Zeitschrift: IABSE reports of the working commissions = Rapports des commissions de travail AIPC = IVBH Berichte der Arbeitskommissionen

Band: 19 (1974)

Rubrik: Session I: National reports

SESSION I SITZUNG I SESSION I

Rapports nationaux

Nationale Berichte

National Reports

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SEMINAR on:
«CONCRETE STRUCTURES SUBJECTED TO TRIAXIAL STRESSES»
17th-19th MAY, 1974 - ISMES - BERGAMO (ITALY)

Etudes et réalisations françaises récentes de structures en béton soumises à des contraintes triaxiales

Recent french studies and realizations of concrete structures subject to triaxial stresses

Neue, projektierte und ausgeführte, französische Betonwerke, die dreiaxialen Spannungen ausgesetzt sind

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I. Introduction - Structures envisagées

Tout ouvrage réel est soumis à des contraintes triaxiales, mais la plupart du temps on peut avec une très bonne approximation négliger l'effet de la triaxialité. Aussi les structures pour lesquelles l'ingénieur s'astreint à tenir compte de la triaxialité des contraintes sont elles assez rares. On peut citer principalement les barrages épais, les fondations massives de certaines machines, et les caissons de réacteurs nucléaires à gaz sous pression.

Dans les réalisations françaises récentes, les structures qui ont donné lieu aux études et aux expérimentations les plus intéressantes sont sans conteste les caissons de réacteurs de la filière graphite-gaz qui, tant par leurs dimensions que par les questions de sécurité extrêmement importantes qu'ils posaient, nécessitaient un soin particulier dans l'élaboration du projet et le contrôle et la surveillance de l'ouvrage.

En revanche, il n'y a pas eu récemment à notre connaissance de réalisations très marquantes dans le domaine des barrages ou des structures industrielles massives. Pour les barrages, en particulier, le rythme de leur construction a beaucoup baissé, en raison du haut niveau d'équipement déjà atteint. Mais les services d'E.D.F. et certains bureaux d'études spécialistes de barrages ont participé très activement à l'étude des caissons de réacteurs, et ont appliqué à cette occasion les techniques mises au point pour les barrages.

Plutôt que de tenter de faire une revue complète des structures soumises à des contraintes triaxiales, il a donc paru préférable de se limiter presqu'exclusivement aux caissons de réacteurs, domaine dans lequel la participation de la France est particulièrement importante et originale.

II. Caractéristiques principales des caissons de réacteurs construits en France

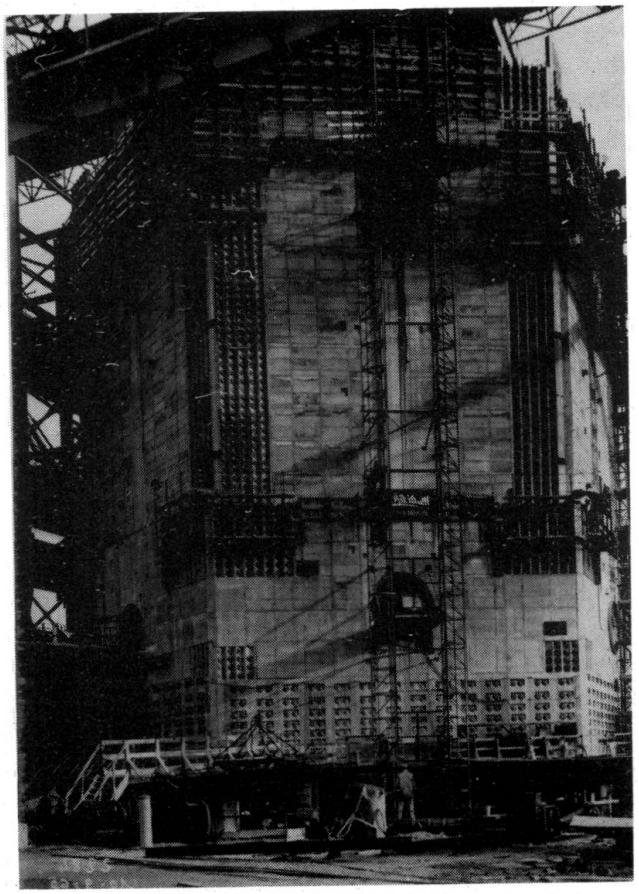
Tous ces caissons ont été construits pour des réacteurs de la filière graphite-gaz et contiennent donc du gaz carbonique en pression et en température. L'étanchéité est assurée par une tôle d'acier, appelée "peau", revêtant la face intérieure du béton précontraint. L'isolement thermique est assuré par un calorifuge fixé sur la peau et un système de refroidissement. De ce fait, le gradient de température dans l'épaisseur des parois est limité à moins d'une dizaine de degrés par mètre en régime permanent.

Pour les caissons des réacteurs G2 et G3 de Marcoule, la pression du gaz est de 15 bars. La structure a grossièrement la forme d'un cylindre horizontal fermé par des coupoles hémisphériques à convexité tournée vers l'intérieur. La cavité intérieure a un diamètre de 14 m et une longueur minimum de 15,50 m. L'épaisseur des parois est de 3 m.

Pour le caisson d'E.D.F. 3 à Chinon, la pression du gaz est de 27 bars. La cavité intérieure est en gros un cylindre vertical de 19 m de diamètre et 21 m de hauteur. La forme extérieure de la structure est à peu près parallélépipédique. L'épaisseur minimale des parois est de 5 m.

Les caissons de Saint-Laurent-des-Eaux 1 et 2 et de Vandellos (Espagne), identiques, supportent une pression de gaz de 27 bars. La cavité intérieure est un cylindre vertical de 19 m de diamètre et 36 m de hauteur, fermé par des dalles planes. La forme extérieure du fût de la structure est hexagonale. L'épaisseur minimale des parois latérales est de 4,75 m, celle des dalles de 6 m et 5,66 m.

Le caisson de Bugey 1 est soumis à une pression de 43 bars. La cavité intérieure est un cylindre vertical de 17,10 m de diamètre et 40 m de hauteur, fermé par des dalles planes. La forme extérieure du fût est cylindrique, avec trois nervures d'ancrage des câbles en saillie. L'épaisseur de la paroi du fût est de 5,50 m, celle des dalles de 7,50 m.



Caisson de Saint-Laurent 1 en construction (photo Baranger)

Saint-Laurent 1 vessel during construction (Baranger photograph)

Saint-Laurent 1 Druckgefäss im Bau begriffen (Photo Baranger)

III. Méthodes et programmes de calcul

Bien que les caissons de réacteurs aient quelques particularités les différenciant des structures courantes (symétrie de révolution plus ou moins parfaite, action de la température), les méthodes et programmes de calcul employés ont évidemment un champ d'application infiniment plus vaste.

D'une façon générale, les programmes utilisés sont basés sur la méthode des éléments finis, maintenant bien connue.

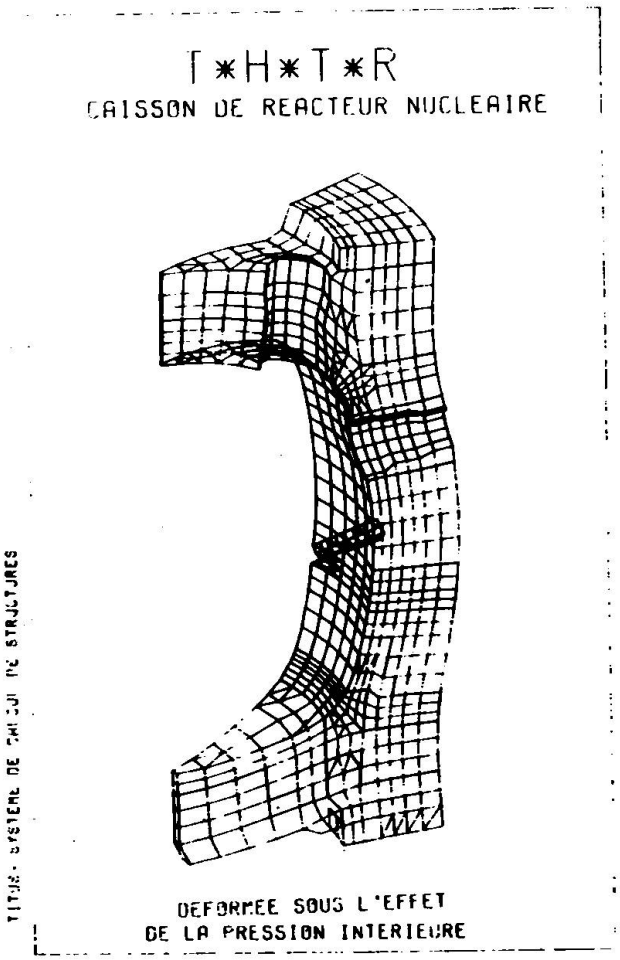
Les points les plus importants à souligner sont les suivants.

Pour l'élaboration et la vérification d'un projet, on se place en général dans le cadre de l'élasticité linéaire. Il peut être avantageux d'effectuer d'abord un dégrossissage par des méthodes simples pouvant paraître grossièrement approchées et négligeant plus ou moins complètement l'aspect tridimentionnel. Ensuite, pour la vérification, il est préférable lorsque cela est possible de simplifier et scinder le problème en étapes successives. On y gagne en temps (donc en argent) et en précision. Par exemple pour un caisson de réacteur, on effectue d'abord une vérification d'ensemble en assimilant la structure à un solide de révolution, et en utilisant un programme axisymétrique bidimensionnel. Ensuite, chaque zone singulière échappant à la symétrie de révolution est étudiée à l'aide d'un programme tridimensionnel. Il faut bien entendu définir correctement les frontières de ces zones et les conditions aux limites imposées pour les contraintes et les déplacements.

Plus que sur les méthodes de calcul, maintenant bien connues, les efforts ont porté ces dernières années sur la détermination automatique du maillage en éléments, avec la possibilité de resserrer le maillage dans les zones délicates, où l'on peut craindre des concentrations et des variations rapides des contraintes, et sur l'amélioration des procédures d'entrée et de sortie, de façon à permettre à l'ingénieur projeteur d'intervenir facilement, de n'appeler dans un gros programme que les parties nécessaires à chaque phase d'élaboration du projet, et de modifier certaines données géométriques ou mécaniques lorsque cela apparaît nécessaire. En particulier, plusieurs programmes comportent la possibilité d'obtenir des dessins automatiques du maillage, des contraintes principales, et des déformations de la structure pour différents cas de charge.

Les caractéristiques mécaniques du matériau peuvent varier en fonction du point considéré de la structure (ferraillage important, revêtement en tôle d'acier par exemple) ou de la température. Aussi la plupart des programmes actuels permettent-ils d'entrer ces caractéristiques comme une fonction des coordonnées de l'élément considéré et de la température.

Lorsque le projet d'une structure est terminé, et qu'il s'agit d'interpréter, soit des essais sur maquette, soit le comportement au cours du temps de l'ouvrage réel, un programme purement élastique est insuffisant. Aussi des programmes plus élaborés ont été développés, en particulier par Electricité de France. L'hypothèse de base est celle de la visco-élasticité linéaire du béton. A l'heure actuelle, les programmes opérationnels sont des programmes monodimensionnels, bidimensionnels, et axisymétriques. Leur exploitation a montré un très bon accord avec les résultats expérimentaux et a confirmé que le modèle rhéologique ainsi choisi représentait bien le comportement du béton.



Caisson de réacteur T.H.T.R. - Déformée sous l'action de la pression (programme TITUS)

T.H.T.R. reactor vessel. Deformation due to pressure (TITUS code)

T.H.T.R. Reaktor-Druckgefäss. Formveränderung durch Druck (TITUS-Programm)

Il n'existe pas à notre connaissance de programme de calcul du comportement d'une structure en phase de fissuration et jusqu'à rupture. Certains programmes pourraient peut-être être utilisés en modifiant les caractéristiques mécaniques des zones fissurées et en effectuant ainsi un calcul pas à pas. Il ne semble pas que ceci présente un grand intérêt, puisqu'on est au-delà du comportement normal de l'ouvrage en service.

Enfin, les calculs à la rupture sont en général effectués manuellement.

IV. Expérimentation sur maquettes

L'emploi de maquettes pour étudier des structures complexes et vérifier leur comportement est déjà ancien. Avant le développement des moyens de calcul modernes, c'était à vrai dire à peu près le seul moyen dont disposaient les ingénieurs.

Si, comme nous l'avons déjà dit, il n'y a pas eu en France ces dernières années d'études importantes ou originales dans le domaine des barrages, de nombreuses maquettes de caissons de réacteur en béton précontraint ont été construites et expérimentées.

Tout d'abord, pour les caissons G2 et G3 de Marcoule, la Direction des Etudes et Recherches d'Electricité de France a dirigé pour le compte du Commissariat à l'Energie Atomique les essais de trois maquettes identiques.

Le choix s'est porté sur des maquettes en béton, constituant des modèles de résistance. L'échelle des contraintes et des caractéristiques mécaniques est donc de 1. L'échelle géométrique choisie était de 1/10, minimum compatible avec la possibilité de mise en place correcte du micro-béton représentant le béton de l'ouvrage réel. Les conditions de similitude n'étaient évidemment pas respectées pour le poids propre, ce qui était peu important en raison de la grande prépondérance des efforts dus à la pression interne.

Un soin particulier a été apporté à la représentation à l'échelle, dans toute la mesure du possible, de la peau d'étanchéité, des câbles de précontrainte, et des singularités, en particulier des divers orifices.

Les essais effectués ont été uniquement des essais en pression, sans aucun essai thermique. Ils ont comporté des essais en phase élastique, dans le domaine des pressions de service, et un essai rapide de rupture au gaz par augmentation de la pression.

L'instrumentation comprenait des appareils de mesure de déplacements (comparateurs), des détecteurs d'ouverture de joints, et pour les câbles des dynamomètres et des détecteurs de rupture. La petitesse de l'échelle n'avait pas permis de prévoir des extensomètres car leurs dimensions auraient été trop grandes pour donner des indications valables compte tenu de la variation du champ de contraintes.

Les résultats obtenus ont été très satisfaisants et ont montré le bon comportement élastique de la structure et sa grande marge de sécurité par rapport à la rupture.

Tous les caissons de réacteurs suivants, E. D. F. 3, Saint-Laurent 1 et 2, et Bugey 1, ont été construits pour Electricité de France. Leurs différentes maquettes, projetées et construites par les constructeurs, en liaison avec la Direction des Etudes et Recherches d'E. D. F., ont été expérimentées par cette dernière. Les essais de chaque maquette ont ainsi profité des enseignements des précédents.

Toutes les maquettes sont des modèles de résistance, pour lesquels là encore les efforts dus au poids propre, très faibles devant ceux de la pression, ne vérifient pas les lois de la similitude.

Pour le caisson d'E. D. F. 3, trois maquettes à l'échelle de 1/6 ont été construites.

Les deux premières maquettes d'E. D. F. 3 ont été essayées uniquement en pression. La mise en pression était hydraulique. De nombreux cycles ont été effectués jusqu'à la pression de service (27 bars) et au-delà. Enfin, les essais se terminaient par un essai de rupture lent par augmentation de la pression.

Ces deux premières maquettes ne différaient que sur un point : sur la première la peau d'étanchéité intérieure en tôle d'acier était remplacée par une membrane en caoutchouc, alors que sur la seconde cette peau d'acier était représentée à l'échelle.

L'instrumentation comprenait des capteurs de déplacement, des extensomètres à corde vibrante (témoins sonores) noyés dans le béton, des jauges à fil résistant placées à la surface du béton et de la peau d'étanchéité, et des dynamomètres pour la mesure des tensions des câbles de précontrainte. Un dispositif d'enregistrement automatique des mesures était prévu.

En outre, la rupture était observée à l'aide d'un circuit de télévision, et des photos et un film en étaient pris par des appareils télécommandés.

Les essais ont montré que les maquettes se comportaient sensiblement comme le laissaient prévoir les calculs, et ont confirmé le surcroît de résistance apporté par la peau d'étanchéité, et l'excellent comportement d'ensemble de la structure.

La troisième maquette a été soumise aux mêmes essais, et aussi à des essais en température, cumulée ou non avec la pression.

A cet effet, elle a été équipée en outre de thermocouples et d'hygromètres. On a simulé en outre des accidents improbables, comme des câbles de précontrainte détendus ou des plages chaudes sur la peau dues à une détérioration du calorifuge de l'ouvrage.

La représentation des conditions thermiques de l'ouvrage réel est impossible sur une maquette de résistance, car l'échelle des temps est le carré de l'échelle linéaire, et il en résulte que les régimes thermiques s'établissent beaucoup plus vite que dans l'ouvrage réel. Or dans ce dernier, la variation très lente de la température fait que le module d'Young intervenant dans l'établissement des contraintes thermiques est certainement sensiblement inférieur au module instantané. On a donc renoncé à étudier sur la troisième maquette d'E. D. F. 3 les régimes thermiques transitoires et on a tenté de reproduire le régime permanent par une montée très lente de la température de la maquette.

Les essais de cette troisième maquette ont été poussés jusqu'à des températures très supérieures à la température de service. Ils ont confirmé l'excellent comportement de l'ouvrage.

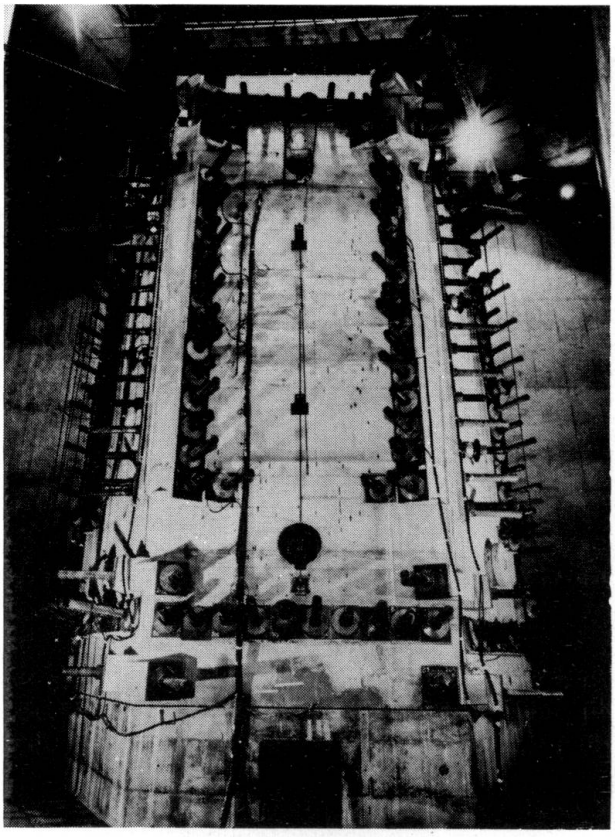
L'essai de rupture de cette troisième maquette a été fait en température (plus exactement avec un gradient thermique dans l'épaisseur du béton). Il a montré que la pression de rupture était sensiblement la même que pour la deuxième maquette et n'était donc pas influencée par l'état thermique de la structure. Ce résultat très intéressant confirme les théories de la plasticité qui montrent qu'un état d'autocontrainte initial est sans influence sur la résistance ultime d'une structure.

L'instrumentation et les techniques d'essai mises au point pour la troisième maquette d'E. D. F. 3 se sont avérées excellentes et ont été reconduites pratiquement sans changement pour les maquettes des caissons de Saint-Laurent 1 et du Bugey, construites et essayées dans le même site que les maquettes d'E. D. F. 3, à Chinon.

Pour le caisson de Saint-Laurent 1, il avait été décidé de construire une seule maquette à l'échelle de 1/5. Le programme d'essais prévoyait :

- des essais élastiques sans température destinés à vérifier le calcul des contraintes dues à la précontrainte et à la pression, jusqu'à des pressions supérieures à la pression de calcul et voisines de la pression de fissuration;

- des essais thermiques avec montée en température rapide ou lente et paliers de stabilisation;



Maquette n° 1 de Saint-Laurent 1 avant rupture (photo Baranger)

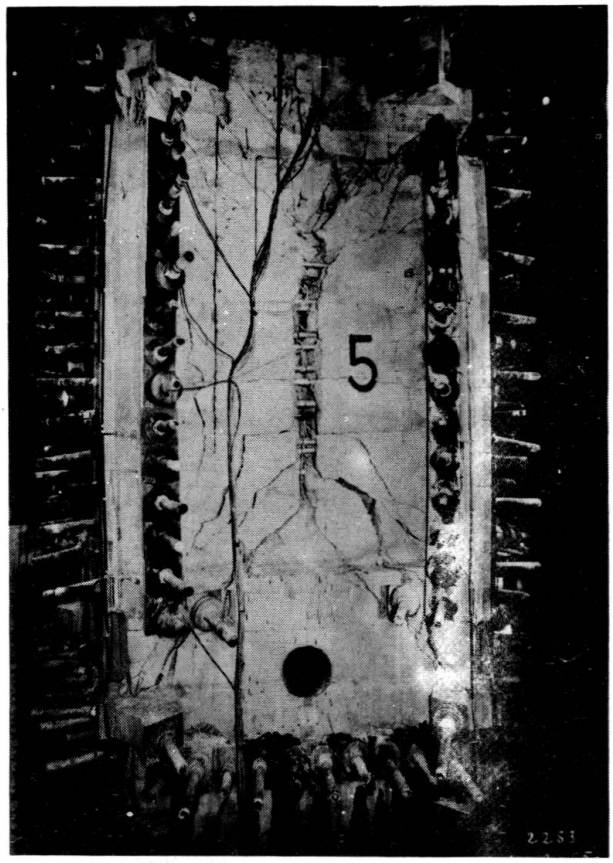
Model N° 1 of Saint-Laurent 1 before rupture (Baranger photograph)

Modell Nr 1 von Saint-Laurent 1 vor Bruch (photo Baranger)

- des essais en pression de courte ou longue durée couplés avec les essais précédents;

- et finalement l'essai de rupture.

Les essais ont montré un bon comportement de l'ouvrage, et en particulier une sécurité très satisfaisante vis-à-vis de la rupture, mais ont mis en évidence une fissuration prématurée de la face externe due à un mauvais tracé de la précontrainte circonférentielle du cylindre.



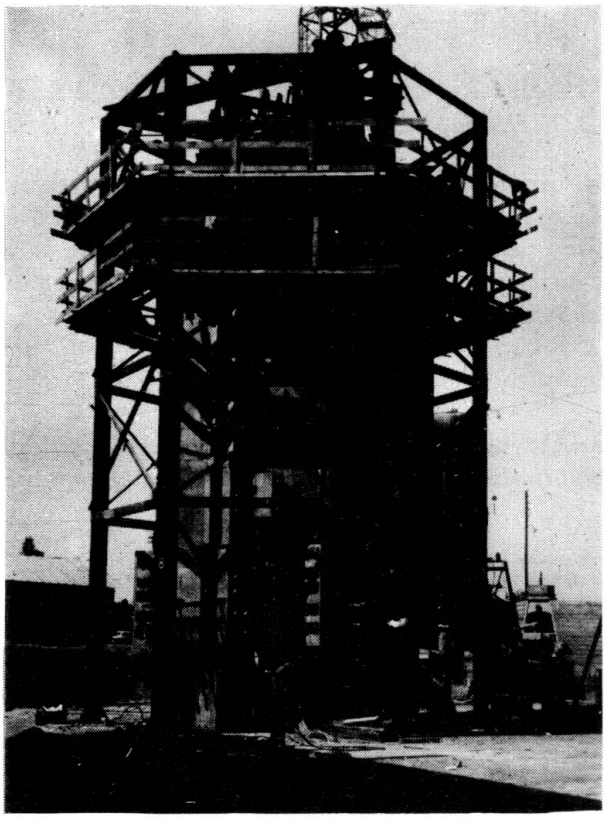
Maquette n° 1 de Saint-Laurent 1 après rupture (photo Baranger)

Model N° 1 of Saint-Laurent 1 after rupture (Baranger Photograph)

Modell Nr 1 von Saint-Laurent 1 nach Bruch (photo Baranger)

Aussi une seconde maquette au 1/5 a-t-elle été construite, avec un tracé de précontrainte modifié et amélioré, et soumise au même programme d'essais. Les résultats ont été entièrement satisfaisants.

Pour le caisson du Bugey, deux maquettes au 1/5 furent construites et essayées. La première était destinée à des essais sans température, la seconde à des essais en pression et température. A la fin du programme d'essais, chaque maquette a été soumise à un essai de rupture. Là encore, les résultats ont été excellents et ont confirmé le bon comportement de la structure projetée.



Maquette n° 1 de Bugey 1

Model N° 1 of Bugey 1

Modell Nı 1 von Bugey 1

Un autre problème délicat a été étudié sur maquettes pour les caissons de Saint-Laurent et du Bugey, celui des vibrations des consoles de soufflantes. Dans ces caissons à circuit primaire intégré, les soufflantes, ancrées dans la paroi massive des caissons, sont entraînées par des turbines portées par des consoles massives en béton précontraint encastrées dans le caisson, et il est indispensable pour que le bon fonctionnement de l'arbre soit assuré que les amplitudes des vibrations de ces consoles restent très faibles pour toutes les fréquences inférieures ou égales à 50 hertz. Après un dégrossissage des formes convenables par

le calcul, différentes formes de consoles ont été testées, sur une maquette spéciale au 1/10 pour Saint-Laurent 1, et sur la deuxième maquette au 1/5 pour Bugey.

V. Mesures et contrôles sur les ouvrages - Comparaison avec les essais sur maquettes - Surveillance en service

Les divers caissons de réacteurs ont été équipés de nombreux instruments de mesure destinés, d'une part à contrôler leur comportement correct lors de la mise en précontrainte et de la mise en service, d'autre part à surveiller leur évolution ultérieure et s'assurer de leur sécurité dans le temps.

Sont employés :

- des instruments destinés à mesurer les déplacements et déformations d'ensemble : repères de nivellement, pendules inversés, fils Invar;

- des instruments destinés à mesurer les déformations locales, aussi bien en zone courante qu'aux points singuliers : témoins sonores et jauges de contraintes (strain - gauges);

- des thermocouples destinés à mesurer les températures, et des hygromètres permettant de suivre les migrations d'eau dans le béton;

- des dynamomètres permettant de mesurer la tension de certains câbles de précontrainte.

On remarquera que la plupart de ces instruments sont ceux là même qui sont utilisés pour l'auscultation et la surveillance des barrages.

Le dépouillement des mesures, leur interprétation, et la comparaison avec les essais sur maquette sont délicats en raison de la complexité de l'histoire du béton de la structure réelle : coulage par plots, échelonné sur plus d'un an, mise en précontrainte en plusieurs phases, etc.. On réussit néanmoins à obtenir en général une très bonne concordance pour les phénomènes rapides (et donc élastiques) comme la mise en pression et certaines phases de mise en précontrainte, et une interprétation très acceptable des autres phénomènes. Mais tout ceci exige un travail considérable et coûteux, et l'emploi d'ordinateurs puissants tant pour le dépouillement, les corrections et l'interprétation des mesures brutes que pour la comparaison avec les résultats d'un calcul élastique ou plus souvent visco-élastique.

La surveillance d'un ouvrage après sa mise en service pose des problèmes plutôt plus simples. Il faut avant tout vérifier qu'il ne se produit pas de variations anormales dans les mesures. Pour cela, on teste, à intervalles réguliers, un certain nombre d'instruments judicieusement choisis, et, connaissant les conditions de charges correspondantes, on réussit à séparer, par voie statistique, d'une part les variations réversibles liées à la pression interne et à l'état thermique de la structure, d'autre part les variations irréversibles liées au retrait, au fluage, à la relaxation des armatures de précontrainte, et éventuellement à une dégradation des qualités de la structure. Cette dernière éventualité ne s'est encore fort heureusement pas réalisée.

Une telle surveillance nécessite l'intervention de spécialistes ayant une pratique courante de cette discipline et est assurée en France par la Division Technique Générale de la Production Hydraulique d'E. D. F., qui assure aussi la surveillance des barrages.

Une mention spéciale doit être réservée au caisson du Bugey. L'instrumentation de cet ouvrage est nettement plus importante que celle des autres caissons, et ceci dans un but de recherche. On peut ainsi espérer que l'exploitation des mesures faites sur cet ouvrage permettra d'accéder à une meilleure connaissance des structures en béton précontraint sollicitées triaxialement et soumises à l'action de la température.

VI. Etudes et recherches en cours

De nombreuses études sur les caissons en béton précontraint ont été effectuées ces dernières années et se poursuivent actuellement, principalement dans le cadre d'un programme de recherches entrepris par le Commissariat à l'Energie Atomique, Electricité de France, et la Société d'Etudes des Caissons Nucléaires.

L'emploi de caissons en béton précontraint pour les filières à eau légère (B. W. R. et P. W. R.) a été particulièrement examiné, et plusieurs projets de caissons ont été étudiés. Des solutions satisfaisantes, tant du point de vue de la sécurité que de l'économie, ont été mises au point.

La principale difficulté dans ces projets est la valeur élevée de la pression du fluide intérieur (80 bars environ pour le B. W. R. et 150 bars environ pour le P. W. R.), qui nécessite des contraintes admissibles élevées dans le béton. Des bétons à haute résistance ont été mis au point dans ce but, et permettent de résoudre le problème.

On pourrait certainement simplifier les projets et abaisser leur coût en tenant compte de la triaxialité des contraintes et de façon plus précise en relevant les contraintes limites admissibles lorsque les trois contraintes principales sont des compressions. Mais les connaissances actuelles sur le béton, si elles montrent indiscutablement que la résistance est augmentée en compression triaxiale, ne sont pas encore suffisantes pour permettre d'évaluer numériquement de façon précise cette augmentation. En particulier, les règlements officiels sont muets sur cette question. Des études et des essais se poursuivent sur ce sujet.

Un autre problème encore mal connu est la valeur des contraintes d'origine thermique. Pour leur calcul, il faudrait connaître la valeur du module d'Young à prendre en compte en fonction de la vitesse d'établissement du gradient thermique et éventuellement du signe de la contrainte correspondante. Pour les caissons de Saint-Laurent et du Bugey, on avait admis que compte tenu de l'inertie thermique de la structure, on pouvait prendre un module égal à la moitié du module instantané pour le calcul des contraintes thermiques en régime permanent. Une telle hypothèse paraît raisonnable mais est évidemment assez arbitraire.

Or, il n'est pas facile de déterminer expérimentalement sur les maquettes la valeur réelle des contraintes thermiques parce que les appareils de mesure ne permettent pas de mesurer directement les contraintes, mais seulement les déformations. C'est seulement au moment de la fissuration, si on connaissait avec précision la valeur de la contrainte de fissuration du béton, qu'un essai permettrait de trouver la vraie valeur de la contrainte thermique.

Il serait donc extrêmement utile de disposer d'un appareil de mesure directe des contraintes. Un tel appareil "capteur de contraintes" a été imaginé par monsieur Beaujoint(E. D. F.) et est expérimenté sur le chantier du pont de Saint-Cloud. Il est malheureusement limité à la mesure des contraintes de compressions, mais pourra certainement rendre de grands services si les essais entrepris sont concluants.

VII. Conclusion

Les réalisations de ces dernières années ont montré que l'état actuel de la technique permettait de projeter des structures tridimensionnelles sûres et dont le comportement était à peu de choses près conforme aux prévisions déduites du calcul ou des essais sur maquettes.

L'expérience déduite de ces réalisations et le perfectionnement constant des méthodes et des programmes de calcul rendent possible dès maintenant des progrès importants tant dans la prévision du comportement des structures que dans l'évaluation de leur sécurité, et donc permettent d'espérer des économies appréciables.

Par exemple, bien que l'expérimentation sur maquettes ait des avantages irremplaçables, et en particulier permette éventuellement de découvrir des erreurs de conception ou des dispositions défectueuses qui auraient pu échapper à la vérification par le calcul du fait d'une mauvaise modélisation de la structure, les progrès actuels du calcul permettent de diminuer notablement le nombre et l'importance des maquettes à prévoir.

Néanmoins, des progrès sont encore à faire, car notre connaissance du comportement du béton sous contraintes triaxiales et sous sollicitations thermiques est encore insuffisante. Il semble que les axes de recherche pour les années futures devraient être la poursuite des études sur la résistance du béton sous charges statiques, sur sa résistance à des charges variables, et sur ses déformations différées, et si possible la mise au point d'appareils de mesure directe des contraintes, sans passer par l'intermédiaire des déformations.

REFERENCES

C. VOUILLON - Système de calcul des structures sur ordinateur."TITUS". (Revue Française de Mécanique n° 40, 1971)

J.M. BOISSERIE - Generation of two-and three-dimensional finite elements (International Journal for numerical methods in ingineering, vol 3, 1971)

J.M. BOISSERIE - Modélisation du caisson du réacteur de Bugey (First International Conference on Structural Mechanics in Reactor Technology, Berlin, Septembre 1972)

G. LAMIRAL, J. COURBON - Les caissons en béton précontraint dans les centrales nucléaires d'E.D.F. 3 et d'E.D.F. 4 (Annales de l'I.T.B.T.P., Février 1965)

P. LAUNAY - Etude et construction du caisson en béton précontraint de la centrale nucléaire de Bugey 1 (Annales de l'I.T.B.T.P., Mai 1970)

N. BEAUJOINT - Similitude et théorie des modèles (Bulletin RILEM n° 7, 1960)

N. BEAUJOINT - Etudes sur modèles de structure de barrages et de génie nucléaire (Annales de l'I. T. B. T. P., Novembre 1961)

M. LIDA, R. AUSANGEE - Modèles réduits de résistance des caissons de réacteurs nucléaires en béton précontraint (Conference on prestressed concrete pressure vessels, Londres, Mars 1967)

J. P. MEROT, M. LANGUILLE - Caissons en béton précontraint pour B. W. R. (Commission des Communautés Européennes, Deuxième réunion d'information sur les cuves de réacteur en béton précontraint, Novembre 1969)

R. LACROIX - Le programme de recherche et développement E. D. F. - C. E. A. -S. E. C. N. pour les caissons en béton précontraint (NUCLEX 1972, Technical Meeting n° 2)

J. P. MAGNAS, A. AUDIBERT - Critères de résistance du béton sous sollicitations multiaxiales (Annales de l'I. T. B. T. P., Supplément du n° 287, Novembre 1971)

RESUME

Les structures les plus intéressantes construites en France ces dernières années sont les caissons de réacteurs. Ces ouvrages sont décrits sommairement. Puis les principales caractéristiques des programmes de calcul utilisés sont données. On décrit ensuite plus en détail les expérimentations sur maquettes, leurs résultats et leur interprétation. On évoque les mesures et les contrôles des ouvrages en service, ainsi que leur surveillance. Un aperçu des études et des recherches en cours est donné. Enfin, l'auteur indique les voies de recherche qui lui paraissent les plus importantes pour l'avenir.

SUMMARY

The most noticeable structures carried out in France during the past few years are reactor vessels. These realizations are briefly described. Then the main characteristics of the computing codes are set forth. More precise descriptions of model tests and of their results and interpretation thereof follow. Measures and checkings of projects in operation, as well as their maintenance, are outlined. A summary of designs and researches in progress is given. Finally, the author points out the lines of research which he considers the most important for the future.

ZUSAMMENFASSUNG

Die bedeutendsten Bauwerke, welche in den letzten Jahren in Frankreich ausgeführt wurden, sind die Reaktordruckgefässe. Diese Bauwerke sind kurz beschrieben. Es folgen die wichtigsten Eigenheiten über den angewandten Berechnungsplan. Die Modellversuche, sowie deren Ergebnisse und daraus hervorgehenden Folgerungen, werden eingehender geschildert. Ferner werden die Messungen und Prüfungen, der sich im Dienst befindenden Projekte und deren Überwachung erwähnt. Es wird eine Ubersicht, der in der Ausführung begriffenen Projektierungen und Forschungen, gegeben. Schliesslich äussert sich der Verfasser über die Forschungswege, die seines Erachtens für die Zukunft am wichtigsten sind.

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SEMINAR on:
«CONCRETE STRUCTURES SUBJECTED TO TRIAXIAL STRESSES»
17th-19th MAY, 1974 - ISMES - BERGAMO (ITALY)

Österreichischer Gesamtbericht

Austrian national report

Rapport général Autrichien

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1. Einleitung

Der weitgesteckte Rahmen des Seminars, von Spannbetondruckgefäßen, Bogengewichtsmauern, Tunnelauskleidungen, Maschinenfundamenten bis zu Verankerungsproblemen, erfordert eine Unterteilung des nationalen Berichtes in Hauptgebiete. Es darf dem Berichterstatter überlassen bleiben, auch nach dem Gesichtspunkt persönlicher Arbeitsgebiete in diesem Bericht den Umfang der einzelnen Teilgebiete abzugrenzen, in dem einen ausführlich zu berichten und in dem anderen sich wesentlich auf Literaturhinweise und fremde Arbeiten zu stützen. Es sollen die Gebiete aus weiterem Gesichtswinkel betrachtet, auch auf benachbarte oder unmittelbar zusammenhängende hingewiesen und, wo am Platz, Vergleiche angestellt werden. Der Bericht umfaßt die Arbeitsgebiete

Sperren,
Tunnel- und Hohlraumauskleidungen,
Spannbetondruckgefäße.

Die ohne Zweifel in Probleme dreiachsiger Beanspruchung mündenden Fragen, z.B. bei Wehranlagen in der Verankerung großer Kräfte der Verschlüsse oder bei Maschinenfundamenten mit mehrachsigen Zwängungsbeanspruchungen aus Temperatur sollen hier nicht betrachtet werden, womit jedoch keines-

falls eine Wertung der Bedeutung dieser Fragen verbunden sein soll.

Auf dem ersten der Arbeitsgebiete besitzt Österreich eine Reihe bedeutender Bauwerke und entspricht damit den anderen Alpen- oder Gebirgsländern, insbesondere auch unserem Gastland. Im zweiten gehen österreichische Ingenieure und Wissenschaftler mit Erfolg eigene Wege. Im dritten, das sich in rascher Entwicklung befindet, werden Wege beschritten, die ebenfalls hoffnungsvoll sind und Beachtung verdienen.

In allen Gebieten ist das Ziel der Ingenieurarbeit, die Sicherheit des Werkes möglichst gut beurteilen, sie richtig wählen zu können, dabei wirtschaftlich zu bleiben und schließlich auch andere an ihn gestellte Forderungen zu erfüllen.

Die Fragen mehrachsiger Beanspruchungen, das Entstehen solcher Zustände oder deren Veränderung durch das Bauwerk, die Wahl der zu ihrer Ermittlung und zur Verfolgung ihrer Veränderung zweckmäßigen Verfahren, Berechnung und Versuch sind Kernprobleme in allen drei betrachteten Arbeitsgebieten.

Die Frage der Beanspruchung ist untrennbar mit der des Materialverhaltens verbunden und mit der zweckentsprechenden Wahl des Werkstoffes und seiner Güte.

Sicherheit ist nicht nur eine Frage der Berechnung, sondern auch der Beobachtung des Verhaltens eines Bauwerkes und der wechselweisen Einflüsse zwischen ihm und der Umgebung. Auch darauf muß eingegangen und verwiesen werden. Sie ist primär auf den Menschen bezogen, auf sein Leben direkt, aber auch mittelbar auf die Befriedigung seiner Lebensbedürfnisse, so daß ihre Bedeutung auch für diesen Punkt betrachtet werden muß.

Nachstehend soll, fußend auf den persönlichen Beiträgen und darüberhinausgehend, über diese Fragen berichtet werden.

2. Sperren

Zunächst einige kurze Bemerkungen zur allgemeinen Bedeutung.

Der Ausbau der Wasserkraftanlagen hat in Österreich und wohl auch in Mitteleuropa seinen Höhepunkt überschritten. Die Ausbauwürdigkeit der noch bestehenden Möglichkeiten nimmt selbstverständlich laufend ab, zumindest unter den derzeitigen wirtschaftlichen Bedingungen der Energieversorgung. Eine Ausnahme mit steigender Bedeutung bilden dabei jedoch die Sperren für hochgelegene Pumpspeicher, die Spitzenstrom zu kurz- und langfristiger Deckung liefern. Sie besitzen noch eine reiche Entwicklung und große Zukunft neben dem grundlastfahrenden Kernkraftwerk, auch im Hinblick auf die Verfügbarkeit dieser Einheiten großer Leistung. Die Beachtung der Probleme dieser Bauwerke, technisch und wirtschaftlich, ist von besonderer Bedeutung und jeder Beitrag hiezu aktuell. Die Vielfalt der Fragestellungen läßt einen vollständigen Überblick nicht zu, der Beitrag Österreichs kann nur umrissen und in Beispielen, ohne annähernd vollständig zu sein, dargestellt werden.

Als ein solches sei gezeigt in BILD 1 die Sperre Schlegeis aus der Kraftwerksgruppe Zemm der Tauernkraftwerke in Tirol. Sie ist eine der weitestgespannten Gewölbesperren der Welt, erbaut 1968/71

Kronenlänge $L = 725$ m
Mauerhöhe $H = 130$ m
$L/H = 5{,}6.$

Beschrieben wird sie von Widmann, Schlosser, Stäuble in [1]. Weiter wird über die Anlagen der Zemm-Gruppe von Widmann in [2] berichtet.

Über den Bau der Staumauer Kops der Vorarlberger Illwerke aus den Jahren 1961/65,

Kronenlänge $L = 400$ m
Mauerhöhe $H = 122$ m
$L/H = 3{,}2,$

berichtet Stocker in [3].

Im Bau befindet sich die Sperre Kölnbrein der Kraftwerksgruppe Malta der Österreichischen Draukraftwerke

Kronenlänge L = 62o m
Höhe H = 195 m
L/H = 3,2.

Wie kaum ein anderes Bauwerk ist eine Sperre so in die Natur und Umgebung eingebunden, daß sie, soll sie gelungen sein, ein Teil von ihr werden muß. Die grundlegende Art der Konstruktion, geschütteter Damm, Gewichtsmauer, Bogengewichtsmauer oder Gewölbesperre greift entscheidend in die Gegebenheiten der Natur ein und wird wesentlich durch sie mitbestimmt, beginnend mit dem örtlich zur Verfügung stehenden Material bis zur Beanspruchung und Festigkeit von Gestein und Gebirge und zu vorhandenen geologisch bedingten Spannungszuständen des Felsens reichend. An dieser Stelle tritt das Grundthema unseres Seminars zutage und zeigt sich in seiner Bedeutung.

Die Einleitung der Fundamentkräfte, ihre Größe und Richtung und die baugeologischen Verhältnisse werden von einer Reihe von Autoren behandelt wie S t i n i [4], K e t t n e r [5], W i d m a n n [6].

Die Formgebung der Sperre, Grundlage der statischen Berechnung und durch sie bestätigt, ist von Einfluß auf die Sicherheitsüberlegungen einerseits und die Wirtschaftlichkeit des Gesamtbauwerkes andererseits. Bei Eintreten von Rissen oder Öffnen von Fugen auf der Zugseite ändern sich örtlich die Steifigkeiten des Tragwerkes. Die Lastaufnahme durch Bogen und Kragträger, um nach diesem Rechenverfahren zu beurteilen, paßt sich den geänderten Steifigkeitsverhältnissen an. Zur Beurteilung von Form und Stärke einer solchen Sperrmauer können dienen die Arbeiten von K e t t n e r [7], W i d m a n n [8] und [9].

Durch geeignete Formgebung konnte bei praktisch gleichbleibendem Betonvolumen die Höhe der Sperre Schlegeis von 1oo m auf 13o m vergrößert und damit das Speichervolumen von 72

auf 127 Millionen Kubikmeter erweitert werden, vgl. hiezu Widmann, Schlosser, Stäuble [1].

Für die statische Berechnung und Ermittlung der Spannungszustände sind verschiedene Verfahren üblich. Angeführt sei als ältestes das Trägerrostverfahren, das je nach der Art der Ermittlung der Lastaufteilung als Trial-Load-Method bei versuchsweiser Lösung oder Lastaufteilungsverfahren, bei Lösung mit einem linearen Gleichungssystem, bezeichnet wird. Einer der ersten, die die Lastaufteilung auch bei einer mehrschnittigen Berechnung mit einem linearen Gleichungssystem durchgeführt haben, war 1949 Jurecka [10].

An der Verfeinerung dieses Verfahrens wird ständig gearbeitet. Verwiesen sei auf den prsönlichen Beitrag von Kettner zu diesem Seminar. Die Anwendung Lagrangescher Polynome für Übertragungsgrößen, aber auch für äußere Lasten und Querschnittwerte bringt wesentliche Vorzüge in der Durchführung der Berechnung.

Angeführt seien weiter Verfahren nach der Schalentheorie und schließlich die Anwendung der finiten Elemente. Aus der umfangreichen Literatur wird verwiesen auf Jurecka [10], Kettner [11], Tremmel [12], Rescher [18], Kettner [7], Widmann [8], Wulz [14], Kettner, Hillbrand [15], Herzog [16] und [17].

Die errechneten Beanspruchungen im Sperrenbeton bestimmen die zu wählende Festigkeit. Hierüber, über die Festigkeit unter lang andauernder Belastung, wechselnder Belastung sowie über kurz dauernde Lasteinwirkung berichtet Widmann in seinem Beitrag zu diesem Seminar, insbesondere aber auch über die Beurteilung der Festigkeit des Betons in mehrachsigen Beanspruchungszuständen, speziell bei zweiachsigen, fußend auf der Gestaltänderungshypothese, die variiert bzw. erweitert wird. Überlegungen,

welche Festigkeitswerte als Kennfestigkeiten festzulegen und für die Beurteilung maßgebend sind, werden dort angestellt in Berücksichtigung der mindestens zweiachsigen Beanspruchung in Sperrenbeton.

Ausführlich berichtet W i d m a n n in seinem Beitrag zu diesem Seminar über die Frage der Tragreserve von Sperren und die zu wählenden Sicherheiten. Er vergleicht dabei Sperren verschiedener Bauart, Damm, Gewichtsmauer und Gewölbesperre miteinander, geht dabei auf die Definition des Ausdruckes "Sicherheit" ein und kommt zu Schlüssen, welche eine einigermaßen einheitliche Beurteilung verschiedener Sperrentypen ermöglichen.

Die Berechnungsergebnisse werden üblicherweise mittels Modellversuchen überprüft, wie z.B. für die Hierzmann-Sperre an der Technischen Hochschule Graz, T s c h e c h, J a b u r e c k [18], für die Drossen-Sperre an der Versuchsanstalt der Tauernkraftwerke AG, die auch die statischen Modellversuche für die Sperren Kops, Kölnbrein und Schlegeis anstellte, vgl. hiezu M ü h l f e l l n e r [19].

Der Sicherheit dienen während des Baues und nach Fertigstellung die Beobachtungen und Messungen von Spannungen und Verschiebungen in den Sperren und der Umgebung und im Felsen. Über die hiezu verwendeten Methoden und Ergebnisse bzw. deren Auswertung berichten R e i t z [20], R e i t z, K r e m s e r, P r o k o p [21], P e t z n y [22], T r e m m e l [23], W i d m a n n [6], G a n s e r [24], B u c h e g g e r, N i e d e r l, P e t z n y, W i d m a n n [25], P e t z n y, W i d m a n n [26].

Der Bedeutung der Talsperren Rechnung tragend sind jährlich Berichte einem Unterausschuß der Österreichischen Staubeckenkommission vorzulegen, die dann alle 5 Jahre die Talsperre an Ort und Stelle überprüft.

3. Hohlraumauskleidungen

Auch dieses Teilgebiet kann hier nicht ausführlich behandelt werden und soll durch Hinweise auf das Schrifttum und die österreichischen Entwicklungen in seiner Bedeutung gekennzeichnet bleiben.

Hohlraumbauten können verschiedenen Zwecken dienen, als Verkehrswege, zur Führung der Druckleitungen von Wasserkraftanlagen, zur Aufnahme von Maschinensätzen bei solchen. In noch höherem Maß als beim vor beschriebenen Abschnitt ist das Gebirge mit allen seinen Eigenschaften maßgebend für die Arbeit des Ingenieurs.

Bei der Herstellung eines solchen Hohlraumes wird der primär vorhandene Spannungszustand im Gebirge, der dreiachsig ist, gestört. In der Laibung des Ausbruches bleibt ein zweiachsiger Zustand. Die Bruchsicherheit des Materials wird entscheidend verringert. Sie steigt aber mit verhältnismäßig geringer wiederaufgebrachter dritter Hauptnormalspannung oder Schaffung einer Zugfestigkeit normal zur Laibung rasch wieder an. Dies geschieht durch Aufbringen relativ nachgiebiger Spritzbetonschichten möglichst rasch nach dem Ausbruch, oder durch Anker, oder auch durch Kombination beider Verfahren. Nach Umlagerung der Spannungszustände im Gebirge können dann im Schutze dieses Außengewölbes Isolierung und Innengewölbe bzw.Innenverkleidung aufgebracht werden. Über diese "Neue österreichische Tunnelbauweise" wird in der Literatur berichtet von Rabcewicz, Sattler [27], Sattler [28], Gobiet [29], Rabcewicz [30], ferner sei verwiesen auf Kastner [31]. Außen- und Innengewölbe selbst sind mehrachsig beanspruchte Betonkonstruktionen.

Hinsichtlich der Bedeutung der Messungen der Verformung des Gebirges und des Außengewölbes zur Beurteilung des Zustandes der Stabilisierung oder der Sicherheit wird auf die gleiche Literatur verwiesen, in der auch die Modellversuche zur Entwicklung bzw. Bestätigung dieser Bauweise beschrieben sind.

Betreffend die Verwendung kopfbolzenverdübelter Verbundkonstruktionen u.a. für schwere Tunnel- und Stollenauskleidungen wird verwiesen auf S a t t l e r [32].

Für die Bemessung der Panzerung von Druckschächten von Wasserkraftanlagen hat sich zur Bestimmung der Felseigenschaften der Radialpressenversuch bewährt, zu dessen Weiterentwicklung von L a u f f e r - S e e b e r wesentliche Beiträge geleistet wurden, vgl. S e e b e r [33] und L a u f f e r - S e e b e r [34].

4. Spannbetondruckgefäße

Dem explosionsartigen Anwachsen des Energiebedarfes in der ganzen Welt kann nurmehr durch den Bau von Kernkraftwerken entsprochen werden, sowohl aus Gründen der zunehmenden Knappheit an fossilen Brennstoffen als auch wegen der mit deren Verwendung verbundenen Umwelteinflüsse.

