Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 83 (1999)

Rubrik: Session 7: Design criteria

Dynamical Load Factor for Highway Bridge Decks with Pavement Irregularities

José Guilherme SANTOS DA SILVA
Visitor Professor
UERJ/FEN/MECAN
Rio de Janeiro, Brazil

José Guilherme Santos da Silva, born 1965, received his D.Sc. degree in civil engineering-structures from PUC-Rio. His research interests include structural dynamics and solid mechanics.

Summary

Since long time the dynamical actions of moving vehicles on bridge decks have been present in structural engineering concerns. In the last few decades the scientific community has started a continuous effort on the study of the dynamical effects on bridge superstructures due to the vehicle traffic on irregular pavement surfaces. In design practice, most of the technical recommendations use a dynamical load factor applied to the vehicle static effects to take into account all the dynamical actions. One presents now the results of a study to verify the extension of the dynamical effects, displacement and stresses, on highway bridge decks, due to vehicles crossing on the rough pavement surface defined by a probabilistic model.

Keywords: bridge structural dynamics, highway bridges, bridge pavement roughness.

Abstract

Since the late 80's, many efforts have been taken to study the dynamical effects on highway bridge decks due to the interaction of the vehicle suspension flexibility with the irregular pavement surface. Several studies have made evident that such effects are much more important than those for a smooth vehicle movement on the bridge [1,2,3,4].

In design practice, the displacements and stresses of the dynamic actions are taken in account, in general, by means of a dynamical load factor (DLF) applied to the static effects of the moving vehicles. In most of the countries, including in Brazil, the code recommendations propose formulas to evaluate the DLF based only on the bridge span length.

In addition to these points, field reports say that some bridges have been submitted to excitation levels, under usual traffic conditions, which have deteriorated their service conditions and structure durability; this can be an indication of under conservative DLF. So, it is desirable to have the problem parameters quantitatively evaluated to better estimate their participation in the structure disruption and to review quantitatively the definition of the DLF for lower quality pavement surfaces.

This ensemble of papers that have been published since then made evident that the dynamic effects produced by the oscillation of the vehicle vs. rough pavement interaction force is a main factor in establishing the DLF value [4,5,6,7,8,9].

In this work one considers a probabilistic definition for the pavement irregular profile and a mathematical structural model which includes the interaction between the dynamical properties of the vehicle with those of the bridge. The moving load is formed by an infinite train of similar vehicles regularly spaced and running at constant speed, in such way to obtain steady-state mean maximum response quantities of the bridge deck, which are necessary to a fatigue analysis of the deck material; one also considers the generation of a number of pavement surface profiles sufficiently large to sustain a statistical treatment of the results [4,5,8,9].

This analysis methodology is applied to reinforced concrete beam decks, continuos on several supports, with overhangs and with constant box cross section and one observes the node displacements and member stresses where the maxima occurs. A parametric study is developed and one concludes proposing a review on the evaluation of the DLF and on qualitative and quantitative aspects of the problem and the attitudes concerned with bridge design and maintenance.

The main point in this work is the magnitude of dynamical effects relatively to the static values. The roughness of an excellent pavement surface produces response quantities, displacements and stresses to as the bridge deck, that can reach magnitudes to the same order of those due to the static effect of a train of vehicles long enough to load all the bridge. These magnitudes are many times greater than that due to the load mobility alone [10].

References

[1] Ramalho, F. N. M. and Roehl, J. L., "Ações Dinâmicas em Pontes Rodoviárias com Defeitos na Pista", X CILAMCE, Iberian Latin-American Conference on Computational Methods in Engineering, 1989.

[2] Sedlacek, G. and Drosner, St., "Dynamik bei Brücken", Baustatik Baupraxis, Universität. Hannover, 1990.

[3] Wang, T. L. and Huang, D., "Cable-Stayed Bridge Vibrations due to Road Surface Roughness", *ASCE, J. Struc. Engr.*, Vol. 118, n.° 5, pp.1354-1374, 1992.

[4] Silva, J.G.S., "Análise Dinâmica Não-Determinística de Tabuleiros de Pontes Rodoviárias com Irregularidades Superficiais", D.Sc. Thesis, Departamento de Engenharia Civil, PUC-Rio, 1996.

[5] Silva, J.G.S. and Roehl, J.L., "Dynamical Analysis of Bridge Decks with Irregular Pavement Surface Defined by a Profile Spectral Density", XVII CILAMCE, Iberian Latin-American Conference on Computational Methods in Engineering, 1996.

[6] Silva, J.G.S.; Roehl, J.L., "Probabilistic Formulation for the Analysis of Bridge Decks with Irregular Pavement Surface", VII DINAME, International Conference on Dynamic Problems in Mechanics, 1997.

[7] Calçada, R.; Cunha, A., "Stochastic Modeling of the Dynamic Behavior of Bridges under Traffic Loads", IV WCCM, Fourth World Congress on Computational Mechanics, 1998.

[8] Silva, J.G.S.; Roehl, J.L., "Ações Dinâmicas em Tabuleiros de Pontes Rodoviárias Provenientes das Irregularidades do Pavimento", III SIMMEC, 3° Simpósio Mineiro de Mecânica Computacional, In portuguese, 1998.

[9] Silva, J.G.S.; Roehl, J.L., "Parametric Study of Highway Bridge Decks with Irregular Pavement Surface", PACAM VI, VI Pan American Congress of Applied Mechanics and VIII DINAME, International Conference on Dynamic Problems in Mechanics, 1999.

[10] Silva, J.G.S.; Roehl, J.L., "Dynamical Actions on Bridge Decks due to a Train of Vehicles", IV WCCM, Fourth World Congress on Computational Mechanics, 1998.

Size Effect in Concrete Structural Members

Ricardo EINSFELD
Associated Professor
UERJ
Rio de Janeiro, RJ, Brazil

Ricardo Einsfeld received his D. Sc. degree in civil engineering-structures from PUC-Rio. His research interests include non-linear analysis of concrete structures.

Marcus PACHECO
Associated Professor
UERJ/IPRJ
Nova Friburgo, RJ, Brazil

Marcus Pacheco received his Ph. D. degree in civil engineering-soil mechanics from Purdue University. His research interests include numerical modeling of soils and concrete.

Marta VELASCO
Assistant Professor
PUC-Rio
Rio de Janeiro, RJ, Brazil

Marta Velasco received her D. Sc. degree in civil engineering-structures from COPPE-UFRJ. Her research interests include the behavior of high-strength concrete structures

Summary

This paper reviews some recent theoretical and experimental results on the size effect in brittle failure of concrete structures caused by the release of stored energy. The fracture mechanics size effect for geometrically similar structures is simulated in a numerical model developed by the first author. The example presented shows the importance of the size effect in concrete design. In the conclusions, the authors refer to some revisions that should be taken at the Brazilian concrete design code.

Keywords: concrete; fracture mechanics; numerical modeling; plasticity; size effect.

Abstract

The size effect is generally ignored by the current design codes, but recent tests have shown it strong enough to be considered in projects involving large structural members. Most of the recent experimental tests on size effect were conducted by Bazant at the Northwestern University [2]. The tests have been carried out in order to overcome the limitations of previous experimental evidence found in literature.

The size effect phenomenon can be represented by theoretical models that account for the dissipation of strain energy into the fracture process zone. In this study it was used a 2D numerical model developed by the first author[4] that combines finite element analysis routines with interactive computer graphics. The concrete behavior is modeled through the energy-based plasticity formulation of Pramono and William [5]. This formulation covers the full load-response spectrum of the concrete behavior in tension as well as in compression. The concrete model is based on the non-associated flow theory of plasticity, with hardening in the pre-peak regime and fracture energy softening in the post-peak regime.

A numerical simulation was carried out for the three-point symmetric notched beams of various sizes, tested experimentally by Bazant and Pfeiffer [3]. The beams are geometrically similar with the same mesh configuration. The ultimate loads were obtained for the characteristic dimension of the structure d equals to 38, 76, 152 and 304 mm. The material fracture parameters and the width of the fracture process zone were adjusted in the constitutive model in order to obtain the finite element fit over the test data. In Figure 1, the results of the finite element analysis are compared with the mean experimental results obtained by Bazant and Pfeiffer. The nominal stresses were calculated as a strength parameter, using the elastic stress formula ($\sigma_N = 3.75\, P_{max} / b\, d$, *where* P_{max}

is the maximum concentrated load at midspan). The analytical results, obtained substituting the material parameters in an equation that relates the nominal stress to the characteristic dimension of the structure, are also plotted in Figure 1. The plotted values indicate a good agreement between the results.

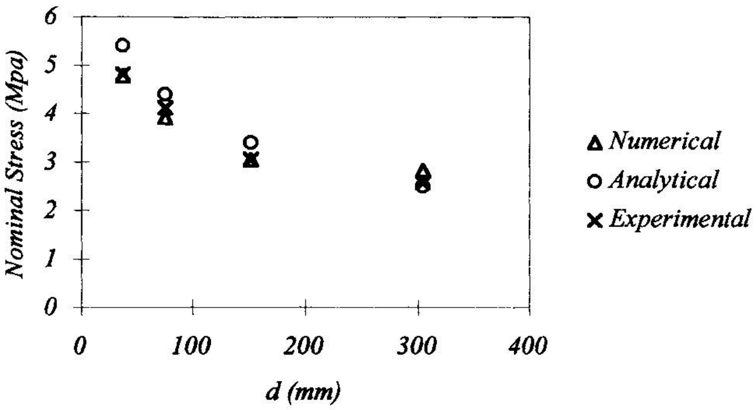


Fig. 1 Results on size effect for three-point bend notched beam

The results show that with the appropriate adjustment of the fracture parameters, the size effect phenomenon can be adequately represented by numerical models that account for the dissipation of the fracture energy.

The size effect should be taken for further development on the Brazilian concrete design code, as it has been done into the ACI Code and CEB-FIB Model Code. An example for code revision can be suggested concerning the minimum tension reinforcement specified in section 6.3.1 of the Brazilian code NBR-6118 [1]. According to the results presented, the minimum reinforcement ratio of 0.0015 specified for CA-50 and CA-60 bars is not sufficient for larger structures, and this value should be revised based on experimental evidences. This example shows that the code must exhibit the transitional size effect between plastic limit analysis and linear elastic fracture mechanics.

References

[1] ABNT, *NBR-6118 - Projeto e Execução de Obras de Concreto Armado*, 1980, 76 pp.

[2] Bazant Z. P., "Should Design Codes Consider Fracture Mechanics Size Effect," *Concrete Design Based on Fracture Mechanics ACI SP-134-1*, 1992, pp. 1-23.

[3] Bazant Z. P., and P. A. Pfeiffer, "Determination of Fracture Energy from Size Effect and Brittleness Number," *ACI Materials Journal*, Vol. 84, Nov.-Dec. 1987, pp. 463-480.

[4] Einsfeld R. A., A. E. Elwi, T. N. Bittencourt, and L. F. Martha, "Numerical Simulation of Fracturing in Concrete Structures Using a Combination of Smeared and Discrete Approaches," *International Journal of Rock Mechanics and Mining Sciences*, Vol. 34, No. 3-4, 1997, pp. 552.

[5] Pramono E., and K. Willam, "Fracture Energy-Based Plasticity Formulation of Plain Concrete," *ASCE Journal of Engineering Mechanics*, Vol. 115, No. 6, 1989, pp. 1183-1204.

Stress-Based Crack Criterion for Concrete at Early Ages

Klaas VAN BREUGEL
Senior Lecturer
Delft University of Technology
Delft , The Netherlands

Klaas van Breugel, born 1952, received his civil engineering degree from Delft University of Technology in 1979 and his doctor degree from the same university in 1991.

Sander J. LOKHORST
Civil Engineer
Holland Rail Consult Utrecht
Utrecht , The Netherlands

Sander Lokhorst, born 1967, received his master degree at Delft University of Technology in 1992. He was employed at TU-Delft until 1997 and since then in a consulting company in Utrecht, The Netherlands.

Summary

In order to enhance the quality of concrete structures, cracking in hardening concrete should be prevented as far as possible. A traditional way to assess the risk of cracking at early ages consists of minimising temperature differentials between warm and cold concrete. In this contribution the results of a recently finished study on the risk of cracking and crack criteria of hardening concrete are presented. The stress development and the tensile stress at the moment of cracking have been measured in a Temperature Stress Testing Machine. Based on the test results a criterion for cracking at early ages is formulated. A procedure for determination stresses in hardening concrete is briefly discussed. In this procedure allowance is made for the effect of the evolution of the microstructure on relaxation of stresses at early ages.

Keywords: Hydration; thermal stresses; cracking; creep; relaxation; modelling; crack criteria

1. Introduction

In the engineering practice the risk of cracking in hardening concrete has often been formulated in terms of allowable temperature differentials. From observations in the practice as well as from theo-retical considerations it has become clear that temperature criteria are very crude. The actual risk of cracking can be overestimated and underestimated considerably, resulting in too conservative or too optimistic designs. Strain or stress criteria are considered more appropirate. In this contribution both experimental and mathematical studies concerning stresses and the risk of cracking in young concrete are dealt with. The aim is to define a stress criterion for cracking of hardening concrete.

2. Outline of research strategy

For the prediction of the risk of cracking in hardening concrete the evolution of the materials properties with elapse of time must be known. In this study the development of materials properties is described as a function of the degree of hydration. The correlation between degree of hydration and strength has been demonstrated many times. More recently also attempts have been made to describe the time-dependent properties of hardening concrete has a function of the degree of hydration. This strategy has also been followed in the theoretical part of this research project. In the experimental part of the project the development of thermal stresses in hardening concrete has been investigated with a Temperature Stress Testing Machine.

3. Determination of stress development and cracking of hardening concrete

The development of stresses in hardening concrete and the risk of cracking was investigated experimentally for a number of different concretes with a Temperature-Stress Testing Machine (TSTM). With this machine stress development due to restrained deformations as well as creep and relaxation behaviour under prescribed thermal conditions were investigated. The mixtures were made with Portland cement and blast furnace slag cement and with water/cement ratios varying from 0.3 to 0.6. Curing of the concrete took place either isothermally or semi-adiabatically. Imposed deformations were restrained in whole or in part.

4. Crack criterion for young concrete and prediction of tensile stresses

For all the concrete specimens considered in the experimental program self-induced cracking occur-red at an average stress/strength ratio 0.75. Based on this data a probabilistic judgement concept is proposed in the form of a nomogram (Fig. 1). The nomogram is based on the assumption of a normal distribution of both the tensile strength and tensile stresses. With the graph an allowable stress/strength ratio η can be determined for each pre-defined probability of cracking which is considered appropriate for the concrete structure in view. This allowable stress/strength level η can be compared with the development of the stress and the strength obtained by simulation.

