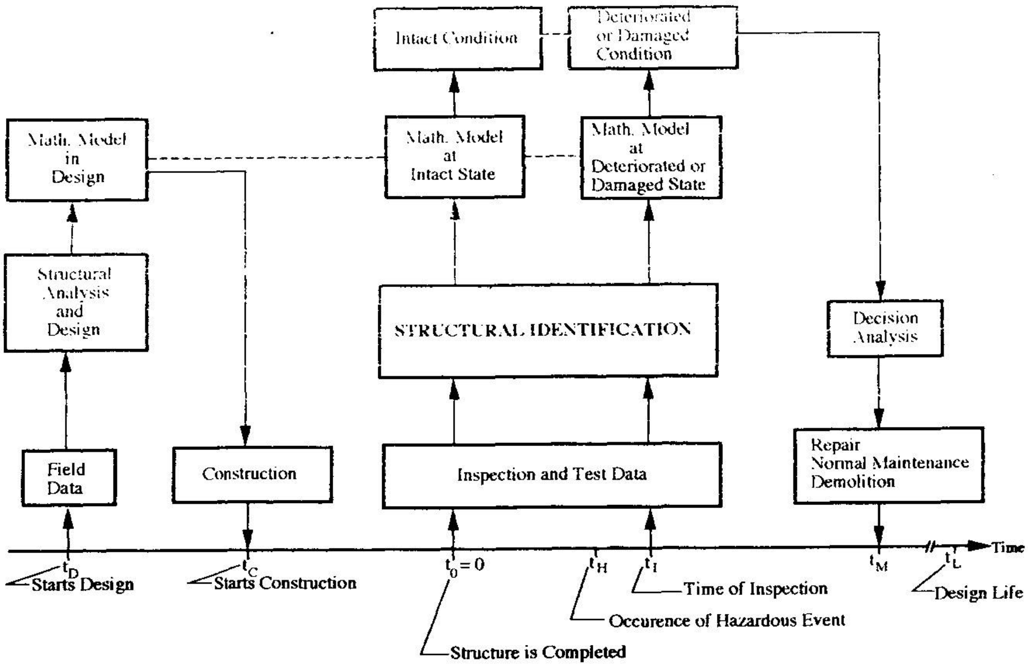

Fig. 1 Role of structural identification during the lifetime of a structure

Recently, with the development of new and sophisticated response measuring instruments and increase in computational capabilities, structural identification has started to be used as a means of evaluating the existing condition of structures. The role of structural identification during the lifetime of a structure can be understood clearly from Fig. 1. In designing structures, an integrated process is followed wherein field data, standard building codes and specifications, structural analysis and engineering experience are used to obtain the design of the structure with specified configuration, dimensions and material properties. The completed design of the structure, however, are based on an assumed mathematical model, the correctness of which have still to be verified. After completion of the structure at t_0 and thereafter, structural identification can play an important role. At the completion of the structure, field testing and inspection can be conducted and the test data can be used in structural identification to perform the following objectives: (1) to verify the validity of the assumed mathematical model used in the design; and (2) to estimate the actual structural properties of the constructed structure and thus the mathematical model can be updated or improved. Results from this identification phase correspond to the condition of the structure at the intact state and can be used as a benchmark for comparison with the results obtained from future condition evaluation investigations. During its service life, especially when the structure is damaged due to an hazardous event such as a strong-motion earthquake, structural identification can again be carried-out to attain the following objectives: (1) to verify the validity of the existing mathematical model; (2) to identify changes in the structural properties of physical parameters governing the response of subassemblies, elements, connections and complete structure; and (3) to update or improve the model for a more realistic representation of the structure. Results from this phase of the investigation can be used in evaluating the soundness of the structure, observing the aging or deterioration of the structure and assessing the extent of damage caused by the hazardous event.

2. THE STRUCTURAL IDENTIFICATION PROBLEM

Structural identification consists of system identification techniques in which mathematical models for a structural system can be found by the use of a set of known inputs and corresponding outputs. Solution of a structural identification problem requires a state vector, X(t), which is governed by a mathematical model in the form of a differential equation

$$\frac{dX}{dt} = f[X(t), u(t), w(t), \theta(t), t] \qquad (1)$$

in which u(t) = input signal, w(t) = system noise and $\Theta(t)$ = unknown parameter vector. A noise corrupted version of the system state vector, Y(t) is observed and is related to the state vector by

$$Y(t) = h[X(t), u(t), w(t), \theta(t), t] \qquad (2)$$

where v(t) = observation noise. In structural dynamics, the order and form of the state equation is known and is derived from the equation of motion of an assumed mathematical model which can be a lumped mass or a finite element model. Hence, the identification problem reduces to estimation of structural parameters in the mathematical model. In this study, the extended Kalman filter with weighted global iteration (EK-WGI) developed by Hoshiya and Saito[1] was used in the identification.

3. GLOBAL AND LOCAL IDENTIFICATION

Based on the size of the structural system under consideration, structural identification can be classified into (1) global; and (2) local. Global identification models the complete structure in the deriving the state equation. The representation usually reduces to simple models such as lumped mass models especially when structures with many degrees-of-freedom (DOFs) are analyzed. Fig. 2(b) is which shows a spring-mass-dashpot model of a plane frame is an example of a global model. The simple modeling is necessary for an efficient and convergent identification. The models used by Hoshiya and Saito [1] are typical examples of global models in structural dynamics. The unknown parameters involved in global identification are those which affect the overall behavior of the complete structure such as frequency and damping or stiffness, **k**, and damping, **c**. In damage assessment of structures, global identification can show only that damage has occurred from the changes in the overall response and characteristics of the complete structure. It is difficult, however, to detect the cause of these changes in the behavior of the structure.

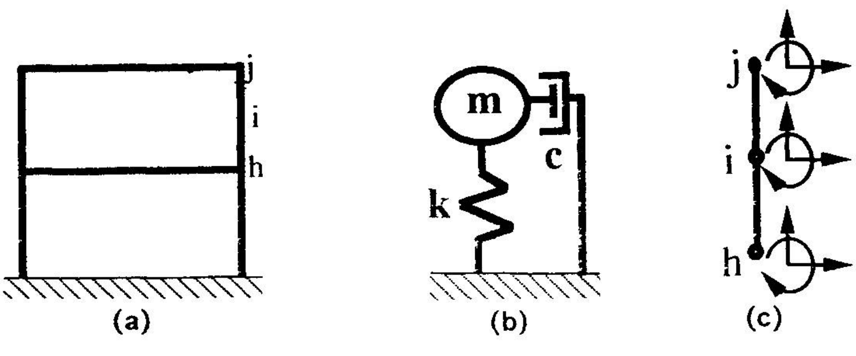

Fig. 2. (a) Plane frame; (b) Global model of plane frame; (c) Local model of frame member

Local identification is introduced as another approach in structural identification. In local identification, only a specific portion of the structure is modeled by substructuring methods. Unlike in global identification, more refined models such as finite element models can be used since the system under consideration is relatively small. Fig. 2(c) shows a model of a plane frame member with three DOFs per node. The unknown parameters involved in local identification are those which represent the structural properties of a substructure composed of an individual member or a system of members. By this approach, it is possible to detect the location and extent of damage in the structure through changes in the parameters of local portions of the structure such as an individual member. Oreta and Tanabe [2] developed a local identification method for framed structures such as plane frames and showed numerical studies to illustrate the identification of the stiffness properties of individual frame members.

As a practical and efficient identification method for damage detection and evaluation of structures, the two approaches can be adopted concurrently: (1) a simple model such as a lumped mass model can be used to represent the complete structure to detect the changes in the overall response; and (2) a refined model can be used to analyze a local and critical area to assess the degree of damage. These two approaches are adopted in the identification of a reinforced concrete space frame which was conducted by the authors.

4. EXPERIMENTAL INVESTIGATION

4.1 Description of Experiment

The test specimen is a reinforced concrete rigid space frame with a weight of 945 kg and height of 110 cm. The top slab is supported by four columns with 7.0 cm square cross-sections. At the midheight of the columns are stiffening beams with I-shape cross-sections. An additional steel mass weighing 935 kg was symmetrically installed at the top slab. The specimen was rigidly bolted on top of the shaking table. Fig. 3 shows you the plan and elevation of the specimen. The four columns are labeled as A, B, C and D.

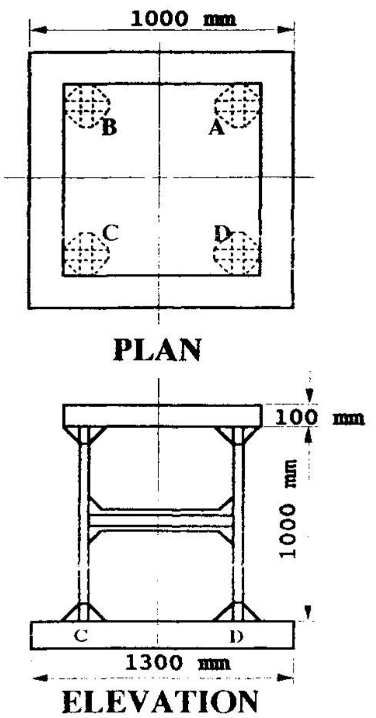

Fig. 3. Test specimen

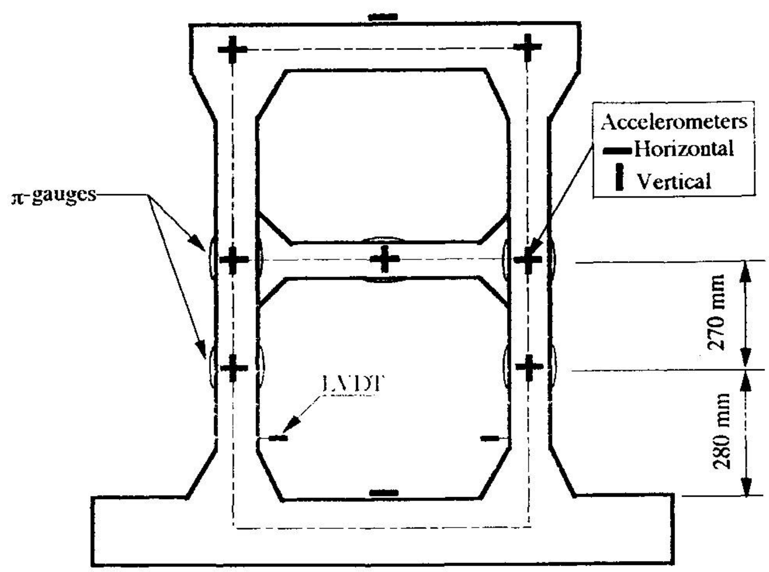

Fig. 4. Location of sensors

To systematically monitor the response of the structure, critical points of the members of the specimen were selected. At these points were installed different measuring devices or sensors. Strain-gauge type accelerometers were installed at the top and bottom slabs to measure the horizontal motion. At specified nodes of the column and beam members, vertical and horizontal accelerometers were fixed. Linear-variable displacement transducers (LVDT) were installed at the bottom ends of the columns. 100 pi-gauges (p-gauges) were fixed at opposite sides of the columns and beams at specified nodes. The locations of these sensors are shown in Fig. 4.

Free vibration tests were performed by hitting the top slab of the specimen by a hammer. This test was performed at the beginning before any shaking table test and after every one or two shaking table tests. The purpose of the free vibration test is to obtain acceleration data at the top slab for global identification of the dynamic properties of the complete structure.

Shaking table tests using a sinusoidal function as input excitation were performed. In the beginning, a forcing frequency of 7.0 Hz and an amplitude of 50 gal were used. To introduce gradual damage to the structure, the amplitude of the sine function was increased to 75, 100, 200, 300, 400 and 600 gal. The vibration data obtained from the different sensors located at the critical points of the specimen were used in the local identification of the stiffness properties of the frame members.

4.2 Global Identification

From the free vibration tests, the measured horizontal acceleration data of the top slab were used as observed data in the system identification by EK-WGI. The dynamic properties such as damping ratio and frequency of the structure which was modeled as a single-degree-of-freedom system were obtained at different stages. Fig. 5 presents the free vibration response of the top slab at the initial and failure stages. The decrease of the frequency from the initial condition to the failure condition is clearly demonstrated graphically and numerically as shown by the results from the identification in Table 1. Initially, the specimen had a frequency of about 6.7 Hz. At the early stages of shaking, the stiffening beams at midheight were damaged. With more intense shaking, cracks started to form in the columns and concrete cover started to spall. As a result, the stiffness of the complete structure decreased. At the failure condition, the frequency of the specimen was about 4.0 Hz which corresponds to more than 41% reduction with respect to the initial frequency.

Table 1. Results of Global Identification

	Damping (%)	Frequency (Hz)
Initial Condition	2.568	6.718
After 200 gal	2.565	6.231
After 400 gal	2.369	5.722
After 600 gal	2.796	5.083
Failure Condition	2.524	3.968

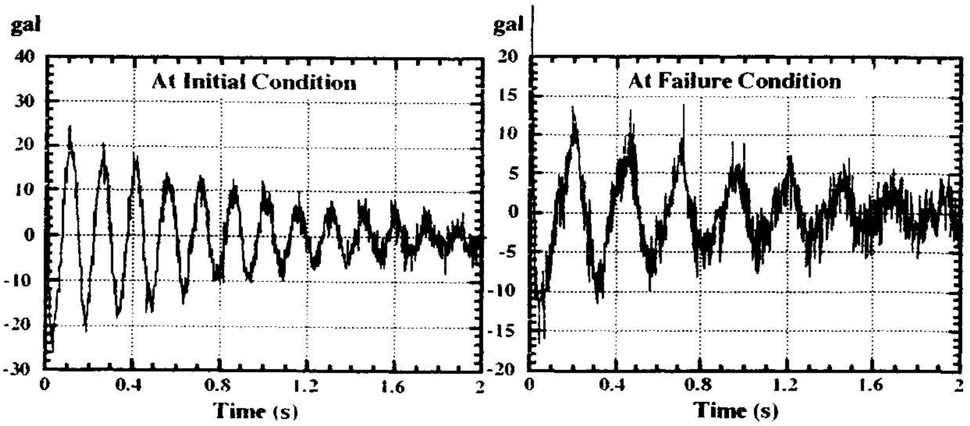

Fig. 5. Free Vibration Response

4.3 Local Identification

To identify the changes in the stiffness properties of the members of the specimen, the element identification method developed in Oreta and Tanabe [2] was adopted. In this method, the differential equation of motion was derived for each column by isolating the member from the structure. The responses at both ends and at the center were observed and used in the identification of the stiffness parameters of the columns. Four stages of the structure are considered. Initially, the response from a low amplitude shaking test of 20 gal was used to identify the stiffness properties at the initial condition. Then the vibration data from shaking table tests of amplitudes of 200 gal, 400 gal and 600 gal were used in estimating the effect of the damage in the stiffness properties of the columns. The results of the local identification of the flexural rigidities (EI), of the columns are shown in Table 2.

Table 2. Results of Local Identification of EI (x 10 ^10 N-cm^2)

	Column A	Column B	Column C	Column D
Initial:	7.901	8.500	8.432	8.000
At 200 gal	7.510	7.465	5.347	7.863
At 400 gal	7.500	7.235	5.166	7.669
At 600 gal	7.806	6.790	4.512	5.880

Tables 2 shows the decreasing value of the stiffness properties of the columns of the structure. Except for column A, the decreasing trend of the flexural rigidities of the columns was observed as follows: (1) Column B: 12.17% at 200 gal, 14.88% at 400 gal, and 20.11% at 600 gal; (2) Column C: 36.59% at 200 gal, 38.73% at 400 gal, and 46.99% at 600 gal; and (3) Column D: 1.71% at 200 gal, 4.13% at 400 gal and 26.5% at 600 gal. It is noted that column C has the largest flexural stiffness degradation followed by column D. This was also observed in the experiment where columns C and D were the worst damaged. The failure of these columns was characterized by spalling of concrete and vertical cracks due to bond failure.

5. CONCLUSION

The results of the study showed that the deterioration of the structure can be observed by system identification. Global identification showed the effect of damage to the dynamic properties of the structure. Frequency decreases with accumulation of damage. Local identification, on the other hand, illustrated the apparent reduction of the stiffness of the columns and indicated the columns which were severely damaged. In general, the identified structural parameters showed the deteriorating trend of the stiffness of the structure. This study must be extended to nonlinear behavior of structures since this behavior is what is usually experienced in severely damaged structures.

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Expert System Application to Buildings Maintenance
Application de systèmes experts à la maintenance des bâtiments
Anwendung eines Expertensystems auf den Gebäudeunterhalt

Youssef Georges DIAB
Maître de Conférences
Université de Savoie
Le Bourget du Lac, France

G. Diab, né en 1964, obtient son diplôme d'ingénieur civil à l'ENTPE, Lyon. Pendant 4 ans il est directeur de la recherche et du développement dans un bureau d'études. Depuis 2 ans il est Maître de Conférences. Il est titulaire d'un doctorat en génie civil.

SUMMARY

Repairing damaged structures has become more expensive in the last years. In many cases, the engineers' experience was not sufficient to decide on the way to repair, and if repair is necessary. There are no exact scientific models for concrete structures repair. The knowledge is dispersed in different guidelines, regulations, books and procedures. The purpose of this work is to develop an expert system for diagnosing buildings pathologies caused by condensation and other physical phenomena and pathologies caused by mechanical defects.

RÉSUMÉ

La réparation des bâtiments est devenue chère, et l'expérience des ingénieurs n'est pas toujours suffisante pour choisir les méthodes de renforcement adéquates aux problèmes rencontrés. Il n'existe pas à l'heure actuelle un modèle scientifique fiable pour le choix de la méthode de réhabilitation car les informations sont dispersées. Il est donc nécessaire de mettre au point une approche pragmatique et scientifique. L'objectif de ce travail est de développer un système expert de diagnostic des bâtiments et de mettre au point une approche complémentaire de réhabilitation.

ZUSAMMENFASSUNG

Die Sanierung von Gebäuden ist eine teure Angelegenheit, und nicht immer reicht die Erfahrung der Ingenieure aus, um zu den angetroffenen Problemen die angemessenen Reparaturmethoden zu wählen. Gegenwärtig existiert kein gültiges wissenschaftliches Modell für die Methodenwahl, denn die Informationen sind in verschiedenen Richtlinien, Vorschriften, Fachbüchern und Verfahrensanweisungen verstreut. Sie müssen in einem pragmatisch-wissenschaftlichen Ansatz aufbereitet werden. Gegenstand der Arbeit ist die Entwicklung eines Expertensystems zur Diagnose von Gebäudeschäden, verursacht durch Wasserdampfkondensation und andere physikalische Phänomene, und zum Einsatz entsprechender Sanierungsmassnahmen.

1. Introduction :

Les bâtiments vieillissent et les propriétés mécaniques de leurs matériaux constitutifs se dégradent, ce qui diminue leurs coefficients de sécurité prévus initialement par les codes de calcul et les réglementations. Ce problème est à l'heure actuelle majeur pour les bâtiments en maçonnerie de briques ou de pierres mais également pour les bâtiments en béton armé ayant un âge supérieur à une trentaine d'années.

Face aux désordres structurels de bâtiments et à la panoplie de solutions de réhabilitation, il est nécessaire d'établir une adéquation entre les uns et les autres afin de prendre les bonnes décisions de réparation et de faire progresser, à l'aide des résultats des calculs et du diagnostic, nos connaissances sur les phénomènes d'endommagement des différentes composantes, dans le but d'améliorer les projets futurs. Ainsi, pour l'analyse de ce problème, cet article s'articulera autour des thèmes majeurs suivants :

1) Nature et origine structurelle de désordres observés sur les bâtiments en béton,
2) Evaluation des coefficients de sécurité résiduels,
3) Inventaire des méthodes de réhabilitation actuelles,
4) Etablissement de règles liant les désordres observés au choix de la méthode de réhabilitation.

2. Nature et origine structurelle des désordres :

L'expérience a toujours été à la base des connaissances des constructeurs. L'examen approfondi des ouvrages existants, l'étude des échecs et des désordres sont donc indispensables pour progresser sans imprudence.Le relevé de dégradations suivi d'une analyse structurelle du bâtiment contribuent à prendre des décisions à caractère très immédiat et à apporter des éclaircissements sur l'origine des dégradations constatées et sur leur évolution future.

A titre d'exemple, contre les fissures de flexion et d'effort tranchant, reconnaissables par leurs positions par rapport à la fibre neutre de la poutre, les dispositions à prendre sont fonction de leurs ouvertures "e" et de leurs longueurs obtenues par un relevé (Mougin, 1986). Ainsi :

+ pour les fissures de flexion, si $e > 0.5$ mm un traitement approprié de la fissure est nécessaire avec suppression de la cause.

+ pour les fissures d'efforts tranchants, si $e < 0.3$ à 0.4 mm, aucune réparation n'est nécessaire.

2.1 Causes liées à la conception et au calcul :

1- Inexactitude ou insuffisance des hypothèses de calcul

* sur les actions et combinaisons d'actions
* sur les déformations imposées
* sur les valeurs des caractères mécaniques des matériaux

2. Inadéquation des modèles de calculs

* pour la détermination des sollicitations et de leurs combinaisons
* pour l'étude de la stabilité générale ou locale (équilibre statique ,contreventement, stabilité de forme...)
* pour la prise en compte des phénomènes différés
* pour la prise en compte des phénomènes thermiques (en particulier gradients)

3. Erreurs de conception, erreurs ou fautes de calcul

* insuffisance des calculs de dimensionnement et insuffisance de la vérification
* absence de dispositions prenant en compte les variations dimensionnelles
* dispositions constructives inadaptées à la transmission des efforts (zones d'efforts concentrées, poussées au vide, zone de reprise de bétonnage)
* acceptation de déformations excessives
* non respect des de l'art .

2.2 causes liées à l'exécution :

1. Insuffisance des documents d'exécution

* insuffisance de leur vérification par le maître d'oeuvre
* erreurs de côtes et absences d'indications de tolérances
* absence de dessins de détails dont dépend la faisabilité
* préparation insuffisante du phasage et des réglages d'exécution

2. Déficiences du contrôle de qualité

* qualité insuffisante ou défectueuse des matériaux, produits et composants, y compris dans les étaiements et ouvrages provisoires
* insuffisance ou absence de contrôle à la mise en oeuvre des aciers et des bétons
* non - respect des prescriptions technologiques (adjuvants, gel, eaux, décoffrages,..)

2.3 Causes liées aux conditions d'exploitation :
* relâchement de la surveillance
* exécution différée de travaux d'entretien
* effets de conditions atmosphériques (eau, gel ,...)
* accroissement de l'intensité et de l'agressivité du trafic (en particulier poids lourds)
* fréquence de passage de convois exceptionnels

2.4 Causes liées au vieillissement de l'ouvrage :
* séquelles d'incidents de construction
* vieillissement des matériaux
* vieillissement de la structure : effet de fatigue

Ces deux derniers phénomènes constituent les principales causes des désordres

2.5 Causes liées à des actions abusives :
* séquelles dues au passage d'engins lourds de chantier pendant les opérations de terrassement
* dépassement des charges autorisées (poids lourds en surcharge)

2.6 Occurrence d'actions naturelles ou extérieures :
* Mouvements d'appuis (affouillements des fondations, tassements différentiels, glissements de terrains)
* Ambiances agressives (embruns, eaux, fumées ,...)
* Incendie
* Séisme

3. Les coefficients de sécurité :

Le calcul des coefficients de sécurité permet de lever toute ambiguïté entre les fissurations "normales" et celles "hors règlements" : Cette étude des coefficients de sécurité résiduels vise à favoriser la réponse aux questions qui se posent aux maîtres d'ouvrages vis à vis des problèmes de fissuration et, au delà, vise à savoir s'il faut entamer des travaux de réparation ou pas.

La fissuration étant du domaine non élastique, l'utilisation des codes de calcul élasto-plastique est nécessaire lorsqu'une étude approfondie sera réalisée. Dans notre approche la variation de la rigidité est utilisée comme artifice de calcul pour prendre en compte la fissuration de la structure. La rigidité étant connue pour la structure saine, on la fait varier suivant l'état de la fissure dans la structure endommagée. Après avoir fixé une valeur caractéristique de contrainte dans les matériaux constitutifs, on détermine soit les contraintes dans les différents éléments de la structure soit les sollicitations. Le rapport des contraintes ou des sollicitations caractéristiques fixées au préalable sur les contraintes ou sollicitations correspondantes calculées sous la structure supposée saine puis endommagée permet d'obtenir le coefficient de sécurité dans l'élément considéré et, partant, dans toute la structure.

La méthode permet de fournir une première approche du degré de sécurité de la structure apparaissant à travers les fissures mais n'a aucun caractère réglementaire. On peut connaître la chute des résistances dans l'ouvrage endommagé par rapport à son état initial (sain), mais aucun seuil critique n'est fixé. Ce dernier est défini selon les pratiques et les exigences des maîtres d'ouvrages. De même, bien que proche de la réalité, la modélisation est une approche simpliste de la réalité mais ne reflète pas le comportement réel d'une structure en béton armé. Celui-ci doit être modélisé directement par l'intégration de la fissuration dans les calculs. Ainsi notre approche se présentera comme suit :

- Une étude de la structure saine supposée homogène, avec le chargement appliquée (déplacements d'appui compris);
- Une étude de la structure endommagée avec le même chargement extérieur.

Cependant, compte tenu du fait que les coefficients de sécurité n'ont aucun caractère réglementaire, il est à l'heure actuelle très souvent impossible de donner une appréciation quantitative des risques que peut faire encourir aux usagers une structure dégradée ou en cours de dégradation. Les appréciations ne peuvent alors être portées que par référence à d'autres ouvrages étudiés précédemment.

4. Observation des dégradations :

4.1 Chronologie d'apparition :

Les dégradations d'un ouvrage, suivant l'origine de leur cause, se manifestent à plus ou moins long terme. Ainsi :

- Lorsque les défauts sont dus à des erreurs de conception, de calcul (tel est le cas dans les nombreux désordres structurels observés) ou même de mise en oeuvre, les désordres apparaissent rapidement dans les mois qui suivent la mise en service.

- Les dégradations des ouvrages bien conçus et bien construits n'apparaissent qu'au bout de quelques décennies, lorsque la couche protectrice des aciers commence à se dégrader. D'où la notion de garantie décennale pour la structure et le gros oeuvre des bâtiments.
Cependant, il est à noter que les dégradations causées par les eaux agressives et les fissures dues au retrait se manifestent pour les premières dans les deux à trois ans qui suivent la mise en service et pour les secondes dès la mise en oeuvre du béton (retrait thermique) et également dès le premier hiver froid et sec (retrait hygrométrique).

4.2 Evolutivité des dégradations :

1- Au début : (1er stade)
* fissures fines * efflorescences * traces de rouille

2- Evolution : (2ème stade)
* multiplication et accroissement de la longueur des fissures * épaufrures dans les angles
* aciers corrodés apparents * gonflement du béton

3- Dégradations avancées : (3ème stade)
* éclatements entre les fissures * éclatements le long des aciers
* aciers corrodés et/ou coupés * altérations superficielles ou dans la masse de béton

4.3 A quel moment intervenir ?

Un ouvrage doit être protégé essentiellement contre les venues d'eau (imperméabilisation des faces exposées, protection des fondations, évacuation des caves, drainage, etc.). Il faut donc surveiller les ouvrages de façon à intervenir le plus rapidement possible et si possible au premier stade où un entretien suffit. Lorsqu'on intervient au deuxième stade des dégradations, l'entretien spécialisé est nécessaire. Au troisième stade, la vie de l'ouvrage peut être en cause et il est nécessaire de faire des réparations souvent longues et coûteuses.

5. Système expert pour le diagnostic et la réparation des désordres dans les structures en béton armé

5.1 Nécessité :

La réparation des endommagements de structures en béton est devenue plus coûteuse à l'heure actuelle et souvent, elle ne donne pas satisfaction. Ceci est dû au fait que dans de nombreux cas, la pratique et l'expérience des ingénieurs ne sont pas assez importantes pour se décider sur le bon travail de réparation. Il y a de bons modèles scientifiques pour la conception de la structure mais pas pour la réparation des désordres qu'elle subit. Cette lacune est dû essentiellement à la dispersion des savoirs dans les ouvrages et manuels et chez les experts (David and Lenat,1982). Il est donc nécessaire d'analyser ces savoirs et expériences et de les rassembler pour en faire en faire un moyen d'aide pour le diagnostic et la réhabilitation des bâtiments.
Une construction peut être examinée sur la base d'observations directes, sur le site ou en laboratoire pour des tests et analyses numériques ; ces observations seront si besoin réajustées ultérieurement. Il faut noter cependant qu'une façon rationnelle d'opérer demanderait une estimation économique des risques pas à pas. Ces risques relatifs à une situation fragile nécessitent une évaluation de connaissances plus profondes, possible par de nouveaux tests ou analyses grâce au système expert.

5.2 Objectifs :

L'objectif de cette étude vise l'élaboration d'un prototype de système qui utilise les techniques de l'Intelligence Artificielle pour faire face à la complexité de l'analyse des désordres d'origine structurelle qui se produisent dans les bâtiments et pour proposer des solutions de réparations.
Quelques principales caractéristiques d'un tel système sont :
- de fournir une liste de désordres sévissant dans les bâtiments et parmi lesquels l'utilisateur retrouvera ceux affectant l'ouvrage qu'il souhaite traiter.
- d'établir les causes des désordres observés parmi une liste de causes potentielles car il faut se rappeler que les endommagements résultent assez souvent de la conjugaison de plusieurs facteurs.
- d'apporter - suivant le coefficient de sécurité qui permet d'indiquer s'il faut entamer des réparations ou non, et, selon l'avis du maître d'ouvrage - des solutions de réparation avec suppression de la cause ou des suggestions d'intervention de " petites ampleurs ". Il reste entendu que le traitement des incertitudes relatives à la connaissance profonde des causes de désordres et aux procédures de réparation (intégration de solutions de réhabilitation dans le contexte urbain) joue un rôle important dans un tel système.

5.3 Petit guide pour la recherche des causes de désordres :

Les désordres dans un ouvrage sont généralement dus à la conjonction de plusieurs causes. Dans les ouvrages en béton, ces désordres se traduisent le plus souvent par l'apparition de la corrosion des armatures qui gonflent et font éclater le béton et/ou par la désorganisation de la cohésion du matériau mais surtout par des fissures dont l'ouverture apparente dépasse les valeurs (0,1 à 0,3 mm) correspondant à un fonctionnement normal du béton armé (fissures fines et réparties) ou des fractures (fissures d'ouverture dépassant le centimètre). Pour ces fissures, le plus couramment observées (Gambardella and Moroni,1989). Ces problèmes qui peuvent être résolus par l'expérience des heuristiques sont adaptés pour l'usage de système expert.

5.4 Intégration des coefficients de sécurité et des facteurs de certitudes :

Un coefficient de sécurité résiduel renseigne sur le "niveau de sécurité" d'un ouvrage existant et indique s'il faut oui ou non entamer une réparation. Cependant, après le diagnostic, on peut toujours se poser la question de savoir si la solution de réparation préconisée est vraiment la bonne. Afin de lever les doutes qui subsistent, certains chercheurs recommandent l'utilisation de facteurs de certitudes (FC) dans les règles. En nous inspirant des travaux de Reinhardt et Sohni(IABSE,1989) de sur l'utilisation de système expert en génie civil, nous pouvons envisager d'intégrer les facteurs de certitudes dans notre système. Ces facteurs viennent étayer les enseignements tirés des calculs de coefficients de sécurité et constituent une argumentation appréciable quant aux choix des solutions de réparation.

En effet, un facteur de certitude est une valeur numérique qui indique une mesure de confiance dans la valeur d'un paramètre. Dans la base de connaissance, il observe l'expérience réelle que les faits ou les opinions ne connaissent pas toujours avec une certitude absolue. Ainsi, un système expert peut prendre en compte deux incertitudes :

* Les faits et relations du champ du problème recouvrent des incertitudes. Souvent l'expert doit faire des constats tels que : *" si les conditions sont rencontrées, le résultat revient presque toujours. Toutefois, un résultat différent peut arriver. "*

* Nous pouvons ressentir un petit doute quant aux conditions d'exécution ou environnementales de l'ouvrage. Par exemple, *" je ne sais pas exactement s'il y a eu certains événements dans la vie de la structure (exemple : températures élevées), mais je suppose que oui "*. L'exemple suivant montrera comment notre prototype peut s'intéresser au coefficient de sécurité et au facteur de certitude.

Supposons que nous trouvions des fissures dans la structure de béton et que nous ayons à en trouver la cause. Les quatre règles suivantes font partie de la base de connaissances :

R1 Si coefficient sécurité = faible
alors réparation = nécessaire

R2 Si désordre = fissures
type de fissure = modèle sinueux
condition d'environnement pendant l'hydratation = basse température
alors cause = perte de chaleur d'hydratation FC = 50

R3 Si début de désordre = premiers jours après mise en oeuvre
type de ciment = pas de ciment de basse température
alors cause = perte de chaleur d'hydratation FC = 90

R4 Si diagnostic = perte de chaleur d'hydratation
alors réparation = réagréage de la fissure par technique du béton projeté

La discussion peut aller dans ce sens. La première règle est nécessaire pour pouvoir entamer un diagnostic et effectuer un traitement du désordre observé. Après avoir répondu que l'on a trouvé un modèle sinueux et que l'élément était du béton de masse, le système peut demander : *" décrivez les conditions d'environnement et indiquez votre degré de certitude "*. Nous pouvons répondre *" froid avec 70 % de certitude "*. Les conditions, dès les premières règles " SI ", sont rencontrées et le système combine le facteur de certitude approprié, et nous entrons dans les règles " ALORS ".

cause = perte de chaleur d'hydratation FC = 35 (70% de 50)

Le système prend alors en compte d'autres causes parce que la conclusion n'est pas vraie à 100 %. Nous regarderons les valeurs des paramètres de la règle 2. Si nous savons que le dommage a commencé pendant les premiers jours de la mise en oeuvre, sans pour autant connaître exactement le type de ciment utilisé, supposons que ce n'est pas un ciment de basse température d'hydratation qui a été employé. Nous répondons avec un degré de certitude de 50 %. Le système expert, quant à lui, utilise les équations suivantes pour combiner les facteurs de certitudes :

[1] (FC (règle)_FC de ("si") x FC de la fonction de conclusion + 50)/100

[2] (FC (antérieur) + FC (règle) x (100 - FC (antérieur) + 50)100

FC(antérieur) est le facteur de certitude avec la valeur des paramètres avant que le système expert apporte l'action du "ALORS" de la règle suivante. Notons que le dernier 50 dans le numérateur des équations est compris pour arrondir et seule la partie entière est utilisée.

Exemple :

[1] FC (règle 2) ((70 x 50 + 50)/100)x 35,5 ====> FC = 35
FC (règle 3) ((50 x 90 + 50)/100)x45,5 =====> FC = 45

[2] FC = 35 +(45 (100 - 35) + 50)/100 + 35 + 29 = 64
======>FC= 64

La cause des fissures est encore la perte de la température d'hydratation, mais la preuve additionnelle augmente le facteur de certitude à 64. Ainsi, de proche en proche, le système en combinant toutes les règles et les facteurs de certitudes rassure sur le diagnostic. L'utilisateur peut alors, sans appréhension, retenir la solution de réparation de la règle 4.

Le traitement des incertitudes à travers le calcul des coefficients de sécurité et l'intégration de facteurs de certitudes dans les règles de diagnostic joue un rôle important dans un tel système. Néanmoins dans ce qui suit, ce thème ne sera pas traité particulièrement dans nos règles de diagnostic ou de réparation puisqu'il peut être abordé séparément du développement du corps principal de notre prototype de système. En effet, un pré - diagnostic, à travers une auscultation sérieuse de l'ouvrage et des calculs rigoureux de coefficients de sécurité, permet de lever un certain nombre de doutes.

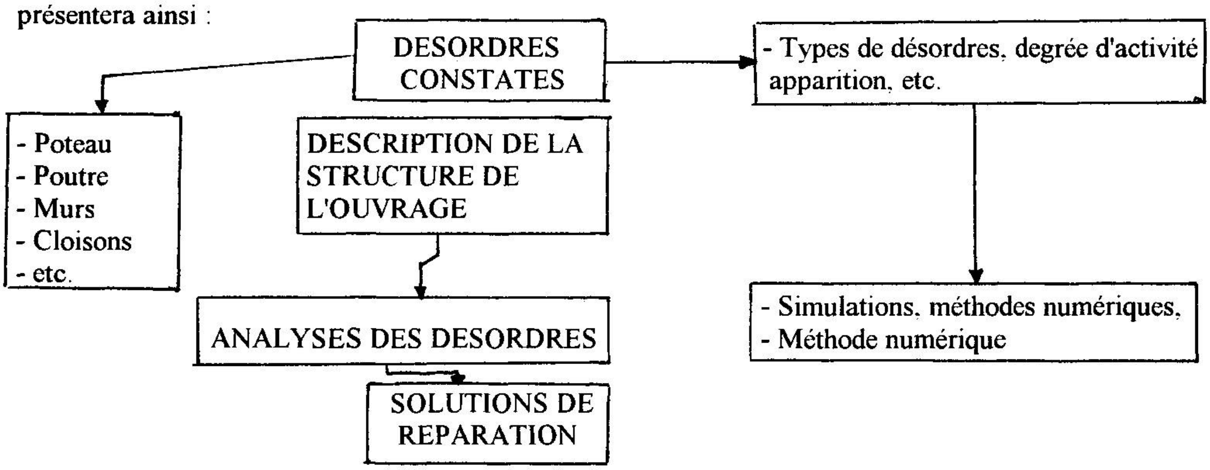
5.5 Pour le diagnostic des désordres d'origine structurelle :

D'abord, la structure et la destination de la construction doivent être spécifiées. Quelques informations importantes peuvent aider à trouver les causes des désordres, comme la nature du sol ou le matériau utilisé. Il sera ensuite établi une base de faits à partir de toutes ces informations disponibles. Auparavant, un examen de la construction sur la base d'observations directes sur le site ou au moyen d'instruments d'auscultation a permis de savoir la nature des désordres qui apparaissent très généralement, comme nous l'avons déjà signalé, à travers les fissures.

Sur la base de faits existants et de désordres constatés, les causes de ces derniers vont être établies et analysées dans un dialogue entre l'utilisateur et le système. Différents types de désordres - tels que l'éclatement du béton, armatures apparentes des poteaux, fissuration généralisée du bâtiment, etc. - sont présentés dans les règles de la base de savoir. On note alors que leurs causes à établir par le système expert sont parfois multiples.

5.6 Pour la réparation des désordres :

Une fois le diagnostic effectué, différentes solutions de réparation seront données. Nous établissons ainsi des règles liant les désordres observés au choix de la méthode de réhabilitation. L'architecture du système se présentera ainsi :



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Correcting Design Deficiencies in a New Five-Story Building

Correction des faiblesses du projet d'un bâtiment de cinq étages

Korrektur von Konstruktionsmängeln eines neuen Bürogebäudes

Mario G. SUAREZ
Civil Engineer
Schupack Suarez Eng. Inc.
Norwalk, CT, USA

Mario G. Suarez is Vice-President of Schupack Suarez Eng.. He obtained civil eng. degrees from the Univ. of Havana and the Univ. of Illinois. He has been involved with prestressed concrete design and technology for more than forty years.

Daniel J. McCARTHY
Civil Engineer
Schupack Suarez Eng. Inc.
Norwalk, CT, USA

Daniel J. McCarthy, a Senior Structural Engineer, has been with Schupack Suarez Eng. since 1982. He obtained civil eng. degrees from the Univ. of Connecticut and Columbia Univ. His activity relates to evaluation, repair and strengthening of various types of structures.

SUMMARY

A five-story office building was investigated to assess its structural integrity immediately following construction, prior to occupancy. Each floor level consisted typically of two-way post-tensioned concrete slabs, except the fourth level which was built in reinforced concrete. Significant structural deficiencies were identified. Some elements had little or no reserve against failure at normal service loads. A proposed load test was judged to be inappropriate, as it would have likely resulted in failure of structural elements. Repair and strengthening measures involving various methods and materials were engineered.

RÉSUMÉ

Un bâtiment de cinq étages a été contrôlé pour vérifier l'intégrité de la structure avant son occupation. Chaque étage est constitué d'une dalle précontrainte, excepté le quatrième étage construit en béton armé. Une étude compréhensive a identifié les points faibles de la structure. Certains éléments de cette structure ont peu ou pas de réserve de résistance ultime aux charges de service. Un essai de charge proposé a été refusé car il aurait pro-bablement conduit à la ruine des éléments de la dite structure. La réparation et le renfor-cement, réalisés à l'aide de plusieurs méthodes et différents matériaux, sont présentés.

ZUSAMMENFASSUNG

Ein fünfgeschossiges Bürogebäude wurde direkt nach Errichtung und vor der Inbetriebnahme auf seine statische Zuverlässigkeit untersucht. Die Geschossdecken bestanden aus in zwei Richtungen vorgespannten Platten. Nur die vierte Geschossdecke war in gewöhnlichem Stahlbeton ausgeführt. Ausführliche Berechnungen deuteten auf erhebliche Unzulänglichkeiten des Tragwerksystems hin. Einige Bauteile wiesen wenige oder gar keine Reservekapazitäten gegen Versagen selbst unter Gebrauchslasten auf. Der Vorschlag einer Probebelastung wurde verworfen, da diese leicht zum Versagen einiger Strukturelemente führen konnte. Reparatur- und Verstärkungsmassnahmen mit verschiedenen Baustoffen und Verfahren wurden entwickelt.

1. INTRODUCTION

The structure of a new five story office building was almost finished when the Architect became concerned about its adequacy and the trustworthiness of the Structural Engineer of record. In quest of a second opinion, the Architect contacted a well known consultant, recognized as an expert in prestressed concrete, and requested a limited inspection and evaluation of the structure under construction. The Consultant issued a report listing a number of design-related structural deficiencies, including some on the fourth and fifth floors, which would render the structure unsafe.

Realizing that the need of protracted, in depth engineering assistance, to resolve the problems encountered and to design the strengthening measures required, could most practically and efficiently be provided by a consultant located nearby, the Architect was referred to, and engaged, Schupack Suarez Engineers, Inc. (SSE).

Initially, the objective of SSE's investigation was to confirm the first Consultant's findings regarding the inadequacy of the structural design of the fourth and fifth floors. Later, this was expanded to also assess the structural adequacy of the entire building and to engineer retrofit strengthening measures where the structure was found to be substantially deficient.

2. STRUCTURE DESCRIPTION

Fig. 1 represents the general structural framing of a quadrant of the original as-designed building. The roof, fifth and fourth floor slabs stepped back for architectural purposes. The third, second and first structural levels (not shown) have supporting columns around the perimeter of the building.

The roof and all structural levels, except the fourth, are two-way post-tensioned

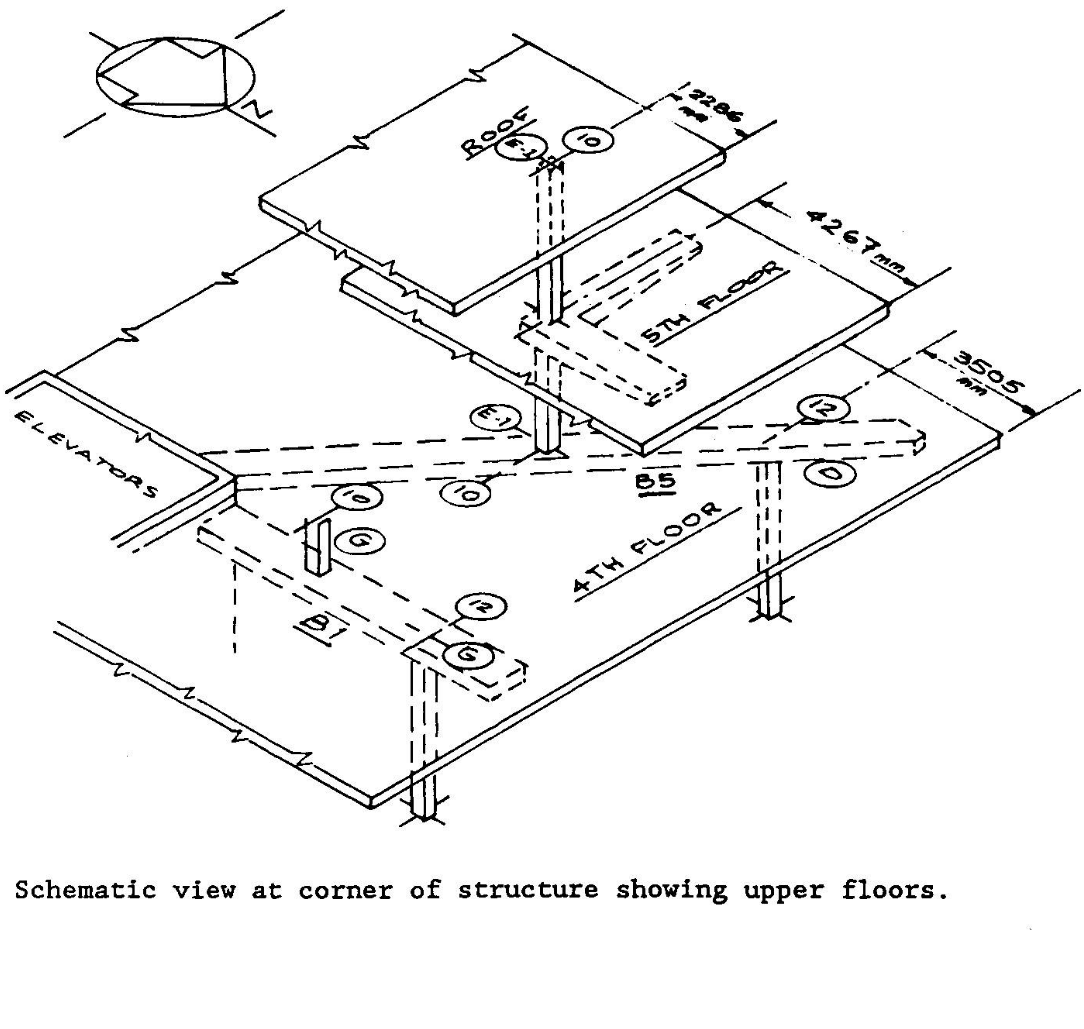


Fig. 1 Schematic view at corner of structure showing upper floors.

flat plates. Typically, based on US current practice, the post-tensioning strands are uniformly spaced in one direction (north-south) and banded along column lines in the orthogonal direction (east-west). Not typical of current practice was the use of relatively long cantilever overhangs of the roof and fifth level slabs, and the short beams cantilevered from the exterior columns, composite with the fifth floor slab.

The fourth level was constructed as a conventionally reinforced concrete slab and beam frame. As is seen in Fig. 1, the exterior perimeter columns supporting the roof and fifth levels did not extend below the fourth level but, instead, were supported on and transferred their loads to shallow, reinforced concrete transfer girders.

There is a three level parking garage structure which is north of and contiguous to the office building, and separated from it by expansion joints. It has been excluded from this report because of space constraints.

3. ANALYSIS

The behavior of two-way flat-plate reinforced or prestressed concrete slab systems, with rather uniform column spacings, is fairly well understood. They can be analyzed by using any one of a number of available "approximate" methods. This is not the case of the fifth and fourth level framings of the structure in question, considering the unusual cantilever beams and shallow transfer girders, as well as the use of relatively large slab overhangs, which make the load path distribution much more complex. Therefore, "approximate" methods of analysis were not considered appropriate.