Die verständlicherweise hohen Ansprüche an die Sicherheit nuklearer Kraftanlagen treffen im bautechnischen u.a. auf den Druckbehälter und den Sicherheitseinschluß. Die zunehmende Größe der Anlagen, räumlich und in der Leistung, brachte neben dem Stahldruckgefäß den Spannbetondruckbehälter, der eine Reihe von Vorzügen in der Herstellung und in der Sicherheit bietet. Er macht u.a. den Sicherheitseinschluß nicht mehr erforderlich. Die Entwicklung hat bei gasgekühlten Reaktoren begonnen, wo der Spannbetonbehälter zur beherrschenden Art der Ausführung geworden ist. In Österreich wurde von der Industrieforschung die Entwicklung auf diesem Gebiet aufmerksam verfolgt und studiert. Dies findet seinen Niederschlag in Beiträgen von N é m e t [35] und [36], K o m o l i [37] und [38].

Frühzeitig reifte der Entschluß, solche Spannbetondruckbehälter so auszulegen, daß sie nicht nur für gasgekühlte

Kernreaktoren, sondern auch für wassergekühlte zur Anwendung kommen können. Dazu erscheinen einige grundlegende Änderungen in der konstruktiven Ausbildung gegenüber den Behältern für gasgekühlte Reaktoren zweckmäßig oder sind notwendig, bzw. es sind Prinzipien einzuhalten, welche sich in primären Forderungen niederschlagen, die wieder sekundäre zwingend nach sich ziehen.

Die erste der primären Forderungen ist die Sicherung der Betriebszyklen durch entsprechende Wahl der Werkstoffe und zugehörigen Beanspruchungen.

Die zweite primäre Forderung ist, Aktivsysteme wie etwa Druckhalter usw. im Primärkreislauf für den Bestand des Gefäßes selbst nicht einzusetzen.

Die dritte primäre Forderung ist, im Primärkreislauf liegende und vom Kühlmittel durchdrungene Isolierungen zu vermeiden. Dies ist insbesondere bei wassergekühlten Reaktoren fast zwingend.

Es folgt aus den Überlegungen ein Behälterprinzip mit heißer Dichthaut in der speziellen österreichischen Fassung. Über ein solches Druckgefäß berichtet der Verfasser in seinem persönlichen Beitrag.

Allgemein sei darüber hinausgehend noch angeführt:
Die heiße Dichthaut selbst ist, da nicht hinter einer Isolierung verborgen, für Inspektion und Reparatur zugänglich, ein aus den obigen Forderungen sich ergebender Vorzug. Sie ist andererseits durch die direkte Beaufschlagung mit dem Kühlmittel härter beansprucht als eine sogenannte kalte Dichthaut, wie sie bei anderen Druckgefäßen besteht. Die Wahl des für sie notwendigen Stahles muß sorgfältig Rücksicht auf alle damit verbundenen Umstände nehmen. Der Werkstoff für sie - eine Entwicklung von VÖEST-ALPINE - ist ein martensititscher Chrom-Nickel-Stahl hoher Festigkeit und geringer Korrosion. Maßnahmen im Spannbetonteil bei Errichtung und Betrieb müssen die Dichthaut entlasten. So ist sie

trotz der hohen Temperatur nur unter ihrer Streckgrenze von 6 Mp/cm2 elastisch beansprucht. Die Zahl der zulässigen Lastzyklen in der Ermüdungsuntersuchung ist dementsprechend ausreichend hoch. Die Temperatur des Spannbetonteiles ist, um dies zu erreichen, auf 1oo bis 12o° C angehoben, um so die Zwängspannungen zu verringern. Im Isolierbeton hinter der Dichthaut steigt die Temperatur dann auf die des Kühlmittels - 3oo° C - an. Die Wärmedehnzahlen von Dichthautstahl und Spannbeton sind mit etwa 12 x $1o^{-6}$ praktisch gleich.

Die hier angedeuteten extremen Last- und Temperaturzustände stellen erhöhte Anforderungen an Werkstoffkenntnis, Berechnung und Ausführung, sowie an die folgende Überwachung durch Meß- und Beobachtungseinrichtungen.

Über Werkstoffgesetze wird im persönlichen Beitrag des Verfassers, betreffend die Relaxation des Stahles der Spannbewehrung bei erhöhter Temperatur und über den Kriechvorgang des Kiesbetons unter der angegebenen Temperatur von 12o° C berichtet. Die Probekörper waren vollständig versiegelt, so daß kein Wasser entweichen konnte. Dies ist im überwiegenden Bereich einer dicken Behälterwand der Fall. Ergänzend hiezu ist in BILD 2 der Kriechverlauf von Betonproben unter veränderlichen Belastungen, welche etwa den Spannungen bei Vorkriechen und Betrieb entsprechen, gezeigt. Die erwartete Stabilisierung scheint dadurch bestätigt zu sein. Des weiteren wurde besonderes Augenmerk auf den Einfluß von hoher Temperatur und deren zeitlichen Veränderungen verwendet. Die Entwicklung von Isolierbeton umfaßte mehrere umfangreiche Versuchsreihen und führte zu einem auch außerhalb des Gebietes der Druckbehälter verwendbaren höchstwertigen Leichtbeton aus Leca mit einer Zylinderfestigkeit bis etwa 7oo kp/cm2 bei einem Raumgewicht von etwa 17oo kp/m3. Ebenso gingen der Festlegung der Zusammensetzung des Kiesbetons Versuche mit verschiedenen Zementen und Zuschlagmaterialien voraus. Sie führten zu einem

BILD 1 Tauernkraftwerke A.G.
Schlegeis Sperre
Schlegeis Dam
Barrage de Schlegeis

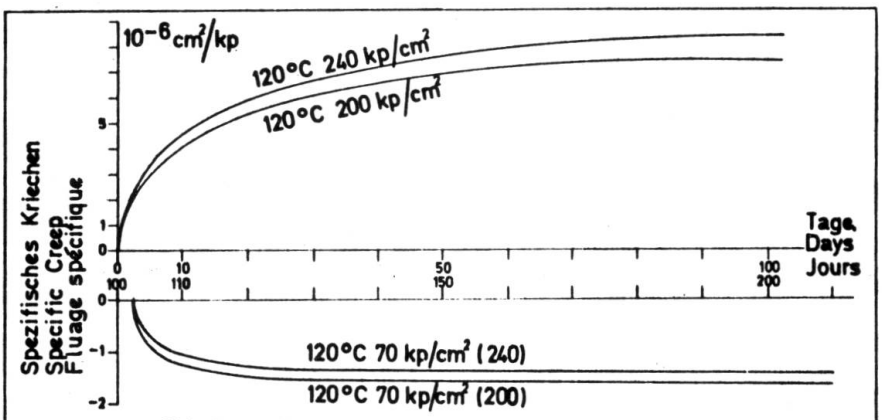

BILD 2 Kriechen mit Teilentlastung
Creep with Partial Relief
Fluage avec décharge partiel

Reaktorbau Forschungs- und Bauges. RFB, Techn. Vers. u. Forschungsanstalt Techn. Hochsch. Graz

Beton aus dolomitischem Zuschlagmaterial hoher Festigkeit, hoher Wärmedehnzahl und relativ geringen Elastizitätsmoduls. Der Einfluß der Temperaturzyklen auf die beiden Werkstoffe wurde untersucht und ist in den BILDERN 3 und 4 dargestellt.

Über den Stand der Entwicklung dieser Arbeiten ist in der Literatur an zuständigen Stellen berichtet worden. Es wird verwiesen auf N é m e t [39] und [40].

Die statische Berechnung solcher Gefäße erfordert Eingehen auf die zeitliche Veränderung von Verformung oder Spannung durch Kriechen und Relaxation, auf die Anisotropie des Materials, etwa gegeben durch Bewehrung. Zur Anwendung gelangten im Verlauf der Bearbeitung verschiedene Verfahren, baustatische für den regulären Mittelteil, Differenzenmethode und finite Elemente für die Endbereiche. Es wird hingewiesen auf N a t h s c h l ä g e r [41] und H o f e r [42].

Die Arbeit an diesem Behälter war Anlaß zur Entwicklung von Rechenverfahren, welche diesen Notwendigkeiten gerecht werden. In persönlichem Beitrag zu diesem Seminar durch W a l l u s c h e k - W a l l f e l d wird darüber berichtet.

Die Bruchsicherheit oder die Tragreserve eines solchen Behälters wird beurteilt aus dem Verhalten des Behälters bei einer fiktiven Steigerung des Innendruckes und dem Verhältnis der hypothetischen Bruchlast zur Gebrauchslast, welches als kennzeichnende Größe genommen wird. Diese fiktive Bruchlast liegt bei Werten, welche im Betrieb auch in Unfallsituationen bei weitem nicht erreicht werden können. Sie entspricht damit der Annahme eines Wassers mit wesentlich höherem spezifischen Gewicht bei der Beurteilung von Sperren. Es wird in einem wie im anderen Fall vorausgesetzt, daß implizit auch andere Sicherheitsüberlegungen dadurch gedeckt sind. So wird etwa ein Versagen oder teil-

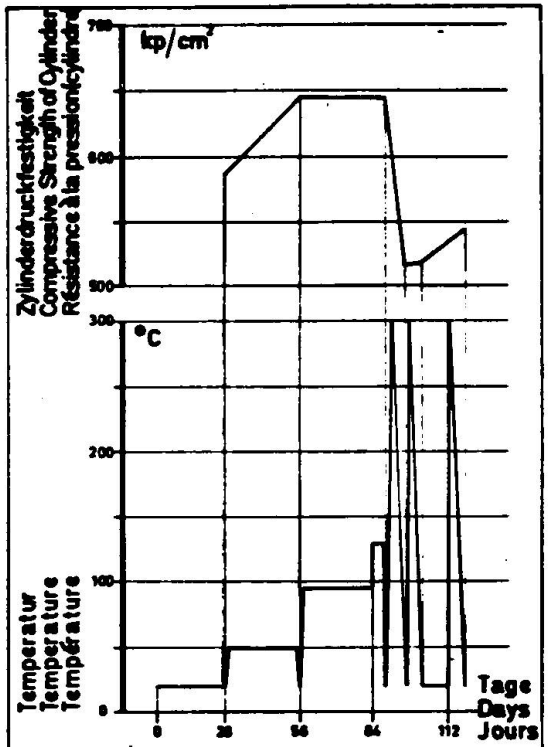

BILD 3 Normalbeton , Temeraturzyklen
Normal Concrete, Cycles of Temperature
Béton normal , cycles de température

Reaktorbau Forschungs-und Baugesellschaft RFB

BILD 4 Isolierbeton Temperaturzyklen
Insulating Concrete Cycles of Temperature
Béton isolant cycles de témperature

Reaktorbau Forschungs-und Baugesellschaft RFB

weises Versagen der Spannglieder oder von Gruppen von Spanngliedern die Sicherheit bzw. die Tragreserve des Gefäßes reduzieren, woraus sich ebenfalls ein Sicherheitsmaß ermitteln ließe, das keinesfalls gleich sein muß dem aus Laststeigerung bestimmten. Der Bruchvorgang bei hypothetischer Laststeigerung, das Entstehen und die Fortpflanzung der Risse, die Reihenfolge des Versagens der einzelnen Tragglieder wurde beim beschriebenen Druckbehälter eingehend überlegt. Die im Beitrag von Walluschek-Wallfeld gebrachten Rechenverfahren geben auch dazu die erforderlichen Handhaben. Es wird ferner verwiesen auf Nesitka [43] und Nesitka und Walluschek-Wallfeld [44].

Über die Bedeutung mehrachsiger Beanspruchungen im Beton bei der Beurteilung der Sicherheit des Druckbehälters berichtet der Verfasser in seinem persönlichen Beitrag zu diesem Seminar. Die Beanspruchungen sind dort für Betriebs- und Ausnahmezustände überprüft. Bei der Beurteilung einer hypothetischen Laststeigerung sind sie zu betrachten, wenn die Bruchgeometrie besonders in der Nähe der Endscheibe des Behälters untersucht wird, in den dort entstehenden Bruchgelenken, wobei wesentlich die plastische Verformbarkeit von Bedeutung wird.

Hinsichtlich der Messungen und Beobachtungen zur Beurteilung der laufenden Verformung, der Spannungen und Temperaturen kann verwiesen werden auf den persönlichen Beitrag des Verfassers und auf Németh, Zemann [45].

Sie sind nicht nur für den Versuchsbehälter zur Erkundung des Materialverhaltens, etwa des Kriechens unter mehrachsigen Spannungszuständen, verglichen mit einachsigen, wesentlich, sondern auch in hohem Maße für die Beobachtung während des Betriebes zur Überprüfung des Verhaltens unter Gebrauchslast. Die Bedeutung entspricht weitgehend der, welche Beobachtungen und Messungen bei Sperrenbauten zukommt.

Der Bericht über dieses Gebiet soll nicht abgeschlossen werden ohne zu erwähnen, daß in Österreich - über den Bau dieses Behälters zur Aufnahme von Versuchseinrichtungen hinausgehend - Entwürfe für Spannbetondruckbehälter zu Druckwasserreaktoren von 1500 MWe und Überlegungen bis 3000 MWe derzeit im Gang sind, über welche im einzelnen hier jedoch noch nicht berichtet werden kann.

Zusammenfassung

Der Österreichische Gesamtbericht bezieht sich auf drei Gebiete, Sperren, Hohlraumbauten, Spannbetondruckbehälter. Er nimmt Bezug auf Fragen der Berechnung, der Baustoffe, der Sicherheit und der Meßtechnik. An Hand der persönlichen Beiträge zu diesem Seminar und darüber hinaus des Schrifttums wird ein Überblick über die Entwicklung und den Stand der Technik in Österreich gegeben.

Summary

The Austrian national report deals with three matters, concrete dams, tunnels and prestressed concrete pressure vessels. It refers on questions concerning calculation, materials, security and measuring techniques. On the basis of the personal reports to this seminar and beyond that the literature, a general view is given of the development and stand of technique in Austria.

Résumé

Le rapport national autrichien traite trois matières, des barrages, des tunnels et des caissons en béton précontraints. Il fait rapport de la calculation, des materiaux, de la sécurité et des techniques de surveillance. Fondé sur les rapports personels et la litérature, une vue générale du developpement et de l'état de la technique en Autriche est présentée.

Schrifttum

[1] Widmann R., Schlosser J., Stäuble H.: Die Bogengewichtsmauer Schlegeis. Ö.Z.E. Jhg. 25, H. 1o

[2] Widmann R.: The Dams of the Zemm Hydro-Electric-Scheme. World Dams Today, 197o

[3] Stocker E.: Bericht über die Bauausführung der Staumauer Kops. Ö.Z.E. Jhg. 23, H. 7

[4] Stini J.: Die baugeologischen Verhältnisse österreichischer Talsperren. Die Talsperren Österreichs, Heft 5. Österreichischer Wasserwirtschaftsverband 1955

[5] Kettner R.: Die Fundamentkräfte der Gewölbesperren. Felsmechanik und Ingenieurgeologie, Suppl. III 1967

[6] Widmann R.: Bogengewichtsmauer Schlegeis. Das Verhalten des Felsuntergrundes während der ersten beiden Teilstauperioden. Rock Mechanics, Suppl. 2. 1973

[7] Kettner R.: Zur Formgebung und Berechnung der Bogenlamellen von Gewölbemauern. Die Talsperren Österreichs, Heft 8, Österr. Wasserwirtschaftsverband 1959

[8] Widmann R.: Zur Berechnung und wirtschaftlichen Formgebung von Bogengewichtsmauern. Österr.Ingenieur Zeitschrift 1961, Heft 8

[9] Widmann R.: Der Parabelbogen im Talsperrenbau. Der Bauingenieur 1961, Heft 6

[1o] Jurecka W.: Die Berechnung bogenförmiger Staumauern nach dem Lastaufteilungsverfahren. Österr.Bauzeitschrift 1949, H. 11 u. 12

[11] Kettner R.: Berechnungsverfahren von Gewölbemauern. Österr.Wasserwirtschaft 1958, H. 8/9

[12] Tremmel E.: Beitrag zur Gewölbemauerberechnung. Österr.Wasserwirtschaft 1951, H. 5/6

[13] Rescher O.: Die Gewölbemauer. Diss. TH Wien, Abhandlungen des Dokumentationszentrums der Technik, H. 1, Wien 1951

[14] Wulz H.: Beitrag zur Vorberechnung von Bogenstaumauern auf der Grundlage des Lastaufteilungsverfahrens. Diss. TH Graz 1969

[15] Kettner R., Hillbrand G.: Elektronische Berechnung von Gewölbesperren. IV. Int. Kongreß über Anwendung der Mathematik in den Ingenieurwissenschaften. VEB-Verlag für Bauwesen, Berlin 1968

[16] H e r z o g M.: Berechnung beliebig geformter Gewölbemauern nach der Schalentheorie. Die Bautechnik 1956

[17] H e r z o g M.: Beitrag zur Berechnung doppelt gekrümmter Staumauern. Die Bautechnik 1957

[18] T s c h e c h E., J a b u r e c k F.: Berechnung von Bogenstaumauern im Vergleich mit den Ergebnissen statischer Modellversuche. Österr.Bauzeitschrift 1951, H. 2

[19] M ü h l f e l l n e r R.: Statische Untersuchungen an einem Modell der Drossensperre. Festschrift Oberstufe Kaprun 1955

[20] R e i t z A.: Beobachtungseinrichtungen an den Talsperren Salza, Hierzmann, Ranna und Wiederschwing. Die Talsperren Österreichs, Heft 1, Österr.Wasserwirtschaftsverband 1954

[21] R e i t z A., K r e m s e r R., P r o k o p E.: Beobachtungen an der Rannatalsperre 1950-1952 mit besonderer Berücksichtigung der technischen Erfordernisse. Die Talsperren Österreichs, Heft 3, Österr.Wasserwirtschaftsverband 1954

[22] P e t z n y H.: Meßeinrichtungen und Messungen an der Gewölbesperre Dobra. Die Talsperren Österreichs, Heft 6, Österr.Wasserwirtschaftsverband 1957

[23] T r e m m e l E.: Limbergsperre, statische Auswertung der Pendelmessungen. Die Talsperren Österreichs, Heft 7, Österr. Wasserwirtschaftsverband 1958

[24] G a n s e r O.: Die Meßeinrichtungen der Staumauer Kops. Die Talsperren Österreichs, H. 16, Österr.Wasserwirtschaftsverband Wien 1968

[25] B u c h e g g e r W., N i e d e r l H., P e t z n y H., W i d m a n n R.: Neuere Beobachtungen. Die Talsperren Österreichs, H. 14, Österr.Wasserwirtschaftsverband Wien 1964

[26] P e t z n y H., W i d m a n n R.: Meßeinrichtungen und Methoden der Auswertung bei Österreichischen Talsperren. 10. Talsperrenkongreß Montreal 1970, Österreichischer Generalbericht

[27] R a b c e w i c z L.v., S a t t l e r K.: Die neue Österreichische Tunnelbauweise, I Entstehung, Entwicklung und Erfahrungen, II Statische Wirkungsweise und Bemessung. Der Bauingenieur 1965, H. 8

[28] S a t t l e r K.: Neuartige Tunnelmodellversuche - Ergebnisse und Folgerungen, Felsmechanik und Ingenieurgeologie. Suppl. IV 111-137, 1968

[29] G o b i e t W.: Beitrag zur Berechnung von Tunnelauskleidungen. Diss. TH Graz 1970

[30] R a b c e w i c z L.v.: Stability of Tunnels. Waterpower 1968/6,7,8

[31] K a s t n e r H.: Statik des Tunnel- und Stollenbaues. Berlin 1962

[32] S a t t l e r K.: Untersuchungen über Verbundträger für den Tunnelbau und Hochbauten. Der Bauingenieur 1964, H. 1

[33] S e e b e r G.: 1o Jahre Einsatz der Tiwag-Radialpresse. 1. Kongreß der ISRM Lissabon

[34] L a u f f e r H., S e e b e r G.: Die Tiwag-Radialpresse. 1. Kongreß der ISRM Lissabon

[35] N é m e t J.: Reaktordruckbehälter aus Spannbeton (Beitrag zur Sicherheit und Wirtschaftlichkeit von Kernkraftwerken). Das Atomkraftwerk 1967, H. 1o

[36] N é m e t J.: Development Work in Constructing a Nuclear Reactor. Build International Nov/Dez/1972

[37] K o m o l i L.H.: Test stand for the Prestressed Concrete Vessel Containing the Helium Loop. Panel on Basic Structural Design Pilosophy, Criteria and Safety of Concrete Reactor Pressure Vessels, International Atomic Energy Agency, Wien 197o

[38] K o m o l i L.H.: Sonderprobleme beim Bau von Kernkraftwerken. Informationstagung des Österreichischen Atomforums, Wien 1972

[39] N é m e t J.: Reaktordruckbehälter aus Spannbeton mit heißer Dichthaut. 1st International Conference on Structural Mechanics in Reactor Technology, Berlin 1971

[4o] N é m e t J.: Statusbericht über das Österreichische Gemeinschaftsprojekt, Spannbetonbehälter mit heißer Dichthaut. 2nd International Conference on Structural Mechanics in Reactor Technology, Berlin 1973

[41] N a t h s c h l ä g e r P.: Technische Berichte 147, 151, 162, 179, 191 der Österreichischen Studiengesellschaft für Atomenergie, Seibersdorf, Österreich

[42] H o f e r G.: EDV-Dokumentation der Vereinigten Österreichischen Eisen- und Stahlwerke, Linz, Österreich

[43] N e s i t k a A.: Berechnung und Sicherheit eines Spannbetondruckbehälters. Zweiter Zwischenbericht zum Gemeinschaftsprojekt Spannbetonbehälter - Heliumkreislauf, Österr.Studiengesellschaft für Atomenergie 1971

[44] N e s i t k a A., W a l l u s c h e k - W a l l f e l d W.: Statische Berechnung, Sicherheitsüberlegungen und Untersuchung des Bruchvorganges von Reaktordruckbehältern aus Spannbeton. 2nd International Conference on Strucktural Mechanics in Reactor Technology, Berlin 1973

[45] N é m e t J., Z e m a n n H.: Untersuchung über elektrische Dehnungsaufnehmer für das Innere des Betons bis 12o° C bzw. 3oo° C. 2nd International Conference on Structural Mechanics in Reactor Technology, Berlin 1973

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SEMINAR on:
« CONCRETE STRUCTURES SUBJECTED TO TRIAXIAL STRESSES »
17th - 19th MAY, 1974 - ISMES - BERGAMO (ITALY)

Recherches et réalisations belges dans le domaine des structures en béton soumises à des contraintes triaxiales

Belgian Researches and Constructions in the Domain of Concrete Structures Subjected to Triaxial Stresses

Forschungen und Realisierungen in Belgien auf dem Gebeit der Betonkonstruktionen unter triaxialen Einflüssen

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I. OUVRAGES REALISES

Notre enquête auprès des Bureaux d'Etudes et des Entreprises montre que peu d'ouvrages ont été calculés en tenant compte de l'état triaxial de contrainte.

Nous citerons :

- Les tunnels sous l'Escaut à Anvers et sous le Rupel à Boom. Ceux-ci sont réalisés par échouage de tronçons en béton armé partiellement précontraint. L'état triaxial de contrainte a été pris en considération dans le calcul des blocs d'about.
- La nouvelle dalle d'essais des structures du Laboratoire Magnel de l'Université de Gand à Zwijnaarde.
- L'enceinte primaire de sécurité du réacteur nucléaire de la Centrale de Tihange, réalisée en béton précontraint.
  Cette enceinte de 42 mètres de diamètre et 63 mètres de hauteur est mince (0,70 et 0,60 m d'épaisseur).
  Elle a été calculée comme une coque de révolution. Les contraintes ont donc été supposées biaxiales. Il n'a cependant pas été tenu compte de l'effet favorable de ce type d'étreinte sur la résistance du matériau, ce qui assure une sécurité supplémentaire.

II. ETUDES EXPERIMENTALES

Nous avons eu connaissance des quatre études expérimentales suivantes, toutes réalisées à l'Université de Liège.

II.1. LES PRESSIONS INTERSTITIELLES DANS LES OUVRAGES HYDRAULIQUES ET LEUR FONDATION [1, 2, 3]*

Notations

ϕ	: potentiel hydraulique.
x, y, z	: coordonnées.
k_{xx}, k_{yy}, k_{zz}	: coefficients de perméabilité principale.
θ	: coefficient de pression interstitielle.
P	: pression.
X, Y, Z	: forces volumiques.
E	: module d'élasticité longitudinale.
ν	: coefficient de Poisson.
ε_z, ε_c, ε_r	: déformations verticale, circonférencielle et radiale.

II.1.1. Existence

Indépendamment des pressions agissant sous la base d'un ouvrage hydraulique (sous-pression), l'existence des pressions dans le corps des ouvrages de retenue hydraulique est prouvée par les suintements d'eau qui traversent des épaisseurs considérables de matériaux pour sourdre à l'aval.
Le passage de l'eau à travers des pores du béton ne peut se concevoir que sous l'action d'un champ de pressions qu'il est convenu d'appeler interstitielles.

II.1.2. Schéma de l'étude du mode d'action des pressions interstitielles

L'examen complet de l'action des pressions interstitielles comporte trois questions :

- la première concerne la valeur de la pression dans les pores;
- la deuxième concerne le concept de l'état de contrainte dans un corps poreux pour lequel les pressions internes interviennent au titre de forces agissant à l'intérieur du massif étudié;
- la troisième question concerne la physique interne des corps poreux.

Les canaux d'infiltration n'occupent qu'une fraction aléatoire de la surface d'une section de résistance. Pour connaître le mode d'action effectif des pressions interstitielles, on est donc forcé de déterminer la valeur de la "porosité superficielle" (différence en pourcentage entre la projection de l'aire totale et la projection de l'aire de solide interceptée par la section considérée). On désigne parfois ce coefficient θ par le terme "aires effectives de pressions interstitielles". Ce dernier problème ne peut être résolu que par la voie expérimentale.

* Références bibliographiques in fine.

II.1.3. Détermination du champ de pressions interstitielles

La détermination du champ d'écoulement permanent et laminaire dans les milieux poreux anisotropes revient à résoudre (dans l'hypothèse où les axes des coordonnées coïncident avec les directions principales de perméabilité) l'équation suivante :

$$\frac{\partial}{\partial x}\left(k_{xx}\frac{\partial\phi}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_{yy}\frac{\partial\phi}{\partial y}\right) + \frac{\partial}{\partial z}\left(k_{zz}\frac{\partial\phi}{\partial z}\right) = 0$$

en tenant compte des conditions aux limites.
Il existe plusieurs méthodes (analytiques, analogiques et numériques) pour résoudre cette équation. Cependant le procédé le plus puissant est celui basé sur l'emploi de la méthode des éléments finis à condition évidemment de disposer d'un ordinateur.

II.1.4. Etats de contraintes créés par la pression interstitielle

A partir des équations d'équilibre et de compatibilité, on peut démontrer que les pressions interstitielles dans un milieu poreux homogène agissent à la manière de forces volumiques

$$X = -\frac{\partial\theta P}{\partial x} \qquad Y = -\frac{\partial\theta P}{\partial y} \qquad Z = -\frac{\partial\theta P}{\partial z}$$

à condition de ne faire agir à la surface soumise à la pression hydrostatique que $P\ (1-\theta)$.
De même on peut démontrer que dans le cas d'une structure constituée de deux parties distinctes (un barrage et sa fondation par exemple) ayant deux coefficients différents de pression interstitielle θ_b et θ_r, l'état des contraintes créé par les pressions interstitielles se calcule en considérant un effet concentré dans le champ de forces volumiques mentionné dans l'alinéa précédent. Cet effet concentré consiste en une ligne de forces agissant dans le joint séparant les deux matériaux différents et dont l'intensité est fournie par l'expression $(\theta_r-\theta_b)P$ où P est la pression dans le joint.

II.1.5. Essais pour la détermination de θ

La méthode la plus classique pour déterminer le coefficient θ consiste à mesurer l'allongement d'un cylindre de matière soumis sur sa périphérie cylindrique à une pression constante p au moyen d'une chambre. Le dispositif d'essai mis au point aux laboratoires de l'Université de Liège comprend deux systèmes réalisateurs et mainteneurs de charge de deux cellules étanches dans lesquelles deux éprouvettes peuvent être étudiées simultanément dans des conditions identiques (Figures 1 et 2). Le premier système sert à comprimer axialement les éprouvettes pour empêcher leur rupture par traction pendant l'essai, et le deuxième système réalise et maintient la pression d'eau dans les deux cellules. Au droit de la section médiane de chaque éprouvette, ont été collées quatre jauges ohmiques. Les deux jauges verticales fournissent la valeur de l'allongement vertical ε_z et les deux jauges horizontales fournissent la valeur de la déformation circonférencielle ε_c.
Ainsi, on peut obtenir :

- une valeur du coefficient de pression interstitielle calculé à partir de la mesure de ε_z, on le désigne ci-après par θ_z

$$\theta_z = \frac{1}{1-2\nu}\left(\frac{E.\varepsilon_z}{P} - 2\nu\right)$$

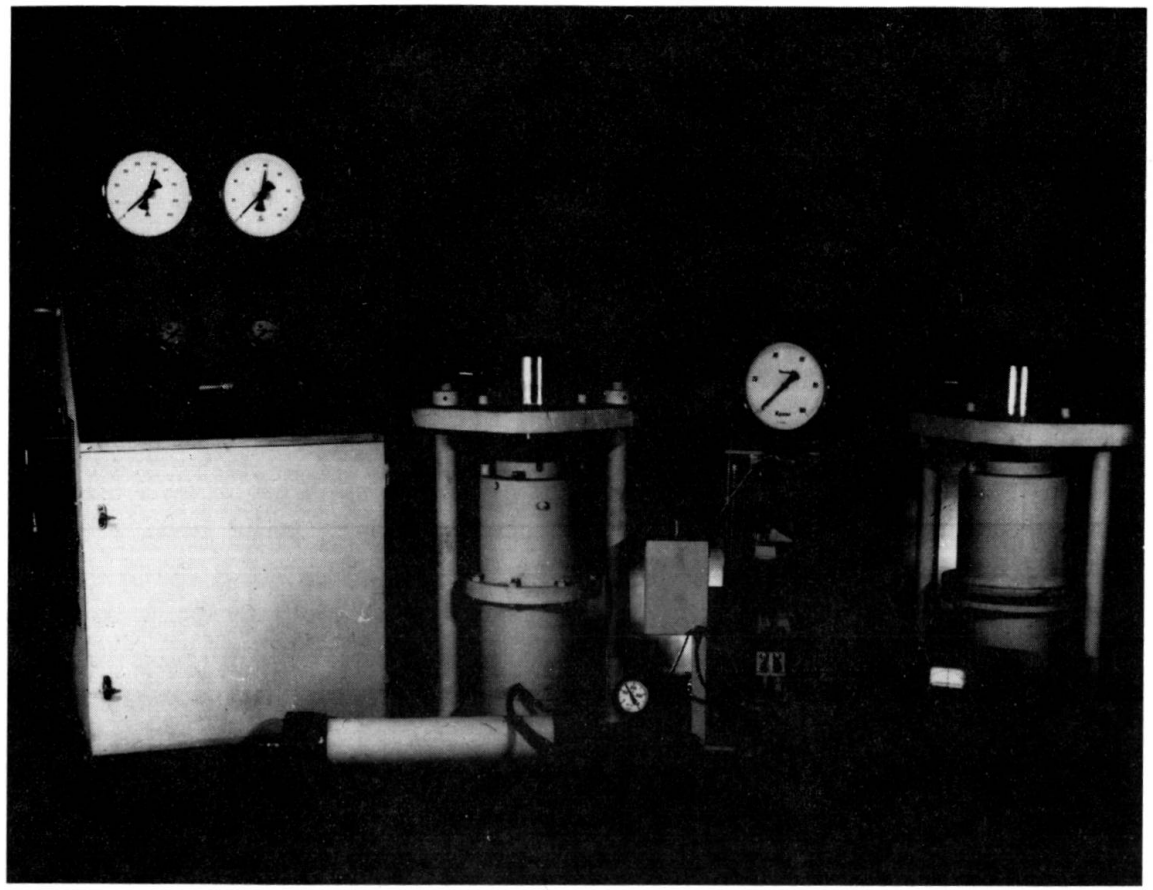

Figure 1

Vue générale du montage expérimental montrant les deux cellules surmontées des vérins réalisant la charge axiale, à gauche, la pompe électrique permettant la mise en pression des deux vérins et au centre, le dispositif de réalisation et de maintien de la pression d'eau dans les deux cellules.

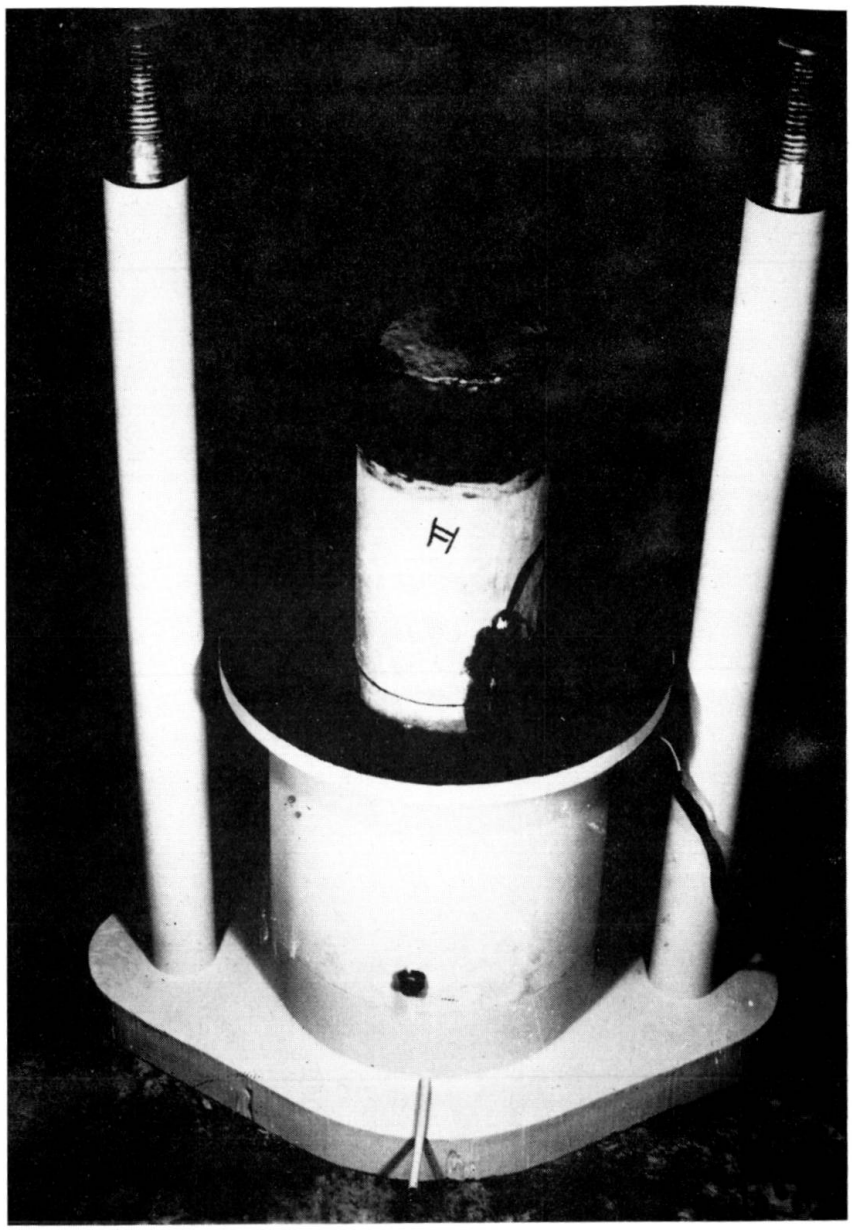

Figure 2

Vue de l'éprouvette équipée de ses couronnes de résine et des jauges ohmiques.

- une valeur θ_c de ce même coefficient par la mesure ε_c

$$\theta_c = \frac{1}{1-2\nu} \left(\frac{E.\varepsilon_c}{P} + 1 - \nu\right)$$

- une valeur θ_δ en tenant compte de l'invariant $\delta = \varepsilon_z + \varepsilon_c + \varepsilon_r$

$$\theta_\delta = \frac{1}{3} \left(\frac{E.\delta}{p\ (1-2\nu)} + 2\right)$$

Théoriquement ces trois valeurs doivent être égales. La pratique des mesures sur mortiers et bétons montre que θ_z est différent de θ_c pour des raisons diverses (homogénéité imparfaite, imprécision des mesures, imperfection de montage, comportement variable dans le temps).
Le tableau n° 1 donne la composition des différents types de mortier et de béton essayés. Le tableau n° 2 donne les résultats obtenus pour la valeur maximale de la pression P = 43,5 kg/cm^2.
On constate d'après ces résultats que les valeurs de θ_δ pour les mortiers et les bétons oscillent généralement entre 0,60 et 0,80. Ce coefficient tend vers l'unité pour des compositions contenant une quantité importante de ciment. Cette tendance a été confirmée par les résultats des essais effectués sur des éprouvettes en pâte de ciment qui ont donné des valeurs de θ comprises entre 0,827 et 1.
D'autre part, afin d'analyser l'évolution de la valeur de θ en fonction de la pression d'eau appliquée sur les éprouvettes, nous avons soumis une série d'éprouvettes de mortier de compositions différentes à plusieurs cycles de mise sous pression d'eau croissante. Ces expériences font apparaître une excellente reproductivité des résultats.
Les figures 3 à 5 représentent les résultats d'un essai. On constate que θ_z croît régulièrement avec la pression, tandis que θ_c et θ_δ restent sensiblement constants quelle que soit cette variable. Cette conclusion confirme l'intérêt qu'il y a à déterminer θ_δ plutôt que θ_z ou θ_c.

II.1.6. Essai pour montrer l'existence d'une ligne de force agissant dans le joint séparant deux matériaux différents

Considérons une éprouvette cylindrique constituée de deux matériaux différents et symétriques par rapport à la section médiane.
La partie centrale de l'éprouvette a un coefficient de pression interstitielle θ_2 et les deux parties supérieure et inférieure sont identiques et ont un coefficient θ_1. Si l'on adopte l'hypothèse de l'existence d'une ligne de force dans les joints et d'intensité égale à $(\theta_2-\theta_1)P$ on peut démontrer que les déformations de la section médiane de l'éprouvette ne dépend que des caractéristiques de la partie centrale, c'est-à-dire θ_2, E_2, ν_2.
Une série d'essais effectués sur de telles éprouvettes nous ont confirmé la validité de cette théorie.

Tableau n° 1

Genres	Type	Ciment HF (kg/m³)	Sable 0/2 (kg/m³)	Eau (kg/m³)	Agrégats 8/22 (kg/m³)	Poids spécifique (kg/m³)
Mortiers	A 1a	392	1566	272		2230
	A 2a	484	1452	294		2220
	A 2b	471	1412	304		2187
	A 2c	458	1374	323		2155
	A 3c	598	1195	345		2138
Béton de graviers roulés	B 1a	202	806	159	1267	2434
	B 1b *	199	797	160	1267	2423
	B 2a	249	748	166	1267	2430
	B 2b	242	726	176	1267	2411
	B 2c	236	707	186	1267	2396
	B 3b *	316	632	185	1267	2400
	B 3c	308	615	197	1267	2387
Bétons de pierrailles concassées	C 1a	219	876	178	1127	2400
	C 1b *	217	867	181	1127	2392
	C 2a	271	812	185	1127	2395
	C 2b	263	790	197	1127	2377
	C 2c	256	769	207	1127	2359
	C 3b *	346	692	202	1127	2367
	C 3c	334	668	219	1127	2348

Tableau n° 2

	Types	Age (jours)	A sec		Sous eau		θ_z	θ_c	θ_δ
			E kg/cm²	v	E kg/cm²	v			
Mortiers	A 1a 1	36	160.000	0,238	145.000	0,195	0,776	0,642	0,687
	2	36	198.000	0,273	184.000	0,228	0,608	0,463	0,525
	A 2a 1	35	148.000	0,211	137.000	0,222	0,827	0,742	0,771
	2	35	161.000	0,293	150.000	0,298	0,590	0,654	0,633
	A 2b 1	50	163.000	0,170	145.000	0,132	0,769	0,781	0,777
	2	50	162.000	0,188	140.000	0,154	0,713	0,813	0,777
	A 2c 1	35	104.000	0,190	89.000	0,182	0,943	0,714	0,790
	2	35	120.000	0,221	92.000	0,196	0,871	0,783	0,790
	A 3c 1	43	164.000	0,213	146.000	0,222	0,650	0,663	0,657
	2	43	102.000	0,135	90.000	0,132	1,044	0,909	0,955
Bétons graviers roulés	B 1a 1	32	175.000	0,227	153.000	0,256	0,645	0,451	0,515
	2	32	167.000	0,142	157.000	0,116	0,760	0,643	0,648
	B 1b 1	63	283.000	0,164	226.000	0,125	0,801	0,896	0,864
	2	63	261.000	0,207	218.000	0,156	0,600	0,901	0,800
	B 2a 1	35	221.000	0,226	182.000	0,220	0,753	0,608	0,657
	1	35	250.000	0,166	231.000	0,140	0,629	0,486	0,534
	2	35	209.000	0,178	172.000	0,227	0,653	0,511	0,559
	B 2b 1	43	228.000	0,189	188.000	0,104	0,845	0,673	0,729
	2	43	233.000	0,173	193.000	0,101	0,876	0,760	0,800
	B 3b 1	66	202.000	0,308	175.000	0,100	0,767	0,884	0,845
	2	66	210.000	0,135	155.000	0,219	0,942	0,721	0,794
	B 3c 1	35	237.000	0,216	204.000	0,163	0,727	0,866	0,820
	2	35	236.000	0,175	209.000	0,167	0,811	0,609	0,677
Bétons graviers concassés	C 1a 1	28	174.000	0,287	149.000	0,266	0,810	0,719	0,750
	2	28	162.000	0,228	146.000	0,237	0,854	0,551	0,652
	C 1b 2	68	160.000	0,256	131.000	0,329	0,371	0,707	0,595
	C 2a 1	35	157.000	0,125	124.000	0,119	0,855	0,797	0,816
	2	35	203.000	0,235	153.000	0,148	0,684	0,736	0,718
	C 2b 1	51	199.000	0,143	172.000	0,137	0,804	0,731	0,758
	C 2c 2	35	169.000	0,205	131.000	0,198	0,621	0,735	0,696
	C 3b 1	66	239.000	0,214	219.000	0,190	0,944	0,823	0,863
	2	66	262.000	0,235	237.000	0,186	0,678	0,732	0,714
	C 3c 1	35	157.000	0,116	141.000	0,101	0,722	0,773	0,758
	2	35	149.000	0,128	119.000	0,107	0,598	0,628	0,618

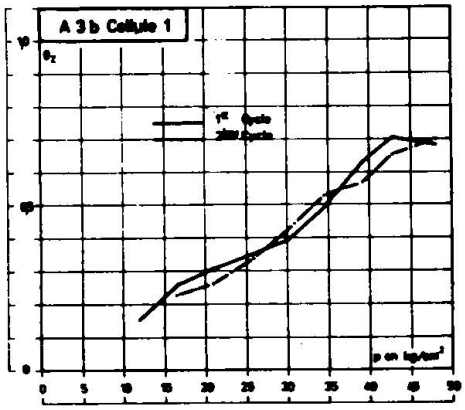

Figure 3

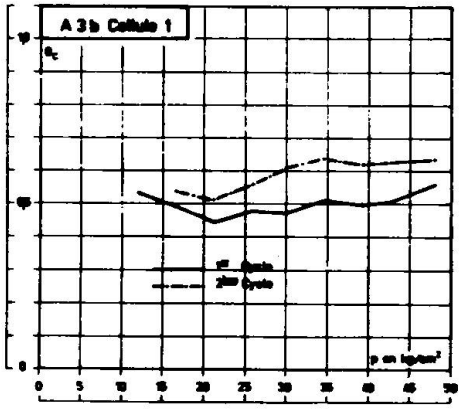

Figure 4

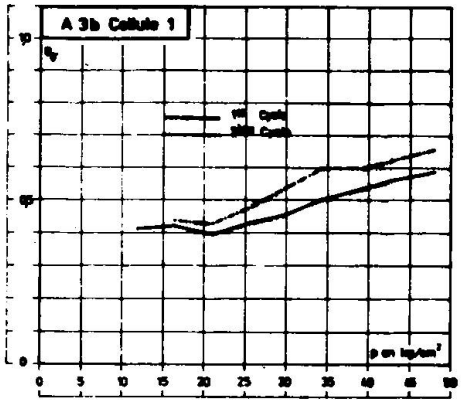

Figure 5

II.2. PRISE EN CONSIDERATION DE L'ETAT TRIAXIAL DE CONTRAINTE DANS LES PROFILS CREUX CIRCULAIRES REMPLIS DE BETON CHARGES AXIALEMENT [4 à 7]

En vue d'augmenter la charge critique d'une colonne tubulaire il y a souvent intérêt à la remplir de béton. De plus ce remplissage permet éventuellement d'accroître la résistance au feu.

Une méthode de calcul des colonnes mixtes acier-béton a été mise au point par le C.R.I.F.* et l'Université de Liège. Cette méthode est basée sur les recommandations les plus récentes du C.E.B. et de la C.E.C.M.

Dans le cas particulier des profils creux circulaires de faible longueur remplis de béton, il y a une interaction latérale entre l'acier et le béton. Ceci a conduit à la prise en compte de l'état triaxial de contrainte du béton. Pour les profils longs le flambement de la colonne se produit avant que la dilation latérale du béton devienne supérieure à celle de l'acier.

La charge critique des colonnes circulaires de faible élancement a été déterminée à partir des deux critères de rupture suivants.

- Béton

$$3J_2 = \left[R_C R_T + (R_C - R_T)\, J_1\right]\left\{1 - \frac{3R_\tau^2}{R_C R_T}\left[1 - \frac{J_3}{2}\left(\frac{J_2}{3}\right)^{-3/2}\right]\right\}$$

où R_C, R_T et R_τ sont les résistances à la compression sur cylindres à la traction et au cisaillement;

J_1, J_2 et J_3 sont les trois invariants du tenseur contrainte.

Dans le cas du profil creux circulaire rempli de béton, on a $\sigma_1 = \sigma_{b\ell}$, $\sigma_2 = \sigma_3 = \sigma_{bt}$, et le critère devient, en supposant $R_T = 0$:

$$(\sigma_{b\ell} - \sigma_{bt})^2 - R_C\,(\sigma_{b\ell} + 2\sigma_{bt}) = 0$$

où σ_1, σ_2 et σ_3 sont les contraintes principales;

$\sigma_{b\ell}$ et σ_{bt} sont les contraintes longitudinale et transversale.

- Acier

Critère de plastification de l'acier soumis à un état biaxial de contraintes (Voir Mises).

$$J_2 = \sigma_e^2 = \sigma_{a\ell}^2 + \sigma_{at}^2 - \sigma_{a\ell}\,\sigma_{at}$$

où σ_e est la limite d'élasticité;

$\sigma_{a\ell}$ et σ_{at} sont les contraintes longitudinale et circonférentielle.

* Centre de Recherches Scientifique et Technique de l'Industrie des Fabrications Métalliques.

La méthode de calcul a été vérifiée par des essais (15) effectués à l'Université de Liège et elle a été confrontée avec les résultats d'essais (22) décrits dans la littérature. Pour l'ensemble des 37 essais la valeur moyenne du rapport de la charge ultime expérimentale à la charge ultime calculée est 1,052 avec un écart type de 0,076 soit un coefficient de variation 7,3 %. Ces valeurs montrent que l'application de ces critères conduit à des résultats très satisfaisants dans le cas des charges ultimes des colonnes circulaires de faible longueur.

II.3. INFLUENCE DE L'ETAT DE CONTRAINTES SUR LES RESISTANCES EN TRACTION ET EN COMPRESSION D'UN BETON

II.3.1. Introduction

Les essais suivants ont été réalisés :

- fendage (brésilien);
- compression diamétrale d'éprouvettes annulaires (brésilien creux);
- compression d'éprouvettes cylindriques dans une cellule triaxiale;
- éclatement d'éprouvettes cylindriques creuses placées dans une cellule triaxiale;
- torsion.

Les résultats de ces essais, interprétés par la théorie de l'élasticité ont été comparés aux résultats des essais uniaxiaux.

II.3.2. Eprouvettes

La composition du béton était la suivante :

- 1.200 kgf de concassé de calcaire (calibre 2/8 mm);
- 590 kgf de sable de Mont St Guibert;
- 400 kgf de ciment portland P 300;
- 210 kgf d'eau.

Ce béton était coulé et vibré en dalles démoulées après 2 jours et conservées une quinzaine de jours sous eau à 20°C. Les éprouvettes étaient alors carottées et replacées sous eau. Elles étaient mises à l'étuve à 105°C trois jours avant leur essai à l'âge de 35 jours. Les dimensions des éprouvettes étaient :

- diamètre extérieur : D = 55 mm;
- diamètre intérieur des éprouvettes creuses : d = 20 mm;
- hauteur : h = 114 mm.

II.3.3. Description des essais et interprétation

- Essais de traction simple

Les éprouvettes étaient collées à l'aide de résine époxyde (Araldite 121 B) sur des bases en acier reliées par une rotule à des tiges fixées dans les mordaches d'une machine de traction.

- Essais de fendage (Brésilien)

Les éprouvettes étaient comprimées diamétralement entre deux plateaux. Le plateau supérieur était articulé pour répartir l'effort longitudinalement. L'initiation de la rupture a été observée à l'axe de l'éprouvette. D'après la théorie de l'élasticité, si P est la charge de rupture, la contrainte principale de traction en ce point vaut $\sigma_{tr}^{f} = \frac{-2P}{\pi Dh}$ et celle de compression $\sigma_c = -3\sigma_t^f$. La troisième contrainte principale est nulle.

- Essais de compression diamétrale d'éprouvettes annulaires (Brésilien creux)

La réalisation de ces essais est la même que celle de l'essai de fendage, excepté que l'éprouvette est annulaire.
La plus grande contrainte de traction a lieu au bord du trou dans le plan des charges, elle vaut d'après la théorie d'élasticité :

$$\sigma_{tr}^{a} = \frac{-2P}{\pi Dh}\left[6 + 38\left(\frac{d}{D}\right)^2\right] .$$

Les deux autres contraintes principales y sont nulles.

- Essais d'éclatement d'éprouvettes creuses placées dans une cellule triaxiale

Les essais ont été réalisés dans une cellule triaxiale classique de la manière suivante :

- application et maintien d'une pression latérale externe σ_e,
- application d'un effort axial P,
- mise en pression σ_i du trou central de l'éprouvette jusqu'à rupture.