For prediction of the tensile stresses at early ages a calculation procedure is propsed in which allowance is made for some microstructural phenomena which occur in hardening concrete. Examples of stress predictions are presented. Preliminary results obtained with this microstructure-based calculation procedure are encouraging.

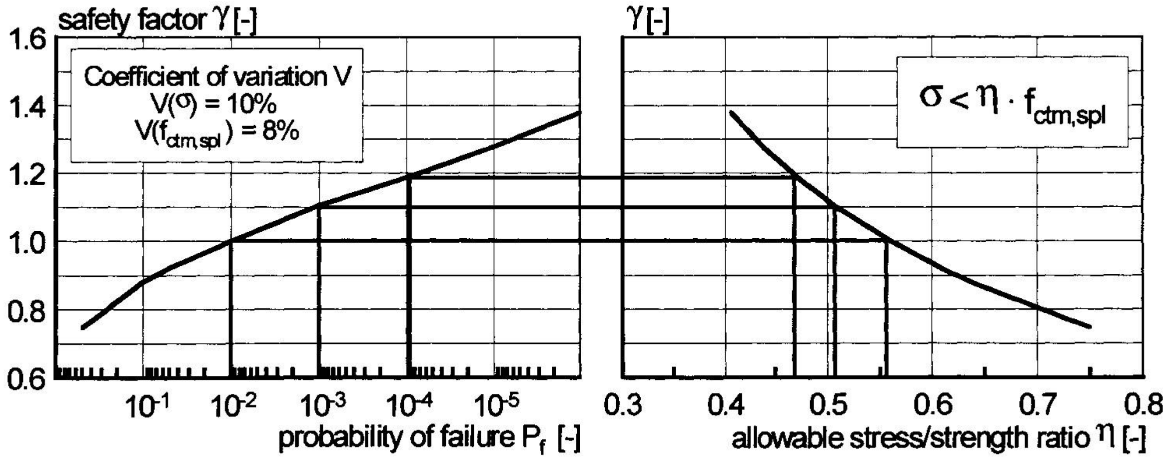


Fig. 1 Design graph for the determination of the maximum allowable stress/strength ratio in a hardening concrete structure

Performance-Based Seismic Design – The Future Practice

Loring A. WYLLIE, Jr.
Senior Principal
Degenkolb Engineers
San Francisco, CA, USA

Loring A. Wyllie, Jr. has over 35 years of experience in seismic-resistant design and building retrofits for improved seismic performance. He is an IABSE Vice President and Chair of the USA Group of IABSE.

Abstract

Performance-based design has been a normal part of building design for many years, and owners frequently make performance-based decisions. Shall we select an expensive roofing system such as slate or copper which will perform well for many decades, or shall we select an inexpensive single ply membrane roof that will require repairs or replacement in 10 to 20 years? Building designers and owners commonly discuss such issues, and owners make selections based on their budget and the years of performance they desire before replacement.

Structural engineering routinely faces similar issues, although they may not be as clear. Since we know most of our structural loads and forces with reasonable certainty, most of our performance decisions center around material performance. We routinely design more durable concrete when it is exposed to weather and chemical attack, and take steps such as coated reinforcement or thicker concrete cover over reinforcement to reduce the possibility of corrosion or deterioration of concrete. The owner is seldom brought into a discussion of these structural issues and is often just informed of future maintenance needs.

But seismic-resistant design is different. Seismic forces in regions of high seismicity are the largest lateral loads we have to consider in design, and are comparable only to large explosive or bomb blasts very close to a building. Seismic-resistant design for most buildings is based on greatly reduced force levels, recognizing the ductility of structural systems to maintain their load-carrying capacity under severe inelastic deformations. This recognition of ductile performance, which is specified as prescriptive material detailing and proportioning provisions in modern building codes, allows us to design structures economically for high seismic ground motions. But the ductile or inelastic performance brings with it extreme damage to the structure. The ultimate goal of conventional building codes is life safety, or providing protection to occupants until they can safely and orderly exit the structure. These codes do not carefully consider the degree of damage that may occur, and if the structure can reasonably be repaired. Thus the performance objective beyond life safety is not clear: the building may be useable, needing only minor repairs, or it may be a total loss, necessitating demolition. More likely, the building's condition will be

somewhere between these extremes, requiring extensive repairs and having to be vacant for many months, causing disruption to the tenants and owner.

Earthquake damage states and performance levels are well defined in the Vision 2000 document published by the Structural Engineers Association of California. The damage states of Negligible, Light, Moderate, Severe, and Complete describe the level of damage experienced by a building during a specific seismic event. These damage states can be roughly compared to five performance levels: Fully Operational, Operational, Life Safe, Near Collapse, and Collapse. These damage states and performance levels are then related to site seismicity and probabilistic studies including recurrence rates for earthquakes of various sizes at the site. Graphics in the paper and Vision 2000 combine these factors with various occupancies in a logical manner.

The structural implications of PBSD can be quite significant. There are a variety of current building codes and standards that include material detailing requirements to achieve basic life-safety performance in an occasional or rare earthquake. This assumes the building is built on a suitable site without potential surface faults, liquefaction, or seismic induced landslides affecting the site. For higher performance objectives structural systems must be selected that have historically performed well in the most severe earthquakes. In general, more rigid structural systems tend to deflect less and experience less overall damage. Building configuration is exceedingly important, as regular buildings tend to outperform irregular or unusually shaped buildings. Seismic load paths need to be direct and well detailed. Structural detail must be carefully selected and detailed for proper performance, based on all our knowledge of inelastic response of structures. The design must be consistent and of high quality, and should be thoroughly peer reviewed by suitable knowledgeable engineers. Then the construction must be well executed, with thorough competent inspection to ensure that the completed building will perform as intended when the earthquakes occur. PBSD is also quite applicable to existing buildings and structures. It has also become routine in California to strengthen deficient existing buildings for improved seismic performance, generally for life safety and to prevent possible collapse or potential collapse. But usually, with some additional work or a different approach, an existing building can be strengthened to a higher performance objective. This is also becoming somewhat common where businesses assess their potential risk or loss of business or manufacturing capability and decide to strengthen some of their facilities to a higher performance objective.

The nonstructural implications of PBSD are perhaps more challenging and difficult to achieve than the structural issues. The nonstructural issues include not only preventing the collapse of ceilings and partitions and sliding or overturning of equipment and furniture, but also the operability of critical building systems such as electricity, water, sewage, and other utility services.

Since no consensus standards or guidelines exist for the higher performance levels of PBSD, a considerable effort will be needed to develop such guidance. Toward this end in the United States, the Earthquake Engineering Research Institute recently developed an Action Plan for the U.S. Federal Emergency Management Agency. The Action Plan outlines all the steps needed to achieve consensus guidelines for attaining PBSD. The program is outlined as a multiple-task effort, which will probably require ten years to complete. The program not only addresses the technical issues, but also the stakeholder and educational agendas, which will need to be addressed.

The New Canadian Bridge Code - Design, Evaluation, Rehabilitation

Roger DORTON
Consultant
Buckland & Taylor Ltd.
Toronto, ON, Canada

Roger Dorton, born 1929, received his civil engineering degree and doctorate from the University of Nottingham. After twenty years with the Ministry of Transportation, Ontario, he joined Buckland &Taylor Ltd. in 1993.

Ranjit REEL
Senior Project Engineer
Delcan Corporation
Toronto, ON, Canada

Ranjit Reel, born 1939, received his civil civil engineering degree from the University of Glasgow. After 32 years with the Ministry of Transportation, Ontario, he joined Delcan Corporation in 1998.

Darrel GAGNON
Senior Engineer
Buckland & Taylor Ltd.
N. Vancouver, BC, Canada

Darrel Gagnon, born 1961, received his master of structural engineering degree from the University of Alberta. He has specialized in bridges and is a member of the team developing the evaluation requirements for the CHBDC.

Summary

The first edition of the Canadian Highway Bridge Design Code will be published in 1999 by the Canadian Standards Association for immediate adoption across the country. The code not only addresses the design of new bridges, but also covers the evaluation and rehabilitation of existing bridges, all in a consistent limit states format in one document. The code has been calibrated to a target safety index of 3.5 for new bridges, with a service life of 75 years. For the evaluation of an existing bridge the material conditions may be assessed by inspection, and operational conditions are known, hence different safety index options and load factors are given in the code, reflecting these various conditions. The durability requirements for all structural materials are addressed in order to improve quality. The code will be issued in both English and French and will be accompanied by a commentary and calibration report.

1. Introduction

At present two bridge codes are in use in Canada, the Ontario Highway Bridge Design Code (OHBDC) and the CAN/CSA-S6-88 Design of Highway Bridges (CSA-S6). Both are now in limit states format, calibrated to a target safety index of 3.5. Although each of the ten Provinces is responsible for its own legal vehicle weights and bridge codes there is an agreement on inter-provincial vehicle weights and a proposed National Highway Policy. With the need to have a single bridge code, the Canadian Highway Bridge Code (CHBDC) has been developed and will replace the OHBDC and CSA-S6. The 1991 OHBDC was selected as the model for the development of the CHBDC. The CSA-S6 Clause 12 on evaluation had been successfully used in several jurisdictions, however, and the Clause 12 approach was maintained for the development of the CHBDC evaluation section. The CHBDC aimed to expand and update the existing Canadian codes in a number of areas, including; eliminating the span length limit of 150 m so the code would address long span bridges, developing a new live load model to represent national traffic loads and to extend its application to long spans, carrying out new calibration studies, expanding the coverage of seismic design to include evaluation and retrofit, including movable bridges, including design provisions for some advanced composite materials, and emphasizing the importance of durability issues by having a separate durability section.

2. Design

The consensus of the Provincial bridge engineers was that the design truck should represent the regulatory loads, based on the inter-provincial transportation agreement. The selected truck, known as CL-625, consists of a gross load of 625 kN, on a 5-axle configuration with an 18 m wheel base, with successive axle loads of 50, 125, 125, 175, and 150 kN. Calibration of load factors, load combinations and resistance factors were based on the CL-625 truck, and for a target safety index of 3.5 and a service life of 75 years produced a live load factor of 1.7 for the ULS combination of live load plus permanent loads. The CL-625 truck and its various axle sub-configurations govern the design of short span bridges and elements and were selected so they would be suitable as the basis for bridge evaluation and load posting as well as design.

For spans over about 50 m the lane loading governs, and long spans, over about 100 m are governed by stationary bumper to bumper traffic. The objective was to have one live load factor of 1.7 applicable to the truck load and the lane load which would produce the target safety level of 3.5 over the full range of spans, short, medium and long. Long span bridges have much higher dead to live load ratios than shorter spans, and this aspect was investigated before arriving at the lane load, which consists of a uniform load of 9 kN/m superimposed on a CL-500 truck (the CL-625 truck with axle loads reduced to 80%). Calibration for permanent loads plus live load was carried out by reliability analysis using Provincial truck survey data, and by design checks on 31 representative bridges using the new CHBDC, the OHBDC and the CSA-S6. Further calibration was carried out for permanent loads plus wind as this is of importance on long span bridges, and produced a wind load factor of 1.65, well above the typical 1.3 value used in North America. To address durability in a consistent manner, the CHBDC identifies the deterioration mechanisms and specifies the protective measures and detailing for durability for each material, based on the deterioration mechanisms and the environmental exposure to which the material is subjected.

3. Bridge Evaluation and Rehabilitation

Bridges are evaluated when either the loads are increasing or the bridge is deteriorating. However, it is often inappropriate to use load and resistance factors from the design sections of a code when evaluating a bridge. The bridge evaluation section of the CHBDC was developed to allow the evaluation to benefit from desirable structural behaviour and the results of bridge inspections. Values of the target safety index β, based on a consistent level of life safety, are tabulated allowing for behaviour of the structural system (e.g. multiple or single load paths), behaviour of the element (e.g. gradual or sudden failure) and level of inspection the bridge has received. Consistency is maintained between bridge design and evaluation by tying the β values to the safety index required by the design sections of the code. For each value of β, load factors were calibrated and shown for various dead loads and live loads for evaluation purposes.

When rehabilitating a bridge, the CHBDC recommends that the bridge be rehabilitated to meet the requirements of the design sections. However, if this proves physically or economically unfeasible the evaluation requirements can be used as a minimum rehabilitation standard.

Comparison of Reinforcement Anchorage Tests with British Code Requirements

Simon CUMMINGS
Research Engineer
Queens University Belfast
Belfast, N. Ireland

Su TAYLOR
Research Assistant
Queens University Belfast
Belfast, N. Ireland

Barry RANKIN
Honorary Lecturer
Queens University Belfast
Belfast, N. Ireland

David CLELAND
Professor
Queens University Belfast
Belfast, N. Ireland

Richard SCOTT
Senior Lecturer
Durham University
Durham, England

Summary

The aim of the research described in this paper was to improve the current method of assessing the shear capacity of existing bridge decks and, in particular, to determine the amount of main reinforcement anchorage below which the shear capacity is affected. This paper presents the results of tests carried out relating to a real bridge. A number of interesting comparisons are made between the test failure loads and the predicted flexural and shear strengths. Conclusions are then drawn relating to the Standard BD44/95 with implications for future assessments. The research indicated that the current bridge assessment method in BD44/95 for reinforcement anchorage and shear strength of slabs is overly conservative.

Abstract

Bridges in the UK are being assessed to confirm their capacity to carry the loading which results from increased lorry sizes conforming to European standards. Assessment is based on the assessment code for concrete bridges (BD44/95) published by the Highways Agency.

The Civil Engineering Department of Queens University Belfast was contacted concerning problems of deficient reinforcement anchorage which had arisen in the assessment of a Bridge in Northern Ireland. It had been found that using the Department of Transport Bridge Assessment Standard BD44/95, the bridge had insufficient shear capacity and would therefore require major repair work.

It was suggested that this problem, which was likely to be more widespread, should be investigated experimentally in order to ascertain if the current code requirements for minimum reinforcement anchorage were overly conservative.

The main reasons for the existing concrete slab bridges failing their assessments are as follows:

(a) the guidelines for shear are more conservative in current codes than in the earlier codes to which some of the earlier structures were designed and

(b) some bridges have been found to have reinforcement details at the supports which provide minimal anchorage to the main reinforcement.