By using linear elastic finite element computer analysis techniques, and explicitly modeling the beams coupled to the slabs, a requisite assumption regarding load distribution was not necessary. In this way, the actual load distribution to slabs, beams, columns and walls would be more rigorously determined, implicitly accounting for the most effective load carrying capability of the as-designed structure. The computer results were verified with approximate hand computations.

Based on the analysis results, concrete components were checked against rated capacity as per code. Where deficiencies were pinpointed and could not be rationalized as structurally acceptable, strengthening was judged to be required. After the strengthening details were formulated, the component structure was re-analyzed to ensure general compliance to the building code.

4. MOST SIGNIFICANT FINDINGS

4.1 Fifth Floor

4.1.1 Deficiencies

The principal deficiencies determined by analysis were: the cantilevered slab/beam assemblies were under-designed and had excessive deflections; most of the supporting columns, especially the perimeter ones supporting the overhangs, were also found to be significantly under-designed.

Fig. 2A shows the deflected shape, as determined by analysis, of the fifth floor slab system, as originally designed, under full service load. It is quite apparent that the slab deflections, especially those of the cantilever overhangs, are excessive, possibly implying that the slab is overstressed. There was some visual evidence of this overstress in the form of slab top and column cracking. Fortunately, the fact that the cantilevered slabs had remained shored all along must have limited the amount and size of cracking observed.

It was estimated that some portions of the cantilevered slab and some peripheral

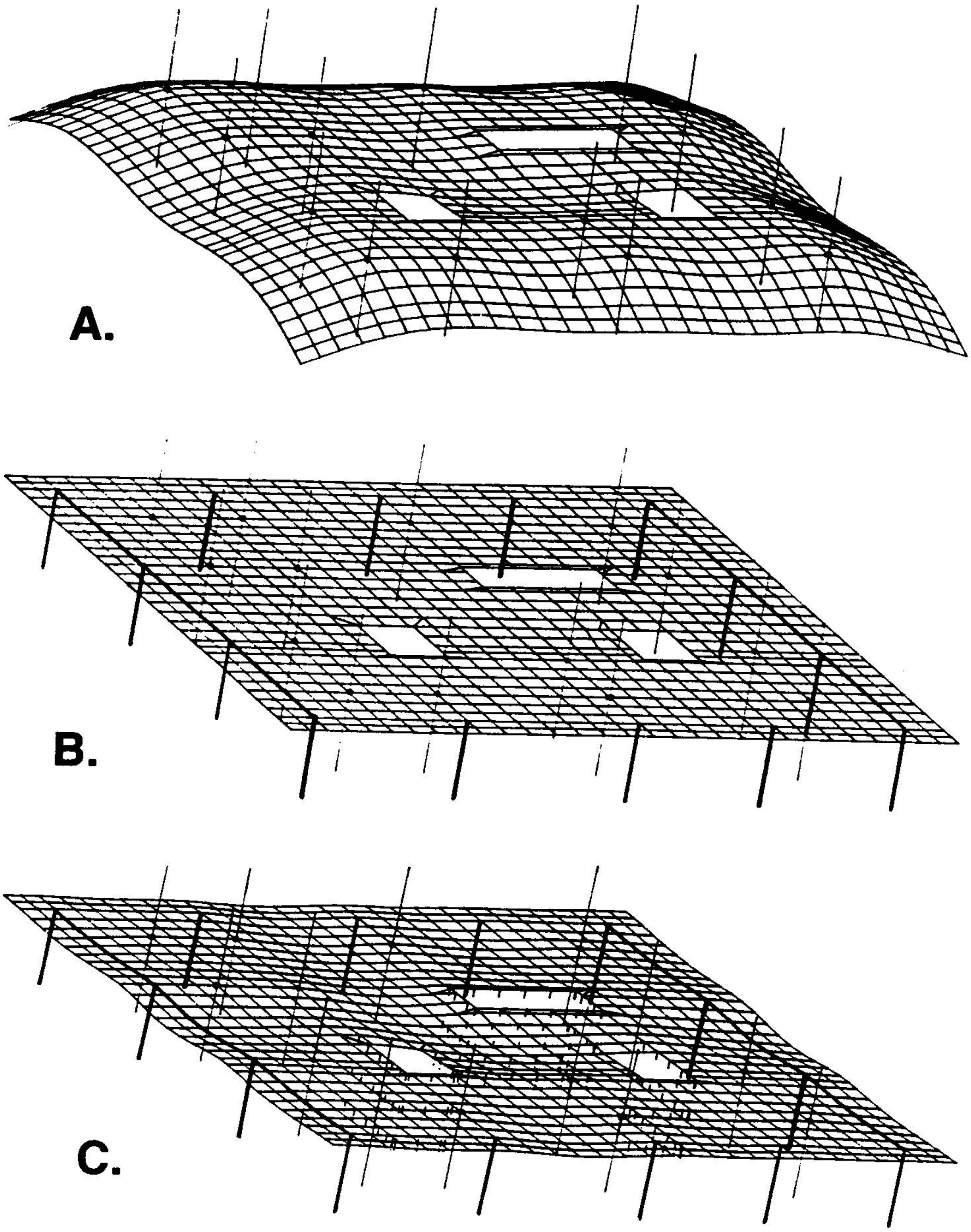


Fig. 2 Finite element models of fifth floor two way posttensioned slab system:

A. Deflected shape under full service loads as originally designed (scal factor = 50).
B. Location of added steel beams and perimeter columns (shown bold o undeflected model).
C. Deflected shape under full service load with added steel beams an perimeter columns (scale factor = 50).

columns would possibly have failed had full service loads ever been achieved. Therefore, a code load test, as requested by the Engineer, was judged to be inappropriate as it would have likely resulted in permanent damage to or partial failure of the structure.

4.1.2 Strengthening Scheme

The major strengthening was accomplished by adding a new line of steel columns (in effect, "permanent" shores) around the periphery of the floor to reduce the slab cantilever (see Fig. 2B). A steel beam was placed on top of the columns parallel to the north and south slab edges in order to control the cantilever deflections in the uniform spaced tendons direction. The new steel beams were preloaded by shimming to predetermined beam deflections, in order to pick up, at least, part of the dead load. The steel columns on the east and west sides were shimmed directly under the short cantilever beams. All the new steel columns sit directly on top of the periphery concrete columns supporting the fourth floor. Consequently, the loads picked up by them is directly transferred down the structure to the footings. This strengthening scheme was chosen because of its relative ease and speed of implementation, and its relatively minor disruption of lease space.

Fig. 2C represents the deflected shape of the strengthened fifth floor under full service loads. Comparing this figure with Fig. 2A, it is readily evident that the cantilever slab deflection and, thus, tensile stresses, have been substantially reduced. The latter, as shown by analysis, appear to be within allowable code levels.

Significant reduction in the cantilever slab stresses and moments resulted in a correspondingly significant reduction of bending moments transferred to the concrete columns.

Where analysis indicated several local regions of slab bottom overstress, these areas were locally strengthened by adding supplemental reinforcing steel bonded to the slab soffit with latex modified shotcrete. A slab top area near the elevator core which analysis indicated was overstressed, since no top steel was called for in the original design, was strengthened by placing reinforcing bars in slots cut into the slab and bonding the bar to the slab with latex modified mortar.

4.2 Fourth Floor

4.2.1 Deficiencies

The principal design-related deficiencies found were: the shallow, reinforced concrete transfer girders, as well as a number of their supporting columns were significantly under-designed; some local slab areas were found to be overstressed (in tension). It is not known why the fourth floor slab was designed in conventional reinforced concrete, when post-tensioning could have been particularly beneficial.

Probing hand calculations made initially by SSE had confirmed the findings of the original Consultant that the transfer girders and slab were highly deficient; so, in order to expedite the design of the necessary strengthening, a full analysis of the as-designed fourth floor was never carried out. The computer analysis actually done contemplated the changes already designed for the fifth floor, including the effect of the loads from the new supporting steel columns. In essence, some of the load which would have been placed on the fourth floor beam/slab system, as originally designed, by-pass the floor and go directly down the building through the steel columns, to the columns and footings below. Even with this improved load distribution, the transfer beams and slab and many of the peripheral concrete columns supporting the fourth floor cantilever were still found structurally deficient.

Evidence of the probable distress which could be anticipated in the transfer girders, under full service loads, was the observed extensive flexural cracking under dead load only, principally under the columns supported by the girders. The magnitude of the cracking of the transfer girders, under partial dead load and relatively light construction loads, clearly indicated a lack of structural strength.

The columns below, supporting the transfer girders, also had evidence of distress under reduced dead load conditions in the form of flexural cracking. Because of the heavy concentrated column load coming down at about midspan of the transfer girders, the top of the supporting columns would bend in towards the center of the building, with cracking thus expected on the outside faces of the columns. This cracking was clearly evident under partial dead load conditions only. Excessive deflections and failure of some structural elements might have occurred had full service loads ever been achieved.

Fig. 3 External post-tensioning of transfer girders and steel collars confining column tops

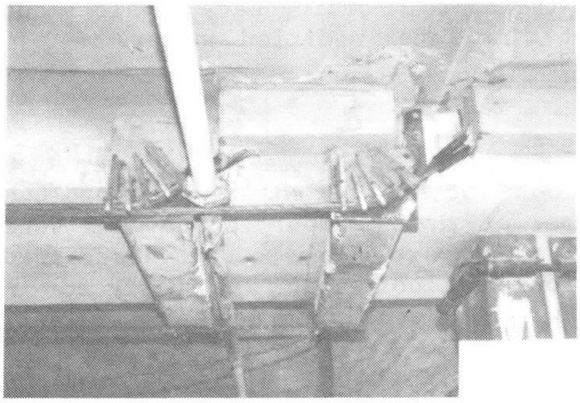


Fig. 4 Close-up of external post-tensioning tendon deviation saddles at transfer girder

4.2.2 Strengthening Scheme

As previously discussed, the analysis of the fourth floor revealed significant structural deficiencies in the transfer girders, slab, and in 7 out of the 16 concrete columns supporting the fourth floor.

The most practical solution to retrofit the transfer girders was determined to be the use of external post-tensioning (see Fig. 3). To fully resolve all structural deficiencies resulting from the analysis would have been impractical. Therefore, a re-analysis of the fourth floor allowing for plastic deformation of the concrete (the creation of hinges) at factored load conditions was carried out. In this way, moments were redistributed to girder and slab regions which would be more convenient to reinforce. Yet, up to factored load conditions, no structural integrity would be compromised.

Based on this re-analysis, using plasticity principles, the external post-tensioning was designed to ensure that the transfer girders achieve ultimate strength at factored load conditions. The deficient top and bottom slab regions were strengthened with additional reinforcement bonded to the slab (as was done on the fifth floor level). The re-analysis revealed that the strengthened slab system would perform adequately if the tops of the 7 deficient columns were allowed to develop a plastic hinge as a connection to the concrete beams above. To ensure the safe plastic deformation of the concrete column tops, confinement was provided using steel angle collars tightened together by high-strength bolts to achieve about 3.4 MPa of lateral pressure (see Fig. 4).

5. EPILOGUE

This case study is, in synthesis, a sad exposé of a structural engineer who totally disregarded the standard of reasonable care that is expected of any professional.

Tripod Column Subjected to Extreme Loading Conditions

Pilier-trépied soumis à des cas de charge extrêmes

Dreibeinige Stütze unter extremen Belastungszuständen

Salah E.E. EL-METWALLY
Associate Professor
El Mansoura University
El-Mansoura, Egypt

Salah El-Metwally obtained his Ph.D in Structural Engineering from Purdue University in 1986. Since then he has been a faculty member at El-Mansoura University. He joined Dar Al-Handasah Consultants between 1990-92.

Hossam M. ABDOU
Assistant Professor
Zagazig University
Cairo, Egypt

Hossam Abdou obtained his Ph.D. in Structural Engineering from the University of Michigan, Ann Arbor. His research interest is related to the analysis and design of concrete structures. He is a member of ACI committees 343 "Concrete Bridge Design" and 368 "Earthquake Design of Structures".

Sami HADDAD
Partner
Dar Al-Handasah
Cairo, Egypt

Sami Haddad has a B.Sc. and M.Sc. in Structural Engineering. He joined Dar Al-Handasah in 1976 and worked as a structural engineer since then. He is currently the Director of the Structural and Construction Management Department.

SUMMARY

The main frames of a unique shading structure are spaced 50 m apart and span 140 m. The frame rafter is formed by a three-dimensional tubular steel box truss which frames into 70 m high tripod columns. The design requirements led to the consideration of many solutions for the tripod column. In this paper the role of conceptual design in arriving to the most efficient solution for the tripod column is illustrated.

RÉSUMÉ

Les cadres principaux d'une structure destinée à procurer de l'ombre sont espacés de 50 m et ont une portée de 140 m. La poutre de la charpente tridimensionnelle est constituée de tubes d'acier et repose, à 70 m du sol, sur des piliers-trépieds. Les exigences du projet conduisent à plusieurs solutions pour le pilier-trépied. L'article illustre le modèle conçu pour arriver à la solution la plus efficace du pilier-trépied.

ZUSAMMENFASSUNG

Die Hauptrahmen eines einzigartigen Sonnendaches bestehen aus dreidimensionalen Rohrfachwerken mit biegesteifem Anschluss an 70 m hohe dreibeinige Stützen. Die Wahl des Tragsystems wurde durch die Forderung nach minimaler Störung des Verkehrsflusses an und unter dem Sonnendach bestimmt, sollte die Rahmentragwirkung erzeugen und die Gründung vereinfachen. Die Entwurfsbedingungen, wie Tragfähigkeit, Duktilität, Unterhaltsfreundlichkeit, Montageverfahren und Bauzeit, erlaubten mehrere mögliche Lösungen für die Dreibeinstütze. Der Beitrag beschreibt die Rolle des Vorentwurfs bei der Entwicklung der wirtschaftlichsten Lösung.

1. INTRODUCTION

A unique shading structure has been designed to cover a very large space in one of the holy places in the Middle East. For the purpose of shading a teflon coated fiberglass membrane has been used, which has a certain degree of translucency in order to allow for natural illumination. The membrane is reinforced by steel cables which are stretched between the main load bearing space frames which are spaced 50 m apart and span 140 m, Fig. 1. In order to control deflection due to any unusual wind loading the main frame is a rigid structure.

The bearing frame was idealized as a three hinged frame and in order to reduce the moments, and thus to transfer loads mainly through axial forces, a space frame was chosen. The main frame rafter is formed of a three dimensional tubular steel box truss which frames into the 70 m high tripod columns. For stability and for controlled deformations the main frames are connected in the longitudinal direction by means of three trusses every bay.

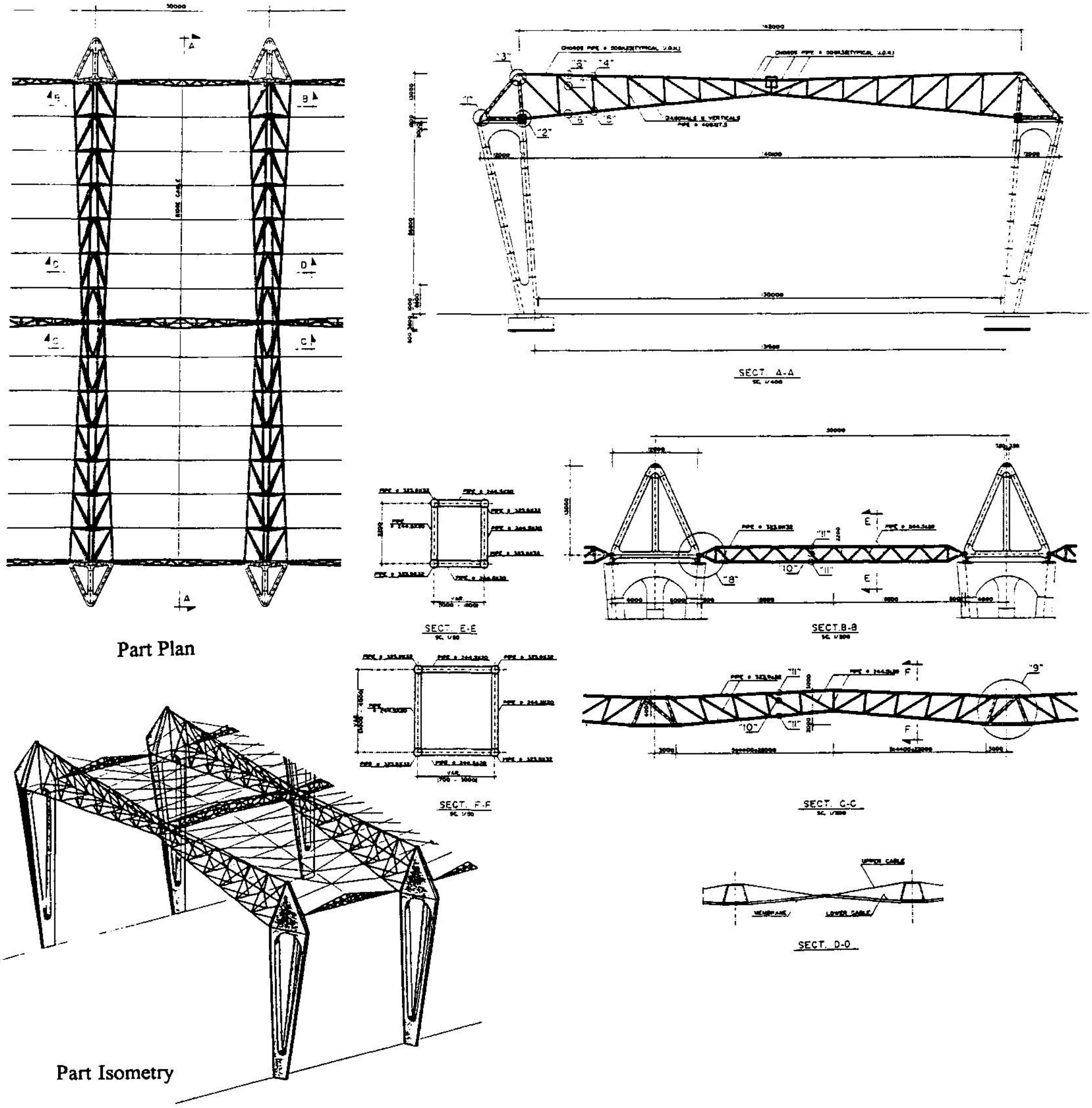


Fig. 1 Shading structure.

The idealization of a structure of such remarkable size and nature represents a great challenge because of its geometry as well as flexibility, very high load magnitudes with their very wide range of variation existed. As a result of structure flexibility a second order analysis had to be adopted during all stages of design.

The choice of the main supporting elements to be tripod columns was based on the need for reducing the interference with the traffic flow around and underneath the shading structure and in order to create the space frame action in addition to simplifying the foundation system. Thus, the foundations of the structure are simple isolated footings.

The large span and spacing between frames will naturally result in forces of very high magnitude. Moreover, the wind magnitude, relatively large compared to the dead and live loads, leads to a very wide range of variation in forces due to its application whether upward or downward. For example, for the outer column the axial force ranges from a small compression to a very high tension (in the order of 1200 ton service). On the other hand, the axial force in inner columns ranged from a small tension to a very high compression (in the order of 1000 ton service).

The significance of the tripod columns dictated the critical consideration of many design factors, such as strength, ductility and maintenance. Also, many construction issues had to be critically considered; for instance, because of time constraints the construction time and hence the construction technique which is also influenced by column height. In order to achieve the design targets different design alternatives were examined and the most suitable two alternatives were either to build the column of precast post-tensioned concrete or to construct a concrete steel composite member.

The two alternatives proposed for column design are presented and a comparison between the two alternatives is drawn. Some unique features of the precast post-tensioned concrete alternative, which seemed to be superior, are discussed.

2. PRESTRESSED CONCRETE SOLUTION

The main reason behind this choice was the high tensile stresses with their range of variation developed in the outer column of the tripod due to load combinations. The wind forces considered in the design were 1.25 kN/m^2 applied to the roof upward and downward and 2.0 kN/m^2 applied to the vertical support. In addition to the load cases due to dead load, live load and wind on the completed structure other load cases were considered such as the load cases during the construction of the tripod as well as during the erection of the main steel trusses, these include the dead weight, wind and construction loads. The structural detailings of the prestressed concrete tripod column are illustrated in Fig. 2.

In order to achieve the highest precision as well as to speed up the construction process the design output was as outlined in the following points.

1. The bottom bulk of the tripod were of cast-in-place concrete. The moments and shears developed due to load combinations at service and during construction at the base are resisted by normal steel reinforcement.

2. The columns of the tripod between the bulk and the top platform consist of 5 precast segments, 8.0 m long each. The prestressing of these columns is anchored in the footing.

3. The prestressing of the outer column of the tripod consist of 18 tendons, of which 12 have 19T15 each while the remaining have 12T15 each. Each of the inner columns have 12 tendons of 12T15 each. For either the outer or the inner columns, 6 tendons 12T15 will be tensioned during the erection of the columns while the remaining tendons will be tensioned after the construction of the tripod column is completed.

4. The segments of the columns will be tied together during construction by means of prestressing

(using the 12T15) and epoxy. The tendons that will not be tensioned until the completion of the tripod column will be coupled between segments; however, not all the tendons will be coupled at the same location. All segments contain about 1.6% normal reinforcement. Each segment has a hole in the middle with a tube attached at the bottom for alignment with lower segments.

5. The columns of the tripod will be braced together during construction by a temporary system at levels multiple of 8.0 m from the top level of the bottom bulk, Fig. 2, to be removed after the placement of the top platform.
6. After the erection of all the straight segments, the precast platform will be placed on the top of the columns and will be anchored to the straight segments with the final prestressing of the tendons. The prestressing at the top of the platform will be anchored to a system of bearing steel plates with brackets which will carry the main steel truss. All the prestressing will be bonded using non-shrink grout except those anchored against the steel bearing plate will be unbonded in the top 2.0 m in the platform region.

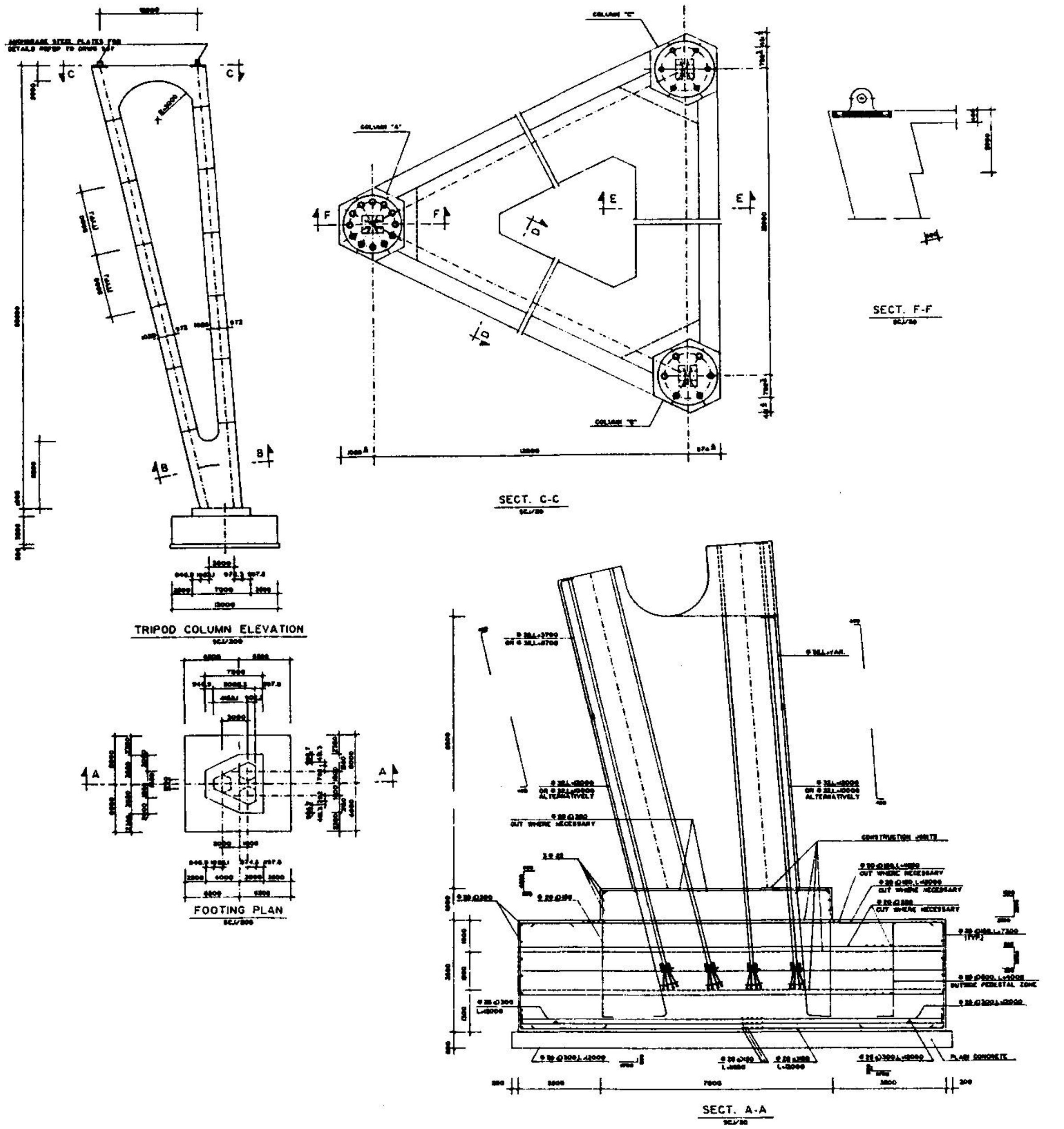


Fig. 2 Prestressed concrete column solution.

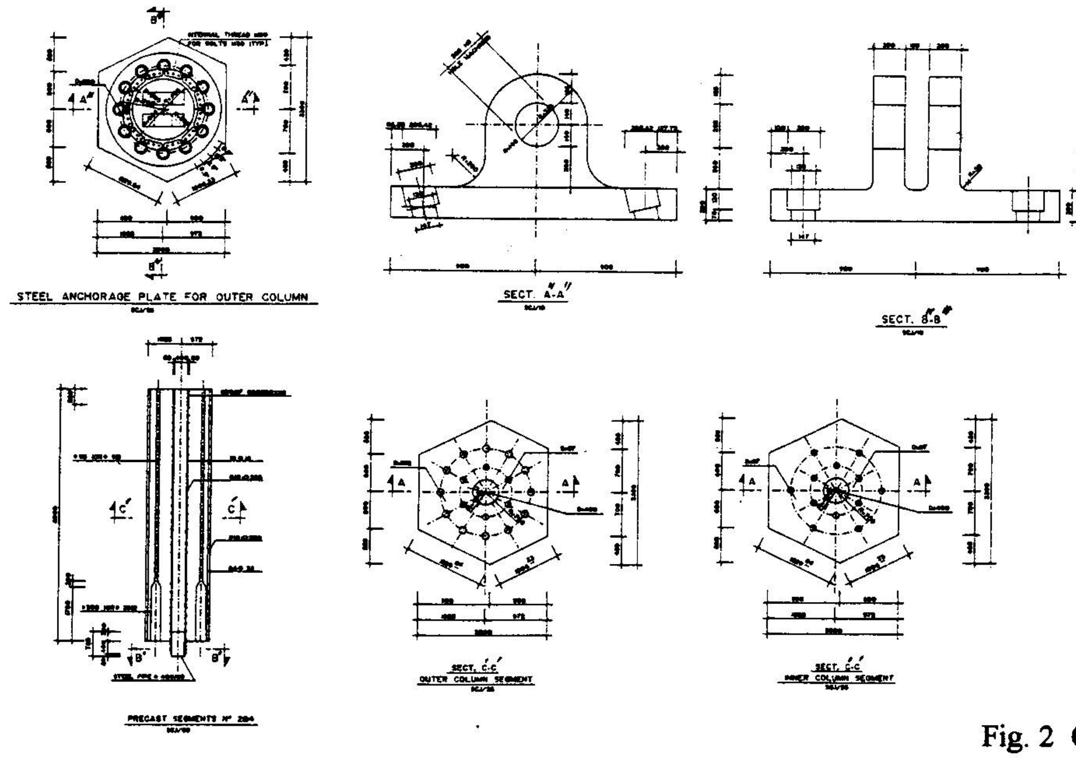


Fig. 2 Cont.

3. CONCRETE STEEL COMPOSITE SOLUTION

In this solution, the tripod column is designed as composite concrete steel members supported on isolated footing. In this type of design, 1100×35 mm steel pipes encased with 300 mm reinforced concrete ring are used; in addition, precast reinforced concrete moulds (cladding) are used as a permanent shutter and also to give the final shape of the column, Fig. 3. The steel pipes will be infilled with lightweight concrete after casting the concrete encasement. The magnitude of wind load and the cases of loading due to construction were the same as in the prestressed concrete solution.

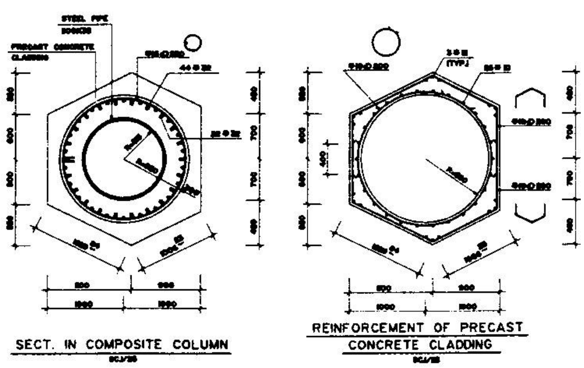


Fig. 3 Composite column solution.

In order to achieve the highest precision as well as to speed up the construction process the design output was as outlined in the following points.

1. The steel pipes will be shipped in sections of about 15.0 m long and welded on site.
2. The steel pipes will be anchor bolted inside the footing.
3. The bottom bulk of the tripod will be made of cast-in-place concrete around the steel tubes.
4. The cladding (concrete moulds of height 4.0 m) will be placed around the steel pipes followed by placement of a circular reinforcing cage between the cladding and the steel pipes. Concrete will then be poured between the steel pipes and the cladding.
5. The bracing of the tripod columns will be as proposed in the prestressed concrete solution.

6. The steel pipes are infilled with concrete after casting the concrete encasing the pipe.
7. The top platform will be cast-in-place reinforced concrete.

4. COMPARISON BETWEEN THE PRESTRESSED AND COMPOSITE ALTERNATIVES

Upon looking carefully at the prestressed and composite design alternatives for the tripod column it can be realized that the prestressed concrete alternative is preferred to the composite alternative. The main arguments for this conclusion are summarized in the following points.

1. Since prestressed concrete experiences mainly compressive stresses normal reinforcement and prestressing steel are protected against environmental conditions; hence, the prestressed tripod columns are more durable. On the other hand, in the composite column the structural steel will be exposed to environmental attack even if encased with concrete because the member will experience both tensile and compressive stresses which will allow for concrete cracking.
2. In the prestressed concrete alternative higher quality control can be achieved since the column segments are precast and on-site casting is eliminated. Moreover, on site welding in the case of the composite solution will be eliminated along with all its requirements of extremely high quality control during welding as well as testing of the welded joints.
3. The prestressed concrete column will require only on-site assembly which will speed up the erection process. As for the composite alternative, on site concrete casting is required in addition to on-site welding which will necessitate longer construction time.

5. CHARACTERISTIC FEATURES OF THE PRESTRESSED CONCRETE SOLUTION

The transfer of loads from the truss into the columns is achieved through the brackets attached to the prestressing bearing plates as illustrated in Fig. 2. This detail transmits the reactions of the truss directly to the prestressing steel in case of tension and to the concrete section in case of compression. In case of tension, additional tensile strain takes place in the prestressing steel while the compression strain in concrete is relieved. In case of compression, the compressive strain in concrete increases while the tensile strain in the prestressing decreases.

The force applied to a column, either tension or compression, as a reaction from the truss, F, is the algebraic difference between the change in forces in the prestressing steel and concrete. As a result of the very large axial stiffness of the concrete section compared to the prestressing steel the change in prestressing force is small. The changes of the force in the column prestressing, ΔP_{sp}, and the column concrete section, ΔP_c, are

$$\Delta P_{sp} = E_{sp}\, A_{sp}\, (F) \,/\, (E_{sp}\, A_{sp} + E_c\, A_c)$$

$$\Delta P_c = E_c\, A_c\, (F) \,/\, (E_{sp}\, A_{sp} + E_c\, A_c)$$

where E_{sp} and A_{sp} are the elasticity modulus and area of the prestressing steel, respectively, and E_c and A_c are the elasticity modulus and area of the concrete section. In the design of the tripod column legs, the prestressing force is set so that the column is always under compression.

6. CONCLUSION

This paper demonstrates the role of conceptual design in arriving to an optimal solution in structural design. The obtained solution should have the features of robustness and durability and should be easy to construct as well.

ACKNOWLEDGEMENT

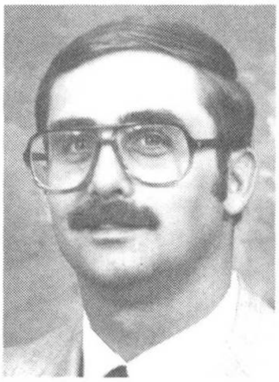
The project discussed in this paper has been designed in Dar Al-Handasah Consultants (Shair & Partners)", Dar Cairo, Cairo, Egypt.

Durable Parking Structures: a Level of Service Approach

Durabilité des structures de garages: une approche du niveau de service

Dauerhafte Parkhäuser - ein Ansatz zum Gebrauchsniveau

Kenneth E. NAPIOR
Structural Engineer
Walker Parking Consultants
San Francisco, CA, USA



Ken Napior, born in 1950, earned an architectural engineer degree at the California Polytechnic State University, San Luis Obispo, CA. He specialises in the design and construction of post-tensioned and reinforced concrete buildings. He has extensive experience in design, construction, and investigation.

SUMMARY

This paper presents a new approach to selecting design criteria based on desired level of service. Selecting the proper criteria can mean the difference between a happy or disappointed client. For instance, the engineer can estimate the slab thickness based on experience. Using this estimate, he can select a slab thickness thicker or thinner, based on the severity of the exposure, location of the project and budget. No one set of design standards is suitable for all situations.

RÉSUMÉ

Cette communication présente une nouvelle approche du choix des critères d'étude qui dépendent surtout du niveau de service désiré. La justesse d'une telle sélection détermine le degré de satisfaction ou de contrariété du maître de l'ouvrage. Ainsi, l'ingénieur peut choisir l'épaisseur d'un plancher en fonction de son expérience. Partant de là, il peut la prévoir plus ou moins grande, selon les conditions de service, la situation de l'ouvrage et le budget dont il dispose. De ce fait, aucune règle générale de projet ne peut répondre à chaque situation particulière.

ZUSAMMENFASSUNG

Im Beitrag wird ein neuer Ansatz zur Wahl von Entwurfskriterien vorgestellt, der auf dem gewünschten Gebrauchsniveau beruht. Die richtigen Kriterien zu finden kann den Unterschied zwischen einem zufriedenen oder enttäuschten Bauherrn ausmachen. Zum Beispiel kann der Ingenieur die Deckendicke nach seiner Erfahrung wählen. Von diesem Anhaltspunkt aus kann er die Decke dicker oder dünner gestalten, je nach Grad der Umwelteinwirkungen, der Lage des Bauprojekts und des Kostenziels. Kein einzelner Satz von Entwurfsanforderungen ist für alle Situationen geeignet.

1. INTRODUCTION

This paper presents a new approach to selecting design criteria based on desired Level of Service. Selecting the proper criteria can mean the difference between a happy or disappointed client. For instance, the engineer can estimate the slab thickness based on experience [12]. Using this estimate, they select a slab thickness thicker or thinner, based on the severity of the exposure, location of the project and budget. No one set of design standards is suitable for all situations[1,9].

2.0 LEVEL OF SERVICE

Traffic engineers developed a system of classifying flow conditions by Level Of Service (LOS). Signalized intersections where traffic is virtually free flowing represents LOS A, the highest level of service. As congestion increases, the level of service decreases. The lowest, LOS F, results in gridlock. Traffic conditions are considered least acceptable for design purposes with LOS D. At this level, delays occur, but are acceptable to regular users.

The LOS concept for the functional design of parking structures was developed by parking consultants in the 1980's [7]. Structural engineers can now apply the level of service concept to selection of the structural criteria, that affect long term performance and durability.

The structural shell for a parking structure is approximately 45 percent of the total building cost [5]. The criteria selection in the schematic phase affects the initial cost and maintenance costs of the structure[7,8]. Owners know that the initial and future values of the project depend upon knowledgeable choices. However, until now, there was no quantitative way to assess the level of service of the design criteria.

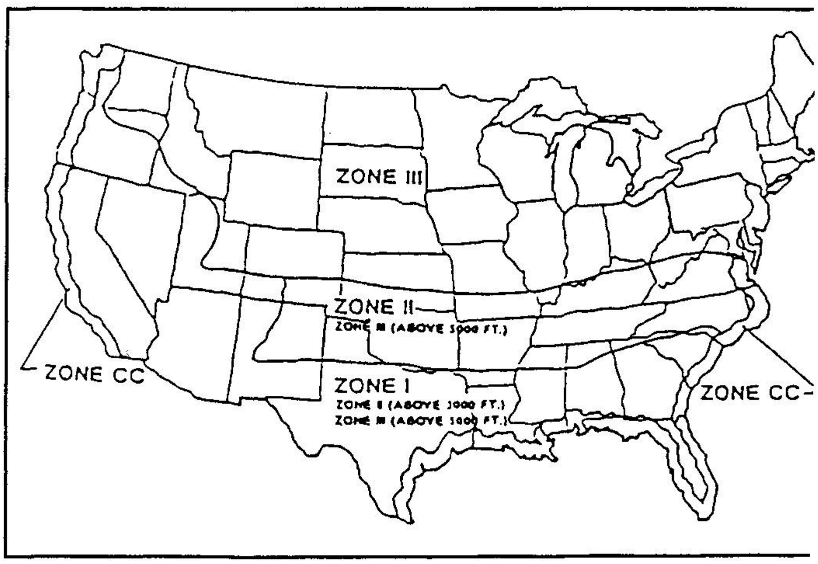
3.0 EXPOSURE

In the United States, five exposure zones are suggested [5] to assist in identifying exposure conditions, Figure 1).

Zone I: Areas with the mildest conditions and neither freezing nor salt is present.
Zone II: Areas where freezing occurs and de-icing salts are never or rarely used.
Zone III: Areas where freezing and de-icing salts are common.

Coastal Chloride Zone I, (Zone CC-I) Areas in Zone I and within 10 miles of a major body of salt water.
Coastal Chloride Zone II (Zone CC-II) Areas in Zone I, and II that are within one half mile of a major body of salt water.

Climates can range from mild exposure to an extremely harsh environment, with large fluctuations of temperature. Chemical attack by de-icing salts is a major concern [2]. Where salt is commonly used on roads and in parking structures, the engineer must account for the increased risk of corrosion. Chloride penetration has been a major catalyst for the onset of deterioration [12,13].



4.0 DESIGN, DETAILING, AND MATERIAL SELECTION

Proper selection of material, detailing and construction practices, affect the durability of the parking structure. Consideration must be given to adequate drainage, detailing for volume changes, material selection and finishing, etc. [1,10]. These criteria are shown in Table 1 [8]. The engineer selects criteria within the budget and durability requirements of his/her client. Thus, the cost of the project is based on quality of the material specified, labor costs, and geographic area.

5.0 DURABILITY ANALYSIS

An approach to assess mathematically the LOS of a slab has been developed by the author. Selections are made from a list of 26 design criteria, see Table 2. An LOS rating is assigned to each criteria. The rating system for some criteria is based on quantity. For example, in a severe climate area, air entrainment may vary from 0 to 7%. A value of 0 and 7% is assigned LOS F and A respectively. The LOS rating for other items is based on their inclusion. For example, water cured concrete is rated LOS A in both a mild and severe climate. If the slab is not water cured, then LOS C is assumed in a mild climate zone, and LOS D in a severe climate zone. The severity of exposure of the structure directly affects the LOS rating.

The relative value of each selection is based on its degree of importance to the overall system. For instance, epoxy-coated reinforcing has a larger relative value than the lateral load carrying system in a severe climate zone for seismic zone 1 and 2. However, in a mild climate zone in seismic zone 4, epoxy coating has a lower relative value than the lateral load carrying system. In a mild climate zone, criteria that are not required, such as corrosion inhibitors, epoxy-coated reinforcing, etc., are non-rated, since their contribution to durability is minor.

The relative importance of some criteria increases, if other complimentary criteria are excluded. This is reflected in the overall LOS rating for the floor system. Omitting epoxy-coated reinforcing becomes significant in a severe climate zone, if criteria such as corrosion inhibitors or sealers are excluded. Thus, the relative value of each criteria is affected by the total selection of the concrete durability system.

The durability analysis includes the purchase costs and installation of each item. The 40 year life cycle cost includes purchase, installation, annual maintenance, removal (water proof membrane), and periodic replacement costs for items such as sealers and membranes. Other items such as aggregate have a one time cost distributed over 40 years.

T A B L E 1

LEVEL OF SERVICE (LOS) FOR STRUCTURAL DURABILITY IN PARKING STRUCTURES
SI Units

	ZONE I			ZONE III		
FLOORS:	LOS A	LOS B	LOS C	LOS A	LOS B	LOS C
Aggregate, low shrinkage type (1)	yes	yes	no	yes	yes	no
Air Entrainment (2)	4%	4%	4%	7%	7%	7%
Beam Depths (3)	L/20	L/24	L/26	L/20	L/20	L/24
Cathodic Protection (4)	no	no	no	no	no	no
Concrete Strength, MPa (5)	35	28	28	35	35	28
Conventionally Reinf. Slab Thickness (6)	L/28	L/36	L/40	L/28	L/32	L/40
Corrosion Inhibitor (7)	no	no	no	yes	yes	no
Dead Load Deflection (8)	L/600	L/480	L/360	L/600	L/600	L/480
Drainage (9)	yes	yes	yes	yes	yes	yes
Encapsulated PT Tendons (10)	yes	partial	partial	yes	yes	yes
Epoxy Coated Reinforcement (11)	no	no	no	yes	yes	no
Expansion Joint Spacing, m (12)	90	115	140	75	90	110
Increased Rebar Cover , mm(13)	25	25	28	50	40	40
Joint Sealants (13)	yes	no	no	yes	yes	yes
Lightweight Aggregate (14)	yes	yes	yes	no	no	yes
Min. Avg. Prestress, MPa, floors (15)	1.40	1.00	0.86	1.60	1.40	1.00
Min. Avg. Prestress, MPa, roof (15)	1.60	1.30	1.00	1.60	1.60	1.20
Post Tensioned Slab (16)	yes	yes	yes	yes	yes	yes
Precast Slab (16)	yes	yes	yes	yes	yes	yes
PT Slab Thickness (17)	L/40	L/44	L/48	L/36	L/40	L/44
Silica Fume (18)	no	no	no	yes	yes	no
Surface Sealers (19)	yes	no	no	yes	yes	yes
Temp. Tendons Avg. Prestress, MPa, floor (15)	0.86	0.86	0.70	1.00	0.86	0.86
Temp. Tendons Avg. Prestress, MPa, roof (15)	1.00	1.00	0.86	1.00	1.00	0.86
Water/Cement ratio (20)	0.40	0.50	0.55	0.40	0.40	0.45
Waterproof Membrane (21)	yes	no	no	yes	yes	no
Water Cured Concrete (16)	yes	no	no	yes	yes	no
Vibration Perception (22)	slight	distinct	strong	slight	distinct	strong

For notes, please see next page.

Footnotes for Tables 1

1. Always use the best aggregate available.
2. Always use at least 4%.
3. Consider in conjunction with dead load deflection and vibration.
4. Do not use with pre- or post-tensioned reinforcement [2].
5. Never use less than 4000 psi.
6. Conventionally reinforced (non-prestressed) slabs are not recommended in Zones II or III.
7. Corrosion inhibitors may contribute little if used with silica fume concrete or with precast concrete.
8. Consider in conjunction with beam and slab depths.
9. The first line of defense is good drainage.
10. Always use.
11. Never depend on epoxy coated reinforcement alone.
12. Consider independently of other durability measures [3,11,12].
13. Inexpensive protection [5].
14. Check long term shrinkage rate.
15. Always needed. Consider independently of other durability measures.
16. Always recommended.
17. Consider in conjunction with dead load deflection and vibration.
18. Not needed with precast concrete.
19. If used with silica fume concrete, only an initial coating needed.
20. Always preferred
21. Not normally needed, except over habitable areas.
22. Probably the most difficult item to deal with, because it is the most subjective [6].

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Floor Slab Durability Analysis

WALKER Parking Consultants/Engineers   Table 2
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Project: Example
Date: January 1995
City: San Francisco

This project has been designated for a mild climate, without ice and de-icing salts. The user should satisfy themselves that adequate safe guards are utilized in the selection of the criteria to protect the structure against premature deterioration.

FLOORS:	Selection	LOS	Initial Cost/sq. m	Life Cycle Cost/sq. m
Concrete				
Aggregate Type	Hardrock	A	$0.01	$0.00
Aggregate Shrinkage Rate	0.04	A	$0.01	$0.00
Air Entrainment Rate	4.00%	A	$0.01	$0.00
Corrosion Inhibitors	No	NR	$0.00	$0.00
Sealers, Penetrating Type	Yes	A	$0.06	$0.00
Silica Fume	No	NR	$0.00	$0.00
Strength at 28 days	35 MPa	A	$0.02	$0.00
Water/Cement Ratio	0.4	A	$0.03	$0.00
Water Cured Concrete	Yes	A	$0.00	$0.00
Reinforcing Steel				
Cathodic Protection	No	NR	$0.00	$0.00
Concrete Cover at Top	25 mm	A	$0.02	$0.00
Encapsulated PT System	Yes	A	$0.01	$0.00
Epoxy Coated Reinforcing Steel	No	NR	$0.00	$0.00
Min. Effective Prestress in Floor Slab	1.4 MPa	A	$0.01	$0.00
Min. Effective Prestress in Roof Slab	1.6 MPa	A	$0.01	$0.00
Temp. Min. Effective Prestress in Floor	0.86 MPa	A	$0.01	$0.00
Temp. Min. Effective Prestress in Roof	1 MPa	A	$0.01	$0.00
Structural Considerations				
Dead Load Deflection Ratio	L/600	A	$0.01	$0.00
Beam Span/Depth Ratio	L/20	A	$0.01	$0.00
Expansion Joint Spacing	76 m	A	$0.01	$0.00
Joint Sealers	YES	A	$0.00	$0.00
Lateral Load System	FRAME	A	$0.01	$0.00
Min. Slope on Floors	2.00%	A	$0.00	$0.00
Slab Span/Depth Ratio	L/43	B	$0.01	$0.00
Vibration Perception	SLIGHT			
Waterproof Membrane on Floors	NO	C	$0.00	$0.00
THIS SELECTION HAS A DURABILITY RATING OF		**A**	**$0.23**	**$0.01**

Diagnosis and Repair of Buildings Damaged by Fire Exposure

Évaluation et réparation de bâtiments endommagés par un incendie

Diagnose und Reparatur von Gebäuden mit Feuerschäden

Osama EL-NESR
Dr. Eng.
Ain Shams Univ.
Cairo, Egypt

Osama El-Nesr, born 1960, received his Ph.D. at Ain Shams University, Cairo. Since 1992 he is involved in research work on the behaviour of structures exposed to fire at Braunschweig University, Germany. He is working at Ain Shams University in the field of reinforced concrete structures.