La contrainte maximum de traction a lieu à la surface du trou, elle est circonférencielle et vaut :

$$\sigma_{tr}^{e} = -\frac{\sigma_i\,(d^2+D^2) - 2\sigma_e D^2}{\pi(D^2-d^2)} .$$

Les deux autres contraintes principales valent :

$$\sigma_\ell = \frac{4P}{(D^2-d^2)} \quad \text{et} \quad \sigma_r = \sigma_i .$$

- Essais de torsion

Les éprouvettes ont été collées sur les plateaux d'une machine de torsion classique, les plateaux étant en place dans la machine.
Les contraintes maxima de traction ont lieu à la surface de l'éprouvette, sur des facettes inclinées à 45° sur les génératrices, elles valent :

$$\sigma_{tr}^{t} = \frac{-16\,M_t}{\pi D^3}$$

M_t étant le moment de torsion appliqué. La contrainte principale de compression $\sigma_C = -\sigma_{tr}^{t}$, la troisième contrainte principale est nulle.

- Essais classiques de compression simple

- Essais de compression triaxiale

Ces essais ont été réalisés de la manière classique en mécanique des sols; maintien de la pression latérale σ_ℓ et augmentation de l'effort axial P jusqu'à rupture. Les pressions latérales ont atteint 1900 kgf/cm^2.

II.3.4. Résultats obtenus

Pour chaque ensemble d'essais, la valeur moyenne de la contrainte de rupture, l'écart type et le coefficient de variation ont été calculés ainsi que les valeurs moyennes des autres contraintes principales. Les résultats sont donnés au tableau ci-après où figure également le rapport de la contrainte de rupture relative à un type d'essais à la contrainte de rupture en traction ou compression simple.

TYPE D'ESSAI	Nombre d'éprouvettes	Valeur moyenne de la contrainte de rupture kg/cm^2	Ecart type kg/cm^2	Coefficient de variation	$\frac{\sigma t_r}{\sigma^{ts}_{tr}}$ ou $\frac{\sigma c_r}{\sigma^{cs}_{cr}}$	Valeur moyenne des deux autres contraintes principales kgf/cm^2	
Traction simple	12	σ^{ts}_{tr} = - 26,6	2,7	0,10	-	σ_θ = 0	σ_r = 0
Fendage	6	σ^{f}_{tr} = - 35,6	4,0	0,11	1,34	σ_θ = 107	σ_l = 0
Bresilien creux	12	σ^{a}_{tr} = - 134	11	0,08	5,04	σ_θ = 0	σ_l = 0
Eclatement	6	σ^{e}_{tr} = - 53,8	6,1	0,11	2,02	σ_r = 0	σ_l = 41,2
	5	= - 63,8	13,2	0,21	2,40	= 5	= 57,7
	5	= - 57,3	9,8	0,17	2,15	= 10	= 61,6
	5	= - 73,4	10,4	0,14	2,76	=15	=82,8
	5	= - 71,0	15,5	0,22	2,67	= 20	= 89,8
	5	= - 80,1	14,7	0,18	3,01	= 25	=106
Torsion	12	σ^{t}_{tr} = - 39,9	7,4	0,19	1,50	σ_r = 0	$\sigma_{45°}$ = 39,9
Compression simple	6	σ^{cs}_{cr} = 300	33	0,11	—	σ_θ = 0	σ_r = 0
Compression triaxiale	6	σ^{tr}_{cr} = 475	85	0,18	1,58	σ_θ = 20	σ_r = 20
	4	= 845	110	0,13	2,82	= 100	= 100
	5	= 1771	140	0,08	5,90	= 500	= 500
	4	= 3012	41	0,01	10,0	=1000	= 1000
	3	= 3997	73	0,02	13,3	=1500	= 1500
	3	= 5095	38	0,01	17,0	= 1900	= 1900

A partir des essais de traction simple et de compression simple et triaxiale une courbe intrinsèque a pu être tracée (cfr. figure 3), elle a pour équation :

$$\sigma = 0{,}07194\ \tau^{1{,}4697} - 26{,}6 \quad (\text{en kgf/cm}^2).$$

II.4. DETERMINATION EXPERIMENTALE DE L'ETAT TRIAXIAL DE DEFORMATION DANS LES ELEMENTS EN BETON ARME SOUMIS A DES CHARGES STATISTIQUES [9]

II.4.1. But

Le but de cette étude expérimentale est de déterminer la répartition des composantes de la déformation à l'intérieur d'éléments en béton soumis à différents états de sollicitation. Elle permet en particulier d'analyser la validité de deux hypothèses suivantes, habituellement émises dans les études théoriques et expérimentales concernant le comportement du béton :

- la connaissance d'une ou de deux composantes de l'état de déformation suffit pour déterminer cet état avec une précision acceptable, en négligeant les autres composantes;
- l'état de déformation interne des éléments étudiés peut être déterminé à partir des déformations mesurées sur les faces extérieures.

II.4.2. Dispositif de mesure

Des mesures de l'état de déformation à l'intérieur d'éléments en béton (éléments cylindriques comprimés et tendus, poutres fléchies et prismes comprimés) ont été effectuées à l'Université de Liège à l'aide de capteur comportant douze jauges électriques. Ces jauges étaient collées à mi-longueur des arêtes de tétraèdres formées de bandes en cuivre ou en araldite longues de 70 ou 100 mm. Les six dilatations mesurées permettent de déterminer les six composantes du tenseur déformation.

II.4.3. Résultats principaux

- Relation entre les déformations sur les faces extérieures et les déformations à l'intérieur des éléments

  Plusieurs auteurs ont observé des différences considérables entre les déformations mesurées sur les faces extérieures et celles déterminées à l'intérieur des éléments en béton soumis à compression.

  Lors de l'étude considérée, l'absence de différences importantes et systématiques entre les indications des jauges de surface et celle des jauges situées sur les arêtes des tétraèdres montre que la longueur des jauges et des arêtes des tétraèdres ont été choisies judicieusement et que l'introduction des capteurs à l'intérieur de la poutre ne perturbe pas de façon sensible les champs de déformation. L'ensemble des résultats ne permet pas de formuler des conclusions définitives sur les relations entre les déformations des faces extérieures et les déformations intérieures des éléments comprimés; en effet,

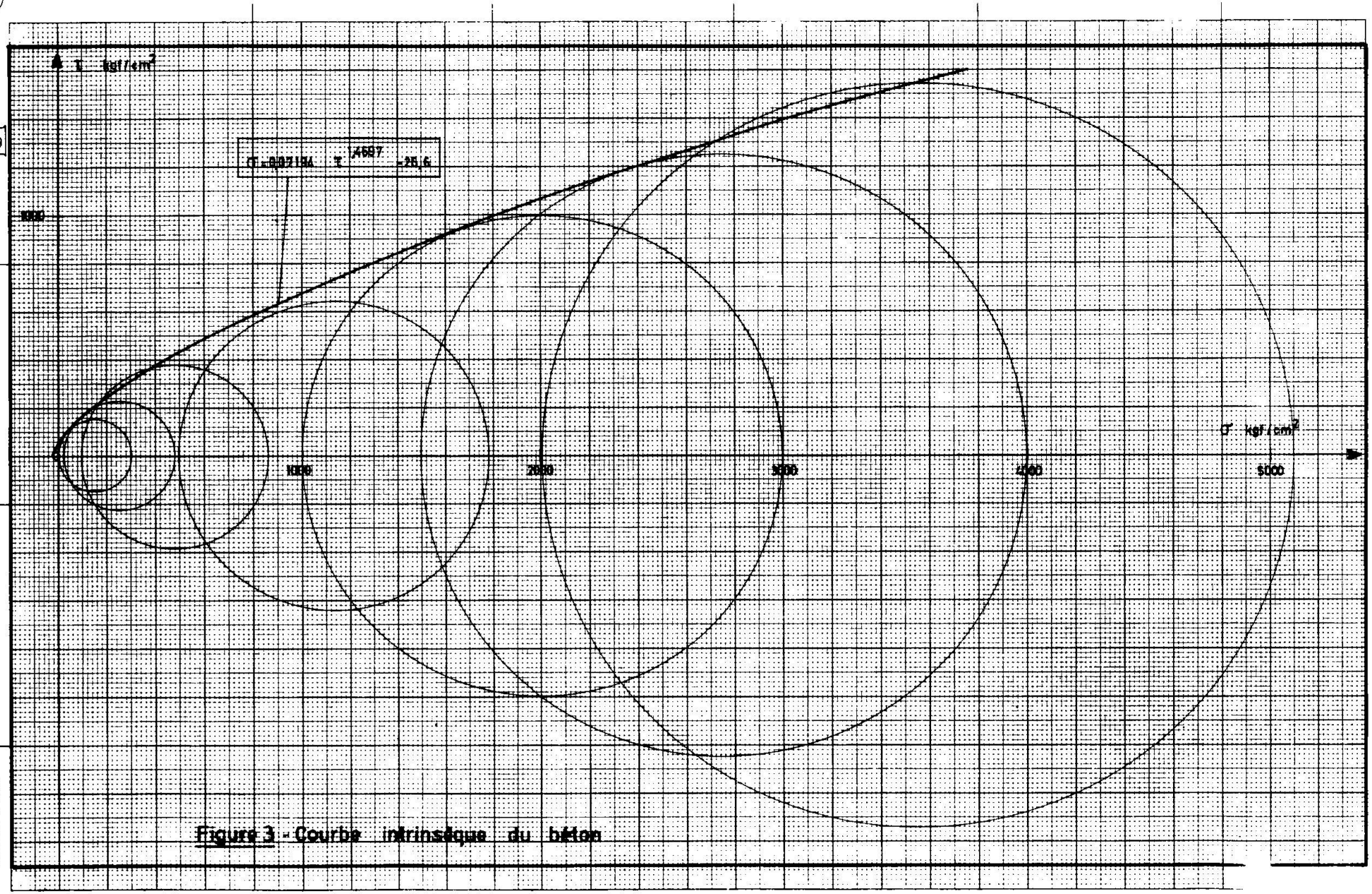

Figure 3 - Courbe intrinsèque du béton

dans les poutres fléchies, les champs de déformation varient de façon importante sur la hauteur de la zone comprimée, ce qui semble entraîner des perturbations dans les relations envisagées.
Les résultats montrent cependant que, dans la plupart des cas, les déformations de la surface extérieure sont inférieures aux déformations internes, ce qui correspond aux résultats des observations faites par de nombreux chercheurs.

- Analyse expérimentale du coefficient de Poisson

L'étude n'a pu mettre en évidence l'évolution du coefficient de Poisson à l'intérieur des éléments considérés, à cause du trop petit nombre de capteurs placés.
Elle a par contre permis l'analyse de l'influence de la charge et de l'âge du béton.
Il est apparu que les valeurs de ν augmentent systématiquement avec la charge; cette évolution croissante est en concordance avec la plupart des résultats qui figurent dans la littérature et qui proviennent de mesures effectuées à la surface des éprouvettes.
Il a été observé d'autre part que l'évolution de ν diffère suivant l'âge du béton. Dans le cylindre tendu, on observe une augmentation rapide de ν, tandis que dans le cylindre comprimé, les valeurs de ν évoluent peu jusqu'à l'âge de 154 jours. Dans le prisme comprimé pour lequel on dispose de mesures effectuées à 620 jours, on observe une diminution importante de ν. Dans la poutre fléchie, après un accroissement rapide de ν observé à 56 jours, on constate dans la suite une légère décroissance.
L'ensemble des capteurs de chaque élément conduit au même type d'évolution de ν en fonction de la charge et de l'âge du béton.

- Non-homogénéité des déformations à l'intérieur des éléments en béton

Par non homogénéité des déformations on entend les différences entre les déformations des différentes zones du béton des éléments.
L'ensemble des mesures effectuées au cours de cette étude permet d'observer une certaine stabilité des propriétés locales du béton, notamment en ce qui concerne les directions des différentes composantes de déformation et leurs rapports.
Si on peut admettre que les propriétés locales du béton sont relativement stables, sous les différentes charges et aux différents âges, on constate par contre que ces propriétés varient fortement d'une zone à l'autre dans les éprouvettes d'essai.
Les expériences mettent clairement en évidence des différences importantes de déformabilité des différentes zones des éléments en béton. La répartition des zones fortes et des zones faibles est déterminée, soit au cours de la confection des éléments, soit au cours de leur première mise en charge.

- Précision de la méthode expérimentale proposée

L'étude d'erreurs effectuée conduit notamment aux conclusions suivantes :

- les erreurs sur les valeurs des déformations principales ne dépassent jamais 5 $\Delta\varepsilon$; dans la plupart des cas, elles sont

comprises entre 1,5 $\Delta\varepsilon$ et 2,0 $\Delta\varepsilon$, $\Delta\varepsilon$ étant l'erreur d'une indication fournie par une jauge simple;

- les erreurs sur les directions des contraintes principales dépendent dans une certaine mesure de la situation des axes de la rosette envisagée par rapport aux directions principales; dans certains cas de sollicitations, où toutes les déformations principales sont presque identiques, cette erreur peut atteindre 45° ou même davantage, ce qui met en doute l'intérêt des mesures; cependant le plus souvent, ces erreurs se situent entre 1° et 10°.

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REFERENCES BIBLIOGRAPHIQUES

[1] CAMPUS F., DEHOUSSE N.M., HAMOIR J. - Les pressions interstitielles dans les ouvrages hydrauliques et leur fondation. Matériaux et Constructions, Vol. 2, n° 10 et 12, 1969.

[2] DEHOUSSE N.M., DIAB B. - Essais sur les pressions interstitielles dans les ouvrages hydrauliques. Rapport des laboratoires du Génie Civil et d'Hydraulique Fluviale, 1972.

[3] DIAB B. - Contribution à l'étude du complexe barrage-fondation et explication des séismes dus au remplissage de certains réservoirs. Thèse de doctorat, Annales des Travaux Publics de Belgique, n° 6, 1972-1973.

[4] GUIAUX P., JANSS J. - Comportement au flambement de colonnes constituées de tubes en acier remplis de béton. C.R.I.F. MT 65, Bruxelles, novembre 1970.

[5] ANSLIJN R., JANSS J. - Le calcul des charges ultimes des colonnes métalliques enrobées de béton. C.R.I.F. MT 89, Bruxelles, avril 1974.

[6] TADENSZ, GODYCKI, CWIRKO - Scinamie w Zelbecie. Arkady Warscawo 1968.

[7] JANSS J. - Neuvième congrès de l'A.I.P.C., pp. 125-132, Amsterdam, mai 1972.

[8] FAGNOUL A., BONNECHERE F. - Appareils à sollicitations pluriaxiales du laboratoire de mécanique des sols de l'Université de Liège. Colloque International R.I.L.E.M., Cannes 1972.

[9] BAUS R., BRANDT A., BRENNEISEN A. - Analyse des déformations des éléments en béton sous charges statiques. Mémoires C.E.R.E.S. (nouvelle série) n° 42, Liège, mars 1973.

RESUME

Le rapport expose principalement les quatre recherches suivantes:

1. Les pressions interstitielles dans les ouvrages hydrauliques et leur fondation.
2. Prise en considération de l' état triaxial de contrainte dans les profils creux circulaires remplis de béton chargés axialement.
3. Influence de l' état de contraintes sur les résistances en traction et en compression d' un béton.
4. Détermination expérimentale de l' état triaxial de déformation dans les éléments en béton armé soumis à des charges statiques.

SUMMARY

The report mainly deals the following researches:

1. Pore pressure in hydraulic works and their foundations.
2. Consideration of triaxial stress state in axially loaded hollow circular profiles filled with concrete.
3. Stress state influence on concrete strength in tension and compression.
4. Experimental assessement of the triaxial strain state in reinforced concrete members subjected to static loads.

ZUSAMMENFASSUNG

Es werden hauptsächlich die folgenden Studien beschrieben:

1. Zwischenraumdruck in hydraulischen Anlagen und ihren Grundlagen.
2. Berücksichtigung des triaxialen Stress-Zustandes in axial geladenen, runden, mit Beton gefüllten Profilen.
3. Einfluss des Stress-Zustandes auf Spannung und Druckwiderstand.
4. Experimentelle Bestimmung von triaxialen Deformierungen in Eisenbeton-Elementen unter statischer Ladung.

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SEMINAR on:
«CONCRETE STRUCTURES SUBJECTED TO TRIAXIAL STRESSES»
17th-19th MAY, 1974 - ISMES - BERGAMO (ITALY)

Construction and Calculation of Three-Dimensional Structures with particular reference to Prestressed Concrete Reactor Pressure Vessels

Konstruktion und Berechnung Dreidimensionaler Bauwerke am Beispiel eines Spannbeton- Reaktordruckbehaelters

Construction et calcul de structures tridimensionnelles avec référence en particulier aux caissons en béton précontraint pour réacteurs nucléaires

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Preface

Load-bearing members of structures can be idealized in most cases by one dimensional or two dimensional systems (beams, arches, plates, shells). Hence, it was not necessary so far, to investigate the three dimensional state of stress and strain. Just in recent time, extremely loaded structures subjected to triaxial stresses get more and more important. Particularly, thick walled vessels are to be mentioned here, e.g. reactor pressure vessels for nuclear power plants. In various respects these vessels typify complicated three dimensional stress and strain problems, since considerably high static and possibly dynamic loads and elevated temperatures occur. If prestressed concrete is used, non-linear and time-dependent material behaviour further complicates the design work.

Thus, problems concerning prestressed concrete pressure vessels for nuclear power plants shall be discussed in the following which are the main object of investigations in the field of concrete structures subjected to triaxial stresses in the Federal Republic of Germany. Problems of construction, calculation of the states of stress and strain under service load conditions, and calculation of ultimate load behaviour will be considered.

1. Constructional Problems

1.1 General remarks

The first task in projecting a structure is the design of the construction, because it influences among other things the choice of the method of stress calculating

and the choice of the erecting procedure. Thus, reflections on prestressed concrete pressure vessels shall be opened by discussing several important constructional problems.

Usually prestressed concrete reactor vessels are characterized by thick cylindrical walls and deep top and bottom closures. At numerous points walls and end slabs are penetrated by openings of various diameters. In order to withstand the internal pressure, above all the vessel must be tight against coolant. This is obtained by coating its inner surface with a steel liner.

Most important constructional problems are the technique of prestressing, the design of penetrations and inner haunches between end slabs and cylindrical wall, the performance of liner anchoring, as well as problems concerning construction joints and concrete casting process.

1.2 Prestressing

Prestressing is an essential element for a prestressed concrete pressure vessel. The prestressing system has to be appropriate and economical. In early reactor vessels, horizontal prestressing was performed by installing cables inside the wall and anchoring in buttresses, like conventional container design (Fig. 1.1). The difficulties of this procedure are obvious. Arranging of so many horizontal and vertical prestressing cables yields significant overlapping in the anchorage zones. During prestressing, reductions of prestressing force arise due to friction. These facts enlarge prestressing steel consumption. Furtermore the large number of anchoring elements needed is expensive. Hence, this method is uneconomic, additionally to its other disadvantages. For that reason, much effort was done to develop a circumferential prestressing suitable for structures like these. For that purpose, special wire-winding procedures were created, by which a number of wires can be tensioned simultaneously. In Fig. 1.2 a prestressing system using a wire-winding machine is shown. Here prestressing wires are wound into annular channels. In another system, well arranged wires are wound polygonally across guide borders used as intermediate anchoring simultaneonsly (Fig. 1.3). For vessels with smaller diameter, wire-winding technique is hardly applicable. For those cases, a system has been developed, where prestressing tendons are assembled on the vessel in a pretensioned state. Fig. 1.4 shows the principle of this technique. The concrete vessel is cast and hardened previously. Annular closed prestressing tendons are tensioned against a straining ring above or below the vessel an then moved into their final position in this tensioned state. There, prestressing forces are transmitted to

Schnitt im Zylinderbereich

Schnitt in der Bodenplatte

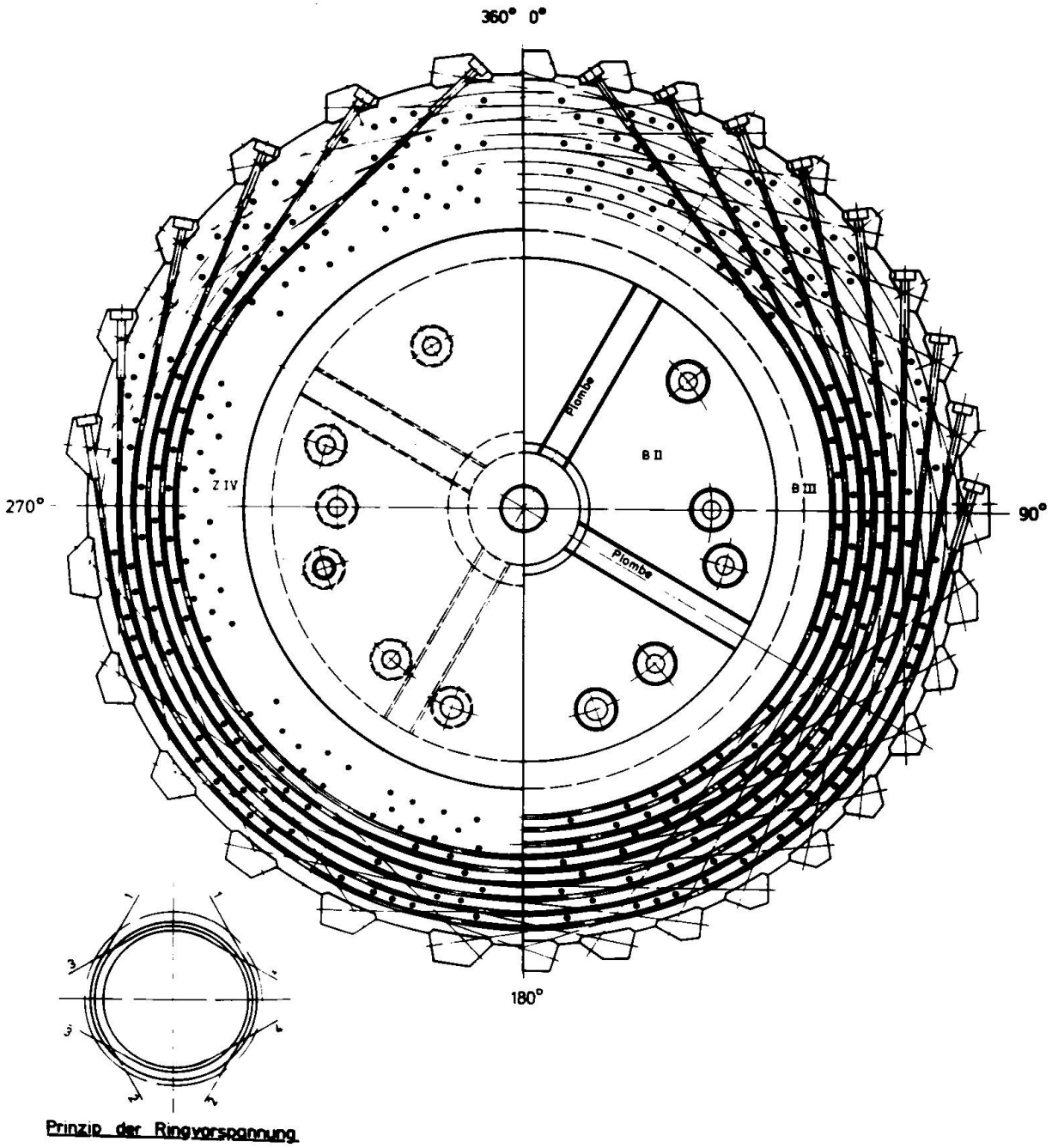
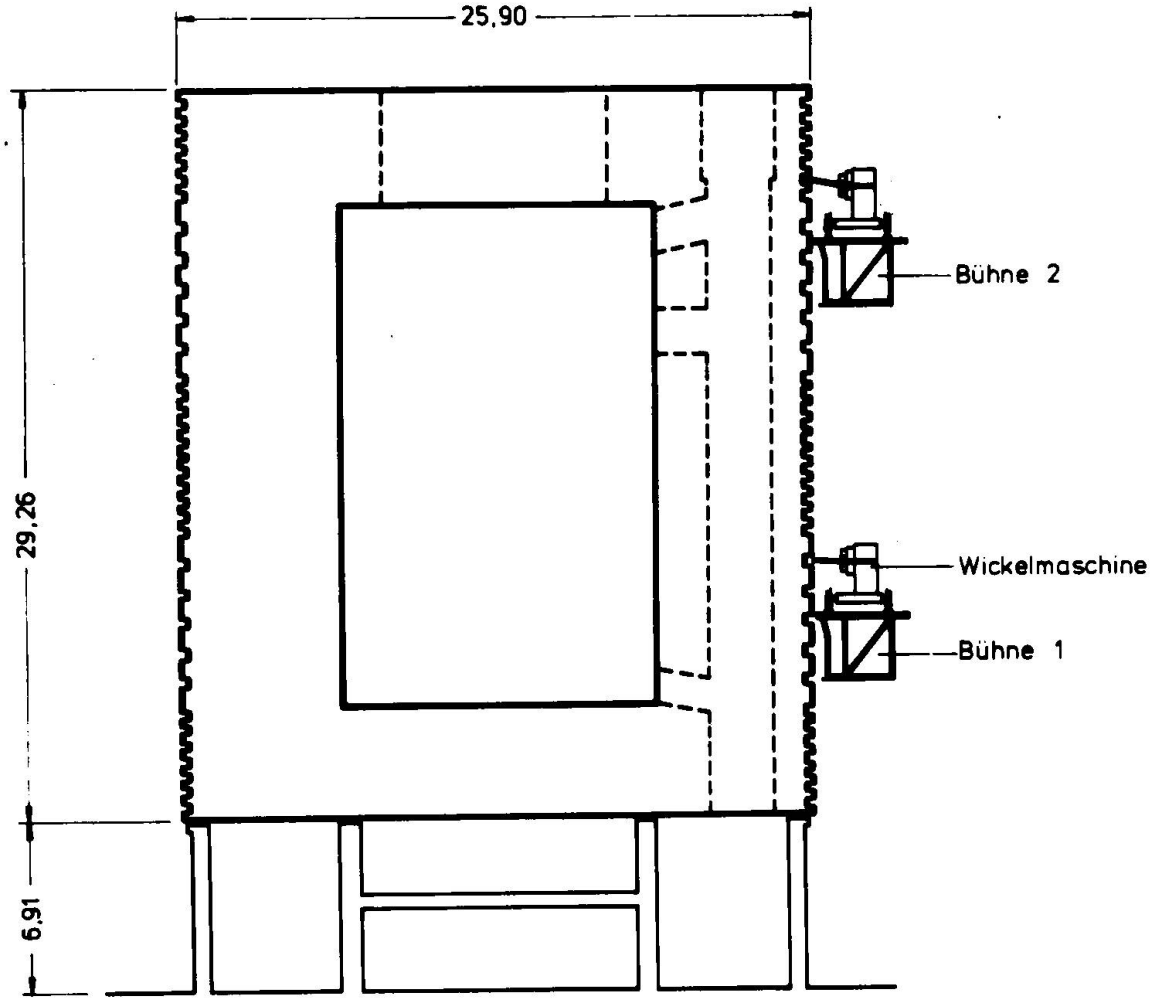


Bild 1.1 SBB - Lisenenvorspannung
PCRV Buttress prestressing system



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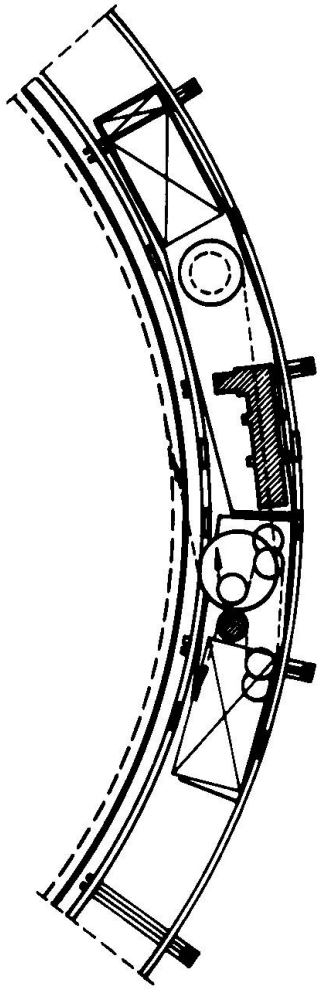


Draufsicht Wickelmaschine

Bild 1.2

SBB-Wickelvorspannung
PCRV Wire-winding prestressing system

the vessel by unloading the straining ring. This method of prestressing can also be carried out with immediate bond. In this case, each straining ring is loosened after the corresponding lifts have hardened (Fig. 1.5).

1.3 Structural problems due to local stresses

In the regions of penetrations, buttresses - if existing -, and inner haunches, local zones with stress concentrations arise. In these regions, besides high compressive stresses also significant tensile stresses can occur which practically cannot be compensated by prestressing. Regions like these with local stresses yield some special structural problems, caused particularly by an accumulation of bonded reinforcement. Fig. 1.6 demonstrates the reinforcing arrangement for buttresses. In Fig. 1.7 bonded reinforcement around the haunch between bottom closure and cylindrical wall can be seen. Most exceptional difficulties due to stress concentrations are to be solved in the standpipe region of the top slab and in zones around major penetrations. Substantial advantages with special regard to constructional problems could be gained by using steel fiber reinforced concrete. This is a new type of material which is today in the stage of development or testing, respectively. Its main characteristic is admixing short steel fibers (about 25 to 3o mm long and o.25 to o.4 mm in diameter) to a suitable concrete mix. This yields a quasi-homogeneous material with uniform strength qualities in all three dimensions. Particularly tensile strength of the concrete is increased. Fig. 1.8 outlines the increase of bending tensile strength in dependence of the quantity of steel fibers added. It is obvious that in regions where tensile stresses of a certain amount occur usual reinforcement can be avoided by using this special material. Fig. 1.9 shows the construction of the haunch between bottom slab and wall, if steel fiber reinforced concrete is used. In Fig. 1.1o a comparison between designs with conventional reinforced concrete and steel fiber reinforced concrete is pointed out for the zone around the pebble outlet penetration of a high temperature reactor.

1.4 Liner

The liner - i.e. the steel membrane at the inside surface of the vessel - acts as a gas-tight skin, that has to prevent leakage of the contained coolant through the concrete walls of the vessel. Its function as load carrying part is negligible in most regions of the vessel. The thickness of the membrane should be chosen as thin as possible in order to get minimal intermediate reactions between liner and concrete. On the other hand, requirements like necessity of producing a good quality weld and use of the liner as internal formwork of the concrete

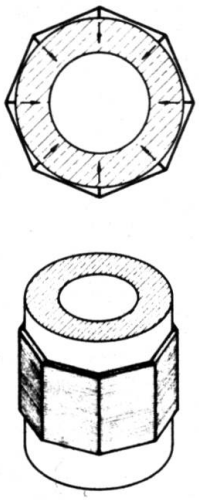


Bild 1.3 SBB - Prisma - Vorspannung
PCRV Prisma prestressing system

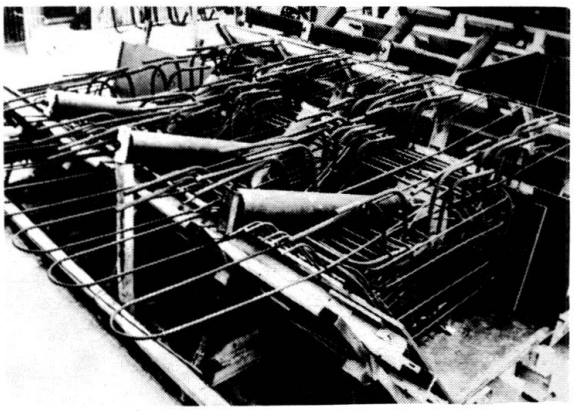


Bild 1.6 Bewehrung einer Fertigteillisene des THTR - Spannbetondruckbehälters
Precast buttress reinforcement of the THTR vessel

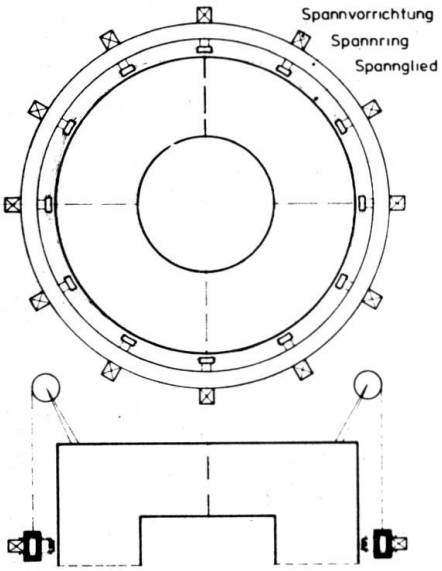


Bild 1.4
SBB - Ringvorspannung (mit Spannringen)
PCRV circumferential prestressing (with straining rings)

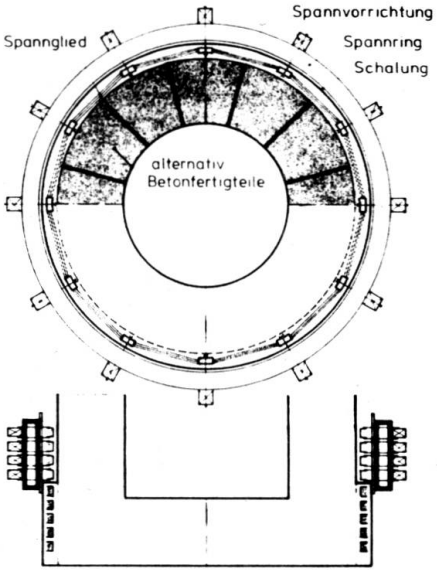


Bild 1.5 SBB - Ringvorspannung (mit Spannringen und sofortigem Verbund)
PCRV circumferential prestressing (with straining rings and immediate bond)

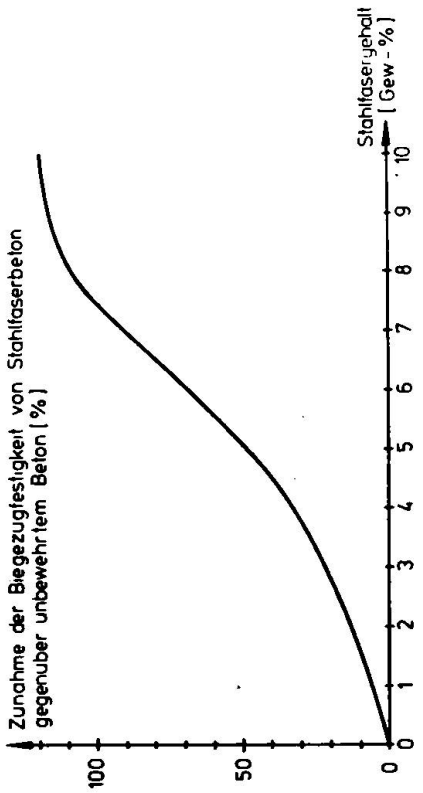


Bild 1.8 Zunahme der Biegezugfestigkeit von Stahlfaserbeton in Abhängigkeit vom Fasergehalt
Bending tensile strength increase of steel fiber reinforced concrete versus content of fibers

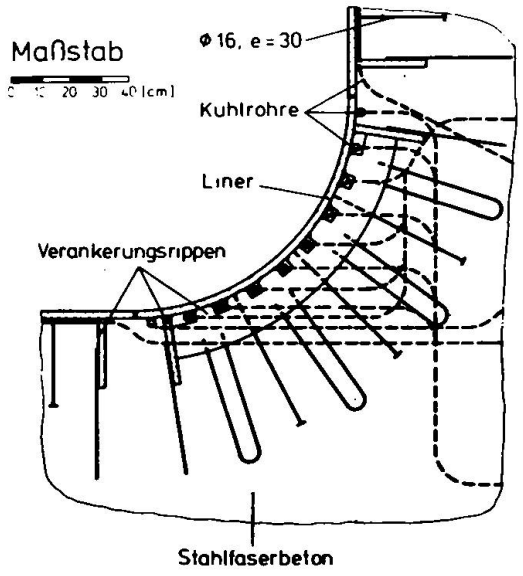


Bild 1.9 Voute eines Spannbetonbehälters bei Verwendung von Stahlfaserbeton
Haunch of a PCRV using steel fiber reinforced concrete

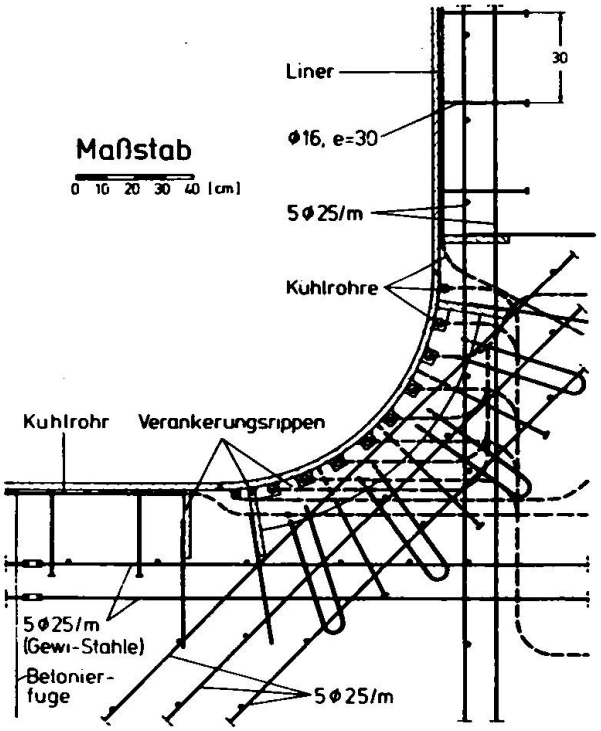
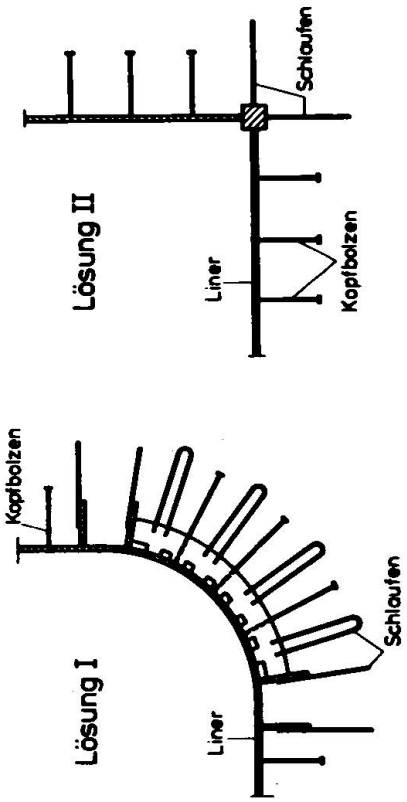


Bild 1.7 THTR-Spannbetonbehälter, untere Voute
THTR vessel, lower haunch



Linerübergang vom Boden zum Zylinder
Transition region between bottom and wall liner

Bild 1.11

set certain limits for its minimum thickness. Additionally the liner has to be protected against buckling due to high compressive loads caused by vessel deformations or temperature restraints by a well balanced proportion of thickness to concrete anchoring. Thicknesses between 12 and 3o mm were found to be most suitable, where the thicker steel part rather concerns the flat top and bottom liners. For anchoring the liner in the concrete, butt welded stuts with diameters from 12 to 2o mm and distances from c. 18 to 3o cm are commonly used in the meantime. Generally, small relative displacements between liner and concrete are admissible. So, anchoring by bolts only - i.e. a junction by spring elements in principle - is sufficient. However, in the haunches between cylindrical wall and end slabs as well as in zones around penetrations, possibly an almost rigid joint can be necessary. Due to strong changing of stress or strain gradients high local forces have to be transfered between liner and concrete at these points. In Fig. 1.11 two construction alternatives are pointed out for the lower haunch,e.g. At first sight, the rounded version (solution I) seems to have certain advantages concerning the stress state within the vessel. But the liner has to be anchored in the concrete so as to fulfill its function even under these strongly changing load states in the haunch region. Hence, slipping of the liner from the cylinder or slab plane into the rounded region or vice versa has to be provided by strong ribs welded on the liner. This generally requires additional reinforcement which on the other hand cannot be easily arranged near the liner due to even these ribs. Moreover, feeding pipes for the liner cooling system must be installed in this zone, (Fig. 1.7). All this needs high accuracy and carefulness in constructing and reinforcing this detail of the structure. These problems get substantially more simple, if a construction like solution II in Fig. 1.11 is chosen. Reinforcement can be arranged much better, because there are no ribs.

1.5 Concrete Quality, concrete casting, construction joints

Mass concrete conditions and high demands on quality of prestressed concrete reactor vessels require carefulness in specifying the optimum concrete mix and fixing the lifts and bays for casting. Besides high compressive and tensile strength under ambient and elevated temperatures, it is of great importance that thermal expansion, shrinkage, creep, and elastic deformations are as low as possible. Creep deformations significantly influence the undesirable loss of force in the prestressing tendons. Furthermore, a high concrete density is required to give a good absorption of neutron and gamma rays. High thermal conductivity influences above all the dimensioning of the cooling system which

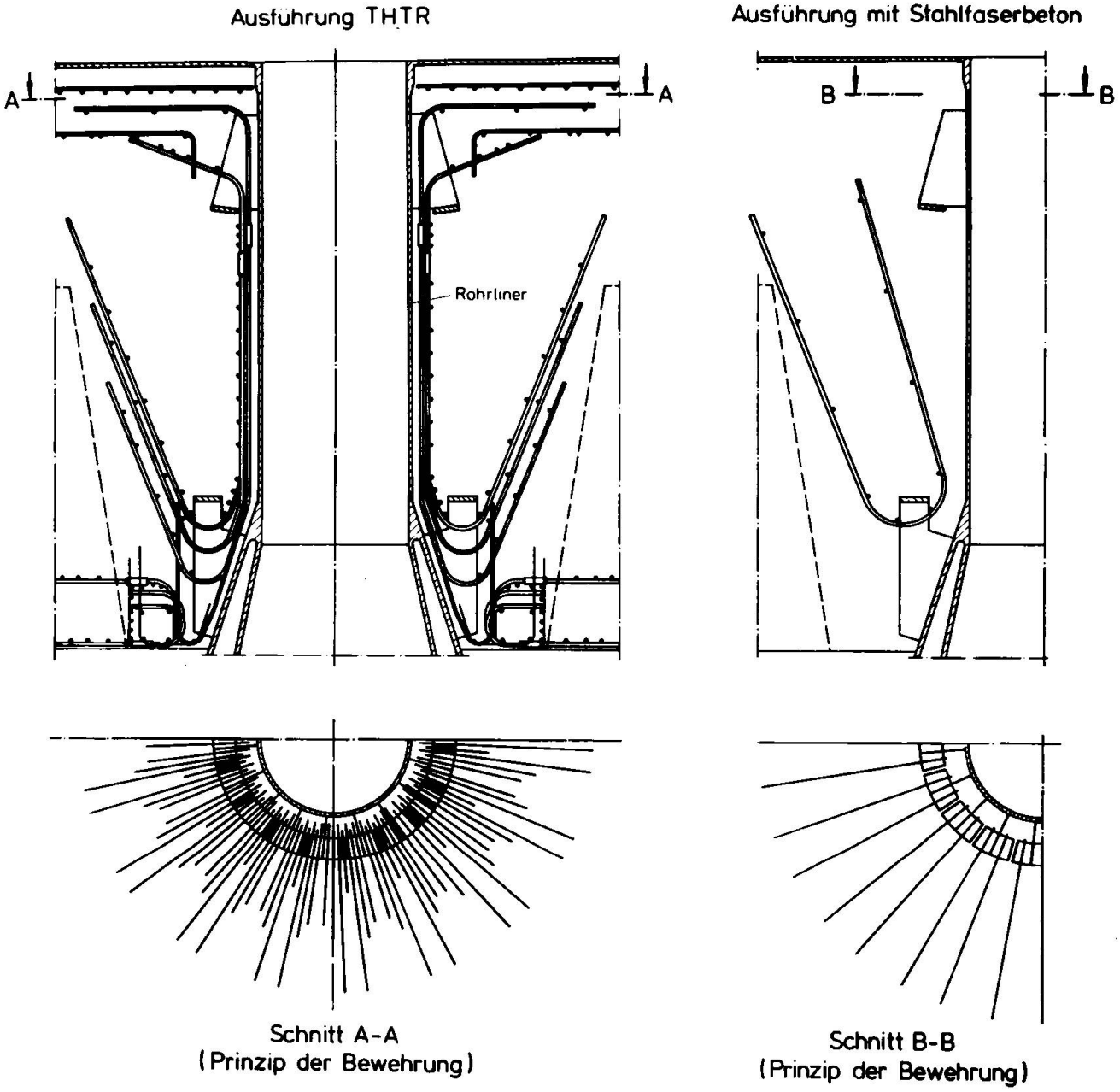


Bild 1.10 THTR-Spannbetonbehälter, Bewehrung um das Kugelabzugsrohr
THTR vessel, reinforcement near pebble outlet penetration

has to hold a provided temperature in the concrete. A reasonable choice of subdivisions in casting lifts and bays is important for quickly deducing hydratization heat. It has been found to be suitable to make casting lifts not deeper than 2 meters. Lifts are almost subdivided into several bays which are of some advantage for continuous progress of work, too. However, much more formwork is necessary then, and the well-known problems with vertical construction joints arise.

Designing a prestressed concrete pressure vessel yields altogether even more problems then mentioned here, for example the difficulties in concrete casting below the liner bottom in order to get a continuous supporting of the liner by concrete. But discussing all these special questions in detail would be beyond the scope of this report.

2. Experiences in the Application of Dynamic Relaxation

2.1 Introduction

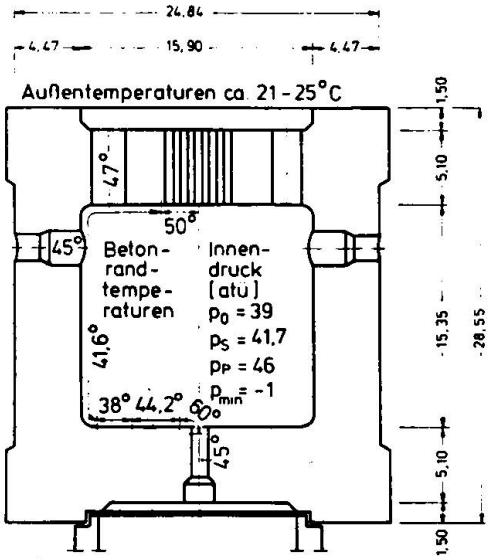
Prestressed concrete pressure vessels are three-dimensional structures, whose state of stress and strain is to be calculated three-dimensionally. These calculations have to take into account the complicated geometry of the vessel with large openings in the walls, the unhomogeneous and partly nonlinear behaviour of the material as well as the liner and the penetration liners with their influence on the vessel.

Fig. 2.1 shows the vessel of the THTR-nuclear power station as an example for such a structure. The inner surface of the vessel and the penetrations are covered with the mentioned liners. The high internal pressure of the vessel and its high temperature load require high prestressing. These are the essential loads compared with other loads like dead weight, installations etc.

The three-dimensional calculations of prestressed concrete pressure vessels are mainly done today by two methods. These are the finite element and the dynamic relaxation method. Both made important progress in the last years. For calculating three-dimensional or plane structures they seem to be of the same efficiency. For the THTR pressure vessel the dynamic relaxation mainly was preferred. This method was developped to the state of today in our institute. Because of these reasons it will be reported about the dynamic relaxation here.

2.2 Description of the dynamic relaxation

For the calculation of prestressed concrete pressure vessels the restriction on infinitesimal deformations is sufficient. All nonlinear problems concerning the behaviour of the material can be reduced to linear step-by-step calculations if using numerical methods. Thereby the linear elasticity can be used for the behaviour of the material, too.



p_0 = Betriebsdruck p_P = Prüfdruck
p_S = Störfalldruck p_{min} = evakuierter Behälter

Temperatur- und Innendruckbelastung

Bild 2.1 THTR - Spannbetonbehälter, Geometrie und Belastung aus Reaktorbetrieb
THTR vessel, geometry and loads due to reactor service

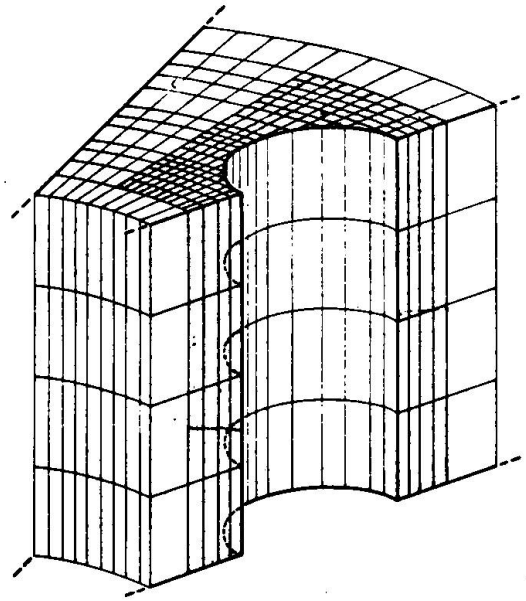


Bild 2.2 Dynamische Relaxation, Beispiel einer Rasterteilung
Dynamic relaxation, an example for a computing mesh

The basis of the dynamic relaxation in the case of the three-dimensional continuum is the system of differential equations, comprising the 3 dynamic equations of equillibrium and the 6 constitutive equations. At first it seems not to be suitable to introduce the time as fourth dimension into the calculation of static problems. But this leads to differential equations of hyperbolic form which are easy to solve with difference methods. For dynamic relaxation a special form of difference method is to be used, which solves the difference equations dependent on time. To get the static solution it is necessary to add damping terms, which are proportional to the velocity. By well chosen damping coefficients, only a short time is taken to lead the dynamic problem to the wanted static solution.
The solution of the initial value and boundary value problem begins from an arbitrary initial stress-strain-state of the structure, loaded by forces or enforced displacements at the boundaries or by the volume loads. Often an approximate stress-strain-state is known, from which the calculation can be started in order to get the solution in a shorter time. The boundary conditions are considered at every step of the iterative process of the numerical calculation. The stability and convergence of the numerical solution can be guaranteed by a simple criterion. Because it is not necessary to know the exact course of the vibration process, some simplifications are possible for the calculation. One of them is the possibility to store only the values of the last step of the iterative calculation. Only these values are necessary to get the values of the next iteration step. Because of this problems with thirty thousand or more unknows can be calculated only in the core storage of computers of middle size which are today available in universities or important companies everywhere. The needed storage for the calculation is smaller than for other numerical methods like finite elements etc. The basis of the dynamic relaxation is described in detail in literature for example /2.1/ and /2.1o/, so that the description of this method will not be continued here.

Similar to the calculation of stresses and deformations the state of temperature is computed by difference equations. The basis is the known differential equation of Fourier. The calculation here realistically describes the time dependant building up of the temperatures in the continuum. By that, stationary and unstationary states of temperature can be computed, vis. /2.5/. With the knowledge of the state of temperature and its time dependent variations in the vessel walls the calculations of stresses and deformations caused by thermal loads can be performed.