The second of these is a particular problem because BB44/95 assumes that any shear capacity depends on a minimum anchorage length of 12 times the bar diameter. Enhanced shear, up to 3 times the normal shear capacity may be assumed if the load is applied close to the support but this is dependent on an anchorage length of 20 times the bar diameter. No enhancement is permitted for anchorage lengths between 12 and 20 times the bar diameter. In an attempt to improve the requirements of BD44/95 an investigation of the specific problem of deficient shear capacity in a real concrete bridge was carried out at Queens University Belfast, where a number 1/3 scale models representing a typical bridge were tested.

The test results are compared with the predicted ultimate flexural and shear strengths according to BD 44/95 (An example of this is presented in Figure 1). It is shown that the reinforcement anchorage requirements of this standard are overly conservative and shear enhancement is possible for reinforcement anchorage lengths less than 12 bar diameters.

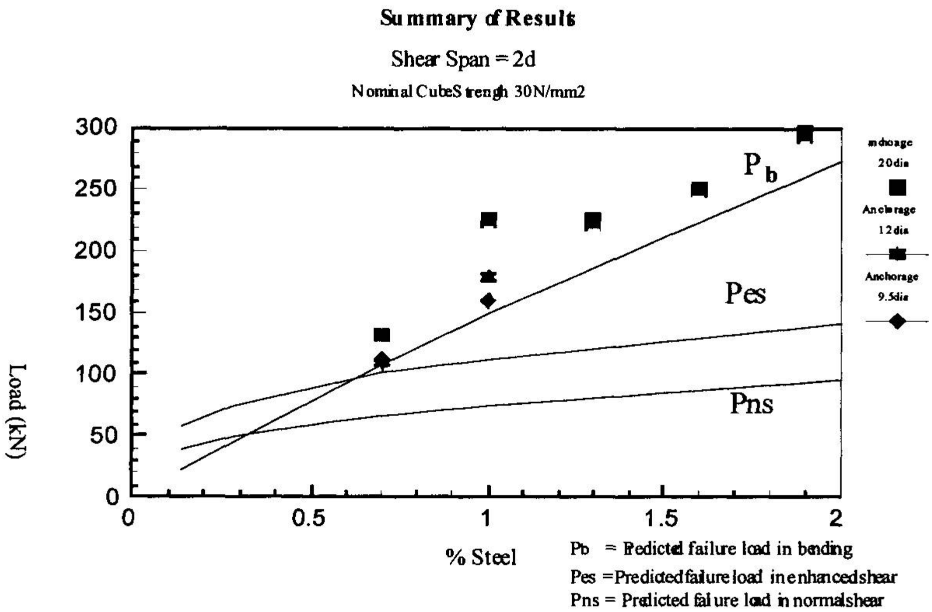


Figure 1 Comparison of Actual to Predicted Failure Loads

If the quality of bridges in the future is to be improved then there must be a more complete understanding of the effects of reinforcement anchorage. This will in turn prevent overly conservative codes of practice requiring expensive strengthening operations on bridges which do not need it.

Eurocode Comparison Calculations for Storebælt Bridges

Jens Chr. KAERN
M.Sc., Ph.D., Civil Engineering
RAMBØLL
Copenhagen, Denmark

Mr. Kærn graduated from the Technical University of Denmark in 1978. Since 1981, he has been employed by RAMBØLL. Mr. Kærn was lead engineer of the detailed design of the steel box girder for the suspension bridge of the Great Belt Link.

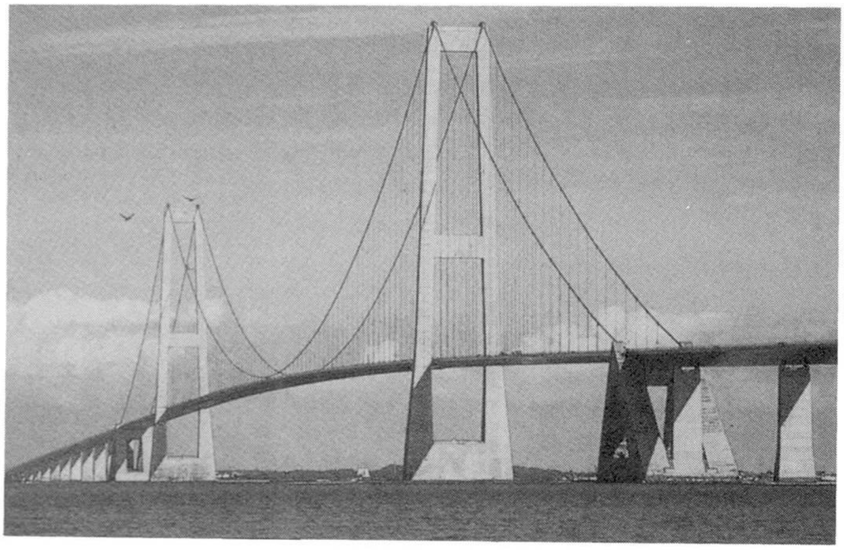
Lars HAUGE
M.Sc., Civil Engineering
COWI
Copenhagen, Denmark

Mr. Hauge graduated from the Technical University of Denmark in 1986. Since 1990, he has been employed by COWI, where he at present is head of the department for design of major bridges. Mr. Hauge was in charge of the detailed design of the steel box girder for the suspension bridge of the Great Belt Link.

Abstract

This paper describes the main results from a desk study, where the consequences of applying the Structural Eurocodes, especially ENV 1993-2 for steel bridges, as design basis for Great Belt East Bridge, have been investigated. All main structural steel elements of the superstructures have been investigated. The bridges were originally designed according to a purpose-made design basis and the Danish national code.

The comparative study was carried out jointly by RAMBØLL and COWI, who also performed the detailed design of the bridges. The Great Belt, East Bridge consists of a suspension bridge with a main span of 1624 m and side spans of 535 m and approach bridges with 23 nearly identical spans with a span length of 193 m.



Great Belt, East Bridge

The general loads as well as the partial coefficients for loads and materials are based on the Structural Eurocodes. Project specific loads such as wind loads, ship impact, etc. are taken from the Design Basis for the Great Belt, East Bridge.

The traffic load of the Eurocode is based on Load Model 1, defined in European Pre-standard ENV 1991-3, Eurocode 1. The load corresponds to an exceedance probability of 0.1 per 100 years, whereas the traffic load on the Great Belt, East Bridge corresponds to an exceedance probability of 0.02 per year. Consequently, the Eurocode traffic loads are substantially higher. The uniformly distributed traffic load (UDL) in the Great Belt, East Bridge design basis is 18% lower than in the Eurocode, when the influence length is below 500 m. The maximal axle load is 15% higher in the Eurocode, whereas the maximum wheel pressure is about 13% larger in the Great Belt, East Bridge design basis. These differences will influence the local conditions of the deck structure.

The total load safety (TLS) factor for the dead load, which is defined as the total loads multiplied with the material partial coefficient, increases by 10 % from the Great Belt, East Bridge, design basis to the Eurocode. The TLS enhancement factor for the uniformly distributed design traffic load is identical for the two code systems. In the article all partial coefficients and TLS enhancement factors for difference loads are described.

The maximum positive and negative design bending moments are for the approach bridge girders 1.19 times larger in the Eurocode. The ratio between the material coefficients is 0.81, meaning that the TLS enhancement factor is 1.19 * 0.81 = 0.97. The conclusion of the investigation is that the area of the bridge dimensioned by the tensile stress corresponding to the global moment will remain unchanged, if designed according to the Eurocode.

In contrast to elements dimensioned for tensile stress, a further difference in the load bearing capacity for global as well as local plate stability applies. The results of the comparison are that the enhancement factor from the Great Belt, East Bridge design basis to the Eurocode ranges between 1.0 and approximate 1.35, increasing with increasing slenderness ratio.

The Eurocode does not apply to bridges with a span of more than 200 m, implying that the suspension bridge over Storebælt is not covered. The comparison is however done anyway. The TLS enhancement factor for the hangers is calculated and the results vary from 0.98 to 1.03. This means that the dimensions of the hangers in average will be the same, if dimensioned according to Eurocode. The TLS enhancement factor for the main cable is determined to 1.20, meaning that the cross section of the main cable should have been 20% larger, if calculated according to the Eurocode. This is due to the fact that the contribution of the dead load constitutes 70% of the maximum characteristic force in the main cable, and that the partial coefficient for the dead load is 1.35 according to Eurocode, and 1.1 according to the Great Belt, East Bridge design basis. The conclusion is therefore that Eurocode have too high safety level for dead load dominated structure.

The fatigue analysis has been carried out using Load Model 3 and the characteristic lifetime for the orthotropic steel deck has been determined. The results for the two most critical welds (the deck plate/trough weld and the trough splice welds) show that the SN-curves in the Eurocode and the traffic Load Model 3, in the case of the Great Belt, East Bridge, are very conservative.

In relation to this work we have found that the Eurocode does not specify how the local stresses from the axle loads shall be combined with the global stresses in the orthotropic steel deck, which is necessary for a rational design of orthotropic steel decks.

The final conclusion is that the Structural Eurocode can be used as design basis for both approach and suspension steel bridges, but has - for unique structures as the worlds 2. longest spanning suspension bridge - to be accompanied by purpose made design specifications.

Modelling of Post-Tensioned Reinforced Concrete Flat Slabs

György FARKAS
Dr Eng.
Technical University
Budapest, Hungary

György Farkas, born 1947, received his civil engineering Ms.C. degree from the Technical University of Budapest. The title of his Ph.D. dissertation was "Design of Prestressed Concrete Flat Slabs". Research fields: modelling of reinforced concrete structures, high performance concrete in construction.

Summary

Experiments with existing prestressed concrete floors have shown that with use of traditional models the calculated value underestimate the real load bearing capacity of reinforced concrete flat slabs. Based on theoretical and experimental considerations propositions for estimating the variation of the prestressing force during the loading and models to calculate the load carrying capacity of post-tensioned concrete flat slabs with second order effects taken into account are developed in this paper.

Keywords: flat slabs, post-tensioning, load carrying capacity, second order effects.

1. Introduction

Because of their technical and economical advantages, post-tensioned reinforced concrete flat slabs are widely used around the world to construct various type of buildings and other constructions. After a short presentation of linear elastic methods used mainly in serviceability limit states, proposition for estimating the variation of the prestressing force during the loading and analytical formulae to estimate the load carrying capacity of prestressed post-tensioned concrete flat slabs taking second order effects into account will be developed in the paper.

2. Modelling in serviceability Limit States

Linear elastic methods of analysis should be used to determine the internal forces of the slab in this states. The influence of post-tensioning may be taken into consideration with the use of the equivalent load method. In the case of applying non uniformly distributed tendons in plain, the variation of the normal forces in the slab has significant effect on its behavior. Linear elastic finite element method with combined plate-membrane or mainly for waffle slabs, with grid-membrane elements gives suitable result in this case.

3. Modeling to estimate the load bearing capacity

Experiments [1] show that the classical methods do not give appropriate results in ultimate limit states to estimate the load bearing capacity of reinforced concrete flat slabs post-tensioned by unbonded tendons. These results indicate that to obtain more exact estimation of the load carrying capacity of these structures it is reasonable to take also second order effects into consideration.

The analysis of a post-tensioned reinforced concrete internal floor field in ultimate limit states may be transformed to the examination of the equilibrum of a compressed and a tensioned membrane. Based on the principle of the equilibrum of the compressed membrane, we developed a membrane dome model. This model permits to determine the load carrying capacity of the floor filed by the expression

$$q_u = p \cdot 3 \cdot t / R^2, \tag{1}$$

where p represents the radial component of the uniformly distributed prestressing forces, f_c is the rise of the substitutive membrane dome, t is the thickness of the slab and R is its equivalent radius at the base. If the prestressing tendons of the floor are concentrated mainly to the column lines, a double arch model is proposed to determine the ultimate limit load. The horizontal component of the reaction of the third-degree parabolic arches is equilibred by the prestressing forces. From this condition the uniformly distributed load bearing capacity of a floor field will be given by the lower value from the expressions

$$q_{ux} = P_x \cdot 9 \cdot t / \ell_x^{\,2} \cdot \ell_y \quad \text{and} \quad q_{uy} = P_y \cdot 9 \cdot t / \ell_x \cdot \ell^2{}_y, \tag{2}$$

where P_x and P_y are respectively the prestressing forces in direction x and y concentrated in column lines. When considerable deflections develop on the floor the compressed membrane and arch models are no more applicable. The use of a tensioned membrane analogy or a modified yield line analysis can give appropriate results in this case. In any case the elongation of the prestressed tendons due to the deflection of the slab during the loading has an important influence to the calculated load bearing capacity of the floor. The effect of elongation of the tendons to the load bearing capacity of the slab must be taken into consideration. In the case of large deflection of the slab the yield line analysis of the theory of plasticity can be applied to determine the load carrying capacity of the floor. With the supposition that the effect of the slab's prestressing is replaced by an equivalent external upward pressure, the ultimate limit load of an internal floor field prestressed by uniformly distributed tendons may be calculated according by the following expression

$$q_u = \frac{8 \cdot p}{\ell}\left[1 + \frac{16}{3} \cdot \frac{f_t}{\ell} \cdot \frac{w}{\ell} \cdot \frac{1}{\varepsilon_p} + \frac{8}{3} \cdot \left(\frac{w}{\ell}\right)^2 \cdot \frac{1}{\varepsilon_p}\right] \cdot \left(\frac{f_t}{\ell} + \frac{w}{\ell}\right), \tag{3}$$

where p is the uniformly distributed prestressing force in direction x or y, ℓ is the span of the slab in the same direction, f_t is the maximum deviation of the tendon, w is the maximum deflection of the slab and ε_p is the strain of the tendon due to the efficient prestressing force.

4. Conclusions

Experiments with existing post-tensioned reinforced concrete flat slabs have shown that the use of classical models generally underestimate the real load bearing capacity of these structures. Second order effects are reasonable to be taken into consideration for dimensioning these floors. Application of a membrane dome model, a double arch model and a modified yield line analysis are shown in this paper to estimate the ultimate limit lead of prestressed concrete flat slabs using unbonded tendons. The numerical result approve the suitability of the developed method.

5. References

[1] Farkas Gy., "Strengthening of Prestressed Floors by Additional Post-tensioning", *FIP Symposium on Post-Tensioned Concrete Structures*, Symposium Papers, Vol. 1., 1996, pp. 454-462.