SUMMARY

This paper summarises the philosophy and procedures to be adopted in assessing the damage by fire to structural elements. It describes the design and execution of the repairs necessary on the basis of such assessment, especially to restore the structure so that it will fulfil its original performance requirements, including fire resistance.

RÉSUMÉ

Cette communication présente un sommaire de la philosophie et des procédures pour évaluer le dommage dans les structures sous l'effet du feu. La réparation des structures endommagées, et le calcul nécessaire, sont décrits afin d'atteindre la performance originale et satisfaire les réglementations, spécialement en ce qui concerne la résistance au feu.

ZUSAMMENFASSUNG

Der Aufsatz beschreibt die Philosophie und das Verfahren, welches bei der Beurteilung von Feuerschäden an Gebäuden angewendet werden soll. Es wird die Bemessung und die Ausführung der auf Grund der Beurteilung notwendigen Reparatur beschrieben, mit dem Ziel die ursprünglichen Anforderungen, insbesondere der Feuerfestigkeit zu erfüllen.

1. INTRODUCTION

The objective of a rapair is to restore the original functions of a structure in the most economical way. Before the repair of a building can be started, the damage must be carefully investigated. It must be known which part of the structure or structural element is damaged and to what extent.

It may be useful to discribe what kind of damage are to be expected after a fire, in order to know to what to look for. Therefore, a classification system is useful for the grading of the damage.

There are a number of publications which give advice on the assessment and repair of fire damaged structures. The most comprehensive is Conseil International du Batiment (CEB W14) Report "Repairability of Fire Damaged Structures", 1990 [4].

Though this paper concentrates mainly upon reinforced concrete structures as these are most readily repaired, consideration is also given to damage and repairability of other types of engineering structures and materials.

2. MATERIAL DAMAGE

The deterioration of material properties occuring during a fire exposure, may, in many cases, not be fully restored after the cooling down to normal temperature conditions.

Residual mechanical properties of the materials used in concrete structures, after cooling depend on a number of parameters, such as: kind of steel reinforcing, maximum temperature attained, stress-level during the manufacturing process, rate of heating and cooling, and duration of maximum teperature level.

2.1 Concrete

The residual strength of concrete can be tested by the well known measures. The dependance of decreased strength on the temperature reached before as well as on the cooling down conditions can be taken from Fig.1-a. As a rough solution, the decrease of the elastic modulus versus temperature may be taken as equal to that of strength.

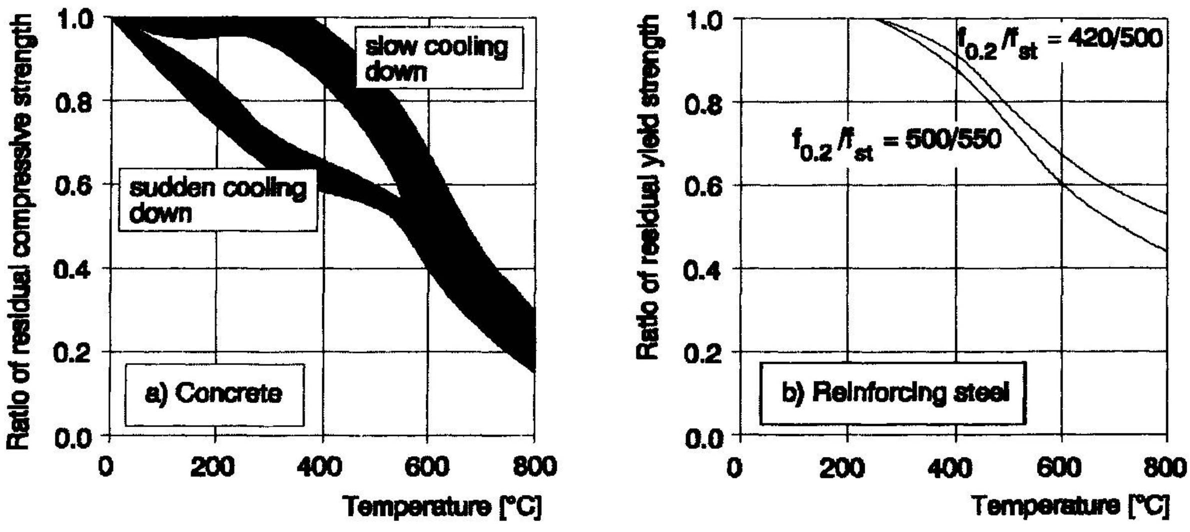

Fig. 1 Residual strengths of concrete and reinforcing steel as a function of temperature reached

2.2 Reinforcing Steel

Hot rolled reinforcing steel recovers its original yield stress if the temperature does not exceed approx. 700°C. With cold worked steel, the work-hardening effect which increases the strength of such typed of reinforcement under normal temperature, surffers regression if exposed to temperatures > 400°C. With temperature lower than 400°C, a residual hardening due to an ageing effect may be observed. Hardening effects, possibly accompanied by a loss of ductility also may be caused by sudden cooling down.

The reduction of the modulus of elasticity may roughly be taken to be in the same range than for the yield stress, Fig.1-b.

3. TEMPERATURE DEVELOPMENT IN CONCRETE CROSS-SECTIONS

The parameters influencing the development of a fire in a compartment, and in consequence the temperature development in concrete elements, are documented in reference [2]. However, a reliable re-construction of the time-temperature process which may have affected the load bearing structure in the individual case to be judged, will very often not be possible.

4. DAMAGE ASSESSMENT

The load-bearing capacity of structural members made from reinforced concrete is reduced through the action of elevated temperatures, because the strength of the concrete as well as that of the steel and the bond between the two materials are diminished. The reduction in load-bearing capacity can be estimated from the maximum temperature, the temporal temperature change and the duration of the heating period.

If the reinforcement has lost strength or ductility, additional reinforcement must be provided to ensure the function of the structure. The design of the repaired members as well as the size and spacing of the additional reinforcement must comply with the code of practice. They must also be practicable in respect of thickness of the concrete cover required.

One special problem can be the placement of the shear reinforcement. There will be no problems for free standing columns or beams, but difficulties will occur in complying with the normal code of practice when additional stirrups have to be placed on reinforced floor beams. To make closed stirrups it is necessary to drill holes through the floor slab at each side of the beam at sometimes very short distances. This will reduce the compression zone of the beam and also affect the shear capacity of the slab. It will also mean that a number of reinforcing bars in the slab are cut.

5. REPAIR TECHNIQUES

It is essential that a repair must restore any loss of strength, maintain durability and fire protection. In situations where, following a fire, there is still sufficient strength and cover for durability then a thin hand or spray applied material could be used to restore a loss in fire protection. Equivalent thickness of concrete cover for various materials may be found from manufacturers. Some information is given in reference [3] and in BS 8110 [1].

5.1 Resin Repair

As a result of the study, it has been necessary to make particular comment on resin repairs. These are commonly used to overcome problems of reinforcement corrosion but may not provide the necessary protection in the event of a subsequent fire.

Resin repairs may consist of a variety of configurations of patch or infill of Epoxy, Polyester and Acrylic mortar. Resins are often used for repairs to lightly spalled areas and, though they may perform quite satisfactorily in normal service, there is no comprehensive information on the performance of such repairs or that of the materials when subject to heat or an actual fire test. What information does exist, including some published papers documented in Ref.[4], indicates that these materials may soften at relatively low teperatures (80°C). As a consequence, it is possible that some resin repairs may not provide adequacy in compression zones. Accordingly it is recommended that resin repairs only be used when either :

- performance data can be supplied to show that the particular formulation has adequate fire resistance and retains its structural properties under the envisaged fire condition, or
- the material is adeqately fire protected by other materials and retains its structural properties at the expected fire temperatures at the relevant depth in the section, or
- loss of strength or other properties of the material will not cause unacceptable loss of structural section or fire resistance.

The designer should refer to the manufacturers literature for details of the performance of the various materials available. A wide variety of materials exists and their specifications are liable to changes at relatively short notice. In general, the materials will be capable of providing good bond and compressive strengths, the flexural and tensile strengths may exceed that of concrete but the thermal expansion is considerably larger than concrete and this may be a point to be considered where the temperature range is large. For further information, the designer should refer to the Concrete Society Technical Report No.26, "Repair of concrete damaged by reinforcement corrosion" [5].

5.2 Polymer Modified Mortars

In many instances, hand repairs of small areas can be effective by use of polymer-modified cementitious mortars. These repairs will generally be to areas or patches of between 12 mm and 30 mm depth. In particular, styrene butadiene rubber (SBR) modified mortars appear suitable. There is limited test information on such mortars but it is expected that they will be satisfactory in a fire as they should behave as a cementitious product. It is to be expected that the use of a small sized aggregate will improve the performance of mortars compared to concretes due to a lesser tendency for damage caused by aggregate splitting in the event of a subsequent fire. In other respects, these mortars are also described in the Concrete Society Technical Report No.26, "Repair of concrete damaged by reinforcement corrosion" [5] to which the designer should refer.

5.3 Cement Mortars

These may be hand applied to damaged areas but great care in surface preparation is necessary in order to ensure adequate adhesion. Generally, mortars will be applicable to well-defined areas placed in layers using good rendering practice, up to a total 30 mm thickness.

5.4 Plaster

This may be readily applied to both plain and roughened concrete surfaces. It can restore a degree of fire resistance but will not replace cover requirements for durability.

5.5 Sprayed Mineral Preparations

This technique will not assist where strength restoration or replace cover requirements for durability are required. Where internal repairs of a minor nature are necessary, these systems will restore fire resistance and shape to damaged members. For particulars, the designer should consult specialist contractors.

5.6 Alternative Supports

The designer can consider the use of alternative supports such as further columns or new beams to sub-divide floor spans. Such schemes may be well prove economic as they may allow lesser restoration to damaged members. Ideally, new supports would be in reinforced concrete but there is no reason why steel, timber or masonry should not be used providing the required strength, fire resistance, durability and appearance are achieved.

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Improved Ductility of Loaded Concrete-Filled Members

Ductilité accrue de tubes en acier remplis de béton

Verbesserte Duktilität belasteter, mit Beton ausgefüllter Stahlquerschnitte

Audronis K. KVEDARAS
Professor
Vilnius Technical University
Vilnius, Lithuania

Audronis Kvedaras, born in 1937, received his engineering degrees at Kaunas Polytechnical Institute. At Vilnius Technical University he is responsible for research and lecturing on steel and composite structures.

Antanas SAPALAS
Professor
Vilnius Technical University
Vilnius, Lithuania

Antanas Sapalas, born in 1959, received his engineering degrees at Vilnius Technical University. Now in the Department of Steel Structures, he is responsible for research and lecturing of steel and composite structures, including their fire resistance.

SUMMARY

A relatively recent development in steel structures is the use of concrete-filled structural hollow sections. Researchers are trying to find more accurate and uniform ultimate strength calculation methods for various composite tubular elements under different loadings. Concrete filled steel tubular linear members are discussed. Because of the interaction between circular steel shell and solid or hollow concrete core, they achieve great ductility, which allows them to provide higher values of ultimate strength.

RÉSUMÉ

L'idée d'utiliser des tubes en acier remplis de béton est assez nouvelle. Les auteurs proposent des méthodes de calcul plus simples et plus précises pour différents éléments composites et différents cas de charge. Ils démontrent que grâce à l'interaction entre les tubes en acier et le béton, ce dernier subit de grandes déformations plastiques et la capacité portante des éléments augmente.

ZUSAMMENFASSUNG

Ein verhältnismässig neues Entwicklungsgebiet ist die Füllung hohler Stahlquerschnitte mit Beton. Die Verfasser versuchen, die Grenztragfähigkeit der sehr verschieden belasteten rohrförmigen Elemente einfacher und genauer zu berechnen. Durch die Interaktion zwischen der kreisförmigen Stahlschale und dem kompakten Beton wird eine grosse Duktilität erreicht, und somit eine höhere Bruchfestigkeit.

1.PRELIMINARY REMARKS

1.1.The Peculiarities of Structural Composite Framework

Structural framework of concrete filled steel tubes finds every time wider application because permits to gain a lot of technical and economic advantages [1,2,3]. There are already many favourable examples of the successful application of concrete filled structural steel hollow sections to both - building and bridge structures. Advantages which can be gained by filling hollow circular or rectangular steel tubes with concrete are stimulating researches in the field. The enhancement of concrete strength associated with circular concrete filled tubes can be accounted for. It is assumed the increase in concrete strength due to triaxial effects and time offsets any possible weakening effects due to creep and shrinkage.

Although a lot of speculations are done by various researchers in the field of expression of the influence of interaction between the external steel shell and the internal concrete core on the increase or decrease of their strength due to triaxial or triaxial-diaxial effects, but it must be admitted there the paramount importance of improved ductility of such loaded concrete filled members. Concrete filled steel tubular linear members because of interaction between the steel shell and concrete core during loading obtain great ductility which, we are sure, allows them to enhance the ultimate strength and to create a possibility for partial self repairing even after the rupture. Interaction occurring because of the difference between the values of Poisson's ratios of steel shell and concrete core allows to look for steels with more favourable distribution of their cross-section or strength and strain properties. There the big importance is playing very high reliability or safety of structural systems collected of such concrete filled steel tubular linear members under various abnormal loadings.

So below the attention is paid on the fundamentals of influence creating the state of such improved ductility in the most simple model - in loaded concrete filled steel tubular member. The usual concrete cores of steel tubes are most suitable only for the first stage of development of structural composite framework, because brittle concrete core and elastic plastic steel shell being loaded in composite linear member and interacting together make a favourable improved ductility in quite simple way. It is likely to be used more effective ductile materials in cores of composite linear members instead of concrete such as some kinds of new composites with more deeply expressed properties of self-repairing though the self-regulation of stress strain state by improved ductility, especially under abnormal loading, for instance, occurring during erthquake or different explosive actions.

So the interest in concrete filled steel tubular structures as well as in all other effective types of steel-concrete composite systems which under loading usually the favourable improved ductility in different members and especially in joints are producing by interactions, is now world-wide.

1.2.Situation with design rules and methods.

Advantages which can be gained by filling steel tubes with concrete are stimulating the preparation of design rules and methods. Usually any combination of axial loads and end moments about the principal axes may be considered, including cases where unequal moments are applied to both ends of the columns. The enhancement of concrete strength associated with circular concrete filled tubes can be accounted for by the procedure. Then design procedures differ in very large scale. At [6] is clearly shown that the confining effect of the tube enhances the strength as the unconfined strength is 34% greater, on average, than the test results. The Eurocode 4 [7] equation gave an average estimation of strength 2% below the average test strength with the failure load underestimated in 26 cases from 44; this is unacceptable for application to design (even after material factors are applied).

There authors are trying to find the way for more accurate and uniform method of calculation of ultimate strength of various composite tubular elements under very different loadings and actions with an easy possibility to avoid too large scatter between the calculated values and the test data, taking advantage of the mathematical theory of elasticity [4] and the theory of plasticity [5] to simplify the solutions.

2.THE FIELD AND RESULTS OF THEORETICAL INVESTIGATIONS

2.1.Behaviour of short compressed members

Our theoretical investigations at first dealt the strength, stress - strain behaviour and limit states of short axially compressed members with solid concrete cores enhanced with circular steel hollow sections. The stability of composite members of the same cross - sections have been investigated too, especially because it was based on the strength results of short elements. For description of stress - strain behaviour the presumptions of elasticity or plasticity theories usually are used. Application of the theory of plasticity as a rule leads to analysis of hollow and solid cylinders, loaded separately with coaxial forces and internal pressure [5]. The Prandtley's diagram is usual there to describe the relation-ships between the stresses and strains (Fig.1).The yield relationships in such case are ordinary expressed as

$$\sigma_r^2+\sigma_r\sigma_\Theta+\sigma_\Theta^2=k^2, \tag{1}$$

or $$0{,}5(\sigma_1-\sigma_2)=k, \tag{2}$$

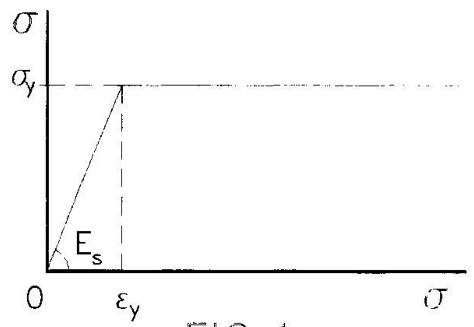

FIG.1

in dependence of criterion of beginning of plastification taken in analysis; where $k=(1/3^{0.5})\sigma_y$ for the Eq.1 (if the HubberMises-Hanley criterion is used) or $k=0{,}5\sigma_y$ for the Eq.2 (if the Sen-Venanh-Tresca criterion is used). These and some other fundamental relationships of the theory of plasticity are useful in analysis of stress-strain behaviour and limit state of concrete filled steel tubular members, but the assumption of independent action of coaxial forces and internal pressure is in contradiction with the reality, because in short compressed circular composite members the only external coaxial pressure is acting on them. In such cases the internal radial and tangential stresses may occur only if the different values of Puasson's ratio for different materials of components of composite cross- section exist. Then the necessity to compensate the breaks occurring on the circles of contacts between the different media arises [4]. But it is impossible by the means of methods of the theory of elasticity [4] to evaluate the increased stiffness even on the beginning of plastification (i.e. when the Puasson's ratios as of concrete core so the circular steel shell get equal each other ($\nu_b=\nu_s=0{,}5$) neither by requesting the reasoning of complex variable functions for more comfortable description nor auxiliary problems about the plain strains following to [4]. The above mentioned evaluation is possible only if to request the law about the equality of axial strains of both media - concrete core and circular steel shell:

$$\varepsilon_{zb}=\varepsilon_{zs}. \tag{3}$$

In the solid concrete core the triaxial stress behaviour of all-round compression exist and in the circular thin-walled steel hollow section only diaxial stress behaviour may be accounted. Then during the plastification of both izotropic media (concrete core and steel shell) when above mentioned different stress behaviours exist, the Eq.3 should exist too. That is being after analogy of the auxiliary problem about the plains strains of elastic coaxial media with different Puasson's ratios [4]. So on the contact circle (between internal surface of steel tube and external surface of solid concrete core) the radial stress σ_r of interaction between the both media should occur

$$\sigma_r=(\overline{R}_bE_s-R_{yn}E_b)/(E_s-0{,}5E_b), \tag{4}$$

where R_b and R_{yn} - the characteristic strength of composite and yield strength of steel; E_b and E_s - the modulus of elasticity of the both materials respectively. There are taken assumptions about relationships between the stresses and strains of concrete and steel at limit state of plastification according to those given in Fig.1. The occurring interaction between the both materials of short composite member in compression allows not only now to look on diaxial stress strain behaviour in both materials, but also to fix a limit state at quite improved ductility usually impossible to carry in each separated single medium. Then according to [5] if action on circular tube consist of coaxial and radial pressure the longitudinal yielding begins from the internal fibres and comes to an end with the external one. So for concrete filled circular steel tubular member it is permissible to assume the plastification firstly covers the internal concrete core and only after that it arises in steel tube. On the ductility of concrete core it is possible to look as on the pseudoplasticity because of process of microcracking of concrete.

2.2. Ductility and stress strain analysis

In the steel shell just before the longitudinal yielding is the elastic plastic stress strain behaviour which may be analysed according to the fundamentals of the version of small elastic plastic strains of the theory of plasticity. It is known that for steel tubes of complicated stress behaviour (tension compression) quite suitable is the law of generalised curves, which is being expressed as

$$\sigma_i = E_i \varepsilon_i \cdot \tag{5}$$

where E_i - secant modulus determined on generalised curve according to generalised strain; σ_i - generalised stress for diaxial stress strain behaviour is being expressed as

$$\sigma_i = \sqrt{\sigma_1^2 + \sigma_2^2 - \sigma_1 \sigma_2}; \tag{6}$$

ε_i - generalised strain, expression of which for diaxial stress strain behaviour is

$$\varepsilon_i = \frac{2}{3}\sqrt{\frac{1-\nu+\nu^2}{(1-\nu)^2}\left[\varepsilon_1^2 + \varepsilon_2^2 + \varepsilon_1\varepsilon_2\left(\frac{3\nu}{1-\nu+\nu^2} - 1\right)\right]}; \tag{7}$$

σ_1, σ_2, ε_1, ε_2 -longitudinal and radial stresses and strains of steel shell respectively, ν - Poisson's ratio of material.

Usually it is assumed that generalised curve is universal for any stress state. So it may be determined according to the curve of uniaxial stress strain behaviour described by the data of standard steel tension test (Fig.2.).that the beginning of the full longitudinal yield of steel shell corresponds to the constant value of secant modulus of steel

$$E_{ui} = 0{,}67 \cdot E_s \cdot \tag{8}$$

That means that exactly fixed ultimate value of generalised strain of steel shell (as elastic plastic deformation criterion of ultimate strength) is known

$$\varepsilon_{iy} = 1{,}5 \cdot R_{yn} / E_s \cdot \tag{9}$$

Then the value of radial strain is

$$\varepsilon_2 = (1/E_s) \cdot (\sigma_2 + 0{,}5 R_{yn}) \tag{10}$$

and the longitudinal one

$$\varepsilon_{1y} = -0{,}5 \cdot \left(\varepsilon_2 + \sqrt{3 \cdot (\varepsilon_{iy}^2 - \varepsilon_r^2)}\right), \tag{11}$$

where σ_r is the radial stress of interaction, above marked as σ_r.

There is found very important thing The secant modulus of concrete at fixed value of generalised strain is

$$E_{ui,b} = E_b / \varepsilon_{iy}, \tag{12}$$

where R_b - the characteristic strength of concrete cylinders.

2.3. Ultimate strength expressions

After the values of ε_{1y}, ε_r, E_{ui}, $E_{ui,b}$ are known, the ultimate values of longitudinal stresses in steel shell and in concrete core have to be determined by

$$\sigma_1 = (4/3)\cdot E_{ui}\cdot(\varepsilon_1 + 0{,}5\varepsilon_r). \tag{13}$$

Then the ultimate strength of composite member have to be expressed by

$$N_u = \sigma_{1s}A_s + \sigma_{1b}A_b \geq N\ . \tag{14}$$

Approximately $\sigma_{1s} \approx 1{,}074R_{yn}$ for circular steel hollow section, $\sigma_{1b} \approx 1{,}64R_b$ for solid concrete core and $\sigma_{1b} \approx 1{,}32R_b$ for hollow concrete core. So improved ductility of steel shell enhances ultimate strength. In some cases the ultimate strength of composite member may be expressed at the moment when the radial stresses of steel shell reach the yield point. Usually that depends on the stock of elasticity of concrete filled circular steel tubular member. Then the value of σ_{1s} and σ_{1b} may be a little higher ($R_{yn} \leq \sigma_{1s} < R_{un}$; $\sigma_{1b} \approx 1{,}74R_b$), depending on the generalised elastic plastic deformation. If walls of steel tubes are superthin, the local stability of such shells have to be checked because might be $\sigma_{1s} < R_{yn}$. and criterion of ultimate strength $\varepsilon_{1u} > \varepsilon_{1y}$. The design values of ultimate strength of materials of composite members are a little less as a rule.

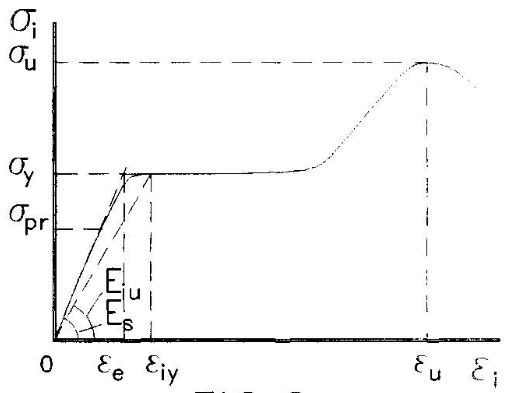

FIG.2

3. The efficiency of composite members

Increase in design strength of stub composite column depends on the yield point of steel R_{yn}, characteristic cylinder strength of concrete R_b and reinforcing factor $\mu_r = A_s/A_b$. The casual relationship of those three quantities

$$\eta = \mu_r \cdot R_{yn}/R_b. \tag{15}$$

The magnitude of increase in ultimate strength of composite member can be expressed as

$$K_E = N_{exp}/N_{de}, \tag{16}$$

where N_{exp} - experimental value of ultimate strength of member in compression at the beginning of longitudinal yield of steel shell; N_{de} - superposition of ultimate strengths of steel shell and concrete core determined separately as for pure reinforced concrete element:

$$N_{de} = R_{yn}A_s + R_bA_b. \tag{17}$$

The relationship between the parameter η and the steel contribution factor δ from [7] can be expressed as

$$\eta = \delta/(1-\delta). \tag{18}$$

Fig.3. deals with the illustration of casual relationship between K_E and η for short spun concrete filled steel tubular element in compression for only one value of the relative thickness of hollow concrete core

β_i=2,0. If look on any other values of β_i the family of such curves should be plotted. The factors K_E values have been calculated for the group of spun concrete filled steel tubular buckle members: K_E =1,21 - in uniaxial compression; K_E = 1,27 - in eccentric compression; K_E = 1,32 - in bending. The short stubby column of the same cross - section has K_E = 1,17. That means more high efficiency of buckle differently loaded members than short one and is in contradiction to requirement of [7] not allow confinement effects when slenderness ratio is $\lambda > 0{,}5$. On the ground of similar diversities erroneous conclusions about the efficiency of only short coaxially compressed concrete filled steel tubular members sometimes are being done.

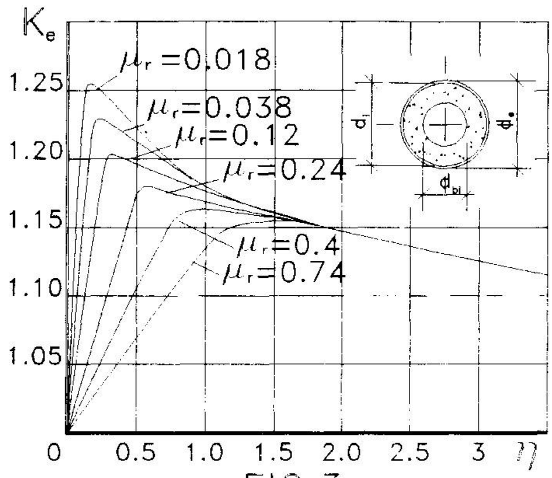

FIG.3

4.Conclusions

1. An improved ductility of concrete core and steel shell allows enhance the strength of concrete filled steel tubular members with great efficiency and safety.
2. The improved ductility of loaded concrete cores and circular steel shells allows to prepare the quite accurate way for analysis of ultimate strength of short composite elements on the base of principles of the theory of plasticity.
3. Design procedure of various composite members is most efficient and reliable if based on the results of design of short concrete filled circular steel tubular columns.

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Strength Reduction in Masonry Due to Dynamic Loads

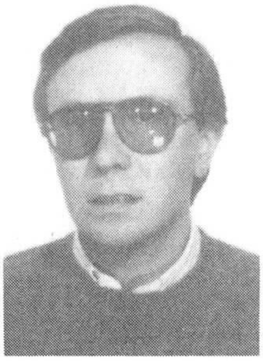
Réduction de la résistance de la maçonnerie sous charges dynamiques

Festigkeitsrückgang im Mauerwerk unter dynamischer Last

Nicola AUGENTI
Professor
Univ. "Federico II"
Naples, Italy

Nicola Augenti, born 1944, received his civil eng. degree at the Univ. of Naples, Italy, where, he teaches Comprehensive Design of Structures. He has carried out studies for the diagnosis and repair of structural damage and worked on the strengthening and restoration of historical buildings.

Paolo CLEMENTE
Researcher
ENEA Casaccia
Rome, Italy



Paolo Clemente, born I959, received his civil engineering degree and his Ph.D. in structural engineering at the Univ. of Naples. Since 1985 he has been researcher at ENEA Casaccia and has worked in structural and earthquake engineering.

SUMMARY

The results of a broad experimental investigation of the behaviour of scaled models of well arranged masonry panels, made of yellow tuff stone and cement mortar, are shown. Force-displacement diagrams have been plotted both in the case of dynamically prestressed and non- prestressed specimens. Significant strength reductions have been observed in cases with a high number of cycles.

RÉSUMÉ

L'article présente les résultats d'un grand nombre d'essais sur des panneaux de maçonnerie de tuf jaune et de mortier. Des diagrammes forces-déplacement ont été réalisés dans les deux cas de maçonnerie précontraintes ou non. Une réduction notable de la résistance a été constatée dans le cas de la maçonnerie soumise à un grand nombre de cycles de charge.

ZUSAMMENFASSUNG

Gezeigt werden die Ergebnisse einer experimentellen Studie über das Verhalten eines gut gebautem Mauerwerks, bestehend aus gelbem Tuff, Mörtel, und Zement . Die Diagramme Stärke-Bewegung von dynamisch belasteten und unbelasteten Modellen werden gezeichnet. In dem Fall von einer grossen Zahl von Zyklen wurde eine empfindliche Festigkeitsreduktion des Mauerwerkes gefunden.

1. INTRODUCTION

Masonry constructions behave very well under vertical loads but they are not suitable to bear horizontal forces, especially those due to seismic actions. Strong vibrations may cause the collapse of the structure but also dynamic stresses due to microvibrations play an important role in the lifespan of masonry structures. In fact, vibrations of small amplitude but characterized by a high number of cycles cause the reduction of the masonry strength due to the deterioration of the mortar and to its detachment from the bricks. In this condition the lifespan of structures cannot be long.
This is certainly true for the masonry building of the Naples area, characterized by the traditional weaving, called "a sacco". It is made by two sheets of bricks with a rubble fill between them. This kind of masonry is not suitable for buildings in seismic area.
In this paper a well arranged masonry is proposed and its behaviour is investigated. In a well arranged masonry wall the two sheets are connected by cross stones, without rubble fill.
The aims of the research project are to investigate the load-carrying capacity both under vertical and horizontal loads and to analize changes in the mechanical behaviour of such masonry walls due to dynamic loads. A wide numerical investigation has been performed on 1:6 scaled models of masonry panels. A set of panels has been first subjected to dynamic loads, considering different values of amplitude, frequency and number of cycles.
The panels have been subjected to a fixed vertical load N and a variable horizontal force T at the same time and the corresponding T-Δ diagram (Δ= horizontal displacement) has been plotted.
The experimental results have also been compared with those obtained using 3-D finite element models [3].

2. SPECIMENS AND INSTRUMENTS

As we have already said the tested specimens represent masonry panels on the scale of one to six. The panels are made of little stones of yellow tuff, which characterizes the old buildings of the Naples area. The stones, whose size is 7.0*5.0*2.5 cm³, are extracted by cutting from the original practical used stones; a cement mortar, with a water/cement ratio equal to 0.5, has also been used.
Two kinds of specimens, respectively with the ratio B/H=1 (squat panels) and B/H=0.67 (slender panels), have been considered in order to study the behaviour of characteristic masonry walls. Fig. 1 shows the two models and their size.

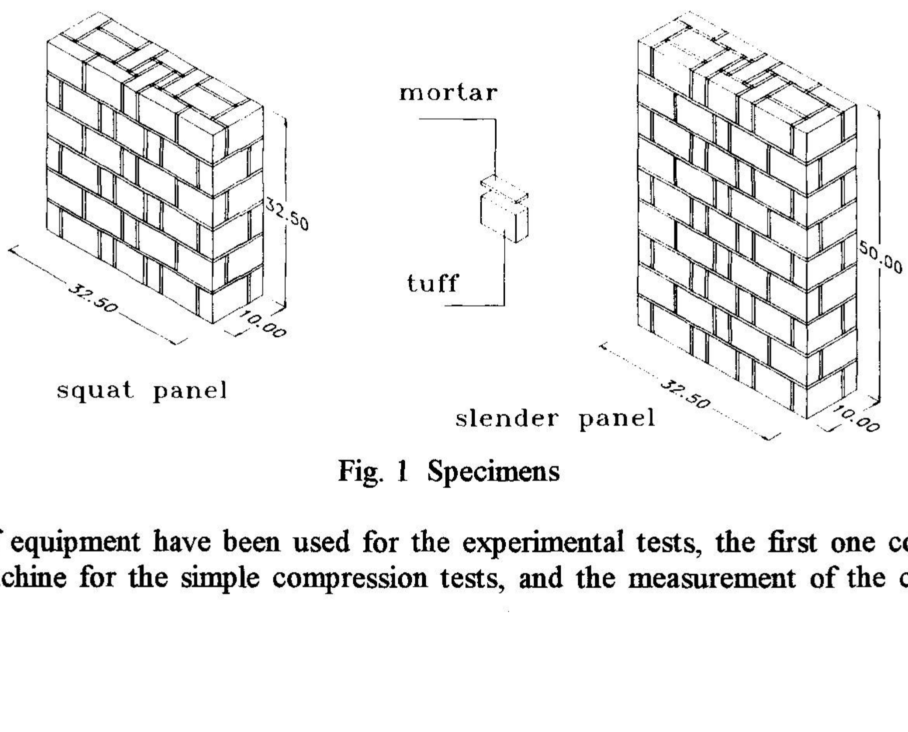


Fig. 1 Specimens

Two kinds of equipment have been used for the experimental tests, the first one consisting of a traditional machine for the simple compression tests, and the measurement of the corresponding

strain. A second equipment has been used to test the panels subject to a fixed axial force and to an horizontal force increasing from zero to the collapse value. This was composed by the following parts:
- three vertical jacks of 30.0 kN with manometers, linked in parallel to an independent oil-dynamic station;
- one hydraulic jack, controlled by an oil-dynamic station, which allows the execution of static and dynamic tests, in which the force law is harmonic with given amplitude and frequency;
- an opposition structure for the jacks, joined to the fixed structures of the laboratory;
- a rigid table on which the models could be constrained;
- a computer which allowed the storing and the drawing in real time of the experimental results.

3. SIMPLE COMPRESSION TESTS

Preliminary compression tests on the basic materials have been carried out. Twelve specimens of tuff and ten specimens of mortar have been tested.
The following values of the characteristic strength have been assumed:

$$f_{bk} = 1.7 \ MPa$$
$$f_{mk} = 13.7 \ MPa$$

respectively for the tuff and the mortar, obtained from the average values using the formula:

$$f_k = f_m - k\,s$$

where s is the mean square error and k=1.64.
Eight slender panels and six squat panels have been subjected to compression test. The results are summarized in Tabs. 1 and 2.

No.	Size (cm)	Weight (Kg)	Collapse load (kN)	Compr. strength (MPa)
1	33.0*9.60*50.0	20.90	160.00	5.05
2	32.5*10.0*50.0	20.20	156.00	4.80
3	32.0*10.1*49.5	20.20	162.00	4.98
4	33.0*10.0*50.0	20.95	170.00	5.15
5	33.0*9.80*49.5	20.00	158.00	4.89
6	32.5*9.70*50.0	20.20	160.00	5.07
7	33.0*10.0*49.5	20.50	156.00	4.73
8	33.0*9.60*50.0	20.50	159.00	5.02

Tab. 1 Results of the tests on the slender specimens

No.	Size (cm)	Weight (Kg)	Collapse load (kN)	Compr. strength (MPa)
1	32.5*10.0*32.5	13.00	147.00	4.52
2	33.0*10.0*32.5	13.10	150.00	4.54
3	32.5*10.0*33.0	13.10	156.00	4.80
4	33.0*9.60*33.0	12.90	154.00	4.86
5	33.0*9.80*33.0	13.20	152.00	4.70
6	32.5*9.80*32.5	12.90	160.00	5.02

Tab. 2 Results of the tests on the squat specimens

From these we can deduce the average collapse stress σ_m, the characteristic strength f_k, the yield axial force N_u and the allowable stress σ_{adm} for both slender and squat panels, reported in Tab. 3.
The experimental values can be compared with those given by the expressions of the Italian Code

[6] and those suggested by other Authors, which relates the masonry strength with the brick and mortar strengths.

	B/H=0.67	B/H=1
σ_m (MPa)	4.96	4.74
f_k (MPa)	4.64	4.29
N_u (kN)	150.93	139.49
σ_{adm} (MPa)	0.93	0.86

Tab. 3 Collapse, characteristic and allowable stresses

We can notice that the strength of the panels is much higher than we would expect. The reason of this behaviour is certainly due to the particular masonry weaving, that we have already called well organized weaving. A well organized masonry panel is quite similar to an homogeneous body in comparison with the traditional masonry, called "a sacco". The loading capacity of a masonry structure depends not only on the strength of the used materials, but also on the weaving.
It is interesting to observe that the panel with the ideal homogenized cross-section would have a collapse axial force:

$$N_u = 161.00\ kN$$

which is coincident with the experimental value.

4. THE BEHAVIOUR UNDER HORIZONTAL LOADS

As we have already said the aim of the project was to investigate the effects of horizontal vibrations on the masonry behaviour subjected to horizontal forces.
Two groups of tests have been carried out: the first one have been carried out on panels that have never been loaded before, the second one on models which have first been subjected to a dynamic load. The collapse tests have been carried out by loading the panels, subject to a fixed vertical load, with a variable horizontal force. Four values of the vertical load have been considered: 30, 45, 52.2 and 60 kN, representing the effects of dead loads and variable vertical loads.
The horizontal load has been increased step by step, from zero to the collapse value, and the corresponding horizontal displacements have been collected. The typical T-Δ diagram have been plotted. During the tests the appearance of the damages (cracks) and the collapse mechanisms have been pointed out.
The vibration effects has been simulated by loading the panels, which were subjected to a uniform vertical load, to an horizontal beating force. Forces variable in the interval [0,5] kN have been applied, with the frequency of 4.2 Hz. Different values of cycles number have been considered: 5000, 10000 and 15000. Forces of lower amplitude or with a lower number of cycles have negligible effects on the load-carrying capacity of the panels. The considered frequency, the maximum value for the used machine, simulated quite well the effects due to ambient vibrations [5]. Seventy models have been tested: thirty with B/H=0.67 and forty with B/H=1.
The two kinds of panels showed different mechanisms of collapse, as we expected. The higher panels reached the collapse by yielding due to the compression at the base corner which is at the opposite side with respect to the horizontal force. The smaller panels showed on the point of collapse the classical diagonal crack due to the shear stress. These behaviours agrees with the theoretical considerations reported in [1] and [2]. According to these considerations we can classify the masonry panels as slender or as squat.
The experimental results are summarized in the following. The diagrams T-Δ (horizontal force against horizontal displacement) are drawn for both dynamically prestressed and not prestressed panels.

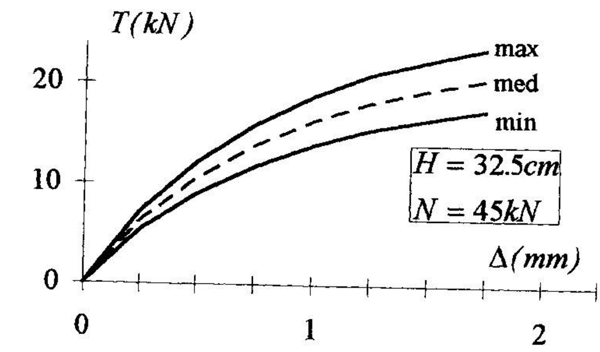


Fig. 2a

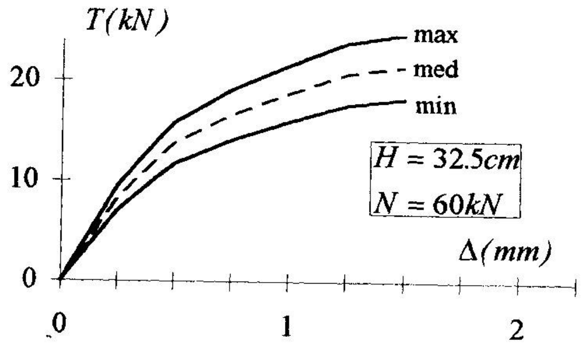


Fig. 2b

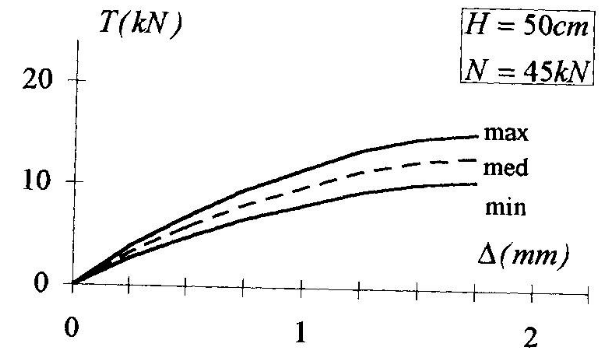


Fig. 3a

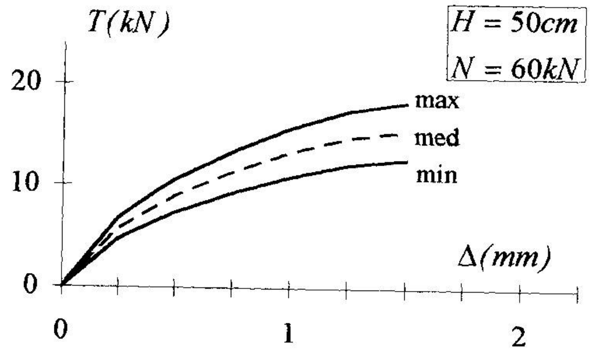


Fig. 3b

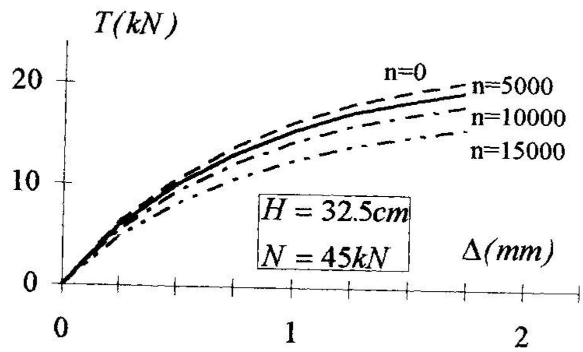


Fig. 4a

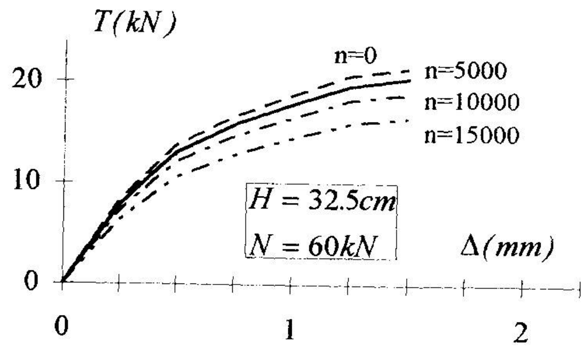


Fig. 4b

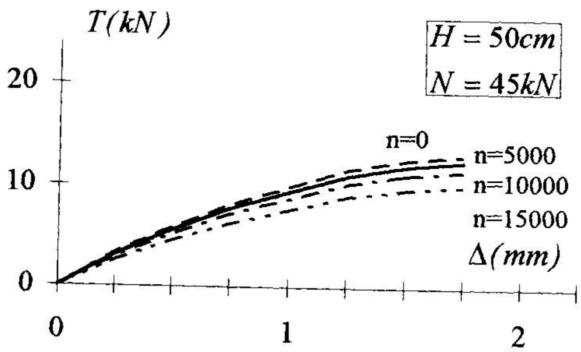


Fig. 5a

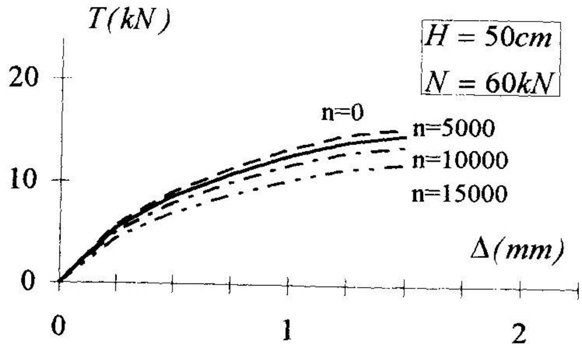


Fig. 5b

Fig. 2 shows the curves relative to the squat models, for two values of N; the similar curves for the slender panels are reported in Fig. 3. The two solid lines define the zone of the plane T-Δ, which contains the experimental curves. The dashed one represents the average curve.
The experimental results appear quite scattered. This cannot be considered an anomaly, but an expected result associated with the real behaviour. In fact, because the masonry is a craftsmanship product, both in the real building and in the models, the strength of the panels cannot be defined by only one curve as we can do for industrial products, but by a region in which we can probably find the structural behaviour.
Figs. 4 and 5 show the average curves relative to the dynamically prestressed specimens; n is the number of cycles. The dynamically prestressed panels behaviour is quite similar to that of the non-prestressed models, as we can see from the analysis of the diagrams. The experimental research has also shown that masonry, characterized by a well arranged weaving and cement mortar, suffer very low reduction of its load-carrying capacity when it is subjected to dynamic loads, especially when the number of cycles n was not greater than 5000. If n>5000 the masonry fabric feels the effects of the dynamic stresses, although the weaving was well arranged, and the curves T-Δ are softer.
The reduction of the collapse load is very low in the case of 5000 cycles, more evident for n>5000 as we can see from the diagrams.

5. CONCLUSIONS

A well arranged weaving of the masonry determines an increasing of the strength under vertical loads, and a noticeable reduction of the vulnerability with regards to horizontal vibration effects. Therefore a well arranged weaving is more suitable than a traditional one to guarantee a longer life to masonry structures. For this reason well arranged masonry buildings may characterize a new builidng age, both in seismic and aseismic areas.

ACKNOWLEDGEMENTS

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Strengthening of the Wooden Covering of the Archaeological Museum in Naples

Consolidation des couvertures en bois du musée d'Archéologie de Naples

Verstärkung der Holzbedeckung des archäologischen Museums, Neapel

Nicola AUGENTI
Professor
University Federico II
Naples, Italy

Nicola Augenti, born 1944, received his civil engineering degree at the University of Naples, Italy, where, he currently teaches Comprehensive Design of Structures. He has carried out studies for the diagnosis and repair of structural damage and worked on the strengthening and restoration of historical buildings.

SUMMARY

The "Gran Salone" of the old building, nowadays seat of the National Museum in Naples, presents three orders of coverings realised in 1735. Damage occurring since the end of the 19th century, necessitated the demolition and the following construction of a new steel roof or lamellar wooden one. The project realised in 1993 saved the existing coverings by introduction of a reversible steel prosthesis and strengthening of masonry and wooden structures. The intervention was carried out without taking down the existing structures and in complete observance of the "Restoring Charter" dictates.

RÉSUMÉ

Le Grand Salon de l'ancien édifice, aujourd'hui siège du Musée National d'Archéologie de Naples, présente trois suites de couvertures en bois réalisées en 1735. Les dommages existants avaient entraîné, dès la fin du 19e siècle, la démolition et la construction d'un nouveau toit en acier ou en bois lamellaire. Le projet réalisé en 1993 a permis de sauver les couvertures existantes par l'introduction de prothèses réversibles en acier et la consolidation des structures de maçonnerie et de celles en bois. L'intervention a été exécutée sans démolir les structures existantes et en plein respect des principes de la "Charte de la Restauration".