2.3 Calculation with dynamic relaxation

For numerical calculations with dynamic relaxation it is necessary, to devide the continuum or the pressure vessel into body elements corresponding to the finite differences. The width of the grid can be different so that in regions

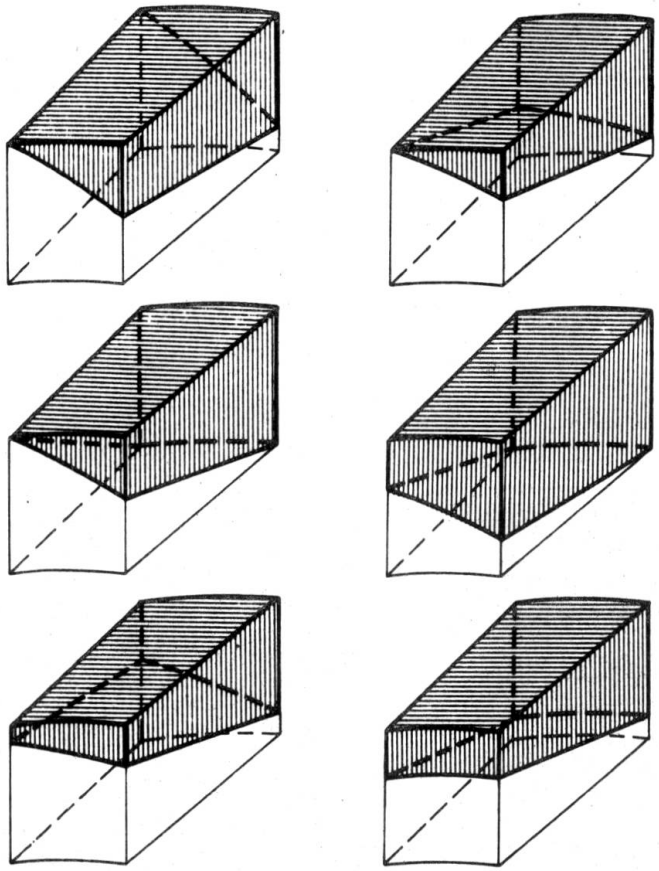


Bild 2.3 Dynamische Relaxation, Beispiele räumlicher Elementformen
Dynamic relaxation, examples for three-dimensional elements

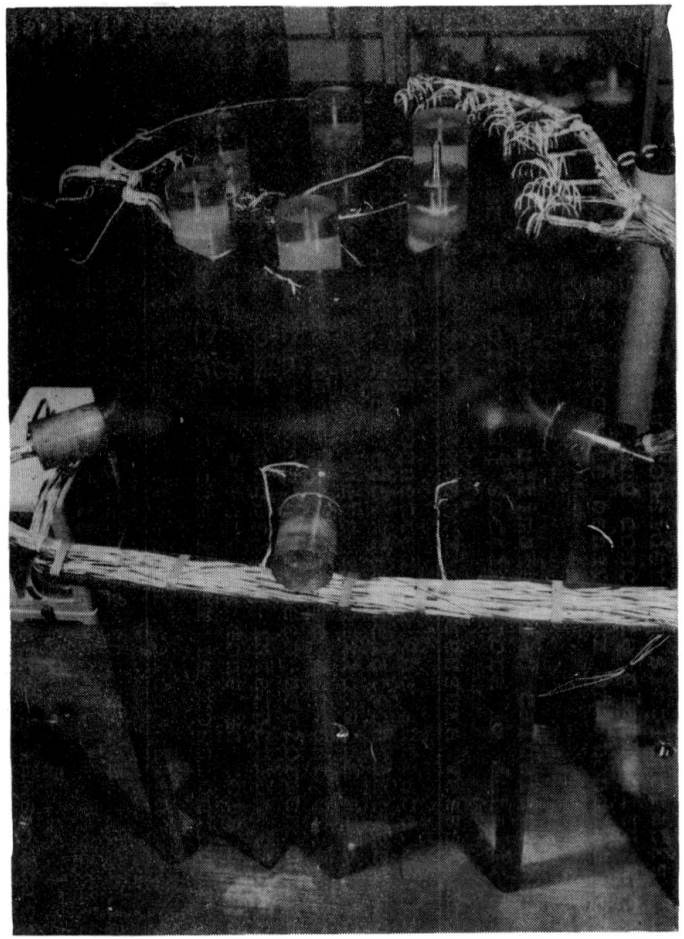


Bild 2.4 THTR-Gießharzmodell M 1:45
THTR 1:45 epoxy resin model

of local stress concentrations a grid with smaller differences is possible, vis. Fig. 2.2. In such a way the grid can be adjusted to the requirements of the structure. For other difference methods the difficulties due to the boundary conditions are well known, especially for boundaries not coinciding with the directions of the coordinates. The special kind of difference method used by dynamic relaxation and the special method of solving the differential equations of the elastic continuum simplify these problems considerably. Moreover any boundaries can be considered, vis. Fig. 2.3. In such a way it is possible to calculate the state of stress and deformation of any three-dimensional or two-dimensional structures. The computation itself is done by an iterative process, described earlier, the way that at different time steps stresses and displacements are calculated successively by the finite difference equations considering the boundary conditions. The calculation is finished, when the vibration is damped to the static solution.

The dynamic relaxation computer programs, set up in our institute, were tested by comparison with known results of special problems or by model experiments. Fig. 2.4 shows the epoxy-resin model of the THTR-vessel, scale about 1:45. At the inner and outer surfaces of the model and within the walls strain gauges had been arranged. For the installation of strain gauges inside the walls a special technic was develloped. Additionally meassurements of the deformations were done at the outer surface of the model. The model was loaded by internal pressure. The calculation of the vessel model and the results of the experiments showed a very good agreement. This comparison is possible for the elastic range of course. As an example of these comparisons Fig. 2.5 shows the tangential stresses at the upper surface of the vessel. Additionally the result of an axisymmetric calculation is plotted at the angle of 30°.

2.4 Application of the method to variable and non-linear material behaviour

More than usual structures, prestressed concrete pressure vessels have to be calculated with due regard to material behaviour of concrete. The higher concrete temperatures yield stronger creep, vis. Fig. 2.6. Shrinkage varies across the wall section depending among other things on temperature gradient and moisture content influenced by temperatures, vis. Fig. 2.7. Also other material properties are influenced by temperature more or less. Thus, material behaviour does not only depend on time, as commonly known, but on temperature, too.

Tensile strength of concrete is only small. In regions with local stresses or in the case of vessel design based on partial prestressing, tensile strength of concrete is exceeded. On the other hand, in local zones

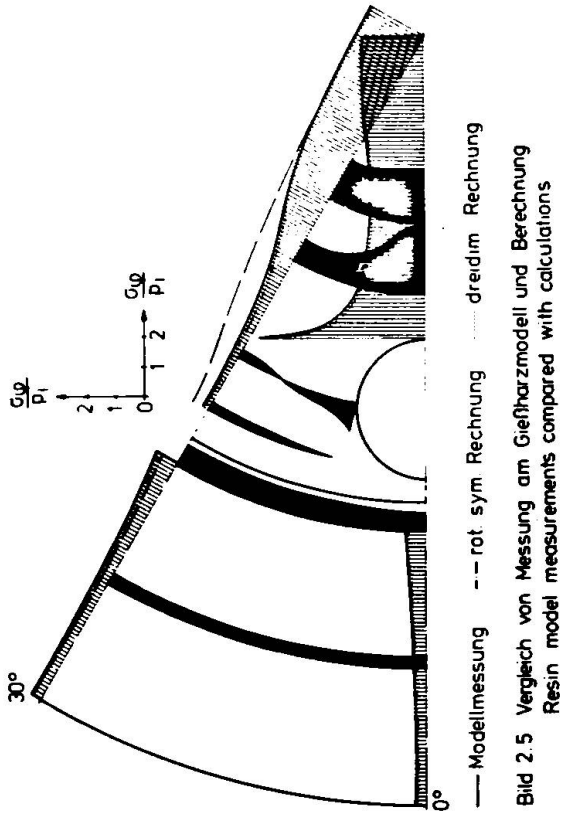
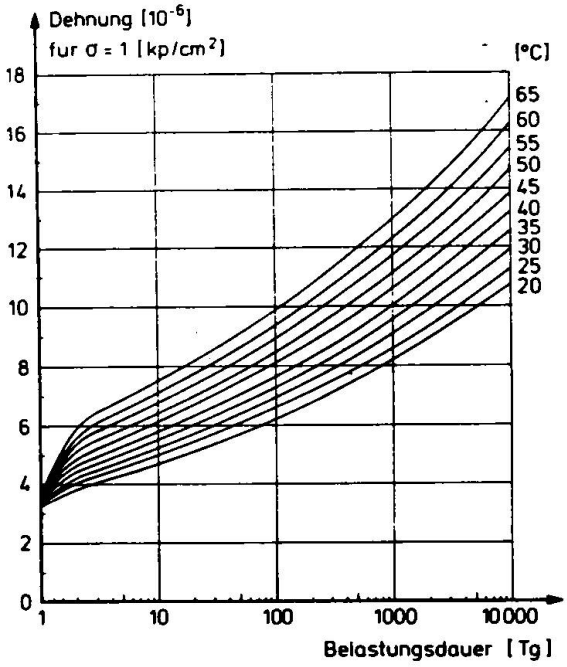
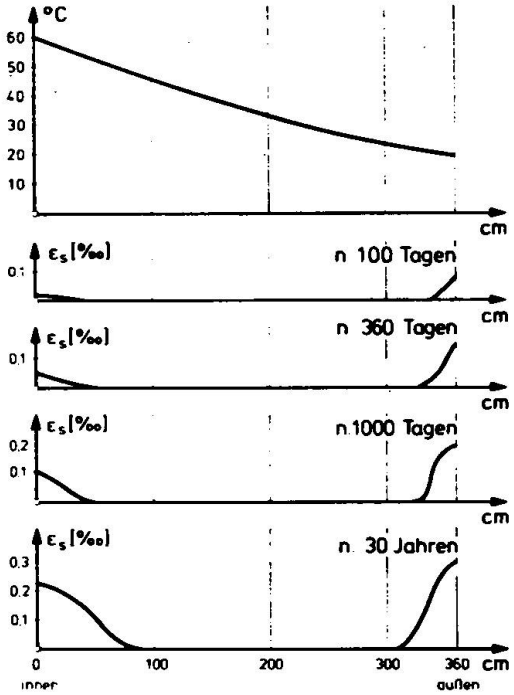


Bild 2.5 Vergleich von Messung am Gießharzmodell und Berechnung
Resin model measurements compared with calculations



$$\lg \varepsilon_{sp} = [1 - \frac{T}{25} \cdot 10^{-(t-t_{BA}+1)}] \cdot [0{,}46 + 0{,}0045 \cdot T + 0{,}12 \cdot \lg(t - t_{BA} + 1)]$$

Bild 2.6 Kriechfunktion eines typischen Behälterbetons
Creep function of a typical vessel concrete



Freies Schwindmaß über die Behälterwandung

Bild 2.7 Verteilung des Schwindens in der Behälterwand
Shrinkage distribution in the vessel wall

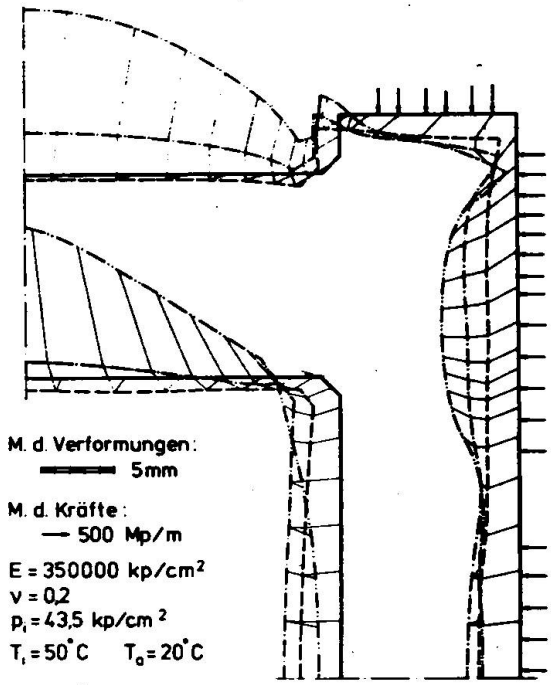


Bild 2.8 Einfluß des Betonkriechens auf die Behälterverformungen
Influence of concrete creep on vessel deformations

with high compressive stresses the limit may be exceeded, up to which linear-elastic concrete behaviour under short time load can be assumed. Computing problems are complicated due to variability and non-linearity of material behaviour, additionally to the requirement in calculating the three-dimensional state of stress and strain, which is a complex problem itself. For calculations like these, load history - i.e. the state of stress and strain imposed previously - is of great importance. Calculations assuming elastic material behaviour are no longer applicable on prestressed concrete reactor pressure vessels except for short time loadings.

Furthermore, the postulation to carry out an ultimate load calculation has to be mentioned here. For this analysis no methods are known from prestressed concrete structures, that could be transformed to the three-dimensional load carrying behaviour of the vessel.

Hence, dynamic relaxation was advanced in order to calculate three-dimensional structures with regard to inhomogeneous and non-linear material behaviour. Thereby, this method was found to be very efficient. In principle these calculations are reduced to step by step elastic calculations. Computations using an elasto-plastic material law were also performed already by non-linear equation systems, but this method is overcome by the solution with step by step elastic behaviour due to its expenditure. Dynamic relaxation is advantageous if used for stepwise elastic solutions, because during the iterative calculating process, that describes the formation of the state of stress and strain, the material law can be altered from one iteration step to the other.

The consideration of variability or non-linearity of material behaviour within the calculation is demonstrated by several examples.

2.4.1 Creep of concrete

Calculations are performed using the superposition method and direct methods as well. The great influence of creep can be seen in Fig. 2.8. Here, deformations of a prestressed concrete pressure vessel at beginning and end of vessel life are drawn up.

Another example is shown in Fig. 2.9, 2.1o and 2.11. This concerns a detail of a vessel wall. A small area is subjected to temperatures elevated up to 85° C . This temperatures possibly can be caused by local failure of thermal insulation. In Fig. 2.9, elastic stresses are plotted. But these stresses cannot develop, since temperature stresses decrease already to some extent during the arising of the temperature field. In Fig. 2.1o, the quasi-stationary temperatures stresses after creep can be seen. If the reactor is shut down for a time, temperatures in this previously hot zone cool down to the temperatures existing in the region around. Because temperature stresses were already decreased largely, this yields an inverted temperature load in the region regarded. As creep recovery is essentially less than creep during first temperature increase, this unloading causes a

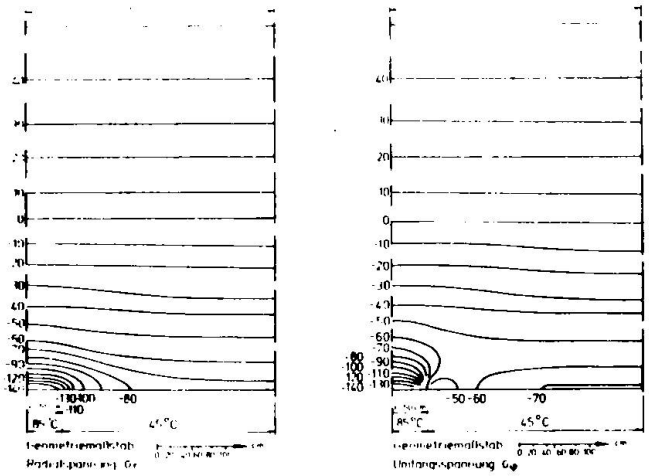


Bild 2 9 Heißer Bereich, elastische Temperaturspannungen
Hot spot, elastic temperature stresses

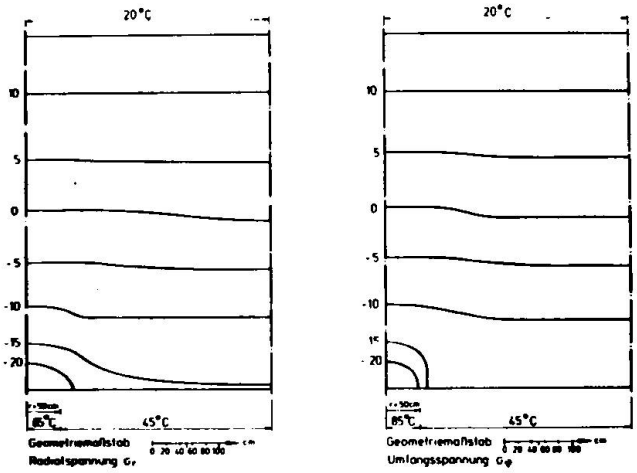


Bild 2 10 Heißer Bereich, Temperaturspannungen nach Kriechen
Hot spot, temperature stresses after creep

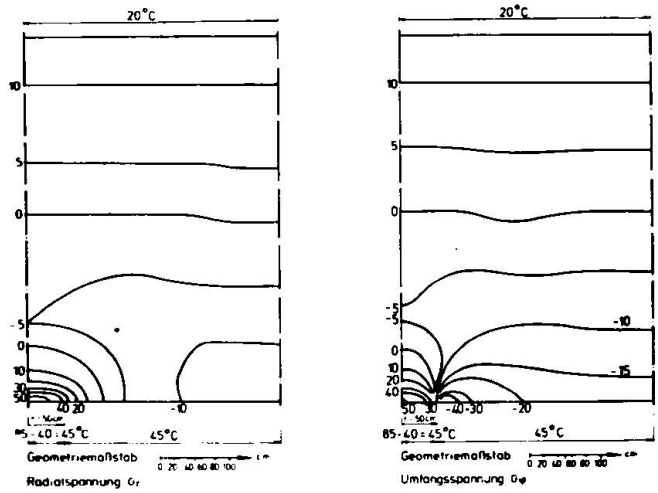


Bild 2 11 Heißer Bereich, Temperaturspannungen nach Kriechen und Abkühlung
Hot spot, temperature stresses after creep and cooling down

tension field , vis. Fig. 2.11. In the example shown here, these tensile stresses exceed tensile strength of concrete. That means, this region is in a cracked state.

2.4.2 Consideration of cracking

In order to estimate boundary strains and cracking at steel coated surfaces realistically, calculations with consideration of cracking are necessary. An economic design of bonded reinforcement can be gained only by calculations like these, too. Thus, procedures for taking account of crack development and reinforcement efficiency were developed based on dynamic relaxation using a fictive material law and assumption of single cracks as well. If distinct single cracks are considered, load carrying of bonded reinforcement is taken into account by special bars, where intermediate reactions act on the concrete as single, line, or volume forces according to respective conditions and bond assumptions. However, knowledge about bond is not yet so extensive as to be sufficient for an accurate calculation. In this field, experimental investigations are still necessary in order to derive laws for the calculation. That is why at present computations using a fictitious material law are preferred.

Both methods are generalized to such an extent that directions of crack propagation and direction of reinforcement can be arbitrarily chosen. Till now calculations were performed for the case of rotational symmetry. Fig. 2.12 shows an example of a calculation like this. It refers to the THTR vessel subjected to 1.6 operating pressure. The cracked regions are marked. Due to high circumferential prestressing of the vessel, cracks mainly occur in the r-z-plane at this pressure. Calculations for the general three-dimensional case are under work now.

2.4.3 Elasto-plastic calculation

As mentioned before, also in this case the iterative computation is carried out by use of stepwise constant elastic material coefficients, that are variable over the calculated region. Since the computation itself is similar to that one concerning cracking, it shall not be discussed in detail here.

2.4.4 Fracture analysis

Fracture analysis is possible by using integral description of the deformation behaviour of cracked zones, or by calculating structures with single cracks. Elasto-plastic behaviour of steel and concrete may be of importance, too. In this field, some research work is still necessary in order to get more detailed knowledge about material behaviour of reinforced concrete at different modes of fracture. With respect to computing technique, for example the dynamic relaxation is a suitable method in order to perform these calculations.

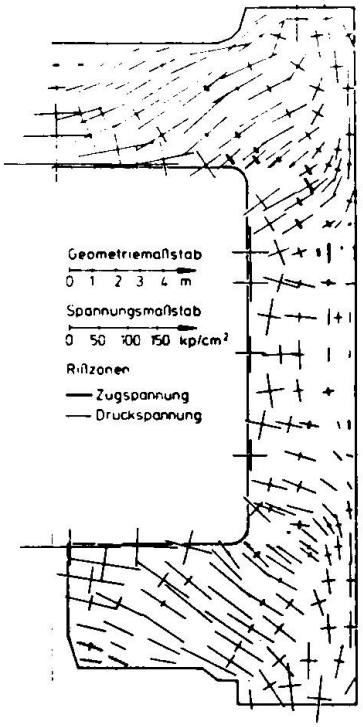
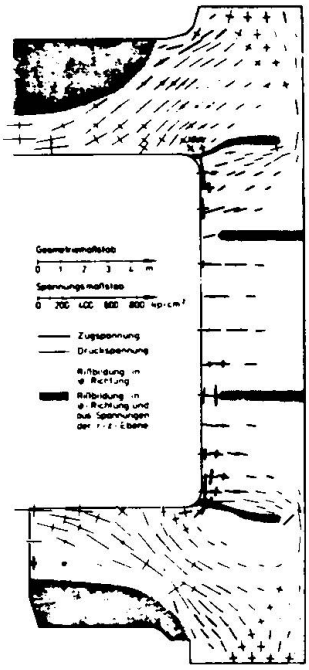


Bild 2.12 THTR-Behälter, Rißbildung bei 1,6-fachem Betriebsdruck
THTR vessel, crack propagation at 1.6 operating pressure



Hauptspannungen der r-z-Ebene bei $p = 2.8\,p_0$

Bild 2.13 THTR-Behälter, Nachweis des Grenzzustandes der Tragfähigkeit
THTR vessel, ultimate load analysis

Fig. 2.13 shows the THTR vessel subjected to 2.8 times operating pressure calculated in a still rather simplified way considering single cracks, dispersed crack regions, and concrete plasticity with presupposed laws for concrete behaviour and bond between concrete and steel.

How to improve ultimate load calculations is reported in the following section more in detail.

2.4.5 Interaction between the pressure vessel and its liner

The liner is connected to the surface of the concrete by studs, ribs etc. The relation between forces and deflections of these connexions is non-linear. The calculation of them has to take account of this non-linearity. Such calculations have been performed already for special cases to get the interaction between liner and concrete. Much effort is done in this field now. Moreover the buckling of the liner is considered by this research.

2.5 Further possibilities of calculation

The short description of the dynamic relaxation has shown the efficiency of this method with regard to the calculation of three- or two-dimensional structures. These calculations can be done with material laws, which are dependent on time, temperature, strain etc., including non-linear behaviour. As example the calculation of a prestressed concrete pressure vessel was described. The dynamic relaxation makes use of solving static problems by computing damped vibrations. It is obvious, that in the same way dynamic problems can be solved. With dynamic applications one would call this method the direct numeric integration of the equations of motion. The possibility of the direct integration was successfully used for calculating dynamic problems such as earthquake, or inpact forces for three- or twodimensional structures. This was done by taking into consideration cracking of concrete and elasto-plastic behaviour of the materials.

3. On calculations of ultimate load behaviour

3.1 Introduction

Ultimate load safety calculations for prestressed concrete pressure vessels for nuclear reactors (PCRV) up to this time always were performed using the method of kinematic rigid body mechanisms - in more or less refined mode, vis. [3.1] - and in most cases in connexion with large scale model tests. Since a long time it is desired to gain more generally valid methods herein, i.e. to formulate deformation and fracture processes in concrete structures when loaded up to the ultimate state using a general material law, which could be applied to three dimensional continua of any shape. Investigations in this field, that are done within the scope of the research work sponsored by the Bundesminister für Forschung und Technologie, shall be demonstrated by an example in this part of the report. The example presented here is chosen from the field of investigations on light water reactor vessels.

In the Danish AEC Research Establishment Risø, model tests for removable vessel top closures corresponding to the Nordic PCRV prototype were undertaken [3.2]. One of these models, called LM 3, and test results gained with it are object of the calculations mentioned here, which were performed to check material assumptions chosen. Fig. 3.1 shows the testing arrangement. The model acts as closure of a steel pressure vessel and is fastened by a supporting and sealing construction corresponding to the prototype. Special characteristics are:

- strong steel flange for supporting and sealing arrangement,
- bearing of the slab by 4o inclined struts,
- stiffening of the flange by 4o stiffening ribs,
- no bonded reinforcement.

3.2 Calculation fundamentals

The calculations were carried out on the basis of the dynamic relaxation method mentioned in section 2. The non-linear material behaviour of concrete and steel liner during increasing load was implied in the computing process as follows:

- Instead of continuous increase of load, single load steps were computed, here having distances of 4o kp/cm^2 . Refined scaling - in principle desirable - was renounced, because these were only testing calculations as a small part of more extensive parametric studies.
- Within each load level several computing steps are done, until changing of material behaviour has stabilized. For this, normally three computing steps were sufficient.

In each of these steps - from one load level to the next one as well as within the same load level - a new stress and deformation state is determined using the material

parameters of the previous step. According to this new state, material coefficients are changed if necessary in compliance with distinct criteria. Thus, non-linear material behaviour is simulated by a number of steps, each of them with linear material behaviour.

For description of concrete characteristics the following facts are mainly taken into account:
- plastification based on the von Mises criterion with a certain modification for better fitting on concrete behaviour gained from experiments,
- crack propagation (In the case of cracking, material turns from the isotropic into orthotropic state.),
- fracture due to multiaxial compression (This means complete collapse of internal structure in the region regarded.).

Any occuring of fracture is determined by the general failure criterion for concrete subjected to multiaxial load, vis. [3.3] e.g.

3.3 Example

Before results are discussed in detail, it should be mentioned that this report represents an instantaneous phase of current research work. That means that in some details investigations are still going on. But it can be said already that the present state seems not to be discouraging.

Two calculation procedures were carried out:
In the first one, the stiffening ribs were disregarded and the flange was assumed to have no bending stiffness, too. Hence the result will be conservative. In the second calculation the flange was set to be completely rigid in vertical direction due to the ribs. This is a more realistic, but somewhat optimistic approach.

First some results of the case without stiffening ribs shall be pointed out. Fig. 3.2 shows the propagation of tensile cracks with tangentially running crack areas. Cracked regions in dependence of pressure and crack directions are outlined. As to be seen in this figure, ample cracking starts in the central region of the upper side of the slab, while above the supporting zone only narrow cracks can be observed. Around 16o kp/cm^2 crack propagation decreases. But now, starting approximately in the middle between centerline and outer surface, inclined cracks propagate. In connexion herewith tangential vertical cracks arise above this zone at the upper surface at pressures about 2oo to 24o kp/cm^2. In the model test cracks like these were observed at circ. 25o kp/cm^2 - a remarkable accordance. In the following, cleavage fractures occur in the whole upper region of the slab, while downwards at half of slab radius a distinct weakening of the uncracked area is taking place. At this position ultimate failure occurs in the test at 37o kp/cm^2 . Up

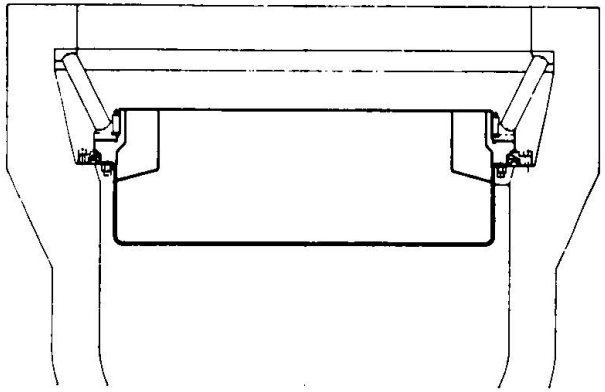


Bild 3.1 Versuchsanordnung
Test arrangement

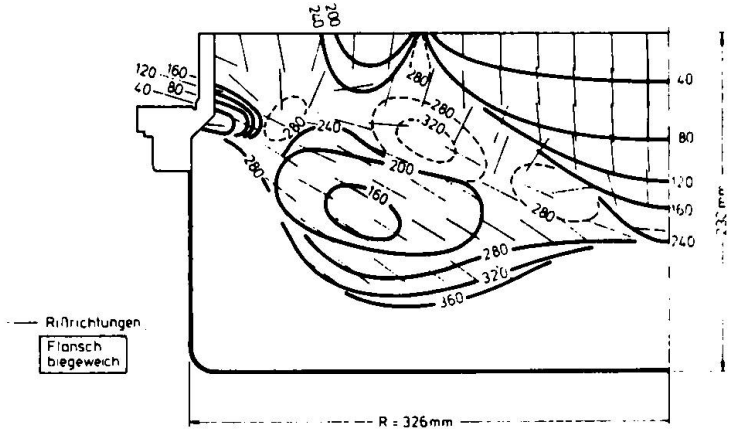


Bild 3.2 Bereiche mit Tangentialrissen bis p = 360 kp/cm²
Crack patterns, tangential cracks up to p = 360 kp/cm²

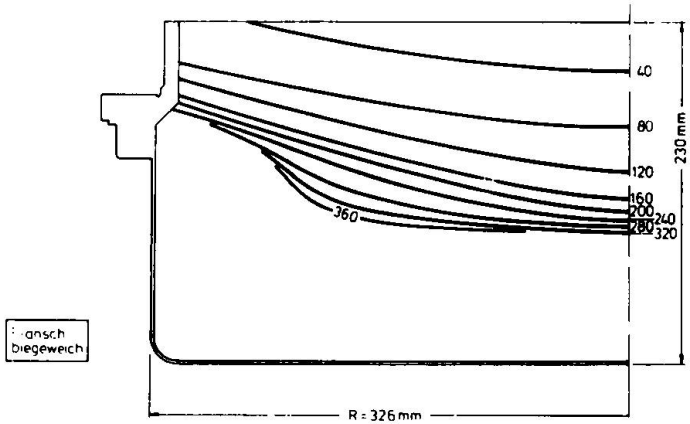


Bild 3.3 Bereiche mit Radialrissen
Crack patterns, radial cracks

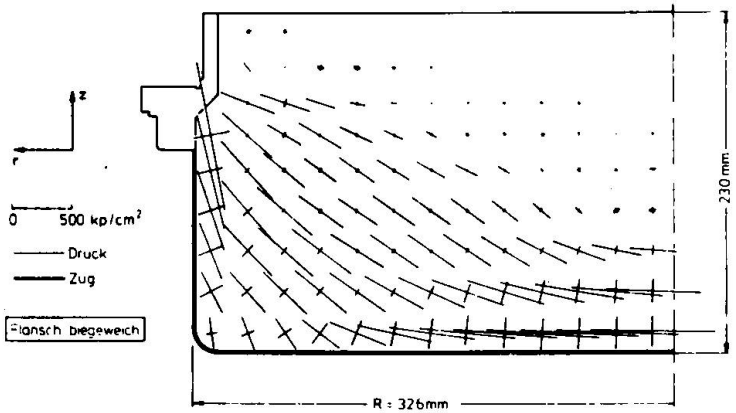


Bild 3.4 Hauptspannungen in der r-z-Ebene bei p = 280 kp/cm²
Principal stresses in the r-z-plane at p = 280 kp/cm²

to now it has not been possible to simulate this final phase in the calculation satisfying, but efforts on this subject will go on.

Fig. 3.3 shows the patterns for radially running cracks. It can be seen, that already at 8o kp/cm^2 pressure the whole upper surface of the closure is cracked. But at higher pressures expansion of crack regions decreases. This shows also, that no bending failure can be expected here. Rather an unmistakable pressure dome developes. This fact can also be recognized in Fig. 3.4, where principal stresses in the r-z-plane at a pressure of 28o kp/cm^2 as an example are plotted. The whole upper region does in fact not participate in the load carrying function of the slab.

The following figures show corresponding drawings for the second calculation with stiffened flange. In Fig. 3.5 it can be noticed, that tangentially cracked regions in the slab center remain smaller than in the former example. Here however, the inclined crackes continuously run forward from the supporting zone into the midst of the closure, until at 32o kp/cm^2 the two crack regions are joining. In this case too, ab about half the outer radius, the weakest point developes at which finally ultimate failure occurs. There is no fundamental difference in radial cracking between both calculations (Fig. 3.6), merely the crack regions are somewhat smaller due to the stiffer system. Fig. 3.7 outlines the formation of a pressure dome in this example too. (The remaining principal tensile stresses result from stress redistributions after cracking, because in tensile cracks, if not too wide opened, shear forces still can be transfered. Secundary effects like these will be further investigated.)

In the following some deformations shall be pointed out as a comparison between calculations and model testing. First of all the vertical displacement at slab center (Fig. 3.8): The dash-and-dot lines describe the results of the calculation with weak flange, the continuous lines of that one with stiffened flange respectively. Both curves fit the results of measurement quite well. As expected, the first calculation is somewhat conservative, the second one unconservative. Caused by a comparatively rough computing mesh and rough load step subdivision the resulting curves are not always exactly continuous. This could be compensated by suitable refinements.

Further on, comparisons with measured concrete strains shall be presented. Fig. 3.9 represents strain in the apex of the pressure dome. There is a good accordance of test results with the calculation of the weaker system, while fitting of the other calculated curve is not as good. Nevertheless also this one is rather satisfying, particularly since there are several influences on measurement such as load cycles and time effects

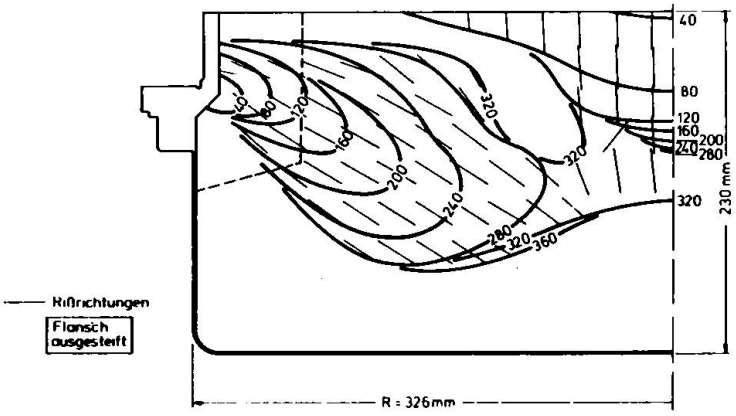


Bild 3.5 Bereiche mit Tangentialrissen bis $p = 360\ kp/cm^2$
Crack patterns, tangential cracks up to $p = 360\ kp/cm^2$

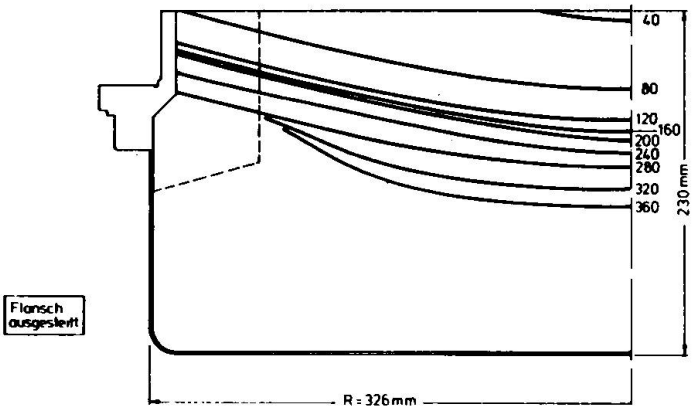


Bild 3.6 Bereiche mit Radialrissen
Crack patterns, radial cracks

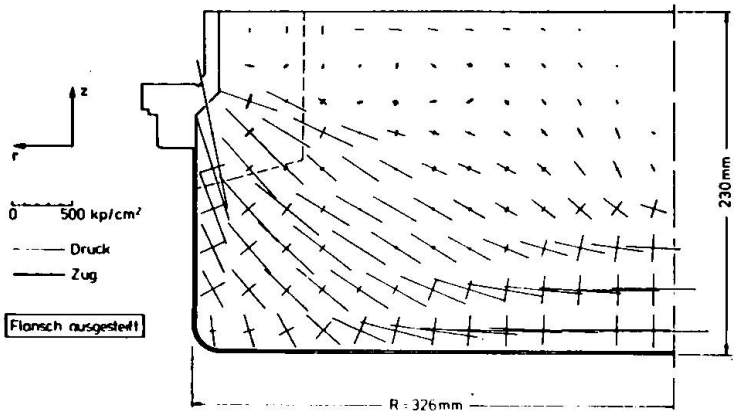


Bild 3.7 Hauptspannungen in der r-z-Ebene bei $p = 280\ kp/cm^2$
Principal stresses in the r-z-plane at $p = 280\ kp/cm^2$

that can modify test results. A confrontation of measurement and computation in the region of neutral fiber at the centerline (Fig. 3.1o) shows basically similar behaviour. Quantitative differences may be caused by the mentioned roughness of calculation, but also by local material conditions and consequently doubtful statement of testing. However trying to get much better accordance in details like these is not so necessary in the author's opinion. Finally in Fig. 3.11 tangential strains at about half radius and about midheight of the closure are compared. Also here qualitative accordance can be recognized. Whether reasons for quantitative differences are more in calculation or more in model testing again cannot be estimated.

3.4 Conclusion

All things considered, this comparison shows that there seems to be a good chance to simulate in the way described non-linear behaviour of concrete in three demensional continua of any shape when loaded up to failure. As mentioned earlier, there are still some minor problems to be solved. But in a measurable space of time, surely it will be possible to perform ultimate load investigations for PCRV by theoretical methods only.

References

/1.1/ W. ZERNA, G. SCHNELLENBACH: Probleme des Stahlbetons und Spannbetons bei Reaktorgebäuden für Kernkraftwerke, Vortrag auf dem Betontag 1971 in Berlin

/1.2/ W. ZERNA, G. SCHNELLENBACH, S. WRAGE: Konstruktive und ausführungstechnische Probleme bei Spannbeton-Reaktordruckbehältern, 2nd International Conference on Structural Mechanics in Reactor Technology, Berlin, 1973, paper H 3/1

/2.1/ G. SCHNELLENBACH: Beitrag zur numerischen Berechnung des räumlichen Spannungszustandes in Hohlzylindern mit Ausschnitten, Dissertation, Bochum, 1969

/2.2/ V. HANSSON: Zur praktischen Berechnung von Scheiben mit beliebiger Geometrie und allgemeinem Materialgesetz, Konstruktiver Ingenieurbau - Berichte, Heft 14, Vulkan-Verlag Essen

/2.3/ G. SCHNELLENBACH: Allgemeine Berechnung von Spannbeton-Reaktordruckbehältern unter Berücksichtigung von nichtlinearen Spannungs-Dehnungsgesetzen nach der Methode der Dynamischen Relaxation, First International Conference on Structural Mechanics in Reactor Technology, Berlin, 1973, paper H 3/7

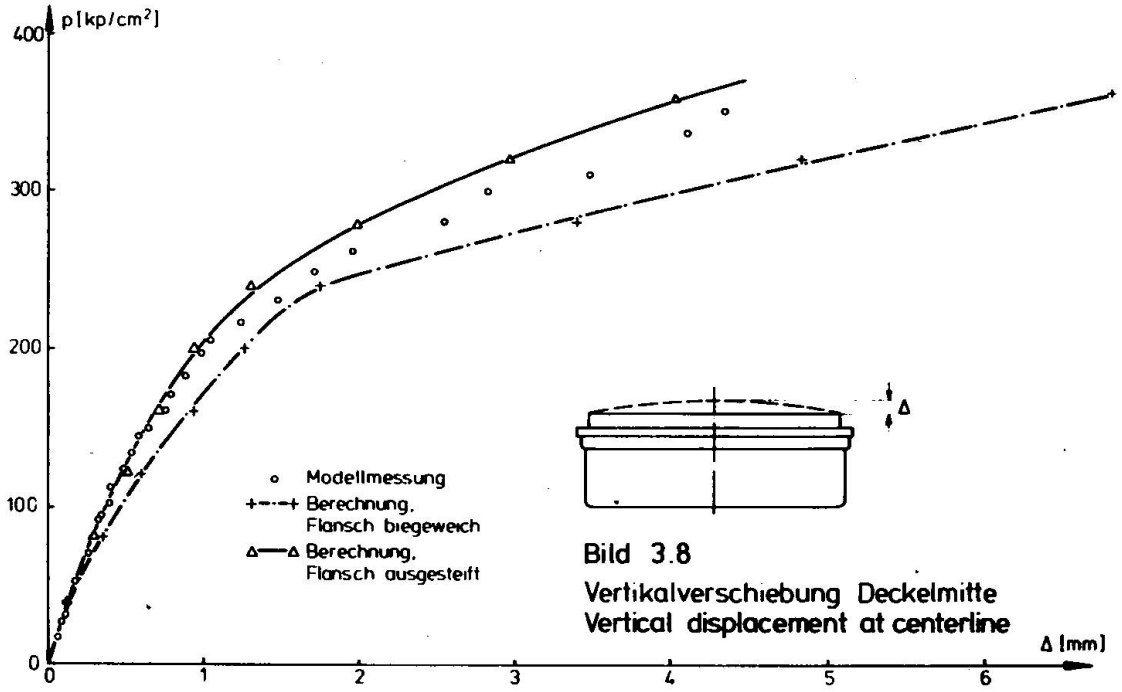


Bild 3.8
Vertikalverschiebung Deckelmitte
Vertical displacement at centerline

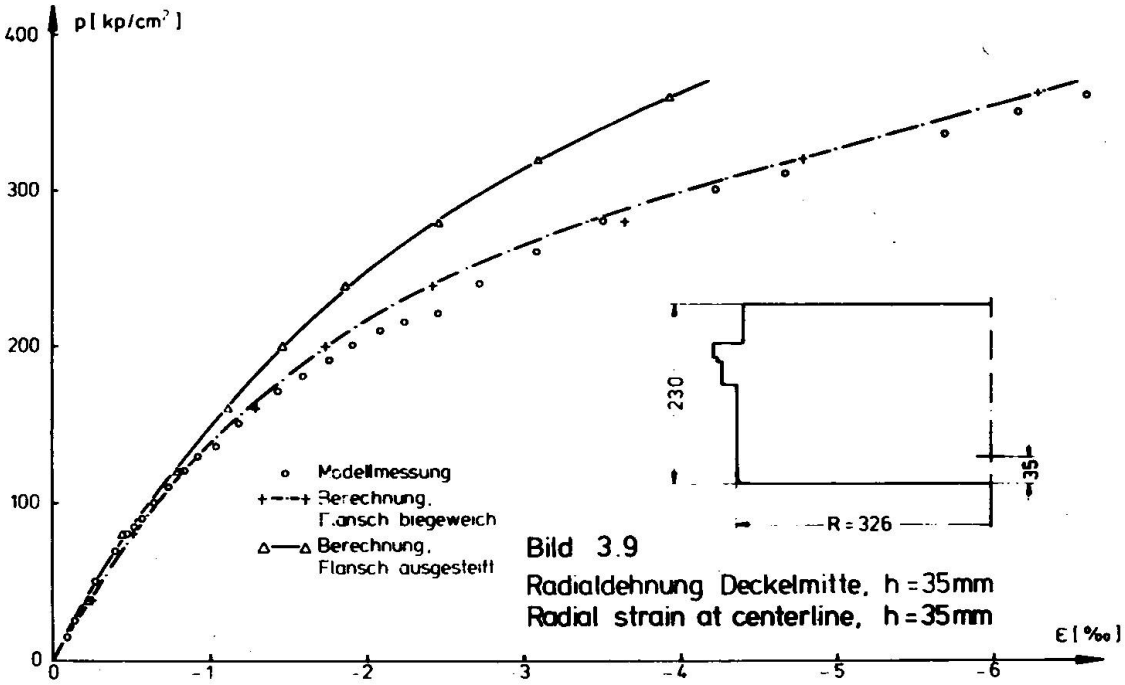


Bild 3.9
Radialdehnung Deckelmitte, h = 35mm
Radial strain at centerline, h = 35mm

/2.4/ G. SCHNELLENBACH: Statische Nachweise für Spannbeton-Druckbehälter im Gebrauchszustand, Konstruktiver Ingenieurbau - Berichte, Heft 1o, Vulkan-Verlag Essen

/2.5/ H. SCHULTZ: Beitrag zur Berechnung von Temperaturverteilung und Temperaturspannungen bei Reaktordruckbehältern, Technisch-wissenschaftliche Mitteilungen des Instituts für Konstruktiven Ingenieurbau, Ruhr-Universität Bochum, Mitteilung Nr. 72-2

/2.6/ V. HANSSON: Die dreidimensionalen elastischen Spannungszustände im Bereich der Öffnungen des THTR-Spannbetonbehälters - Ein Vergleich zwischen numerischer Berechnung und Meßergebnissen eines elastischen Modells, First International Conference on Structural Mechanics in Reactor Technology, Berlin, 1971, paper H 5/4

/2.7/ R. STÖVER: Modellstatische Untersuchung der Spannungen und Verformungen des Modells eines Pod-Boiler-Spannbeton-Reaktordruckbehälters, Bericht Nr. 1o/1 der Forschungsgruppe Reaktordruckbehälter, Institut für Konstruktiven Ingenieurbau, Ruhr-Universität Bochum, 1973

/2.8/ V. HANSSON: Werkstoffannahmen zum Kriechen und Schwinden des Betons in Spannbeton-Reaktordruckbehältern, Bericht Nr. 3 der Forschungsgruppe Reaktordruckbehälter, Institut für Konstruktiven Ingenieurbau, Ruhr-Universität Bochum, 197o

/2.9/ V. HANSSON, K. SCHIMMELPFENNIG: Das Verhalten der Werkstoffe Stahl und Beton in Spannbeton-Druckbehältern, Ermittlung der Werkstoffeigenschaften und Rechnungsannahmen, Konstruktiver Ingenieurbau - Berichte, Heft 1o, Vulkan-Verlag Essen

/2.1o/ W. ZERNA, G. SCHNELLENBACH: Zur Berechnung von Spannbeton-Reaktordruckbehältern, Beton- und Stahlbetonbau, 65. Jahrgang, Heft 11/197o

/2.11/ F. STANGENBERG: Berechnung des 1:5-THTR-Druckbehältermodells und Vergleich mit Meßergebnissen des Modellversuchs, Bericht Nr. 9 der Forschungsgruppe Reaktordruckbehälter, Institut für Konstruktiven Ingenieurbau, Ruhr-Universität Bochum, 1972

/2.12/ V. HANSSON, G. SCHNELLENBACH: Spezielle Probleme bei der Berechnung von Spannbeton-Reaktordruckbehältern unter Berücksichtigung instationärer Temperaturfelder und nicht-linearen Materialverhaltens, 2nd International Conference on Structural Mechanics in Reactor Technology, Berlin, 1973, paper H 1/3

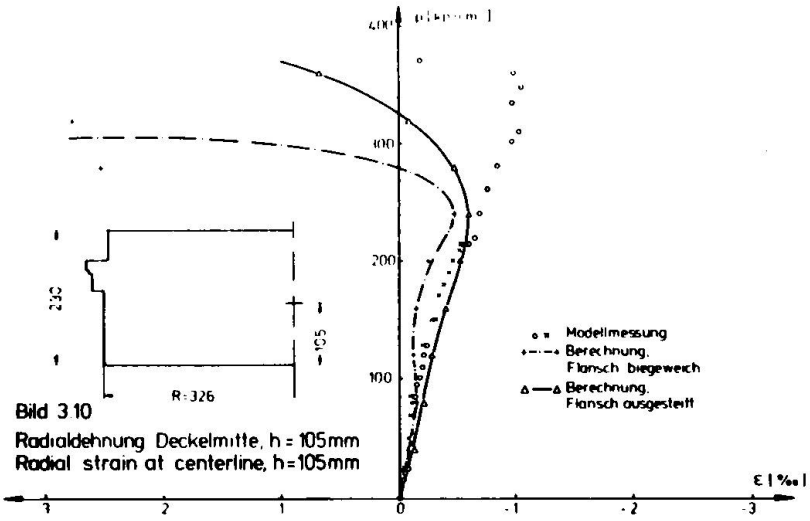


Bild 3.10
Radialdehnung Deckelmitte, h = 105mm
Radial strain at centerline, h=105mm

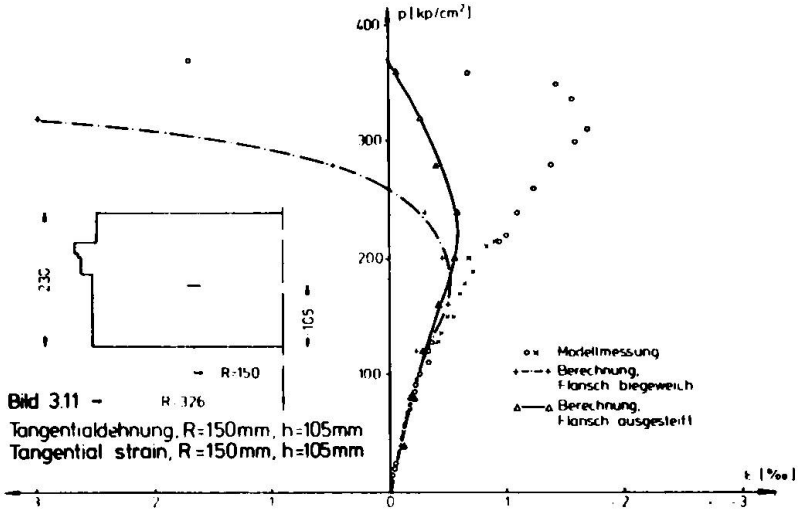


Bild 3.11 –
Tangentialdehnung, R=150mm, h=105mm
Tangential strain, R=150mm, h=105mm

/3.1/ K. SCHIMMELPFENNIG, V. HANSSON: Untersuchungen zur Grenztragfähigkeit von Spannbeton-Reaktordruckbehältern, 2nd International Conference on Structural Mechanics in Reactor Technology, Berlin, 1973, paper H 2/3

/3.2/ S.I. ANDERSEN, N.S. OTTOSEN: Ultimate Load Behaviour of PCRV Top Closures, Theoretical and Experimental Investigations, 2nd International Conference on Structural Mechanics in Reactor Technology, Berlin, 1973, paper H 4/3

/3.3/ K. SCHIMMELPFENNIG: Die Festigkeit des Betons bei mehraxialer Belastung, Bericht Nr. 5 der Forschungsgruppe Reaktordruckbehälter, Institut für Konstruktiven Ingenieurbau, Ruhr-Universität Bochum, 1971

Résumé

Le calcul et la construction de structures en béton soumises à des contraintes triaxiales on eu un progrès significatif par les caissons en béton précontraint pour réacteurs nucléaires. Ce rapport veut donner un panorama du travail de recherche dans ce domaine, soit du point de vue du calcul que de la construction. On a considéré différentes techniques de précompression du caisson, concentrations locales de contraintes et les requises conséquentes pour le projet et la réalisation, problèmes qui concernent la "peau" d' étanchéité, la compositions et le coulage du béton. On traite la calcul tridimensionel du caisson, avec considération particulière pour la méthode de la relaxation dinamique. En particulier, on souligne comment tenir compte de: conditions géométriques complexes; variations en fonction de l' espace et du temps et non-linéarité des propriétés des matériaux; influence des variations de température.