Stability Studies of Water Tower's Vertical Flanges

Kresimir HERMAN
Professor
University of Zagreb
Zagreb, Croatia

Kresimir Herman, born 1934, received his BCE, M.Sc. and Ph. D. at the University of Zagreb where he teaches Theory of elasticity and Continuum mechanics.

Vladimir SIGMUND
Professor
University of Osijek
Osijek, Croatia

Vladimir Sigmund, born 1956, received his BCE and M.Sc. at the University of Zagreb and Ph. D. at the University of Sarajevo. Teaches Continuum mechanics at the University of Osijek.

Ivica GULJAS
Assistant
University of Osijek
Osijek, Croatia

Ivica Guljas, born 1966, received his BCE and M.Sc. at the University of Zagreb. He is an assistant professor for Structural stability and dynamics at the University of Osijek.

Summary:

High reinforced-concrete water towers are commonly used for water storage in the low regions. Water tanks, with big storage capacity, are placed at the top. Middle carrying part has to be stable enough and to secure the overall structural stability. To set the whole area under tank as a carrying element is not economically sound and is ugly from the architectural standpoint. Therefore, thin vertical flanges are radially placed around the central tubular part as buttresses. Designing of such flange-buttress elements is complicated as the problems of their out-of-plane and overall structural stability are coming together.

In order to study this problem simplified model studies of the water tower's middle part. The models were exposed to incrementally increasing vertical loads. Out-of-plane deformations of the flanges were registered and critical buckling forces and forms were determined. Then the sophisticated analytical model of the overall structure has been done. Critical flange's buckling load and buckling forms of the scaled experimental model and that of sophisticated numerical model correlated very well. Model studies gave good sense of the load carrying ability for a full-scale structure. Calibrated numerical models were used to study the influence of various parameters on the flange's and overall structural stability.

Keywords: water tower, vertical flanges, and experimental model, buckling analysis, comparison

1. Introduction and Conclusion

The flange's out-of -plane stability can determine the overall structural stability. Sometimes these elements are from the architectural reasons made with openings. Their numerical analysis without very sophisticated mathematical models presents a problem. This study was undertaken in order to gain insight in the behavior and stability of the vertical flanges and to distinguish among various parameters that influence that stability.

Typical water tower's structural cross section and dimensions, for a particular case, are presented on Fig. 1. The Model 1 has uniform buttress flanges and Model 2 has the same geometry but its flanges had openings at their upper and lower part. Both models have six radially placed flanges around the middle tubular element.

Experimental analysis is performed on a simplified (quasi 3D) scaled structure with the intention of finding the first critical buckling force and form of a flange in system. Analytical models represented the whole system without simplifications. Their results were compared with the experimental ones. Parameters that contribute to the flange carrying ability are than varied in order to distinguish among them.

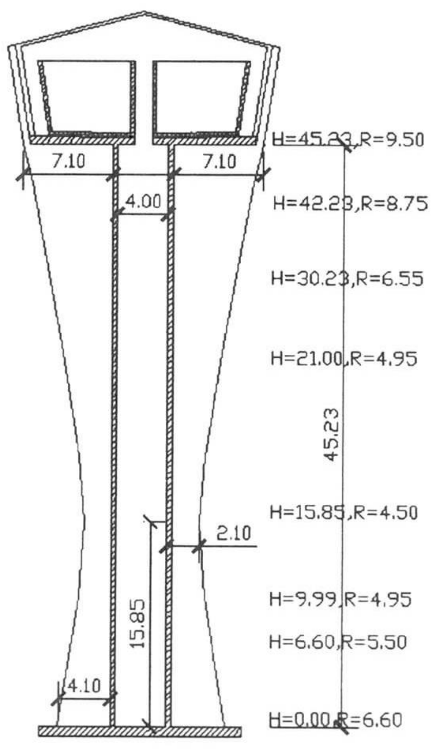


Figure 1Water tower-Model 1

The experimental tests were done on a simplified model in a scale 1:50. By incrementally applying loads and registering the displacements, critical buckling forces and forms for the models were determined. Critical buckling force in Model 2 was 2 times smaller than that in Model 1. Restraining rotation of the flange's edges

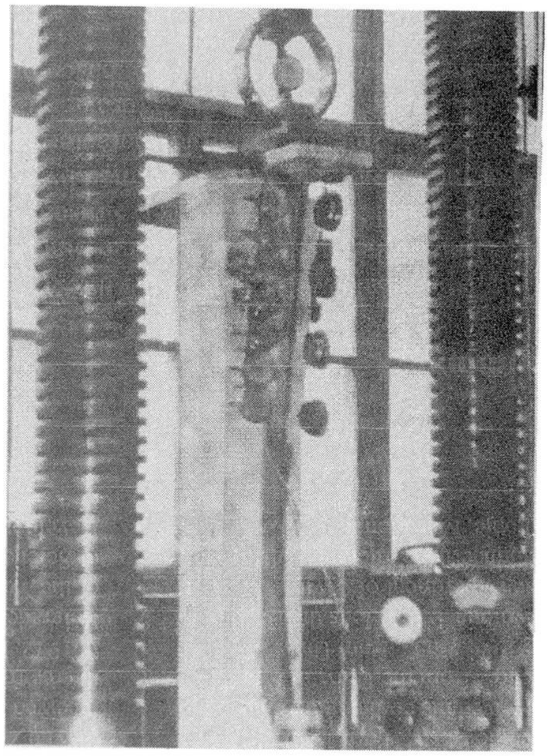


Figure 2 Model test set-up

was found to be very important. Determined forces are directly, through the application of model analysis, applicable to a full-scale structure, and as it turned out represent the lower bound.

The analytical models were sophisticated and represented detailed structural models. Critical buckling forces and forms of the scaled experimental model and that of numerical models correlated very well. The Model 1 has additional reserves due to the spatial distribution of flanges, which was not simulated in the experiment. Spatial distribution of the flanges had significant influence on Model 1 and little influence on the Model 2 structural stability. Model 2 fails locally in the flange-opening region. By implementing openings in the flanges, critical buckling force is reduced several times (up to 3 times) and implementing openings in the flanges minimizes spatial reserves of the system. While simplified plane analysis can be approved for structures with such flanges (Model 2); it gives us only a lower bound for a structure with solid flanges. Structures with solid flanges have much greater stability reserves due to the spatial (3D) structural stiffness distribution.

Calibrated numerical models were used to study the influence of various geometric parameters on the flange's and overall stability. From the limited parametric analysis has been observed that for load carrying ability of the vertical flanges the most important factors are its protruding length outside the central tube and the elements' thickness while flange's height plays a minor role.

Conclusions achieved from the simplified experimental models proved to give us good orientation in dealing with very complicated problems. Such problems are local and global stability analysis of buttress flanges encountered in high water tower tanks. Usage of simple models can obviously serve us as a rule of thumb for complicated numerical analysis or as an additional tool to the simple numerical methods.

High Strength Concrete Beams Subjected to Reversed Loads

M.A.A OLIVEIRA
Graduated Student
Campinas University
Campinas, SP, Brazil

A.L. MORENO Jr
Assistant Professor
Campinas University
Campinas, SP, Brazil

Abstract

The transfer mechanisms of the shear force from section to another of the beam only recently are being explained and, even so, partially. This way, a definitive quantification of the contribution of the same ones in the transmission of the shear force becomes only speculation.

Tests of beams conducted by Leonhardt defined the portions of contribution of shear carrying mechanisms in a beam with web reinforcement. In these studies, it was ended that a big portion of the shear force was transmitted by the web reinforcement and a remaining minority portion can be attributed to the aggregate interlock and dowel action of the longitudinal reinforcement.

About the first subject, many shear behaviour models, seeking to determine the real contribution of the web reinforcement in the transfer of shear force has been proposed. However, a model that determines, with accuracy, this portion of contribution, was not still presented.
The shear strength of the concrete beams can be affected by factors that influence the efficiency of one of the shear carrying mechanisms. Leonhardt admits the existence of approximately 20 of these factors.

Besides numerous, the influential factors in the internal shear strength are complex and, most of the time, interrelated. Among these factors it can be included, for example, the compression strength of the concrete and the type of load (normal or reversed load).

Serious doubts still stay regarding the shear behaviour of a reinforced concrete beam subjected to reversed loads. While Alatorre and Casillas concluded that the shear behaviour of these beams would not be altered, Japanese investigations , reached to the conclusion that the portion of shearing, resisted by the web reinforcement, would be reduced in more than 50% in the beams subjected to reversed loads.

The Brazilian Code, NBR 6118, do not mention any special procedures for shear design of beams subjected to reversed load. The American Code, ACI 318, suggests that all shear force will be resisted by web reinforcement only. About the shear behaviour of high strength concrete beams subjected to reversed loads any special designs procedures are mentioned in both Codes.

The experimental investigation here described had its motivation in doubts lifted about the shear strength behaviour of high strength concrete beams subjected to alternate loads. It were experimentally analysed 12 beams with identical geometry and longitudinal reinforcement. The concrete compression strength and the type of load, normal or alternate, were the adopted variables. A first series constituted by 6 beams without web reinforcement and a second series by beams with this reinforcement. For each series, there were three ranges of concrete compression strength. Of the

beams with the same concrete compressions strength, one was subjected to normal loads and another to alternate loads.
A concentrated load, F, was applied at the midspan of the beams until the expected shear failure. In those tests that included reversed loading the concentrated load was applied in one of the faces until the wanted load increment, then the beam was discharged and the load was applied in the other face. The load increment was of 5 kN. The specimens were instrumented for deflection and steel strain measurements.

The cracking pattern of the specimens tested monotonically was similar. The fist cracks to appear in the specimens were short vertical flexure cracks in the midspan. Continued increases in the load led to the formation of inclined cracks that extended out from the vertical cracks to form flexure-shear cracks. With increasing load the existing cracks developed and new cracks were formed until they had extended through the longitudinal steel at both layers. When the loading was reversed, an additional pattern of cracks appeared in the apposite direction.

After the formation of flexure-shear cracks, the shear carrying mechanism was assumed to consist of contribution from the compressed concrete above the crack, aggregate interlock or friction forces along the crack dowel forces along the longitudinal reinforcement and, for beams with web reinforcement, consists of stirrups crossed by the inclined cracks too.

The pattern of stress evolution was the same for all the beams. The stirrup stresses were not perceptible until inclined cracking occurs. After inclined cracking, the stirrup stresses increase with applied shear. Starting from the mobilisation of the stirrups, the growth of the stresses accompanied, in an approximate way, the foreseen growth by the classical analogy.

It can be observed that in any stage of load of the beams the stresses in the web reinforcement were greater than the values predicted by the classical analogy (shear force carried by the web reinforcement only), even for the beams subjected to reversed loads. This fact checks that, although the contribution offered by the concrete of the compressed zone can be unimportant, it still exists the collaboration of another alternative mechanisms, that not the one formed by the web reinforcement, in the shear strength of high strength concrete of beams submitted to reversed load. The dowel force of the longitudinal reinforcement can be placed as a great collaborator.

It can be concluded that exists a decrease of the shear strength in high strength concrete beams submitted to reversed load in relation to the same beam submitted to normal load. This reduction was shown itself significant, but insufficient to approach the shear behaviour of these beams of the classical model – where the total shear force is carrying by the web reinforcement.

The Brazilian Code do not mention any special procedures for shear design of beams subjected to reversed load. The American Code suggests that all shear force will be resisted by web reinforcement only. About the shear behaviour of high strength concrete beams subjected to reversed loads any special designs procedures are mentioned in both Codes. The results obtained in this work showed that the procedures of ACI 318 , where all shear force would be resisted by web reinforcement only, could be very conservative

High Strength Concrete Beams Subject to Axial Compressive Stress

Adriano MARINHO
Civil Engineer
Univ. Estadual de Campinas
Campinas, SP, Brazil

Adriano Marinho, Structural Engineer that obtained his MS from the Civil Engeneering at Campinas State University. Scholar Ship by FAPESP

Armando Lopes MORENO JR
Professor
Univ. Estadual de Campinas
Campinas, SP, Brazil

Armando L. Moreno Jr, Doctor Professor Assistant of Civil Engeneering at Campinas State University – UNICAMP, Campinas – São Paulo, Brazil.

This paper forms part of a comprehensive investigation of the shear behaviour of high strength concrete beams submitted to axial compressive stress and attempts to search the basis for the development of simple design procedures for these structural elements, particularly the study of the shear force portion absorbed by the concrete, V_C.

Most current building codes express the shear strength of beams in terms of the shear force portion carried by transverse reinforcement, V_S, and the shear force portion "carried by the concrete", V_c. This "concrete contribution" can be assumed as:

$$V_{C,ACI} = \left(1.4\sqrt{f_c'} + 17.2\rho_w \frac{V_{ud}}{M_u - N_u \frac{(4h-d)}{8}}\right) b_w d \text{ [kN], to ACI 318-86(1)} \tag{1}$$

$$V_{C,NBR} = 0.15 * b_w * d * \sqrt{f_{ck}} * \left(1 + \frac{M_0}{M_{sd}}\right) \text{ [kN], according to NBR 6118(2).} \tag{2}$$

$$V_{C,REV} = 0.6 * f_{ctd} * b_w * d * \left(1 + \frac{M_0}{M_{sd}}\right) \text{[kN], according to NBR 6118 review.} \tag{3}$$

$$V_{C,CEB} = V_\theta - V_{45} \text{ [kN], according to CEB FIP-1990 (4)} \tag{4}$$

$$V_\theta = \left(\frac{A_{sw} * f_{yd}}{s}\right) * z * (\cot g\theta + \cot g90) * \operatorname{sen} 90 \tag{5}$$

considering V_{45} and V_θ the total shear force ($V_c + V_s$) according to the "plasticity truss model" with the angle of inclination of the diagonal concrete struts, θ, equal to 45 degrees for V_{45} and variable from 18.4 up to 45 degrees for V_θ.

Based on the research's results herein presented, a new equation is proposed for predicting the ultimate shear force portion "carried by the concrete", V_c:

$$V_{C,PROP} = 0.6 * f_{ctd} * b_w * d * (1+\kappa)^2 \text{ [kN], with } (1 + \kappa)^2 \leq 2 \tag{6}$$

with the compression degree, *k*, assumed as:

$$\kappa = \frac{N}{N_{nec}} \tag{7}$$

where N is the compressive axial load applied in the beam and N_{nec} is the service compressive axial load necessary to avoid tension stresses in required cross section of the beam, assumed as:

A total amount of seven beams, with a concrete compressive strength around 85 N/mm^2, were analysed. All of them had identical geometry and reinforcement, longitudinal and transversal. The only variables were the intensity of the compressive axial load applied and its application point through outs the beam's height.