ZUSAMMENFASSUNG

Der grosse Salon des alten Gebäudes, der heute die Stelle des nationalen archäologischen Museum von Neapel ist, zeigt drei Holzbedeckungsreihen, die 1735 realisiert wurden. Die bestehenden Zerrüttungen hatten, seit dem Ende des neunzehnten Jahrhunderts, den Abbruch und den Aufbau eines neuen Daches aus Stahl oder aus Lamellenholz bedingt. Das 1993 durchgeführte Projekt hat die bestehenden Bedeckungen durch die Einführung der reversiblen Stahlprothesen und die Verstärkung der Holz- und Mauerstrukturen zu bewahren erlaubt. Der Eingriff ist ausgeführt worden, ohne die bestehenden Strukturen abzubauen und unter Befolgung der Vorschriften der "Charta der Restaurierung".

1. SUBJECT OF THE INTERVENTION

The National Archeological Museum in Naples occupies a building founded in 1582 by the Naples Viceroy Don Pedro Giron. The most representative part of the monument is formed of a stately room of more than 20.000 cu.m. called "Gran Salone", whose roof was rebuilt in 1735 by arch. Giovan Antonio Medrano with three wooden covering orders .

Fig. 1 Outside view of "Gran Salone" covering.

The first upper order, represented by a roof realized through Palladian chestnut trusses with spans of about 23 m. The second intermediate one is formed by another roof of similar wooden trusses, alternate to the first ones, having impost height and pitches inclination smaller than that ones substaining the upper roof. The third lower one is formed of a depressed wooden vault with stiffening ribs hanged to the trusses, whose intrados is painted with valuable frescoes (Fig. 2) .
In the original project these structures had different functions: the upper roof constitued the covering, while the second trusses order supported the depressed vault below by suspensions. After more than two and half centuries, damages caused by time and men have deeply changed the structural behavior.
This covering has a great importance in the Technique of Construction history. It represents a rare example of great span wooden structure and after 250 years preserves, almost unchanged, its original aspect and technology.

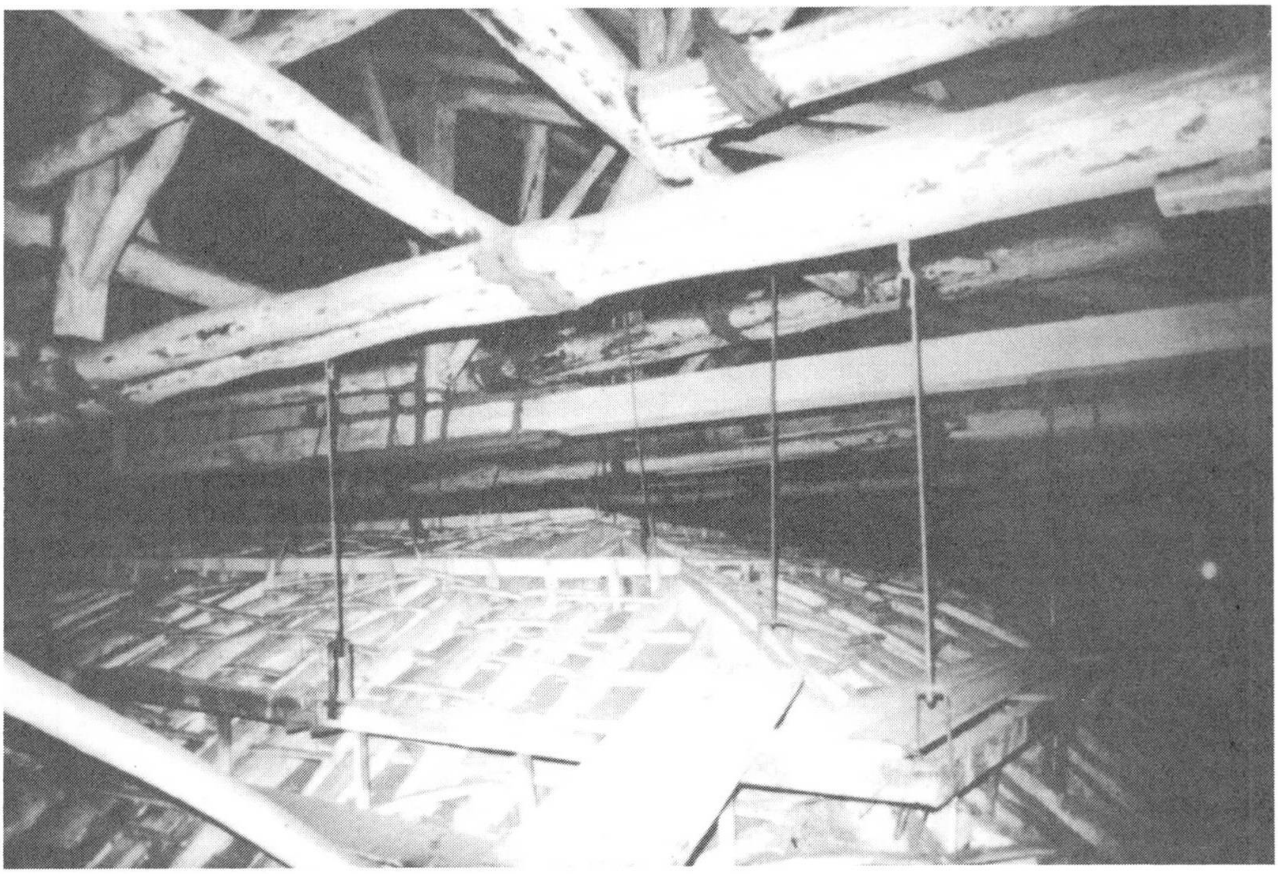

Fig. 2 Inner structure of wooden covering: lower trusses and depressed wooden vault with stiffening ribs before intervention.

2. DIAGNOSIS OF THE DAMAGES

The difficult choice between preserving existing structures and their replacement engaged many engineers since the end of the last century. For the roof of the "Gran Salone" a long research allowed to identify the most important diseases: great deformations in the vault, an elongation of 15 cm in the tie beams, partly disjointed knots and greatly deformed elements in the trusses, significant settlements of the bearing walls. A finite element analysis of the structures demonstrated that the above damages are partly due to the seismic events occurred during the past two centuries and half, and partly to some inadequate past interventions. Particularly, studying the original covering system and the next transformations, the author came to the conclusion that the horizontal differential movement, happened during the seismic oscillations between the main walls where trusses lean, caused most of the slidings between the wooden tie beams (which were already subject to the dead load strains and to relaxation of the joints); these slidings caused a lot of the observed damages.
In 1911, then, the vault was overhanged to the upper trusses to replace some of the lower tie beams. In this way, both the upper trusses and the lower ones were damaged.

3. INTERVENTIONS OF STRENGTHENING

The design solution, proposed by the author, and accepted by the Italian Ministry of Works which provides its execution, is based on two fundamental principles: to assure the safety of people and of the archeological treasures, and to warrant the conservative recovery of the existing covering complex.
The intervention philosophy consists in making the three structures statically independent. Through the proposed strengthening interventions, the two trusses orders could separately support their own weight and the overloads strictly pertinent to them. On the contrary the vault structure was not even able to substain itself. Neither it was possible to retrofit the existing trusses and to secure the vault load to then, since it would have upset their original organism. To integrate the vault self-supporting capacity, a spatial steel structure (prosthesis) endowed of adequate resistance and stiffness, but little visible in the structural contest, has been introduced (Fig. 3) . It was designed so as to be assemblable on site and compatible with the existing wooden structures. This avoided the demolition of the existing structures which would have disturbed the equilibrium and depreciated the historical value of the covering.

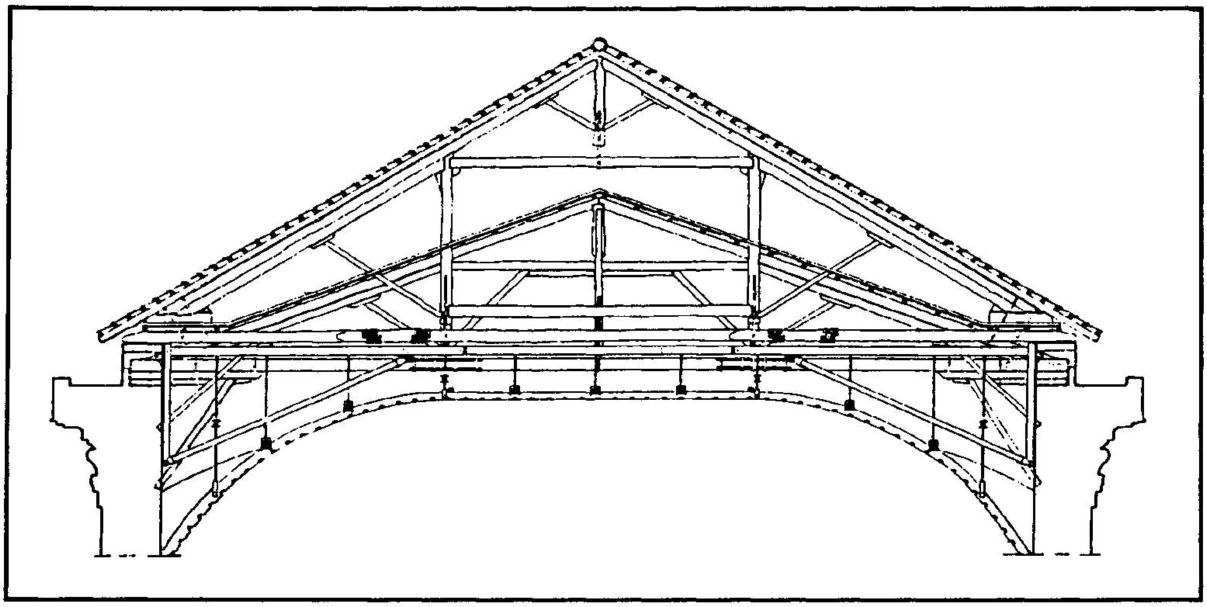

Fig. 3 Covering section after intervention: in evidence (down from above) upper trusses, lower trusses, steel prosthesis and the wooden vault now suspended to them.

On the other hand, it wouldn't be possible to carry out a shoring underneath which would have damaged the frescoes and the stuccoes of the vault. Anyway every other intervention had higer cost. The project solution proposed takes in two different interventions:
- strengthening and static improvement of the wooden and masonry structures;
- realization of the prosthesis sustaining the vault and connected structures.

At first it needed to confer to the perimeter walls of the "Gran Salone" the necessary strength to stand both vertical and horizontal actions transmitted by wooden existing structures and steel prosthesis. To this end, the strengthening of the whole western and eastern masonry faces was carried out by injecting cement mixtures with volumetric stability to improve compression strength.

Afterwards, a reinforced concrete beam was made, at intervals, on the top of the main walls but under the existing trusses to join the walls and support the existing lower trusses and the new steel prosthesis.
The wooden trusses were reinforced, temporarly, by windbracing placed in the vertical plane of the king posts, in the pitch planes and in some horizontal planes of the tie beams.
Eighteen space beams, realized with elements compatible for weigth and dimension with the existing wooden structures, made up the new supporting structures of the vault.
Every prosthesis is made up by two lateral beams (composed with 2 NPU240 and 2 O 159/4) and a central one (composed with 2 NPU240) . At first, they were calculated in exercise and breaking conditions under the action of permanent loads, thermic variations and seismic actions. Afterwards, a prosthesis prototype, assembled in the "Gran Salone" temporarly, was subject to a load test .

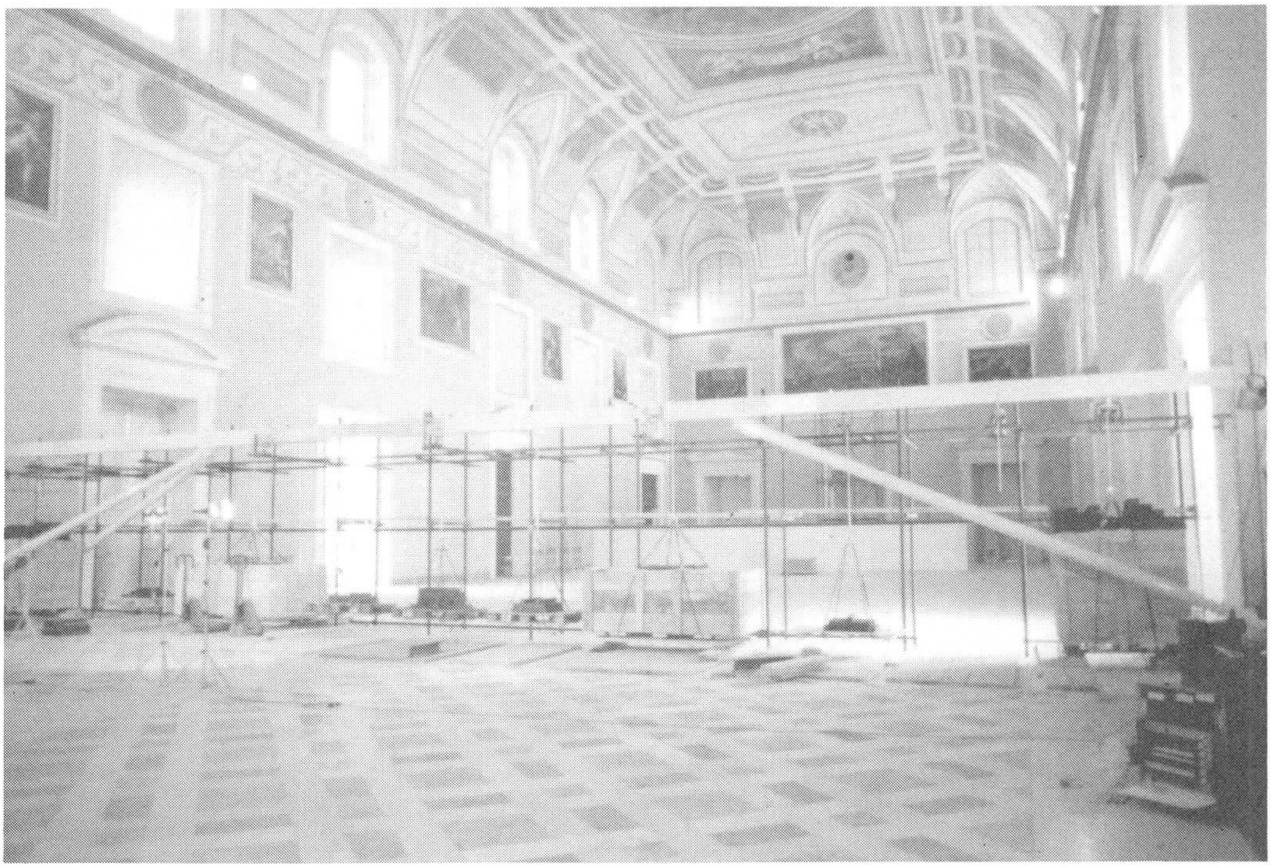

Fig. 4 Inside view of "Gran Salone" where a prosthesis prototype has been subjected to a load test.

These structures were assembled joining the costraint systems (relized with stainless steel) to the new reinforced concrete beam and to the existing masonry and, subsequently, assembling every prosthesis with pieces lifted and transported one by one . After the assemblage of all the prosthesis, the new suspensions were put under tension checking the painted vault movements by monitoring its strains . In this way, it was possible to eliminate the old joints between wooden trusses and vault. The reinforcement of the existing wooden structures has been obtained by completely restoring the disjointed connections in the trusses, replacing the broken elements, adding braces in the horizontal, vertical and pitch planes, disinfesting all the trusses by wooden parasites.

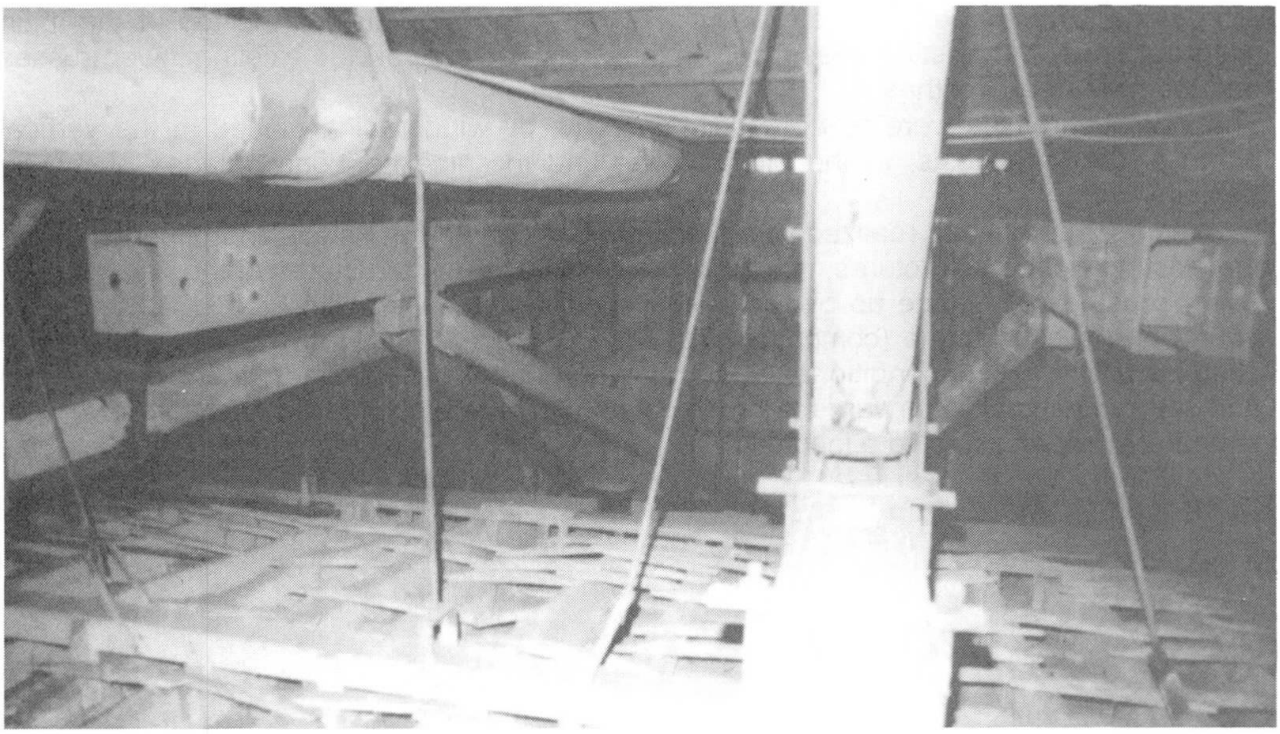

Fig. 5 Assemblage stages of prosthesis lateral beams.

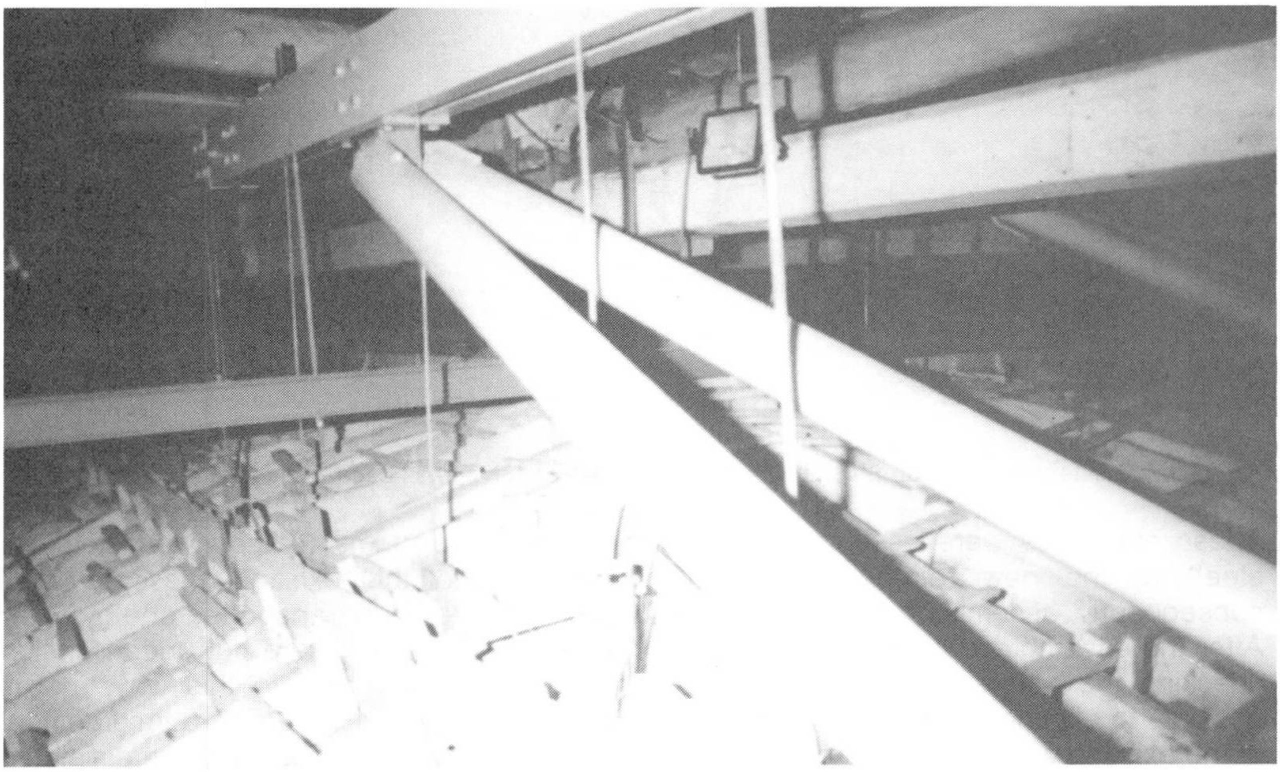

Fig. 6 New suspensions to steel prosthesis of the wooden vault after intervention.

Repair of the Inner Shell of Natural Draft Cooling Towers

Assainissement de la surface intérieure des tours de refroidissement

Sanierung der Innenschalen von Naturkühltürmen

Bernt LANGENEKE
Dr. Ing.
RWE Energie AG
Essen, Germany

Bernt Langeneke, geboren 1943, studierte Bauingenieurwesen an der TH Darmstadt und Wirtschaftswissenschaften an der RWTH Aachen, wo er auch promovierte. 1975 begann er seine Tätigkeit bei der Bauabt. der RWE Energie AG, wo er heute dessen Bereich Bau leitet.

Dieter BUSCH
Dipl.-Ing.
RWE Energie AG
Essen, Germany

Dieter Busch, geboren 1951, studierte Bauingenieurwesen an der Ruhr Universität Bochum. Er arbeitete 4 Jahre in einer Baufirma und 3 Jahre an einer Universität, bevor er zur Bauabteilung der RWE Energie wechselte. Dort leitet er die Abteilung Sonderingenieurbau.

SUMMARY

First signs of degradation on the inner shells of RWE's natural draft cooling towers appeared in 1978. Some of the concrete surfaces revealed a high degree of scaling, resembling exposed aggregate concrete, and in some areas corroded reinforcement bars had broken off large pieces of concrete. Extensive studies of the towers were undertaken between 1978 and 1982 on the causes of the damage and on rehabilitation options. The resulting program and procedures have become well recognised for cooling tower upgrades and are being applied by many other cooling tower operators.

RÉSUMÉ

Les premiers dégâts aux parois internes des tours de refroidissement sont apparus en 1978. La surface en béton a été attaquée et des phénomènes de corrosion ont provoqué un effet d'écaillage important produisant une surface similaire à celle du béton délavé. Entre 1978 et 1982, de vastes analyses portant sur la cause des dégâts, ainsi que sur les possibilités d'assainissement, ont été conduites. Le programme et la méthode d'assainissement qui en ont résulté, sont devenus une référence qui est appliquée maintenant à l'échelle internationale dans le domaine de l'assainissement des tours de refroidissement.

ZUSAMMENFASSUNG

Erste Schäden an den Innenschalen von Naturkühltürmen des RWE traten 1978 auf. Die Betonoberfläche sandete stark ab, partiell ergab sich eine waschbetonähnliche Oberflächenstruktur, und korrodierte Bewehrungseisen sprengten Betonflächen ab. An Kühltürmen wurden zwischen 1978 und 1982 umfangreiche Untersuchungen über Schadensursache und Sanierungsmöglichkeiten durchgeführt. In Labortests und an Probeflächen in einem Kühlturm wurden die von verschiedenen Herstellern für diesen besonderen Anwendungsfall vorgeschlagenen Materialien erprobt. Das daraus entstandene Programm und die Vorgehensweise haben sich zum Standard der Kühlturmertüchtigung entwickelt und werden von vielen anderen Kühlturmbetreibern angewendet.

1. INTRODUCTION

Engineers always try to build concrete structures in an economical way by using as little concrete and steel as possible. In Europe, throughout the 70s, the use of newly developed calculation systems led to thinner and thinner steelreinforced concrete shells. A good example for this development are the shells of natural draft cooling towers. These constructions may be as high as 180 m (Chivaux, France), with a wall thickness of sometimes down to 16 cm. Considering a layer of crossed reinforcement on the outside and inside there is not much space left for the concrete coverage and the concrete itself.

The inner shells of RWE Energie's natural draft cooling towers built in the 70s showed first signs of degradation in 1978. The power station personnel detected huge amounts of sand filling the water basins. When searching for the cause, it was revealed that the surface of the inner side of the cooling tower shells was damaged in different ways. Sand particles were attached to the surface, corroded reinforcement became visible and great areas of coarse aggregates formed the surface, which implied that the surface of the shell as it had been after construction had deteriorated during the course of operation.

2. INSPECTION AND TESTING

2.1 Examining the concrete

The first step following the detection of the damage was a careful inspection of the cooling towers. The strength of the surface of the concrete was examined by cutting it with a sharp knife and measuring the depth of the cut. In some areas the knife cut several millimeters into the concrete, while it was impossible to perform cuts on normal concrete. The compressive strength of the concrete was tested and found to be equivalent to the design values. The amount of corroded reinforcement was counted and added up to several hundred meters.

However, the most important aspect investigated was the carbonation depth in relation to the concrete coverage of the reinforcement. The results were even worse than expected. The concrete coverage designed for 1.5 cm, in large areas appeared to be only between 0.5 cm and 1.0 cm, while the carbonation had already reached a depth of more than 1.5 cm in some areas and 1.0 cm on an average. [1]

This made a repair of the concrete shells a precondition for further operation.

2.2 Reasons for the damage

After having obtained these results, the reasons for the degradation were examined. The cooling towers inspected had all been built with the help of a climbing scaffolding, concreting every day on an average. So every ring of concrete had its own environmental influences throughout construction, i.e. rain and wind. Moreover, the treatment of the surface after having removed the scaffolding varied depending on how good it was done. Not enough attention had been paid to the concrete coverage of the reinforcement and to the sealing of the concrete surface. Since cooling tower shells are exposed to strong winds and great changes of temperature, small cracks can always be found in the surfaces.
In addition to the above-mentioned, operation also constituted an unexpected attack. During operation, treated water ran down the surface and washed out the cementitious matter. Measurements showed that the pH value of this water was at times as low as 4. Thus, this may be regarded as a chemical attack to the surface and explains the loosening of the surface and the wash-out effect. The water and the air accounted for the high carbonation of the concrete surface which, compared to today's standards, had undergone inadequate curing.

Moreover, effects of microbiology with organisms living on the surface and producing acid were revealed, as well as frost effects in the lower part. However, both proved to be of no great influence compared to the others. [2]

2.3 Materials testing

A main aspect in choosing the appropriate kind of repair system is the time available. Most of the cooling towers are connected to base-load power stations which implies that they are only shut down for six weeks every three years. Therefore, upgrading has to fit into this six-week maintenance period.

A wide range of concrete coating materials were available for the upgrading process. But as stated above, there were certain requirements for the material to be used. After discussing the subjects with experts from universities and production companies, a laboratory test series was set up for testing proposed materials with regard to

- resistance to CO_2, S_d (CO_2) > 50 m
- resistance to H_2O vapor, S_d (CO_2) > 10 m
- changing temperatures
- swelling behavior
- adhesive strength
- reapplying layers of coating

The materials showing a good results were then applied as test areas to the concrete surface inside one of the cooling towers. After all the tests, two materials of different companies based on equivalent materials proved to be the best. These are specially designed epoxy-resin coatings.

3. UPGRADING MEASURES

3.1 Access equipment

For the treatment of the concrete surface, access to all areas of the cooling tower shell must be possible. This is achieved by travelling working platforms fixed with steel cables suspended from the cornice. The basic equipment is predominantly used for the maintenance of the facades of huge buildings. In order to use the system in cooling towers it was necessary to develop travelling catheads for the cooling tower cornice which are remote-controlled so that they are able to travel horizontally on demand. The up and down movements are controlled by winches installed on the platforms. By using this equipment, it is possible to reach every spot on the shell.
Due to the hyperbolic shape of the shell it is necessary to prestress the cables since otherwise the lower part of the shell would be out of reach. On the other hand, it is necessary to install rolling spacers in the upper part to prevent damage caused by the ropes and to the ropes moving on the surface.

All the equipment, particularly the working platforms, have to be examined, tested and approved by the German labour protection authorities. There has been no major accident during the upgrading of cooling towers throughout the last 15 years.

3.2 Surface treatment

For the upgrading of concrete, surface treatment is most important. Due to the limited time available it was not possible to do the blasting of the concrete by hand either with water or sand. Therefore, the company, which secured the order from RWE Energie, built unmanned high-pressure water-jet cleaning devices. The moving construction is similar to the one mentioned above.

The construction consists of a steel or aluminum structure in which three of four rotors are installed. These rotors have a diameter of about 60 cm and are mounted in such a way that the areas reached by each one overlap. Inside each of the rotors are four water-jet nozzles which are slightly tilted, thus rotating by the water pressure alone. The pressure is adjustable up to 800 bar, while the normal pressure required is about 400 bar. The water-jet nozzles clean an area with a width of 1.5 m and travel at a speed of 9 m per minute. Consequently, they are able to prepare an area of about 800 m² in an hour. For an area of 26,000 m² it takes about 5 days, including all interruptions and adjustments.

3.3 Upgrading the concrete

The concrete treated according to this method is examined carefully and the adhesive strength is tested. It has to be 1.5 N/mm² on average. Afterwards, the reinforcement bars as well as possible weak concrete areas which have been detected are sandblasted by hand. Before being double coated with a thickness of 1,000 μm they have to be cleaned according to SA 2 1/2 of the DIN standard. In general, there is no reprofiling of the reinforcement with mortar due to the lack of time. The average amount of reinforcement to be treated is about 3,000 m with a maximum of up to 5,500 m. The treatment of the reinforcement is the most time-consuming work.

The coating is applied in two layers by using airless spraying systems. While the primer is an unpigmented surface-penetrating system, the top coat is of gray colour. It has to be applied to a thickness of 200 μm measured after drying. Therefore, an amount of 500 gr/m² for the primer and 600 gr/m² for the final coat is used.

4. COOLING TOWERS WITH CLEANED FLUE-GAS DISCHARGE.

4.1 Additional attack

During the upgrading of the damaged cooling tower in the above-mentioned way, a new environmental law was released in Germany. For meeting its requirements, desulphurisation plants had to be built for almost every power station running on lignite coal. After careful investigations, the general management of RWE Energie AG decided to lead the cleaned flue gases into the cooling towers and to discharge them together with the vapor. The reasons for this decision were, besides environmental reasons, i.e. a lower impact on the surrounding, that this method would not require the reheating of the gases cooled down by the washing procedure as well as the building of new stacks. [3]

Consequently, the coating systems applied as well as new ones underwent a new series of laboratory testing procedures. The temperature and the acid content were the important factors for these tests which proved that the materials already applied remained to be the best choice. The retrofitted cooling towers for flue gases are also coated with epoxy resin, but at least a total of 1.2 kg/m² are used which amounts to 350 μm of dry thickness.

4.2 Outside

Since there now was an acid content in the vapor which got in touch with the outside of the shell, a protection had to be created for the outside, too. As for the coating, an acrylic coating was chosen in order to let remaining moisture out of the concrete. Due to the fact that it was possible to do all the work while the cooling tower was in operation, time was not that important anymore. Therefore, all the reinforcement bars were reprofiled before coating the complete concrete surface.

4.3 Repairing the coating

Since some of the cooling towers which are used for the discharge of the cleaned flue gases had not received the three layers of coating with the necessary thickness, some repair work meanwhile had to be carried out on the coating. Thus, new methods of surface treatment were required. Since it is not possible to remove the remaining and destroyed coating by high-pressure water without causing further substantial damage to the concrete, sand blasting was the only viable solution. However, due to all the plastic fillings, the sand and removed coating and concrete could no longer be dropped to the basin.
In order to save time and money an automatic sand-blasting device, similar to the water system, was developed. In addition, a screen consisting of brooms was attached to the sand-blasting device, and the dust and sand were collected with a type of oversized vacuum cleaner. After some testing periods, two different methods emerged which work almost dustfree so that they fulfill the requirements.

5. RESULTS AND OUTLOOK

Throughout the last 15 years, 18 cooling towers have been upgraded by RWE Energie AG. The coatings on the cooling towers which are not involved with the discharge of cleaned flue gases are in good condition, even after 15 years of operation. The others, with a coating that complies with the quality assurance system developed in the past years, are also in a good condition which proves the system's worth.

The above-mentioned method of upgrading cooling tower shells has been used in about 30 other cooling towers in Israel, the Czech Republic, Slovenia, France, Poland, Belgium and Spain, to mention a few countries.

New cooling towers which are to be built and which must cope with the discharge of flue gases - as planned in Germany - should have a concrete design which no longer requires an additional protective coating.

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Strengthening Techniques Using Additional Reinforcement

Techniques de renforcement par armature secondaire

Verstärken durch zusätzliche Bewehrung

György IVÁNYI
Professor
Essen University
Essen, Germany

György Iványi received his engineering degree from Budapest Univ. in 1962, and his Ph.D. from Braunschweig Univ. in 1967. Since 1978 he has been Prof. for Structural Concrete at Essen University. He authored several papers on prestressed concrete and strengthening of structures.

Wilhelm BUSCHMEYER
Dr.-Ing.
Essen University
Essen, Germany

Wilhelm Buschmeyer received his eng. degree from Braunschweig University in 1976, and his Ph.D. in 1983, from Essen University. His research interests include partially prestressed concrete with and without bond and strengthening of concrete structures.

SUMMARY

A new technique for strengthening reinforced or prestressed concrete structural elements, using additional reinforcement in grooves, embedded into shotcrete or high slump con-crete with fine coarse aggregate, was developed. In this paper, details of design and construction of the new technique are compared with other strengthening methods. Several applications to bridge superstructures are presented.

RÉSUMÉ

Une nouvelle technique pour le renforcement de structures en béton, ayant pour principe le renforcement de la zone de traction, a été développé. À cette fin, des fentes ou des rainures sont faites dans le matériau et, après y avoir mis l'armature secondaire, elles sont remplies de béton projeté ou, dans le cas de surfaces horizontales, de béton à grains fins de consistance fluide. Les critères d'utilisation ainsi que les applications de la nouvelle technique sont présentés et comparés avec les méthodes existantes. Plusieurs exemples pour le renforcement des structures de ponts sont également présentés.

ZUSAMMENFASSUNG

Für die Verstärkung von Betonbauteilen durch Ergänzung der Zugzone wurde eine neue Technik entwickelt, die es gestattet, die Zusatzbewehrung im tragenden Betonquerschnitt unterzubringen. Hierzu werden Schlitze oder Nuten durch Hochdruckwasserstrahl hergestellt und nach Einlegen der Zusatzbewehrung durch Spritzbeton bzw. bei waagerecht liegenden Flächen durch einen fliessfähigen Feinkornbeton gefüllt. Auslegungskriterien und Anwendungen der neuen Technik werden vorgestellt, mit den bekannten verglichen und Beispiele für Brückenverstärkungen gezeigt.

1. INTRODUCTION

The performance of concrete structures with respect to load bearing capacity, serviceability and durability conditions or fatigue safety level can be improved by strengthening them. In general, the following methods are available for this purpose: increase of the cross sectional resistance by additional concrete parts, reinforcement or prestressing, change of the existing structural system to a more favourable one and finally by textural injections using fine cement grouts. A systematic presentation of various strengthening methods was given elsewhere [1]. In this paper special aspects of strengthening techniques using additional reinforcement are reported and a new technique shown.

2. BASIC REQUIREMENTS FOR ADDITIONAL REINFORCEMENT

In order to obtain an efficient strengthening of a concrete structural element, the chosen technique must guarantee a high quality bond of the additional reinforcement to the existing concrete.In particular, an effective bond is necessary in order to ensure the following criteria:

- the load bearing capacity can be calculated as the sum of those of the existing and the additional reinforcement (Fig. 1);
- the cracking behavior of the strengthened concrete member shows no differencies to that of the usual one (Fig. 2);
- the necessary anchorage lengths are as usual.

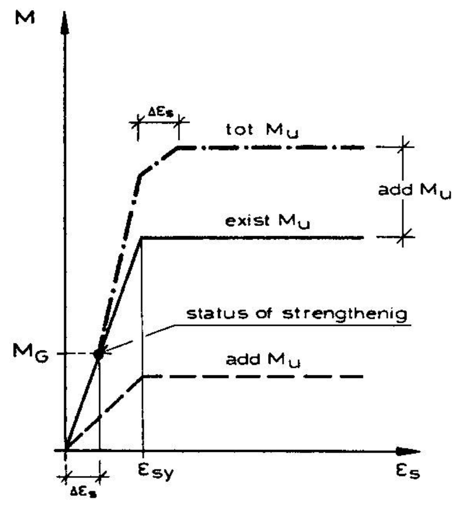

Fig.1 Ultimate of the strengthened concrete member

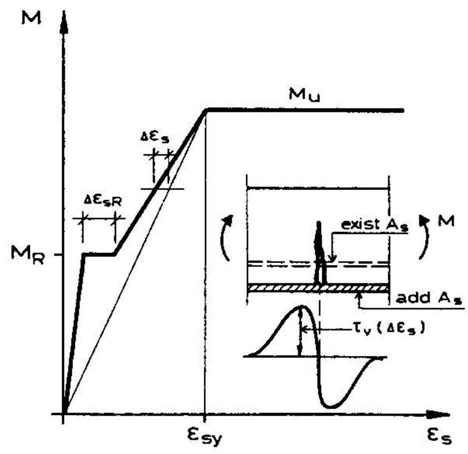

Fig.2 Changes in steel strains during cracking

Therefore, bond strength and bond stiffness of the additional reinforcement should be comparable with the existing one.

3. ADDITIONAL REINFORCEMENT AT THE BOUNDARY LAYER

The well-known technique of strengthening of concrete structures by additional reinforcement using the shotcrete technique was extended for quite some time with a new one using epoxy-bonded glued steel or plastic sheats [2]. In both cases, the strengthening will be realised at the boundary layers of the structural elements, in the concrete cover zone (Fig. 3). Therefore, intensive surface preparation works are necessary, in order to ensure the sufficient bond properties between both parts of the cross section. Because of the fact that the boundary layers of any concrete structural element are generally not the best part of them, the usable shear capacity of the bond area is highly limited. Therefore, reinforcing bars embedded in shotcrete often have to be bonded to the cross section by additional dowels. Glued steel sheats can also be applied taking into consideration the limited shear capacity of the prepared surface, in particular at the anchorage zones on both sides of cracks formed after gluing and at the edges. Therefore, the maximum thickness of the sheats, which influences directly the possible strengthening ratio of a structural element, have to be oriented to these facts [2].

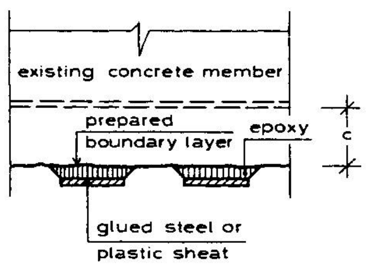

Fig.3 Additional reinforcement at the boundary layer

4. ADDITIONAL REINFORCEMENT IN GROOVES

4.1 Properties and construction

At the early ninties, a new strengthening technique was developed at Essen University in cooperation with a German construction company, using additional reinforcement in grooves, embedded into shotcrete or in case of non-existing shotcreting conditions into high slump concrete with fine coarse aggregate [3, 4].

The grooves are mechanically prepared by high pressure water jets allowing only minimum tolerances of the designed geometry. The additional reinforcement will be arranged in the sound part of concrete cross section (Fig. 4) and the bond characteristics of the existing concrete can fully be used. Moreover, the geometry of the grooves and the spacing of them can than be actually chosen taking into consideration the required bond characteristics for the designed bar diameters and construction aspects. These characteristics show very clearly that the new strengthening technique is highly advantageous in comparision with the known ones. The strengthening quality of rebars in grooves have been confirmed also experimentally. Test series at Essen University have shown, that strength, deformation and cracking behavior of concrete beams strengthened this way is the same as that of beams with identical reinforcement, constructed in one step.

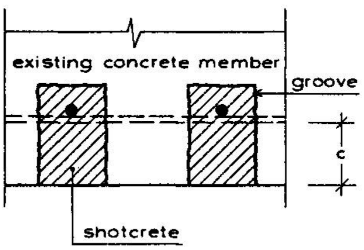

Fig.4 Additional reinforcement in grooves

Practically, the groove geometry is to be defined decisively from the constructional point of view (Fig. 5). For an embedding of good quality, a minimum distance should be chosen between rebar and groove surface of the magnitude of 20 to 30 mm. The tolerances of the groove geometry result from the grading of the coarse aggregate of the existing concrete. Therefore, approximately one half of the mean size of coarse aggregate has to be taken into consideration as a realistic measure of tolerance. Between two grooves, a minimum distance of 60 to 80 mm should exist. Following the geometric requirements, it is clear that the new method cannot be used for the strengthening of 'filigree' structures.

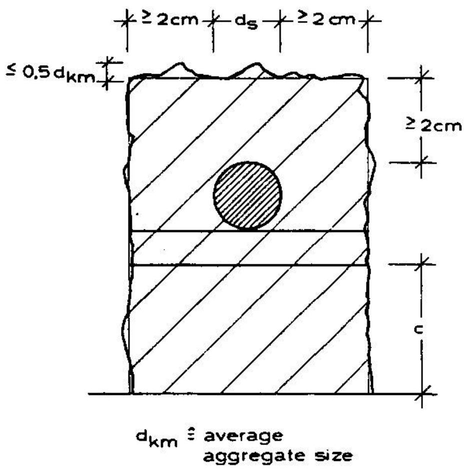

Fig.5 Groove geometry

For the embedding it should be used a concrete-mix with a maximum size of coarse aggregate of 8 to 16 mm. The optimum quality of embedding can only be guaranteed, if the curing conditions are carefully carried out. This means, chemical compounds are not acceptable for this purpose.

4.2 Applications

Until now, three bridge superstructures have been strengthened with the new technique. The first case was a hollow slab pedestrian bridge, strongly cracked in the transition zone between the hollow and solid parts of the superstructure at the bottom side. After the crack injection with fine-cement grout, a completely new longitudinal reinforcement was applied in grooves and embedded into shotcrete (Fig. 6). The second use was the partial strengthening of a deck slab. Because of local defects in the insulation, a great number of the transverse prestressing tendons were heavily corroded and in many cases also failed. The function of these tendons was replaced always by two additional rebars of ∅ 28 mm. A high slump concrete with fine coarse aggregate was used for the embedding of the rebars. The third use was the local strengthening of a prestressed bridge superstructure to remove the resistance of spliced tendons to fatigue fracture in the working zone. The additional longitudinal reinforcement of each joint consisted of two up to eight rebars of ∅ 20 to ∅ 28 mm (Fig. 7).

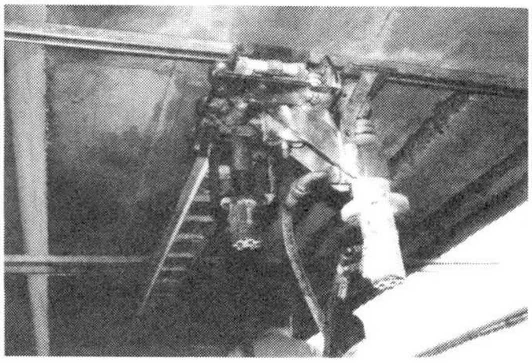

Fig.6 Construction phase of groove preparing

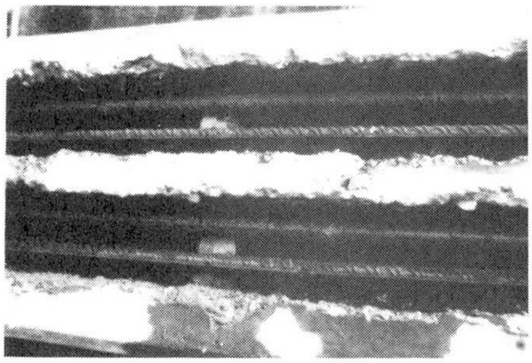

Fig.7 New longitudinal reinforcement

5. CONCLUSIONS

In three applications to date, the advantages of the new strengthening technique could be shown very impressively. The equipment for the cutting procedure of grooves is very efficient and costs for the additional reinforcement per ton are less than by other methods. Numerous further applications are in the design phase.

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Strengthening Masonry Structures with Retro-Reinforcement

Consolidation de constructions de maçonnerie au moyen de barres d'acier

Verstärkung von Mauerwerkbauten durch nachträgliche Bewehrung

Stephen GARRITY
Lecturer
University of Bradford
Bradford, United Kingdom

Stephen Garrity, born 1956, obtained his degrees in civil and structural engineering from Nottingham and Manchester Universities. Following several years working for consulting engineers and a Highway Authority, he took up a senior lectureship at Bolton Institute, moving to Bradford University in 1990 where he specialises in concrete and masonry.

SUMMARY

Retro-reinforcement, a cost-effective method of increasing the tensile strength and ductility of existing masonry using stainless steel reinforcement, is briefly described. Recent experience shows that such reinforcement can be installed with very little disruption to the users of the structure and that it is possible to avoid any noticeable alteration to the original masonry finishes. Although retro-reinforcement has been used mainly to rehabilitate low-rise buildings and arch bridges, it is also likely to be suitable for improving the impact resistance of masonry and as a seismic retro-fit technique.

RÉSUMÉ

Le "rétro-renforcement" est une méthode rentable qui permet d'augmenter la résistance à la traction et la ductilité des maçonneries existantes au moyen de barres en acier inoxydable. Des expériences récentes montrent que ce renforcement peut être installé sans perturber outre mesure les utilisateurs du bâtiment et en évitant toute modification apparente des finitions de la maçonnerie originelle. Quoique le "rétro-renforcement" ait été, jusqu'ici, principalement utilisé dans la rénovation de bâtiments à hauteur limitée et de ponts en arc, il est probable qu'il puisse également améliorer la résistance à la flexion de la maçonnerie, sous l'effet de chocs et servir de technique pour augmenter la résistance aux séismes.

ZUSAMMENFASSUNG

Die nachträgliche Bewehrung ist eine kosteneffektive Methode zur Steigerung der Zugfestigkeit und Duktilität bestehender Mauerwerke durch Edelstahlbewehrung. Neueste Erfahrungen zeigen, dass eine solche Bewehrung bei minimaler Störung der Benutzer eingebaut werden kann und dass es möglich ist, jegliche wahrnehmbare Veränderung der Originalmauerwerkfassaden zu vermeiden. Obwohl die nachträgliche Bewehrung bisher hauptsächlich für die Renovierung niedriger Bauten und Bogenbrücken verwendet wurde, ist es wahrscheinlich, dass sie sich auch für eine Verbesserung des Mauerwerkschlagwiderstandes und als eine seismische Nachrüstungstechnik eignet.