Summary

Calculation and construction of concrete structures subjected to triaxial stresses was significantly advanced by erecting prestressed concrete pressure vessels for nuclear power plants. This report shall give a survey of research work in this field, that includes problems of calculation and problems of construction as well. Different vessel prestressing techniques, local stresses and resulting requirements on design and manufacture, problems concerning the liner, as well as concrete mix and concrete casting demands are discussed. Three-dimensional calculation of prestressed concrete pressure vessels is reported with particular consideration of the dynamic relaxation method. Especially it is pointed out how to take account of complicated geometry conditions, of space and time dependent variations and non-linearity of material properties and of the influence of variable temperatures. Finally, the present state of investigations on ultimate load behaviour of concrete structures subjected to triaxial stresses is outlined. For this, ultimate load calculation results of removable closures for light-water reactor vessels are presented.

Übersicht

Die Berechnung und Konstruktion von Betonbauwerken mit dreidimensionaler Beanspruchung erhielt mit dem Bau von Spannbeton-Druckbehältern für Kernkraftwerke wesentliche Impulse. Dieser Beitrag soll über Erfahrungen und Forschungsarbeiten auf diesem Gebiet, die sowohl Probleme der Berechnung als auch der Konstruktion betreffen, einen Überblick geben. Behandelt werden Möglichkeiten der Behältervorspannung, örtliche Störungszonen und ihre Anforderungen an Konstruktion und Ausführung, Probleme im Zusammenhang mit dem Liner sowie Fragen der Betonzusammensetzung und des Betoniervorganges. Über die dreidimensionale Berechnung von Spannbetonbehältern wird am Beispiel des Verfahrens der Dynamischen Relaxation berichtet. Insbesondere werden Möglichkeiten der Berücksichtigung geometrischer Besonderheiten, des örtlich und zeitlich veränderlichen sowie des nichtlinearen Materialverhaltens und des Einflusses unterschiedlicher Temperaturen aufgezeigt. Schließlich wird über den derzeitigen Stand von Untersuchungen zum Bruchverhalten dreidimensional beanspruchter Betonkonstruktionen gesprochen. Dazu werden Ergebnisse von Bruchberechnungen beweglicher Deckel für Leichtwasser-Reaktordruckbehälter vorgestellt.

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SEMINAR on:
« CONCRETE STRUCTURES SUBJECTED TO TRIAXIAL STRESSES »
17th - 19th MAY, 1974 - ISMES - BERGAMO (ITALY).

The U.K. Experience in the Analysis and Design of Triaxially Stressed Concrete Structures Supported by Model Investigations with Particular Reference to Prestressed Concrete Pressure Vessels

Le support des essais sur maquettes dans l'analyse et le projet de structures en béton chargées triaxialement, avec référence en particulier aux caissons pour réacteurs nucléaires. Expériences réalisées dans le Royaume-Uni

Die Erfahrungen im U.K. in 'Analyse und Design von dreiachsigen Spannbeton-Konstruktionen unterstützt durch Modelluntersuchungen unter besonderer Berücksichtigung von Spannbeton-Druckbehältern

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F.K. GARAS, Ph.d., B.sc., M.I.C.E., Head of Structures Research Laboratory
Taylor Woodrow Construction Limited, Southall, Middlesex U.K.

1. INTRODUCTION

Most concrete components and structures are in a state of triaxial stress in one form or other although, for practical design, gross simplifications have to be made:

(i) to reduce design time and cost

(ii) since a knowledge of triaxial behaviour is not generally available to designers

(iii) since appropriate methods are not readily available

(iv) since the external forces cannot always be defined with precision

However, for complex structures, such as dams, pressure vessels, tunnels, offshore oil drilling platforms, a greater understanding of the triaxial behaviour is required.

Over the last fifteen years in the U.K. extensive research and development has been carried out on material properties, methods of analysis and model testing for such structures. In the case of prestressed concrete pressure vessels for nuclear power stations, the stringent demand for reliable information as to the behaviour of these critical structures both as regards their safety and their operation performance has required exhaustive investigations on many fronts. Aspects covered include material behaviour throughout the operating life of 30 years, overall and localised behaviour of the vessel by model testing, and full instrumentation and observation of the final structures during proof pressure testing, commissioning and operation. The authors have been closely involved in the design, development and construction of a number of vessels both in the U.K. and overseas.

In attempting to cover for the U.K. the major structural situations where triaxial stress states are significant, the authors realise that the items presented may not do justice to the overall effort by universities, government and commercial establishments in the various fields. In the paper only selected aspects have been included to demonstrate the width of progress.

This has involved examining the following main areas of activity:

(i) material research

(ii) methods of analysis

(iii) model and component testing

(iv) observation of the structures' behaviour

(v) current design methods

All these factors should be assessed in order to simplify or improve design methods or to enable more advanced structures to be built with confidence. Further, each of the above factors has to deal with two main loading conditions, the operating condition and the overloaded state.

In both categories an understanding of any triaxial stress situation may require a knowledge of the following parameters:

(i) short term elastic/plastic behaviour

(ii) long term, creep and shrinkage performance

(iii) response to dynamic, cyclic and random loads

(iv) the behaviour at ambient or elevated temperature

The following "matrix" chart encompassing the various components of the design process, has provided a background to the following sections covered in the paper: materials research, confined concrete components, pressure vessels, dams, tunnels, etc.

Materials Analysis Models Field Data Design	Short/Long Term Loading	Static/Dynamic Loading	Operational/ Overload States	Normal/Elevated Temperature

DESIGN DEVELOPMENT 'MATRIX'

2. FUNDAMENTAL RESEARCH

2.1. Triaxial Loading

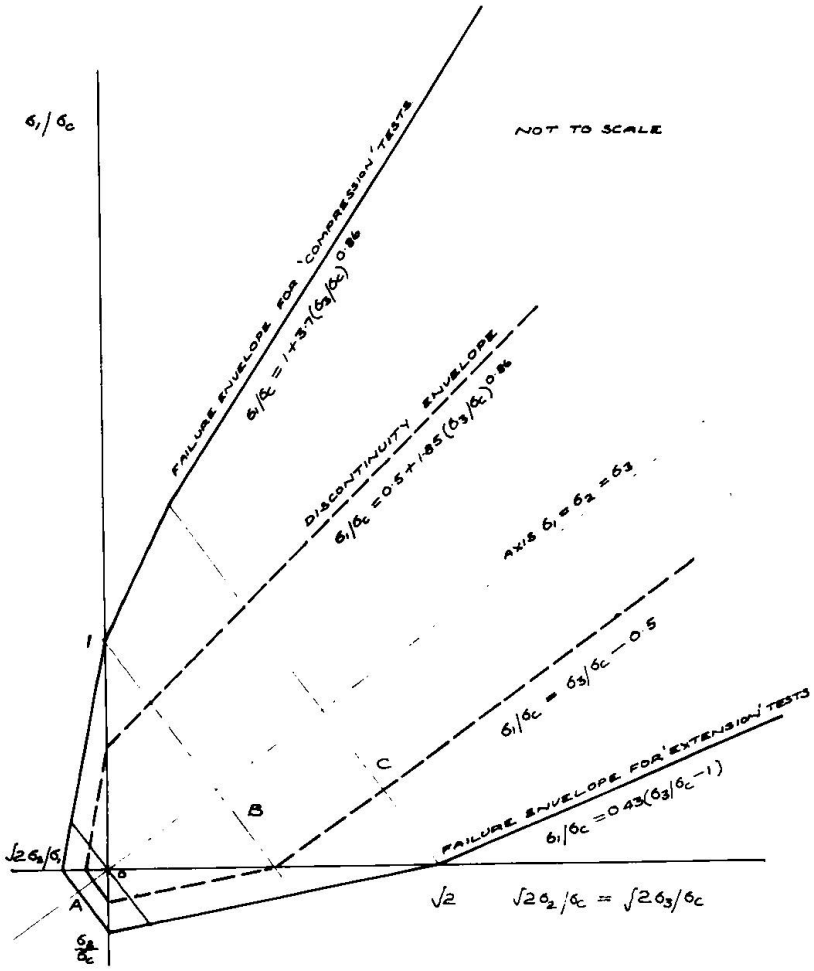
The stress/strain behaviour of concrete up to failure under short term loading has been extensively investigated by Newman and Newman(1 & 2) primarily using biaxial compression/tension and hydrostatically contained triaxial specimens. The experimental programme covered mortar, artificial, and natural aggregate concretes.

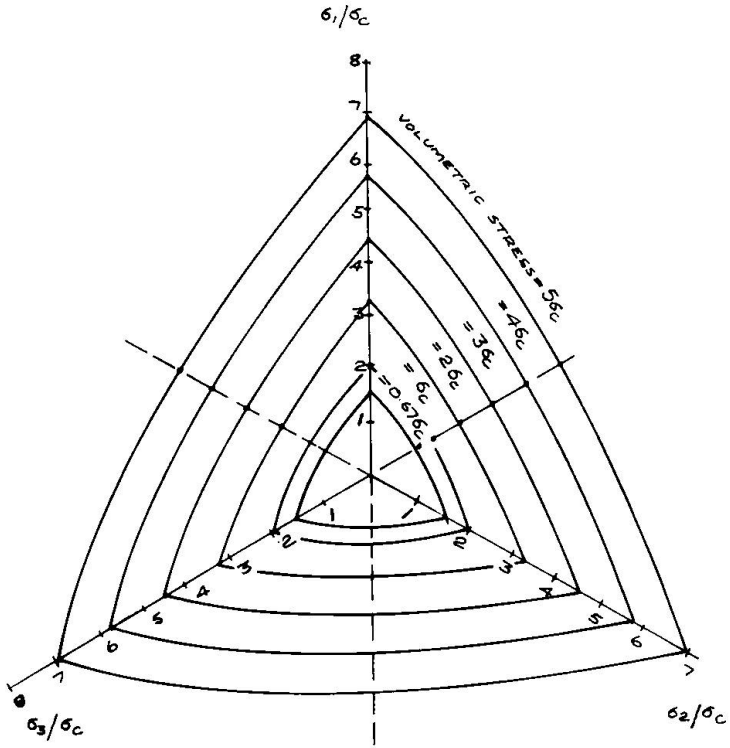
This research could provide valuable basic data for analysis programmes involving non-linear triaxial behaviour to failure.

From their work, together with comprehensive analysis of the results of other researchers, a triaxial compression/tension failure representation (Fig. 1) was produced initially including the level of onset of major internal dislocation of the

FIG. 2

ULTIMATE AND DISCONTINUITY FAILURE SURFACES FOR CONCRETE (NEWMAN & NEWMAN)





POSSIBLE DESIGN CHART FOR ULTIMATE STRENGTH FAILURE CONCRETE IN THE TRIAXIAL COMPRESSION REGION (NEWMAN & NEWMAN)

material, the discontinuity level, which was considered also to be of importance to the long term sustained and cyclic loading stability of the material. Methods of utilising the triaxial compression/tensile performance for design have been developed (Fig. 2.)

Another comprehensive analysis of published data on the triaxial strength of concrete has recently been reported by Hannant(3) from which he derived simplified design nomograms for handling combinations of multiaxial stress to predict failure.

Perhaps the above work highlights the care needed in utilising the triaxial strength of concrete where one stress is relatively low, and thus high biaxial states occur. The strength gain over the uniaxial compression value is both marginal and uncertain. This may limit its application to that in which low deviatoric stress states exist.

Shear

The above work has not considered the behaviour of concrete in shear under multiaxial stress states, of importance particularly in concrete pressure vessel design. Intensive experimental work by Garas and Langan(4 & 5) has shown that deep sections of various geometric and restraint states (Fig. 3) give ultimate shear strengths several times greater than for normal, low restraint, shear conditions (Table 1 and Fig. 8b).

TABLE 1 Nominal Shear Stresses in Beams and Slabs as a Function of Compressive Cylinder Strength of Concrete (f_c)

Nominal Shear Stress at Failure = $K \times \sqrt{f_c}$ (f_c in N/mm^2)

Type of Element	K
1. Beams a. Shallow unrestrained	0.17 - 0.33
b. Deep restrained (span/depth = 2.5)	1.33 - 1.58
2. Slabs a. Shallow unrestrained	0.50 - 0.75
b. Deep restrained (span/depth = 2 & 2.5)	1.50 - 2.00
3. Small Discs (span/depth = 1 to 3)	
a. Unrestrained	0.83 - 1.67
b. Restrained	2.25 - 5.17
4. Anchor Block Models	4.42 - 7.42

Even in tensile zones, aggregate interlock was believed to contribute to the ultimate shear strength although large deformations along cracks were recorded.

2.2. Creep

Sustained long term loading effects on concrete deformation behaviour, covering the working stress levels used in structures, have been extensively investigated particularly in relation to concrete maturity for normal and elevated temperature conditions by Browne and Blundell(6). Their experimental work was confined to uniaxially loaded concrete since published data available had demonstrated that generally Poisson's ratio could be regarded as constant under multiaxial sustained load and within the range of 0.15 - 0.23 for different states of stress including elevated temperature (Fig. 4).

FIG. 3

DIFFERENT STRUCTURAL FORMS SUBJECTED TO BIAXIAL COMPRESSION AND SHEAR (GARAS & LANGAN)

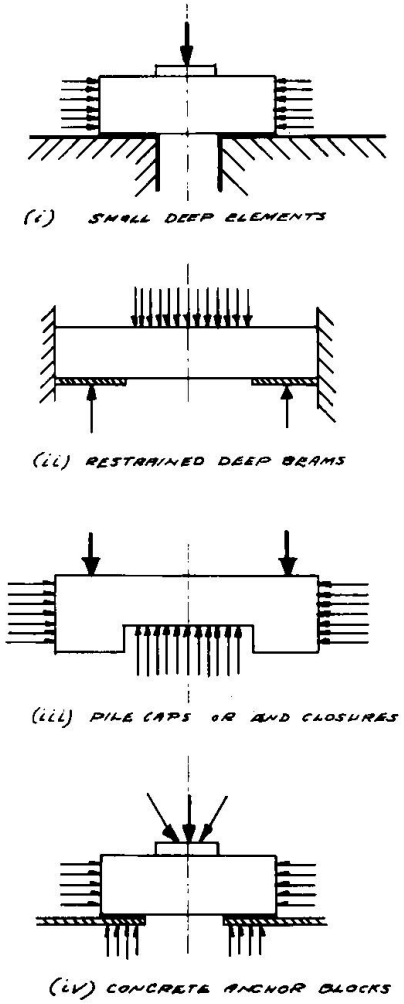
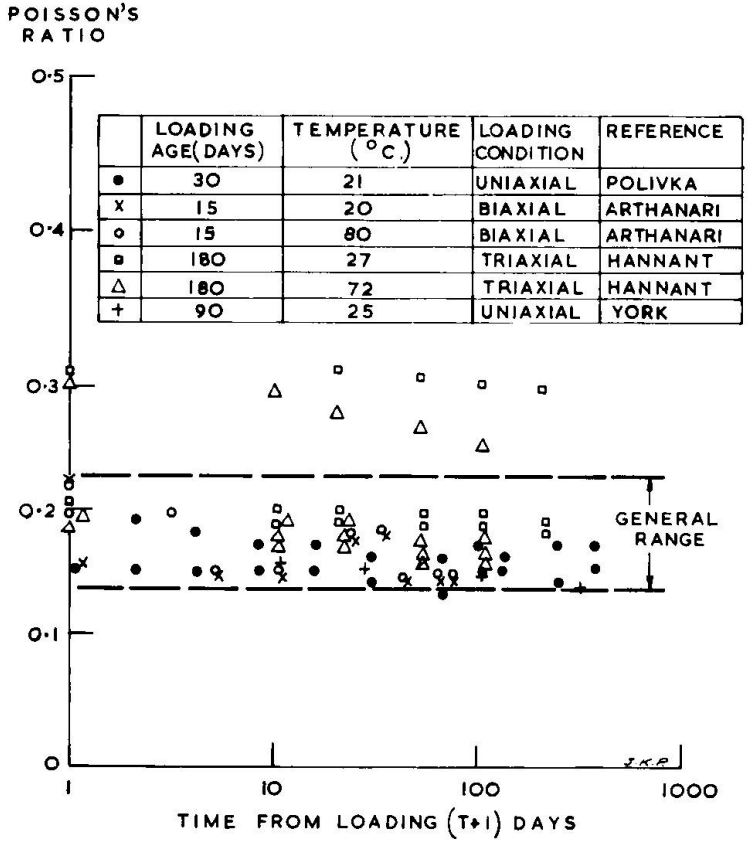


FIG. 4

POISSON'S RATIO FOR SEALED CONCRETE UNDER UNIAXIAL AND MULTIAXIAL SUSTAINED LOADING CONDITIONS.

(BROWNE & BLUNDELL)



	LOADING AGE (DAYS)	TEMPERATURE (°C.)	LOADING CONDITION	REFERENCE
●	30	21	UNIAXIAL	POLIVKA
x	15	20	BIAXIAL	ARTHANARI
o	15	80	BIAXIAL	ARTHANARI
▫	180	27	TRIAXIAL	HANNANT
△	180	72	TRIAXIAL	HANNANT
+	90	25	UNIAXIAL	YORK

They also showed from long term tests (Fig. 5) the close similarity of creep for different concretes, provided the elastic modulus of the aggregate exceeds 10×10^6 psi (7×10^4 N/mm^2). This enables the designer to use one set of standard creep data for such concretes.

Recently Illston and Jordaan(7) have confirmed the similarity of creep Poisson's ratio to the elastic value for uniaxial, biaxial and triaxial stress states (Fig. 6) and have developed a method of handling elastic and creep deformation under changing multiaxial stress states with time. Illston and Sanders(8) have also investigated the effect of temperature change under sustained load for mortar using torsionally loaded specimens. This work showed that temperature change induces only an initial increase in the creep rate, although the maximum period of loading reported was 3 months only.

The effect of cyclic loading (585 cycles/min) on creep and delayed failure has been investigated by Whaley and Neville(9) showing that cycling can increase the non-elastic time dependant deformation of concrete.

2.3. Analysis

The exhaustive development of analysis methods based on the finite differences, finite element and dynamic relaxation techniques have provided useful tools to handle the elastic behaviour of 3-D stressed structures covered in this paper. Increasingly larger problems have been analysed facilitated by the introduction of substructuring techniques and the ready availability of finite element packages on a bureau usage basis using large core computers.

Current trends in analysis methods have been towards non-linear applications to deal with both geometries (large deflection, buckling) and material properties(10) (plasticity, creep, crack propogation etc.) Lewis and Irving(11) and England(12) have developed methods of analysis to predict time dependant behaviour incorporating temperature, for concrete pressure vessels. However, the authors are not aware of the extent to which these methods have been applied to vessel design, the effective modulus approach still being generally in use.

3. CONFINED CONCRETE COMPONENTS

Concrete Filled Tubes

Extensive work by Sen, Chapman and Neogi(13 - 15) on testing and developing design methods for concrete filled steel tubes (Fig. 7a) with axial and eccentric loads, showed that with height/diameter ratios over 15, the effect of biaxial passive restraint provided by the steel tube was negligible due to the dominating effect of buckling. However, tests on concentrically loaded concrete filled stubb columns showed that for diameter/tube wall ratios from 17 to 37, only after the longitudinal stress of the steel has approached its compressive yield stress does the restraining effect on the concrete become operative (Fig. 8a). The strength of the concrete was nearly 2 times its uniaxial value and the column load was up to 1.6 times the sum of the uniaxial compressive strength of the steel and concrete.

Utilisation of this passive restraint technique of concrete in tubes so far in normal construction appears to be limited, particularly since in buildings, problems such as end connection design, have not as yet been resolved nor the performance of such columns under fire. However, applications could include bridge support columns and piles and limited information available suggests that this appears to be so.

FIG. 5

THE EFFECT OF AGGREGATE MODULUS OF ELASTICITY UPON CREEP OF CONCRETE. (BROWNE & BLUNDELL)

CREEP ASSUMED AS UNITY FOR $E_{AGG} = 0{\cdot}69 \times 10^5$ N/mm².

COARSE AGGREGATE CONTENT = 45 TO 55 % BY VOLUME.

$$\text{RELATIVE CREEP} = \frac{\text{CREEP}}{\text{CREEP AT } E_{AGG} = 0{\cdot}69 \times 10^5 \text{ N/mm}^2}$$

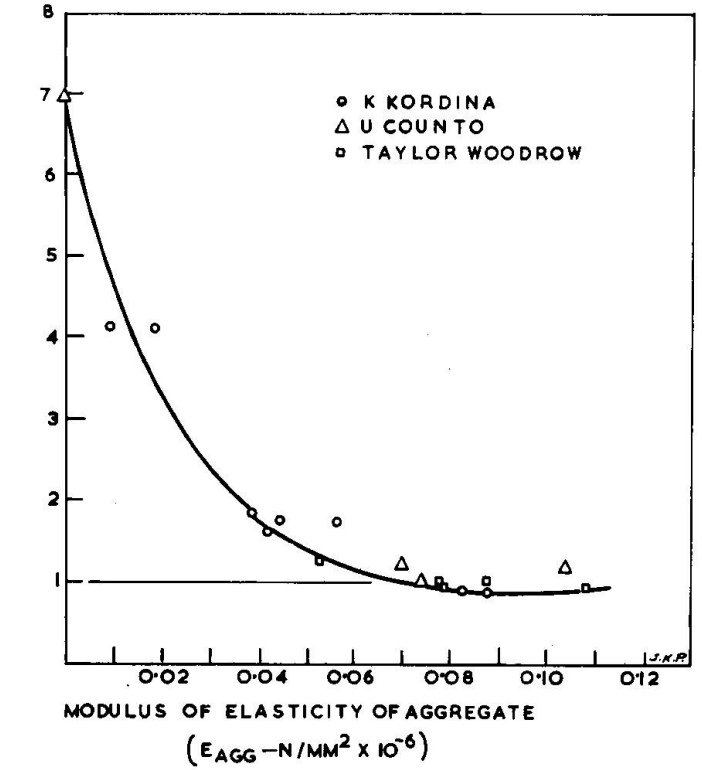
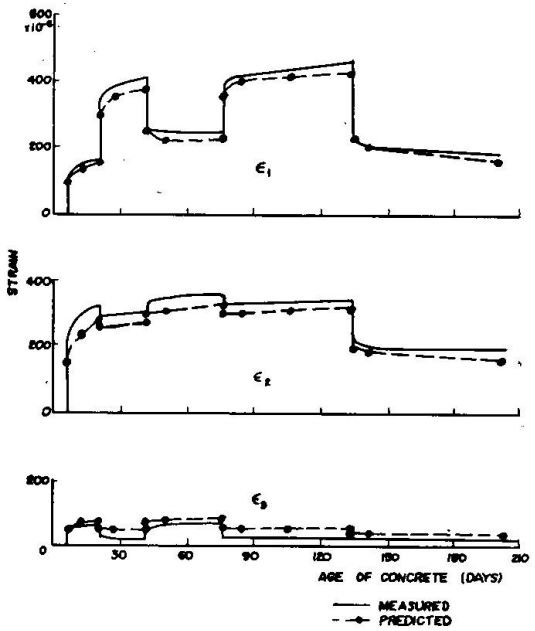


FIG. 6

PREDICTION OF STRAINS DUE TO A VARIABLE TRIAXIAL STRESS SYSTEM (ILLSTON AND JORDAAN)



CONFINED CONCRETE COMPONENTS.

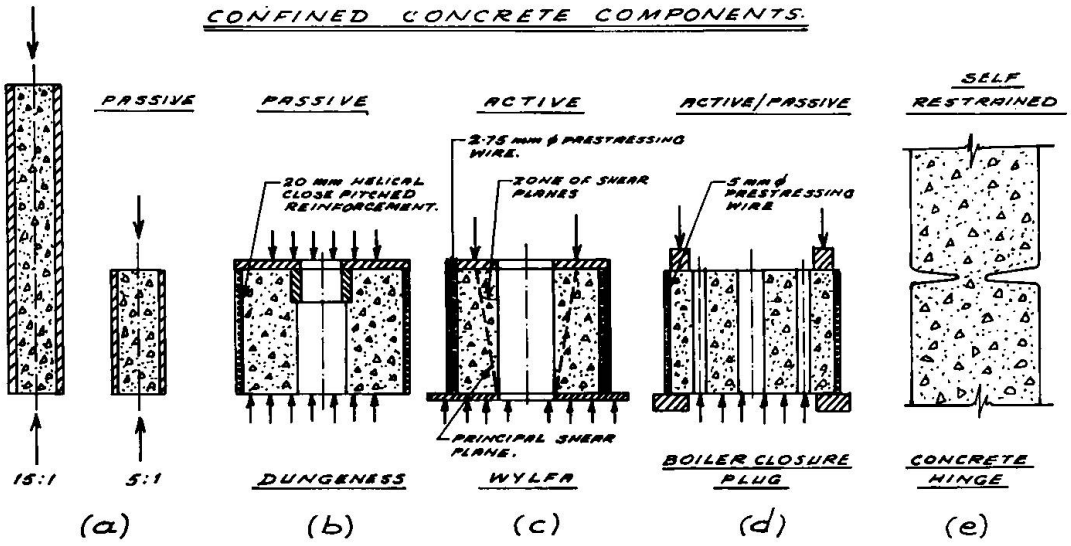


FIG 7

EFFECT OF HOOP RESTRAINT ON THE AXIAL AND SHEAR STRENGTH OF CONCRETE.

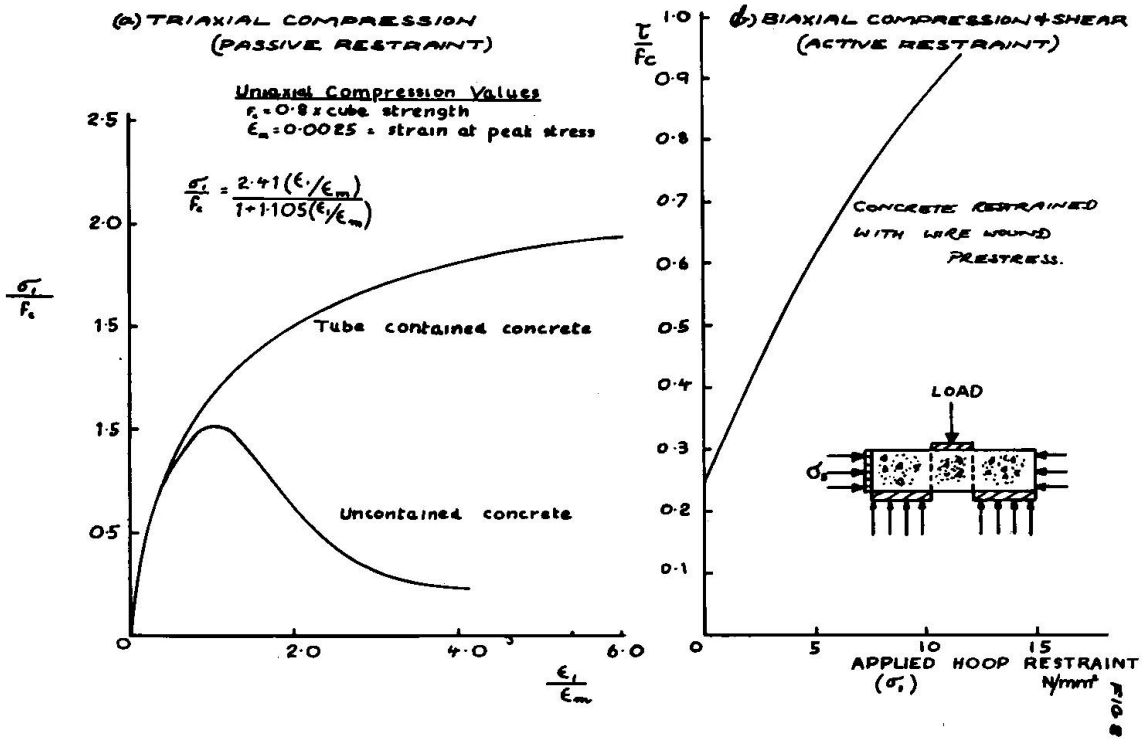


FIG 8

Prestress Anchorages

The authors are not aware of any recent development of further practical design methods for prestress anchorages where high tensile splitting stresses arising from excessive bearing load may be controlled by reinforcement. Perhaps re-examination of the experimental work of Roe and Zielinski(16 & 17) might be fruitful in the light of current development of multiaxial concrete behaviour theory and analysis techniques.

A particularly successful application of the passive restraint technique is in the design of the prestressing bearing plate assembly for the two concrete pressure vessels for the Dungeness 'B' nuclear power station(18). Here, a precast concrete block for the 660 tonne tendons (Fig. 7b) is contained to withstand the high prestress loads by a continuous winding of 19 mm dia. high tensile deformed bar, the unit subsequently being embedded in the vessel concrete. Considerable development work, including long term loading tests, were carried out to confirm the adequacy and safety of the component.

The limitation of the passive restraint technique is that accumulation of strain is required before the benefit of the potential restraint is mobilised.

3.2. Active Restraint

A more direct method is to apply an active restraint to concrete in the form of prestress. Again, the main use so far has been as prestress anchorage blocks for large tendons (584 tonne) for the two pressure vessels for the Wylfa nuclear power station(19) which has been in operation for four years.

Fig. 7c shows the features of the cylindrical block wound with high tensile 2.65 mm dia. wire to a stress of 1300 N/mm^2. The mode of shear failure as shown in Fig. 7c was established by overload tests on 1/3 scale models and nominal strengths of the order of 37 N/mm^2 were achieved (i.e. 20 x the conventional shear strength).

Both short and long term elevated temperature tests were carried out on the full size blocks showing acceptable prestress relaxation, predictable creep using uniaxial specimen data and no significant influence of a sustained temperature of 40°C. No fully acceptable method of analysis of the design was available at that time.

The active restraint combined with the passive restraint has been recently introduced on a highly critical component for the Hartlepool and Heysham pressure vessels which is the closure for the boiler cavity in the pressure vessel walls (Fig. 7d). A full description of the details and the support test work are given by Garas at this Seminar(20).

3.3. Concrete Self Restraint

The use of concrete itself to provide its own restraint to axial and non-axial loads has been employed successfully for many years in concrete hinges (Fig. 7e) for many bridge structures and recently for precast tunnel linings(Section 6.3).

Tests on various hinge configurations were carried out by Base(21) both as regards their load bearing and rotation capability. Reinforcement through the throat was thought to be unnecessary even under shear conditions. Compressive strengths several times the concrete cube strength were achieved without causing crushing of the concrete.

4. CONCRETE PRESSURE VESSELS

4.1. Introduction

In the U.K. extensive programmes of work have been undertaken to develop a number of prestressed concrete pressure vessel (P.C.P.V.) designs for nuclear power stations. This technology started in about 1959 and since then there have been considerable advances which are indicative of the extensive research and development activity which has been devoted to this field. The studies have been concerned with the fundamental problems involved and have utilised reasonably well established techniques such as model and computer analysis. One outcome of this technology was the introduction in 1973 of the British Specification for Prestressed Concrete Pressure Vessels (22).

A considerable amount of this work has also been reported in International conferences(23 - 26) and technical journals. It is only possible in this paper to give below a brief account of the total development work, related to analysis, design research and field observations.

4.2. Vessel Description and Design Conditions

Fig. 9 shows the layouts of the vessels which were completed or are being built in the U.K. The main features of these vessels, the method of analysis adopted and the type of models constructed and tested for each vessel are summarised in Table 2.

Basically, the vessels are three dimensional bodies of simple geometry, but they are made complex by the requirements for steps, penetrations and ribs, causing a variety of stress concentration conditions. Current techniques of analysis depend upon setting up various mathematical models which are capable of numerical solutions.

As a standard procedure, each vessel is designed for the following conditions:

(1) A linear elastic phase which covers all the normal operating conditions:

(a) construction, prestress, commissioning including the proof test.

(b) early and long term (30 years) operation including intermittent reactor start up, shut down and fault conditions.

(c) local stress concentrations and prestressing anchorage stresses.

In addition to compliance of a vessel to permissible stresses, the vessel also has to cater for strain compatibility with a steel internal liner and overall movement tolerances in relation to reactor components. The above conditions can be thoroughly examined by readily available analysis programmes.

(2) An ultimate load analysis which involves large deflection with small increases of pressure where the structure behaves as a mechanism.

An adequate prediction by either theoretical or model analysis of overload behaviour is essential to confirm that a proper margin of safety exists for normal loads, currently set at 2.5 times the design pressure(22).

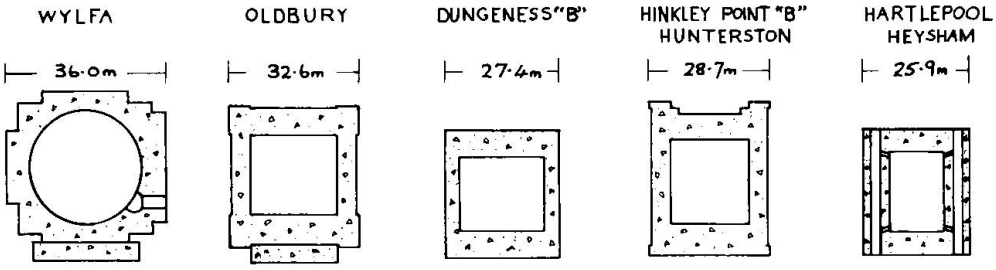
4.3. Design and Analysis

As described in Table 3, basically two methods of analysis, which account for 3-D stresses are used:

(i) finite element analysis(27)

(ii) dynamic relaxation(28)

UNITED KINGDOM PRESSURE VESSELS

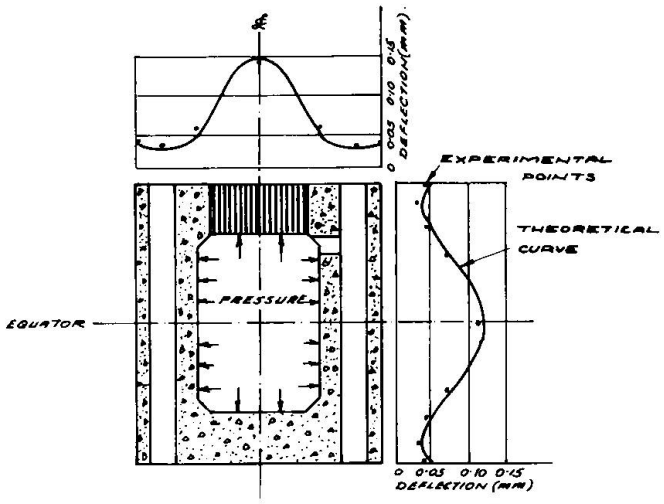


No.		WYLFA	OLDBURY	DUNGENESS "B"	HINKLEY POINT "B" HUNTERSTON	HARTLEPOOL HEYSHAM
1	POWER OUTPUT	590 M.W.	300 M.W.	607 M.W.	625 M.W.	622 M.W.
2	DESIGN PRESSURE	423 P.S.I.	385 P.S.I.	520 P.S.I.	615	644
3	MINIMUM FACTOR OF SAFETY	2·65	3·0	2·5	2·5	2·5
4	VOLUME OF CONCRETE	27,700 CU. YDS	24,288 CU. YDS	16,400 CU. YDS (EST.)	22,600 CU. YDS	15,180 CU. YDS
5	TEMPERATURES °C					
	a). MEAN GAS COOLANT	331	331	485	483	467
	b). CONCRETE INTERNAL	45	55 - 60	55	70	55 - 60
	c). CONCRETE EXTERNAL	30	25 - 30	25	—	30 - 35

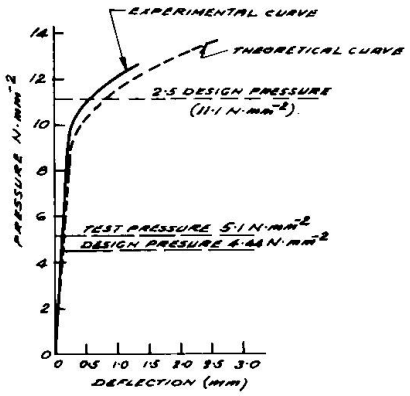
FIG. 9

PCPV. MODELS: TYPICAL CORRELATION BETWEEN THEORY AND EXPERIMENT.

(LANGAN, O'FLYNN AND WELCH)



EXTERNAL DEFLECTIONS AT 4·41 N·mm^{-2} PRESSURE.



EQUATOR DEFLECTIONS VS. PRESSURE.

FIG. 10

TABLE 2 - Summary of Development Work on U.K. Pressure Vessels

Vessel	Organization	Method of Analysis	Model Analysis			
			Type of Model	Scale	No. of Models	Study of
Wylfa	B.N.D.C.	2D - Dynamic relaxation	Spherical PV	1/12	1	E O F T U
	T.W.C.		Spherical PV	1/40	1	E O U
Oldbury	T.N.P.G.	2D - Finite difference and dynamic relaxation	Cylindrical PV	1/8	1	E O F T U
	McAlpine					
	C.E.G.B.	2D - Finite element	Cylindrical PV	1/8	1	E O T U
Dungeness 'B'	A.P.C.	2D - Finite difference	Cylindrical PV	1/10	1	E O U
			Cylindrical PV	1/26	1	O U
			Isolated End Slab	1/26	1	O U
			Isolated End Slab	1/72	1	O U
Hunterston 'B' & Hinkley Point 'B'	T.N.P.G.	2D - Finite elements	Cylindrical PV	1/10	1	E O
	McAlpine					
Hartlepool & Heysham	B.N.D.C.	3D - Dynamic relaxation	Cylindrical Multicavity PV	1/10	1	E O
	T.W.C.	3D - Finite elements	Isolated End Slabs	1/24	15	E O U
General	U.K.A.E.A./I.C.	3D - Finite elements	Spherical PV	1/12	1	E O T U
	U.K.A.E.A. (Foulness)		Cylindrical PV	1/20	10	F U
	G.E.C. - Simon Carves		Cylindrical PV	-	1	E O T U
	T.W.C.		Cylindrical PV	1/20	1	E O F T
	T.W.C.		Cylindrical PV	1/40	2	E O
Future H.T.R.	B.N.D.C.		Cylindrical PV	1/40	4	E O U
	T.W.C.		Isolated End Slabs	1/26	1	E O U
				1/8	3	E O U
	T.N.P.G. McAlpine		Cylindrical PV	-	-	-

KEY

B.N.D.C.	=	Nuclear Design & Construction Limited	E	=	Elastic response
T.W.C.	=	Taylor Woodrow Construction Limited	O	=	Overpressure
T.N.P.G.	=	The Nuclear Power Group	F.	=	Fault condition
McAlpine	=	Sir Robert McAlpine & Sons Limited	T	=	Temperature
A.P.C.	=	Atomic Power Construction Limited	U	=	Ultimate load
C.E.G.B.	=	Central Electricity Generating Board			

In the early development of the design of some of the vessels, the application of these analytical tools was limited by the size of computer available. In recent years, the availability of greater capacity computers has encouraged the continued development and application of both techniques.

Extensive work in the application of finite element analysis to P.C.P.V.'s have been carried out at a number of research centres, particularly at the Central Electricity Generating Board's Berkeley Nuclear Laboratories and the University of Swansea. Some of this work has been reported by Lewis et al(11) and at this Seminar by Zienkiewicz et al(10).

The determination of the ultimate pressure of current vessels has so far been only based on simplified analytical methods. Usually the location of principal hinges, in the case of cylindrical vessels, or major crack patterns in the case of spherical vessels, are determined by the observation of models and theoretical analysis, based on the principle of least work(29 & 30).

In the case of end slabs in cylindrical vessels, where the top slab contains multiple penetrations, the designer has only partial ability to analyse the mechanisms of shear failure. In most cases, as shown in Table 2, safe design is achieved by extensive model testing of this area of the vessel(5 & 31).

4.4. Models

Table 2 shows that at least 48 realistic vessel models up to 1/8 scale full size and end slab models have been tested over the last 15 years in the U.K.

These have been subjected to a range of operation and overload conditions including load and thermal cycling and were primarily related to specific vessel designs.

The purpose of a vessel model was to verify the elastic state and to confirm that the vessel would meet the specified overpressure requirement. Frequently these models were pressurised in excess of the specified requirement without significant damage.

In general terms, comparison between predicted and measured behaviour was close (29, 32, 33) (Fig. 10) although the elastic property of the model concrete required modification in a number of cases.

Instrumentation of both model vessels and end slabs was extensive and results have been generally published. The information has provided already reference data for analysis development (11).

Continuing change in vessel geometry and design together with the concern for the safety of such structures will inevitably demand further testing of large scale physical models in the future to confirm the behaviour of the structure and its reserve of strength. Even the development of powerful analytical tools to handle, for example, ultimate load analysis will not, the authors believe, be sufficient to replace such models.

4.5. Observations on Constructed Vessels

All P.C.P.V.s constructed so far have been subject to a cold proof pressure test normally at 1.15 x the design pressure. Since the Central Electricity Generating Board has insisted on each vessel being comparatively instrumented (e.g. about 400 embedded strain gauges) to record temperature, strain and overall movement, this facility has enabled their design and active performance to be correlated. Published data(34 & 35) on the proof pressure tests showed very close correlation between the two.

For the commissioning heating/cooling runs and early reactor operation, so far reasonably satisfactory prediction by design has been accomplished(11 & 36).

However, it should be emphasized that the interpretation process for instrumentation data from embedded strain gauges etc., in the pressure vessels is an extremely difficult process frequently underrated since reliability of data and its effective interpretation depends on considerable attention to all aspects involved, including supporting long-term concrete property data, the realistic calibration of gauges for concrete creep and proving trials to simulate long term gauge durability at elevated temperatures. Careful selection of gauge location is required in relation to analysis grids in areas where reasonable strain magnitudes will be detectable (i.e. no strain change does not mean no stress).

5. CONCRETE DAMS

5.1. General

Dams are generally designed for a number of loading cases including water pressure, gravity loads and temperature variation, and in some cases earthquake shocks. Furthermore, the effects of uplift and time have to be considered.

A very active research programme into the design and analysis of selected forms of arch dams (Fig. 11) was initiated by a number of British Universities with the assistance of practising engineers. This work was fully reported in a symposium held in London in 1968(37) and is summarised in Table 3.

The table shows that considerable effort was devoted to the basic dam shapes selected and that a number of computing methods were developed. However it has been stated that such methods have not been fully utilised by the practising engineer due to the problems of digesting the methods and the fact that he still has to assume elastic behaviour of the ground and dam materials.

A review of this work and performance of the analysis methods was presented at the 1970 Congress on Large Dams in Montreal(38).

Recent work has tended to move away from model testing to improve stability and accuracy of theoretical analyses (Universities of Southampton and Wales) and to developing suitable methods for handling non-linear behaviour and ground variability (Bristol, Wales). Generally, it is now possible to predict temperature effects, gravity and seismic loads with sufficient confidence without model analysis.

Although only a limited amount of model testing is at present underway (e.g. dynamic water/structure interaction at Bristol University) it is considered that theoretical studies on variable foundation conditions (Imperial College) could well be supported by back-up model tests.

5.2. Seismic Loading

For dams in earthquake areas, seismic stresses have to be superimposed on to static stresses causing the structure to be loaded more severely than at any other time. The accurate prediction of possible seismic loads are therefore a matter of great significance in determining the safety of the design.

In one investigation(39) a check was made on conventional methods of earthquake design and the Hendrik Verwoerd Dam on the Orange River, South Africa, was used as a basic model. The finite element method of analysis was adopted. A full account of the dynamic response of the structure to a given earthquake was taken and maximum values of the stresses and deflections within the arch during the passage of the earthquake were produced.

TABLE 3 - Summary of Development Work on Arch Dams

Dam	Organization	Analysis - Method of Development	Background and Capability	Model Analysis			
				Material	Scale	No. of Models	Study of
Stithians Dam	I.C.	3D - Dynamic Relaxation	H, S G, H, S	Araldite Micro-concrete	1/100 1/60	1 1	E E, F, UE
Monar Dam	I.C.	3D - Dynamic Relaxation	S, H	Cement mortar	1/60	2	E, SP, F
Hendrick Verwoerd Dam	I.C. I.C., R.A.E., B.U.	3D - Finite Element 3D - Finite Element	G, H, S D	 Micro-concrete	1/100 1/200	1 1	Static and overall behaviour SS
Dam Type 1	B.U. U.N., R.C., L.C.T.	3D - Dynamic Relaxation Photoelastic method	H, S H, S	Rubber Thermo setting resin	1/50	1	Displacements SP
Dam Types 2, 5	U.N., R.C., L.C.T.	Photoelastic method	H, S	Thermo setting resin			SP
Dam Type 3	U.N., R.C., L.C.T. I.C.	Photoelastic method 3D - Dynamic Relaxation	H, Different Valleys G, H, RV, S	Thermo setting resin Cement mortar		 1	SP E, F
Dam Type 4	U.N., R.C., L.C.T. B.U. I.C.	Photoelastic method 3D - Dynamic Relaxation 3D - Dynamic Relaxation	H, S H, S H, RV, S	Thermo setting resin Rubber Cement mortar	 1/4	 1 2	SP Displacements E, F
Gravity Dam	I.C.	2D - Photoelastic method	G, H, S	Aluminium Alloy Thermo setting resin	1/400	1	E, SP
General	U.W. I.C. B.U. R.P.T.	3D - Finite Elements 3D - Finite Elements Finite Elements 3D - Dynamic Relaxation	T, AF, H, S, G EP, AD, SW, S, G, T G, H, S, D D, VF				

Capabilities of System

AD = Influence of Abutment Displacement
AF = Arbitrary Foundation
D = Dynamic Analysis
EP = Elasto-plastic Problem
G = Gravity Load
H = Hydrostatic Load
RV = Rigid Valley
S = Static Analysis
SW = Effect of Self Weight
T = Temperature
VF = Variable Valley Flexibility

Model Studies

E = Elastic Response
F = To Failure
SP = Stress Pattern
SS = Seismic Sensitivity
UE = Uplift Effects

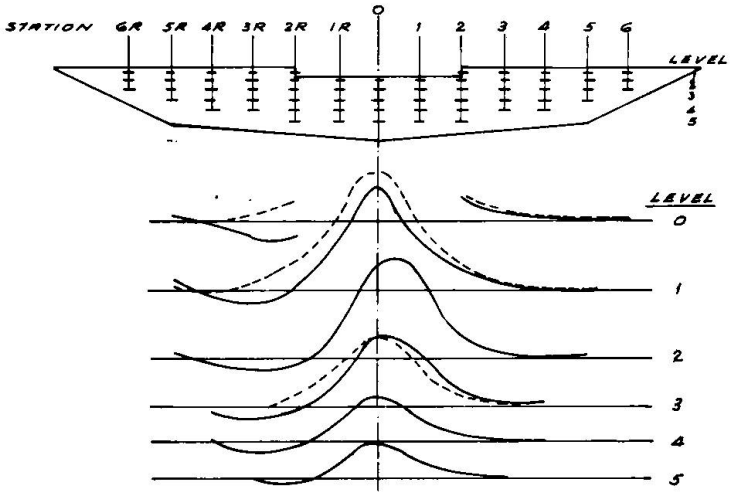
Organizations

I.C. = Imperial College, London
R.A.E. = Royal Aircraft Establishment
B.U. = Bristol University
U.N. = University of Nottingham
R.C. = Richard Costain Limited
L.C.T. = Lanchester College of Technology, Coventry
U.W. = University of Wales
R.P.T. = Rendel, Palmer & Tritton & Partners

FIG. 12

SEISMIC INVESTIGATION ON THE HENDRIK VERWOERD ARCH DAM (BACK et al)

FUNDAMENTAL SYMMETRIC MODES: RESERVOIR EMPTY AND FULL.

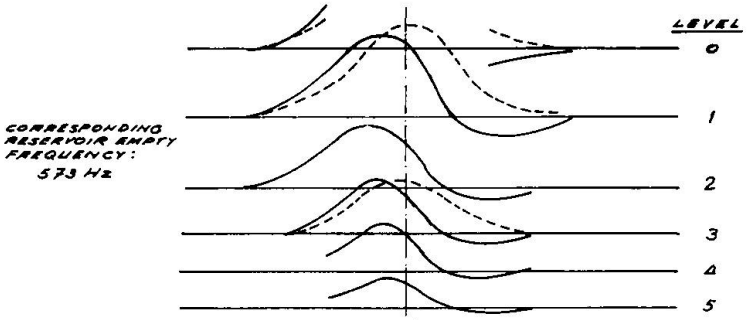


MODE AT 5.73 Hz RESERVOIR EMPTY

——— MEASURED MODE SHAPE

- - - - - CALCULATED MODE SHAPE

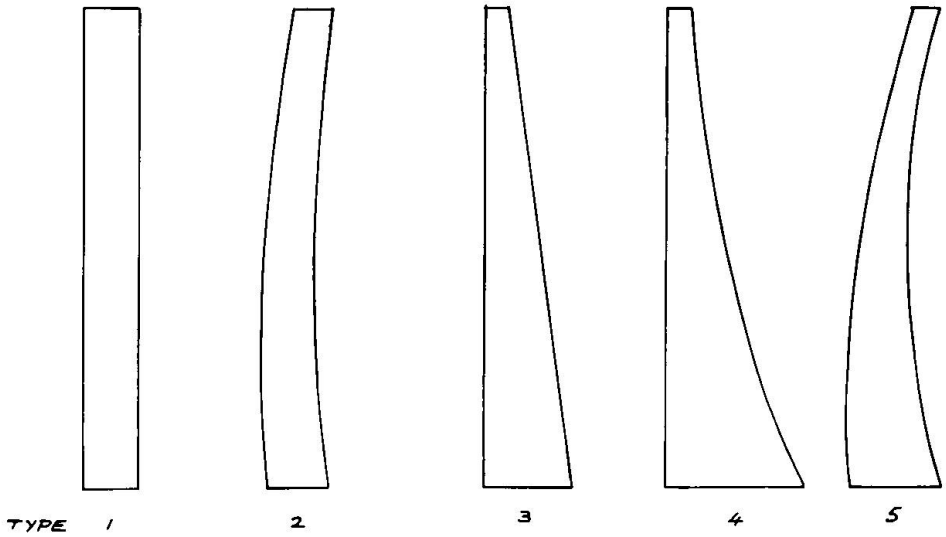
CALCULATED FREQUENCY : 5.48 Hz



MODE AT 4.34 Hz RESERVOIR FULL

——— MEASURED MODE SHAPE

- - - - - CALCULATED MODE SHAPE ON FREQUENCY: 4.88 Hz



CROWN CROSS-SECTIONS OF ARCH DAM MODELS SELECTED FOR STUDY BY THE BRITISH COMMITTEE ON ARCH DAMS (1958-1968)

FIG. 11

The work was supplemented by a micro-concrete model to a scale of 1 : 200 which was built at Imperial College with a valley block large enough to give adequate flexural restraint to the dam. Fig. 12 shows the calculated and measured distortions; these are not strictly comparable because no allowance has been made in the calculations for curvature of the face of the dam in a vertical direction.