The loading on the beams was made up of a concentrated load, applied at the midspan of the beams, and of a compressive axial load, applied at their external faces.

A web reinforcement ratio, ρ_w,of 0.252% was adopted for all the beams. Logically this value was chosen in order to be inferior to the web reinforcement ratio, which was calculated according to the classical truss model - $\rho_{w,45}$, once the shear failure by yielding of the web reinforcement was required. Strains in the longitudinal bars and shear reinforcement were measures by strain gages.

The intensity and position difference point application of the compressive axial load resulted in a variability of the compression degree, *k*, in the investigated beams.

All the tested beams have reached the ultimate shear capacity by the yielding of the web reinforcement as it was expected.

Failure was sudden and complete, particularly in beams with high compression degree, κ.

Results of the tests conducted here have shown that the truss model can be extended to high-strength concrete beams subjected to axial compressive stress, on the shear design. At least the ones which have been made of concrete with a compressive strength until 85 N/mm^2.

The shear force portion "absorbed by the concrete", V_C, calculated according ACI code equation, NBR code equation and CEB code equation, were compared with the experimental data obtained in this work and it could be noticed that the all code equations were very conservative for high strength concrete beams herein tested, underestimating the influence of shear force portion "absorbed by the concrete", V_C, at the shear strength design.

It was possible to notice for this research's beams, as it was already observed by Leonhardt for the usual strength concrete beams subject to axial compressive stress, the possibility of a bigger reduction shear load portion, V_c, due to the compression degree increased, *k*, within the beams). This reduction possibility may be explained due to the fact that the longitudinal compression stresses delay the crack beginning of the beams and, therefore, they delay the mobilised of the web reinforcement too.

The new proposal, presented in this paper, **eq. (6)**, has provided satisfactory values of "shear force portion absorbed by the concrete", V_c, for high strength concrete beams submitted to axial compressive stress.

Experimental and Analytical Investigation of Sandwich Panels

Vladimir SIGMUND
Professor
University of Osijek
Osijek, Croatia

Vladimir Sigmund, born 1956, received his BCE and M.Sc. at the University of Zagreb and Ph. D. at the University of Sarajevo. Teaches Continuum mechanics at the University of Osijek

Kresimir HERMAN
Professor
University of Zagreb
Zagreb, Croatia

Kresimir Herman, born 1934, received his BCE, M.Sc. and Ph. D. at the University of Zagreb where he teaches Theory of elastivity and Continuum mechanics.

Darko SIGMUND
Architect
"Sigmund" Ltd
Zagreb, Croatia

Darko Sigmund, born 1949, received his BCE and M.Sc. at the University of Zagreb. He is a specialist for modular coordination and rationalisation in design and building.

Summary

In order to optimize outer wall's thickness and to reduce the required construction phases (costs) carrying walls built as "sandwich" panels (insulated wall panels) seem to be a reasonable alternative. Finished wall consists of inside insulation and two concrete wythes connected by shear connectors. It acts as composite wall up to a certain point. Load carrying ability of such elements is investigated experimentally and analytically. The experimental investigation consisted of trial loadings of model "sandwich" bords, up to the failure, under vertical, horizontal and in plane loading. The obtained results are compared with modified (based on the tests) analytical values for uniform concrete sections. Analytical values correlated well with the experimental results. Based on the results and published test results of the simmilar structures, suggestions for design of load carrying composite "sandwich" wall panels are given.

Keywords: "sandwich" wall, composite action, trial loading, shear transfer, analysis, design

1. Introduction

Insulated wall ("sandwich") panels are used in the TVD process that suits any architectural shapes and standards of buildings. The basic building elements are „TVD thermo- panels" which consist of a prefabricated steel frame (made out of two spaced inter-connected mesh reinforcements) and the insulation in-between, dry-set and completed „in-situ" with spraying of concrete, thus forming a purely monolithic structure with uniform concrete walls and slabs.

Mesh reinforcement is standardized steel fabric. Patented shear panel connectors (deformed stainless steel reinforcement bars) together with polyethylene tiles meshes and thermal insulation and form a space truss system for a panel board. Thermal insulation, held in place with the space truss, serves as a caisson for spraying of concrete and eliminates the need of formworks. It gives its physical characteristics to the finished wall, stiffens the lattice and serves as a horizontal or vertical support for spraying of concrete. Thermo panel boards (Fig. 1) are 0.60 to 1.20 meter wide and 2.60 to 4 meters high, so that one single person can easily handle them. They are easy to manipulate and any architectural shape can be achieved. The building phases for a story are (1) Erecting and assembling of thermo-panels and cutting out of the openings; (2) Setting up the plumbing and electrical installations on the panels; (3) The first spraying with concrete; (4) Assembling of the ceiling and pouring of the slab concrete; (5) The second final spraying with concrete and surface finishing. Concrete wythes are the loads bearing part of the "sandwich" walls. Thickness of the finished insulated walls varies depending on its use (bearing and non-bearing outer- and inner-walls, slabs and base plate). From the structural standpoint, the walls are acting as composite walls consisting of insulation between the two concrete wythes of equal or unequal stiffness. The possible wall loading types are axial vertical loading, eccentrical vertical loading, in-plane shear load and wind lateral loading. As behavior of the composite panels and their carrying abilities are not standardized, each of the typical loading situations was simulated on model panels up to the failure and the panel behavior was observed.

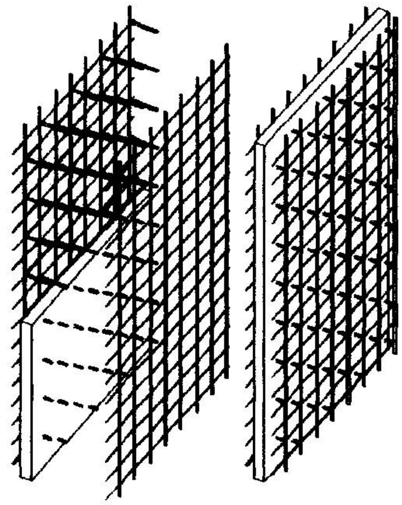


Figure 1 TVD thermo-panel board

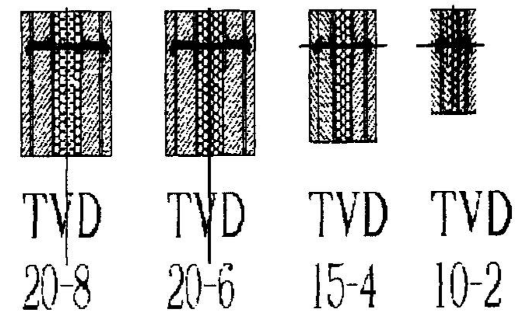


Figure 2 Typical cross-section of the wall

2. Experimental tests and analytical calculations

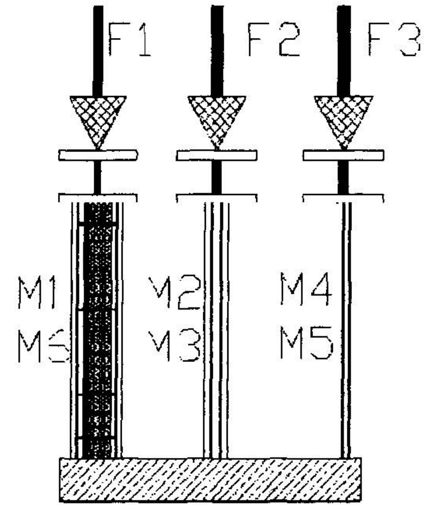


Figure 3 Tested models of short walls

Experimental tests were needed to study achieved composite action and behavior of the "sandwich" walls under various loading situations. The tests were divided in:

(A) Tests of short walls under axial loading. The models represented composite wall, one concrete wythe and uniform concrete section represented joined both wythes;

(B) Tests of composite "sandwich" panels which represent long walls, under axial load and moment, lateral and in plane loading.

The experimental investigation consisted of trial loadings up to the failure for vertical (composite and uniform concrete sections), horizontal and in plane loading. Results obtained from the tests on "sandwich" wall panels are compared with modified analytical values for uniform concrete sections.

Modifications of analytical formulas were based on the results of experiments and published tests of the similar structural systems. Measured and analytical values, calculated by the suggested, methods are correlated and it can be seen that suggested modified formulas are representing well the actual behavior. Calculated values are always on the safe side, except in the highly plastic region. Due to the special problems occurring in composite elements at the load application region, standard values of the safety coefficients have to be increased. The investigation has shown that insulated ("sandwich") walls executed in-situ by spraying concrete, can be analyzed with the modified methods that are commonly used for design of uniform reinforced/concrete sections.

Experimental Investigation of Industrial Hall Structures in the Full Scale

Danilo RISTIC
Professor
IZIIS, University of Skopje
Skopje, R. Macedonia

Danilo Risti}, born 1950, received his civil engineering degree from the Univ. of Skopje in 1974, Ph.D. from Univ. of Kyoto in 1988. He is the Head of the Department of Civil Engineering Structures, IZIIS, Skopje, Macedonia.

Slavko ZDRAVKOVIC
Professor
Civil Engineering Faculty
Nis, Yugoslavia

Slavko Zdravkovi}, born 1947, received his civil engineering degree from the Univ. of Nis in 1972, Ph.D. from the Univ. of Belgrade in 1981. He is an Expert of the Yugoslav Federal Ministry, of Science, Technology and Development.

Dragan ZLATKOV
Assistant
Civil Engineering Faculty
Nis, Yugoslavia

Dragan Zlatkov, born 1958, received his civil engineering degree from the Univ. of Nis in 1983, defended his Master thesis in 1998 at the Univ. of Nis.

Summary

This paper deals with a part of the integral report prepared within the framework of the project: Attesting of the Static and Dynamic Stability of Typified Moduli of the Hall Program of the Precast RC Structural System "AMONT-Krusce, Yugoslavia". In this phase of the project, adequate field experimental studies were performed by applying the method of measurement of ambient vibrations in order to define the dynamic characteristics of four different structures constructed according to this structural system. For the future application, more successful analytical modeling of this structural system is made possible based on the obtained experimental results.

Keywords: precast structure; ambient vibration method; dynamic characteristics.

1. Introduction

This work represents a part of the integral report prepared within the framework of the project: Attesting of the Static and Dynamic Stability of Typified Moduli of the Hall Program of the Precast RC Structural System "AMONT-Krusce, Yugoslavia".

For the purpose of definition of basic dynamic characteristics of constructed structures of precast hall program "AMONT-Krusce" an experimental nondestructive method based on measuring of ambient vibration has been applied. By use of this method resonant frequencies can be registered, that is corresponding vibration periods, and can be determined damping coefficients as well as shapes of vibrations at the defined resonant frequencies. These measurements are carried out at already constructed full-scale structures, so that obtained results reflect the real state of the structure. The results are the needed base for real analytical modeling of the structure that has to be done as a part of the analysis of seismic stability of the structure.

Complete processing and analysis of the obtained experimental data have been carried out and a part of the results from the investigation is presented in this paper.

2. Testing results

The integral report contains results from the performed experimental tests on the dynamic characteristics of four structures constructed according to the "AMONT" system. A part of testing results in the form of Fourier spectra for measured ambient vibrations is shown in Fig.1.

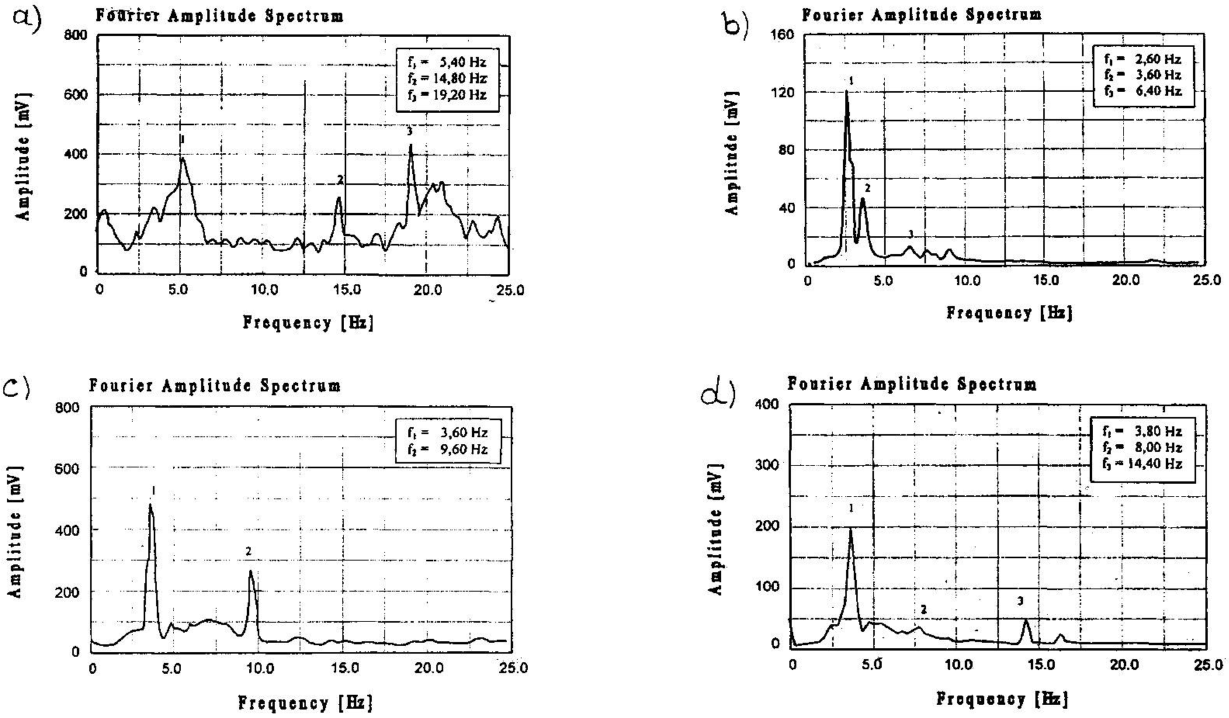


Fig. 1 Fourier spectra for measured ambient vibrations in transversal direction for: a) one-bay one-story structure; b) and c) two different two-story structures with two bays in the second story level, one without filling walls and the other with filling walls, respectively; d) two-story structure with one bay in the second story level.