1. INTRODUCTION

Engineers have been using reinforcement to improve the performance of masonry for many centuries. The Romans used iron cramps in molten lead to fasten stone blocks together or to tie stone facing to rougher masonry backing [1]. During the Italian Renaissance (1420-1580), iron tie rods and chains were used to resist the thrust in vaulted masonry construction and iron anchors were used to tie floors to masonry walls. Also, by this time, it had become common practice in many parts of Western Europe to use various iron components to tie masonry blocks together. One of the earliest recorded examples of the use of iron bar reinforcement in a masonry structure was by Soufflot who used a variety of horizontal, vertical and diagonal iron bars to strengthen the portico of Sainte Geneviève (the Panthéon) in Paris between 1770 and 1772 [2]. In the UK, Sir Marc Isambard Brunel used hoop iron to reinforce the caissons for the Wapping to Rotherhithe tunnel beneath the River Thames in London in 1825. The success of this project led Brunel to carry out a programme of testing of reinforced brickwork between 1835 and 1838. Initially he carried out load tests on a series of reinforced brickwork beams; this was followed by the construction of two semi-circular reinforced brick arches cantilevering from a prototype bridge pier. The use of reinforcement in the arches precluded the need for costly centering and falsework. Around the time of Brunel's tests, C.W. Pasley carried out similar tests on reinforced brickwork beams constructed with different mortar types. The results demonstrated improved performance with mortar containing a cementitious binder instead of lime [3]. By the turn of this century, the development of Portland cement and improvements in the performance and production of steel led to reinforced masonry being eclipsed by the emergence of reinforced concrete construction. Nevertheless, reinforced masonry continued to be used, notable examples being the Sidwell Street Methodist Church in Exeter, England and the Church of St.Jean de Montmartre in Paris, designed and built between 1900 and 1906 by the French structural engineer Paul Cottancin [3,4].

Much of the development of reinforced masonry followed the widespread damage to masonry buildings caused by seismic activity. As a result of the earthquake in Lisbon on 1 November 1755, timber members arranged in the form of a cage or *gaiola* were built into masonry walls to improve their seismic performance. A similar internal timber frame was also proposed for masonry buildings in Calabria, Italy following a series of earthquakes in 1783. Iron was suggested as an alternative to timber and it was eventually adopted as the principal reinforcing material in Southern Italy after a series of strong tremors struck the region in 1854 [2]. A great deal of credit for the development of seismically-resistant reinforced masonry construction must go to Sir Alexander Brebner, the Chief Engineer to the Government of India, who, between 1918 and 1923, carried out an extensive experimental study leading to the widespread use of reinforced masonry construction for slab and wall construction in buildings, culverts and bridges. Such structures were found to perform very well in the 1934 Bibar earthquake which prompted the extensive use of reinforced masonry in the reconstruction of Quetta (in what was then Northern India) following the devastating earthquake of 1937 [3,5]. In the USA, in spite of the poor performance of many unreinforced masonry buildings in the 1906 San Francisco earthquake, reinforced masonry only became recognised as an acceptable form of earthquake resistant construction after the 1925 Santa Barbara and 1933 Long Beach quakes and the earlier successes in India. It is now used fairly extensively in many of the seismically active regions of the world, particularly for low to medium rise building construction.

It is clear from the above brief review that many generations of engineers have recognised the potential benefits of building some form of reinforcement into new masonry construction. It should also be possible to strengthen or improve the performance of *existing* plain masonry structures by installing reinforcing bars, retrospectively, in regions where tensile stress is likely to develop or where it is desirable to introduce a measure of ductility. This paper describes a method of installing reinforcement into existing masonry structures hereafter referred to as "***retro-reinforcement***".

2. RETRO-REINFORCEMENT

The different stages of retro-reinforcing an existing masonry structure are summarised as follows:-

a) Any existing covering such as external render, plaster, vegetation or dirt that has accumulated on the surface of the structure is removed to expose the masonry.

b) Grooves or slots up to 10mm thick are cut into the mortar bed joints of the masonry; for repairs to brick masonry walls, the grooves are usually 50 - 70 mm deep. Although the use of a power disc cutter is often adequate for most repair work, in some cases it is necessary to exercise more control over the depth of cut. Equipment capable of achieving this with the added benefits of reduced dust emission and improved operator handling has been developed by Proprietary Reinforcement Engineering of England who are specialists in this type of work.

   In any case, sufficient mortar should remain in place to support the existing bricks or blocks. In some cases, dislodged bricks or stones must be reinstated before continuing with the repair. In some cases, for example when repairing severely deteriorated arches or other spanning members where the reduction in structural integrity caused by the cutting of grooves is a major concern, it is necessary to install the reinforcement in comparatively narrow band widths and to provide temporary support to the adjacent masonry.

c) The interior of the existing masonry is carefully inspected to assess its condition. It may be necessary to grout any large voids present before continuing with the repair; the characteristics of the hardened grout should be compatible with the existing masonry.

d) Dust and debris are removed using compressed air or by washing with clean water.

e) A layer of cementitious grout is then injected into the back of the groove using a manually operated gun; the grout should also be firmly pressed into the groove to ensure maximum compaction. When injecting grout into a groove cut into a soffit, or where the existing masonry contains many voids, the use of low pressure mechanical pointing equipment is recommended.

f) A length of 6mm diameter ribbed stainless steel reinforcing bar is then pushed into the layer of grout. Care is taken to ensure that the bar is fully surrounded by grout. Stainless steel helical rod or flat strips of stainless steel may be used instead of reinforcing bar.

g) A second layer of grout is then injected over the reinforcing bar.

h) Additional bars, rods or strips are inserted, as required, and encapsulated in grout. In a 50-70mm deep groove, it is usual to insert only two bars and the outer 15mm is left free of grout for repointing.

i) The grout is left to cure and the sawn grooves are repointed with mortar to match the existing masonry. In many cases special sands or pigments must be used in the repointing mortar to obtain the required finish.

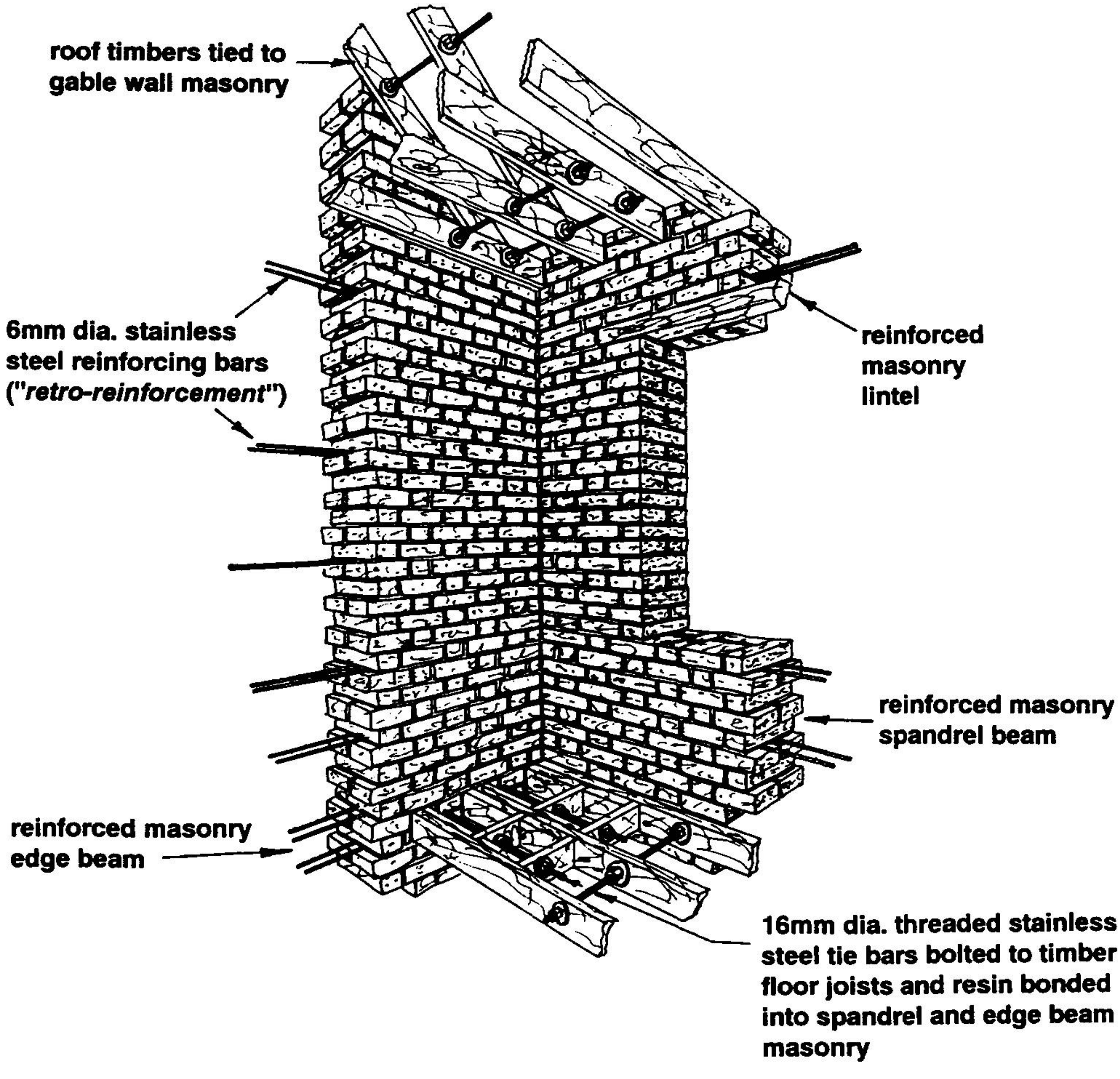

Fig. 1 Typical retro-reinforcement repairs to low-rise masonry construction

To date, retro-reinforcement has been used mainly to repair low-rise masonry buildings; a general example is given in Figure 1. Usually, the steel reinforcement is installed in the bed joints of regularly coursed brick or stone masonry. Although this is the simplest form of retro-reinforcement and is the easiest to disguise, it is possible to install diagonal and vertical bars in masonry structures using a similar approach.

3. DISCUSSION

The principal advantages of retro-reinforcement are that it is both simple and quick to install. Furthermore, the repairs and resulting disruption are kept to a minimum because the engineer can target specific regions of the structure where the reinforcement is needed. In some cases the reinforcement can be installed entirely from the outside of the building and it may only be necessary to evacuate the rooms immediately adjacent to the walls undergoing repair to prevent any imposed loading that might impair the development of full bond between the grout and both the reinforcement and the existing masonry. The noise of the disc cutter and the dust created during groove cutting must also be taken into account when planning repair work of this type.

Another important feature of retro-reinforcement is that it does not cause any noticeable alteration to the original appearance of a structure, particularly if the reinforcement can be installed in the bed joints of the masonry. Also the addition of reinforcement does not significantly increase the self weight of the structure. This can be a major advantage when repairing old and historic buildings built on poor foundations. Although retro-reinforcement is a very versatile technique, it is not a solution to ***all*** masonry repair and strengthening problems. It may not be the most appropriate repair system to use where the mortar joints are very thin; although the bricks or blocks could be cut, it would be difficult to disguise the repair work. Its main uses are likely to be where there is a need to provide or increase continuity in a masonry structure and where greater tensile strength and ductility are required. Typical examples include building structures either damaged by or at risk from seismic activity, blast effects or other forms of impact loading. The technique has also been successful for the repair and strengthening of short span masonry arch bridges.

When designing or specifying any form of repair or strengthening works it is not only very important to correctly diagnose the cause of deterioration or cracking but also to determine the effects of the proposed remedial works on the behaviour of the complete structure. In some cases, retro-reinforcement will only be appropriate if used in conjunction with other remedial work such as pre-grouting of the masonry. Where cracking has been caused by excessive movement of the foundations, it is usually necessary to stabilise the ground or relieve some of the load from the foundations to minimise the risk of further gross deformation.

With retro-reinforcement, as with all refurbishment, it is essential that the work is carried out with care. Even though the technique is very simple and therefore there is reduced scope for error, to ensure maximum composite action it is important to remove dust and debris from the grooves cut into the existing masonry and to accurately batch and mix the grout materials.

4. SUMMARY

Retro-reinforcement has already proved to be an effective and economical means of repairing and strengthening existing low-rise masonry buildings of historical interest [6]. Experience to date shows that such reinforcement can be installed with very little disruption to the users of the building and that it is possible to avoid any noticeable alteration to the original masonry finishes particularly where reinforcement is installed in the bed joints of regularly coursed masonry. The technique also offers considerable potential for the repair and strengthening of masonry arch bridges, seismic retrofitting of low to medium rise masonry buildings and the armouring of masonry structures against impact damage.

5. ACKNOWLEDGEMENT

The University of Bradford and Proprietary Reinforcement Engineering (P.R.E) Limited are jointly developing the retro-reinforcement technique for the repair and strengthening of historic buildings, the rehabilitation of masonry arch bridges, seismic retrofitting of low to medium rise masonry buildings and the armouring of masonry structures against impact damage. The assistance of David Atkins, director of P.R.E. Limited, in preparing this paper is gratefully acknowledged.

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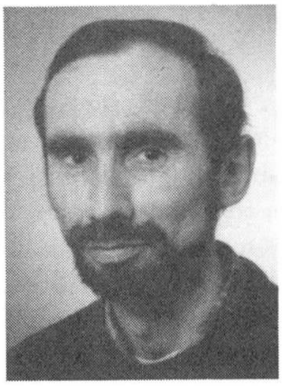
Innovation in Structural Timber Design with Prestressed Timber Joints

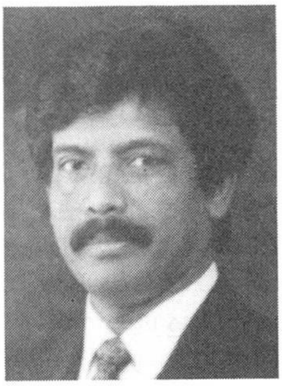
Projet de constructions en bois avec des assemblages précontraints

Erneuerung von Holzkonstruktionen mit verstärkten Holzverbindungen

A.J.M. LEIJTEN
Civil Engineer
Delft Univ. of Technology
Delft, The Netherlands



P. RAGUPATHY
Research Fellow
City University
London, UK



K.S. VIRDI
Professor
City University
London, UK

SUMMARY

A timber joint is presented with a new type of prestressing connector. The timber is locally reinforced with densified veneer wood. The high strength and stiffness of the joint is combined with excellent ductile properties. This enables structural timber to be applied much more efficiently than with traditional joints. It is shown that timber savings of up to 30% can be realised.

RÉSUMÉ

Un nouvel assemblage pour le bois est présenté avec un nouveau type de connecteur précontraint. Le bois est renforcé localement avec du bois lamellé. Une grande résistance et une grande rigidité de l'assemblage sont combinées à une excellente ductilité. Ceci permet à la structure en bois d'être utilisée plus efficacement que lorsqu'elle comporte des assemblages traditionnels. Il en résulte une économie de bois de 30%.

ZUSAMMENFASSUNG

Eine neue Holzverbindung mit einem neuartigen Verbindungsmittel wird vorgestellt. Das Holz ist im Bereich der Verbindungen mit Presssperrholz verstärkt. Die Verbindungen zeigen neben aussergewöhnlich hohen Tragfähigkeiten und Steifigkeiten auch ein exzellentes plastisches Verhalten. Aus diesen Eigenschaften der Verbindung ergeben sich neue Möglichkeiten für wirtschaftliche Holzkonstruktionen. Holzeinsparungen bis zu 30% sind möglich.

1 INTRODUCTION

In many cases timber structures suffer due to the low strength, stiffness and splitting of timber joints. Although in essence the timber joints with mechanical fasteners like bolts and dowels behave in a ductile manner the unpredictable appearence of splitting cracks which initiate failure does not allow designers to take any plasticity or non–linear behaviour into account. In fact, due to splitting, the moment capacity of the joint will never be higher than about 40% of the bending capacity of the timber member. On the other hand, the spacing requirements of the dowel type fasteners in a joint frequently govern the dimensions of the timber members. The application of plastic theory in structural timber design with traditional joints is hindered because of the unreliable cracking. All together, mechanical timber joints limit the efficient use of timber as a structural material. Investigations aimed at improving the reliability of strength and stiffness of joints have been a research topic at the Delft University of Technology for many years.

2 REINFORCED JOINTS WITH DENSIFIED VENEER WOOD

It was Robert Stoeckhart, who first patented the densification of solid wood in Leipzig, Germany, in 1886. Finally in 1922 the Austrian Pfleumer brothers found a more effective method by trial and error. Since then this material is commercially available mainly in Europe. The main thought behind the densification is that the mechanical properties change proportionally with density. In a nutshell, the densification is described as follows. Solid wood or stacks of veneers are heated up to 145°C and compressed, perpendicular to the grain, to 20 MPa. This forces the outer cell wall to become plasticized, allowing the cell to drift and move within the conglomerate of cells so as to close all veins. For poplar, this process is indicated in figure 1. When the fibres are settled, a rapid temperature drop will freez this situation. Only immersion in water for a long time will activate the material memory. The maximum density obtained is about 1380 kg/m^3. Although many wood species can be densified, the highest mechanical properties are obtained with beech. Tension and compression strength values of maximum 300 MPa can be achieved [1]. For practical application a minimum density is usually specified. As densified veneer wood (dvw) consists of cross–layered veneer sheets, the mechanical properties are less grain direction dependent than for wood. No special precautions are required to glue this material to the timber.

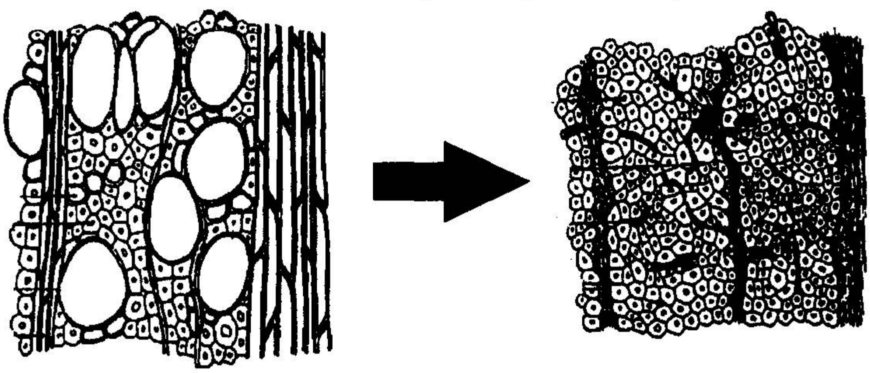


Figure 1: Poplar before and after densification.

The dvw is glued to the surface of the timber members where high concentrated loads are introduced by the fasteners to prevent splitting. The embedding or bearing strength of the dvw is about 130 MPa compared to 20–25 MPa for spruce [1].

3 THE EXPANDING TUBE DOWEL

One of the major problems in timber joints with mechanical fasteners is to eliminate the clearance space. Although tight fitting holes lead to a reduced clearance, the fasteners are in that case hard to drive in. When the holes are slightly unequally spaced, splitting can already be initiated at that stage. Therefore, very precise drilling equipment is required. To overcome this, there are efforts to drill wide holes and to use injection resin as in steel structures [2]. However, this method is rather expensive while quality control is difficult and doubtful.

To solve all these problems, an innovative new idea was developed and tested. Why not fit in a tube as dowel type fastener in wide holes and expand the tube by pressing the tube ends together? A central rod prevents any inward movement of the tube. This method has proved to satisfy all requirements easily. The clearance is completely removed. Due to this expansion procedure, the hole diameter is enlarged and the dvw and timber prestressed, which lead to an increased stiffness at the initial loading stage, see figure 2. On the other hand, the physics of a tube allows very large plastic deformations. The tube material is every cheap; galvanized welded gaspipe Fe360 (according to DIN 2440/ ISO 65). Diameters of 17.2 to 33.7 mm have been used. Details about production requirements are available on request. There are no patents that hinder the application commercially.

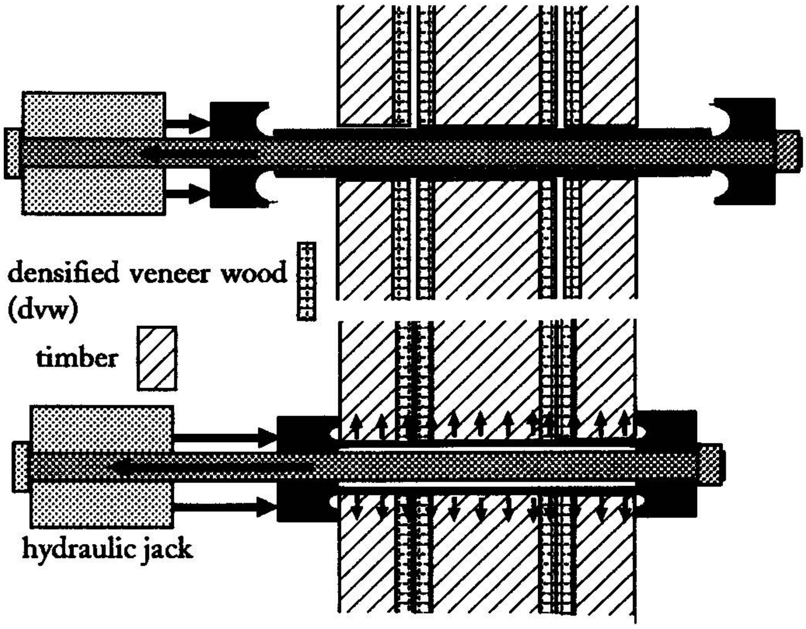


Figure 2: The principle of the prestressing procedure

4 TEST RESULTS

To show the capacity of this joint, portal frame corners were tested and the results are presented in figure 3. The moment–rotation characteristics of two test series are given. The difference is, the spacing of the fasteners. Each joint has four 35 mm diameter tube–dowels at the corner and the dvw thickness is 18 mm density 1300 kg/m^3. At the right hand side of the graph the bending stress in the glued laminated members of 600x110 mm is given as well. The data of test 3.5d187 should not be considered as the joint was tested twice due a computer break down. All tests ended because the stroke length of the jack was reached without any timber failure.

5 FRAME DESIGN EXAMPLES

In what follows, the application of such joints is indicated in a single storey and three–storey timber frame. The main objective is to determine the potential improvements in design that could be achieved by using the reinforced joints, especially in terms of the timber (cost) saving ability and the question of rotation capacity required at the ultimate limit state.

5.1 Program SWANSA

The structural design analysis was carried out using the computer program *SWANSA*, which takes into account material and geometrical nonlinearities, together with nonlinear semi–rigid behaviour of connections. The background theory is explained in more detail in [3]. The structural analysis is initiated with assumed cross–sectional geometry and connection characteristics, for combinations of dead, imposed and wind loads at ultimate (ULS) and serviceability limit state (SLS) conditions.

5.2 Geometry of the structures

The portal frame (Structure 1) considered in this analysis is idealised as shown in figure 4. In the structure as built, the frame consists of two 56 x 550 mm deep members and a single column. For

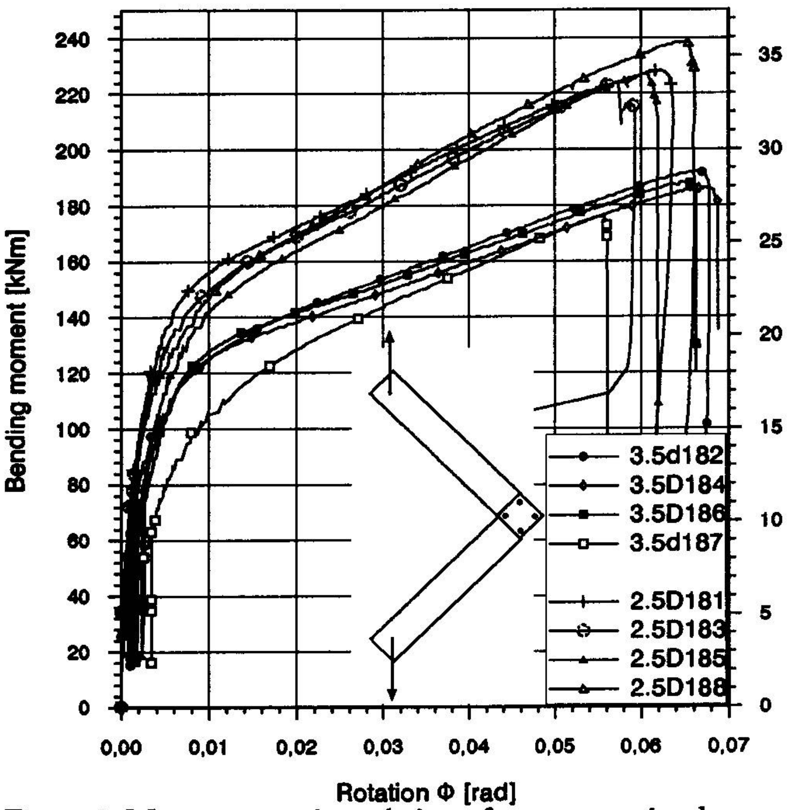


Figure 3: Moment–rotation relation of two tests series dvw reinforced prestressed timber joints, taken from [2].

purposes of present analysis, the beam is idealised as a single member of size 112 x 550 mm. The beam is connected to either column using four tube–dowels of 35 mm diameter set in a square pattern in the connection zone. The columns which are also of uniform cross–section are fixed to the foundation, using a connection detail similar to the beam to column connection at the top of the column. The spacing of the frames is taken as 5 m.

The three–storey frame (Structure 2) is idealised as shown in figure 5. The beams and columns are of uniform cross–section, and are connected to the columns using four 35 mm diameter tubes set in a square pattern in the connection zone. The connection detail is, in fact, identical to that of Structure 1.

5.3 Material Properties

The structures are made of glued laminated timber of strength class GL30, with a characteristic bending strength of 30 MPa. The design bending strength is 21.2 MPa, according to Eurocode 5. The modulus of elasticity is 12000 MPa for SLS and 6800 MPa for the ULS calculation.

5.4 Moment rotation characteristic of the connection

For the calculations with the traditional joints, the stiffness and strength values are used in accordance with EC5. Dowels ranging from 12 to 20 mm in diameter were chosen. The moment rotation characteristic of the traditional joints is linear elastic. The design moment and rotation capacity is given in Table 1. The moment–rotation relations of the reinforced joint, required in the frame analysis with semi–rigid joints, are derived from figure 3. The moment capacity of the beams is 121 kNm for Structure 1 and 130 kNm for Structure 2.

6 LOADING AND LOADING COMBINATIONS

The Permanent load, Variable load and Wind load used in the calculations for both structures are shown in Table 2. The structures were analysed for five load combinations, factored in accordance with Eurocode 1 clauses on the combination of actions, as shown in Table 3 below.

Load type	*Structure 1*	*Structure 2*
Permanent load (DL)	*1.5 kN/m*	*9.36 kN/m*
Variable load (LL)	*2.5 kN/m*	*6.00 kN/m*
Wind load (WL)	*12.9 kN/m*	*1.704 kN/m*

Table 2: Loading

Load Combination	*Limit State*	*Combination Formulae*
1	*Ultimate*	*DLx1.1 + LLx1.5*
2	*Ultimate*	*DLx1.1 + LLx1.5 +WLx1.5x0.6*
3	*Ultimate*	*DLx1.1 + LLx1.5x0.7 +WLx1.5*
4	*Serviceability*	*DLx1.0 + LLx1.0*
5	*Serviceability*	*DLx1.0 + LLx1.0 +WLx1.0*

Table 3: Load Combination factores Eurocode 1

Traditional joints with dowels			
Diam		*Mmax.*	*Rot.max.*
[mm]	*number*	*[kNm]*	*[rad]*
15 mm	*22*	*31.3*	*0.0036*
16 mm	*16*	*33.6*	*0.0040*
20 mm	*13*	*38.3*	*0.0045*

Table 1: Moment rotation data

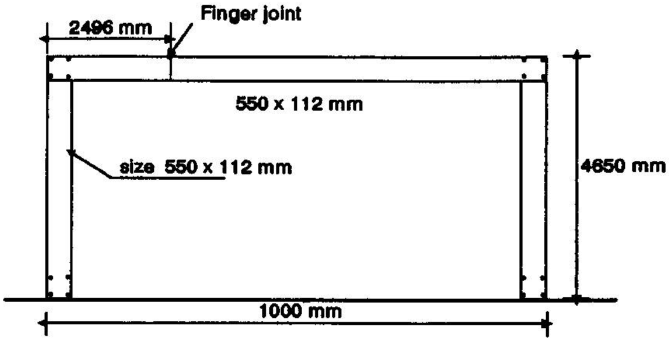


Figure 4: Structure 1

In the ultimate limit state calculations, the load factor on live load is varied, while the other load factors are kept constant. The maximum value of the live load factor obtained from the computations is compared with the code specified load factor. The serviceability calculations are made for the specified load factors. The principal objective of serviceability calculations is to check for limiting deflections. When the live load factor is substantially more than the value of 1.5 required for the ultimate limit state, substantial safety margins are available. For these cases the beam sizes have been reduced until comparable safety margins are obtained as with traditional design.

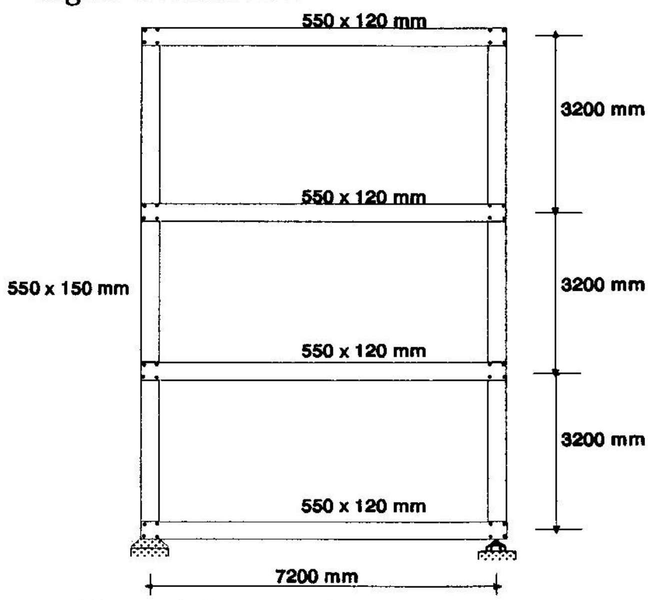


Figure 5: Structure 2

7 RESULTS

In all three ULS load combinations (1–3), the computations indicate failure at the beam midspan for Structures 1 and top storey beam of Structure 2, when the extreme fibre stress reaches the maximum design value of 21.2 MPa. In Table 4a, 4b and 5 the maximum load factors on live load for Structures 1 and 2 are presented for ULS design as well as for the serviceability limit conditions (SLS).

STRUCTURE 1							
Traditional design U.L.S.				Reinforced design U.L.S.			
Member sizes	*num. of dowels*	*Load Comb.*	*LL factor*	*Member sizes*	*num. of dowels*	*Load Comb.*	*L.L. factor*
550x112 mm	*13 of 20 mm*	*1*	*2.38*	*550x112 mm*	*4 tubes 35mm*	*1*	*5.85*
"	*16 of 16 mm*	*1*	*2.00*	"	"	*2*	*6.78*
"	*22 of 12 mm*	*1*	*1.82*	"	"	*3*	*9.52*
				500x100mm	"	*1*	*5.42*
				480x100mm	"	*1*	*4.68*
				450x100mm	"	*1*	*3.94*
				400x90 mm	"	*1*	*2.69*

Table 4a: Overview of Live Load factors for Structure 1, ULS.

Reinforced design SLS

Member sizes	*num. of dowels*	*Load Comb.*	*Vertical defl.*	*Vert.defl. limit*	*Load Comb.*	*Lateral defl.*	*Lat.defl.limit*
550x112 mm	*4 tubes 35mm*	*4*	*8.23mm*	*30mm*	*5*	*5.7mm*	*8.75mm*
500x100mm	"				*5*	*8.0mm*	*8.75mm*
480x100mm	"				*5*	*10.0mm*	*8.75mm*
Vertical deflection limit is 0.003xspan; Lataral deflection limit is h/500							

Table 4b: Overview of Live Load factors for Structure 1, SLS.

STRUCTURE 2							
Reinforced design ULS							
sizes	*num. of dowels*	*Load Comb.*	*LL factor*				
550x120 mm	*4 tubes 35mm*	*1*	*3.99*				
550x120 mm	"	*2*	*3.92*				
550x120 mm	"	*3*	*5.51*				
Reinforced design SLS							
sizes	*num,of dowels*	*Load Comb.*	*Vertical defl.*	*Vert.defl. limit*	*Load Comb.*	*Lateral. defl.*	*Lat.defl. limit*
550x120 mm	*4 tubes 35mm*	*4*	*11mm*	*21mm*	*5*	*14.7mm*	*19.2mm*
500x100mm	"	–	–	–	*5*	*20.9mm*	*19.2mm*

Table 5: Overview of Live Load factors for Structure 2.

The maximum value of the joint rotation obtained anywhere in the structures is 0.02543 radians. The maximum rotation in Structure 2 is about 10% higher than that for Structure 1, but is again well within the maximum rotation deduced from tests for this type of joint. Without any reduction of safety the dimensions of the timber beams can be reduced from 550x120mm in the traditional design to even 480x120mm for the prestressed dvw reinforced joint. About 30% of timber is saved which more than balances the production cost of the reinforced joint and is therefore of commercial interest.

8 CONCLUSIONS

It has been shown that the application of the prestressed dvw reinforced joint instead of the traditional joints with dowels opens new frontiers in structural timber design. The bending capacity of timber joints which traditionally is maximum 40% of the bending capacity of the timber members is increased to at least 100%. Combined with the semi–rigid behaviour of this new innovative joint, timber structures can be designed much more more efficiently. Timber saving of up to 30% is possible when applied in portal frames. The bending capacity of the timber is in all cases still the governing factor for ULS design.

9 ACKNOWLEDGEMENT

The authors wish to acknowledge the industrial partners, Lignostone (dvw) in Ter Apel (NL) and De Groot Vroomshoop (NL) and the Firm Holzbau (I) as well as the European Commission for their financial contribution of this research project within the R&D programe FOREST. The results used were obtained in collaboration with the following research laboratories, University of Brighton (GB), University of Florence (I), LNEC Lisbon (P), Tech. University Karlsruhe (D) and City University London (GB).

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Lifespan Evaluation of a Nuclear Vessel in Terms of Prestress Loss

Prévision sur 40 ans des pertes de précontrainte d'une enceinte nucléaire

Abschätzung der Lebensdauer von Spannbeton in Nuklearreaktoren

Laurent GRANGER
Research Engineer
EDF SEPTEN
Paris, France

Laurent Granger, born in 1969, is Dr. Eng. of ENPC, Paris, France. For 3 years, he was involved in a research program for Electricité de France (EDF) aiming at predicting the life span of Pressurised Water Reactor Vessels in terms of prestress loss.

Jean-Michel TORRENTI
Research Engineer
ĆEA
Saclay, France

Jean-Michel Torrenti, born in 1953, is Dr. Eng. of ENPC, Paris, France. Since 1994, he is the head of the Concrete Research Laboratory of the Commissariat à l'Energie Atomique (CEA).

SUMMARY

The mechanical strength and tightness of the reactor buildings of French nuclear power plants are provided by biaxial prestressing of the concrete. To evaluate the "life" of the containment, in the sense of loss of prestress, the authors propose a physical-chemical model of the delayed behaviour of concrete, which is applied to the Paluel power station in Normandy. The modelling of the structure and the comparison of the simulations with in situ measurements are described over a period of 10 years.

RÉSUMÉ

La tenue mécanique et l'étanchéité du bâtiment réacteur de centrales nucléaires françaises sont assurées par une précontrainte biaxiale du béton. Afin d'évaluer la "durée de vie" de l'enceinte au sens de la perte de précontrainte, les auteurs proposent une modélisation physico-chimique du comportement différé du béton qui est appliqué à la centrale de Paluel en Normandie. L'article présente la modélisation de la structure et la comparaison des simulations avec les mesures in situ sur une période de 10 ans.

ZUSAMMENFASSUNG

Die Festigkeit und Dichtigkeit der Nuklearreaktoren in französischen Atomkraftwerken werden durch zweiachsige Vorspannung des Betons gewährleistet. Um die Lebensdauer, im Sinne von Vorspannverlusten, dieser Behälter abzuschätzen, stellen die Autoren ein auf physisch-chemischen Grundlagen basierendes Modell zur Bestimmung des zeitabhängigen Verhaltens von Beton vor, welches zur Berechnung des Paluel Kernkraftwerks in Normandie verwendet wurde. Die Modellierung des Bauwerks wird gezeigt und die numerischen Ergebnisse mit in situ Messungen über eine Periode von 10 Jahren verglichen.

1. THE INDUSTRIAL PROBLEM

The reactor building of a 1300 MWe nuclear power plant consists of 2 concentric containments (Fig. 1). The inner containment, biaxially prestressed, from 90 to 120 cm thick, is designed to withstand an internal pressure of 0.5 MPa, which leads to a mean initial prestress of 8.5 MPa along zz and 12.0 MPa along $\theta\theta$. The outer one, designed to withstand external aggressions, is made of reinforced concrete. The construction of the containments lasts 5 years; the prestressing begins at the end of the 2[nd] year and takes 1 year, in a complex site staging. The reactor is commissioned approximately 7 years after the start of work. In the operating stage, the inner containment is subjected externally to "atmospheric conditions" (15°C, 60% HR) and internally to a temperature and a relative humidity of 30°C and 45 %.

In an accident condition, the tightness of the structure depends mainly on the residual prestress of the concrete. But the devices for surveillance of delayed strains (wire strain gauges, Invar wires, etc.) reveal kinetics of strain that regulation models [3, 5], fail to incorporate in a satisfactory manner. To improve the "management" of the set of power stations, through a better evaluation of their life, EDF undertook in 1992 a vast programme of study [6] with a view to predicting the true creep behaviour of the containments. This study includes many shrinkage and creep tests on concretes reconstituted in the laboratory, together with numerical modelling with a view to prediction of the *in situ* strains in 40 years.

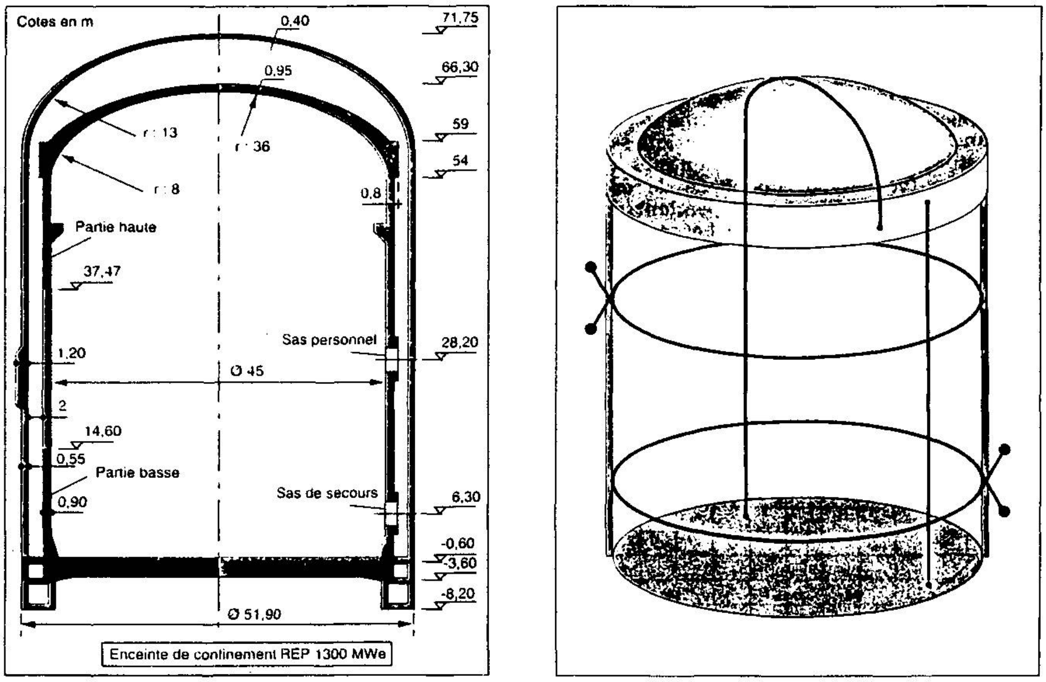


Fig. 1 Simplified diagram of containment and prestress.

2. MODELLING OF THE DELAYED BEHAVIOUR OF CONCRETE

An elastic calculation, by finite elements, in which the structure is subjected to an internal pressure (simulating the prestress), shows that in the running part, 15 m < z < 45 m, the shaft is deformed as an infinite cylinder, not constrained by the dome or the foundation raft. This calculation is used to validate the material approach, chosen over a relatively cumbersome numerical calculation of the whole structure.

In what follows, we shall distinguish drying creep from basic creep and drying shrinkage from endogenous shrinkage according to the commonly accepted definitions [2]. The engineering model we propose, of the equivalent continuum type, is based on a very simple principle: <u>each physico-chemical component receives specific numerical treatment</u> [1,2].

- As for the linear thermo-elastic strain, $\underline{\underline{\varepsilon}}_e$, we write, conventionally:

$$\underline{\underline{\varepsilon}}_e = \frac{1+\nu}{E}\underline{\underline{\sigma}} - \frac{\nu}{E}\mathrm{tr}(\underline{\underline{\sigma}})\underline{\underline{1}} + \alpha\Delta T\underline{\underline{1}} \qquad (1)$$

- The basic creep is modelled as a function of the relative humidity h (% RH) and temperature T (K). The creep function J_{bc}, taken from [5], is then:

$$J_{bc}(t,t_c,h,T) = \frac{1}{E_0} + h\frac{T-248}{45}\cdot\frac{28^{0,2}+0,1}{t_c^{0,2}+0,1}\Phi_{bc}(t_{eq},t_c=28,h=1,T=20°) \qquad (2)$$

$$t_{eq}(t) = \int_{s=t_0}^{t}\exp\left(-\frac{U_c}{R}\left(\frac{1}{T(s)}-\frac{1}{293}\right)\right)ds \qquad (3)$$

The viscoelastic constitutive law we use makes it possible to take the temperature and humidity history into account. If, at time t_n, the stress, the strain, the temperature, and the humidity, assumed constant over a time interval, are known, the strain for $t \in [t_n, t_{n+1}]$ is found by writing:

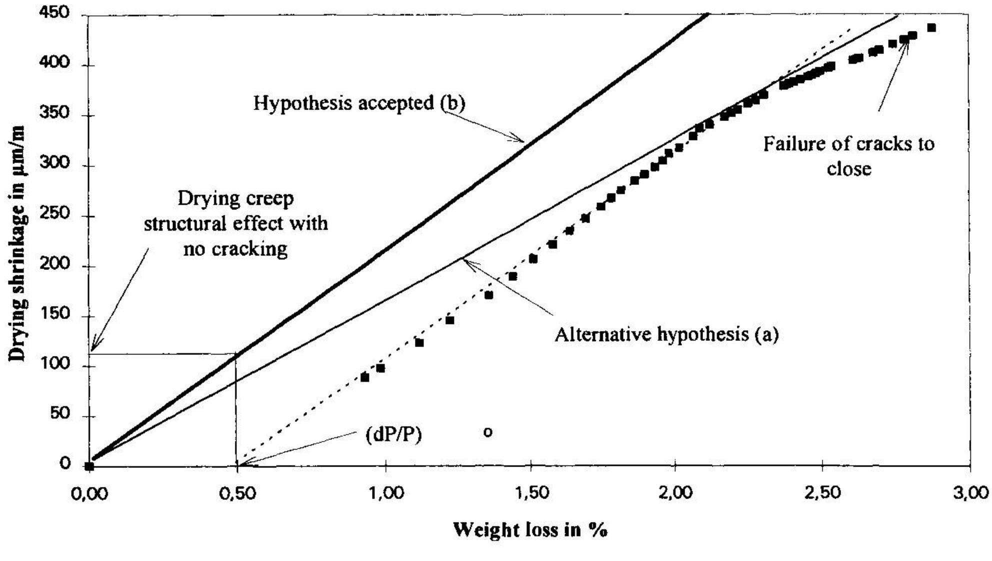
$$\varepsilon_n(t) = \varepsilon_{n-1}(t) - \sigma_{n-1}J_{bc}(t,t_n,h_{n-1},T_{n-1}) + \sigma_n J_{bc}(t,t_n,h_n,T_n) \qquad (4)$$

This formulation, which corresponds to an unloading and complete reloading, then indeed satisfies the elementary criteria of continuity.

- The <u>drying shrinkage</u> ε_{ds} is taken as proportional to the weight loss $(\frac{\Delta P}{P})$ [1, 2] :

$$\varepsilon_{ds}(t) = k\left[\left(\tfrac{\Delta P}{P}\right)_t - \left(\tfrac{\Delta P}{P}\right)_0\right] \qquad (5)$$

The term $k(\frac{\Delta P}{P})_0$ results from the fact that the shrinkage induces a skin cracking of the material by blocked strain. This cracking, rarely visible to the naked eye, can be found in curves of drying shrinkage versus weight loss (Fig. 2).



<u>Fig. 2</u> Drying shrinkage versus weight loss.

Various authors [10] have observed, on very small specimens, a *quasi* linear relation between drying shrinkage and weight loss. The curve above, if the skin cracking did not appear, could be analyzed in two ways:

a) According to the dashed curve [4, 2].

b) But in our opinion [6, 7], the behaviour is represented rather by the solid curve, since the cracks can in all likelihood not close, or only very poorly.

• The drying creep ε_{dc} [7, 2] results from the sum of an intrinsic creep (int) proposed by Bazant and a structural effect (str) linked to the drying shrinkage:

$$\Delta\varepsilon_{dc}^{int} = \lambda\sigma|\Delta h| \qquad \varepsilon_{dc}^{str} = kF(\sigma)\left[\left(\tfrac{\Delta P}{P}\right)_{load} - \left(\tfrac{\Delta P}{P}\right)_0\right] \tag{6}$$

where $\left(\frac{\Delta P}{P}\right)_{load} \leq \left(\frac{\Delta P}{P}\right)_0$ is the weight loss that has already occurred at the age of loading. From [9], $F(\sigma)$ is given by:

$$\sigma \leq 0 \Rightarrow F(\sigma) = 0 \qquad 0 \leq \sigma \leq 15 \Rightarrow F(\sigma) = \frac{\sigma}{15} \qquad 15 \leq \sigma \Rightarrow F(\sigma) = 1 \tag{7}$$

The total delayed strain ε_{tot} is then found by adding together the different contributions:

$$\varepsilon_{tot}(t) = \varepsilon_e(t) + \varepsilon_{ds}(t) + \varepsilon_{bc}(t) + \varepsilon_{dc}^{int}(t) + \varepsilon_{dc}^{str}(t) + \varepsilon_{tc}(t) \tag{8}$$

where ε_{tc} is a transcient thermal creep. It should be noted that only the basic creep must follow the constitutive law established in (4). As for the sequencing of the calculations, a thermal calculation is performed first of all. It is followed by a calculation of hygral diffusion (transient, nonlinear) in which the coefficient of hygral diffusion is a function of the water content C(x,t) and of the temperature T(x,t) given by the previous calculation:

$$\frac{dC}{dt} = \mathrm{div}(D(C,T)\underline{\mathrm{grad}}(C)) \tag{9}$$

Finally, a 3rd calculation, viscoelastic, uses the foregoing results and calculates, at each time step, the total delayed strain at each point of integration of the mesh. Note that the fact of linking the three calculations in this order presupposes some conventional decouplings between the various delayed strains (9). The main physical parameters of the model are then determined from the results of the experimental programme, mechanical tests, a weight loss test, and a complete test of delayed behaviour (endogenous shrinkage, total shrinkage, basic creep, and creep at 50 % RH).