Recently, attempts have been made to overcome the non-applicability of standard seismic data to a specific location by recording in the area of the dam micro-seismic activities(40). This approach may be limited due to the problems of extrapolation of such data to a full scale earthquake condition where the non-elastic behaviour of both the ground and the concrete may be dominant.

5.3. Field Instrumentation

In the U.K. there are at least 30 large concrete dams which have been instrumented and have provided records of performance over the last 20 years. Moffat(41) of the University of Newcastle is attempting to establish a record centre for instrumentation data for U.K. dams in order that comparison of long term behaviour of dams can be made. Further, it would enable guidance to be given on instrumentation in order to avoid common problems in the performance and analysis of dam instrumentation. The important field of structural deterioration as regards safety, would also be included in this work. This latter aspect is one which the designer/analyst may tend not to consider.

A recent comprehensive investigation into the performance of the Clywedog Buttress dam in Wales was undertaken for the Construction Industry Research and Information Association(42). Measurements were made of internal stress and strain together with external deflection of the dam and measurement of seepage and uplift pressures in the dam foundation. A simple method of using basic creep data measured separately was developed to deduce stress from strain readings.

6. CONCRETE TUNNEL LININGS

6.1. General

Although a considerable amount of experience has been gained in the design and construction of tunnels, the design of tunnel linings remains an art rather than a science, because of the inherent variability of the main design parameter - the ground. It is not possible to set down a definite list of design rules nor has a code of practice to meet any contingency been produced. It is a rare event if a projected tunnel and it surrounding ground is exactly similar to a previously completed tunnel. Extrapolation of the basic parameters of previous work is only really applicable in the case of tunnels of similar diameters , wall and known ground conditions. Further, it is only the large capital projects such as the Channel Tunnel where meaningful savings can be achieved by refinements in design methods(43)

Currently, global factors of safety of up to 4 times the design loading are used except for major projects where extensive information on ground conditions can reduce the factor to more realistic levels.

6.2. Design Problems

In designing a tunnel lining the stiffness of the lining has to be related to the characteristics of the ground since the strength of the lining is related to both factors. Since the characteristics of the ground before and after instrumentation of the tunnel are usually different, the behaviour of both are considered. The geological conditions of the ground are difficult to predict and no satisfactory method is as yet available. Further judgement related to tunnelling experience is needed to estimate ground loads since full depth loading may not always occur.

6.3. Methods of Design and Analysis

Many tunnel designs have been based on empirical methods which involve simple proportioning of the parameters of the lining to that of previous designs or by the use of a simplified approach(44). However, analytical methods of designing tunnel linings are available which can cope with most design requirements. Finite element techniques, for example, have been developed for the numerical analysis of a large number of boundary conditions, mostly in two dimensions and occasionally in three. The practical usefulness of these methods has been restricted by the uncertainties as to the ground conditions, e.g. degree of fissuring, water flow, the non-isotropic elastic/plastic material properties, and also the variation in ground conditions along the length of the tunnel. Further non-uniform loading can arise during erection.

To overcome the non-uniform loading of unreinfoced precast concrete linings, a lining composed of flexible cross joints (Fig.13a) has been used in London clay for the Victoria Underground line(45) and currently in the Fleet line(46) under construction. Reduction in bending movements resulted in a better distribution in stress and increase in strength. Fig. 13b also shows the effect of joint rotation on the ultimate strength of the joint. Multiaxial self-restraint was considered to be responsible for the high load achieved. The same concept is being considered for use in the Channel Tunnel.

6.4. Field Instrumentation

In the last few years some effort has been devoted to the instrumentation of actual full scale tunnelling situations, particularly by the Building Research Establishment and the Transport and Road Research Laboratory. In situ measurements, for example, were carried out in the Cargo tunnel at Heathrow Airport, London(47), Victoria Underground line(45) and currently for the Fleet line of the London Underground(46).

In the case of the Victoria line(45), results showed that after six years, the horizontal diametral deformation of a 4 m dia. experimental length of tunnel had increased by as much as 10 mm, showing the continuing change of state in the surrounding clay.

In the case of fissured rock, load, pressure and hoop measurements were made on the bolted concrete lining with grouting for the Severn and Wye cable tunnel(48). As shown in Fig. 14, the results demonstrated a non-uniform loading of the lining and a redistribution of stress with time, showing stability had not been reached within the 7 month period of measurement.

Considerable data in designing reinforced concrete linings in rock environments will be gained from the in situ measurements to be carried out during the preliminary driving for the Channel Tunnel at present under construction.

7. OTHER AREAS OF ACTIVITY

7.1. Triaxial Stresses in Construction

In the construction of mass concrete structures whether dams, pressure vessels or concrete foundations for buildings, uncertainty has prevailed for many years as to the effect of size and temperature rise from cement hydration on cracking and joint movement.

Work by Hughes of Birmingham University(49) resulted in a method of designing the steel reinforcement requirements to control cracking from thermal contraction strains.

FIG. 14

TUNNELS IN FISSURED ROCK:

MEASURED HOOP STRAIN IN BOLTED RING SEGMENTS AFTER GROUTING.

SITE: TUNNEL UNDER SEVERN – WYE ESTUARIES.
INTERNAL DIA. = 3 METRES.
(HASWELL)

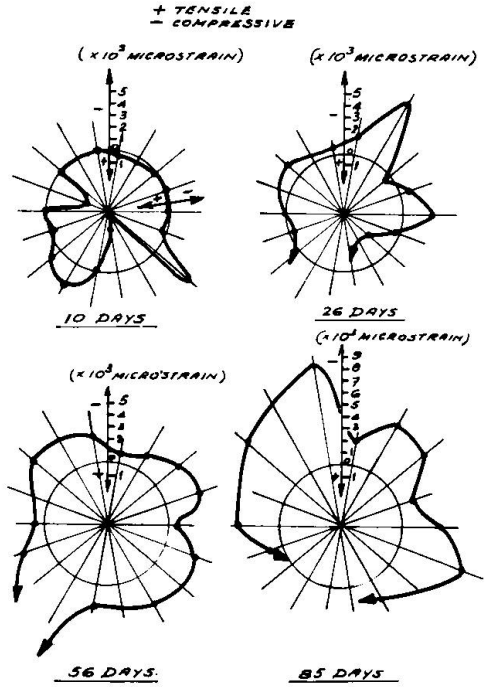
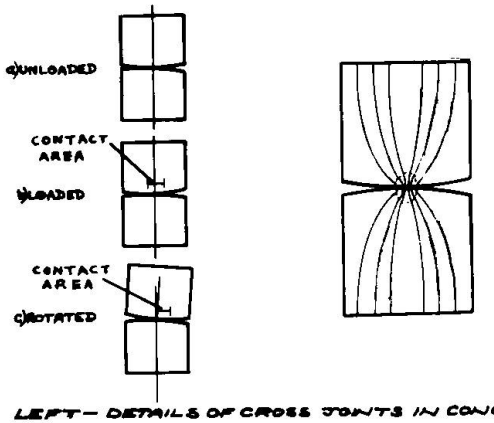
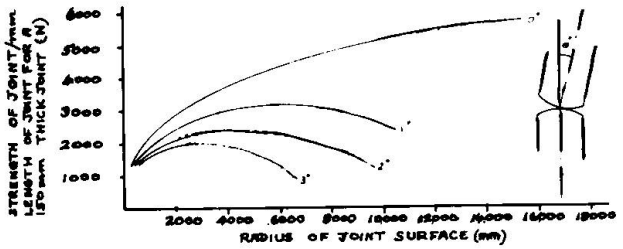


FIG. 13

EXPANDED FLEXIBLE PRECAST CONCRETE TUNNEL LINING IN THE VICTORIA LINE (MORGAN AND BARTLETT).



LEFT – DETAILS OF CROSS JOINTS IN CONCRETE LINING.
RIGHT – FLOW LINES OF LONGITUDINAL STRESS.



RELATIONSHIPS BETWEEN STRENGTH OF JOINT AND RADIUS OF SU FACES FOR V. DEGREES OF JOINT ROTATION.

Instrumentation of 200 m^3 pours in pressure vessel construction and form foundation structures together with experimental work at the authors laboratories (50) have demonstrated the degree of restraint that can build up (Fig. 15) and the stresses that are locked into a pour as a result of the heating and cooling cycle that takes place over the first 2 - 3 weeks after placing. Such locked-in stresses, besides being of interest in relation to cracking and joint movement, are not considered at all at present in subsequent design or analysis considerations. The above work has also included the influence of concrete additives and cement types as well as lift height and bay length on the temperature rise and its effects.

7.2. Buildings and Piled Foundations

In the design of piled rafts for buildings in London clay, Hooper(51) has reported on a finite element analysis undertaken on the foundation of 90 m tower block on London clay in which load cells were installed. Good correlation between the theoretical loads and displacements after six years was found. Field results showed that the load distribution between the piles and the raft for the completed structure was in the region of 6 : 4 and that the foundation stiffness was 10 times greater than the raft itself due to rigidity of the substructure, the contribution of the superstructure being relatively small.

Perhaps in the near future more attention will be given to the global interaction of the building with dead and live loads and the way load is transmitted to the ground.

7.3. Offshore Structures

With the discovery of the North Sea Oil, there is a rapidly increasing demand for offshore concrete gravity-type structures to be used for oil production and/or for oil storage. These structures are massive and although the geometry may be simple they require careful design to cater for stability, wave forces and sea bed conditions.

Their unique feature is they consist of a number of massive components, e.g. raft, pods and towers, which are interlinked (Fig. 16). Individual components demand a sophisticated and lengthy type of analysis. Computer programmes using finite elements and grillage types of analysis are being used. These design tools are capable of describing the behaviour of each component and the interaction between them. Further development work into the application of these methods is in progress.

The combined action of the various components and the partial passive restraint which is provided by the continuity of the structure, create triaxial states of stress. This no doubt will enhance the limit of cracking and the ultimate strength. It has been established that lateral restraint, especially in slabs(52 & 53) increases considerably their flexural and shear strength due to the increase in the compressive membrane forces. Advantage might be taken of the triaxial state of stress in many parts of such structures and hence increases in the allowable stresses may be possible.

FIG. 15.

DEGREE OF RESTRAINT TO FREE THERMAL MOVEMENT DURING THE HEAT OF HYDRATION THERMAL CYCLE IN A MASS POUR (BROWNE & BLUNDELL)

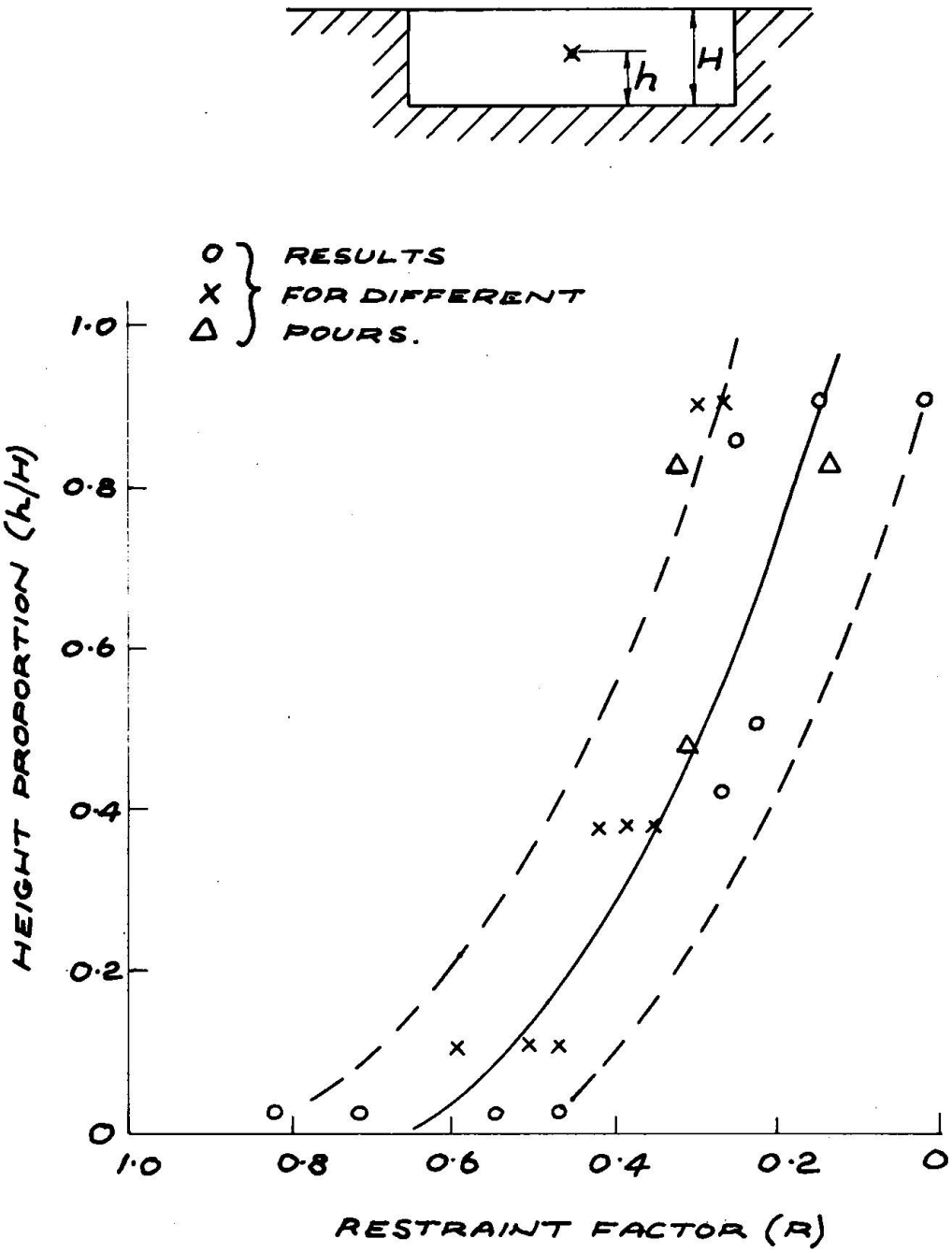
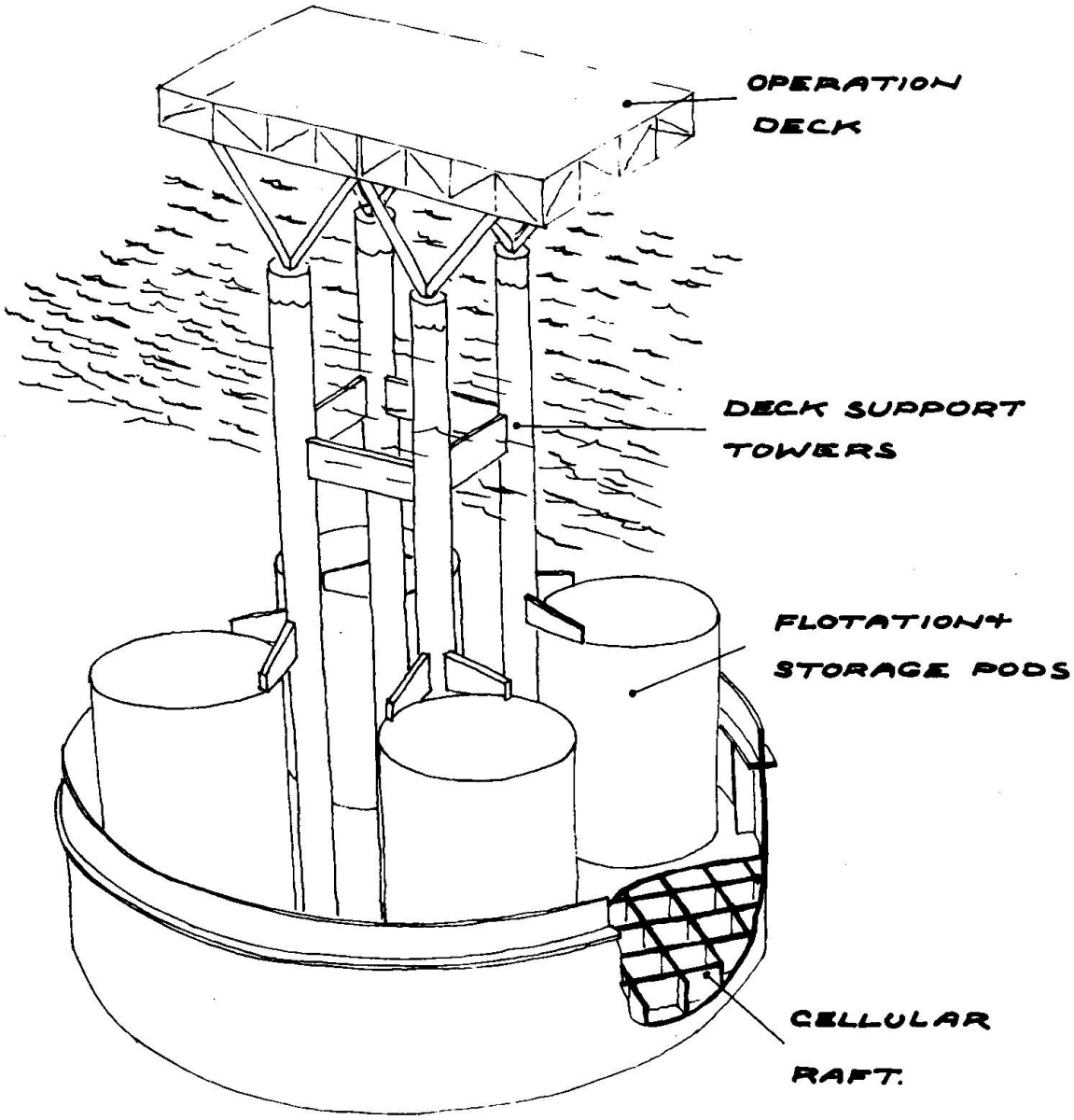


FIG. 16

OFF-SHORE DRILLING RIG

CONCRETE GRAVITY STRUCTURE



8. CONCLUSIONS

8.1. In the U.K., research and development is active on all fronts and in particular, the demands on understanding the detailed behaviour of concrete pressure vessel for the nuclear power station programme has stimulated considerable advances both in universities, government and commercial establishments.

8.2. Development of analysis and design methods, to handle overload and time dependant states as well as variable natural boundary conditions, are being supported by model analysis and to a somewhat lesser degree, by feed-back from field instrumentation.

8.3. So far, the design standards and codes of practice[54] have only to a limited degree provided guidance on the many factors outlined in the paper involved in the design of 3-D stressed structures but they have left the way open for progress in this field. In certain applications, such as dams and tunnels, uncertainties in environmental loadings etc. have prevented the preparation of such documents since design depends so much on specialised experience in the field.

8.4. The benefits of higher strength achieved under triaxial compressive states have been so far only utilised in small component applications where the applied stress states can be controlled to a close degree.

8.5. Finally, the authors believe that this Seminar will provide a valuable link to all those engineers involved in widely different applications of concrete in triaxially stressed states since cross-fertilisation of techniques of design, analysis, material and model behaviour combined with field measurements could well stimulate progress.

Recommendation

To ensure that future progress is broad based in the total 'matrix' area defined in the Introduction to this Report, it is suggested that the IABSE or some other appropriate international body should, if possible,

(i) set up a reference bank for data on all aspects of this important subject

(ii) arrange that an analysis of data in each component field is produced in a form directly usable to the designer.

ACKNOWLEDGEMENTS

The authors wish to express their appreciation to the Directors of Taylor Woodrow Construction Limited for permission to prepare and publish this report and to their colleagues who have assisted in its preparation. The report could not have been produced without valuable discussion with external practising engineers and research workers as well as reference to their published information. This aid has been well appreciated.

REFERENCES

(1) Newman, K. and Newman, J. B. "Failure Theories and Design Criteria for Plain Concrete" Proc. Civil Engineering Materials Conference 1969 Southampton Univ., Publ. Wiley - Interscience, London, 1971.

(2) Newman, K. and Newman, J. B. "The Cracking and Failure of Concrete under Combined Stresses and its Implication for Structural Design" Proc. Rilem International Symposium on the Deformation and the Rupture of Solids subject to Multiaxial Stresses, Cannes, October, 1972.

(3) Hannant, D. J. "Nomograms for the Failure of Plain Concrete Subjected to Short-term Multiaxial Stresses" The Structural Engineer No.5, Vol.52, May 1974.

(4) Garas, F. K. and Langan, D. "Capacity of Prestressed Concrete Vessels to Resist Shear" Paper SP 34-12, ACI Seminar, Concrete for Nuclear Reactors, Berlin, 1970.

(5) Langan, D. and Garas, F. K. "Behaviour of End Slabs in Cylindrical Prestressed Concrete Pressure Vessels", paper H3/4 First International Conference on Structural Mechanics in Reactor Technology, Berlin 1971.

(6) Browne, R. D. and Blundell, R. "The Behaviour of Concrete in Prestressed Concrete Pressure Vessels", Paper H1/1, Nuclear Engineering and Design, Volume 20(1972) No.2, July 1972.

(7) Illston, J. M. and Jordaan, I. J. "Creep Prediction for Concrete under Multiaxial Stress", Journal of Am. Conc. Inst., March 1972.

(8) Illston, J. M. and Sanders, P. D. "The Effect of Temperature Change upon the Creep of Mortar under Torsional Loading" Mg. Conc. Res. Vol. 25, No. 84 September, 1973.

(9) Whaley, C. P. and Neville, A. M. "Non-elastic Deformation of Concrete under Cyclic Loading" Mg. Conc. Res. Vol. 25, No. 84, September 1973.

(10) Zienkiewicz et al. "Finite Element Analysis of some Concrete Non-Linearities Theory and Examples" Seminar on Concrete Structures subjected to Triaxial Stresses, Bergamo, 1974.

(11) Lewis, D. J., Irving, J. and Carmichael, G.D.J. "Advances in the Analysis of Prestressed Concrete Pressure Vessels" Nuclear Engineering and Design No.20, 1972, pp. 543 - 573.

(12) England, G.L. "Time Dependent Stresses in Creep - Elastic Materials A general method of calculation" Proc. Conf. on Recent Advances in Stress Analysis, Royal Ae. Soc., London, 1968.

(13) Neogi, P. K., Sen, H.K., and Chapman, J.C. "Concrete Filled Tubular Steel Columns - Uniaxial Behaviour under Eccentric Loading" Construction Industry Research and Information Association, Technical Note 3, January, 1969.

(14) Sen, H.K. and Chapman, J.C. "Ultimate Load Tables for Concrete - Filled Tubular Steel Columns" Construction Industry Research and Information Association, Technical Note 13, August 1970

(15) Sen, H.K., Chapman, J.C. and Neogi, P.K. "Concrete filled Tubular Steel Columns - Triaxial Effects in Circular Steel Columns under Concentric Loading" Construction Industry Research and Information Association Technical Note 39, June 1970.

(32) Langan, D. "Correlation between Theory and Experiments - Wylfa Vessel" Paper 39, Conference on Prestressed Concrete Pressure Vessels, Institution of Civil Engineers, London 1968.

(33) Hornby, I.W., Verdon, G.F. and Wong, Y.C. "Testing the Oldbury Vessel Model" Paper 41, Conference on Prestressed Concrete Pressure Vessels, Institution of Civil Engineers, London, 1968.

(34) Eadie, D. McD "The Behaviour of the Oldbury No.1 Reactor Pressure Vessel during Prestressing and Proof Pressure Tests" Nuclear Engineering and Design Vol. 54:1967 pp. 295.

(35) Carlton, D., Smith, J.R. and Bellwood, G.N. "Prestressing and Proof Pressure Testing of the Wylfa Prestressed Concrete Pressure Vessels and Comparison of Measured Data with Predicted Performance" Paper H4/2 First International Conference, Structural Mechanics in Reactor Technology, Berlin, 1971.

(36) Browne, R. D., Smith, J.R. and Bamforth, P. "The Performance of the Wylfa Nuclear Pressure Vessels during Commissioning in Relation to Design Analysis" Taylor Woodrow Internal Report No. 014J/73/1630 June 1973.

(37) The Institution of Civil Engineers "Arch Dams - a Review of British Research and Development"Proc.of the Symposium, London, 1968.

(38) Roberts, L.M., Chitty, L. and Crook, D.E. "The Application of Modern Methods of Analysis to Arch Dams" Proc. of Congress on Large Dams Vol. 4, pp 167 - 180, Montreal, 1970

(39) Back, P.A.A., Cassell, A.C., Dungar, R., Gaukroger, D.R. and Severn, R.T. "The Seismic Design Study of a Double Curvature Arch Dam" Proc. of the Institution of Civil Engineers, Vol.43, Paper No. 7181. June 1969.

(40) Haigh, I.P. "Informal Discussion on Design of Dams in Earthquake Areas" Proc. of the Institution of Civil Engineers, May 1974.

(41) Moffat, A.I.B. "Instrumentation and Performance Studies on Concrete Dams" News and Views of the British National Committee on Large Dams, November 1973.

(42) Construction Industry Research and Information Association, Instrumentation Investigations at Clywedog Dam, Technical Note 9, 1970.

(43) Tough, S.G. "Present Practice in Tunnel Design, Tunnelling in Rock" Series of Lectures, South African Institution of Civil Engineers, 1973.

(44) Howells, D.A. "Design of Tunnel Linings" Civil Engineering and Public Works Review, September 1971.

(45) Morgan, H.D. and Bartlett, J.V. "The Victoria Line - Tunnel Design" Proc. of the Institution of Civil Engineers, Paper 7270S, Vol. December 1969.

(46) Donovan, H.J. "Expanded Tunnel Linings" Tunnels and Tunnelling March 1974.

(16) Zielinski, J. and Rowe, R.E. "An Investigation of the Stress Distribution in the Anchorage Zones of Post-tensioned Concrete Members" Cement and Concrete Association, Research Report No.9, London 1960.

(17) Zielinski, J. and Rowe, R.E. "The Stress Distribution associated with Groups of Anchorages in Post-tensioned Concrete Members" Cement and Concrete Association, Research Report No.13, London, 1962.

(18) Garas, F. K. "Dungeness 'B' Power Station - Ultimate Load Behaviour of Bearing Plate Assemblies Under Short and Long Term Loading" Taylor Woodrow Internal Report No. 14H/70/1455, London, 1970.

(19) Rowland, A.H. and Garas, F.K. "Wylfa Prestressed Concrete Pressure Vessel" Taylor Woodrow Internal Report No. 147/66/1174, London, 1966.

(20) Garas, F. K. "Analysis and Behaviour of Triaxially Prestressed Concrete Components for Prestressed Concrete Pressure Vessels" Paper 2/4, Seminar on Concrete Structures subjected to Triaxial Stresses, Bergamo, 1974.

(21) Base, G.D. "Test on Four Prototype Reinforced Concrete Hinges" Cement and Concrete Association, Research Report No. 17, London 1965.

(22) B.S. 4975: 1973 "Specification for Prestressed Concrete Pressure Vessels for Nuclear Reactors" British Standards Institution, London 1973.

(23) Institution of Civil Engineers: Proceedings of Conference on Prestressed Concrete Pressure Vessels, Institute of Civil Engineers, London 1968.

(24) Institution of Civil Engineers: Symposium on Model Technique for Prestressed Concrete Pressure Vessels, British Nuclear Energy Society, London 1969.

(25) International Association for Structural Mechanics in Reactor Technology; First International Conference on Structural Mechanics in Reactor Technology, Berlin 1971.

(26) International Association for Structural Mechanics in Reactor Technology; Second International Conference on Structural Mechanics in Reactor Technology, Berlin, 1973.

(27) Zienkiewicz, O.C. and Holister, G. S. "Stress Analysis", London, John Wiley and Sons, 1965.

(28) Welch, A. K. "Application of Dynamic Relaxation to Elasticity Problems" Ph.D Thesis, Southampton University, 1969.

(29) Langan, D., O'Flynn, M. and Welch, A. K. "The Design of Pod Boiler Pressure Vessels with Particular Reference to Hartlepool Nuclear Power Station" paper H4/4, First International Conference on Structural Mechanics for Reactor Technology, Berlin 1971.

(30) Finigan, A. "Ultimate Analysis of the Dungeness 'B' Vessel" Conference on Prestressed Concrete Pressure Vessels, Institute of Civil Engineers, London, 1968.

(31) Brading, K.F. and Hills, G. "The Use of Structural Models in Developing Pressure Vessel Design" Conference on Prestressed Concrete Pressure Vessels, Institute of Civil Engineers, London, 1968.

(47) Muir Wood, A.M. and Gibb, F.R. "Design and Construction of the Cargo Tunnel at Heathrow Airport" Proc. Institution of Civil Engineers, Paper 7357, London, January 1971.

(48) Haswell, C.K. "Tunnel under the Severn and Wye Estuaries" Proc. Institution of Civil Engineers, Paper 7650, Vol. 54, London, August 1973.

(49) Hughes, B.P. "Reinforcement for Thermal Stress" Symposium of Massive Reinforced Concrete Structures, Birmingham, Concrete Society, September 1973 (To be published).

(50) Browne, R.D., and Blundell, R. "The Early Age Behaviour of Mass Concrete Pours" A Symposium of Massive Reinforced Concrete Structures, Birmingham, 1973 (To be published).

(51) Hooper, J.A. "Observations on the Behaviour of a Piled-Raft Foundation on London Clay" Proc. Institution of Civil Engineers, Paper 7662, December 1973.

(52) Park, R. "Membrane Action at the Ultimate Load of Laterally Restrained Concrete Slabs" CEB Report, Dalles-Structures Planes, October 1965.

(53) Jones, L.L. and Wood, R.H. "Yield-line Analysis of Slab" Chatto and Windus, London 1967.

(54) CP.110 1972 "The Structural Use of Concrete", British Standards Institution, London, 1972.

SUMMARY

The paper first summarises the parameters that should be considered in assessing progress in the areas of materials research, theoretical and model analysis, design and field observation relevant to triaxially stressed concrete applications. The main part of the paper consists of a survey of Research and Development in the United Kingdom on structures which could utilise triaxially stressed material properties. The application to nuclear pressure vessels is discussed in detail . Reference is also made to other types of structures, such as dams and tunnels, highlighting the major design problems in relation to the stress states.

RESUME

Le rapport traite d' abord les paramètres qu' on doit considérer pour obtenir des progrès dans les domaines de la recherche sur matériaux, de la analyse théorique et par maquettes, du projet et de la réalisation, en relation aux applications du béton sous contrainte triaxiale. La partie principale du rapport représente un panorama des recherches et des développements dans le Royaume-Uni, sur structures qui pourraient utiliser les propriétés des matériaux chargés triaxialement. On discute en détail l' application aux caissons en pression pour réacteurs nucléaires. On fait aussi référence à d' autres types de structures, comme barrages et tunnels, en soulignant les problèmes plus importants du projet, en relation aux états de contrainte.

ZUSAMMENFASSUNG

Der Artikel fasst zuerst die Parameter zusammen, die zur Erzielung von Fortschritten auf den Gebieten der Werkstoff-forschung, theoretischen und Modell-Analysen, Beobachtungen bei Design und Ausführung von dreiachsigen Spannbeton-Konstruktionen zu berücksichtigen sind. Der Hauptteil des Artikels bietet einen Überblick über Forschung und Entwicklung in Grossbritanien hinsichtlich solcher Konstruktionen, für die die Eigenschaften von dreiachsigen Werkstoffen von Nutzen wären. Auf die Verwendung für Spannbeton-Druckbehälter wird ausführlich eingegangen. . Auch andere Typen von Konstruktionen, wie Deiche und Tunnels, werden erwähnt, wobei die hauptsächlichen Design-Probleme im Zusammenhang mit der Druckspannung hervorgehoben werden.

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SEMINAR on:
«CONCRETE STRUCTURES SUBJECTED TO TRIAXIAL STRESSES»
17th-19th MAY, 1974 - ISMES - BERGAMO (ITALY)

Philosophie sur la technique des modèles statiques adoptée à l'ISMES pour les structures massives

Philosophy on Statical Models Techniques used at ISMES for massive Structures

Philosophie der von ISMES angewanten Technik der Statischen Modellen für massive Strukturen

Prof. Ing. Emanuele FUMAGALLI, Directeur ISMES - Istituto Sperimentale Modelli e Strutture
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Le présent rapport s' occupera, à titre prioritaire, des essais à rupture des modèles à l' étude, car pour les structures massives la seule connaissance des contraintes en domaine élastique résulte insuffisante pour une évaluation croyable du facteur de sécurité de l' ouvrage.

En effet, seulement un essai poussé jusqu' à rupture peut donner une valeur du facteur de sécurité globale basée sur la résistance structurale d' ensemble.

Logiquement, la crédibilité d' une pareille vérification dépend, dans une mesure déterminante, de la correspondance en similitude entre caractéristiques des matériaux modèle et prototype et en régime élastique, mais surtout en régime élasto-plastique jusqu' à rupture. A ce sujet, l' analyse des paramètres de similitude indiqués par la note (^) - paramè-

(^) Il est reconnu que l' expression fondamentale suivante doit être respectée dans le secteur des modèles statiques: $\zeta = \rho\lambda$ (1) où:

$\zeta = \sigma/\sigma' = E/E' = R_c/R'_c = \ldots\ldots$ représente l' échelle de reproduction des grandeurs ayant les dimensions d' une contrainte spécifique: contraintes, modules de Young, résistances à la compression, etc.

$\lambda = L/L'$ représente l' èchelle de reproduction géométrique des grandeurs linéaires

$\rho = \gamma/\gamma'$ représente l' èchelle des forces spécifiques de masse.

très sur lesquels se basent principalement les choix des techniques expérimentales à adopter - revêt une importance fondamentale. Les choix sont conditionnés, en principe, par la nécessité ou non d' appliquer au modèle les forces de masse qui lui sont propres.

A ce sujet, trois secteurs différents d' application peuvent être envisagés à l' ISMES:

1. Modèles de caissons en béton précontraint pour réacteurs nucléaires

Dans un autre rapport on a déjà traité en détail de ces modèles. Il y a néanmoins lieu de remarquer que,dans ce cas, les contraintes de poids propre peuvent être considérées comme secondaires vis-à-vis des contraintes appliquées par la précontrainte et par la pression à l' intérieur du caisson.

Il en dérive que selon l' expression (1) (^), le choix du "rapport d' efficience $\xi = 1$" permet l' emploi de microbétons.

Les recherches sur les microbétons on été conduites en plusieurs laboratoires. En résumé, on peut affirmer que ces matériaux sont ceux qui se rapprochent le plus, par comportement du béton ordinaire par suite de leur affinité de concrétion. Leurs caractéristiques fondamentales - modules élastiques, résistances pour des contraintes simples et composées - se situent dans des zones de dispersion assez proche de celles de un béton normal.

Nous ne saurions en dire autant des caractéristiques de creep. L' excès de mortier augmente en effet dans les microbétons les effets de retrait et les caractéristiques plasto-visqueuses. A ce sujet, cependant, il nous faut souligner que les techniques des modèles ne représentent pas un moyen suffisamment valable pour le contrôle des structures à des échéances trop prolongées. Nous pouvons donc en conclure que les résultats sur modèle en microbéton assurent un bon degré de crédibilité des essais effectués en domaine élastique et à rupture; des derniers uniquement si on les rapporte à des délais comparables à ceux d' un normal essai d' acceptation.

2. Modèles statiques ou traditionels de barrages

Ce secteur d' application des modèles est désormais traditionnellement utilisé pour les barrages à arc et à arc-poids. En pareil cas, il faut tenir compte, aux fins de l' équilibre de la structure pendant les essais à outrance, de la composante de poids propre, tout au moins en ce qui concerne l' ouvrage en béton.

L' examen des paramètres de similitude indique que si l' on emploie les microbétons, selon un rapport d' efficience $\xi = 1$, il s' avère nécessaire d' utiliser un matériau de densité augmenté dans le rapport $\gamma' = \gamma\lambda$.

A l' ISMES on fait recours généralement à une installation de de charge artificielle en utilisant des tirants suffisamment distribués dans

le modèle (fig. 1). Néanmoins, pour éviter un encombrement excessif, on préfère utiliser des matériaux moins résistants ayant un rapport d'efficience: $3 \leq \zeta \leq 8$, ce qui donne la possibilité de réduire la densité à la valeur $\gamma' = \gamma \cdot \lambda / \zeta$.

Cela permet, en ajoute, d'effectuer des cycles de poids propre pour la détermination des contraintes relatives.

Les matériaux qui répondent le mieux à ce but se sont les microbétons en pierre ponce. On peut affirmer qu'ils respectent de façon satisfaisante les paramètres de similitude, exception faite des caractéristiques de résistance à la rupture pour les états de contrainte composés. La

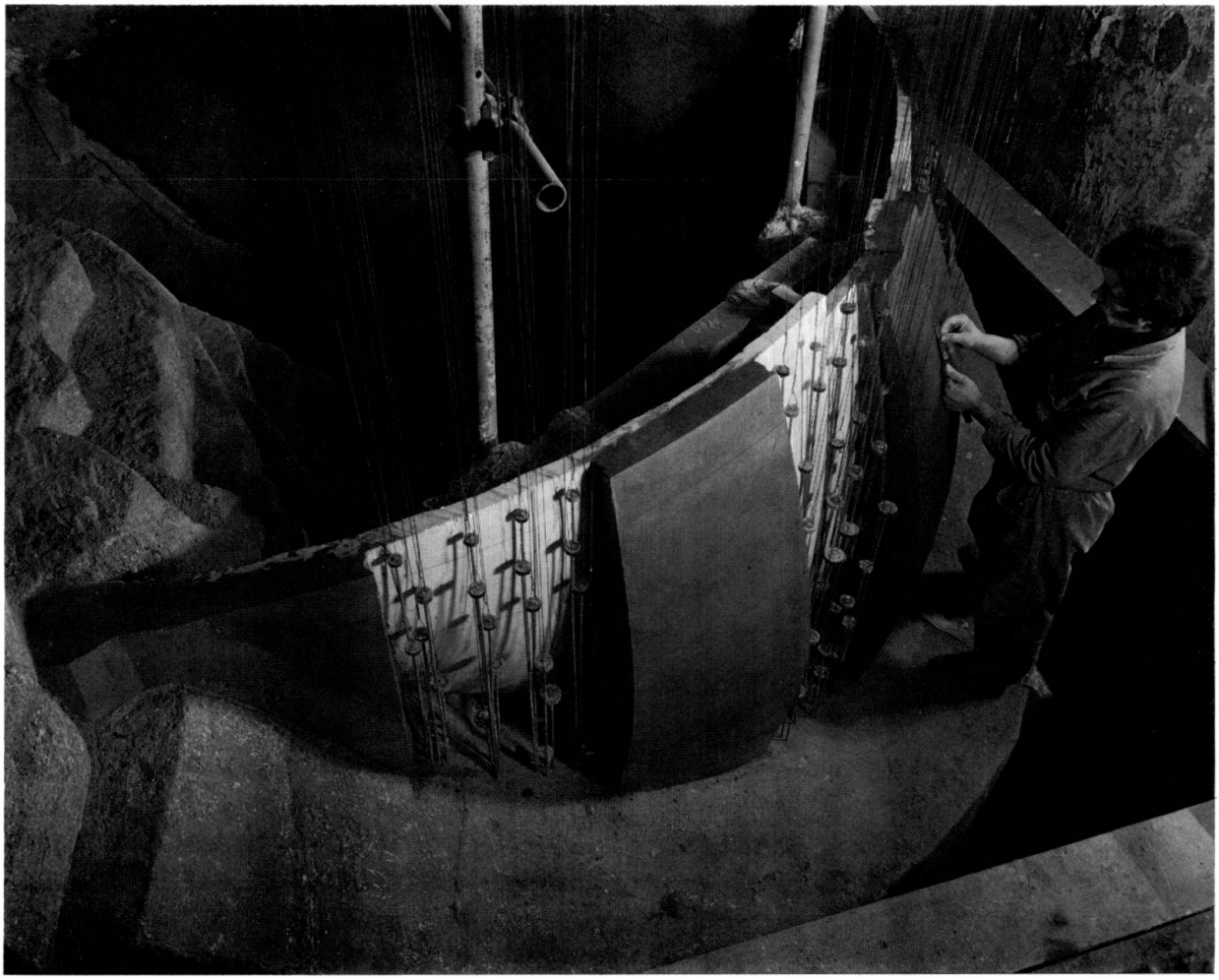

FIG. 1

Tirants de poids propre en acier répartis dans les blocs d'un barrage - modèle.

Steel wires of dead load distribued in the blocks of a model dam.

Eigengewicht-Stahlspannstangen an den Blöcken eines Modell-Staudammes verteilt.

nature poreuse de l' agrégat provoque en effet des tassements plastiques relativement prématurés, surtout pour des états de contrainte triaxiaux de seule compressione et particulièrement poussés. En effet, la courbe intrinsèque à rupture (diagramme $\tau - \sigma$) est sensiblement plus aplatie vers l' axe des σ , c' est-à-dire, elle dénonce des résistances au cisaillement plus réduites. (fig. 2).

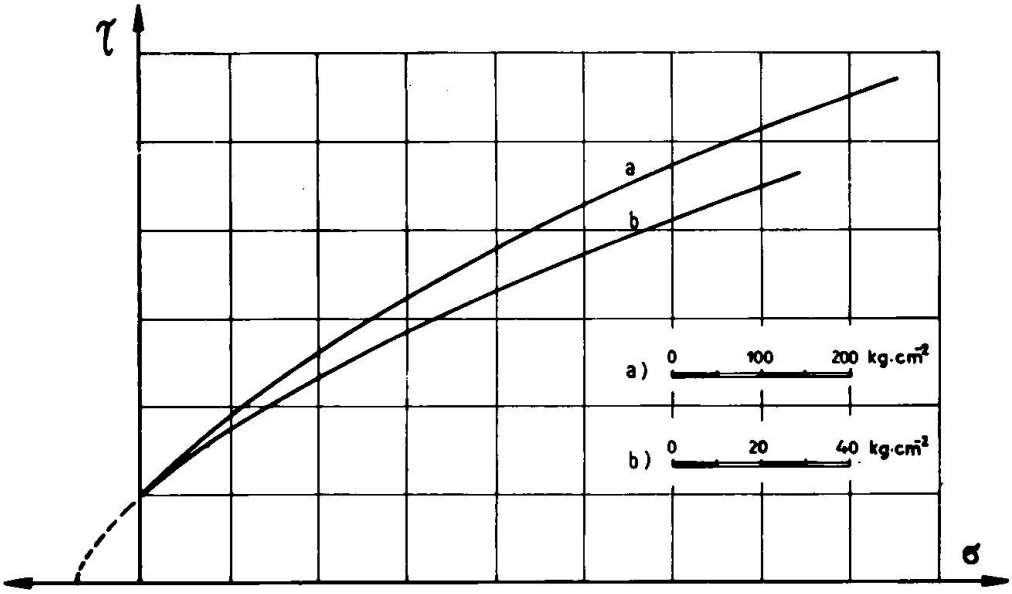

FIG. 2

Courbe intrinséque de rupture: a) béton normal; b) microbéton en pierre ponce.

Intrinsic collapse (failure) curve: a) normal concrete; b)microconcrete in pumice.

Effektive Bruchkurve: a) normaler Beton; b) Mikrobeton aus Bimsstein.

Une certaine discordance à ce sujet peut être admise, car rarement les ruptures se vérifient pour des contraintes triaxiales de compression pure et simple. En plus, la correspondance imparfaite du matériau-modèle est, dans ce cas, de nature déficitaire, et par conséquent elle répond à des critères conservatifs.

Le modèle statique traditionnel constitue par ailleurs un instrument particulièrement efficace aux fins des recherches sur le fonctionnement statique du tampon du barrage où, en présence d' arcs à forte épaisseur, les calculs deviennent difficiles et les résultats discutables.

Vis-à-vis d' un problème d' une telle importance, on se limite à rapporter une brève documentation sur l' intérêt unique des informations sur le fonctionnement statique de la structure obtenu par le modèle.

Dans la fig. 3 on constate l' allure des contraintes par arcs

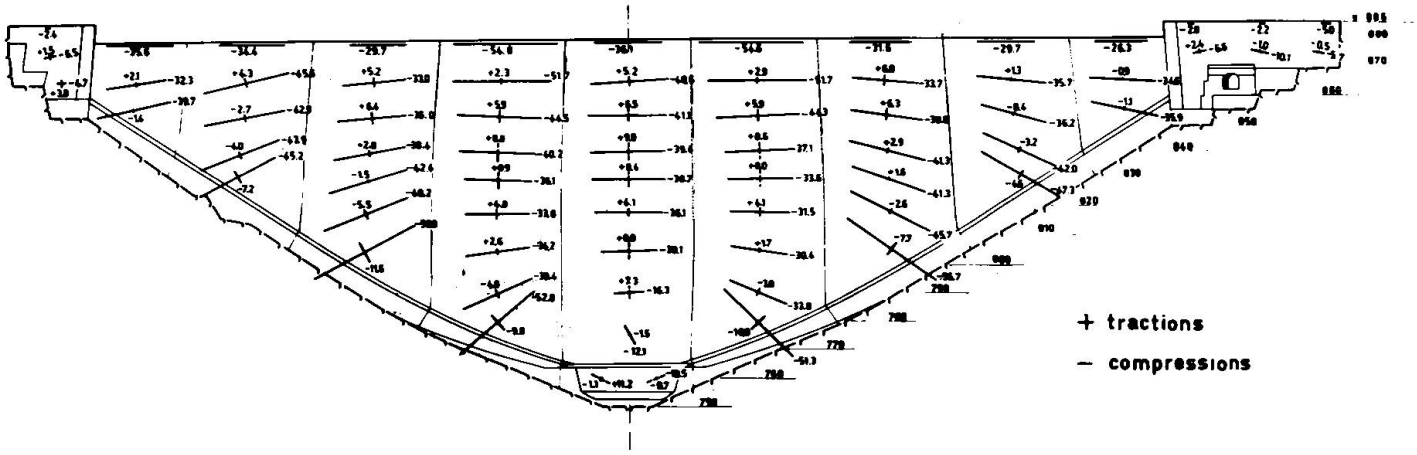

FIG. 3

Modèle du barrage de Las Portas (Espagne): essai de charge hydrostatique, allure de contraintes principales, surface à l' aval.

Model of Las Portas dam (Spain): hydrostatic load test, principal stresses, downstream.

Modell des Staudammes in Las Portas (Spanien); Versuch hydrostatischer Belastung: Verhalten der Hauptspannungen, Luftseite.

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plongéants en correspondance du corps inférieur du barrage engendré par la charge hydrostatique.

Dans la fig. 4 on présente, pour le même ouvrage, la répartition de la charge hydrostatique par éléments résistants, en correspondance de la console centrale. On peut remarquer que l' effet voûte se réduit considérablement aux cotes inférieures, pour laisser la place à une forte contribution résistante de flexion et de torsion.

La fig. 5 dénonce les excentricités des résultantes le long des arcs d' un modèle-barrage qui s' accentuent particulièrement en correspondance des arcs inférieurs.

Dans la fig. 6 on présente le cadre des contraintes principales mesurées dans le plan d' un arc de tampon.

La fig. 7 présente enfin le cadre des fissurations relevées dans un modèle de barrage après les essais à rupture.

3. Modèles géomécaniques de massifs rocheux

Un trosième secteur, plus complexe, concerne les modèles géomécaniques. Ces modèles permettent de contrôler la stabilité des massifs rocheux intéressés par des systèmes de discontinuité plus ou moins importants et défavorables. L' emploi d' une installation artificielle de poids propre, réalisée au moyen de tirants, est d' ordinaire inacceptable du fait des liaisons que les tirants introduisent dans la structure discontinue.

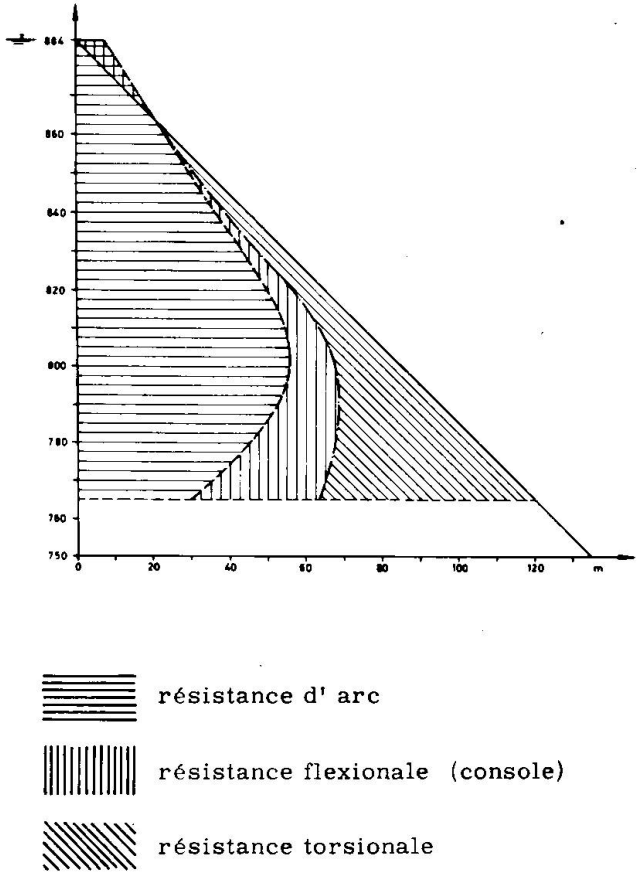

FIG. 4
Modèle du barrage de Las Portas (Espagne): répartition de la charge hydrostatique relative à la console principale.
Model of Las Portas dam (Spain): distribution of hydrostatic load for the central cantilevel.
Modell des Staudammes in Las Portas (Spanien): Verteilung der hydrostatischen Last bezüglich des mittleren Kragsteins.