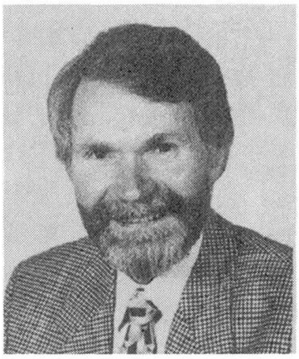
3. Conclusion

Taking into account the integral results of defined real dynamic characteristics of chosen representative halls constructed in "AMONT" structural system, the following can be stated:

- Correct mathematical modeling and analysis is made possible based on the results of measured real dynamic characteristics of constructed buildings for the purpose of the future application.
- Very divergent opinions about the achieved "level of rigidity of connection" of some precast structural elements, which before has led to parametric analyses and fluctuation of results, are avoided now.
- Very useful information about influence of masonry and other finishing elements on dynamic characteristics of two similar two-floor structures for two different phases of construction is obtained.
- It is important to note that measured values are direct indicators only of the "initial" dynamic characteristics, which can be significantly changed after the first cracks, and particularly after the connections come into nonlinear range of behavior.

Punching of Flat Slabs – Comparison of Design and Construction

Uwe ALBRECHT
Prof. Dr.-Ing.
FH Nordostniedersachsen
Buxtehude, Germany



Uwe Albrecht, born 1940, received his civil engineering degree from the University of Hannover and his doctorate from the Technical University of Darmstadt. After 16 years in the construction industry he was appointed Professor of Structural Engineering in Buxtehude (near Hamburg) in 1983

Summary

The provisions for the design and construction of reinforced concrete flat slabs differ considerably. The punching provisions will be compared with respect to the shear capacity, the common shear resistance of concrete and shear reinforcement and the relevant detailing of the reinforcement specified in

Germany	DIN 1045
	E DIN 1045-1
Eurocode	EC 2
UK	BS 8110
USA	ACI 318
Canada	CSA A23.3
Model Code	CEB-FIP 1990

Keywords: Concrete structures, flat slabs, punching, shear resistance, shear reinforcement, integrity reinforcement

1. Shear resistance

The shear resistance is calculated on a critical perimeter u which is located $0.5d$ to $2d$ from the face of the column (d.... effective depth, h....overall thickness). Fig. 1 shows the location for an interior column with rectangular cross-section $c_1 \cdot c_2$. Some codes require that the shear resistance outside the shear reinforced zone should be checked at successive perimeters u_2, u_3.

The shear resistance is expressed by

V_{Rd1} ....provided by the concrete

V_{Rd2} ....maximum shear resistance with shear reinforcement, the capacity is limited by the concrete strength of the struts

Fig. 2 results from the material data of C35 (cube) or C30 (cylinder) and the assumption

$d = 0.85\ h$ $\qquad \alpha = (c_1 + c_2) / 2\ h$

All codes, with the exception of DIN 1045, use partial safety factors which are, however, not identical. Therefore the comparison is based on service loads. For simplification, a common safety factor γ_F for permanent (G_k) and imposed load (Q_k) has been chosen for each code and the size factor refers to d = 25cm.

With shear reinforcement, the shear capacity may be enhanced up to $\approx 40\%$ according to DIN 1045 or up to a maximum of >60% according to BS 8110 and CEB-FIP 1990. Fig. 3 shows the shear reinforcement required for V_{Rd2}, assuming a yield stress of $f_{yk} = 500$ N/mm². For better comparison, the shear resistance specified in BS 8110 and CEB-FIP 1990 has been limited to $V_{Rd2} = 1.6\ V_{Rd1}$.

2. Conclusions

The paper reveals on the one hand considerable differences with respect to the shear capacity and to the amount and fitting of shear reinforcement and integrity reinforcement and on the other hand that design and detailing form an integral whole. The different effect of the flexural reinforcement ratio on the shear capacity is remarkable, particularly as the North American codes do not take the reinforcement ratio into account at all.

The rules for the common shear resistance of concrete and the shear reinforcement are manifold because of the different assessment of the effectiveness of the shear reinforcement. There is a need for tests to overcome this contradiction. Some codes require an integrity reinforcement provided by continuous bottom bars passing within the column cage to prevent progressive collapse of the structural system in the case of local punching. Others do not even mention this reinforcement.

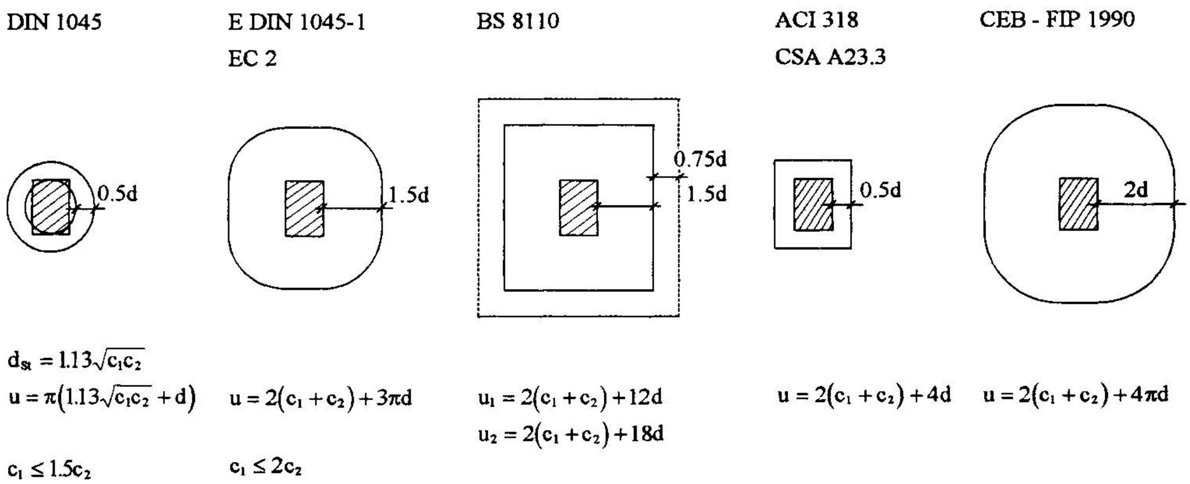


Fig. 1 Critical perimeter

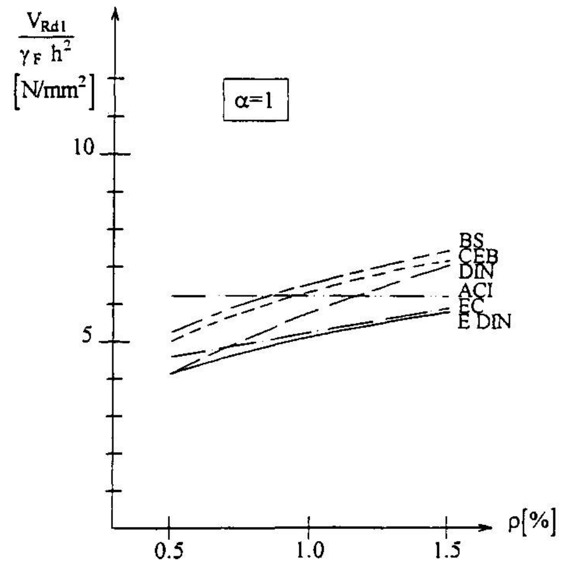


Fig. 2 Interior column:
Shear resistance without shear reinforcement

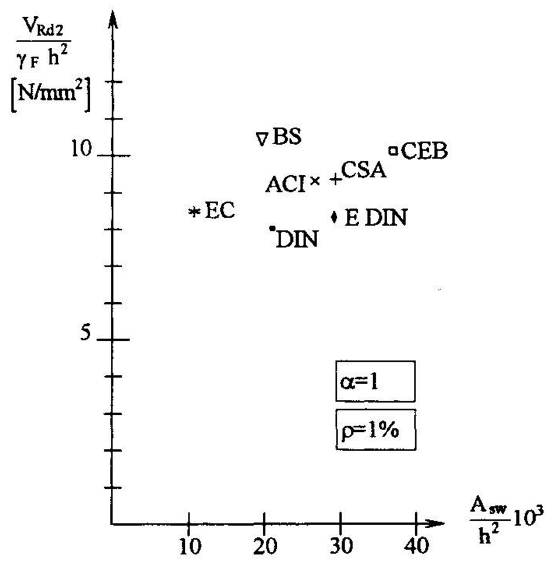
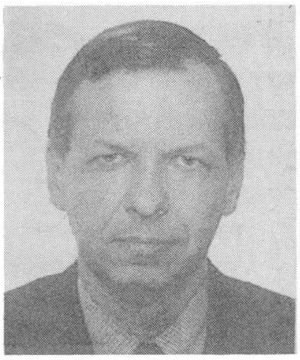


Fig. 3 Interior column: Shear reinforcement

Optimisation of Composite Waffle Slab Structure Design

Petr HÁJEK
Assoc. Professor
Czech Technical University
Prague, Czech Republic



Petr Hájek, born 1955, received his civil engineering degree from the Czech Technical University in Prague. Assoc. Professor of building construction at the CTU. Research activities: optimisation in building design, structural analysis, optimum design of composite waffle slabs, environmental structural design

Summary

The quality of structural design and the resulting quality of the structure's performance depends on the level of knowledge of structural behaviour. The structural behaviour of RC waffle floor slabs and composite waffle slabs has been theoretically analysed and verified in a wide range of experiments. The results of the tests, supported by theoretical conclusions, have confirmed significantly better structural properties of the composite waffle slabs than the assumptions commonly considered in analysis models. Primary theoretical assumptions of high torsional rigidity of waffle slabs have been proven. The ribs of the tested specimens were however not reinforced with shear and torsional reinforcement.

Keywords: reinforced concrete, waffle slab, ceramic fillers, hollow bricks, experiments, torsion, flexure, optimisation, quality design

1. Introduction

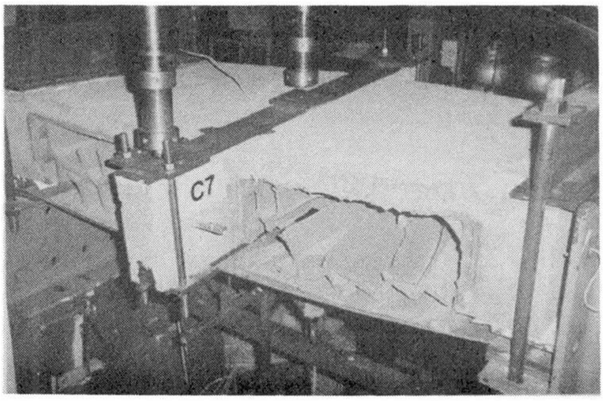
The development of construction technologies should be influenced by the effort to decrease consumption of material and energy sources while increasing the structure's serviceability, durability and reliability throughout its entire expected life. This general need for quality design and the resulting quality of the structure's performance is determined by the level of knowledge of the structural behaviour of the corresponding structure. A complex optimisation of material and energy flows within the whole life of the structure should therefore become a necessary part of the quality design approach.

A better understanding of the structural behaviour of composite waffle structures is the necessary basis for the development of more realistic and precise structural analysis models and for the improvement of code requirements with the general aim to decrease the cost and to increase the serviceability and reliability of the structure i.e. to increase the quality of the design as well as the quality of the final waffle structure performance.

2. Experimental Investigation

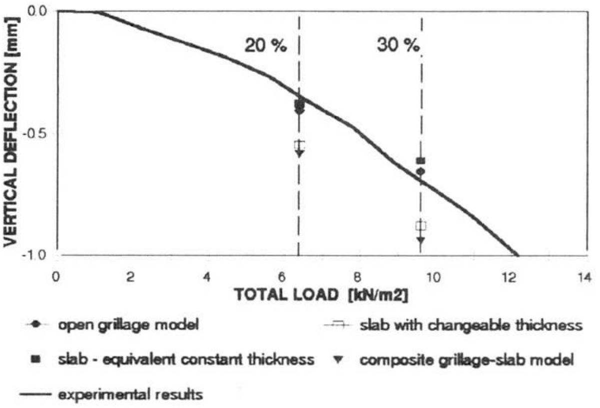
To investigate the structural behaviour of composite waffle slabs exposed to flexural and torsional loads, three types of specimens were tested. Test specimens were subjected to different combinations of flexural and torsional loads. A full scale test on a composite waffle slab with ceramic fillers (size 3.15m × 3.15m) was carried out in 1996. The results of all these tests supported the theoretical assumptions and verified the proposed analysis models.

3. Composite Action of Ceramic and Concrete in the Composite Section



The interaction of fillers with the concrete part of the section is determined by the mechanical properties of the fillers. In the case of brick fillers the ceramic material has a relatively high compression strength while the tensile strength is very low. Moreover, brick ceramic material is brittle. Thus, the interaction of ceramic fillers with concrete can only be considered if any part of the filler is in the pressure zone of the section. The quality of interaction depends on the bond strength between ceramics and concrete. Experiments showed, that the part of the ceramic filler which was in direct contact with the concrete, was in an effective composite action until structural failure of the whole composite structure occurred. The internal parts of hollow brick fillers failed just before the structural failure, usually in the stage when deflections were over the corresponding serviceability limits.

4. Comparison of Analysis Models with Experiments



A composite waffle slab structure can be considered in structural analysis models as an open grid with appropriately substituted properties for the corresponding beam elements or as a slab structure with an equivalent thickness. Four proposed analysis models have been compared with the experiments. The comparison is presented in the graph. Deflections in the centre of the slab are shown for the load steps representing 20% and 30% of the total load when structural collapse occurred during the loading test.

5. Conclusions

1. The results of testing have confirmed high torsional as well as flexural rigidity and ultimate bearing capacity of composite waffle slab structures, even when the ribs were without shear and torsional reinforcement. The composite waffle slab behaves very similarly to the full RC slab with reduced thickness.

2. The significant coupled action of hollow brick elements with concrete has been proven. The ultimate bearing capacity in flexure of a composite waffle slab was approx. 15 to 30% higher than that of an RC waffle slab without fillers. The ultimate bearing capacity in torsion of a composite waffle slab was even approx. 60 to 90% higher.

3. New analysis equivalent models for structural analysis of composite waffle structures have been described and compared with the experimental results. This comparison confirmed the possibility of using the simpler slab model with constant thickness or the grillage model.

4. Optimisation of reinforcement of composite waffle slab structures is possible by using the same principles and corresponding code conditions which are generally used for full RC slabs.

The theoretical and experimental research of composite waffle slab structures is supported by the Grant Agency of the Czech Republic, Grant No. 103/98/1480 and Grant No. 103/98/0091.