- The free water content is given by $C_0 = w_0 - 0{,}9 \cdot 0{,}22c_0$ where c_0 is the weight of cement and w_0 the quantity of water. C(h) is evaluated by recent desorption results.
- $J_{bc}(t, 28, h = 1, 20°)$ is fitted to the basic creep test.
- $D(C,T) = A \cdot 10^{-13}\exp(0{,}05C)\frac{T}{293}\exp(-\frac{U_T}{R}(\frac{1}{T} - \frac{1}{293}))$ is fitted to the weight loss test.
- k and $(\Delta P/P)_0$ are fitted to fig. 2.
- λ is fitted to give the total strain.
- α = 10.10-6 °C^{-1} and U_T and U_c are values taken from the literature.

The results of the various simulations are given in fig. 3, which shows in particular the shares of the various delayed strains calculated on a laboratory sample of 16 cm diameter..

3. SIMULATION OF STRAINS ON CONTAINMENT

When the various parameters of the model have been determined, it is possible to predict the results on a structure (fig. 4) from the staging of prestressing and the boundary conditions (in temperature and humidity) and compare the simulations performed with measurements made *in situ* over a period of 10 years (fig. 5).

The study of the containment is limited to study of an annulus 6 m high and 90 cm wide calculated in axisymmetry. To model the initial prestress, the test body is subjected to a

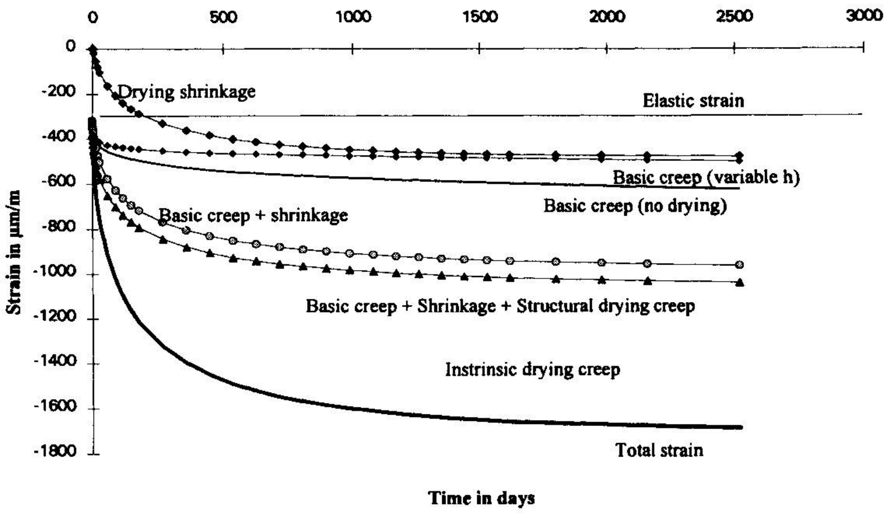


Fig. 3 Breakdown of delayed strains of Paluel on specimen Ø 16 cm.

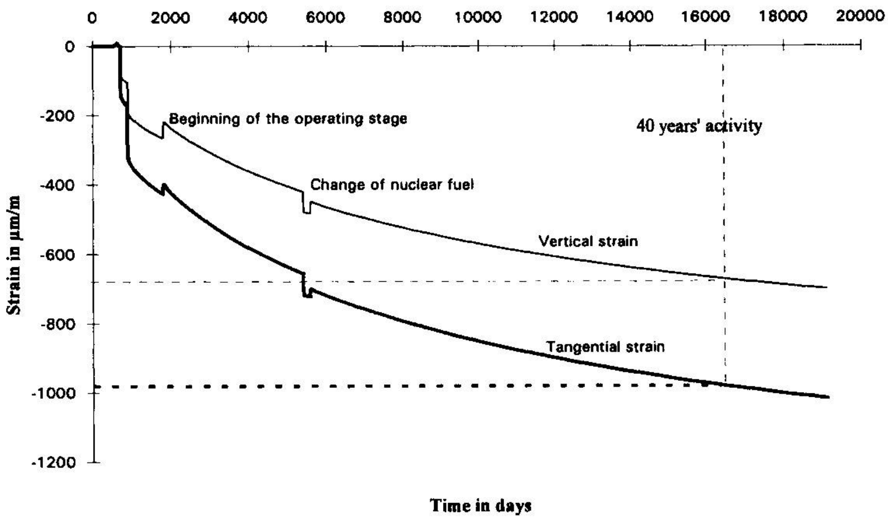


Fig. 4 Simulations of Paluel containment for a constant initial prestress.

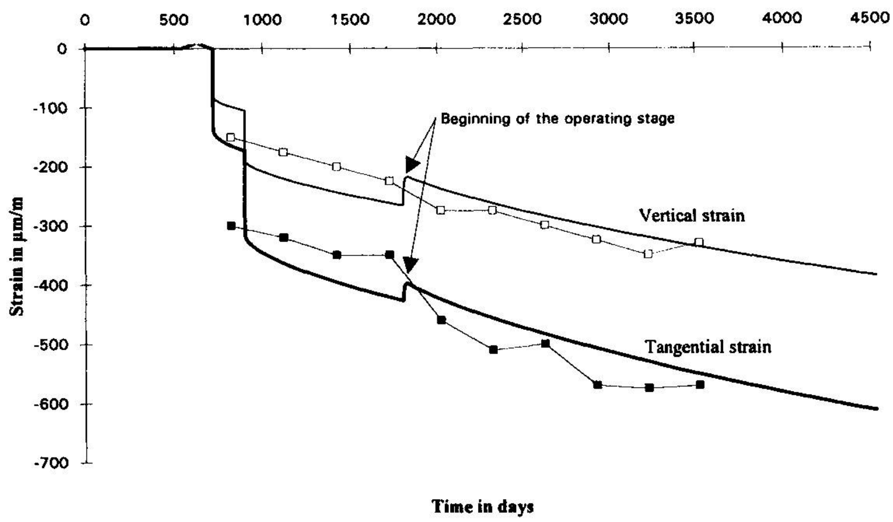


Fig. 5 Comparison between simulations and *in situ* measurements over a 10-year period.

constant pressure along e_{zz} of $p_{zz} = 8.5$ MPa and a constant pressure along e_{rr} calculated as follows $p_{rr} = \frac{\sigma_{\theta\theta}}{R_{ext}} = \frac{12}{24} = 0,5$ MPa to model the prestress along $\theta\theta$.

4 . CONCLUSION

In structural calculations, the behaviour of the concrete is taken into account in accordance with regulations that give an average response for the material. It should however be recalled that the delayed strains of a particular concrete (for a given range of strengths) can be rather far from the tendency indicated by the regulation. For sensitive industrial applications, it is therefore recommended that a study of the delayed behaviour of the concrete used should be undertaken when the building of the structure is started. In the case of nuclear containments, the shrinkage and creep results obtained in the laboratory are used to judge the delayed strains to come and therefore the life of the structure. However, the phenomena of delayed strains can be investigated only by relatively long tests (2 years); this time is often incompatible with construction site planning. If it is desired to guard against the hard-to-control influence of the constituents (aggregates, binder), the use of a high-performance concrete (compatible with the design criteria of the structure), one that is particularly good in terms of shrinkage and creep, [8], can help to substantially reduce the risks related to losses of prestress.

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Monitoring, Maintenance and Replacement Policies for a Stock of 100'000 Structures

Surveillance, maintenance et remplacement d'un parc de 100'000 ouvrages d'art

Ueberwachung, Instandhaltung und Erneuerung eines Baubestandes von 100'000 Bauwerken

Frédéric DELORME
Head, Structures Division
French National Railroads (SNCF)
Paris, France

Frédéric Delorme, born in 1960, is a graduate from Ecole Polytechnique and ENPC, Paris. For three years, he has been in charge of French Railroads' structures division.

SUMMARY

French National Railroads (SNCF) manage a stock of 100'000 structures whose design, age and operating conditions present high variability. This paper describes the various arrangements (monitoring rules, weighting methods and maintenance strategies) which ensure safety, and more generally, the reliability of structures, and their use, a number of them being more than hundred years of age.

RÉSUMÉ

La SNCF gère un parc de 100'000 ouvrages d'art, dont le type de structure, l'âge et les conditions d'exploitation sont extrêmement diversifiés. Cet article décrit les différentes dispositions (règles de surveillance, méthodes de cotation des ouvrages et stratégies de maintenance) qui lui permettent d'assurer la sécurité, ou plus généralement la fiabilité, de l'exploitation d'ouvrages dont beaucoup sont aujourd'hui plus que centenaires.

ZUSAMMENFASSUNG

Die SNCF verwaltet einen Baubestand von 100'000 Bauwerken, deren Strukturtyp, Alter und Betriebsverhältnisse sehr vielfältig sind. Dieser Artikel beschreibt die verschiedenen Bestimmungen (Ueberwachungsregeln, Verfahren für die Kotierung der Bauwerke, Instandhaltungstrategien), die ihr ermöglichen, die Sicherheit -oder allgemeiner die Zuverlässigkeit- des Betriebes von Bauwerken zu gewährleisten, die heutzutage in grosser Anzahl mehr als hundertjährig sind.

1. DESCRIPTION OF THE STOCK OF STRUCTURES HELD BY FRENCH RAILROADS

The French Railroads'stock of structures consist of 80 000 bridges, 1500 tunnels and 8 million m2 of walls and stone pitching. The replacement cost of this stock is estimated at FFr 180 bn. This stock also presents a high geographic dispersion.

Most of the structures are very old and very few of them have been subject to renewals. Some of them date back to the dawn of the railroads' construction and are more than 150 years old. 75 % of masoned structures are over 100 years old, 45 % of steel bridges over 80 years old and 65 % of tunnels exceed one hundred years of age.

From the safety point of view, the structures requiring the sharpest scrutiny are the older steel bridges, the rail bridges in water environment and tunnels.

As a result of increased operating speeds and loadings, far in excess of their early design specifications, the steel bridges built between 1860 and 1920 have entered an irreversible fatigue process, which calls for a sustained and proactive policy in terms of monitoring, maintenance and replacement.

Since the average age of foundations in water-environment is 115 years, it is necessary to conduct a tighter monitoring and a specific strengthening due to the degradation of their condition, to the sudden changes in their land environment (lower river beds, remodeled banks), or to the risks that could be incurred in the event of the displacement of a bridge foundation.

After the Vierzy rail accident in 1974, French National Railroads have implemented over the last twenty years an organisation which provides an in-depth appraisal of the actual condition of tunnels and enables to plan repairs whenever necessary. Their current condition is therefore satisfactory, both in terms of safety and durability.

The past experience on these three types of structure has shown that their service life could be extended not only on the basis of initial structural capacity reserves but also with the adjustment of their monitoring and maintenance conditions. Performance standards have been enhanced in the process. A number of methods successful developed for tunnels, such as the weighting method described later on, have been extended to all types of structure. New monitoring and maintenance policies have been implemented as a result of the recent changes affecting the actual condition of the structures. These methods will be presented in this article.

2. PRINCIPLES GOVERNING THE MONITORING OF STRUCTURES ON FRENCH RAILROADS

The safety management of rail structures is first and foremost based on the reliability of visual examinations and is therefore conditioned by the competence of human operators and notably structure inspectors who have this fundamental duty to accomplish. There does not exist any mechanised means measuring and characterising their condition, contrary to some other rail infrastructure components, such as the right-of-way.

Monitoring starts with the analysis of the initial or reference situation. Subsequently, all the sections of a structure will have to be inspected in-depth, as part of detailed five-year inspections. These inspections appraise all the deteriorations existing on the structure. However, there is still the possibility of a rapid change. Therefore, detailed inspections have to be supplemented by annual inspections focusing more particularly on critical areas.

Special inspections may be decided when environmental conditions are likely to jeopardize safety : steel structures which are vulnerable to thermal loads (such as puddled iron) and subject to special inspections during very cold winter periods. Foundations in river environment are also inspected to detect any local scour after major floods.

These inspections are supplemented by examinations on foot on a weekly basis or over short cycles to investigate the track performance and by lineside audits determining the quality of monitoring and maintenance over a certain territory under one civil engineering authority.

3. PRESENTATION OF THE WEIGHTING METHOD FOR STRUCTURES DEVELOPED BY FRENCH RAILROADS

The weighting system serves the purposes that are described hereunder :

3.1 Possessing a reliability indicator for structures and measuring the condition and changes affecting the various stocks (broken into categories or geographical perimeters).

The weighting system for structures to be introduced this year is aimed at providing a better and more immediate comparative assessment of the various French Railroads regional stocks as well as pathologies affecting those structures and the changes they are likely to undergo. It is therefore a predictive tool which assists in the setting-up of preventive maintenance policies.

Naturally, the weighting of a structure taken in isolation has no absolute meaning. It has only a relative value and relevance is achieved if it is placed into the context of a statistical base encompassing the whole population of structures.

The weigthing system does not come in lieu of continuous and periodic monitoring steps. It is only an add-on to give an overall and synthetic assessment of the condition of structures. The weighting system is not meant to detect immediate risks in terms of performance and safety, since this is part of the local civil engineering authority to monitor those. So, this weighting system has to be understood as a "reliability" indicator for structures -and a predictor of their failures and therefore of their ultimate safety- rather than a measurement tool for immediate safety.

3.2 Obtaining ultimately statistical models of wear and tear per type of structure

Economists claim rightly that to be able to choose between two repair scenarios on a structure (for instance choosing between "protracted therapy" or replacement), it is necessary to make long-term net present values calculations so as to achieve the economic optimum.

This theory is thwarted today by the lack of scientific knowledge of engineers on the fatigue modes affecting structures. This is due to their very slow rate of change compared to other industrial equipment which are less of a durable nature (i.e. rails).

At the end of the day, the weighting system for structures will materialize in the form of major statistical data banks on the evolution of pathologies per type of structure. Models on ageing will be constructed as a function of their evolutionary factors (traffic, environment). In return it will be possible to predict a potential evolution of structures and their probable condition in the future.

3.3 Steering the national and regional maintenance policies

The weighting system will provide multiple indicators which will greatly improve maintenance policies as shown in the two previous experiments (weighting system for tunnels after the Vierzy accident), and weighting system to classify major iron structures of UIC International Union of Railroads) (groups 1 to 6).

It will be possible to justify more accurately the funding scenarios proposed for the maintenance and regeneration of structures.

As an illustration of financial targets :
- replacement every 10 years of all steel structures showing a weighting in excess of 60 points, which would mean a fatal fatigue failure in the next 10 years ;
- preventive treatment every 5 years on all masonry disjointings with a mark for disjointing in excess of 70.

From such objectives, it will be possible to establish the annual financial flows to be devoted to the various categories of structures thanks to the marks which will exceed the maximum threshold prescribed for a given pathology.

The assessment of average repair costs will be treatable statistically : from a homogeneous sample of structures of the same type and with the same mark, it will be possible to set simple economic rules applicable to a large family of comparable structures.

The weighting of a structure includes four chapters :

Chapter A - Condition of the structure
Chapter B - Evolutionary factors
Chapter C - Effects of this condition on operation and maintenance,
Chapter D - Strategic policy to be adopted on the structure.

The weighting of A and B is conducted as part of a detailed inspection by the inspector in charge (there are two types of inspectors : non-specialist and specialist inspectors, the latter handling special techniques.

Chapter A is a physical description of the pathologies affecting a structure. It includes a record of duly referenced failure codes. Each part of the structure is analysed and recorded : homogeneous segments or spans can be put together. The chapter A form is completed during detailed inspections which means that it is a five-year photograph of the condition of the structure.

Chapter B is a record (which is not evolving much from one inspection to the next) of the factors governing the evolution of the various pathologies.

It is from chapter A that it is possible to know the condition and therefore the reliability of the various stocks in French Railroads, at the regional or local level, and to monitor the evolution of their condition every five years so as to possess ultimately statistical models on wear and tear per type of structure (identification of cases where deterioration thresholds are exceeded).

With chapter A and B, the purpose is to predict the future over a ten-year horizon without any repair on the structure and therefore to predict the aggravation of defects over time.

The record on the consequences of the condition over operation and maintenance (Chapter C) is a quality indicator to be monitored in real time as opposed to the condition weighting which only takes place every five years. It records the safety steps taken such as strengthened monitoring, operating restrictions, track repair or temporary reinforcement which all reflect insufficient monitoring or maintenance.

The strategic policy (chapter D) is determined for each structure. Chapter D provides a strategic repair criterion irrespective of the condition of this structure. It depends on the tracks which are intersected or supported and on the repercussions for third parties and the environment.

Assessments contained in chapter B, C and D are special selection criteria between structures having the same condition mark. B, C and D can be used concurrently with other (mainly financial) criteria, so as to prioritize repairs.

Details concerning the determination method for the condition mark A :

A tunnel, a wall, a stone pitching can be broken down into segments with a homogeneous condition.

A bridge can be broken down into elementary parts (mechanically speaking) : foundations, bearings, deck, arch, membrane,...

As for the whole structure, the parts or segments receive a mark over 100 (maximum mark). For each part of the structure, Ci coefficients are introduced so as to be able to reconstitute the total mark for the whole structure from the individual marks allocated to each part.

The weighting formula adopted is the following :
N = sum (Ci x sum (Nij),
where Ci is the coefficient of the structure related to part i and Nij is the condition mark reflecting damage j of part i.

For bridges, the structural coefficient are 0.9 for deck and 0.1 for bearings and, for foundations in water environment, 0.5 for foundations and 0.5 for the superstructure.

There are two ways to query from the system :

- the first way is to outline the most serious defects requiring an immediate repair : in that case, the question is on those cases where the predetermined severity threshold for each basic pathology are exceeded ;

- the second one is to identify the general condition of a structure or part of structure in order to assess economic requirements. In that case, the question is on most cases where a certain mark on the condition of the structure or part of the structure are exceeded.

4. DEVELOPMENT OF MAINTENANCE STRATEGIES

To define the financial flows required for maintenance and renewal of structures, three methods have been selected for cross-examination purposes :

4.1 Method using the weighting of structures : case of steel structures

SNCF has surveyed in 1993 the steel structures in excess of 30 meters over high density lines and constructed before 1900. A hundred structures have been identified by a team of specialists and have received a mark on a very detailed grade, validated over a few examples. The marks were given between 40 and 280. The most critical decks (mark in excess of 120) must be exchanged within 15 years, so as not to be safety critical. This experimental approach has been extrapolated to the whole population of older structures. It was possible to predict that 130 000 tons of steel will have to be replaced by 2010 ; this amount corresponds to a total budget of 5300 MF.

4.2 Using next repair periods : example of foundations

The average age of the 480 foundations in water environment was 124 years at the time they were treated (some of them have been treated under an emergency). Knowing the age of foundations and the average repair cost per foundation (1.6 MF per bearing), it is possible to quantify an annual financial flow of 150 MF until 2010 to be able to put in place a true preventive policy in lieu of the emergency exchanges (the average age for foundations to be treated should be around 130 years so as not to be safety critical).

4.3 The use of global management ratios

The use of global ratios of the OECD type is not the most reliable method since the level of ratios recommended depends on the actual design and condition of structures. However, comparing the results of this method with the existing ones has enabled SNCF to make sure that the expenses predicted for maintenance and regeneration of structures were correct.

SNCF has a backlog of work to catch up to ensure the durability of its stock of structures, and more particularly its steel structures and foundations in water environment. French Railroads should spread evenly their operating costs and investments so as not to penalize the accounts of the company or its train operations. Such penalties would be unacceptable for the company.

Therefore, an average financial flow of about 0.5 % of the value as new of its stock over 15 years (0.7 % for bridges, 3.5 % for steel bridges, and 0.2 % for tunnels), excluding all the ancillary costs related to signalling and protection work (speed restrictions and train diversions) (i.e. the sole civil engineering cost).

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Seismic Protection through Elastomeric Base Isolation

Protection sismique grâce aux isolateurs en élastomère

Seismischer Schutz durch Elastomere Isolation

Markus PETSCHACHER
Dr.sc.techn.
Vienna Consulting Engineers
Vienna, Austria

Markus Petschacher, born 1962, received his diploma as civil engineer at TU Graz, Austria. After a short time in practice he was assistant at ETH-Zurich and worked in the field of reliability theory, where he received his Ph.D. in 1993. His main activities, at VCE, are in research and innovative design.

SUMMARY

Isolation systems are applied for various structures requiring protection against dynamic excitation. By means of a probabilistic design approach, criteria for elastomeric bearings are discussed considering different stochastic quantities. Employing a simple mechanical model, major limit states are analysed. The calculated sensitivity factors of design parameters offer information for the design of seismic protection devices.

RÉSUMÉ

Des systèmes d'isolation sont utilisés pour différentes constructions nécessitant une protection contre des forces dynamiques. Les critères de dimensionnement pour les appareil d'appui en élastomère sont discutés grâce à une approche probabiliste prenant en compte les différentes variables stochastiques. Les états-limites principaux sont analysés en utilisant un modèle mécanique simple. Les facteurs de pondération des paramètres de dimensionnement, ainsi calculés, donnent des informations pour la conception de l'appareil d'appui.

ZUSAMMENFASSUNG

Isolationssysteme werden für verschiedene Bauwerke angewendet, die einen Schutz gegen dynamische Anregung benötigen. Mit Hilfe eines probabilistischen Ansatzes, der verschiedene stochastische Grössen berücksichtigt, werden Entwurfskriterien für Elastomerlager erörtert. Unter Verwendung eines einfachen mechanischen Modells werden wichtige Grenzzustände analysiert. Die errechneten Sensitivitätsfaktoren der Entwurfsparameter bieten Informationen für die Lagergestaltung.

1. INTRODUCTION

The high level of respected reliability of public lifeline systems such bridges and viaducts and required low maintenance costs are stimulating this research. Society is directly affected by a collapse or a reduced utility of a bridge. Such problems become transparent when thinking about High Speed Railways (HSR) build mainly on viaduct systems. Severe earthquakes may partly damage the structure or moderate earthquakes may diminish the operation speed. The second argument in particular affects the maintenance costs, which are being considered more and more by public owners. Looking at the problem from this point of view, the bearings are, beside their semantic meaning, one of the most important links in the chain.

A passive base isolation is the applicable solution for a viaduct HSR structure. Such a structure may be isolated in horizontal and vertical directions. In this paper basic contribution is given on laminated elastomeric bearings for carrying the vertical loads. The characteristic properties will be evaluated by means of a probabilistic approach. The strain due to shear deformations and effects of non-linearity will be neglected. The development of an highly adaptable rubber isolation system (HARIS) will be supported by the Industrial and Materials Technologies program of the Brite-EuRam III action. Principal information is given in [1].

2. PRINCIPLES OF BASE ISOLATION

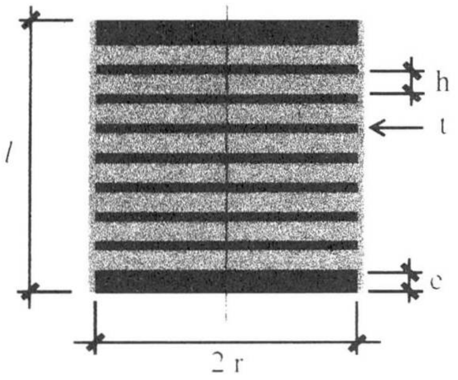
To achieve seismic isolation the superstructure must be decoupled from horizontal accelerations occurring during an earthquake. Anti-seismic devices must have a certain horizontal stiffness able to move the fundamental response frequencies of the structure well below 1 Hz. This is valid for a sufficient rigid soil, where the frequency range is characterized by a low energy content. The structure will behave like a rigid body in the horizontal plane. In the case of too soft soil, base isolation will not be applicable. The decrease of accelerations by means of filtering elements has the disadvantage of increasing the displacements. This can be limited by adding energy dissipating elements with a good self-centering capability to return the structure into its initial position. Another possibility of influencing the damping behaviour is determined through the phenomenon, that the damping remains finite while the lateral stiffness falls to zero. It can be shown, that damping of the bearing may well be significantly greater than that of its constituent elastomer.

The following discussions are based on a simple SDOF model assuming linearity and considering only the longitudinal direction of the bridge. The natural period T_0 of this oscillator is related to its equivalent mass M and stiffness K:

$$T_0 = 2\pi\sqrt{\frac{M}{K}} \tag{1}$$

The elastic response spectrum S_a on a specific site is normalized and depends on the eigenperiod T. The probability of occurrence is given implicitly through the Peak Ground Acceleration (PGA). However, the shape of the spectrum does depend on the site conditions. As mentioned, the discussions are related to a planned HSR. Therefore, a site-specific spectrum is determined from Taiwan. The normalized horizontal acceleration spectrum must be scaled by the PGA value, which is based on a certain return period and is related to the specific site. For a moderate period the spectral acceleration S_a is given by:

$$S_a = Z\frac{c}{T_0} \tag{2}$$



Unit shear stiffness:

$$R = G\pi r^2 \frac{t+h}{h}$$

Unit tilting stiffness:

$$T = \frac{3}{4} G\pi^4 \frac{t+h}{h}\left(1+\frac{r^2}{6h^2}\right)$$

Total height of bearing:

$$l = nh + (n-1)t + 2e$$

Fig. 1 Characteristic values of a circular shaped laminated rubber bearing.

where Z is the PGA and c is a factor depending on the soil type, here set to $c = 1$. Eq.2 assumes the damping to be 5% of critical. From this the peak deflection of the elastomeric device may be estimated according to the relation:

$$D = \omega^2 S_a = T_0 Z \frac{c}{(2\pi)^2} \tag{3}$$

The isolation system must be designed to achieve a suitable long fundamental horizontal period. Employing Eq.1, the required stiffness K_s of the device can be calculated. The unit shear and tilting stiffness for a circular shaped device are given in Fig.1.

A bearing must sustain the dead load DL and the train load TL, resulting from one or two trains. A dynamic factor Φ is considered to determine the statical equivalent load effects due to the rolling stock. The vertical load for one bearing is:

$$V = \frac{1}{2}(DL + 2\Phi TL)\frac{L}{2} \tag{4}$$

An elastomeric bearing is characterized by two shape factors. The so-called primary factor S_1 is equal to the ratio between cross section and lateral surface areas of each layer and affects the vertical stiffness, which varies approximately proportional to its square. A low S_1 value indicates a vertical system isolation. The secondary shape factor S_2 equals the ratio between diameter and total rubber thickness and affects mainly the bearing capacity relative to the buckling phenomena.

3. STOCHASTIC MODEL

3.1 Shear stiffness

Uncertainties in the G modulus result from the limited possible accuracy of material production fitting a certain Shore grade value, relative to the expected mechanical property. The method of measuring the shear modulus is very sensitive and is effected by environmental temperature, strain rate and rubber layer thickness, which are neglected herein. Considering the mentioned uncertainty a log-normal distribution is assumed.

The shear stiffness depends mainly upon the vertical loading. In extreme deflection situations a different stiffness behaviour can also be seen relative to the type of end-plate construction, dowel or bolt connection. For a circular shaped laminated elastomeric bearing (LEB) the shear stiffness K_s according to [2] is:

$$K_s = \frac{V^2}{2qT\,tan\,(ql/2) - Vl} \tag{5}$$

In Eq.5 the radius r, the total height l of the bearing and the vertical or compressive loading V are used. The quantity q is defined as $q^2 = V/T(V/R+1)$.

Table 1 Basic variables **X** used in analysis

X	Type	μ	σ	Description
G	LN	1.0	0.1	shear modulus [MPa]
DL_1	N	245	20	self weight [kN/m]
DL_2	LN	172	17	superimposed dead load [kN/m]
TL	T1L	71	10	HSR train load [kN/m]
φ	LN	1.06	0.05	dynamic factor [ ]
T_0	LN	1.87	0.1	natural eigenperiod [s]

3.2 Maximum horizontal deflection

The shear effect is mainly limited by the *roll-out* effect or by cavitation of the rubber. Both phenomenons are caused by large deflections. In extreme deflection situations a different stiffness behaviour can also be seen relative to the type of end plate construction, dowel or bolt connection.

If the end-plates are fixed by bolts the maximum shear deflection is limited by rupture of rubber, initiated through cavitation or extreme shear strains. Cavitation, an effect known from liquids, is also possible for elastomers. It describes an elastic instability when the hydrostatic tension reaches only few MPa, about 3G in the case of rubber. This phenomenon is caused by pure tension, as well as by extreme shear actions and may lead to rupture. Following [4], cavitation is expected to occur in the end rubber layer at a deflection of $d \cong r/2$, when a compressive axial force is assumed. The theoretical value can be calculated from the following expression:

$$D_{cav} = \frac{Vr}{3(V+K_s l)} \frac{2x_m}{3x_m^2 - 1} \tag{6}$$

The location of the minimum pressure can be calculated from:

$$x_m^2 = \frac{(2-3\rho) \mp \sqrt{9\rho^2 - 8\rho}}{2} \tag{7}$$

where $\rho = p_m/(2v)$ with the normalized compressive load $v = V/(\pi r^2 E)$. Cavitation can be evaluated when setting $p_m = -1$.

In the case of dowelled connections of the end-plates to the super- and substructure the *roll-out* effect will limit the possible shear deflections. The end-plates would be free of bend and at large deflections there could not be a hydrostatic tension in the end rubber layer. The *roll-out* threshold is defined by:

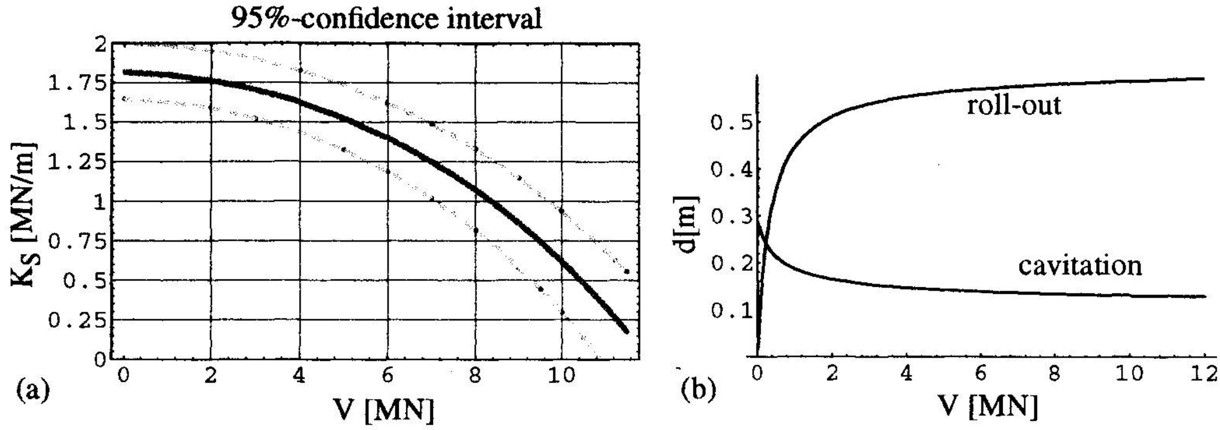


Fig. 2 The scatter of the shear stiffness (a) and critical deflections causing roll-out or cavitation (b).

$$D_{roll} = \frac{2Vr}{K_s l + V} \tag{8}$$

Cylindrical bearings may be deflected to 80% of their diameter without failure. Devices with bolted end-plates may have deflections 30% higher than the diameter given a suitable compressive load. Failure will occur at smaller deflections, when the bearing is subjected to a tensile load. The mechanism of failure is characterized by a crack propagation, initiated in the region of maximum tension.

3.3 Maximum compressive load

The instability load of a laminated elastomeric bearing is given, when the shear stiffness given in Eq.5 reaches zero. In [6] a formula for circular shaped bearings is given by:

$$P_{cr} = \frac{R}{2}\left[\sqrt{1 + \frac{4\pi^2 T}{R l^2}} - 1\right] \tag{9}$$

The dead load is divided into the self weight of the box girder DL_1 with normal distribution and the superimposed dead load DL_2 consisting of ballast, sidewalks and tracks, which is defined by a log-normal distribution. The railway load takes the transverse load distribution for a ballasted track into account, and is given by an equivalent uniformly distributed load TL. A Gumbel distribution is employed considering the time variability. The dynamic factor Φ is taken as lognormal quantity reflecting the uncertainties of the dynamic behaviour. The distribution types are selected according to [3] and summarized in Tab.1 with their characteristic values.

4. RELIABILITY ANALYSIS

The influence of different compressive loads on the scatter of the shear stiffness in Eq.5 is evaluated by a 95% confidence interval plotted in Fig.2. A suitable Johnson curve is fitted to the first four statistical moments. With this information an approximate confidence interval can easily be calculated. Bearings can sustain a certain compressive load V given in Eq.4 depending on the shear deformations. The elastic instability is given by the limit state formulation in Eq.10. The sensitivity of the reliability factor β with respect to the basic variables given in Tab.1 and design parameters $\{r,n,h,t\}^T$ are calculated by a FORM analysis using the VaP program [5].

$$M_1 = P_{cr}(r, h, t, n) - V \tag{10}$$

Setting the parameters to {0.3, 8,0.015, 0.005} a calculation gives the sensitivity values reported in Tab.2, which are relative to the reliability index β. Further, in the event of a server earthquake a LRB must resist to extreme horizontal displacements.

$$M_2 = D_{cav} - D \tag{11}$$

A simple approach for evaluating the extreme horizontal displacement is used by employing Eq.3 with a PGA of $Z = 0.3 m/s^2$ and assuming a connection of the end-plates by bolts. Therefore the cavitation effects becomes the critical event. In the same manner as Eq.10 the influence of certain parameters will be evaluated in Eq.11.

Table 2: Sensitivities of basic variables and design parameters.

X	DL_1	DL_2	G	Φ	TL	r	n	h	t
M_1	0.286	0.337	-0.626	0.045	0.641	211	-2.07	-2140	68.3
M_2	0.367	0.2502	-0.514	0.037	0.589	6.77	-0.050	-43.9	1.17

5. CONCLUSIONS

The property of the elastomeric material is characterized mainly by the shear modulus G and influences therefore in consequence the stiffness of the device. It can easily be seen that the scatter of the shear stiffness is proportional to that of the shear modulus. The sensitivity factors due to a FORM analysis are showing the same importance of shear modulus and load contribution. This defines the starting point for further investigation.

The influence of various design parameters in terms of reliability measures have been quantified. These can be used to discuss different device layouts with respect to failure of probability or appropriate cost models. For the research work is now in progress only principal statements are given in this paper. But one central postulate can be drawn, that a precise manufactured elastomeric material will be most important to guarantee a reliable device.

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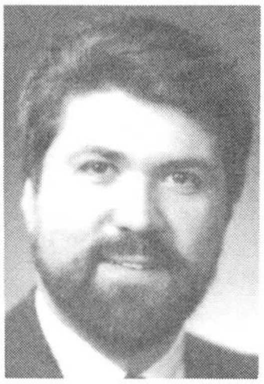
Seismic Retrofit of a Four-Storey Building in British Columbia

Consolidation vis-à-vis des séismes d'un bâtiment de quatre étages

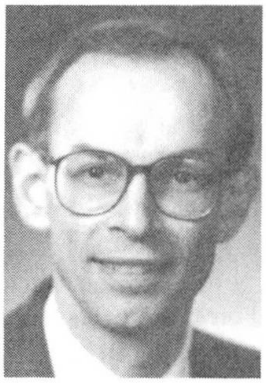
Erdbebensanierung eines vierstöckigen Gebäudes in British Columbia

Peter TIMLER
Senior Specialist
Sandwell Inc.
Vancouver, BC, Canada



Peter Timler, born 1958, received a Bachelor's Degree in civil eng. from McMaster University, Hamilton and a Master's Degree in Applied Science from the Univ. of Alberta, Edmonton. During his career he has participated in research, consulting and fabrication and erection.

John SHERSTOBITOFF
Senior Specialist
Sandwell Inc.
Vancouver, BC, Canada



John Sherstobitoff, born 1955, received a Bachelor's Degree in civil eng. from the Univ. of British Columbia, Vancouver and a Master's Degree in Applied Science from the California Institute of Technology, Pasadena. He has concentrated recently on managing seismic retrofit projects.

SUMMARY

The externally mounted steel frame for the seismic retrofit of the BCIT SW1 Main Building presented unique design constraints. The main structure is part of a four-building complex arranged in a rectangular pattern forming a central courtyard. The retrofit of these struc-tures used the courtyard as the optimum location for strengthening. Minimising window coverage and providing a visually unobtrusive retrofit and erectable system within three summer months, while meeting the specific ductility requirements for the members and connections necessitated both the design and detailing of all the connections.

RÉSUMÉ

La charpente extérieure métallique pour la consolidation du bâtiment principal de BCIT SW1, a présenté des difficultés très spéciales. La construction principale fait partie d'un complexe de quatre immeubles placés autour d'une cour centrale. Cette cour a été utilisée comme le lieu optimal pour la consolidation des structures. Minimisant l'obstruction des fenêtres et utilisant une consolidation discrète, le système a été réalisé en trois mois d'été. Il satisfait les exigences pour les éléments et les connections.

ZUSAMMENFASSUNG

Der an der Aussenfassade angebrachte Stahlrahmen zur Erdbebensanierung des Hauptgebäudes SW1 des British Columbia Institute for Technology bot einzigartige Entwurfsbeschränkungen. Die Hauptstruktur ist Teil eines aus vier Gebäuden bestehenden Komplexes, welche symmetrisch um einen rechtwinkligen Innenhof angeordnet sind. Der Innenhof wurde als optimal für die Verstärkung der Struktur bestimmt. Sowohl der Entwurf wie die Detaillierung der Rahmenverbindungen wurden im Ingenieurbüro angefertigt, da besondere Bestimmungen in Bezug auf die Duktilität von Rahmenteilen sowie für Verbindungen nach der kanadischen Norm einzuhalten waren. Ferner mussten Richtlinien betreffend minimaler Fensterverdeckung und akzeptabler Struktur berücksichtigt werden, sowie die besonders kurze Konstruktionsdauer während drei Sommermonate.

1. INTRODUCTION

Since 1989, British Columbia has developed an awareness for the evaluation and upgrading of the province's vulnerable structures. The Ministry of Advanced Education funded the seismic upgrade of the primary laboratory teaching facility at the British Columbia Institute of Technology (BCIT), located in Burnaby, B.C. The facility is part of a four-building complex surrounding a central court-yard area (Fig. 1). The intent of the project was the upgrading of the building to 100% of current code requirements as specified in the British Columbia Building Code (BCBC) 1992.

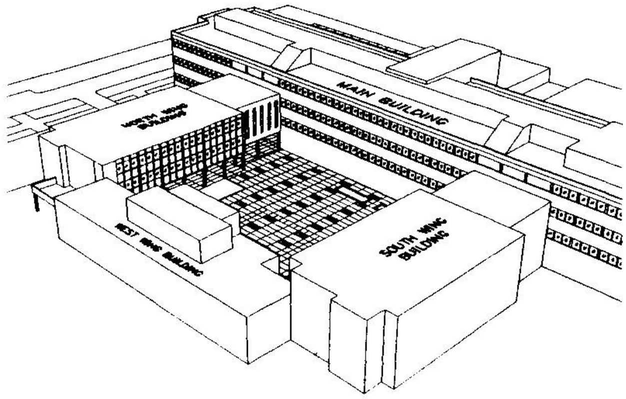


Figure 1 - SW1 Complex

2. EXISTING BUILDING

The Main Building (1962) is a four-storey structure constructed of 125 mm and 150 mm lightweight concrete floor slabs supported on steel beams and concrete encased steel columns. Conventional spread footings are founded on dense non-liquefiable soil. The existing lateral load resisting system consisted of four nominally reinforced concrete stairwells distributed along the building's length. The elastic capacity of various structural elements ranged from 20% to 40% relative to current code requirements.

3. PROJECT CONCEPT DEVELOPMENT

A previous seismic retrofit report for the building recommended incorporation of some 20 new internal reinforced concrete shear walls. The scheme, while sound in strengthening the structure, would have required phased construction over a two or three summer period. The estimate for this upgrade scheme of $3.3M CDN did not include any non-structural seismic restraint or other building improvements.

The project's conceptual focus was to meet the following objectives: 1, minimizing the project capital cost; 2, completing the project in one three-month period; 3, minimizing disruption to mechanical and electrical systems; 4, ensuring that the teaching laboratories would resume classes following summer recess; and 5, maximizing the aesthetic value of the completed project.

Five options were evaluated during the conceptual design stage: 1, strengthening the existing concrete stairwells and foundations; 2, providing new interior concrete shear walls; 3, incorporating new interior steel bracing; 4, using a mix of interior and exterior steel bracing; and 5, employing a combination of exterior steel bracing with exterior concrete shear walls.

After evaluation of all the options, the combination of external steel-bracing with external shear walls was selected (Fig. 2). This option best fulfilled all objectives at approximately 50% of the cost of the preliminary 20 internal shear wall retrofit concept. A benefit from the external retrofit scheme of the Main Building in Phase I would be the provision of some of the structural system upgrading required for the remaining three buildings of the SW1 complex proposed for Phase II.

Phase I's scope was to raise the level of structural resistance of only the Main Building to current building code standards but would also include restraint for all non-structural, mechanical, and electrical items. Phase II, would continue the seismic upgrade of the remaining three buildings. For the full seismic restraint requirement of Phase I, a three-sided steel bracing system together with external concrete end shear walls would be required. Since aesthetics were important to the success of this project, attention to detail required the services of an architect. Incorporation of repetitive visual softening into the details of the braced frame, in particular the gusset plate connections, challenged the designer and ensured the delivery of a non-industrial type bracing system appearance.

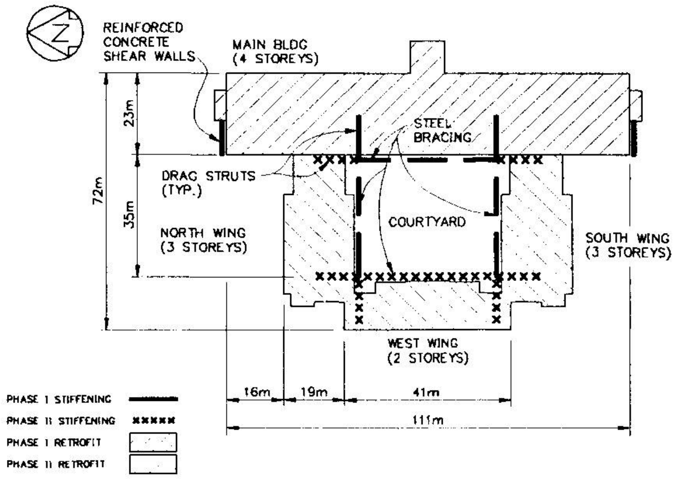


Figure 2 - SW1 Complex Retrofit Plan

4. DUCTILE BRACED FRAME DESIGN

A ductile-braced frame scheme with an R value of 3.0 was selected for several reasons. The ductility rating of this system would approach equivalence with that of the proposed reinforced concrete end walls having an R=3.5 and overall member force levels would be reduced significantly. However, greater demand in the design and detailing of connections would be required to ensure the necessary ductile behaviour. Reducing the force levels was the primary consideration because the irregularly long north-south plan dimension created a large torsional component.

A wide flange beam and column grillage incorporating hollow structural steel (HSS) bracing was selected as the preferred system. Several bracing arrangements were studied for the building faces requiring stiffening. Full concentric braced systems were compared to partial concentric braced systems. For the partially braced options force levels at the base would require HSS 305 x 305 x 13 braces for tension and compression resistance, however, the stringent width-thickness ratios for ductile braced frames and the extreme demand that would be placed on the foundations precluded their use. A more manageable pattern of forces would be transferred into the foundations by the full bracing options.

Design of exposed steel retrofits requires increased consideration of appearance. Of particular importance to this project were: 1, minimizing window coverage from connection hardware; 2, providing an unobtrusive appearance; 3, restricting erection to a small mobile crane; and 4, completing the project within one summer recess. From these reasons, the non-traditional consideration of full in-house detailed engineering of the connections evolved. It was soon apparent that other benefits would follow from such an engineering approach.

As most local steel detailers would be relatively unexposed to the concept of ductility provisions for connection design, the lack of pertinent experience would lead to an intolerable prolonged shop drawing approval process. Furthermore, a general design philosophy capable of producing connection symmetry and uniformity would require all the restraints recognized by the consulting design engineer. By providing a set of connection design loads on the contract set, as is the standard industry practice, the design concept conformance would not be ensured without the lead and commitment of the bracing systems engineer-of-record. It became clear that the design engineers' responsibility would require extension beyond the usual provision of connection design loads and specification of connection type and would have to include full details to comply with the intents of the code. Responsibility for the connection design could ensure complete control of aesthetic uniformity and avoid any industrial type connection appearance. The completion of the project necessitated a higher degree of participation by the consulting design engineer than traditional design projects warrant. Failure to provide this degree of commitment would otherwise compromise the aesthetic qualities of the project and more significantly, the project's schedule.

5. BRACING CONNECTION DESIGN

Selection of framing connections was contingent on the erection sequence of the members and minimization of field welding. End-plate connections on beams with bracing gusset hardware were selected since they allowed manageable erection for both the frame connections and the connections to the buildings. The gusset plates, which would receive the slotted HSS braces prepared with angle end clips, would enable quick erection bolting followed by the brace-to-gusset fillet field welding.

The project architect, having reviewed the size implications of the connection arrangement, requested a modification which incorporated the use of scalloped gusset plates. This architectural consideration would impose additional constraints on the engineering design of the project; not only were all the connections to be detailed, but a common geometric relationship between all joints for aesthetics was to be incorporated.

Intrinsic in the connection development was the provision of an out-of-plane yield zone to satisfy the codes stipulation of avoiding brittle failures on gussets through hinge formation upon brace buckling, (Fig 3). This plastic region on the gusset reported to be achieved [1] maintains the brace end back approximately two times the gusset plate thickness from a plane created by a line connecting the gusset plate vertical and horizontal extremities. Because of the predetermined hinge locations at either end of a brace, its effective length could be modified to approximately 80%. The slenderness reduction would be essential to design acceptance of smaller braces and to overstrength provisions of the code so that, consequential lower force levels would result for the connection and foundation designs.

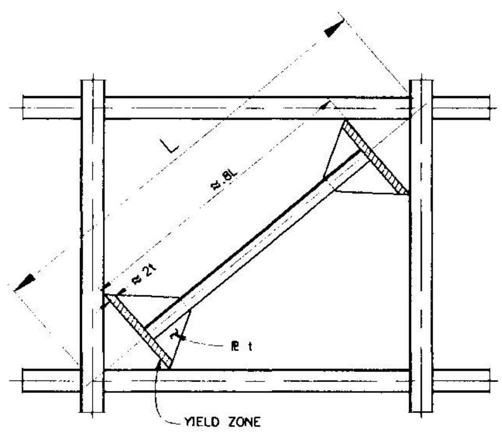


Figure 3 - Researched Out-of-Plane Yield Zones in Gussets

Design of a complete joint, i.e., top and bottom bracing connections on either side of the column centreline, was not straightforward. Each multiple joint condition was examined for the worst design case. The governing brace force per side of connection established the geometry of the entire connection so that symmetry could be approached. Overall joint force equilibrium was an essential component of the design of each element of the connection for the earthquake load condition.