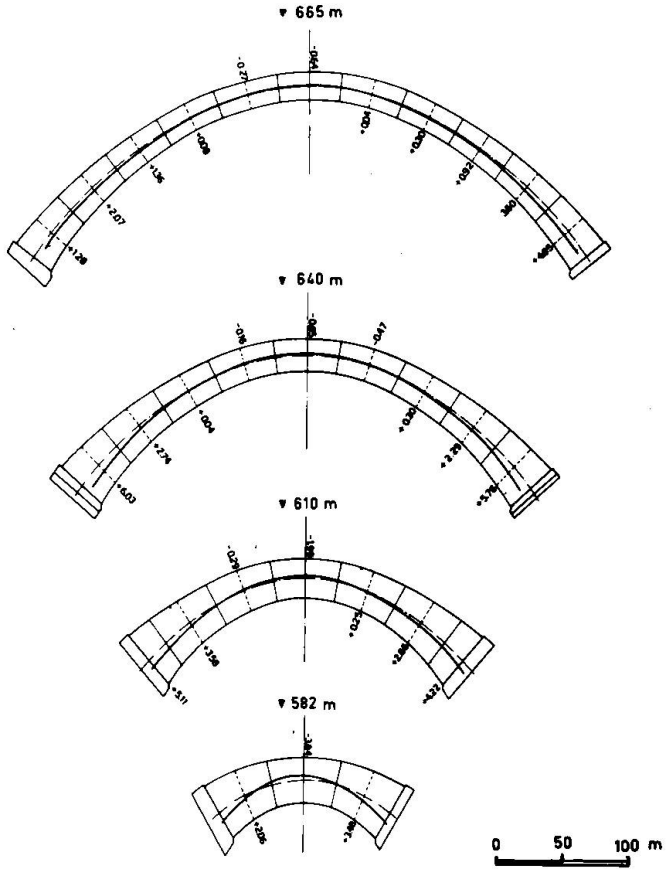

FIG. 5
Excentricités (en mètres) des résultantes transmises dans les arcs du barrage de Almendra (Espagne) par la charge hydrostatique. - Eccentricities (in meters) of the resultants transmitted in arches of Almendra dam (Spain) due to hydrostatic load. - Excentrizität (in Metern) der in den Bögen des Staudammes in Almendra (Spanien) übertragenen Resultanten infolge der hydrostatischen Belastung.

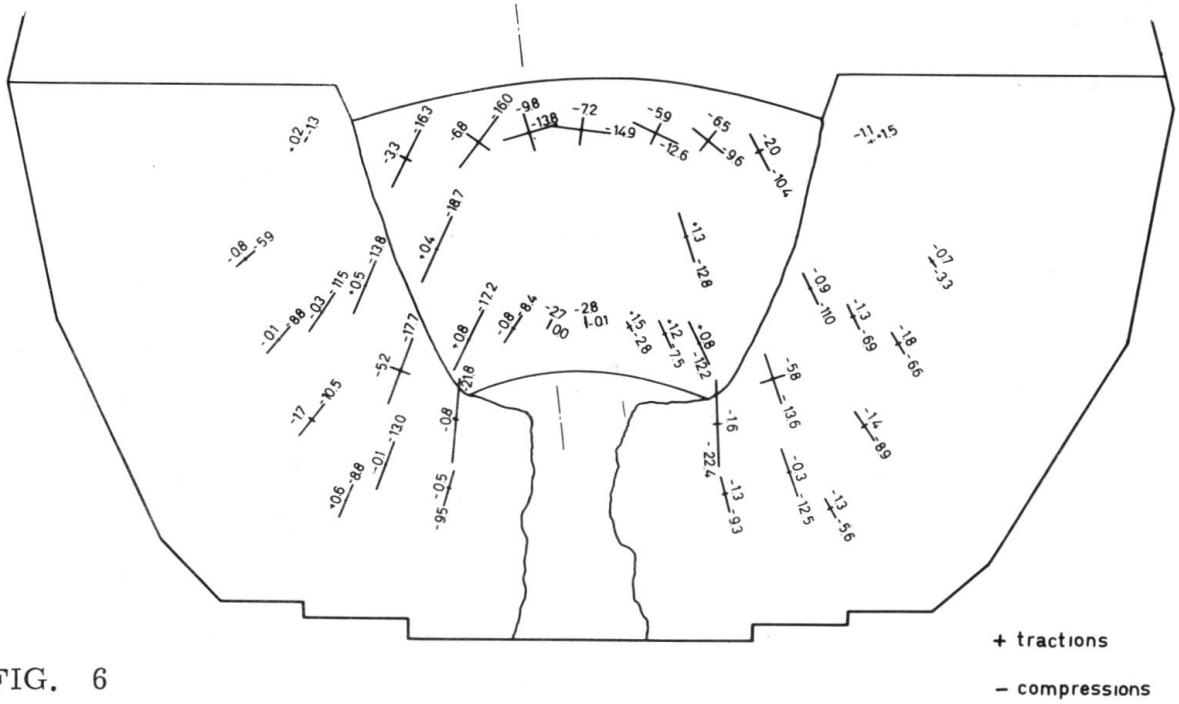

FIG. 6

Distribution des contraintes principales de charge hydrostatique en correspondance d' un arc de tampon.

Distribution of principal stresses (of hydrostatic load) in a concrete plug arch.

Verteilung der Haupspannungen infolge der hydrostatischen Belastung bei einem Pufferbogen.

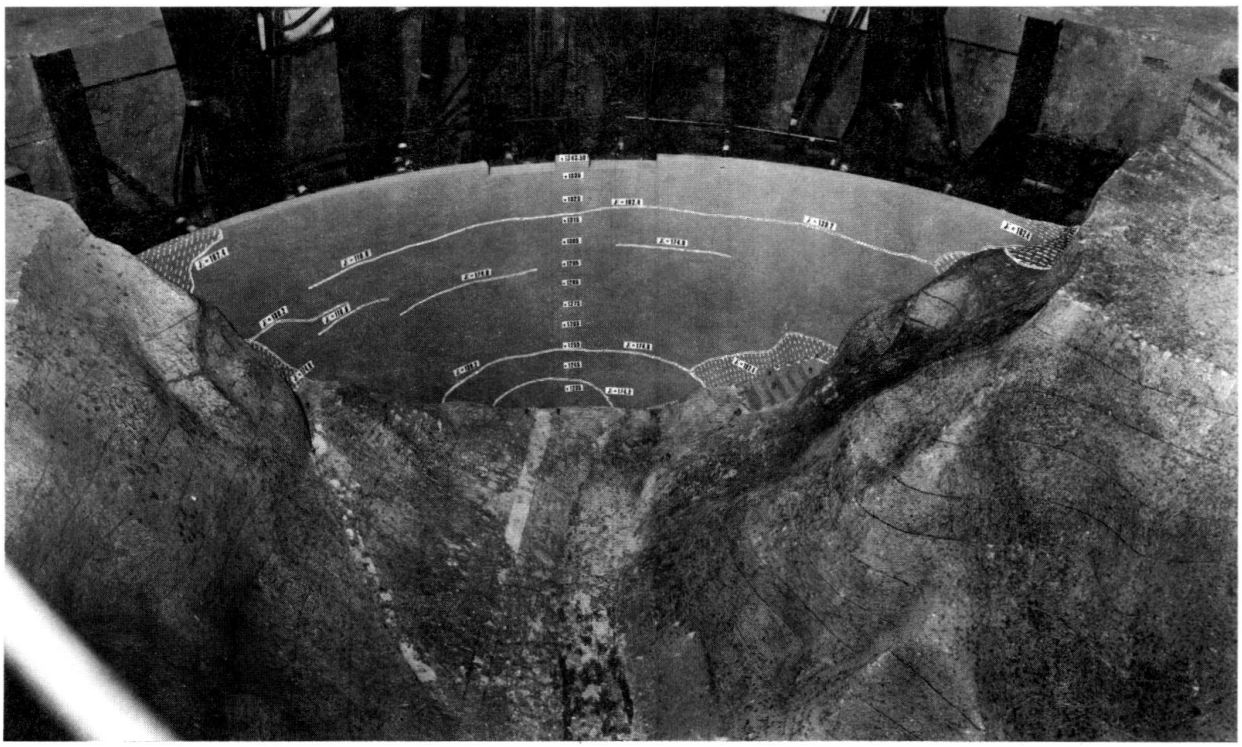

FIG. 7

Modèle du barrage de Agua del Toro (Argentine): cadre des fissures après les essais de charge hydrostatique poussés jusqu' à la rupture.

Model of Agua del Toro dam (Argentina): state of cracks after hydrostatic load tests up to failure.

Modell des Staudammes in Agua del Toro (Argentinien): Rissbild nach dem Versuch der hydrostatischen Belasting bis zum Bruch.

La solution la plus raisonable consiste à employer des matériaux disposant d' un rapport d' efficience $\zeta \simeq \lambda$ et par conséquent d' un rapport $\rho = 1$ ($\gamma = \gamma'$).

Compte tenu des dimensions des systèmes à reproduire, l' échelle géométrique est normalement fixés entre: $50 \leq \lambda \leq 150$, autrement dit, on s' adresse à des matériaux spéciaux disposant de caractéristiques mécaniques extrêmement réduites.

Pour mieux préciser les domaines et les limites d' application des modèles dans le secteur géomécanique, on doit préciser qu' il concerne, en principe, seulement les systèmes en roche cohérente.

Par contre, les modèles du secteur géotechnique ou des terrains, n' offrent qu' une crédibilité très réduite, surtout dans les cas où la présence d' eau revêt une importance fondamentale.

On doit considérer, à ce sujet, les deux processus fondamentaux qui interviennent dans la déformation et dans la rupture d' un terrain: la compressibilité oedométrique (pour laquelle les pressions interstitielles et leurs gradients sont déterminants) et les glissements de couche, auxquels il faut imputer fondamentalement les processus de déformation plasto-visqueux. Les deux phénomènes en question obéissent à des lois physiques et à des paramètres de similitude esentiellement différents, de sorte que le respect simultané des deux phénomènes résulte théoriquement impossible.

Les modèles des systèmes rocheux disposent, au contraire, de une plus grande crédibilité. On doit toutefois également préciser leurs limites d' application. Ils sont principalement employés pour vérifier la stabilité des massifs d' appui aux barrages.

On s' adresse à ces modèles lorsqu' il y a lieu de penser que la rupture finale du barrage puisse se vérifier par tassement ou effondrement des appuis.

La roche-matrice possède généralement des caractéristiques mécaniques élevées et relativement élastiques, et intervient de façon limitée dans les processus de déformabilité et de rupture.

Une importance déterminante doit être attribuée, par contre, aux caractéristiques d' ensemble du massif rocheux avec ses complexes accidents: stratifications, systèmes de diaclases, failles, schistosités , etc.

Les causes d' instabilité du massif son constituées d'ordinaire par les discontinuités les plus importantes et défavorables qui intéressent d' une manière étandue la masse rocheuse.

Le modèle reproduit un schèma géomécanique (figs. 8 et 9) élaboré par le géologue sur la base d' une évaluation critique et conservative de la situation "in situ". Ce schèma doit tenir compte des accidents qui peuvent intervenir, sous l' effet de composantes de poussée appliquées, dans la rupture de l' équilibre statique du système constitué par l' ensemble barrage-réservoir-roche d' appui.

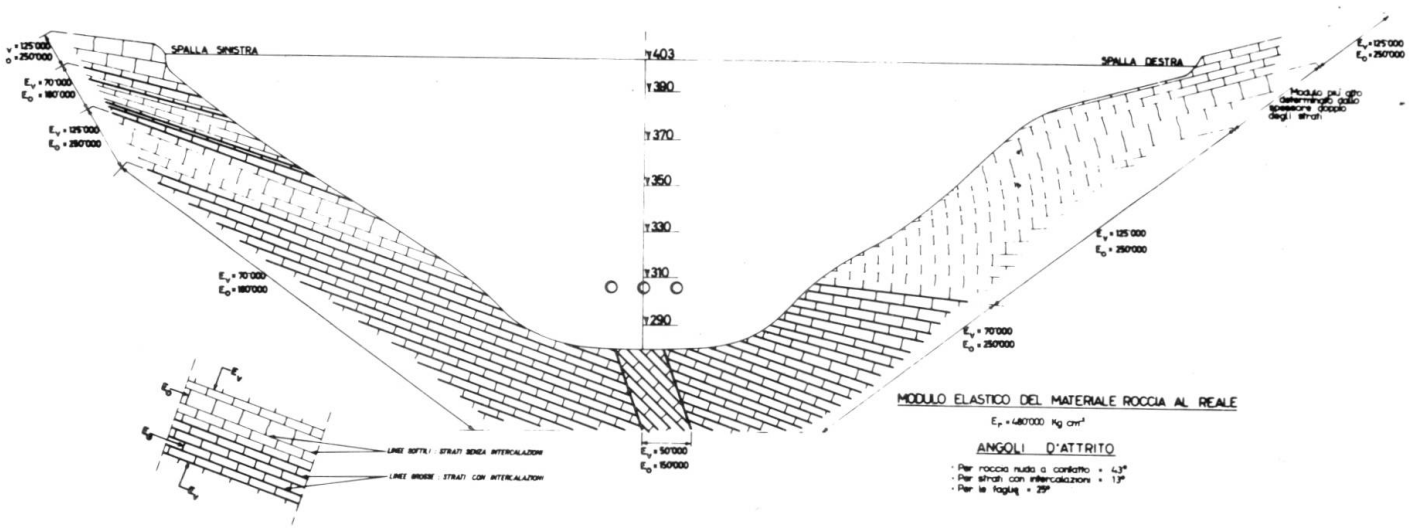

FIG. 8

Schèma reproduit pour le modèle géomécanique du barrage de Grancarevo (Yugoslavie).

Schema of geomechanic model used for the Grancarevo dam (Yugoslavia).

Reproduziertes Schema für das geomechanische Modell des Staudammes in Grancarevo (Jugoslavien).

FIG. 9

Reproduction par briquettes de la roche de fondation du modèle géomécanique de Grancarevo (Yugoslavie).

Reproduction by bricks of a foundation rock of the Grancarevo geomechanical model.

Reproduktion mit kleinen Ziegeln des Gründungsfelsen des geomechanischen Modells des Staudammes Grancarevo (Jugoslavien).

Le schèma, reproduit dans le modèle, est vérifié jusqu'à rupture afin de s' assurer sur l' existance d' un facteur de sécurité de minimum. C' est pour ça que la technique et les modalités de reproduction des discontinuités plus défavorables et importantes deviennent particulièrement délicates.

Si on considère un diagramme contraintes (τ) - déplacements obtenu à l' issue d' un essai de cisaillement sur éprouvette isolée en situ, en correspondance d' un plan de discontinuité de la roche, on constate d' ordinaire la présence d' un pic initial dû à une cohésion résiduelle ou à des forces d' imbrication initiales engendrées par la rugosité de la surface de contact (fig. 10 a et b). Le pic de maximum de résistance une fois franchi, les valeurs des τ se réduisent et la courbe se dispose selon une allure relativement rectiligne, qui donne la valeur réelle de la seule résistance de frottement.

Dans le modèle, en général, on néglige toute résistance de cohésion initiale et les courbes de glissement qu' on obtient entre les surfaces de discontinuité sont du type représenté dans la fig. 10 a' et b'.

On estime, en effet, que la résistance de cohésion est due à des liaisons fragiles et locales que l' on peut considérer négligeables. Il suffit de considérer à ce sujet le travail réduit de déformation nécessaire pour casser ces liaisons, vis-à-vis du travail bien plus important demandé pour les processus de glissement plastique le long des plans de discontinuité.

D' autre part, il faut encore constater l' impossibilité d' une collaboration mutuelle et continue des liaisons de cohésion le long de surfaces très étandues.

En effet, les zones de rupture plastique se répandent spatialement à l' intérieur de la masse, par frontières successives qui s' élargissent graduellement en profondeur et en étendue.

Les valeurs des angles de frottement normalement adoptés dans le modèle, en cas de roche saine à contact, sont comprises entre : $35^{o} \leq \varphi \leq 45^{o}$.

Les conditions sont essentiellement différentes en présence de inclusions (mylonite, argille, tourbe, etc.). Dans des cas pareils, l' angle de frottement se réduit considérablement, selon les conditions d' humidité des matériaux interposés. Vu les valeurs de frottement extrêmement réduites que l' on constate dans les cas-limites ($\varphi \leq 10^{o}$ pour certaines argilles) il va sans dire que leur fidèle reproduction revêt une importance déterminante sur le comportement du modèle et sur sa crédibilité.

Il est bon de remarquer ici que, en présence d' inclusions humides, les processus de déformation par glissement se développent en domaine eminemment visqueux, et par conséquent sont reliés à la variable "temps".

Malheureusement, les lois qui président aux processus de glis

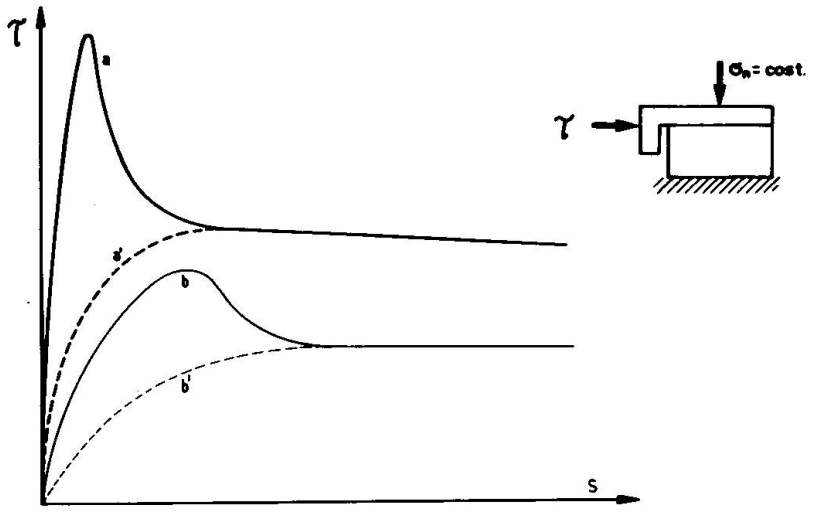

FIG. 10

Essais de cisaillement en roche; schèma indicatif des diagrammes (τ) - déplacements:

Shear tests in rock; indicative schema of stress (τ)-displacement diagrams:

Schubversuche im Felsen: Indikatives Schema der Spannungen (τ) - Verschiebungen Diagramme:

a) courbe relative à roche en contact direct avec cohésion résiduelle
curve inherent to rock in direct contact, with residual cohesion
Kurve bezüglich des Felsens im direkten Kontakt mit resticher Kohäsion

a') courbe reproduite sur le modèle en absence de cohésion
curve reproducted on the model without cohesion
Kurve bezüglich eines Modells ohne Kohäsion

b) courbe en présence d' intercalations plastiques et de cohésion résiduelle
curve when plastic intercalations and residual cohesion are presented
Kurve mit Kunststoffeinschiebungen und restlicher Kohäsion

b') courbe reproduite sur le modèle en absence de cohésion
curve reproducted on the model without cohesion
Kurve eines Modells ohne Kohäsion

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sement visqueux dans le secteur de la mécanique des roches sont encore relativement trop peu connues pour qu' on puisse procéder à une leur reproduction d' une façon croyable.

Actuellement, à l' ISMES, on se limite à relever au cours des essais les déformations finales, à modèle entièrement stabilisé. En d' autres termes, on se limite à vérifier la structure en condition d' équilibre statique, à complet épuisement des processus de déformation.

Malgré ces limitations, il est intéressant de remarquer que les essais sur modèle géomécanique donnent des informations de grand intéret. Chaque tassement plastique local est suivi d' une redistribution générale des contraintes et d' une variation du comportement statique de le ouvrage, de sorte que les essais à outrance permettent de suivre l' évolution graduelle des conditions d' équilibre en domaine de grandes déformations, jusqu' au collapsus. Sous cet aspect, le modèle géomécanique constitue un moyen d' investigation unique en son genre.

Pour les processus de déformation et de rupture d' un massif rocheux il faut encore établir une nette différenciation entre ce qui peut se produire à la surface (où la roche, en état de contrainte biaxiale, offre une résistance dépendant surtout de la cohésion, et où par conséquent la rupture se manufeste comme phénomène essentiellement fragile), et ce qui peut se produire à l' intérieur ou en profondeur (où, en présence de compressions hydrostatiques considérables, la résistance dépend de façon prépondérante de l' angle de frottement interne et les ruptures en plasticité se manifestent sous l' effet des cissions élevées). Au fur et à mesure de l' augmentation de la charge, les zones plastiques intéressent la structure résistante par frontières successives, situées à des profondeurs et à des distances toujours croissantes de la surface d' application des forces.

Le recul de ces frontières fair céder graduellement les liens de cohésion, tandis que les forces résistantes, qui se rattachent au frottement interne du matériau, sont appelées à collaborer dans un cadre de résistance mutuelle, par l' intermédiaire d' une redistribution graduelle du champ des contraintes. Tout cela se produit logiquement en domaine de grandes déformations.

Nous pouvons nous demander quelles sont les conditions d' instabilité qui résultent des discontinuités présentes dans le massif rocheux (lithoclases, failles, stratifications, schistosités, etc...). Si nous limitons notre examen à des considérations d' ordre général, nous remarquerons que les risques d' instabilité au niveau des discontinuités sont d' autant plus grands que ces discontinuités sont plus superficielles. Dans la zone de surface, en effet - nous l' avons dit - non seulement la cohésion manque, mais aussi les états de contraintes triaxiales isotropes importantes y font défaut.

Lorsque, par contre, les surfaces de discontinuité pénètrent dans la roche, la stabilité dépend surtout de l' angle de frottement entre les surfaces de contact.

Lorsque la valeur de cet angle est à peu près égale à celle de l' angle de frottement interne propre de la roche - c' est le cas ordinaire des contacts entre roches vives - le glissement de ces surfaces, si planes et si étendues soient-elles, ne se produit que dans des cas exceptionnels.

Même l' éventuelle présence d' une contribution réduite de résistance apportée par la cohésion n' est pas considérable en pareil cas; la rupture par décohésion se propage, en effet, sans offrir aucune résistan-

ce d' ensemble, et le travail de déformation qu' elle accomplit est extrêmement réduit.

Sur la base de cette remarque, les critères de stabilité conseillent souvent de négliger, dans le modèle, la cohésion dans les plans de discontinuité. Par contre, lorsque l' angle de frottement entre les surfaces de discontinuité s' avère sensiblement inférieur à l' angle de frottement interne, la ruine se produit de préférence par glissement des couches, tout au moins quand ces dernières sont orientées défavorablement. C' est le cas du modèle géomécanique de la culée gauche du barrage de Ca' Selva (fig. 11), où l' on constate, au couronnement, des ruptures fragiles par décohésion entre les surfaces des blocs et fissuration de ces derniers; aux cotes inférieures, on remarque une prédominance de glissements entre les plans de stratification sub-horizontaux, dans lesquels on avait reproduit un angle de frottement $\varphi \simeq 20^{o}$.

Les résultats sur modèle peuvent se démontrer particulièrement utiles pour une comparaison avec les déplacements mesurés sur le ouvrage, au cours de la première mise en charge.

Sur la base du schèma en fig. 13:

- la courbe a) représente la ligne d' inflexion du modèle traditionnel, mesurée au cours des essais à outrance en clé de couronnement. Vu que dans le modèle la roche est normalement reproduite avec emploi de matériau élastique homogène et isotrope et sans discontinuité, la courbe se développe selon une allure pratiquement linéaire jusqu' au seuil de plasticité, auquel font suite les premiers processus de fissuration;
- la courbe b) représente la ligne d' inflexion correspondante du modèle géomécanique qui accuse, par effet de discontinuités, des déplacements plastiques déjà pour des charges relativement réduites.

La courbe de déformation pour le barrage réel (s' il serait possible de la tracer dès à l' origine) est comprise entre les deux courbes citées - au moins si les hypothèses adoptées au cours des essais résultent acceptables. Les deux courbes a) et b) doivent enserrer, en effet, la courbe hypothétique de déformation jusqu' à rupture du barrage et fournissent, de toute façon, les limites extrêmes de dispersion entre lesquel les peut varier le facteur de sécurité de l' ouvrage.

On observe en plus que les deux courbes de déformation présentent des points de résistance maximum auxquels correspondent des déformations pratiquement coincidentes.

Il existe en effet, dans le système, des conditions de congruence entre les tassements plastiques de la roche et les capacités de l' ouvrage en béton de s' adapter par déformation intrinsèque pour suivre d' une façon solidaire les tassements dits. On peut en déduire qu' en domaine de grandes déformations les processus de fissuration et de collapsus de la structure sont engendrés par les contraintes intrinsèques conséquent aux déplacements anélastiques de la roche plutôt que par l' intensité des charges appliquées.

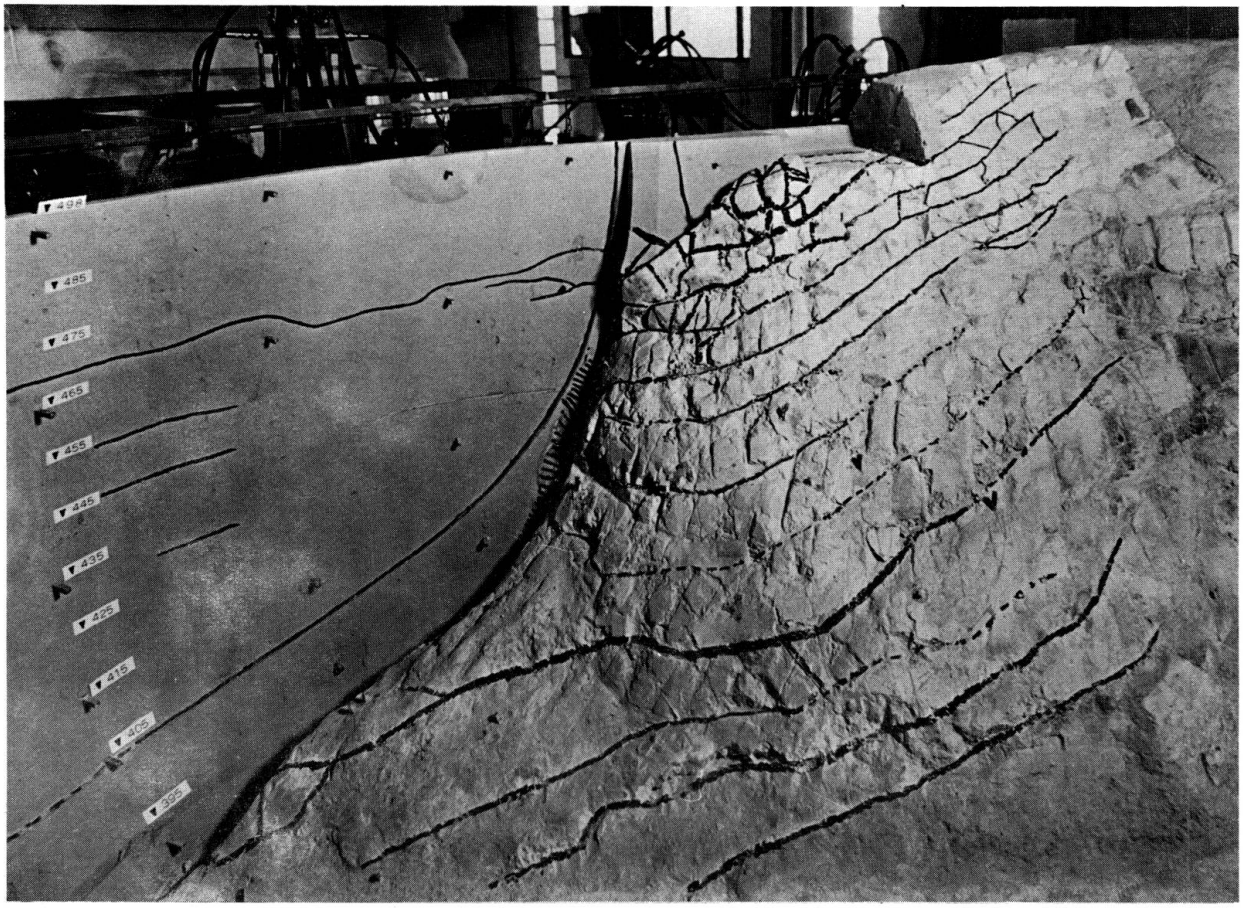

FIG. 11

Modèle géomécanique de Ca' Selva (Italie) essai à rupture, affaisement du rocher par glissement des couches de stratification.

Ca' Selva dam geomechanical model (Italy): after failure test for sliding of stratification strates.

Geomechanisches Modell des Staudammes Ca' Selva (Italien): Bruchversuch, Felsensackung durch Rutschen der Schichtungsflächen.

Il est très intéressant de représenter, séparément, en fonction de l' intensité de la charge appliquée (fig. 12), les déformations globales ① , les déformations réversibles ② , les déformations résiduelles anélastiques ③ . A ce sujet, le diagramme de la composante radiale relatif à la clé de la console centrale du modèle géomécanique du barrage de Susqueda (Espagne) (fig. 12) est très représentatif.

A la charge normale ($\gamma = 1$) les déformations sont de nature éminemment réversible; entre autres, elles ne s' écartent pas considérablement des déplacements mesurés sur le modèle traditionnel. Pour des charges plus élevées, les résidus anélastiques s' accroissent avec la

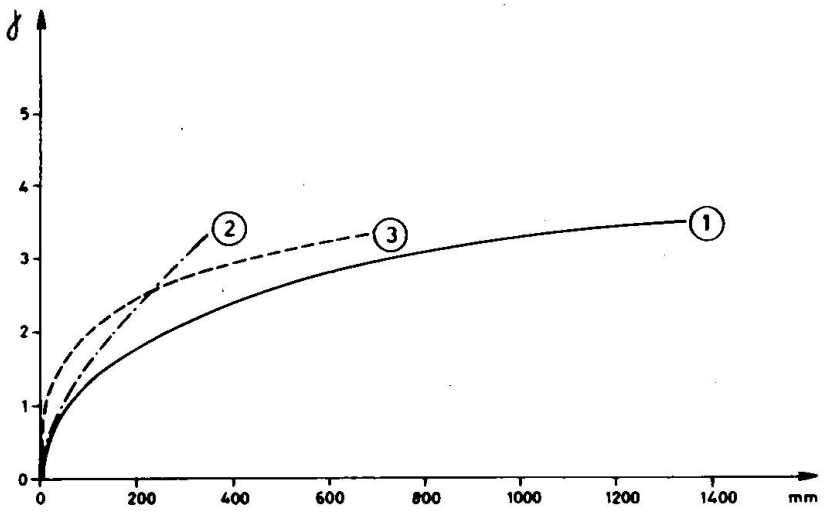

FIG. 12

Courbes des déplacements radiaux mesurés en clé du modèle géomécanique de Susqueda (Espagne):

Curve of radial deflections measured in arch of crest of Susqueda geomechanical model (Spain):

Kurven radialer Verschiebungen nach dem geomechanischen Modell des Staudammes in Susqueda (Spanien):

① déplacements globaux
global displacements
Gesamtverschiebungen

② déplacements réversibles
reversible displacements
Reversibele Verschiebungen

③ déplacements anélastiques
anelastic displacements
Anelastische Verschiebungen

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charge selon une allure considérablement exponentielle, tandis que les déformations réversibles ne s' écartent pas sensiblement de la linéarité.

Nous comprenons de toute évidence, sur la base de ces observations, qu' il peut se démontrer inexact et insidieux de tirer des conclusions sur la sécurité de l' ouvrage de l' examen des seules déformations en domaine élastique, lorsque la rupture se produit par tassement ou effondrement de la roche d' appui.

Bien plus, on a des cas dans lesquels les déplacements anélastiques reliés à la logique des discontinuités sont orientés en direction totalement différente par rapport aux déplacements réversibles reliés à la logique élastique ou des états de contrainte.

Seul un essai poursuivi jusqu' à l' effondrement permet d' ob

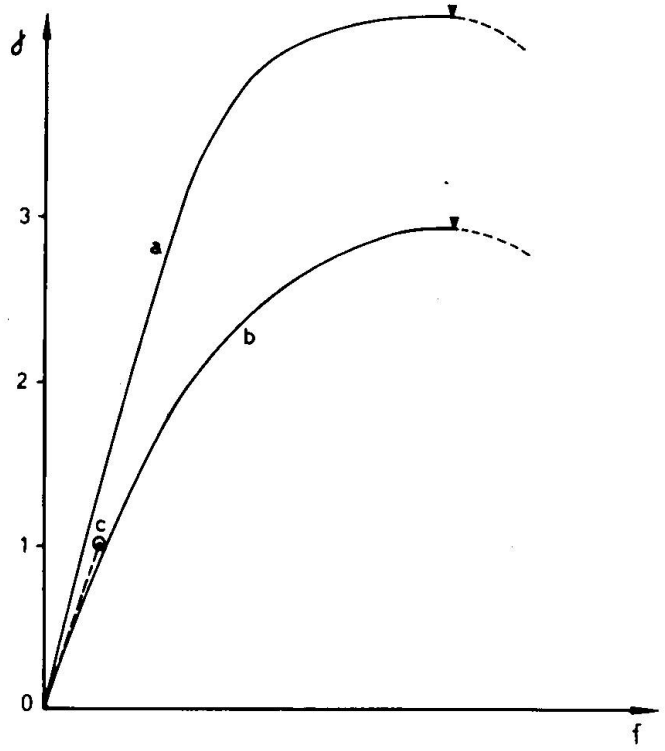

FIG. 13

Courbe représentative des flèches radiales mesurées en clè de couronnement pendant les essais de charge hydrostatique poussés jusqu'à rupture:

Curve of radial deflections measured in arch of crest, during the hydrostatic load test up to failure:

Kurve der radialen Durchbiegungen an der Krone während der hydrostatischen Versuchen bis zum Bruch:

a) modèle statique traditionnel
traditional static model
traditionelles statisches Modell

b) modèle géomécanique
geomechanical model
geomechanisches Modells

c) flèche du barrage depouillée de tout autre effet
dam's deflections due only to hydrostatic load
Staudammdurchbiegungnur wegen hydrostatischer Belastung

---

tenir des informations relatives au massif rocheux, les seules susceptibles de permettre, grâce à une interprétation objective du comportement réel du système barrage-rocher de fondation, la détermination d'un facteur de sécurité établi selon des critères extrêmement conservatifs conséquent à un contrôle d'ensemble du système.

CONCLUSIONS

Dans le rapport on a mis au point les critères d'utilisation des modèles pour structures en béton qui travaillent eminemment pour des états de contrainte triaxiale.

On a constaté, même si dans un cadre très réduit, que tant que il résulte difficile la détermination des contraintes triaxiales et une corrélation croyable entre ces derniers et les modalités de rupture, un essai à outrance sur modèle représente encore la vérification la plus significative sur le réel degré de sécurité de l' ouvrage.

Un secteur particulier est représenté enfin par les essais sur modèles géomécaniques de barrages. Trop souvent, on a oublié le contrôle du massif rocheux jusqu' au point de pouvoir affirmer que, à front des calculs très affinés et souvent sophistiqués sur la structure en béton dans lesquels la déformabilité de la roche aux limites est introduite par paramètres conventionels, on a ignoré totalement les dangers reliés aux processus de déformation et à l' instabilité de la roche au point de pouvoir affirmer "hic sunt leones".

Les accidents, quelquesfois disastreux qui se sont vérifiés , nous obligent à considerer l' équilibre d' ensemble du barrage avec ses appuis et, à ce but, la technique des modèles géomécaniques s' est démontrée très valable comme moyen de recherche d' avant-garde.

RESUME

Le rapport s' occupe des principes sur lesquels se basent les techniques adoptées à l' ISMES pour la reproduction de modèles de structures massives.

Vis-à-vis de la nécessité ou non de reproduire l' effet de poids propre, on peut envisager les trois différents secteurs d' application pris en examen:

1. celui des caissons en béton précontraint pour réacteurs nucléaires, pour lesquels l' influence du poids propre est négligeable;
2. celui des barrages à arcs, pour lesquels l' effet de poids propre est donné artificiellement par cables en acier;
3. celui des massifs d' appui aux barrages (modèles géomécaniques) pour la reproduction desquels on utilise des matériaux très déformables.

Dans le secteur des modèles géomécaniques on traite, en particulier, les techniques de reproduction des massifs rocheux d' appui aux barrages, les critères adoptés pour en vérifier l' équilibre, et enfin, l' intérêt que les résultats - obtenus par les modèles essayés à outrance - revêtent pour vérifier l' équilibre d' ensemble d' un barrage en appui sur un rocher intéressé par importantes discontinuités.

SUMMARY

The paper deals with the principles on which the techniques used at ISMES for modelling massive structures are based.

With reference to the problem of the self weight, three main

application fields have been considered:

1. Prestressed concrete pressure vessels, where the influence of self weight can be neglected.

2. Arch dams, where the effect of self weight is artificially obtained by means of steel cables.

3. Geomechanical models of rock abutements, where very soft materials are used.

This last part deals in particular with the techniques of reproduction of the abutement, the criteria to verify the equilibrium and eventually the interest of the results of models tested up to failure to evaluate the overall equilibrium of a dam with a rock abutement interested by important discontinuities.

ZUSAMMENFASSUNG

Der Bericht befasst sich mit den Prinzipien, die der von der ISMES angewanten Technik für die Reproduktion von Modellen massiver Strukturen zu Grunde liegen.

Was die Notwendigkeit die Wirkung des Eigengewichtes zu reproduzieren, anbetriff drei verschiedene Anwendungssektoren betrachtet werden:

1. Der Sektor der vorgespannten Betonbehälter für Reaktoren, bei denen der Einfluss des Eigengewichtes keine Rolle spielt.

2. Der Sektor der Bogen-Staudämme, bei denen das Eigengewicht künstlich mittels Stahlkabeln wiedergegeben wird.

3. Der Sektor der massiven Stützen an Staudämmen (geomechanische Modelle) für die Reproduktionen, wofür sehr verformende Materialien gebraucht werden.

Auf den Sektoren der geomechanischen Modellen werden hauptsächlich die Reproduktionstechniken der massiven Stützenfelsen für Staudämme, die für die Festlegung des Gleichgewichtes angewanten Kriterien und schliesslich das Interesse für die Ergebnisse - die mittles geprobten Modellen erreicht wurden - behandelt, um das Gesamtgleichgewicht eines an einem Felsen mit wichtigen Diskontinuierlichkeiten gestützten Staudammes festzulegen.

IABSE
AIPC
IVBH

SEMINAR on:
«CONCRETE STRUCTURES SUBJECTED TO TRIAXIAL STRESSES»
17th-19th MAY, 1974 - ISMES - BERGAMO (ITALY)

Triaxial State of Stress "Tiny Walled" PCPV for HTGR. Comparison with a Conventional "Thick Solution"

Etat de contrainte triaxiale dans un PCPV pour un HTGR à parois minces. Comparaison avec la solution conventionelle à parois épaisses

Multiaxialen Spannungzuständen in dem PCPV für HTGR Typ Reaktor. Vergleich zwischen die Dünn- und Dickwandigenlösungen

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1. INTRODUCTION

In the field of the civil nuclear engineering work design stand up the prestressed concrete pressure vessels (PCPV), structures typically subjected to a triaxial state of stresses.

Considering that for the different types of "PCPV filière" a quantity of concrete of the order of 3,000 to 20,000 m^3 and of prestressing steel from 500 to 3,000 ton can be involved, the economical implications arising in fitting a more or a less degree of conservatism in the design hypothesis clearly appear.

In addition, the high values of the ground basis acceleration (that for many Nuclear Plant sites is notable (Caorso, Italy a = 0.24 g)) push the designers to re-examine in deeper detail the margins adopted for the analysis of such structures in order to reduce the imposing masses to be stabilized under the seismic event and, of course, the relevant additional costs.

This paper is related to the Italian contribution to such efforts that involve different research branches and laboratories.

The research is sponsored by the DSR (Research and Development Department) of ENEL (Italian Generating Electricity Board) and devised and coordinated by CPN (Nuclear Design and Construction Center)

of ENEL - Rome with the cooperation of ISMES (Experimental Institute for Models and Structures) Bergamo, and CRIS (Hydraulic and Structural Research Center) of ENEL - Milan, with the aim to give a contribution to the development of more advanced and economical solutions in the topics of the Nucleothermoelectrical plants.

The PCPV that represent the objective of this research are not built in Italy at the present or envisaged in the near future, but they are considered in perspective for the high interest of the problems involved that they represent as structures typically subjected to a triaxial state of stress.

This topic requires in fact the development of the more advanced design tools and basic researches on materials, thus giving a contribution on a better understanding and approaching on the behaviour of the great family of the structures under triaxial state of stresses. For instance, remaining in the nuclear field, the containement buildings.

In this perspective, I have proposed at the 1st and 2nd Conference in Berlin on Reactor Technology (1, 2) a so-called "tiny-walled" solution of a PCPV for a high temperature gas reactor (whose thickness is not less than the limit required for a biological shielding of about 2.5 m) in which, in my opinion, the resources of the concrete triaxially stressed are better utilized that in the conventional solution, called from now on "thick solution".

In line with the topic of this Seminar and as an additional supporting contribution to the proposal, I will give in the following a comment on the design tools selected for the analysis and a comparison between the "thick" and the "thin" solutions.

As far as the general considerations and the collapse state analysis are concerned, I refer to the report that I have presented at the above-said conferences.

This comparison concerns the triaxial state of stress of both the solutions for the typical loading combinations in the working range, that is, in the so-called quasi elastic or linear elastic field of behaviour.

The same comparison was made for the local safety factors calculated on the basis of the Mohr-Caquot method envisaged by the French Code.

Some design philosophy aspects connected with the proposal are also herewith outlined.

2. DESIGNING TOOLS

The true problem for the designer of a PCPV is to define the limits of acceptability of the stress of state of the structure, once satisfactorily defined; these limits, in their turn, will affect the cost.

The stress values, as accepted in the existing codes and standards, impose the construction of oversized structures as proved by the built prototypes and as experienced in our models. In order to elapse from these limitations and better utilize the materials resources, it was necessary to dispose of adequate designing tools able to fill the gap existing in the field of the information on the behaviour of the materials adopted for PCPV in the triaxial stress state.

The first problem consisted in selecting and defining reliable tools to be used easily, rapidly and economically.

Our designings tools are the physical and the mathematical models of the structure, and the rheological model of materials.

More detailed information on these are provided in the Reports (3, 4, 5) which are to be considered an extension of this Report and which are also been presented at the Seminar.

From the complex recycling of the information provided by each of these tools, it was possible to obtain the calibration of each of them and consequently, the check on the reliability and then the quality control of the design.

Both the mathematical and the physical models become essential and complementary design means: the first, to cover the structural design phase (geometries) and the linear behaviours; the second, with the help of the first one (supplying a global information of the elastic phase) to cover the ultimate states, that is, all the non-linear (short time) aspects up to the collapse.

2.1 Physical model

As far as the physical model is concerned, we wish to point out that we should cope with relevant problems to meet the envisaged goals, that is, one model of the same material of the prototype able to provide in short time with limited costs and at the top of the representativity the larger number of information for the whole range of performances from the elastic field to the structural collapse.

These conflicting requirements have been, in my opinion, fully satisfied by our small-scale models equipped with a copper bag liner.

Just to give a feeling about these models, we can say that in nine months we have been able to design, build, and test two models at a cost of the order of 150,000 $.

2.2 Mathematical model

The finite-element method available at the present is a very satisfactory tool to be applied for designing the PCPV structures.

The real problem lies in providing the analyst with the rheological inputs of the material.

From our part, we have tried to solve this problem calibrating the mathematical model on the physical small-scale model, suitably devised for this purpose (axisymmetric model without penetrations (3)). Of course, also the physical models allow the simulation of the reality within certain limits.

The simulation of the actual prototype behaviour for a correct and safe design represent infact the principal problem to be solved. For the constructional stage, for instance, the difficulties arise in modelling the condition of non monolithicity of the structure poured in different steps and portions and along extended periods of time and seasonal variations (of the order of one year and more). This generates problems of differential ageeing for the different portions and, because of the specific dimensions under consideration, problems of shrinkage and thermal effects caused by the hydration heat which impress an indelible mark that will affect all the following behaviours. No to speak of hidden defects of fabrication and their effects on the failure mechanisms.

The problem becomes more and more complex when we would try to simulate the prestressing stage that, for technological reasons, is made in sequences necessarily not axisymmetric involving long periods of time and problems related to the definition of the real friction coefficient and its circumferential variation, obviously again not axisymmetric.

Just to give another example of the design modelling difficulties to be over-come, we know from our small-scale models that in order to obtain stable measurements we must recycle the pressure many times, that means that we need to obtain a structural assessment including for instance some frictional redistributions. Looking at the operational stages, for instance, we should know in advance the heating and cooling, pressurization and depressurization schedule, both in operating and accidental conditions, and what is more important, we should know point by point the time depending governing laws of creep for the different stress state conditions.

We must recognize that the above at now represents, let me say, a topic.

We are therefore obliged to operate a series of choices and simplifications which will enable us to utilize at better all the available calculation tools, making a general synthesis with the scope to sum-up and interconnect all the aspects of the problem, on the safe side, and in the most economic way.

Said design interconnections are going ahead for our research per steps, with the realistic aim to apply the more advanced available tools or/and to devise some new ones in the full respect of an engineeristic and, therefore, productive view of the problem.

For instance, coming back to the mathematical model, the computer programs should provide the designer with an immediate survey of the tensional and deformational state of the whole structure and relevant

safety margins, point by point, and step by step.

The computer programs set up at ISMES are in this respect very efficient as it can be checked from the figures as shown in our papers obtained directly on the plotter.

These plots give an immediate sight of the critical areas to be reinforced, for instance, by appropriate steel reinforcement and, in its turn, the relevant modifications to be applied to the local stiffness and behaviour of materials whose characteristics in the triaxial stress state conditions cannot be analized as per the remaining of the structure.

Stating a design philosophy requiring a so-called elastic behaviour for all the phases of structural performance related to the operating and accidental conditions, we have found that the adoption of a linear semplified model, i. e. a constant elastic modulus E it is already more than sufficient to define in a satisfactory way the probable behaviour of the real structure (as checked on physical models).

We can say, moreover, that the results seem to be also moderately conservative.

2.3 Rheological model

Once derived from the mathematical model the structural state of stress, it is very important to perform a further calibration of this tool, taking into account the results of triaxial test on concrete samples, drawn from the physical model during the pouring stage (3).

At the ENEL laboratories of Niguarda (Milan) a basic research on the behaviour of the concrete subjected to triaxial stress (5) is under development.

In the frame of this general research, said laboratory was asked to cooperate to our research qualifying, at first, the concrete selected for the small-scale physical models (maximum size aggregate 8 mm).

The results of this first investigation has shown that this particular undersized type of conglomerate behaves under triaxial stresses like the full-scale concrete (maximum size aggregate 3 cm), especially as regards the safety aspects, thus confirming the validity of the behaviour of the small-scale models and than their reliability in reproducing the actual structural conditions.

At the present, the cooperation, at its preliminary stage, consists in the application of the advanced know-how developed by the laboratory for the evaluation of the local safety related to the local state of stress of the structure. From this evaluation it is possible to control, per spots, the validity of the safety "maps" computed on the basis of the simplified Mohr-Caquot method (cfr. fig. 5).

In turn, where possible, this information has been recycled to the small-scale models for judgements. It must be remembered that the triaxial tests on sample are carried out on cubic specimens under an

uniform stress distribution state, while in the actual structure the stress distribution is variable.

The complexity of the problems involved in trying to simulate on samples the structural conditions (fairly described in the paper (5)) in order to draw "the constitutive relations" for the given material, asks for a long and very expensive experimental work.

This means that in the future we have to expect supplementary improvements in our design from this research.

3. COMPARISON BETWEEN "THICK" AND "THIN" SOLUTIONS

The following refers to a PCPV solution for high temperature Gas Reactor that will be called "thick", to which design I attended within an international Joint Group, and to my proposal of alternative solution that will be called "thin". For both the solutions, some elements of comparison are herewith given, in order to allow a judgement about the validity of said proposal that seems better utilize the resources of the concrete under triaxial state of stresses.

In the following table some costs relevant to the two solutions are compared showing the great cut-down of the costs and the relevant economical interest arising from this proposal.

	Quantity	Solution		Δ	Costs drop $
		thick	thin		
1 - Concrete					
- Volume	m^3	14,800	5,440	9,360	800,000
- Load	ton	37,000	13,600	23,400	
2 - Prestressing steel	ton	2,700	CPS 3/2 1,000	1,700	3,500,000
			CPS 3/3 1,200	1,500	3,100,000
3 - Reactor Building Volume	m^3	75,400	57,800	17,600	1,500,000

In addition we must consider, the costs drop in: foundations, penetrations, seismic aspects etc. that can lead to total cost - drop amount of the order of 5 - 8 million dollars.

3.1 Aspects of interconnection between mathematical and physical model. Design philosophy for the elastic phase.

Fig. 1 shows the geometrical characteristics of both the solutions and of the four physical 1 : 20 (small) scale models supporting the research.

The general section across the reactor and turbine buildings shows the importance of the seismic problems to be solved.

The reference design inputs data are:

- working pressure $p_w = 40\ Kg/cm^2$
- design pressure $p_{f_c} = p_d = 44\ Kg/cm^2$
- accident pressure $p_a = 48\ Kg/cm^2$
- ΔT across the walls $\Delta = 20^oC$

Fig. 2 shows a comparison, for the four models, with respect to the equatorial and axial deflections, as resulted from the average of symmetrical points examined.

This information, arising from the physical model, allows to perform the calibration of mathematical model and therefore autorize us to give credit to its results for the pressure range p at least between:

$$0 < p \leqslant 1.5\ p$$

This behaviour, that appears from the plot to be "quasi linear" and that is as much valid for the "tick" as for the "thin" structures, comes from the adopted design philosophy to have the structure "fully compressed" for a value of internal pressure:

$$p_{f_c} = 1.1\ p_w = p_d$$

This pressure is normally considered the design pressure for Gas Reactors. As for as the light water reactors B.W. type are concerned, my opinion is that such a value $p_d = p_{f_c}$ shall match with the pressure value corresponding to the opening of the last safety valve, that is:

$$p_{f_c} = p_d \cong 1.25\ p_w$$

3.2 Comparison of the state of stresses in the structure for the constructional and operational phase.

Fig. 3 shows a comparison of the state of stress of structure (principal stresses) between the "thick" and "thin" solutions.

In particular, fig. 3 points out the different specific level of stress values which, in general, are higher, as we have to expect for the "thin" solutions.

On the contrary, it can be noted that both the structures show the same problems of local intensifications for the peculiar points (gussets).