Anchorage of 90-Degree Hooked Beam Bars in Exterior RC Joints

Osamu JOH
Professor
Hokkaido University
Sapporo, Japan

Yasuaki GOTO
Associate Professor
Hokkaido University
Sapporo, Japan

Osamu Joh, born 1943, received his Dr .Eng. from Hokkaido Univ.1971. Research field is seismic design of RC structures and their rehabilitation.

Yasuaki Goto, born 1958 received his D.Eng. from Hokkaido Univ.1996. Research interest is RC beam-column joints by experiment & computing.

Summary

This study experimentally investigated anchorage performances for raking-out failure mode o f 90-degree hooked beam bars in exterior beam-column joints to be used in a middle story, using forty column-shaped specimens with various arrangements of L-shaped beam bar anc horage. Based on the test results, a formula for accurate estimation of anchorage strength w as proposed.

Keywords: anchorage; 90-degree hook; beam-column joint; ultimate strength; developm ent length; concrete cover thickness; axial stress; lateral reinforcement.

1. Introduction

In the previous paper, we divided the anchorage failure of 90-degree hooked bars in exterio r beam-column joints in a middle story into three modes: side split failure, local compressi on failure and raking-out failure. Most of the current structural design codes require that ho oked bars be designed according to certain specifications, such as minimum requirements of concrete cover thickness, bar bend radius and location of the bend in a joint (in order to avoid the three anchorage failure modes, respectively) rather than according to thoretical cal culations of strength. In the same paper[ACI, SP-157, pp.97-116, 1995], we also proposed a formula for estimating anchorage strength of 90-degree hooked bars with raking-out failure mode. In the present paper, we have purposed a modified formula that can estimate ancho rage strength more accurately and can be applied to various designs of hooked bars anchore d in a joint.

2. Experiment

Specimens, about half normal size, had 90-degree hooked beam bars anchored into an exteri or beam-column joint situated at the midpoint of the column height. Two series of specime ns were tested: an inward and an outward hook series, in which the tails of the beam bars were directed to the joint side and the column side, respectively. We tested eleven variabl es, including horizontal development length (Ldh), moment arm of beam (jb), thickness of c over concrete, lateral reinforcement ratio, axial stress (σ o), as shown in the table. A schem a of the dimensions and bar arrangement of the specimens is shown in Fig.1. One tensile l oad, P1, was applied horizontally to the beam bars, where reactions R1 and R2 were suppo rted, and another tensile load, P2, was applied at the top of the column to generate the sa me shear force in both columns.

3. Estimation of Anchorage Strength

Fig. 2 is a schema showing a typical crack pattern in the inward hook series at the final l oading stage. The main cracks related to anchorage strength were in a chain of ⑦-③-④-⑥. The failure plane slid heavily along cracks ④-⑥ so that concrete slippage on this plane worked as a part of horizontal resistance, and the main cracks opened widely at the ultimat e stage so that the hoops crossing the failure plane worked as the remainder of the horizon tal resistance. The following modified formula was proposed for estimating anchorage streng th with raking-out failure:

$$ {}_{cal}T_u = k_N \, ({}_{cal}T_c + {}_{cal}T_w), \quad (1) $$

where ${}_{cal}T_c$ *(horizontal concrete resistance)*= $H / j_b \cdot {}_{cal}T_{ao}$ *for inward hooks,* ${}_{cal}T_c = H / (H - j_b) \cdot {}_{cal}T_{ao}$ *for outward hooks, and H is story height. The value of* ${}_{cal}T_{ao}$ *is analytical horizontal resistance of the failure plane and is defined by the equation* ${}_{cal}T_{ao} = k_c \cdot b_{ce} \cdot L_c \cdot \sigma_e$, *where* k_c *is a coefficient that takes into account tail direction,* b_{ce} *(effective colum n width)* $= S + 0.53\, b_c$, b_c *is the total concrete side cover thickness for beam bars,* L_c *is t he length of horizontal straight bar length minus concrete cover thickness, and* σ_e *(concret e sliding strength)* $= \sqrt{\sigma_B}$. *In Equ.(1),* ${}_{cal}T_w$ *is the horizontal resistance of hoops and is d efined by the equation* ${}_{cal}T_w = k_w \cdot k_b \cdot a_w \cdot \sigma_{wy}$, *where* k_w *(effective hoop stress factor) is 0.8 for inward hooks and 0.9 for outward hooks;* k_b *(effective concrete cover thickness fact or) is 1 at* $C_o \leqq 0.8 L_{dh}$, $3 - 2.5 C_o/L_{dh}$ *at* $0.8 < C_o/L_{dh} < 1.2$, and 0 at $1.2 \leqq C_o$; a_w *is the total sectional area of hoops within the effective zone; and* σ_{wy} *is the yield stress of the hoops. The axial stress modification factor* (k_N) *has to be used to estimate the fact that the colu mn axial stress increased the anchorage strength, but there is a upper limit to the strength -enhancing effect of axial stress. Therefore,* k_N *is defined as follows: in the case of inward hooks,* $k_N = 1 + 0.205\, \sigma_{os}$, *where* σ_{os} *is the minimum between* σ_o *and* $0.08\, \sigma_B$; *and in t he case of outward hooks,* $k_N = 1 + 0.153\, \sigma_{os}$, *where* σ_{os} *is the minimum between* σ_o *a nd* $0.16\, \sigma_B$. *The average of the ratio of observed strength* ${}_{exp}T_u$ *to* ${}_{cal}T_u$ *for 40 specimens was 1.01, and the standard deviation was 0.091. As can be seen from the relation between* ${}_{exp}T_u$ *and* ${}_{cal}T_u$ *plotted in Fig. 3, the new estimations appear to be highly accurate.*

4. Conclusions

The following conclusions were drawn on the basis of the results.

1) The anchorage strength of outward hooks is less than that of inward hooks.
2) Our previous method for estimating horizontal concrete resistance and hoop resistance in anchorage strength was extended to outward hooks as well as inward hooks.
3) A new equation for estimating the anchorage strength of 90-degree hooked bars in exteri or beam-column joints was derived from our previous equation. Our experimental results co nfirmed that estimations using this new equation have a high level of accuracy.

Steel Energy Absorbers for Seismic Building Rehabilitation

Keh-Chyuan TSAI
Professor of Civil ENG
Nat. Taiwan University
Taipei, Taiwan, China

K.C. Tsai, born 1955, received his Ph.D. degree from the Univ. of California at Berkeley in 1988. Licensed Civil and Structural Engineers in Taiwan and California.

Li-Chong HUANG
Project Manger
Federal ENG Consultants
Taipei, Taiwan, China

L.C. Huang, born 1970, received his M.S. degree in Structural Engineering from National Taiwan University joined Federal Engineering Consultant in 1994.

Summary

Experimental responses of two type of seismic energy dissipators, namely the triangular-plate added damping and stiffness (TADAS) device and the shear link energy absorber (SLEA) are briefly summarized. Both two types of energy absorbing devices can sustain a large number of yielding reversals and increase the damping and stiffness of the frame structures. It is illustrated that the force versus deformation relationships of a building structure constructed with these two types of energy dissipator can be adequately characterized by a tri-linear model. Based on the results of extensive response spectrum analyses, optimal stiffness and strength ratios between the device and frame are recognized for the rehabilitated structures. The paper concludes with examples illustrating the seismic design procedures for building rehabilitation using the proposed energy dissipators to resist severe earthquake excitations.

Keywords: energy dissipator, seismic retrofit, rehabilitation, seismic resistant building design

1. Introduction

Modern seismic resistant design procedures generally adopt a set of reduced seismic forces to account for the ductility capacity of the structures. In the meantime, it often requires the compliance of specific ductility design provisions in order to assure a ductile behavior of the system. While a new building can be designed to achieve certain strength and ductility, however, a large number of existing buildings may not possess the needed strength and ductility to sustain strong earthquake excitations. In addition, structural failures of modern buildings observed following some recent earthquakes have suggested that a ductility-based, rather than a performance based, design methodology may not be reliable for modern seismic hazard mitigation of buildings, particularly from the socio-economical point of views. This is evidenced by the brittle fractures of welded steel moment connections due to the difficulties of controlling material properties, construction workmanship and inspections. Thus, in addition to searching for sound technology for the construction of ductility-enhanced members or connections, the general consensus has been on how to find reliable solutions to reduce ductility demands imposed on these structural components. For this purpose, two types of energy absorbing devices have been developed at the National Taiwan University and proven promising in improving the performance of existing and new building structures in resisting severe earthquake excitations. This paper addresses the deign issues of the proposed energy absorbers for the rehabilitation of existing building structures.

2. Steel Hysteretic Energy Dissipators (HEDs)

Since the bending curvature of a transverse load applied at the end of a triangular plate is uniform over the full height of the triangular plate, the plate can deform well into the inelastic range without curvature concentrations. As shown in Fig. 1, the proposed steel triangular-plate added damping and stiffness (TADAS) device consists of several triangular plates welded to a common base plate. Similarly, a short steel wide flange beam under cantilever load can yield in shear while remain elastic in flexure. As shown in Fig. 1, the proposed shear link energy absorber (SLEA) consists of a short steel wide flange beam segment welded to an end plate. Experimental results (see Fig. 1) have confirmed that properly constructed TADAS and SLEA elements possess highly predictable mechanical properties, and can sustain a large number of yielding reversals.

The energy dissipator and the moment resisting frame (MRF) are combined in series, and the force versus deformation relationships of a hysteretic energy dissipated frame (HEDF) can be adequately characterized by a tri-linear model as illustrated in Fig. 2. This paper summarizes the results of the extensive response spectrum analyses for seismic rehabilitation:

• For short period structures (T < 0.8 sec.), a *SR* value ranging from 2 to 4 and an Ω value between 1.2 and 2.0 should be considered. For long period structures (T > 2.0 sec.), it is recommended that a *SR* on the order of 1.0 to 2.0 and an Ω value between 1.2 and 2.0 be considered.

3. Conclusions

The following conclusions can be drawn from this study:

• Experimental response of well constructed TADAS and SLEA can sustain large inelastic cyclic deformations. Thus, they appear promising for seismic resistant constructions of building structures.

• The nonlinear deformational demands imposed on the beam-to-column connections of steel MRFs can be effectively mitigated by incorporating properly proportioned steel energy absorbers.

• The proposed design strategies incorporate the modern two-level seismic-resistant design methodologies, ensure to satisfy the service limit state while facilitate the review of the overstrength factor for controlled nonlinear responses.

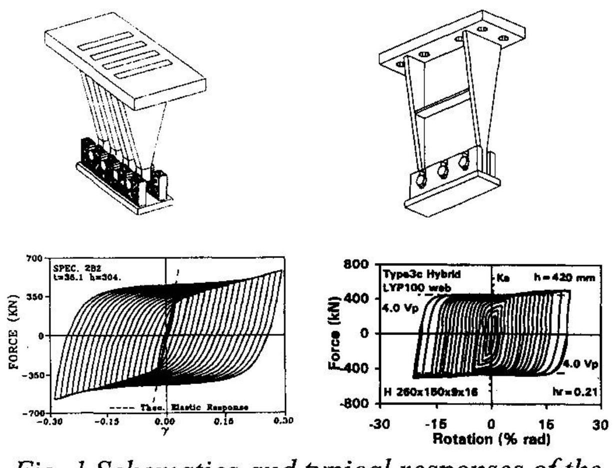


Fig. 1 Schematics and typical responses of the TADAS and SLEA devices

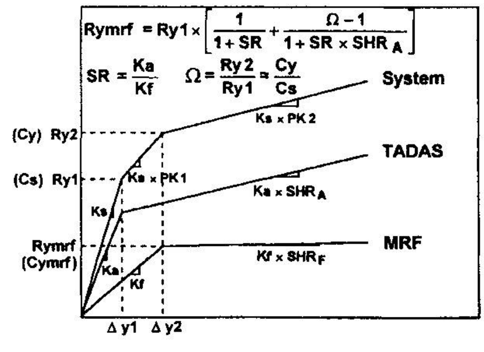


Fig. 2 Tri-linear force vs. deformation relationships of the HEDF system

Thin Walled Steel Hollow Sections with Concrete Infill

S. K. BHATTACHARYYA
Professor
University of Durban-WestVille
Durban, South Africa

S. K. GHOSH
Principal Research Manager
R&D Centre for Iron & Steel SAIL
Ranchi, India

P. K. MAINI
Research Engineer
R&D Centre for Iron & Steel SAIL
Ranchi, India

D. KARMAKAR
Principal Research Engineer
R&D Centre for Iron & Steel SAIL
Ranchi, India

Summary

To overcome the difficulties of hot rolled open steel sections, cold formed hollow sections are used as structural elements. Use of concrete as infill material for the hollow sections may overcome the difficulty of local buckling. This paper presents an experimental and theoretical study on concrete filled column members. Results show that the filled sections are much more efficient than equivalent hollow or concrete sections.

Keywords: Thin walled section; Closed section; steel-concrete composite; infilled section; Finite Element Technique; Experimental Study.

1. Introduction

The hot rolled open steel sections, used as structural members, suffer from a few drawbacks such as buckling about weaker axis, torsional instability etc. Thin walled closed sections are proved to be a viable solution in this direction. Use of infill concrete further enhances the capability of the closed sections.
Several researchers have presented their studies on the behaviour of concrete filled light gauge steel tubular members.
This paper presents an experimental study on the behaviour of concrete filled column members with circular, rectangular and square cross sections along with the development of a theoretical model for the infilled circular section.

2. Experimental Study

An experimental study has been performed with the circular, square and rectangular sections to study the behaviour of the closed form infilled sections. The columns are tested under direct compressive load. Fig.1 shows the test set up and arrangement adopted for testing the specimens.

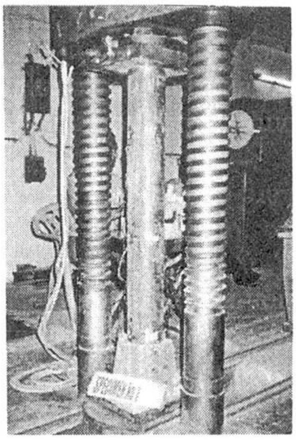


Fig.1: Arrangement for Testing Columns

3. Theoretical Formulation

The circular infilled section is analysed using finite element technique. A computer code is developed for the analysis of concrete filled closed circular columns.

4. Results & Discussion

The results of observed strain data and of theoretical analysis are presented in graphical form.

5. Conclusions

Based on the experimental study and theoretical analysis a set of meaningful conclusions are drawn.

6. References

[1] Bolterman Peter, "Short Thin-walled cylinders under combined loading.", *ASCE Journal of Structural Engineering*, Vol.117, No.3, 1991, pp. 951-971.