For conformance of connection appearance, some controls on gusset geometry were established. Large shear and pass-through forces combined with minimizing connection sizes led to the selection of M24 diameter A490M bolts. The traditional 30° angle straight line distribution, for force dissipation into the gusset plate from the brace end connection was modified. The angle produced from the leading tip of the gusset plate within the slotted region of the brace would be checked to lie within a range of +/- 3° from 15°. Having established the basic required geometry, the trigonometric relationships necessary for calculation of the radii for the scallops could be solved.

DETAILING SCHEDULE				
GENERIC BRACE JOINT	JOINT #3			
	LEVEL	2	℄	J. L.
	L	R	T	B
	✓	✓	✓	✓
DIMENSIONS				
A	675			
B	762			
C	892			
D	664			
E	250			
F	520			
G	6			
H	450			
I	220			
J	100			
K	75			
L	65			
WELDS				
M1	10			
M2	8			
M3	5			
M4	10			
N	10			
O	5			
P	6			
Q	6			
R	CP			
PLATE THK.				
S	28			
T	12			
U	6			
MISC. INFO				
DOUBLER ℄'s (V)	EACH			
θ (DEG.) (W)	40.364			
ERECTION L (X)	L100x100x10			
BOLT # (Y)	25			
BOLT GRADE (Z)	A490M			

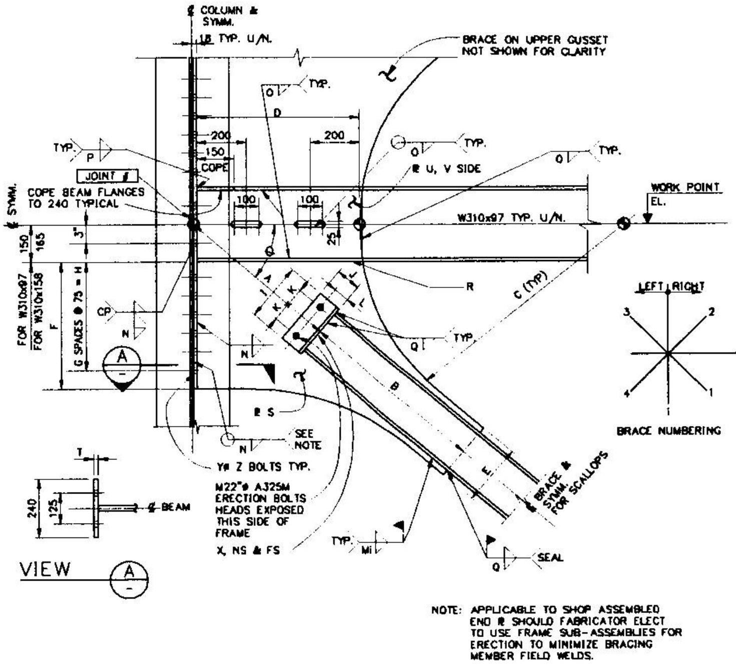


Figure 4 - Generic Bracing to Column Connection Details at Beams

For manageable repetition of the design procedure and to summarize the detail information required, a spread sheet program was developed. Iterative flexibility and geometric parameter sensitivity conditions were incorporated to assist the engineer. The final gusset plate geometry, plate and weld sizes, bolt layout, etc., necessary for complete production of fabrication drawings were tabled and referenced to a generic connection detail (Fig. 4).

6. FRAME-TO-BUILDING CONNECTION DESIGN

Connection design for brace members within a ductile frame is dealt with adequately for the design of new structures in S16.1. For the retrofit of existing buildings, in which new frames are incorporated into the overall structural system, specific design criteria for connection force levels between new and existing structural components is at the discretion of the designer. For this project, the overstrength provision, a crucial consideration in the detailing, ensured full resistance of the frames could be achieved. Connection design force levels of the frames to the buildings for each floor level under consideration, were first limited to twice the calculated earthquake shear. When this force presented an unreasonable number of anchors and their clustering interfered with their efficiency, a total floor shear force calculated on the buckling capacity of storey braces was substituted. Because of the optimization of the bracing sizes this overstrength limitation ranged between 1.3 and 1.7 of the design earthquake shears.

Horizontal reactions from the slabs at each floor level and the roof were transferred through drag struts to the frames along the north and south wings of the complex. Openings cut in the Main Buildings' walls allowed the fabricated struts to be installed from the courtyard. The top flange of each drag strut was bolted with adhesive anchors to the underside of the slab. For accommodation of variations in the soffit elevations, grout dry packed between the connecting flange and the slab ensured pure shear transfer in the anchors. A similar design philosophy as for the shear connections of the frames to the buildings was used for maximum connection force criteria selection.

7. CONCLUDING REMARKS

For this project, the decision of full in-house design of the connections provided the owner's solution within the time frame necessary. Control of aesthetics and integrity in the ductility provisions were maintained by this approach (Fig. 5).

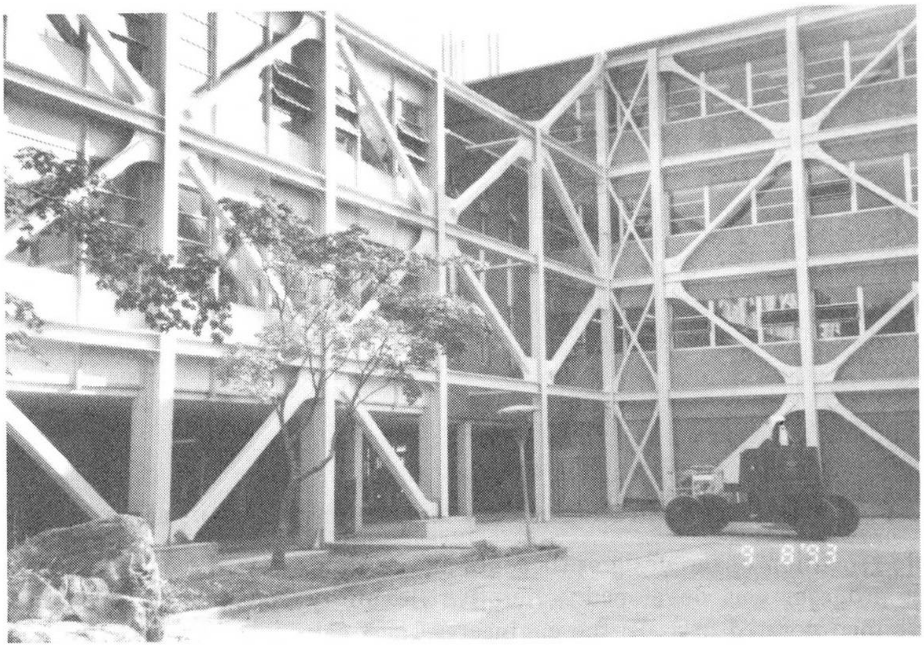


Figure 5 - Courtyard Perspective of Finished Frames

Ductility design requires clear understanding of system, member and connection performance. The existing practice of providing maximum force levels on drawings for detailers on seismic projects is inadequate especially in establishing equilibrium conditions for correct joint design for complex ductile structures. A significant portion of the success of the SW1 Main Building Seismic Upgrade is attributed to the "take-charge" attitude regarding the connection design. The result of this approach wholly complied with the intent of the codes, that is, the engineer-of-record being fully conversant with the performance specification of the connections carried their design through to the proper production of shop drawings by the fabricator. Responsibility for the project's elements was not separated from the engineer at a critical point in the design process.

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Seismic Behavior in Steel Structures of Weak Connections

Comportement sismique de structures présentant des assemblages faibles

Erdbebenverhalten von Stahlbauten mit schwachen Verbindungen

Akira MATSUO
Dr. Eng.
Hiroshima University
Higashi-Hiroshima, Japan

Akira Matsuo, born in 1947, received his Ph.D. at Hiroshima University, Japan, in 1985. He has been involved in design and research of steel structures for 25 years.

Yuji NAKAMURA
Dr. Eng.
Hiroshima University
Higashi-Hiroshima, Japan

Yuji Nakamura, born in 1936, got his Ph.D. at MIT 1968. He worked on the design of steel structures at NKK for 30 years. Now at Hiroshima University, he does a variety of research on steel structures.

Rafeek W. SALIB
Dr. Eng.
Suez Canal University
Port Said, Egypt

Rafeek W.Salib, born in 1958, received his Dr. Ing. at Hiroshima Univ., Japan, 1993. He has been involved in steel structures research work since 1983. Now he is a staff member in Suez Canal Univ., Port Said, Egypt.

SUMMARY

Simple formulation of panel moment-shear deformation for steel beam-to-column connections is presented. Dynamic analyses on low-rise steel frames of the weak column-strong beam are made changing the shear strength of the connection panels. According to the numerical results for the recorded earthquakes, as the shear strength of the connection panel decreases, the plastic deformation of each column decreases and may reach a level of less than half the strong panel cases.

RÉSUMÉ

Une formule simple est, proposée pour exprimer la déformation due au cisaillement dans des assemblages poutres-colonnes. Une analyse dynamique de cadres métalliques avec des colonnes faibles et des poutres fortes est effectuée en faisant varier la résistance au cisaillement des parois. Selon les résultats numériques enregistrés lors de séismes, lorsque la résistance au cisaillement de la paroi décroît, la déformation plastique de chaque colonne décroît et devient inférieure à la moitié de celle constatée pour des parois fortes.

ZUSAMMENFASSUNG

Der Bericht präsentiert zunächst eine einfache Formulierung der Beziehungen von Fachwerkfeldmoment und Scherungsdeformation bei Stahlträgern / Säulenverbindungen. Anschließend wird eine dynamische Analyse niedriger Stahlkonstruktionen mit schwachen Säulen-starken Trägerverbindungen durchgeführt. Die Ergebnisse für die verzeichneten seismischen Wellen legen nahe, dass die plastische Deformation der Säulen mit abnehmender Scherfestigkeit der Fachwerkfeld-Verbindungen zurückgeht.

1.INTRODUCTION

In the frame analyses deformation of the beam-to-column connection must be considered to know the exact behavior of the steel buildings. Many experimental results suggest that the strengths of the H-shaped beam-to-column connections are scattered in a wide range. Authers presented the formulations of the maximum strength for several types of connections through limit analysis[4]. In this paper these formulations are simplified for practical use. Then the formulation of the monotonic and cyclic curves of panel moment ${}_pM$ and shear deformation γ relation of the connection is presented. In Japan about 95% of steel buildings are low-rise. Based upon the Building Law in Japan most of them are designed according to the allowable stress design and the ultimate collapse form is not checked. In such a frame earthquake damage is sometimes concentrated at the weakest story. In this paper the dynamic analyses of 4 and 5-storied frames are made changing the shear strength of the connections to know the seismic performance against several recorded earthquakes.

2.RESTORING FORCE CHARACTERISTICS OF CONNECTIONS

2.1 The maximum strength

Authers presented the maximum strength of a standard type and a non-scallop type of an H-shaped steel beam-to-column connection for the interior column(X-type) and the exterior column(T-type)[5]. Eq.1 is the strength for the standard type.

$$ {}_pM_u=\tau_u V_p+2\pi\sigma_{ywb}t_{wb}a(a+b)/\sqrt{3}+4a(M_{pbf}+M_{pcf})/b+5.2(1+a/b)(M_{ubf}+M_{ucf}) , \quad (1)$$

where V_p, τ_u:the volume and the maximum shear stress of the panel plate, M_{pbf}, M_{pcf}, M_{ubf}, M_{ucf}:the plastic and ultimate bending moment of the beam and the column flange, σ_{ywb}, t_{wb}:yield stress and thickness of the beam web, a:the rigid length at the joint of the flanges and $b=\sqrt{2\sqrt{3}(M_{pbf}+M_{pcf}+1.3M_{ubf}+1.3M_{ucf})/\pi\sigma_{ywb}t_{wb}}$ according to the minimum condition. The first term shows the ultimate panel moment(shear strength) of the panel plate and the following terms the amount of the bending moment directly transmitted through the intersections of the beam and the column flange ${}_FM$, which looks as if the beam and the column reinforced the shear panel. This equation is applied to the 255 combinations of a practically used beam and column H-section(JIS SS400) and ${}_FM$ for each combination is plotted in Fig.1, where ${}_pM_y=\tau_y V_p$. ${}_FM/{}_pM_y$ mainly depends upon the geometrical conditions of subassemblages, as ${}_FM$ and ${}_pM_y$ contain the same material properties. Solid line shows the best fit curve Eq.2 in terms of panel yield ratio R_{py}.

$$ {}_FM/{}_pM_y=0.135/R_{py}+0.098, \quad (2)$$

where $R_{py}={}_pM_y/\sum M_{my}$, $\sum M_{my}$: the total of the yield moment of the beams or columns jointed to the connection, whichever the smaller. A small difference of ${}_FM$ is found between a standard type and a non-scallop type and between X-type and T-type for the practical range(R_{py}>0.5) of T-type[5]. General expression for the maximum strength of the beam-to-column connections of various materials is given as follows using Eq.2.

$$ {}_pM_u=\tau_u V_p+{}_FM =(1/\alpha+0.135/R_{py}+0.098)\,{}_pM_y, \quad (3)$$

where α: yield ratio of steel. Figs.2 show two groups of experimental and predicted maximum

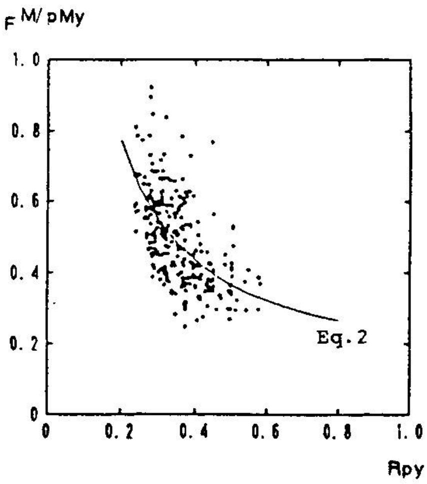

Fig.1 ${}_FM$ for practical sections (X-type with scallop) strengths of the connections depending upon α. Fig.2a contains the data of $0.59<\alpha<0.67$ and Fig.2b $0.67\le\alpha<0.75$. Each prediction is made using the average value of α.

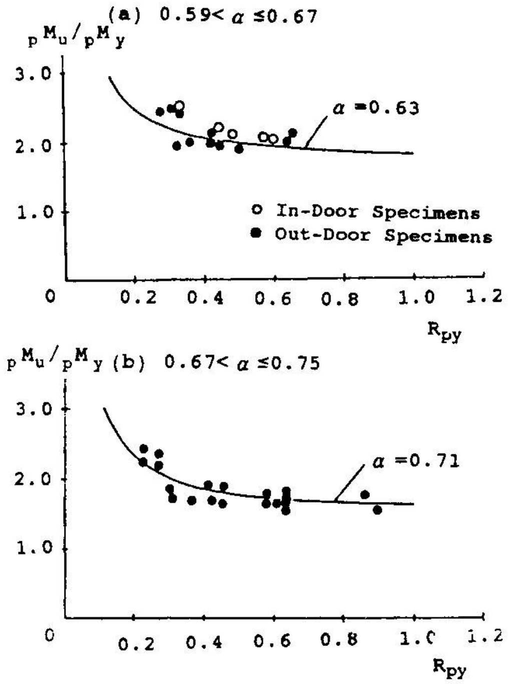

Fig.2 Predicted maximum strength

2.2 Panel moment and shear deformation relation

Monotonic panel moment ${}_pM$ - shear deformation γ relation of the connection is illustrated in Fig.3, where ${}_pM_{st}$, γ_y, γ_{st} and γ_u are given by Eqs.4 and 5 based on a tension coupon test. Cyclic curves can be established by connecting the positive and negative monotonic curves. Figs.4 and 5 show predicted and experimental ${}_pM$ - γ curves of various types of the connections.

$$ {}_pM_{st} = \tau_y V_p + {}_FM = (0.135/R_{py}+1.098)\,{}_pM_y \quad (4)$$

$$ \gamma_y = \tau_y/G,\quad \gamma_{st} = \gamma_y + \sqrt{3}(\varepsilon_{st}-\varepsilon_y), \quad \gamma_u = \gamma_y+\sqrt{3}(\varepsilon_u-\varepsilon_y) \quad (5)$$

3.DYNAMIC PERFORMANCE OF LOW-RISE STEEL FRAMES

3.1 Frames and earthquakes

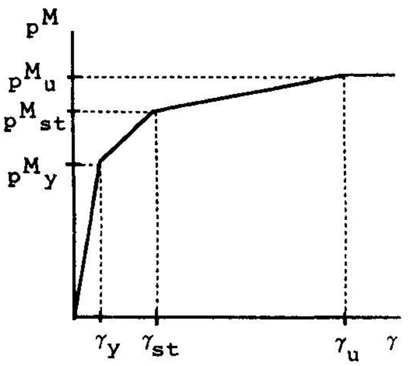

Fig.3 Assumed ${}_pM$-γ relation

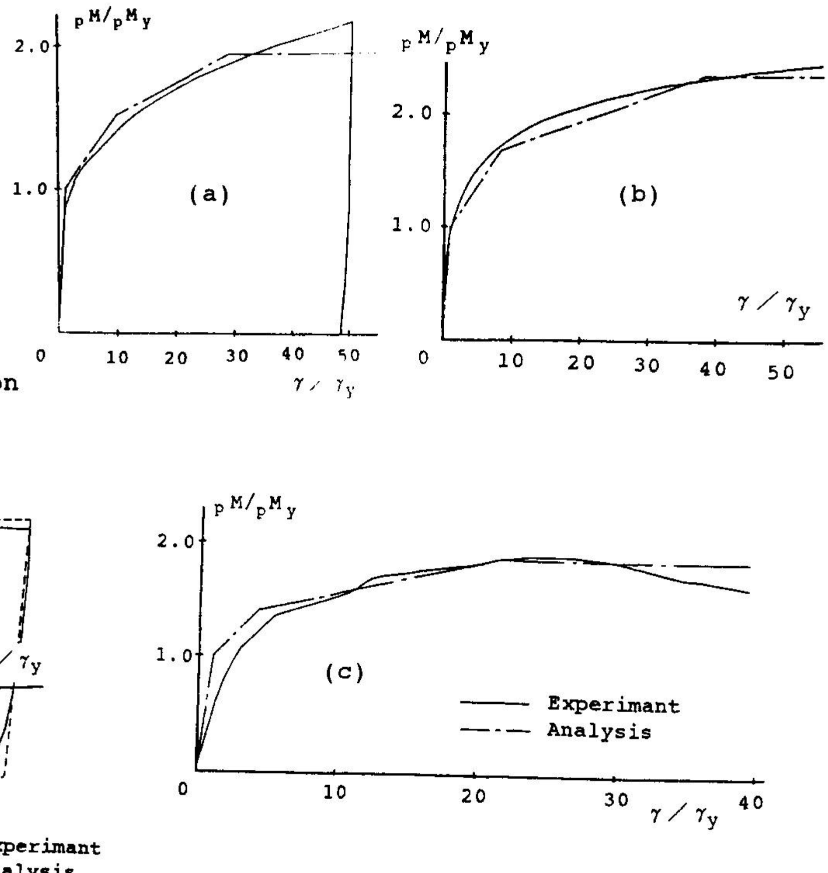

Fig.4 Predicted ${}_pM$-γ relations

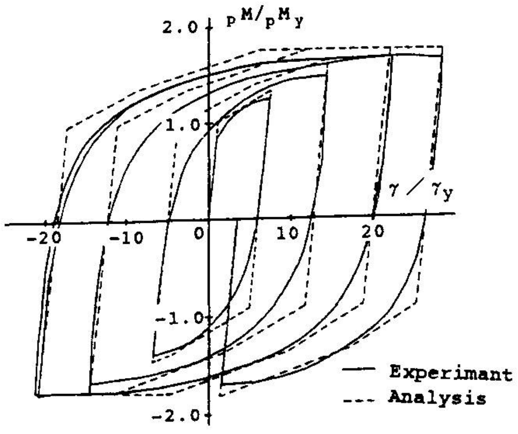

Fig.5 Predicted ${}_pM$-γ relations

For the first time, the sectional properties of the beam and the column for a 4-story 3-bay frame and a 5-story 3-bay frame are determined according to the Japanese aseismic design code(Route 2). A 5-story frame is named "A-frame" and a 4-story frame "B-frame". The sectional properties of the beam in A-frame are increased to 1.2~1.4 times as much as the minimum required value considering the composite effect of the slab concrete. As an analytical model the subassemblages are illustrated in Fig.6 consisting of the interior columns and the connecting beams cut off at their center. The section sizes of the beams and the columns are shown in Tab.1. The shear plate in H-shaped beam-to-column connection is usually reinforced through doubler plates so as to satisfy the allowable atress condition. However, the thickness of the doubler plates depends upon the existing shear stress, nominal thickness of the plate and intension of the designers. Various combinations of strengths(R_{py}=0.5,0.7,0.9,1.2) of the panel plate for each story are selected. The naming rule of the frame is illustrated in Fig.7. The first natural period for these frames are listed in Tab.2.

The frame analytical method is based on the hybrid type complementary energy's principle, where the bi-linear stress-strain relation($E_{st}/E=1/100$) is assumed for a flexural member and $10\gamma_y$ for γ_{st} and $40\gamma_y$ for γ_u in $_pM-\gamma$ relation for the panel plate. Newmark's β method(β=0.25, Δt=1/400sec) is used for numerical integration in dynamic analyses. Two earthquakes recorded in Japan(Miyagi Oki Earthquake 1978.6--MIYAGI NS max acc 350,400gal and Tokachi Oki Earthquake 1968.4--HACHINOHE NS max acc 350gal) are adopted.

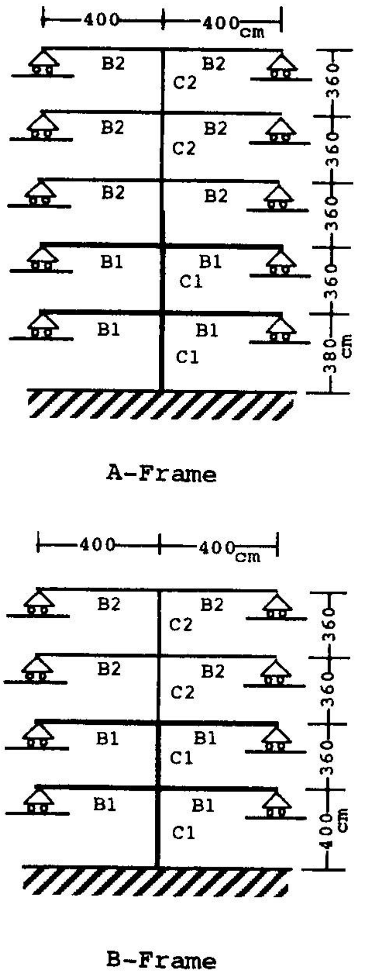

Fig.6 Analytical model

	A-Frame	B-Frame
C1	H-400×408×21×21	H-388×402×15×15
B1	H-582×300×12×17	H-506×201×11×19
C2	H-350×357×19×19	H-440×300×11×18
B2	H-606×201×12×20	H-496×199×9×14

Tab.1 Sections of the beams and the columns

Specimen	T(sec)	Specimen	T(sec)
A-11112	1.120	A-33333	1.069
A-11222	1.108	A-33444	1.062
A-11333	1.100	A-44222	1.077
A-22112	1.100	A-44333	1.059
A-22222	1.088	A-44444	1.052
A-22333	1.108	B-1112	0.969
A-22444	1.073	B-2222	0.944
A-33112	1.088	B-3333	0.929
A-33222	1.077	B-4444	0.920

Tab.2 Natural period of the frames

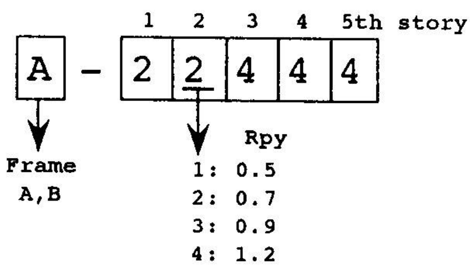

Fig.7 Naming rule of the frames

3.2 Analytical results

Ratio of the energy absorbed by the beams, columns and the connection panels is illustrated in Fig.8. Most of the energy is absorbed by the beams and columns in case of the strong panel A-44444, B-4444(R_{py}>1.0). Around 30% of total energy is shared by the panels in case of A-33333, B-3333(R_{py}=0.9) and more than half in case of $R_{py} \leqq 0.7$. Figs.9a and 10a are the maximum story drift angle R for the frame of equal R_{py} for each story. Figs.9b,c and 10b,c are the total deformation η of the panel and the column, where η_p=a half of the energy shared by the panel/${}_pM_y \cdot \gamma_y$, η_c=a half of the energy absorbed by the column/$2M_p \cdot \theta_p$ and M_p, θ_p: full plastic moment and corresponding elastic deformation of the column. R and η_c of the 4th and the 1st story in A-44444 and B-4444 subjected to MIYAGI NS are much larger than that of other stories, which suggests the local collapse in these stories. The maximum value of η_c is 6 for A-Frame and 15 for B-Frame. However, when R_{py} decreases, R of the collapsed story becomes smaller and that of other stories larger. This corresponds to decreased η_c in the 1st and 4th story and increased η_p over the whole frame. η_c for $R_{py} \leqq 0.7$ is less than half of that for R_{py}>1.0. Figs.11 and 12 are the numerical results for the frames with panels of different R_{py}. When this difference is small, such as A-22333, dynamic performance is almost the same as that of the corresponding frame of equal R_{py} such as A-22222. However, in case of large difference of R_{py}, such as A-33112, A-44222, R and η_p sometimes become large in spite of small η_c.

4.CONCLUSIONS

1) Strengths and panel moment ${}_pM$ - shear deformation γ relation of the H-shaped steel beam-to-column connection are successfully formulated.
2) In the dynamic analyses of the frames of the weak column and the strong beam, the drift R of the story which collapsed for R_{py}>1.0 becomes smaller, when $R_{py} \leqq 0.7$.
3) The column in the 1st story for R_{py}>1.0 is often damaged so much. The maximum deformation of the column η_c is ranged 6 to 15. However, η_c for $R_{py} \leqq 0.7$ decreases to less than half of that for R_{py}>1.0.
4) In case of much different R_{py} within the frame(A-44222,A-33112), drift R and deformation of the panel η_p become sometimes large.

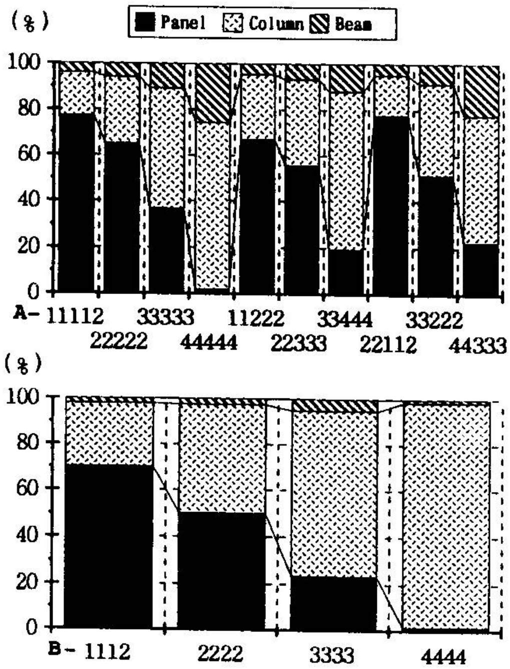

Fig.8 Energy absorbed by each member (MIYAGI 350gal)

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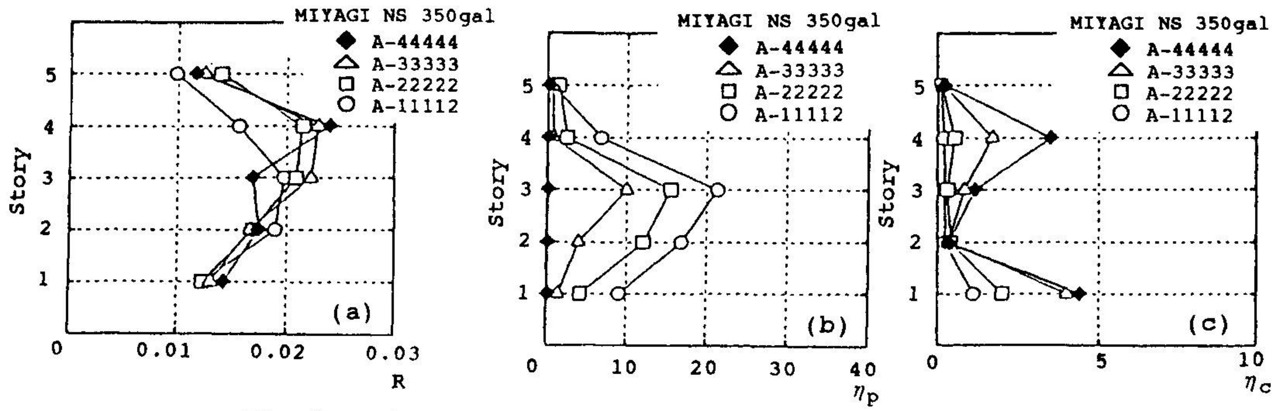

Fig.9 Maximum response of A-frame(MIYAGI 350gal)

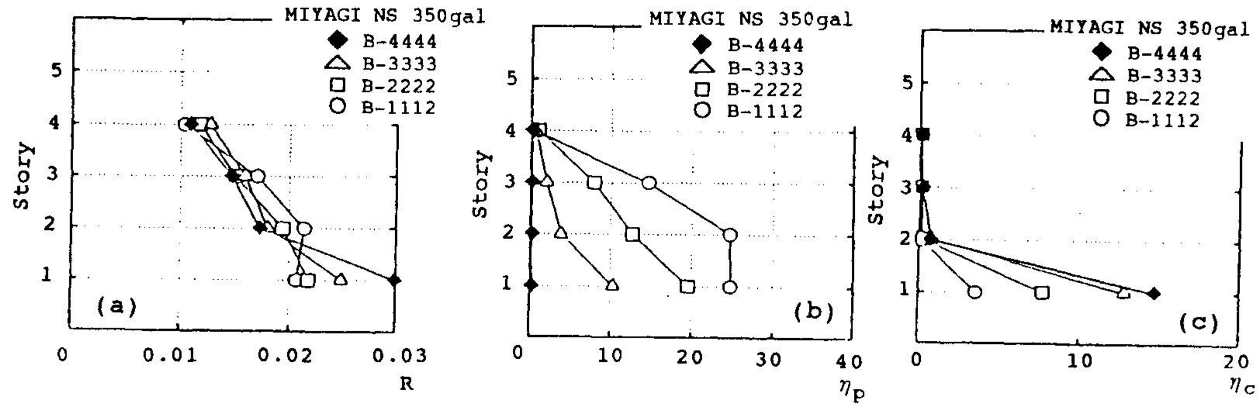

Fig.10 Maximum response of B-frame(MIYAGI 350gal)

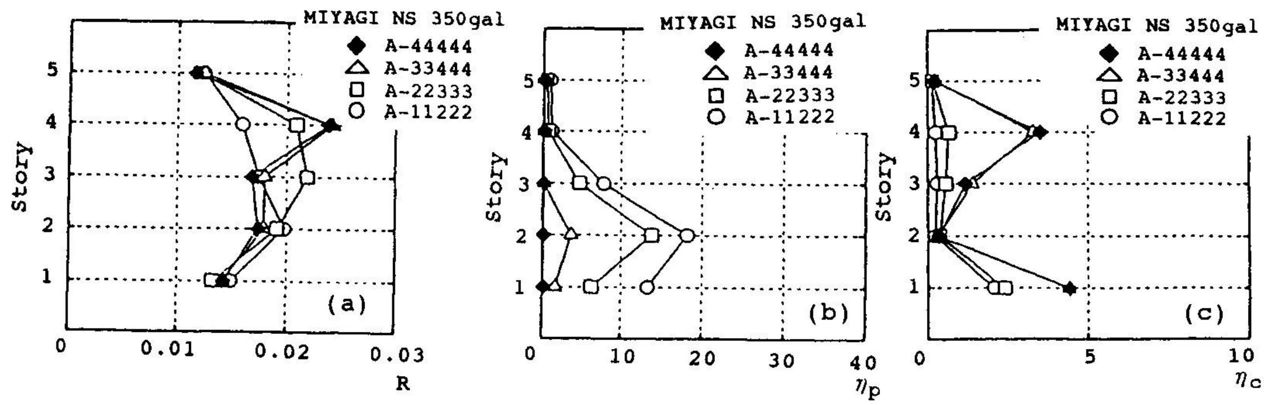

Fig.11 Maximum response of A-frame(MIYAGI 350gal)

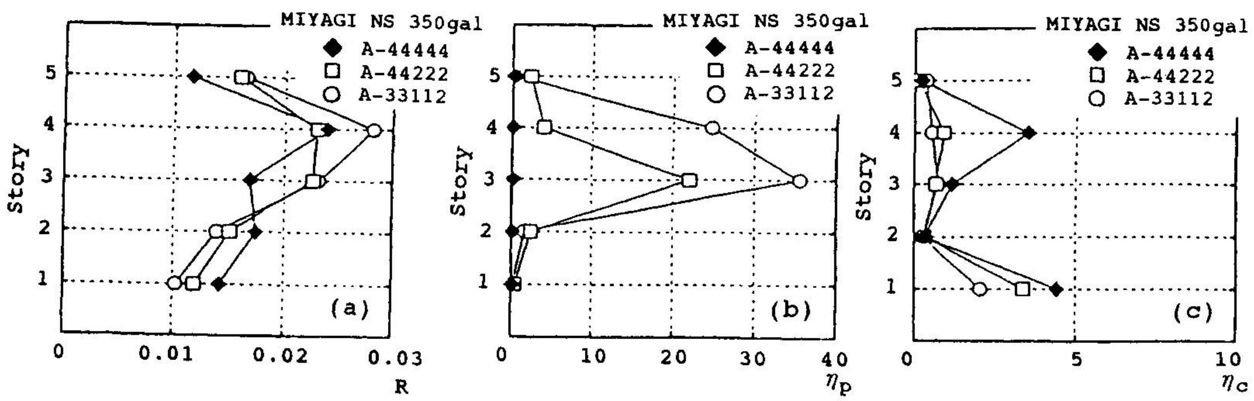

Fig.12 Maximum response of A-frame(MIYAGI 350gal)

Seismic Retrofit of Moment Resisting Frame with Viscoelastic Dampers

Consolidaton parasismique d'un cadre rigide au moyen d'amortisseurs viscoélastiques

Erdbebenertüchtigung eines biegesteifen Rahmens mittels viskoelastischem Dämpfer

Javeed A. MUNSHI
Former Visiting Researcher
Lehigh University
Bethlehem, PA, USA

Javeed Munshi received his Ph.D. in 1993 from Illinois Institute of Technology, Chicago, IL. For four years he has worked on seismic damage mitigation and retrofit of buildings and bridges using base isolation and added damper devices.

Kazuhiko KASAI
Associate Professor
Lehigh University
Bethlehem, PA, USA

Kazuhiko Kasai received his Ph.D. in 1985 from University of California, Berkeley. He has been working on several earthquake engineering topics such as steel EBF systems, seismic pounding of adjacent structures, and supplemental damping devices.

SUMMARY

This paper discusses seismic retrofit of a weak moment-resisting steel frame by viscoelastic dampers. The viscoelastic damper behavior is accurately simulated by using a nonlinear element which takes into account the frequency and temperature dependency of the damper's material. The performance of the frame with and without dampers is investigated under four different earthquake excitations. Both elastic and inelastic member responses are investigated. The elastic response is simulated through a modal method. A simple method based on drift control is proposed for retrofit of weak frames.

RÉSUMÉ

L'article traite de la consolidation parasismique d'un cadre rigide sous-dimensionné au moyen d'amortisseurs viscoélastiques. Le comportement de l'amortisseur viscoélastique est simulé de façon précise à l'aide d'un élément non linéaire qui courante de la fréquence et de la relation de la fréquence et de la température avec les matériaux de l'amortisseur. La performance du cadre est étudiée pour quatre cas de charges sismiques, avec et sans amortisseurs. Le comportement élastique et inélastique des membrures est étudié. Le comportement élastique est également simulé au moyen d'une méthode modale. Une méthode simple basée est proposée pour la consolidation de cadres faibles.

ZUSAMMENFASSUNG

Der Beitrag behandelt die Erdbebenertüchtigung eines unterdimensionierten Stahlrahmens durch viskoelastische Dämpfer. Deren frequenz- und temperaturabhängiges Verhalten wird mittels eines nichtlinearen Elements rechnerisch simuliert. Das Verhalten des Rahmens mit und ohne Dämpfer wird für vier verschiedene Erdbebenanregungen untersucht. Neben dem inelastischen wird auch das elastische Verhalten, in modaler Analyse, simuliert. Für die Nachrüstung der Rahmen wird eine einfache Methode auf der Basis der Stockwerksauslastung als Steuerparameter vorgeschlagen.

INTRODUCTION

The Northridge earthquake of January 1994, by causing fracture damage of numerous moment resisting connections, indicated serious safety and cost implications of steel moment resisting frames. The safety concern of such frames calls for a retrofit scheme, which could ensure safety of both the occupants and the structure under a major earthquake, fulfil the serviceability, and be economical in the long run. Retrofit using VE-dampers result in well controlled response such as significantly small drift and member forces, and relative invariance of response to earthquakes of different characteristics. This paper will focus on the effectiveness of VE-dampers for seismic retrofit of MRFs and address the various issues associated with it.

STIFFNESS AND DAMPING

The key of the retrofit is to limit the story drift ratios to well within 1% so that strength demand of the members is well controlled. In order to estimate the amount of stiffness and damping required for this, consider a single degree-of-freedom (SDOF) system and its acceleration spectrum LMNO for nominal damping (e.g. 3%) as shown in Fig. 1(a). Locate the equivalent elastic force (point A) corresponding to the original building period T_o which is assumed to lie in the constant velocity region of the spectrum. Now locate the corrected elastic force (point B') for the shifted period T of the VE-damped building on a high damping spectrum L'M'N'O'. Points A, A', and B' are also shown in Fig. 1(b) representing the approximate elastic strain energies at peak deformation of the building. Point A' corresponds to force F' and deformation Δ', a situation when only stiffness due to VE-dampers is added to the structure. From Fig. 1(b) the approximate energy contributed by VE-dampers at peak structure deformation is the area under line A'B'. Thus,

$$1/2K\Delta'^2 - 1/2K\Delta^2 = \pi F_d \Delta \tag{1}$$

where $K = K_o + K_d'$, i.e stiffness of original MRF plus storage stiffness of the added dampers. The right hand side is the area of ellipse with equivalent energy, F_d = peak damper force, and Δ_d = peak damper deformation. Here $\Delta_d \approx \Delta$ is assumed by neglecting the brace and column axial deformation. Under the constant velocity, $\Delta' = \sqrt{K_o/K}\,\Delta_o$. Using $F_d = \eta K_d'\Delta$, where $\eta_d = K_d''/K_d'$ = loss factor of VE-material (Kasai et al, 1993), the following equations result:

$$\frac{K_d'}{K_o} = \frac{(\Delta_o^2/\Delta^2 - 1)}{1 + 2\pi\eta_d}, \quad \text{and} \quad \xi = \frac{\eta_d}{2}\frac{K_d'}{K} = \frac{\eta_d}{2}\left[\frac{1}{1 + K_o/K_d'}\right] \tag{2}$$

The force and deformation reduction from F' and Δ' to F and Δ, respectively (Fig. 1(b)) become:

$$\frac{\Delta}{\Delta'} = \frac{1}{\sqrt{(1 + 4\pi\xi)}} = \frac{F}{F'} \tag{3}$$

Note that in Eq. 3 the reduction factor is determined based on the shifted structure period T. Eq. 3 well predicts the reduction of response when compared to those proposed by Newmark & Hall (1973), Kawashima & Aizawa (1986) and also that recommended by NEHERP for passive dissipation system as shown in Fig. 2.

RETROFIT METHOD

Based on the above simplifications, the following method is proposed as a general retrofit scheme of MRF using linear analysis approach:

1. Determine the story drifts (Δ_o) of the unretrofitted MRF under the design earthquake assuming elastic behavior by either modal analysis or equivalent lateral force analysis method.
2. By using ratio of the drift limit (Δ) and the original drift (Δ_o) as well as Eq. 2, determine

required stiffness ratio K_d'/K (Eq.2).

3. Determine the period T of the retrofitted building and its total global damping ratio by static lateral force method (Kasai et. al, 1994).
4. Obtain story drifts of the VE-damped structure for the design earthquake with the damping ratio and compare the drifts with the design/desired drifts. If the drifts are not within limits, revise added stiffness and damping locally if needed (Eq. 2), and go back to step 2.
5. Determine the forces in the members and ensure strength.

UNRETROFITTED MRF

The study considers the 10-story MRF as shown in Fig. 3(a), designed by Anderson and Bertero (1969), satisfying the code minimum strength requirement but intentionally disregarding the code drift limit. The frame has small sections and does not have overstrength. It is very flexible having a long fundamental vibration period of 2.44 sec. El Centro (1940) scaled 1.5 times (peak ground acceleration of 0.52g), Artificial earthquake with spectrum characteristics compatible with NEHERP design spectrum (0.4g), Hachinohe earthquake (Japan), scaled 2 times (0.44g), and Parkfield earthquake (0.49g) are used for the analysis. The analysis assumes rigid floor diaphragms. The steel frame is assumed to have 3% viscous damping.

Elastic Building Case. Fig. 5(a) shows that building develops maximum displacement of 20 to 30 inches and very large story drift ratios of 2% to 3% under the 4 stipulated earthquakes.

Inelastic Building Case. For 5% strain hardening stiffness of steel members and yield stress of 36 ksi, the frame develops large plastic hinge rotations of over 0.015 rad. in columns and beams (Fig. 3). The displacements reach 30 in. with drift concentration of about 3% at several floors (Fig. 6(a). The responses indicate a biased motion and chances of incremental collapse for the building. The column moment and axial forces reach their limiting capacities and indicte a severe ductility demand as understood by comparing with elastic analysis (Fig. 5(a)).

RETROFITTED BUILDING

For limiting the drift ratio to under 1%, against the drift ratio of 2.9% (El Centro), a damper stiffness $K_d' = 0.8\ K_o$ is supplemented as per Eq. 2 at each floor (Fig. 4(a)). The loss factor η is not very sensitive to vibration period (3M ISD110 material). Table 1 lists the additional stiffness and the corresponding damper area required at each story for 1 inch thickness of layer and 1.9 second vibration period of VE-frame (24°C). The braces are sized to be at least 10 times stiffer than the VE-damper to ensure $\Delta_d \approx \Delta$ [Munshi & Kasai, 1994]. Nonlinear analyses of the VE-frame is carried out using writers' VE-finite element which accounts step-by-step for the frequency and temperature induced nonlinearity of response [Kasai, et al, 1993].

Elastic Building Case. Fig. 5(b) shows peak displacement of only 10 to 12 inches, with uniform drifts of less than 1% through the height of buildings. About 20% reduction of column axial forces and 70% reduction of column moments (Fig. 5(b)) significantly reduces the strength demand on the members and connections of the moment frame, which could enhance fatigue and fracture performance of MRFs.

Inelastic Building Case. Negligible inelastic activity of the building is seen with incorporation of this damper configuration as shown in Fig. 4(b). The building behaves almost in elastic manner under the 4 earthquake excitations, and response envelops of the building accordingly are very similar to those of the elastic building as shown in Fig. 6(b). The member forces are well within their corresponding capacities setting aside the chances of damage.

Linear Response Simulation. Though stiffness and damping contribution of VE-dampers will actually vary during an earthquake excitation of the VE-building, reasonable prediction of added stiffness and damping can be made based on its fundamental vibration period. The equivalent viscous damping ratio of the VE-damped building can be determined by using static lateral force method assuming single mode approximation [Kasai et al, 1994]. The VE-damped structure is simulated through Modal Strain Energy (MSE) approach [Munshi and Kasai, 1994], with 16% damping for all vibration modes. Fig. 7 shows that displacement, drift, and column axial forces, are well predicted through the linear approach when compared with nonlinear analysis using the VE-hysteretic element. Note, however, that the damper forces and brace forces predicted through the linear approach are significantly underestimated (Fig. 7) and need to be corrected (see proposed correction method, Munshi and Kasai, 1994).

Effect of Temperature. In the present analysis (24°C), an average maximum temperature rise of 3 to 4°C does not alter global building responses significantly [Munshi and Kasai, 1994]. Analyses of the VE-damped building at 16°C and 32°C shows different responses which are well predicted by modifying the 24°C responses by damper stiffness correction for that temperature.

CONCLUSIONS

Viscoelastic dampers significantly reduce strength and deformation demand on the members and connections and are effective for retrofitting the existing weak and vulnerable moment frames for enhanced performance under strong earthquakes. The amount of stiffness and damping needed can be reasonably estimated based on drift control strategy proposed herein. A simplified linear approach is proposed as a practical analytical method for VE-damped frame.

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Table 1. Retrofit Scheme of Moment Resisting Frame.

Story No. i	Story Stiffness K_o (Kip/in)	Story Drift Ratio Δ_o/h (El Cent) %	Target Drift Ratio $\Delta/h \leq .01$ %	Required Damper Stiffness $K_D'=.8K_o$ (Eq. 2) (Kip/in)	Required Damper Area (in^2) $A=K_D't/G'$	Damper Size Provided (in^2)	Damper Stiffness Provided K_D' (Kip/in)	Story Drift Ratio (Linear Analysis) %	Actual Story Drift Ratio (Nonlin) %
10	21.3	2.25	≤1.0	17.2	111	100	15.0	0.62	0.54
9	27.0	2.88	1.0	21.8	140	100	15.0	0.91	0.82
8	35.8	2.94	1.0	28.9	187	200	30.0	0.92	0.82
7	40.9	2.91	1.0	33.0	213	300	45.0	0.89	0.80
6	48.9	2.62	≤1.0	39.5	255	300	45.0	0.84	0.81
5	55.0	2.15	≤1.0	44.5	287	400	60.0	0.75	0.75
4	62.4	1.97	≤1.0	50.4	325	400	60.0	0.76	0.73
3	92.2	2.26	≤1.0	74.5	481	500	75.0	0.73	0.66
2	77.3	2.63	≤1.0	62.5	403	500	75.0	0.77	0.68
1	82.9	2.07	≤1.0	67.0	432	500	75.0	0.60	0.52

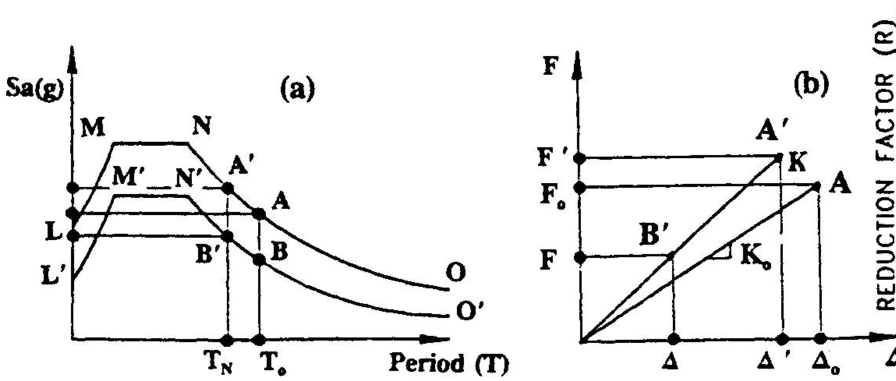


Fig. 1 (a) Schematic Spectrum for VE-System, and (b) Energy Idealization.