[2] Chan S. L., "Strength of cold-formed box columns with coupled local and global buckling" - *Journal of The Inst. of Structural Engineer*, 1990, pp.125-131.

[3] Dutta B. and S. K. Bhattacharyya, "An experimental study on the behaviour of concrete filled light gauge steel tubular members", *Indian Concrete Journal.* Vol.69, No.8, 1995.

[4] Hanbin Ge. and T. Usami T., "Strength of concrete filled thin-walled steel box columns: experiment", *ASCE Journal of Structural Engineering*, Vol.118, No.11, 1992, pp.3036-3054.

Natural Thermal Behaviour of Polish Bridges

Henryk ZOBEL
Associate Professor
Warsaw University of Technology
Warsaw, Poland

Henryk Zobel, born 1950, received his civil engineering degree from Warsaw University of Technology in 1972 and Ph.D. in 1978.

Thakaa ALKHAFAJI
Ph.D.
Warsaw University of Technology
Warsaw, Poland

Thakaa Alkhafaji, born 1957, received her civil engineering degree from Moscow Autom. & Road Construction Institute in 1987, Ph.D. from Warsaw University of Technology in 1994.

Dariusz SOBALA
M.Sc.
Rzeszow University of Technology
Rzeszow, Poland

Dariusz Sobala, born 1966, received her civil engineering degree from Rzeszow University of Technology in 1990.

Summary

Bridges expand and contract due to temperature changes. Actual bridge temperatures may be quite different from idealization used in design. Recent analytical studies have shown that existing procedures are often irrational. Sometimes predicted movements are too small and this leads to damage and deterioration in the bridge. These will significantly shorten the bridge life. In other cases, the movements are too large and this leads to the selection of inappropriate expansion joints and bearings. In case of integral structures, it leads for over-dimensioning. The paper presents results of ongoing research in Poland. Preparation of new design guidelines for selection of bridge temperatures and thermal movements is the aim of this effort.

1. Introduction

Methods of determination of bridge temperatures and thermal movements have been developed for some countries and even put into standards. Extensive studies were done in Europe lately, because of preparation of Eurocodes. The values of bridge temperatures were obtained mostly through computer simulation. Unfortunately, experimental verification of these results through field measurements is rare and random. Therefore, extensive investigation of existing structures is still needed. Polish Standard devoted to thermal loading of bridges is quite conservative and did not change since many years. Growing number of damages of expansion joints, bearings and abutments due to thermal loads proves importance of these phenomena. The separate problem is taking into account thermal actions during construction of bridges. This is a reason that project sponsored by Polish government research agency is developed. New guidance for determination of bridge temperature and thermal movements will be reached through:
- statistical analysis of ambient temperature around Poland,
- determination of relation between ambient temperature and bridge temperature depending on the type of structure,
- 3-dimensional, comparative thermal-structural analysis,
- verification of computer simulation results with field measurements and observations of temperatures and movements in chosen bridges.

2. Bridge Temperature

The calculation of the bridge temperatures is based on the radiation, convection and conduction heat flow. Radiation is provided by sun. It heats the bridge during the day and transmits the heat to the environment on nights. Convection is largely driven by the wind and by air currents caused by moving traffic. Conduction describes the flow of heat within the bridge. Accurate determination of the bridge temperature requires consideration of all 3 components of heat flow including such factors like: air temperature, cloud cover, air pollution, wind speed, the angle of the sun, the time of day, the orientation of the structure with respect to the sun, topographical location, geometry of the bridge, type of cross-section and material of the structure. This is well known that 2-dimensional mathematical model of temperature distribution is usually acceptable for most type of bridges. 3-dimensional heat flow model is required in unusual circumstances only, where the temperature is expected to vary significantly along the length of the bridge.

3. Bridge Thermal Movements

Thermal movements are accommodated by bridge bearings and expansion joints. In other case the piers and abutments may be integrally constructed with the superstructure and than thermal movements are accommodated by pier deflection or movement of the abutment into the backfill. The movements are calculated by a simple uniaxial expansion equation. This model gives reasonable solution for straight bridges working in "normal" meteorological conditions.

4. The project realized in Poland

Extreme temperature weather data over the past 30 years were analyzed for 25 locations in Poland. The locations were selected to cover the variation in weather conditions and terrain within each region. The average distance between meteorological stations is from 50 km in flat area to 30 km in mountains. The data include dates and locations of extreme air temperatures and daily and yearly temperature variation. This weather data are used to calculate bridge temperatures. The measurements of temperatures and thermal movements are made on 2 straight bridges. First one is three span, continuous, plate girder with composite slab highway bridge (l =152 m). The second is simple supported plate girder with orthotropic deck railway bridge (l = 20 m). The movements are measured with photogrammetrical cameras. Photos are numerically analyzed. The comparative computer thermal-structural analysis is done. The temperature distribution within the bridge is done using FETAB program. These data is used for ANSYS system to calculate 3-dimensional thermal movements. The first results of investigations and computer simulation are encouraging. It looks that for composite and steel bridges standard requirement for maximum bridge temperature could be diminished from + 55° C to + 50° C. The temperature difference between top and bottom of structure should be increased from ± 15° C to ± 20° C.

5. Conclusion

The research will benefit Poland in new and rational recommendations for establishing bridge temperatures and thermal movements in design and construction. These recommendations will result in better long-term performance of bridges. It will reduces the initial cost of the bridge and its maintenance costs.

Prestressed Aluminium Arch Dam

Cawas NAZIR
Technical Director
MN Dastur & Co Ltd
Calcutta, India

Cawas Nazir, born 1939, received his Bachelor of civil engineering degree from the University of Nagpur in 1959 & Master of Engineering degree from University of Sheffield in 1962. He has specialised in the Design of Industrial Structures and Foundations.

Abstract

Arch dams provide elegant and economic solutions for dam sites where the valley contracts in plan and where the banks are composed of strong monolithic rock. To provide for a more economical structure and one which could be constructed much faster, a new design has been proposed called the Prestressed Aluminium Arch Dam.

The original concept developed from the fact that arch dams carry their load in compression though ideally, the more logical way of containing any liquid is by tension members. However, since rock is generally weak in tension, it was decided to use a structural system that would take advantage of the tensile membrane action, yet load the foundation in an optional manner in compression for the hydrostatic load. This was hoped to be achieved by constructing the wall from material having high tensile strength and high resistance to corrosion and then prestressing it so that stresses opposite to those originating from hydrostatic load are produced .

The new design presented comprises of a wall composed of horizontally curved multicellular box sections fabricated from structural aluminium alloy and prestressed with high tensile bolts. The bolts and wall section are anchored to anchorage blocks embedded in the banks.

The resulting approach led to a parabolic profile being chosen for the dam with the rise of the arch at the crest level kept at one fourth of the span length and with the wall axis being kept vertical. The principal loading criteria included: a) design of wall as a tensile arch in the reservoir empty condition and prestressed with a force that would produce half the equivalent water load, the stresses induced being predominantly tensile; b) in the final stage when reservoir is full the arch is designed as a conventional arch and the stresses are predominantly compressive. Thus at any stage the structure is effectively designed for only half the water load.

The more important steps in the design process included: fitting of parabolic arch profile to dam site; selecting suitable sections for the wall, foundations and anchorage's; making rigorous stress analysis both for static and dynamic load conditions; and making a model analysis.

The construction aspects briefly describe the various stages of construction viz. diverting of river; laying out the dam; excavation of foundations; construction of foundation/anchorage's; fabrication of aluminium alloy box girders; erecting, welding and prestressing them; installing service facilities; and instrumenting the dam.

As a practical design application an example is given of a 54.5 m high Prestressed Aluminium Arch Dam, for a site where the valley width at crest level is 66.67 m and base width is 12.12 m. The wall forming the body of the dam is made up of aluminium alloy box girders having depth at crest of 1.8 m and gradually increasing to 3.4 m at base. The height between horizontal diaphragms is 2 m and the vertical diaphragms are spaced at 4 m centres. Analysis is carried out for a typical 2 m high box girder at mid height having a parabolic profile of span length 47.92 m and rise of 8.05 m. The principal loading conditions considered are; as a tensile arch under prestressing load to produce an equivalent uniform load of half the hydrostatic load; and as a conventional arch under full hydrostatic load with a temperature variation of 8°C. The analysis is made using the simplified edge perturbation method of shell theory. For the various loading conditions, values of moment, hoop force and shear are evaluated. Based on these forces, the stresses in the various elements of the box girder are calculated. The structural aluminium alloy used is of grade 6061-T6 having an ultimate tensile strength of 294 MN/m^2. The allowable stress considered take into account the buckling factor and an assumed factor of safety of 1.85. For the sections selected, all stresses are shown to be within allowable limits. Based on the prestressing force, the number of high tensile bolts are calculated using steel having ultimate tensile strength of 1500 MN/m^2.

A design is given for a typical wedge shaped anchor block having maximum width of 6 m and embedded 5.25 m into hard granite rock. An analysis based on the failure of the rock wedge due to the prestressing force is made and the anchorage is shown to have an adequate factor of safety against the pull out force.

A preliminary idea of the economics of the new design is obtained by comparing its cost with that for a conventional concrete arch gravity dam using the same parameters of the site as indicated in the example. The main features of the dam are height 54.5 m, radii at top of crest 35.4 m; central angle 150°, thickness at crest 2.6 m and at base 10.13. The major quantities used for comparison are excavation, concrete, aluminim alloy and HT bolts. Based on the prevailing rates a cost advantage is clearly shown.

Other advantages indicated for the new design include: a) the use of aluminium alloy and HT bolts offers a much higher strength to weight ratio leading to significant advantages in transportation, handling, structural design and load bearing capacity; b) prestressing of aluminium wall permits balancing of water load leading to a more economical design; c) repetition of multi cellular boxes forming the body of the dam leads to optimal saving in construction cost and time; d) the wall is free from corrosion and hence maintenance cost are greatly reduced; e) the hollow passages within the dam, lead to better conditions of heat release, creating favourable thermal conditions; f) savings in substructure cost because of substantial reduction of dead weight; g) modern welding processes permit leaking joints to be more easily repaired; h) the cavities in the multi-cellular box construction serve as excellent inspection galleries, permitting the entire face of the dam to be monitored; i) the parabolic arch profile permits of a more gentle arch abutment conjugation and j) the dam has exceptional grace and beauty and neatly expresses its function.

It is concluded that innovative design and modern materials of construction can be used to create a dam type that will be a viable alternative to concrete arch dams.

Development of Bridge Design Codes in Russia

Oleg POPOV
General Manager
Giprotransmost J.S.Co
Moscow, Russia

Oleg Popov, graduated from Moscow Institute of Railway Transport Engineering in 1952. He has more than 46 years of experience in bridge engineering

Vadim SELIVERSTOV
Civil Engineer
Giprotransmost J.S.Co
Moscow, Russia

Vadim Seliverstov, graduated from Moscow Automobile and Road Technical University in 1981. He has been involved in many major bridge projects

Summary

This paper describes the concept and contents of the Russian codes and standards currently used in practice of bridge design and construction. The current design practice based on the limit states specified for the first and second groups are discussed. It also introduces the new system of specifications for design and construction sector. At this moment several activities have started for further development of the structural and engineering bridge design codes. One of the main objectives is to achieve international harmonisation regarding requirements on reliability of structures with Eurocodes and ISO standards.

Keywords: bridges; codes; design; reliability; limit states

1. Introduction

From 1962 the limit state principles were started to be adopted for design of bridges and culverts in the former USSR. At the beginning three groups of limit states were outlined in the bridge design standard – predecessor to the Bridge code currently in use. Later on in 1976 the concepts were refined and in the modern practice two limit states are specified by the State standard ГОСТ 27751-88. Codes are divided into three basic classes – those covering design, construction and workmanship levels, and materials.

2. Codes in Common Use

Main principles for design of bridges in Russia are specified in СНиП 2.05.03-84*. This standard is in one part covering design of bridges in steel, concrete and composite construction and is also used in the republics of CIS. The Bridge code covers design of new and rehabilitation of existing bridges and culverts for highways, railways, tramways, metro lines and also combined (highway-railway) bridges. The requirements specified are for the location of the structures in all climatic conditions in the former USSR, and for seismic regions of magnitude up to 9 (ground acceleration of 0.4g) on the scale of Institute of soil physics (former USSR Academy of science).

Construction and installation practices including the workmanship levels and requirements related to temporary structures are specified by the other code - CHuП 3.06.04-91.

Clearances of highway bridges are specified on the basis of highway classification. The minimum horizontal and vertical clearances are given by the Bridge code. The clearances of railroad bridges are based on the requirements of State standard 9238-83. Besides the highway and railroad clearances the bridges must also satisfy the navigational requirements.

The analysis of stream action on bridges is based on the design flood. The return period of the design flood is specified by the Bridge code and represents a fixed value dependant on the category of railway or highway on which a bridge is located. In the typical practice this range is from 100 year to 33 year flood.

In the existing Russian design practice the evolution of bridge design includes three major stages. These are feasibility study, preliminary design, final design. If the bridge project is simple or the bridge is of small or medium size the design is elaborated in one stage. Normally all stages are developed by a single design company. In the typical practice erection design forms a part of the complete design package. The main advantage of this system is that an interaction of structural solutions with the existing fabrication and erection techniques may be reached.

3. Codes Development and Renewal

A new system of specifications for design and construction in Russia have been introduced by CHuП 10-01-94. This new system was put in power in 1995 and established three levels of normative documents: Federal codes and standards; Regional codes; Standards of branches of industry.

In accordance with the basic principles of new system of normative documents for construction the design of bridges in Russia (besides Building norms and regulations) must also satisfy the requirements of Codes of practices and Regional normative documents. These standards are used to introduce new requirements and also to resolve inconsistencies found in the Bridge code.

In connection with the Moscow Ring Road widening project the "Additional requirements for design and construction of bridges on the Moscow Ring Road" had been worked out. These Additional requirements were approved in 1995. The bridge structures for this project had been designed taking into account the increased live loading specified by the Additional requirements.

The previously worked out Additional requirements formed a basis for the new standard TCH 32. This new standard requires bridges on the specified routes to be designed for increased live loading and abnormal loading. Also it reflects the specifics of bridge design in Moscow and some other new requirements are established in order to improve the reliability and durability of the structures.

4. Conclusion Remarks

In order to interact the existing codes with the new system the preparatory works for revision the codes are currently under way. Some of the newly developed codes have already been issued and put into power. These are the codes which cover surveys for construction.