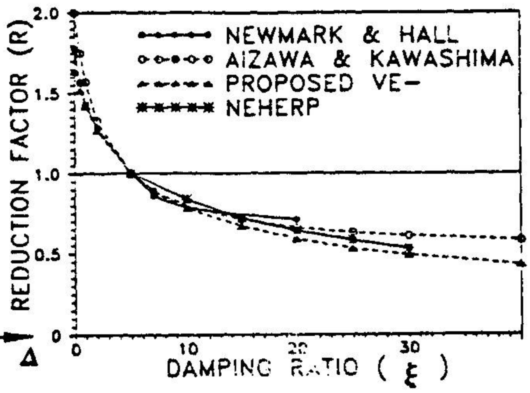


Fig. 2 Response Modification Factor for VE-System.

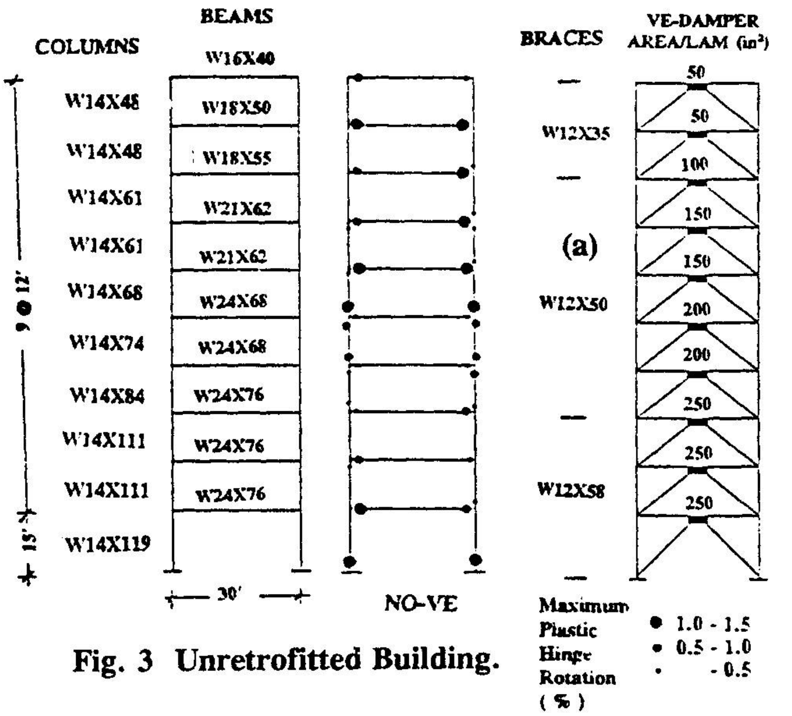


Fig. 3 Unretrofitted Building.

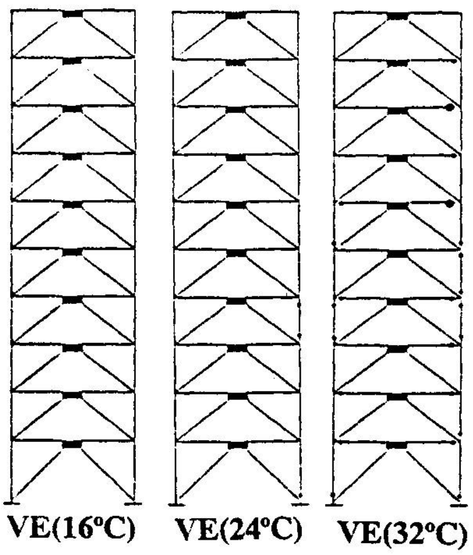


Fig. 4 Retrofitted Building.

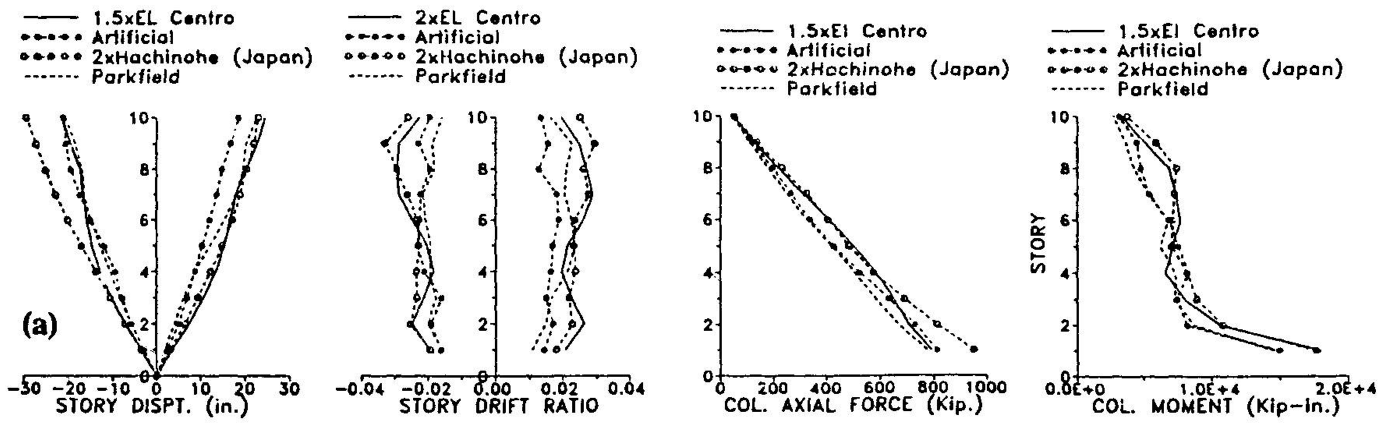


Fig. 5 Elastic Building: Global and Local Responses of (a) Unretrofitted and, (b) Retrofitted Building (Ambient Temperature 24°C).

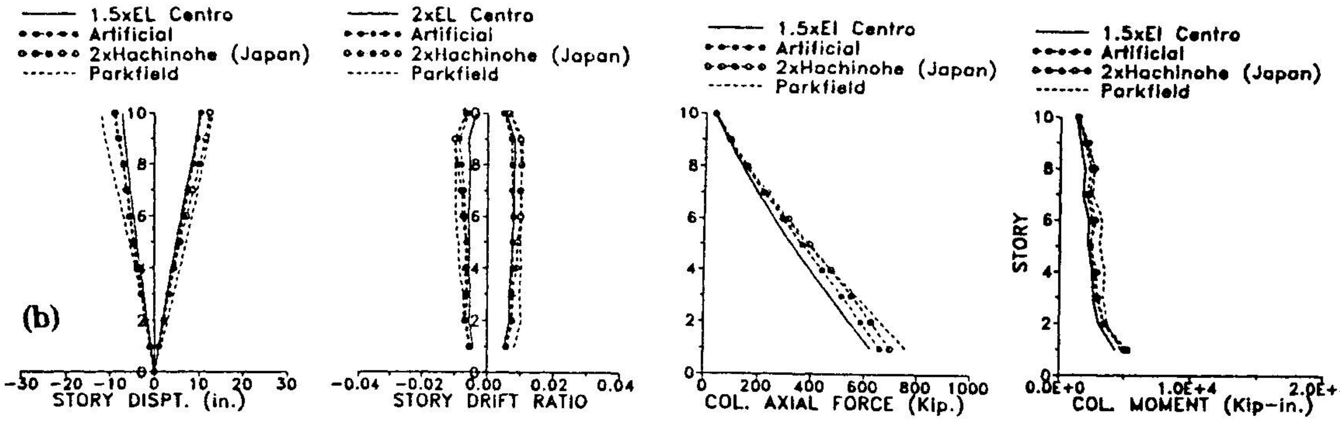


Fig. 5 Elastic Building: Global and Local Responses of (a) Unretrofitted and, (b) Retrofitted Building (Ambient Temperature 24°C).

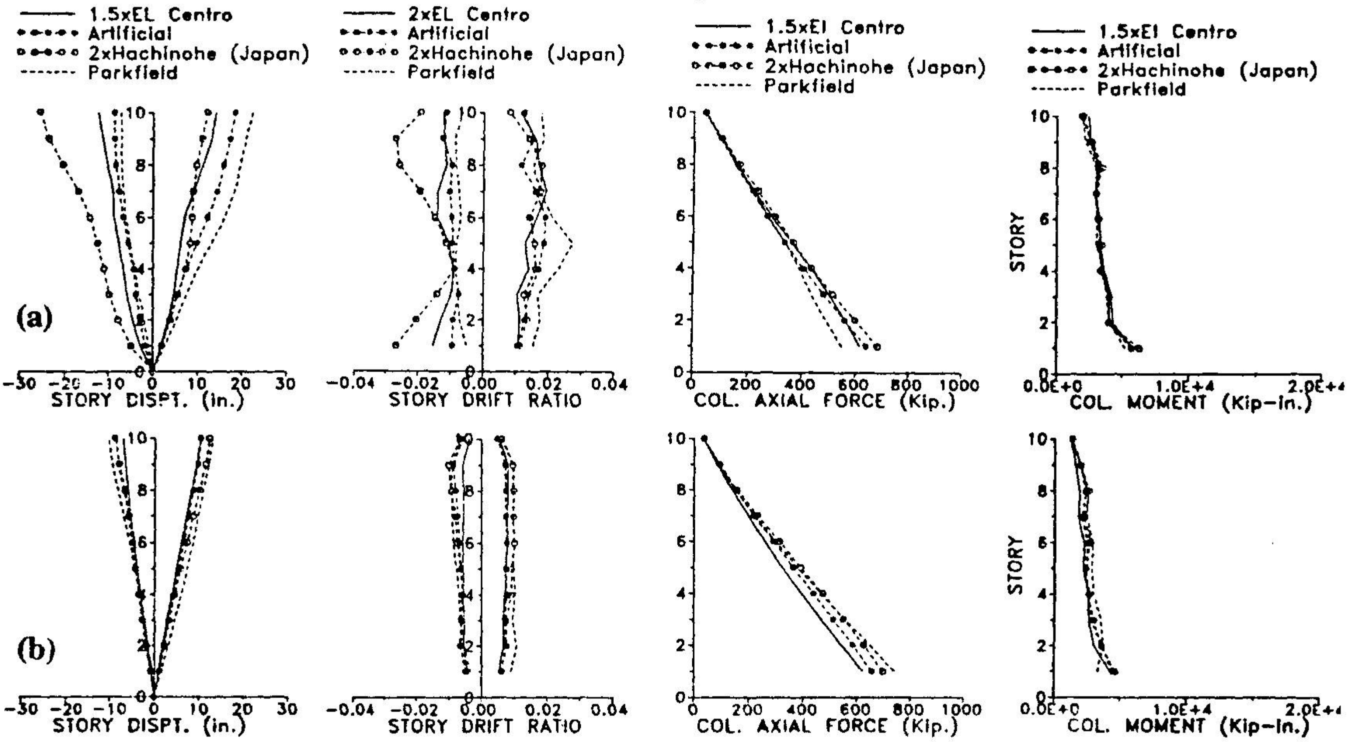


Fig. 6 Inelastic Building: Global and Local Responses of (a) Unretrofitted and, (b) Retrofitted Building (Ambient Temperature 24°C).

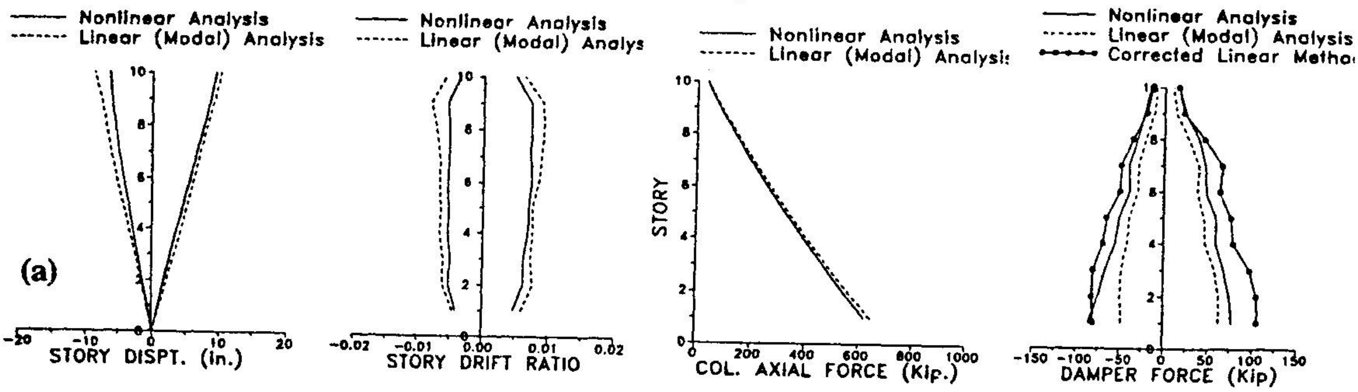


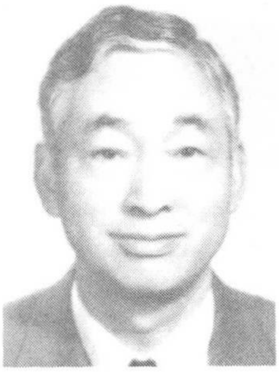
Fig. 7 Linear Response Simulation for El Centro earthquake (24°C).

Seismic Strengthening of Structures

Renforcement des structures contre les effets sismiques

Seismische Verstärkung von Tragwerken

Zhixin XU
Professor
Tongji University
Shanghai, China



Zhixin Xu, born in 1929, received his B.S. degree in civil engineering at Jiaotong University, Shanghai, in 1949. For thirteen years he served in a structural design office. He completed his graduate study at Tongji University in 1964. Since then he has been professor at Tongji University.

SUMMARY

Methods of seismic appraisal and strengthening are presented. Firstly the level of seismic resistance is addressed. It is determined through specific study for important structures and by following code for ordinary structures. The author emphasises his view that the same safety requirements are preferable for residential and public buildings. Three cases of seismic strengthening including public buildings and important hazardous structure are presented.

RÉSUMÉ

L'auteur présente des méthodes d'évaluation et de renforcement de structures vis-à-vis des tremblements de terre. Il définit l'importance du niveau de résistance parasismique au moyen d'études spéciales pour des constructions importantes et de règles pour les constructions habituelles. L'auteur estime que les immeubles d'habitation et les bâtiments publics devraient satisfaire aux mêmes exigences de sécurité. Il présente trois exemples pratiques, permettant d'améliorer le comportement aux effets sismiques des bâtiments publics et à haut risque.

ZUSAMMENFASSUNG

Es werden Verfahren zur Beurteilung und Verstärkung gegenüber Erdbeben vorgestellt. Zuerst wird das erforderliche Niveau der Erdbebenvorkehrungen angesprochen, das sich aus speziellen Studien für wichtige Bauten und Normenbestimmungen für gewöhnliche Bauten herleitet. Nach Meinung des Verfassers sollten Wohnbauten und öffentliche Bauten die Sicherheitsanforderungen für heutige Neubauten erfüllen. Drei praktische Beispiele für die seismische Ertüchtigung öffentlicher Bauten und wichtiger Risikotragwerke werden aufgeführt.

1.INTRODUCTION

It is not unusual that the seismic hazard of a region is identified later higher than the previous assigned level. Seismic strengthening has to be implemented in order to keep buildings and structures safe during future earthquake. During 1966 to 1976 three destructive earthquakes took place in Xingtai, Haicheng and Tangshan. These regions were zoned then as region of fundamental intensity VI in Chinese scale which is almost equivalent to MMI. The epicentral intensity of Tangshan earthquake is XI. For other two earthquakes,the epicentral intensity reaches IX. Since then seiemic hazard of many cities has been reassessed, and fundamental intensity of many a city has been raised. Among them is Shanghai where the fundamental intensity is raised to VII from the original VI. A large scale of seismic appraising and strengthing has been being carried out for various buildings and structures in these regions. The author has been being involved in seismic appraising and design of strengthening. Some important features regarding these works are presented here.

2.LEVEL OF SEISMIC PROVISION

The first problem of seismic appraising and strengthening is the level of seismic provision. This of course depends on the seismic hazard of the region. The seismic hazard of a region is assessed in China mainly by means of the method similar to those proposed by Cornell[1] and Ang[2]. The main points are as follows. For causative faults or areas near a site the probability of occurrence of earthquake of various magnitude is identified. Poisson process is usually adopted for simplicity. Richter's formula for magnitude distribution is used. Also assumed is that the rupture length is related to the magnitude of an earthquake which can take place everywhere along the causative fault,thus the shortest distance from the rupture to the site can be determined. Using some attenuation relationship which defines the ground motion as a definite function of magnitude and rupture distance the seismic hazard of a site is assessed. The result is the values of a certain ground motion parameter,usually maximum acceleration corresponding to various probability of exceedance within a certain period of time.
Theoretically the optimal seismic level of strengthening can be determined through minimization of total cost

$$C_t(a)=E[L(a)]+S(a)$$

where E[L(a)] is the expected earthquake loss under ground motion with maximum acceleration a, S(a) is the cost of strengthening, both reduced to present value. Actually such optimization can hardly be conducted not only due to time consuming but also less reliablity, since both the probability of occurrence of the maximum acceleration and the loss can only be assessed very roughly and with low reliablity.
The practice in China is that for appraising and strengthening the level of seismic provision is specified somewahat lower than designing a new one. For example, the seismic appraising standard (draft) specifies that the earthquake resistant strength can be 15% lower than the design code requirement for reinforced

concrete structure and steel structure. For masonry structure the standard specifies that the required strength of brick wall in region of intensity VIII and IX is 1.5 and 2.5 times that required in region of intensity VII. It is obvious that the safety level for appraising in region of intensity VIII or IX is much too low since the earthquake action in these regions is twice and four times that in region of intensity VII. In the author's view deliberately lowering the safety requirement is inadvisable. Unless careful study is conducted the same safety level as design code is preferable.

3.METHOD OF APPRAISING AND STRENGTHENING

3.1 Masonry building

This type of building is the most vulnerable during earthquake and is the most frequently used for residential buildings in China. The main features determine the earthquake resistant capacity is the layout, regularity of plane and elevation , cross sectional area of transverse and longitudinal bearing walls, height, quality of brick, mortar and construction, width of pier between windows and at corners, bearing length of slab and roof, arrangement of tie beams and so on. According to appraising standard (draft) the seismic resistant capacity is expressed as

$$\beta_{c_i}=\frac{A_i}{A_{b_i}\xi_{o_i}\lambda}\psi_1\psi_2$$

If the smallest value of β_{ci} for the weakest storey,weakest sector is larger than 1, the building is evaluated as safe. A_i is the net cross sectional area of transverse or longitudinal walls at half storey height of i-th floor. A_{bi} is the floor area of i-th floor. ξ_{oi}, an empirical coefficient depends on mortar strength, number of floor level and storeys. λ=0.7,1.0,1.5 and 2.5 for regions of seismic intensity VI,VII,VIII and IX respectively. ψ_1 is a coefficient taking care of the layout and construction quality, ψ_2 ,a coefficient taking care of details.
As has been pointed out by the author, λ is inadequate . The author uses the safety criterion given in the Design Code,which specifies

$$V\leq\frac{R}{\gamma_{RE}}$$

where V is the shear due to earthquake excitation, R is the shear strength of the wall, γ_{RE} is a coefficient considering the increase of allowable strength for earthquake loading.
In selecting strengthening method, effectiveness, reliablity and influence to occupants and normal use have to be considered. This is important for all kinds of structure. The methods of strengthening masonry building commonly used consist of cement grouting, coating with plain or reinforced mortar, constructing shear wall, installing reinforced concrete rigid frame surrounding the building.The last method is the most effctive one against collapse. Its effectiveness has been verified by shaking table test at Tongji[3].Three six storey cinder block masonry

buildings one of which was strengthened by columns at corners and every transverse wall and tie beams at alternate floor levels were tested. The result showed that the strengthened building collapsed under a_{max}=0.45g, while the other two collapsed at a_{max}=0.36g and 0.3g. The crack initiated at 0.13g and 0.07,0.09g.

Case 1
Xianhe is a city in Shandong Province where the fundamental intensity was reassessed as VII from the original VI. Almost all the buildings in the city belong to Gudong Petroleum Company. Here the strengthening of Administration Building of Gudong Petroleum Company is shown in Fig. 1. The strength of mortar was determined by test on specimen cut from wall. The appraising of this building showed defects in some wall sectors in the main building and serious defects in the wing. Constructing shear wall and coating weak wall sector with reinforced cement mortar were used.

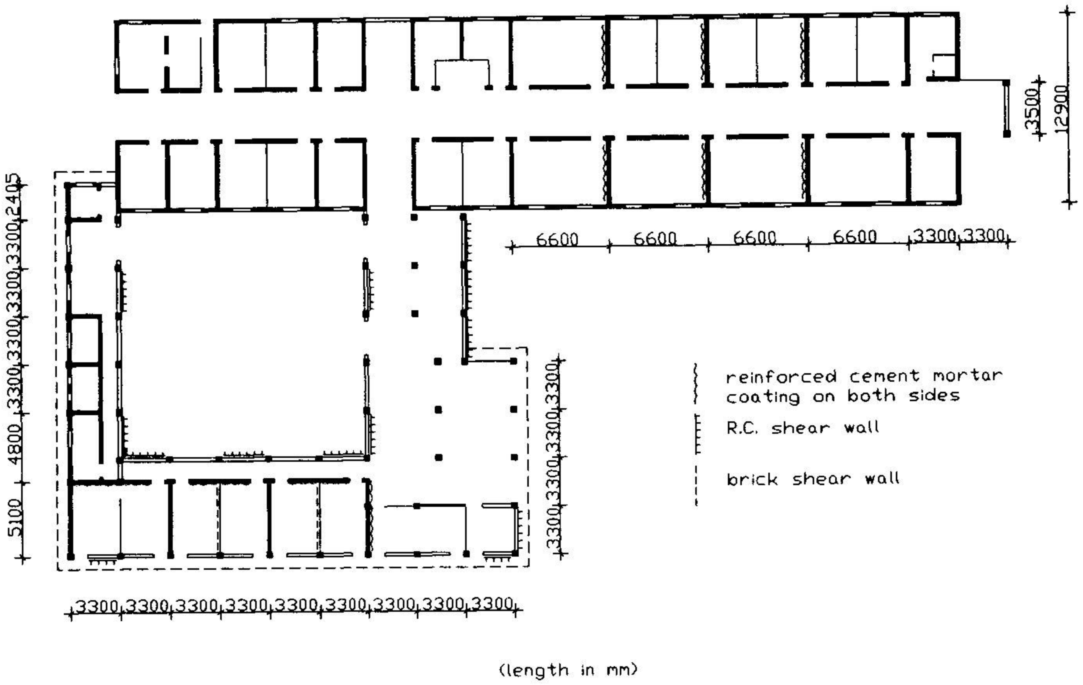


Fig.1 Strengthening of brick masonry building

3.2 Reinforced Concrete and Steel Structures

The Appraising is carried out by visual inspection and analysis. The layout, configuration of plane and elevation, supporting condition of beam, truss, bracing system, connection of nonstructural member can be appraised by visual inspection. The seismic response and the safety of the structure to be calculated in the same way as for a newly designed structure is advisable. The strengthening of reinforced concrete member can use jacketing, strengthened by steel angle or plate, changing structural system by inserting lateral resistant frame, shear wall or bracing system. In doing so care has to be taken that the strengthening system can share the existing load when necessary. Appraising and strengthening steel structure is much the same as for reinforced structure. It is more convenient and easy to strengthen a steel structure than a reinforced concrete one.

Case 2
Baoshan Stadium is a stadium for audience of three thousand. The roof is of precast reinforced concrete slab putting on steel grid structure. Four rows of columns and plateform girders form a rigid frame supporting the grid. Columns in row A are for decoration (Fig. 3). The strengthening to withstand intensity VII was implemented in early ninties. Seismic response analysis revealed that columns in row B and row E do not have sufficient strength. Inspection showed that the support length of roof slab is too short and the dormer window struts are not well braced. Three options of strengthening had been considered: connecting columns in row A and row B to form a frame, strengthening columns in row B and inserting bracing between columns in row D and row E. The first option would destroy the architecture . The last option would cause inconvenience in use and also too rigid the bracing system would be resulting in irregularity of rigidity. Finally columns in row B were strengthened by increasing the section and reinforcement of the column. The stress in columns in row D was thus decreased and no strengthening was needed. Accessaries to improve the supporting condition of roof slab and bracing at dormer window were added.

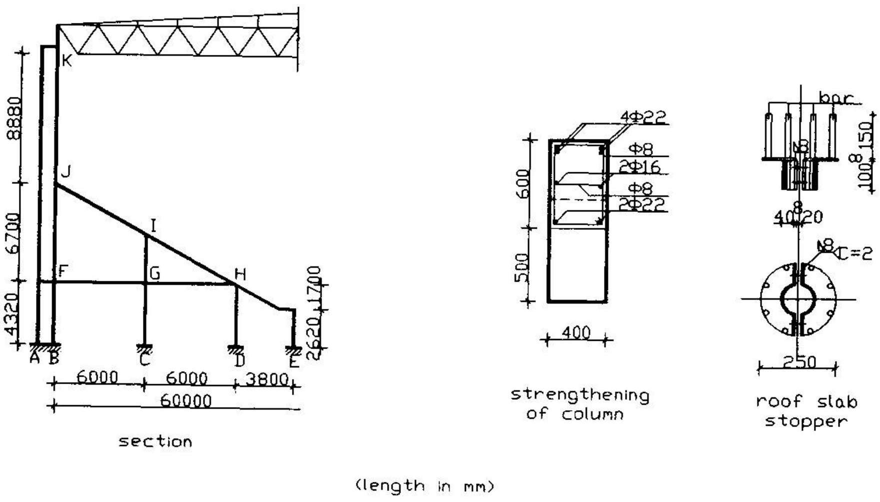
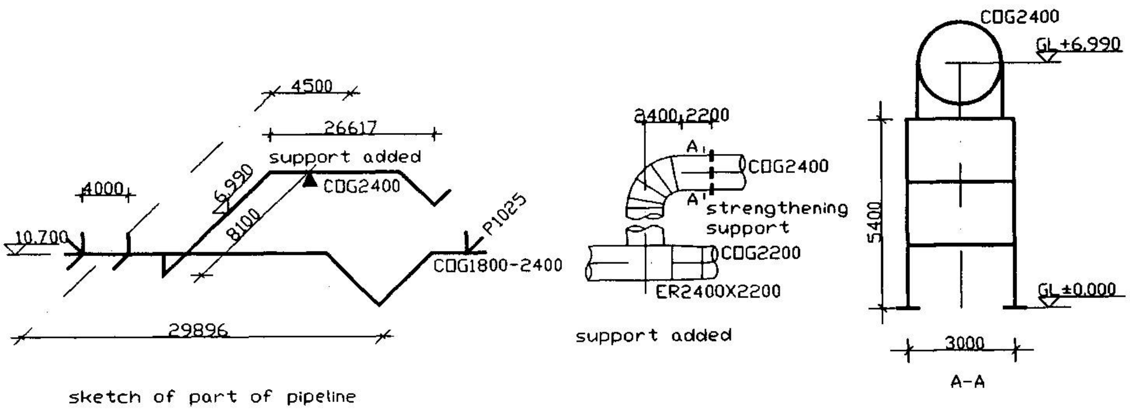


Fig.2 Strengthening of reinforced concrete structures

Case 3
There was no seismic provision for the main pipeline in Shanghai Baoshan Iron and Steel Company during construction. The intensity of site was reassessed as VII according to a specific study. The design response spectrum was also given by that study. In appraising the pipeline was modelled as beam element taking consideration of all the expansion joint, connection and other accessaries. The support was modelled as a spring of six components and three components for rigid support and hinged support respectively. Modal superposition method was used for

seismic response analysis at least twenty five modes being considered. The sum of absolute value of response of modes with natural frequency differing within ten percent was taken and combined with other contributions by means of SSRS. The contributions from the excitation of three directions were combined also by means of SSRS. According to Chinese code, the allowable tensile stress for steel is $[\sigma]=\sigma_u/3=134$Mpa for long term load. A structural factor c=0.3 was apllied to support only that the pipe itself would not yield even under earthquake action.
Several unsafe pipe sectors were discovered.One is shown in Fig.3, where pipe dilivering CO runs in a very complicated way and with unsupported length 40m. The maximum stress (stress concentration considered) at four sections was 465 Mpa to 485 Mpa due to dead load only. Stress due to dead load and earthquake was 544 Mpa to 556 Mpa. The solution was an additional support which raised the pipeline during construction to take reaction from the dead load. The maximum stress was reduced to 76 Mpa (dead load) and 155 Mpa (dead load + earthquake).
Anchor bolts in two fixed rigid frame type support will be overstressed under earthquake. The solution was to insert diagonal members changing the support to a truss. The bending moment of the vertical members was eliminated and thus the tension in anchor bolts was reduced magnificantly.



(length in mm)

Fig.3 Strengthening of steel pipeline

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High Performance Construction Material for Seismic Repair and Rehabilitation

Matériau hautement performant pour la réparation de dommages dûs aux séismes

Hochwertiges Baumaterial für Reparatur und Sanierung von Erdbebenschäden

Neven KRSTULOVIC-OPARA
Assistant Professor
Northeastern University
Boston, MA, USA

Neven Krstulovic-Opara received his MSc. degree from Imperial College of Science and Technology, London, and his Ph.D. degree from Carnegie-Mellon University, Pittsburgh. His research interests include development of high performance fiber reinforced cement composites and fracture mechanics.

Abdol R. HAGHAYEGHI
Principal
Weidlinger Assoc., Inc.
Cambridge, MA, USA

Abdol R. Haghayeghi received his B.S.C.E. from Northeastern University, his M.S.C.E. from MIT, and is currently working towards his Ph.D. at Northeastern University. He has been involved with the design and construction of a variety of rehabilitation and seismic retrofitting projects.

Chia-Ming UANG
Assistant Professor
Univ. of California, San Diego
San Diego, CA, USA

Chia-Ming Uang is an Assistant Professor in the Dept of Applied Mechanics and Engineering Sciences at the Univ. of California, San Diego. He received his Ph.D. in 1986 from the Univ. of California, Berkeley. His research interests include earthquake engineering and seismic repair and retrofit.

SUMMARY

An innovative retrofit technique is presented for improving the seismic resistance of reinforced concrete elements using a newly developed high performance fiber reinforced concrete. The material used in this investigation is made by first placing continuous steel fiber-mats into the form, followed by infiltration of a dense fiber network with a cement-based slurry. The advantages of this novel material - high strength, high toughness, and excellent crack control - makes it ideally suited for increasing strength and energy absorption of structural elements.

RÉSUMÉ

L'article présente un béton de fibres à hautes performances, récemment mis au point pour la rénovation et l'amélioration de la résistance sismique des éléments en béton armé. La méthode consiste à poser des panneaux continus de fibres d'acier dans le coffrage et à enrober ce maillage de fibres très denses par un coulis à base de ciment. Ce nouveau matériau offre une forte résistance, une dureté élevée et un excellent comportement à la fissuration, ensemble de propriétés qui le rendent idéal pour augmenter la résistance et la capacité d'absorption d'énergie des éléments structuraux.

ZUSAMMENFASSUNG

Als innovative Sanierungstechnik zur Verbesserung des Erdbebenwiderstands von Stahlbetonelementen wird ein neuentwickelter hochwertiger Faserbeton vorgestellt. Dabei wer-den zuerst zusammenhängende Stahlfasermatten in eine Schalung eingelegt und danach als dichtes Fasernetzwerk mit einem zementgebundenen Schlamm durchsetzt. Die Vor-teile dieses neuartigen Materials sind seine hohe Festigkeit, hohe Zähigkeit und hervor-ragende Rissebeschränkung und machen es daher ideal geeignet, um die Festigkeit und das Energieabsorptionsvermögen tragender Bauteile zu verbessern.

1. INTRODUCTION

Older reinforced concrete frame structures are usually non-ductile and are thus identified as hazardous. While laboratory studies show that the use of High Performance Fiber Reinforced Concretes (HPFRCs) leads to substantial improvement in seismic response [1-3], conventional materials have been exclusively used in seismic retrofit up until the present. The advantage of HPFRCs is that, when loaded beyond the elastic limit, they exhibit significant increase in both stress and strain capacities, which translates into an enormous increase in the energy dissipation capacity — a feature particularly desirable for earthquake-resistant design.

An existing HPFRC, called Slurry Infiltrated Fiber Concrete (SIFCON), has already been successfully used in the field for non-seismic repair and new construction [4-7]. However, its comparatively high cost and limitations associated with material placement have prevented a widespread field use. Recent development of a new generation of HPFRCs, called Slurry Infiltrated Mat Concrete (SIMCON), promises to eliminate shortcomings of existing HPFRCs and to provide a cost-effective long-term repair and retrofit solution that can be readily implemented in the field. This paper introduces a novel SIMCON-based technique for flexural retrofit of reinforced concrete beams. Research described in this paper is a part of the first stage of a larger ongoing project that aims at developing a SIMCON-based design and construction method for seismic repair and rehabilitation of non-seismically designed beam-column sub-assemblages.

2. BACKGROUND ON HPFRC

Fiber Reinforced Concrete (FRC) is a composite material consisting of cement paste, sand and/or aggregate, as well as fibers made of steel, glass, polypropylene or other tension materials. Presented investigation is focused on steel fibers. Two distinct groups of FRC are: conventional FRC and HPFRC. Conventional steel FRCs are made by premixing up to approximately 3% fiber volume fraction of the discontinuous fibers with concrete [8]. At such a low fiber volume fraction, fibers mainly contribute to the post-cracking ductility and impact resistance. Improvements in first cracking strength, tensile strain capacity, and peak load are not as significant [9]. On the contrary, HPFRCs exhibit significant improvement in both strength and toughness, as shown in Figure 1 a. This is achieved by using continuous steel fibers or high fiber volume fractions of discontinuous steel fibers. However, manufacturing of HPFRCs requires special fabrication methods and careful engineering of the material components. Examples of existing HPFRCs are: fiber reinforced densified small particles (FR-DSP) [10, 11], SIFCON [8, 12, 13], and SIMCON [14]. FR-DSP is made by premixing very short fibers (6 mm in length) with a specially designed matrix. SIFCON and SIMCON are made by pre-placing short, discontinuous steel fibers or continuous stainless steel fiber-mats, respectively, followed by infiltrating the fibers with a cement-based slurry.

Structural Use of HPFRC: Existing SIFCON applications show an unprecedented performance in both repair and new construction. Its use substantially improved durability, ductility, strength, energy dissipation, and eliminated the need for stirrups [1-7, 15]. However, high placement cost and lack of fiber uniformity, associated with manual distribution of discontinuous fibers, prevented a widespread field use. All of these limitations can be overcome using recently developed SIMCON, which is made by infiltrating *continuous fiber-mats* with cement slurry. It thus provides structural performance superior to other HPFRCs [14]. Furthermore, fiber distribution and orientation can be accurately controlled, which allows manufacturing of a unique cement-based fiber composite.

3. EXPERIMENTAL INVESTIGATION

Eight reinforced concrete beams, shown in Figure 1, were tested in flexure. Two beams were used as reference specimens, while the remaining six were retrofitted following three different layouts shown in Figure 1 b. Fiber volume fraction of SIMCON used in this investigation was 5.25%. Sixteen 7.6 cm x 15.2 cm (3 in. x 6 in.) compressive concrete cylinders were cast to determine concrete strength at the time of testing. Behavior of SIMCON in tension and compression was determined in a separate investigation [16]. Average SIMCON strength in tension and compression were determined to be 15.9 MPa (2.3 ksi) at 1.1 % of strain and 68.9 MPa (10 ksi) at a strain of 0.5 %, respectively. Representative behavior of SIMCON in tension is shown in Figure 3.

Materials: SIMCON slurry mix proportions were 1 / 0.31 / 0.6 / 0.3 / 0.045 by weight of Type I Portland cement, water, Ottawa Silica Sand #250, superplasticizer, and microsilica, respectively. Mmat TEC (Ribbon Technology Corporation, Ohio) continuous fiber-mats were used for SIMCON reinforcement. Concrete mix proportions were 1 / 0.47 / 2.60 / 1.27 / 1.33 by weight of Type III Portland cement, water, sand, 0.96 cm. (3/8 in.) size aggregate, and 1.92 cm (3/4 in.) size aggregate, respectively. The yield strengths of the reinforcement was 413.7 MPa (60 ksi).

Specimen Casting and Testing: Light vibration of the forms was used to compact concrete during casting. Cast specimens were moist cured for 9 days, and then left to dry in room atmosphere until retrofitted at the age of 5 weeks. Prior to retrofitting, beams were wetted, wrapped by fiber mats, and placed into forms. Cement slurry was then infiltrated using a manual grout pump. Specimens were demolded after one day, were moist cured for four weeks, and were tested under stroke control in a third point loading, as shown in Figure 1 a.

Observed Behavior: Representative moment-curvature responses are shown in Figure 2 b. Average compressive concrete strength at the age of beam testing was 41.4 MPa (6 ksi). The presence of SIMCON in the compression zone increased strength and strain capacity of the compression zone, thus increasing the lever arm, and leding to an overall increase in strength, energy absorption and curvature at maximum moment. However, delamination of the SIMCON layer occurred shortly after the anticipated flexural capacity was reached [17]. After delamination and with the further increase in beam deflections, significant interfacial friction developed between SIMCON layer and reinforced concrete beam. Hence, after an initial load drop-off, an increase in the load capacity was observed. It can be thus anticipated that if the bond failure was prevented even higher curvature at maximum moment would have been observed.

When SIMCON was cast on all three beam sides both flexural capacity and energy absorption up to the maximum moment exhibited an unprescendented increase of 2.2 and 1.9 times the reference values, respectively. A 25% increase in curvature at maximum moment was also observed. Furthermore, even prior to beam failure maximum crack sizes were less or equal to 0.09 mm (0.0039 in.). Hence, crack sizes were well within the crack size limits of 0.198 mm (0.0078 in.) required by ACI standards for the reinforced concrete elements exposed to severe environments [18].

Analytical Modeling: Behavior of retrofitted beams was further analyzed using the approach described in [17]. Behavior of concrete in compression was represented using the standard model developed by Kent and Park [19], while behavior of reinforcing steel in both compression and tension was defined using the modele developed by Sargin [20]. Behavior of SIMCON in compression was represented using the model developed for SIFCON by Naaman et al. [21, 22], while the behavior in tension was modeled as:

$$\sigma = \sigma_{cu} \left[1 - \left(1 - \frac{\varepsilon}{\varepsilon_{cu}}\right)^{A} \right] \quad \text{for } \sigma \leq \sigma_{cu} \tag{1}$$

$$\sigma = \sigma_{cu} \left[1 - (13 - 2\ V_f) \frac{2\,\delta}{L} \right]^{5} \quad \text{for } \sigma > \sigma_{cu} \tag{2}$$

$$A = \sqrt{E_c \frac{\varepsilon_{cu}}{\sigma_{cu}}} \tag{3}$$

where σ_{cu} is the ultimate composite tensile strength, ε_{cu} is the strain at the ultimate strength, E_c is the composite elastic modulus, δ is the width of the crack opening, V_f is the fiber volume fraction, and L is the fiber length. Predicted analytical response is compared to the experimentally obtained values in Figure 2 b.

5. CONCLUSIONS

Experimental and analytical results clearly show that SIMCON can successfully interact with existing structural elements, substantially increasing load capacity, energy-absorption capacity, and ductility. However, to achieve the maximum benefit of SIMCON retrofit it is important to prevent delamination of the SIMCON layer. Based on the observed cracking mechanism, it can also be anticipated that the use of SIMCON would markedly improve durability of retrofitted elements.

Construction of a SIMCON-based retrofit was much simpler than similar retrofit solutions using SIFCON, reinforced concrete, steel plates or different non-cement based fiber composites. It is thus anticipated that the proposed technique is less labor- and equipment-intensive and more economical than conventional jacketing methods. It uses widely available construction equipment and building expertise, and can thus be easily introduced to the field without major re-training and changes in existing construction practices.

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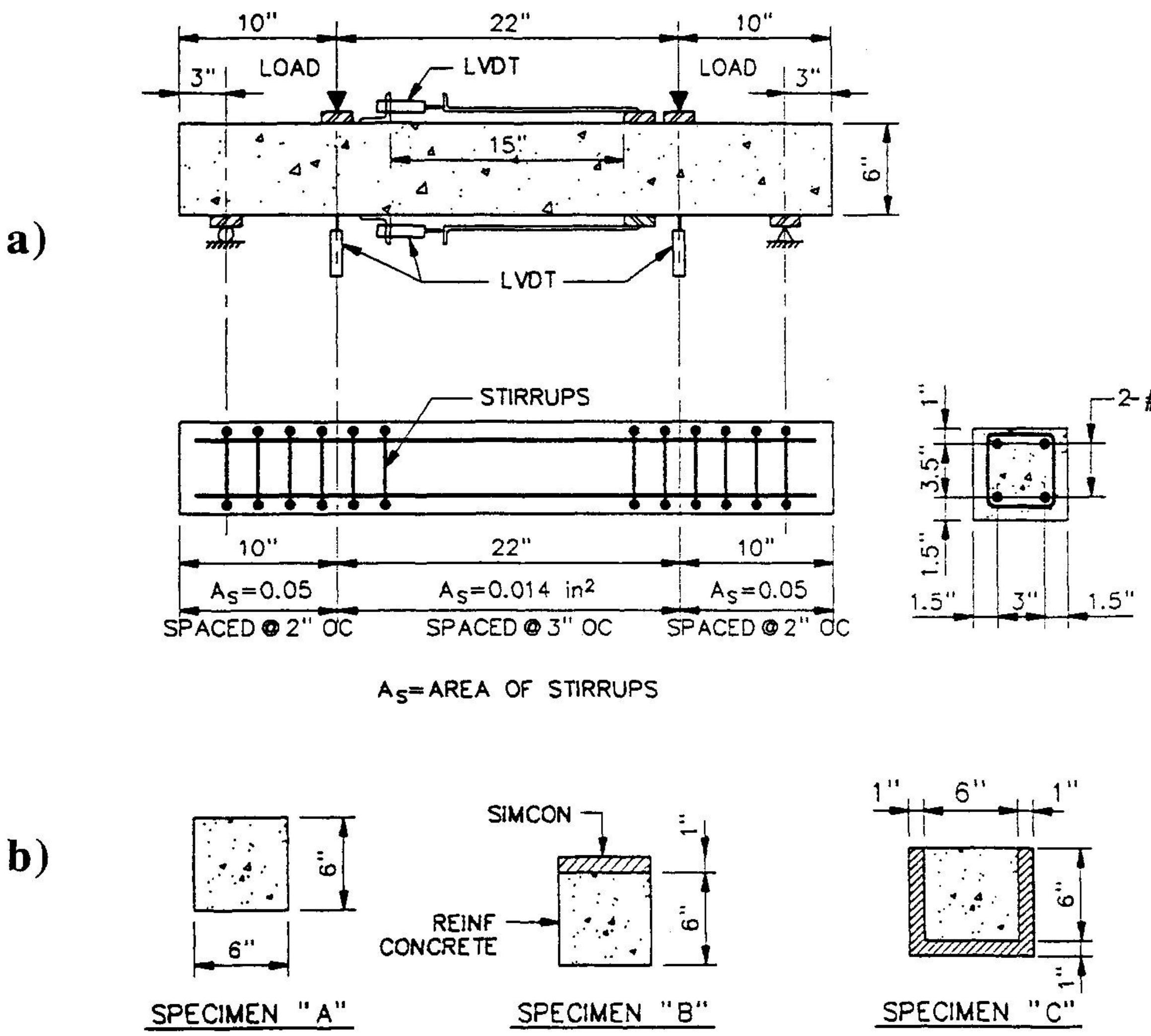
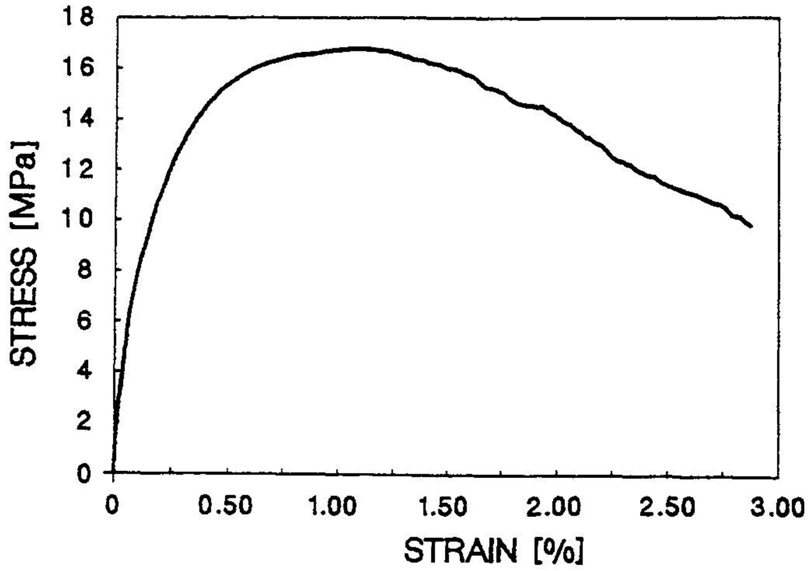


Figure 1: (**a**) Layout of the reinforced beam tested in this investigation. (**b**) SIMCON retrofit layouts.

a)



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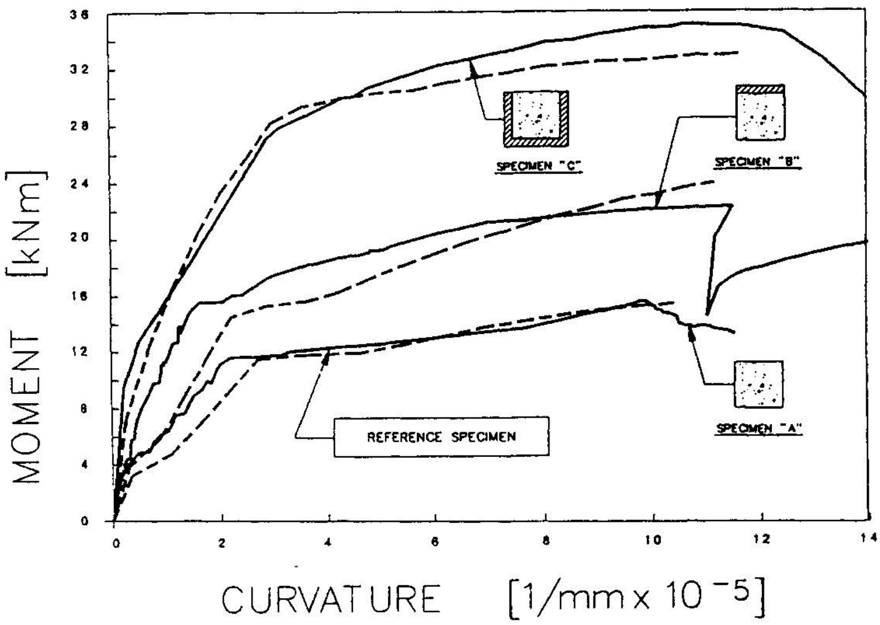

Figure 2: (**a**) Response of SIMCON in tension. (**b**) Experimentally (solid line) and analytically (dashed line) obtained moment - curvature behavior of reinforced concrete beams. Letters "A," "B," and "C" denote specimen types defined in Figure 1b.