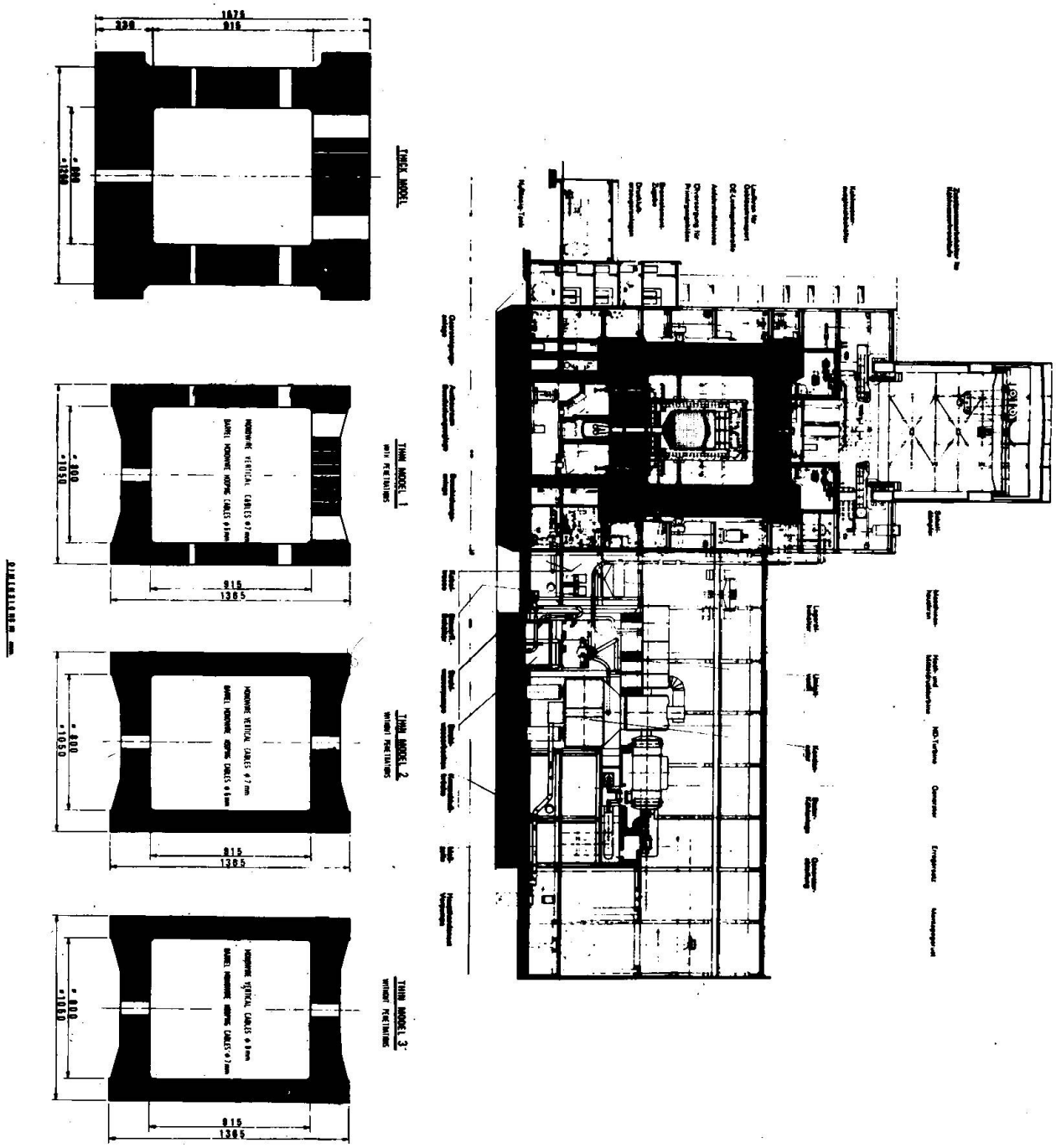
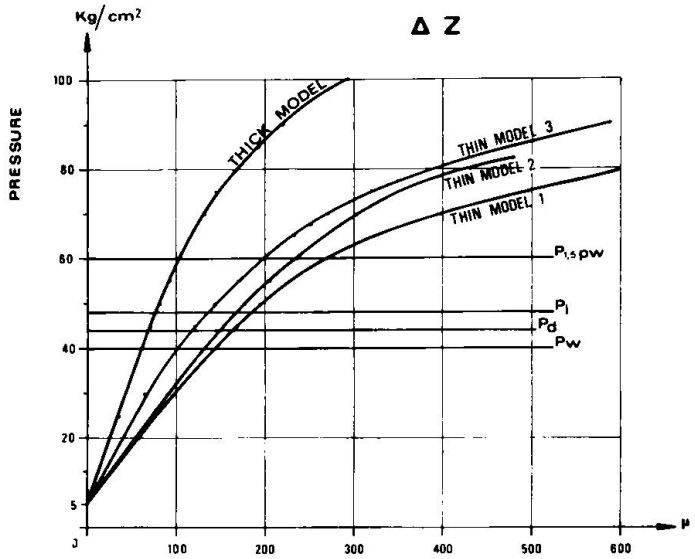


FIG. 1
Geometrical data of "thick" and "thin" solutions.
Données géométriques pour la solution épaisse et mince.
Abmessung beider Lösungen.

P.C.P.V. FOR H.T.R.



TOP CAP AVERAGE DEFLECTIONS AT POINT "A". 1:20 MODEL

EQUIVALENT PROTOTYPE DEFLECTIONS IN μ AT WORKING PRESSURE (40 Kg/cm²)

Deflection type	THICK model	THIN model 1	THIN model 2	THIN model 3
ΔR eq. μ	1422	2252	2280	2162
ΔZ μ	1520	3280	2816	2774

P_w - working pressure 40 kg/cm²

P_d - design pressure 44 kg/cm²

P_l - upper bound pressure 48 " (incident condition)

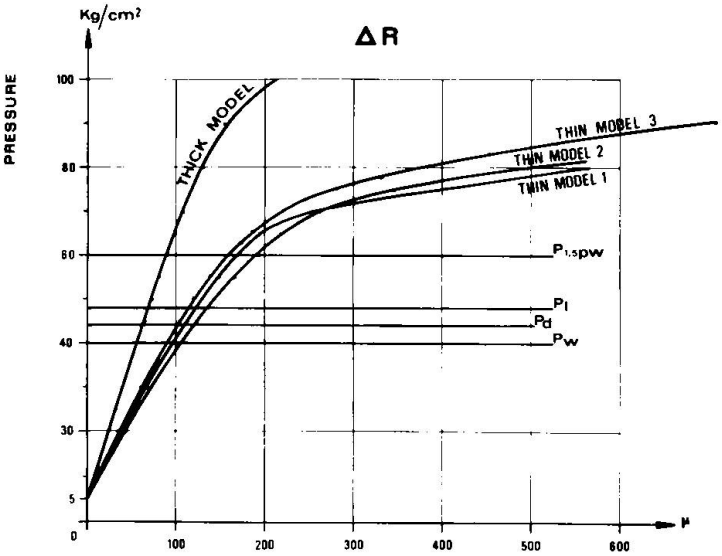
P_f - first clearly visible cracks pressure

THIN MODELS
1) 70 kg/cm² - (with penetrations)
2) 82,5 " - (without ")
3) 90 " - (" ")

THICK MODEL 90 "

P_r - testing reversibility limit pressure

thin 82.5 kg/cm²
thick 116



EQUATORIAL AVERAGE DEFLECTIONS 1:20 MODEL

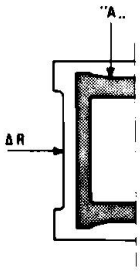


FIG. 2
Deflectionscomparison between the four 1 : 20 scale model.
Comparaisons des flèches des quatre modèles en échelle 1 : 20.
Verschiebungsvergleich zwischen den vier Modellen Massstab 1 : 20.

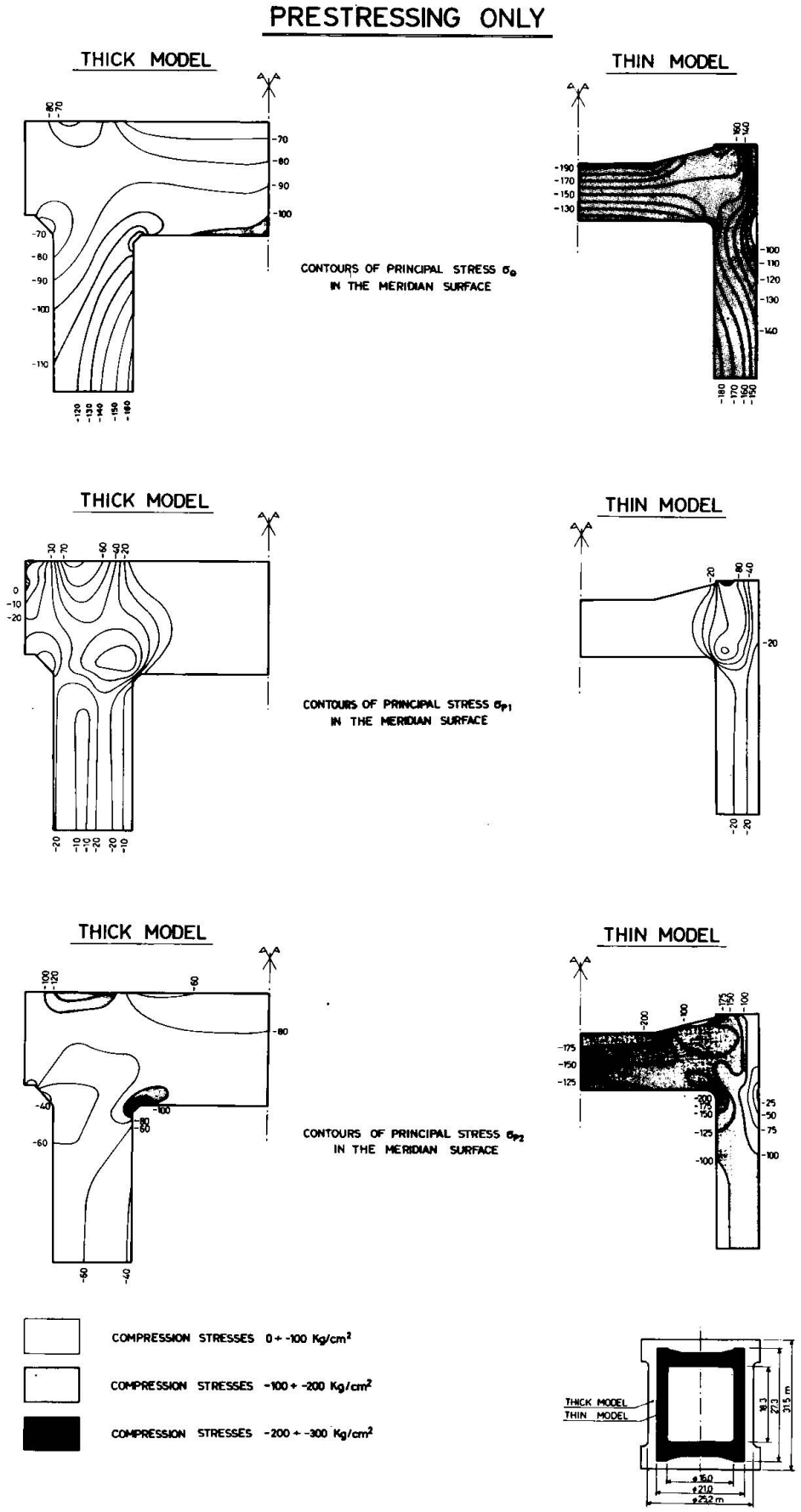


FIG. 3
Comparison between the "thick" and the "thin" solution.
Comparaisons entre la solution épaisse et mince.
Vergleich zwischen der "Dick und Dünnwandigen" Lösung.

The physical models evidenced that the concrete, also at the above-said two different levels of stress perfectly responds to the imposed stress conditions showing a perfect elastic reversibility.

Fig. 4 indicates the same comparison in the normal operating conditions that refer to an equivalent medium-term value of $\Delta t = 10^{o}C$; halved with respect to the design input in order to take into account the medium-term cut-down of thermal stress due to creep (about 1 year).

The thermal loads represent another input that is very difficult to simulate either on the physical or on the mathematical model. On the contrary their effects, are noticeable.

The heating and cooling thermal cycling that can take place during the plant life are theoretically capable even of causing in sequence, due to the creep effects, the inversion of the sign of the thermal stresses.

It is not in my knowledge whether some specific researches have been carried out to ascertain if the known creep laws can be reapplied at any time, in the sense if these laws are still valid even if the creep phenomena have developed their effect for some time, in addition also if the above said creep governing laws are applicable for every combination of the stress ratio in a triaxial state of stresses.

In order to reduce these effects and the problems associated to a correct evaluation versus time of the creep, in my opinion, it is advisable to:

- reduce the wall tickness
- preheat the structure before the pressurization (during the commissioning phase) by means of the liner water cooling system adequately switched to be a water-heating system
- never permit the cooling of the concrete structure during the shut-down conditions in order to exclude inverted thermal gradients across the walls (by means of the said preheating system).

Coming back to the comparison of the state of stresses, it appears from fig. 4 that, even if a certain difference in the specific stress values is maintened between "thick" and "thin" solutions (in the thin solution the specific values are higher), the general trend is practically the same for both structures and the differences are less large.

This is, besides, the prevaling stress condition for more than 90% of structural life, that the physical model warrants to be perfectly stable and satisfactory for both the solutions.

3.3 Safety factors.

Fig. 5 shows the map of the safety factors in the structure defined following the Mohr-Caquot method (3).

From the comparison, it clearly appears that both the "thick"

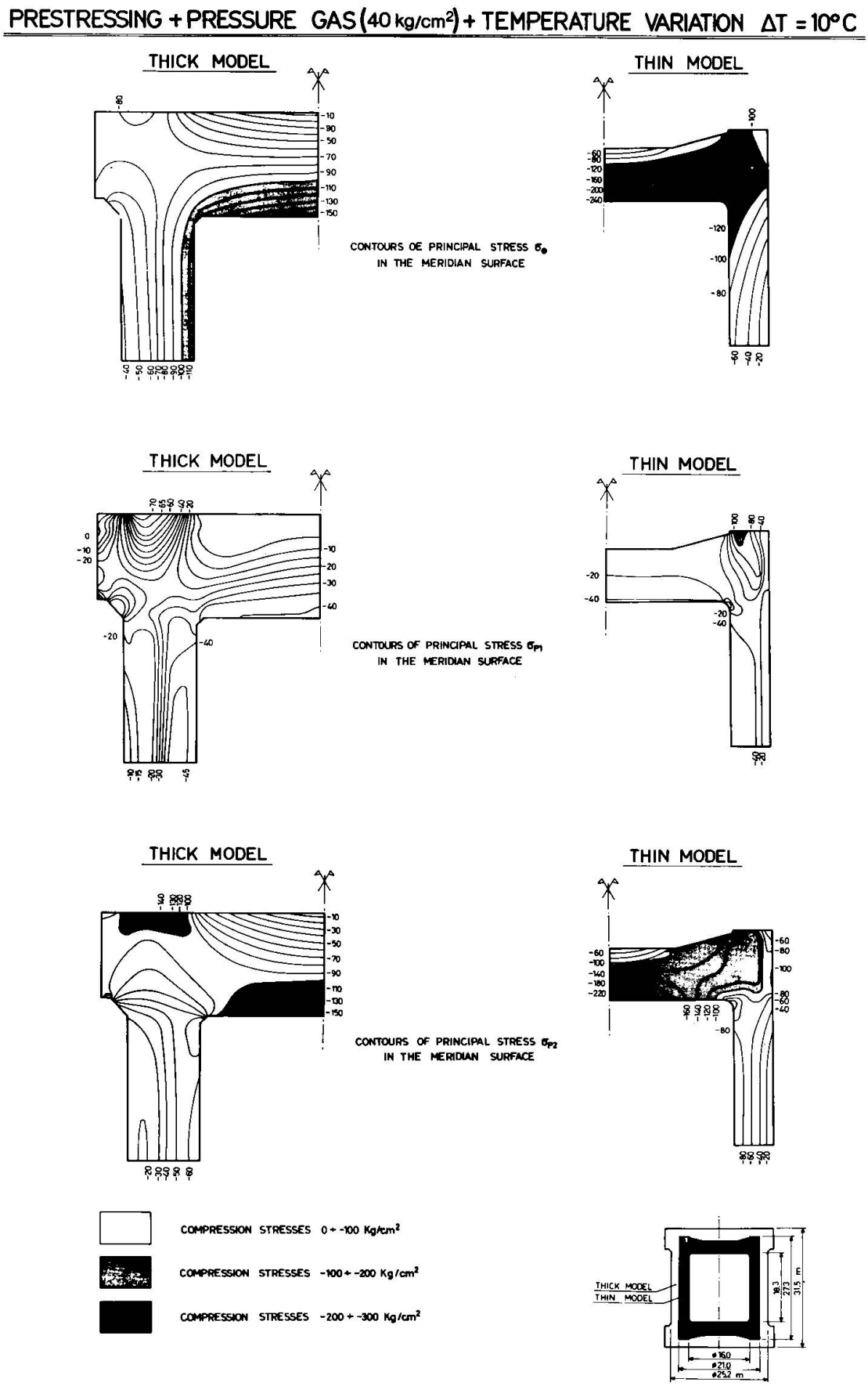


FIG. 4
Comparison between the "thick" and the "thin" solution.
Comparaisons entre la solution épaisse et mince.
Vergleich der "Dick und Dünnwandigen" Lösung.

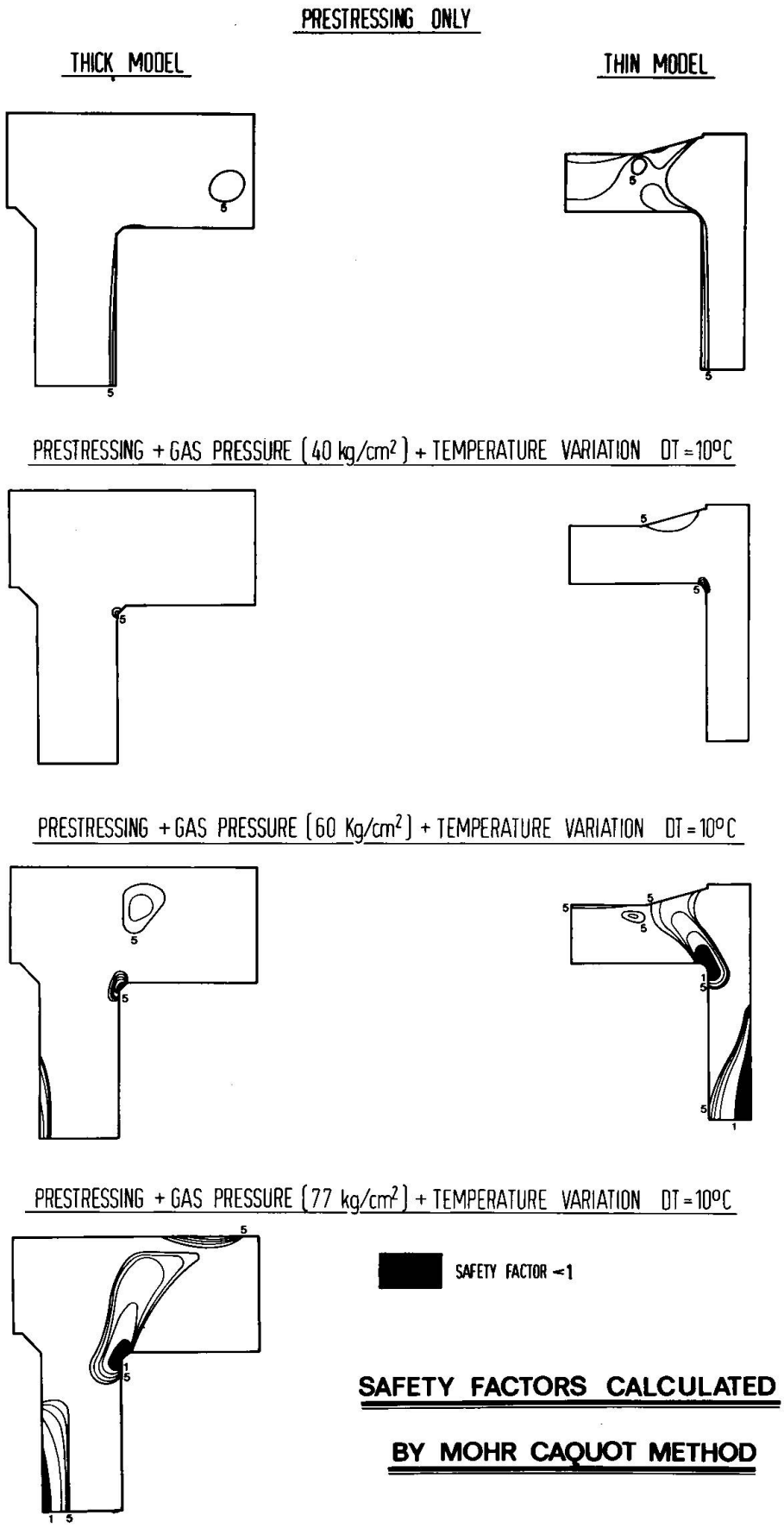


FIG. 5
Comparison between the "thick" and the "thin" solution.
Comparaisons entre la solution épaisse et mince.
Vergleich der "Dick und Dünnwandigen" Lösung.

and the "thin" solutions are provided with large safety margins in the operating conditions, practically the same margins in the barrel for the "pre stressing only" condition and a margin, more reduced but always perfectly acceptable, for the cap slabs for the "thin solution" case.

Taking into account what is evidenced in fig. 7, that both the physical model presents cracks in the external equatorial cortical area of the barrel for pressure values that are in the range of 70 - 90 Kg/cm^2 for the "thin" solution and 90 Kg/cm^2 for the "thick" solution, and the possibility, shown in fig. 2, to accept and extrapolate the elastic behaviour results for pressure of the order of 1.5 times the working pressure and over, we have shown in fig. 5 also the plotter outputs of the safety factors according to the following pressure values:

- thin structure $p_c = 1.5\ p_w = 60\ Kg/cm^2$
- thick structure $p_c = 90/70 \cdot 1.5\ p_w = 77\ Kg/cm^2$

These values were selected in order to make comparable the two solutions considering the behaviour of the physical model.

Taking into account both the plotted values of the fig. 5 and fig. 6, we can notice that:

- the barrel crack pattern in the equatorial area is, topographically speaking, denounced fairly well and, as far as the specific local values are concerned, in a conservative way;
- the "tick" structure is provided with larger margins than the "thin" one, but the relative difference is modest;
- the Mohr-Caquot method is conservative and, at any rate, adequate.

It must be specified that the above-said results refer to the work developed until 1973.

We are now carrying out our safety analysis on the basis of the results acquired from the rheological model and on more advanced methods proposed in the literature which consider also the principal intermediate stress (3).

From the first results in our hands, this check analysis shows that the safety conditions, in the critical areas, are improved (the dimension of the critical areas is restricted and safety factors improve). We consider critical areas those where the safety factor is less than 1.

Moreover, the behaviour checked on physical models is better than the calculated one, in that, the margins in the models look higher.

This is due to the fact that in the reality a stress redistribution occurs, in the sense that a stress migration arises from the critical plastic overstressed areas towards to those (largely predominant in this phase $p \geqslant 1.5\ p_w$) behaving in a linearly elastic way.

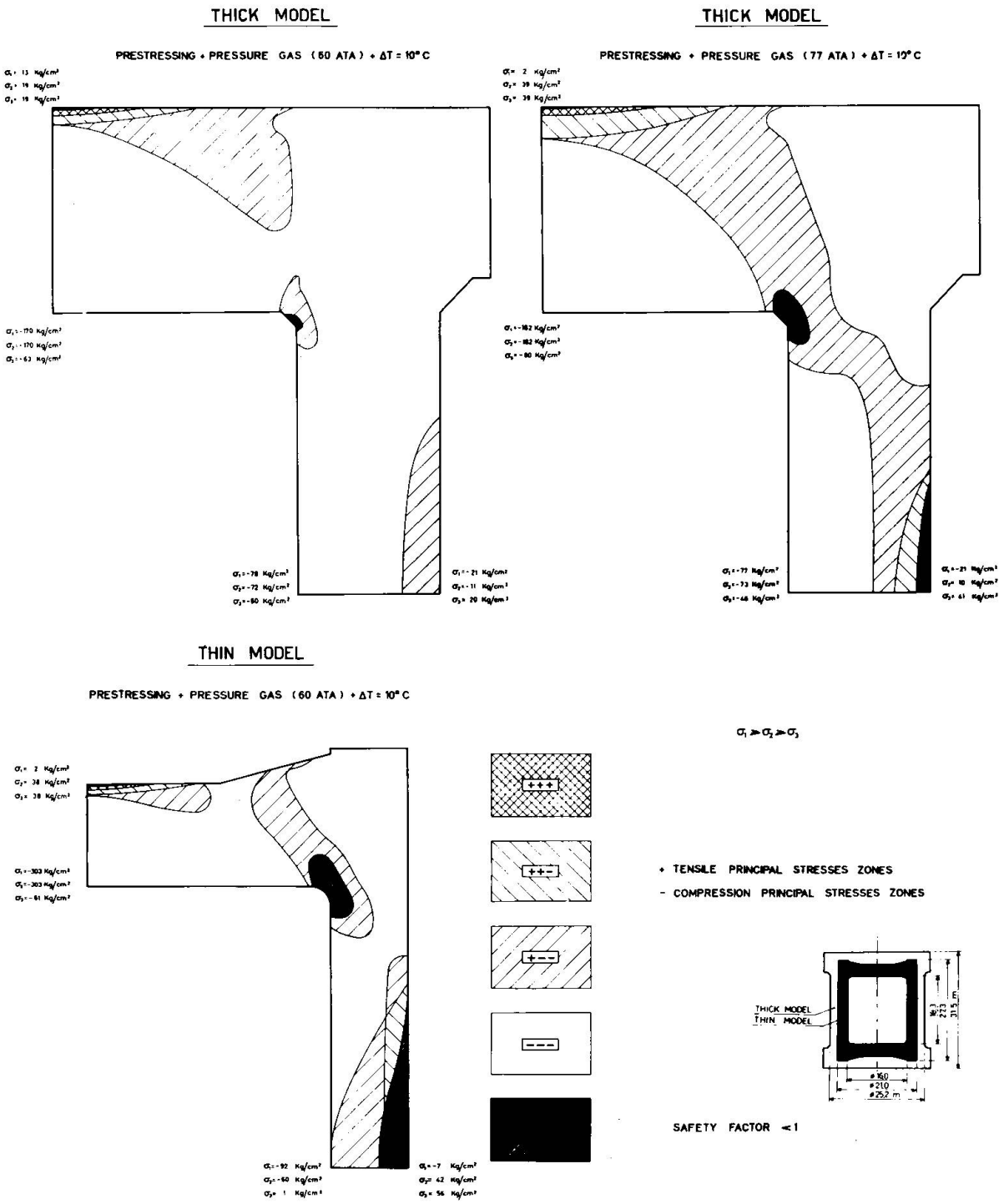
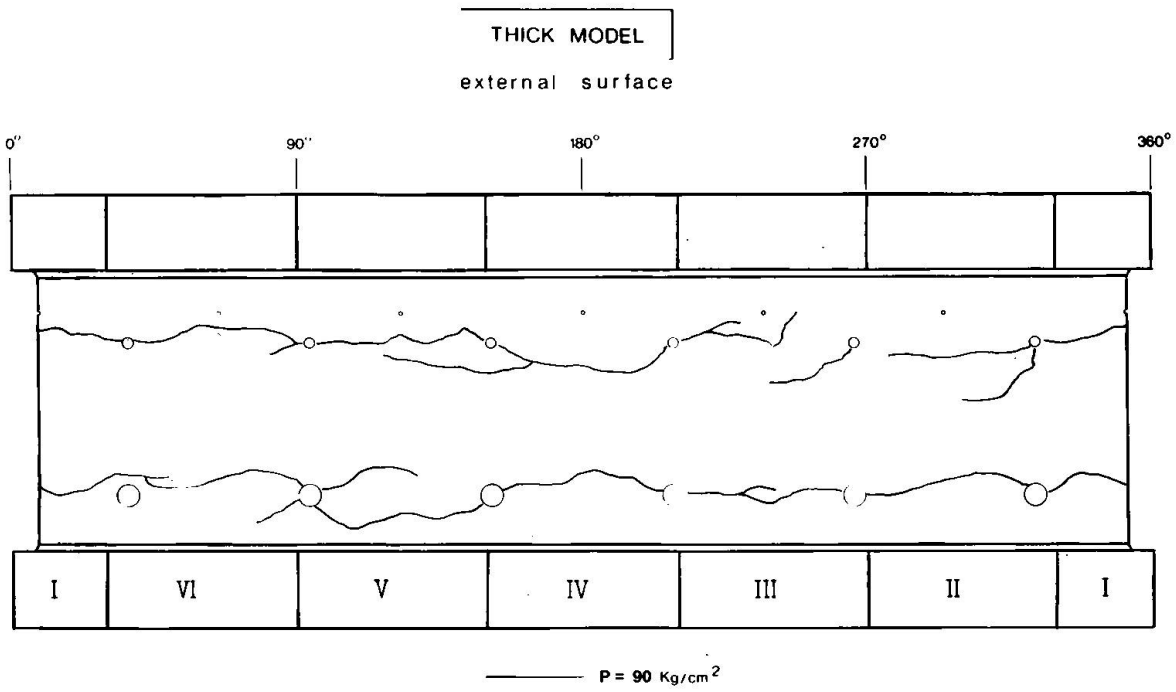


FIG. 6
Comparison between the "thick" and the "thin" solution.
Comparaisons entre la solution épaisse et mince.
Vergleich der "Dick und Dünnwandigen" Lösung.

FIRST CLEARLY VISIBLE CRACKS



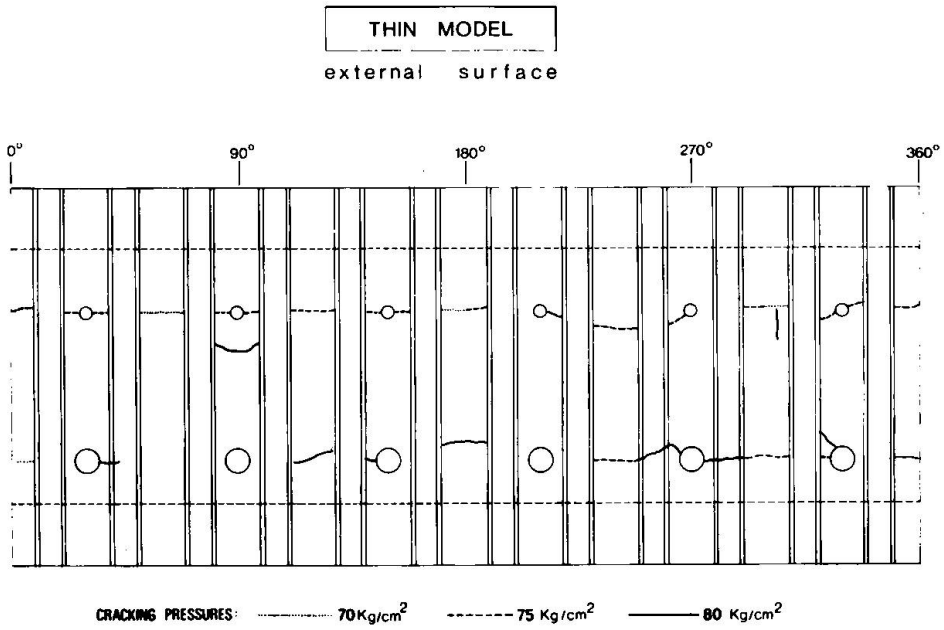


FIG. 7
Comparison between the "thick" and the "thin" solution.
Comparaisons entre la solution épaisse et mince.
Vergleich der "Dick und Dünnwandigen" Lösung.

Very significant in this respect is the behaviour of the cap slabs in the ultimate conditions (as checked on our models) where we assist to the building up of an inverted compressed resistant dome for subsequent partializations per layers (onion type), in the critical plastic tensile areas without impairing the structural integrity.

4. FINAL CONSIDERATIONS

From this necessarily synthetic survey we can, in my opinion, draw the following considerations:

- a problem of a "thick" or a "thin" solution does not exist;
- for a given structure only one design, reliable and satisfactory under all respects. can be provided.

The first point to be considered carefully for an economical viewing of the problem is to try to avoid the superposition of the degrees of conservatism coming from the different technical and safety choices. At first, the design philosophy, shall be stated not only on the basis of a pondered knowledge of all the operating and accidental conditions and their relevant degree of probability, but it must be perfectly acquainted with the structural behaviour so to avoid to ask to the designer unfeasible requirements. One typical example of this is, for instance, offered by the permissible collapse safety factor. This factor is normally stated around 2-2.5 while in the reality it results automatically, at least, higher than 4. For the "thin" solution we were not able to reduce it to less than 3. But the meaningless of this requirements must be sought in the wording "collapse". If we state that for "safety factor" we intend to refer to the value of the ratio between the structural collapse pressure and the working pressure, we do not interpret correctly the safety problem. Indeed we know from our models, on which we experienced the true structural collapse of cables or concrete that, by the point of view of the safety we have to consider a lower pressure level, that is, that level at which the liner integrity (tightness) is lost.

The corresponding pressure level is located in the pressure range of the big deformations (big structural cracks). The theoretical lower bound of this range, in turn, corresponds to the pressure level at which at first the first wire of the prestressing system overcomes its yield limits. Moreover, the true collapse limits are a function of the overpressurization speed that, from our experience, plays a fundamental role in an anticipation of this limit.

When we refer to the PCPV for boiling water reactors, in my opinion, this safety factor can lose at all any significance in case we would size adequately the prestressing cable system.

This sizing must be capable to withstand, within the elastic limits and adequate margins, the pressure levels corresponding to the relief of the safety valves system or/and, because of the high involved cost,

justifying the reasons of such choice, to the higher pressure values, like for instance the pressure at which the leakage through the top lid coupling system begins. Another important point that involves directly the design philosophy of the structure is the problem of the gas permeation through the concrete vessel walls due to accidental liner leakages.

This item can ask for a terrible amount of prestressing and cable layout problems and in my opinion must be, as soon as possible solved and clarified. The more logical proposal is to equip the liner of an efficient suitable gas purging system. At the present, for instance, a triangular distribution of gas pressure across the walls (or rectangular) is envisaged by designers.

Our research has not taken into consideration this kind of phenomena up to now: we intend to consider it in the future. This problem can in fact modify in a drastic way the design and the safety aspects of the structures triaxially stressed.

As far as the permissible stress limits in the structure are concerned, I believe that we shall be prudent and make use of our good engineering feeling considering that this matter is not yet fully explored. In fact, both the so-called "thick" and "thin" structures, present limited regions that would be off-limits under a firm application of the codes. These critical situations are evidenced by the calculation tools, but the physical models prove that these critical areas do not affect at all the overall safety of the structure and, least of all, when a suitable reinforcement is envisaged, thus providing us of the required judgement to accept such conditions.

These considerations must be particularly applied to the cap slab regions grouted of hundreds of penetrations, for which the conventional tools and rules cannot be applied and only the experiments can give us an answer.

5. CONCLUSIONS

Well aware of the present limits of our research work, at any rate, I believe that the results acquired have given evidence to the fact that in the PCPV structures, the strenght reserve, intrinsic of the triaxial state of stresses, can be better utilized. Therefore, I believe that the research work has accomplished our primary objective that was to give a contribution in promoting important cost reductions in the field of nuclear energy and more advanced design methods.

Along these necessarily fragmentary and synthetic paper descriptions, I have tried to put in evidence the importance of defining a suitable design philosophy adequate and aware of the structural behaviour modes.

At the present, our research is oriented towards the PCPV for Boiling Water Reactors, and we will try to apply our experience and im-

prove our techniques on this subject for the future.

ACKNOWLEDGMENTS

I owe warm thanks to the Prof. Ing. G. Oberti for his precious advice during the research. I am also indebted to ENEL - CPN Manager Ing. R. Lepore and ENEL-CRIS Manager Prof. Ing. A. Marazio for the assistance given us and to all the valid contributors.

REFERENCES

(1) F. Scotto "Tiny-Walled 1:20 Prestressed Concrete Pressure Vessel Model for THTR Reactor Type"
Paper H5/4, First International Conference on Structural Mechanics in Reactor Technology, Berlin, September 1971.

(2) F. Scotto "Thin-Walled Prestressed Concrete Pressure Vessel for High Temperature Reactors: Experimental Investigations on Three 1 : 20 Scale PCPV Models and Design Philosophy Proposals"
Paper H4/5, Second International Conference on Structural Mechanics in Reactor Technology, Berlin, September 1973.

(3) M. Fanelli, R. Riccioni, G. Robutti
"Finite Element Analysis of Prestressed Concrete Pressure Vessel"
IABSE Seminar on Concrete Structures subjected to Triaxial Stresses, Bergamo, 17-19 May, 1974.

(4) E. Fumagalli, G. Verdelli "Small Scale Models of PCPV for High Temperature Gas Reactors. Modelling Criteria and Typical Results"
IABSE Seminar on Concrete Structures subjected to Triaxial Stresses, Bergamo, 17-19 May, 1974.

(5) R. Bellotti, P. Rossi "New Prospects for Evaluating the Degree of Safety in Concrete Structures subjected to Multiaxial Stresses"
IABSE Seminar on Concrete Structures subjected to Triaxial Stresses, Bergamo, 17-19 May, 1974.

SUMMARY

The paper is related to a proposal made by the Author at the 1st and 2nd Conferences on Reactor Technology held in Berlin concerning a tiny walled solution of a PCPV for HTGR. With this solution, on the opinion of the Author, the resources of concrete in the triaxial state of stress is better utilized than with the conventional "thick" solution normally adopted.

A comment on the design tools selected for the analysis and a comparison among the triaxial state of stress and relevant punctual safety factors of the "thick" and the "thin" solutions are presented in order to give the possibility to judge about the reliability of the proposed "tiny - walled" solution.

RESUME

Dans ce rapport on décrit une proposition faite par l' Auteur à la 1e et 2e Conférence sur la technologie des Réacteurs Nucléaires tenue à Berlin concernante une solution à parois minces pour un PCPV d' une filière HTGR. Avec cette solution, à l' opinion de l' Auteur, les resources du béton dans l' état de contrainte triaxiale sont mieux utilisées qu'avec la solution conventionnelle à parois épaisses normalement adoptée.

Un commentaire sur les instruments de projet adoptés pour la analyse et une comparaison établie entre l' état d' effort triaxial et les relatifs coéfficients de sécurité de la solution à parois minces et de celle à parois épaisses sont soumises pour donner la possibilité de juger la fiabilité de la solution proposée.

ZUSAMMENFASSUNG

In diesem Bericht illustriert man einen Vorschlag schon in der ersten und, zweiten Konferenz über die Reaktortechnologie in Berlin von dem Verfasser gemacht. Der Vorschlag betrifft eine Dünnwandigenlösung eines PCPV für HTGR Typ Reaktor. Nach dem Verfasser, ist der Beton in den zu multiaxialen Spannungszuständen untergezogenen Strukturen besser als in Dickwandigenlösung nutzbar gemacht.

Der Beitrag behandelt die Berechnungtechniken fur die Analysen und beschriebt einen Vergleich zwischen Dünn- und Dickwandigenlösung unter multiaxialen Spannungzuständen und die bezuglichen Sicherheitfaktoren.

IABSE
AIPC
IVBH

SEMINAR on:
«CONCRETE STRUCTURES SUBJECTED TO TRIAXIAL STRESSES»
17th-19th MAY, 1974 - ISMES - BERGAMO (ITALY)

Concrete structures under combined stresses: a review of canadian research

Betontragwerke unter mehrachsiger Beanspruchung: ein Überblick über Kanadische Forschungsarbeiten

Structures en béton sous contraints triaxiales: vue d'ensemble des recherches conduites au Canada

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Summary :
Research in Canada into the behaviour of concrete structures under three-dimensional stresses is reviewed. Studies involving the response of materials and structural components include comprehensive investigations into confined concrete under static loading; methods of confinement include lateral binders, lateral prestressing and steel pipes. Dynamic loading of concrete which is restrained laterally with binders has also been studied extensively. Research into the creep, strength and fatigue of concrete under multiaxial stresses is reported. The analysis of structures under three-dimensional stresses includes a comparison of the approximate space frame analysis with coarse and fine mesh 3 D finite element analysis when applied to an arch dam. Lastly, results of photoelastic studies applied to a dam and to conduits are summarized.

Zusammenfassung :
Es wird über Kanadische Forschungsarbeiten über das Verhalten von Betontragwerken unter mehrachsiger Beanspruchung berichtet. Untersuchungen über das statische und dynamische Verhalten der Baustoffe und Bauglieder wurden vor allem an Beton durchgeführt, der durch Querbewehrung, Quervorspannung oder Stahlrohre umschnürt war. Zudem wird das Verhalten des Betons unter Kriech- und Ermüdungsbelastung diskutiert.
Als Beispiel für die Berechnung von Bauwerken unter dreiachsiger Beanspruchung wird ein Staudamm als räumliches Stabtragwerk und mit Hilfe eines groben und feinen Rasters von finiten Elementen berechnet und die Ergebnisse werden verglichen. Zum Schluß werden die Resultate von photoelastischen Untersuchungen an einem Staudamm und an Druckrohrleitungen zusammengefaßt.

Résumé :
Vue d'ensemble des recherches conduites au Canada sur le comportement des structures en béton soumises à un état triaxial de contraintes. Des études sur la réponse des matériaux

et des éléments structuraux comprennent des recherches poussées sur le béton fretté soumis à des charges statiques. Le frettage est assuré par des cadres, une précontrainte transversale, l'enveloppement dans des tubes en acier. Le chargement dynamique du béton fretté par des cadres a également été étudié de facon intensive. La recherche sur le fluage, la résistance et la fatigue du béton sous contraintes pluriaxiales est présentée. Le calcul de structures sous contraintes triaxiales comprend une comparaison du calcul approché à l'aide d'un portique spatial et du calcul par la mèthode des éléments finis (maillage spatial grossier et fin) pour un barrage-voûte. Pour finir, des résultats d'études de photoelasticité sur un barrage et des conduites sont résumés.

1. Introduction

Most concrete structures are subjected to multiaxial stresses but for only very few is the multiaxial state of stress analyzed and considered in design. In many cases one or two stress components are so small that they can be neglected; in others the stress components not considered in the analysis are of significant magnitude but are beneficial so that neglecting them does no harm to the structure. Then there are cases where the multiaxial state of stress is expressed indirectly in empirical design equations and is therefore incorporated in the design without being especially mentioned. As an example of the latter case the shear design of flat slabs may be mentioned.

In addition to those structures requiring three-dimensional stress analysis, there are structural elements such as beams and columns which are idealised in the theory of elasticity as one-dimensional elements in a skeletal frame analysis. In these members a triaxial state of stress is introduced by the lateral confinement of the concrete. The beneficial effect of the confinement is normally not expressed in design codes because it does not noticeably affect the strength of the member. But with a general trend towards limit design (plastic design) for reinforced concrete structures the increased ductility resulting from the confinement is of paramount importance. Reinforcement confining the concrete in the directions perpendicular to the axis of compression produces a ductile material out of an otherwise brittle (unconfined) material. Besides the above-mentioned cases there are concrete structures, particularly massive ones such as dams and prestressed concrete pressure vessels the design of which requires complete knowledge of all the components of stress. The analysis of these structures requires highly sophisticated analytical and / or experimental techniques which are generally dealt with by specialists.

Recent research work in Canady concerned with three-dimensional stresses in concrete structures can be divided in two categories :

a) Research involving the response of materials and structural components.

b) Methods of analysis.

The response of concrete and structural components to multiaxial stresses has been investigated chiefly at the Universities of Waterloo and Calgary. The research has included experiments under static, dynamic, fatigue and sustained loading conditions. One of the main purposes of the experimental research was to find the response of confined concrete to axial load. The confinement was either produced by lateral binders (non-prestressed and prestressed) or by steel pipes. The results obtained from prism tests were found to provide accurate information to predict the ductility and for stability of reinforced or prestressed members subjected to axial load and / or bending moments.

Creep deformation under sustained multiaxial loading was investigated and reasonable assumptions for the creep analysis in mass concrete structures were suggested. Failure under triaxial stresses is a complex problem, with concrete changing gradually from a brittle to a ductile material with increasing volumetric (compressive) stress. This observation led to a recent study on the fatigue behavior of concrete under combined stresses at The University of Calgary.

Methods of analysis have been concerned with the three-dimensional finite element method. Analytical research on this topic is reported from The University of Calgary. Finite element programs have been developed to analyze concrete arch dams and other massive concrete structures. The analyses were in all cases based on linear elastic material properties.

2. Research on laterally confined concrete

2.1. General

The following are the chief practical means of confining concrete laterally : (1) to provide binders (2) to wrap the member with prestressed wires and (3) to encase the concrete in a steel tube. The first method is the most common. In a structural component binders support the longitudinal reinforcement during pouring of the concrete, prevent the longitudinal bars from buckling, provide shear reinforcement and confine the concrete in the core. Two extensive studies have been concerned with the effect of lateral reinforcement on the strength and behavior of confined concrete, one under static load (1) (2), the other under dynamic load, more specifically under various rates of deformation (3) (4) (5).

2.2. Confined concrete under static load

The study by Sargin [1] and Sargin et.al. [2] at the University of Waterloo represents a comprehensive experimental program involving the following test parameters : concrete strength; size, spacing and strength of the lateral reinforcement;strain gradient; thickness of cover and casting position. Sixty three prisms, of dimensions 125 x 125 x 510 mm, were tested, twenty two of which were of plain concrete; 14 specimens were tested under eccentric load. No longitudinal reinforcement was used in any of the specimens. The cylinder strength of the concrete varied between 16.8 N/mm^2 and 37.6 N/mm^2. The lateral reinforcement consisted of 5 mm, 6 mm or 10 mm diameter plain or 6 mm deformed bars, or of 0.6 mm or 0.9 mm thick plain steel sheet invelopes. The steel had yield strengths between 260 and 480 N/mm^2. The specimens were tested to failure by applying 10 to 30 deformation increments after approximately 60 % of the expected maximum load had been reached. Test duration was 30 to 90 minutes.

In the analysis of the test results, laterally reinforced concrete is treated as a composite material consisting of core and cover.

The parameters of an analytical stress-strain relationship, which was found to describe the mechanical properties of plain and confined concrete adequately, were determined

using a regression analysis of the test results. A stress-strain relationship of the following form was proposed by Sargin [1]

$$\sigma = k_3 f'_c \frac{A x + (D - 1) x^2}{1 + (A - 2) x + D x^2} \qquad (1)$$

where $A = E_c \epsilon_o / k_3 f'_c$

$x = \epsilon / \epsilon_o$

f'_c = cylinder strength of plain concrete

ϵ_o = strain corresponding to maximum stress

ϵ = strain at stress σ

k_3 = ratio of maximum stress to cylinder strength

D = parameter mainly affecting the slope of the descending branch

For concentrically loaded plain concrete specimens the following values were found (units in N/mm^2) :

$E_c = 5975 \sqrt{f'_c}$

$k_3 = 1.00$

$\epsilon_o = 0.24\ \%$

and for eccentrically load plain concrete specimens :

$E_c = 6530 \sqrt{f'_c}$

$k_3 = 1.00$

$\epsilon_o = 0.30\ \%$

If variations in k_3 and ϵ_o due to strain gradient are expressed in terms of kd , the position of the neutral axis, a unified set of equations is obtained for both the concentric and eccentric loading with

$$k_{3c} = k_{30} \{1.0 + q'' [0.007 \quad 125/kd + 0.015 \ (1-0.25\ s/b_e)]\} \qquad (2)$$

and

$$\epsilon_{oc} = \epsilon_{oo} \{1.0 + q'' [\ 0.25 \quad \sqrt{125/kd} + 0.154 \ (1-0.7\ s/b_e)]\} \qquad (3)$$

where subscripts c and o refer to the core and plain concrete, respectively and

$$q'' = 12.05\, p''_e\, f''_y / \sqrt{f'_c} \qquad (4)$$

p''_e = volumetric ratio of lateral reinforcement
f''_y = yield strength of lateral reinforcement (N/mm^2)
b_e = width of enclosed core (in mm)
s = spacing of ties (in mm)

The parameter D is adequately represented by

$$D = 0.65 - 7.25\, f'_c \times 10^{-3} \qquad (5)$$

The results obtained from Eq. (1) together with the empirical parameters are compared with sthe strength of confined concrete predicted by a formula based on a theoretical analysis using a classical Boussinesq approach. To this end, the following assumptions were made : (1) the material is linear elastic (2) the medium is semi-infinite (3) the lateral confinement acts uniformly along straight lines and (4) the ties yield before or when the concrete reaches its maximum strength. This theoretical analysis resulted in the following equation [1] :

$$k_{3c} = k_{30} + 16.4\, p''_e\, f''_y \left[\log_e (1+R_s^2) - 3 + (3/R_s) \tan^{-1}(R_s) \right] / \pi f'_c R_s \qquad (6)$$

where $R_s = b_e/s$.

Good agreement was observed between the values of k_{3c} determined from the experimentally based empirical formula and Eq. (6) .

Using Eq. (1) and the appropriate parameters allows accurate prediction of the moment-curvature relationship of reinforced and prestressed concrete members (see Fig. 1).

It was concluded in the study that the effect of lateral reinforcement is dependent on the type, spacing, amount and grade of the binders in addition to the quality of the concrete. The spacing was the most important variable. The concrete confined by the reinforcement was improved in ductility and strength but the concrete outside the core is adversely affected.

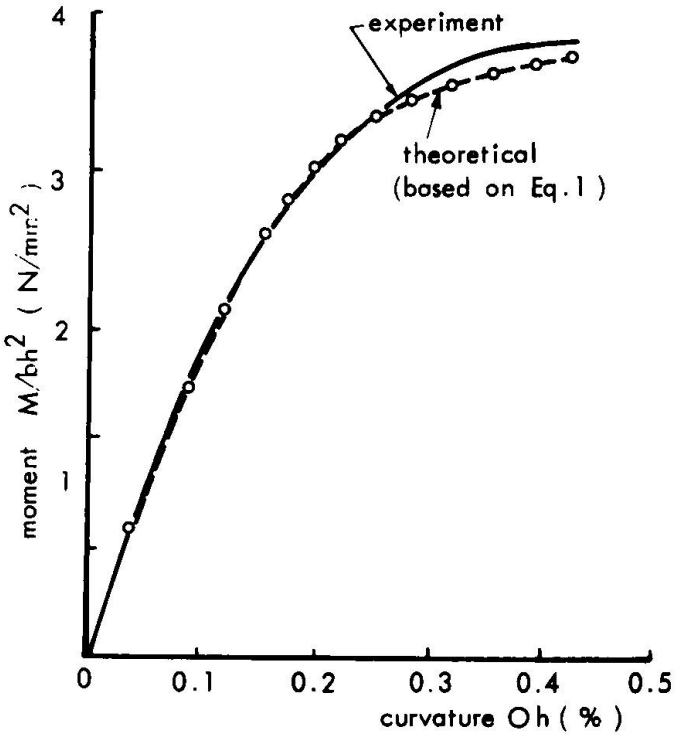


Fig. 1. Moment-curvature diagram for eccentrically loaded column (h = total depth of member)

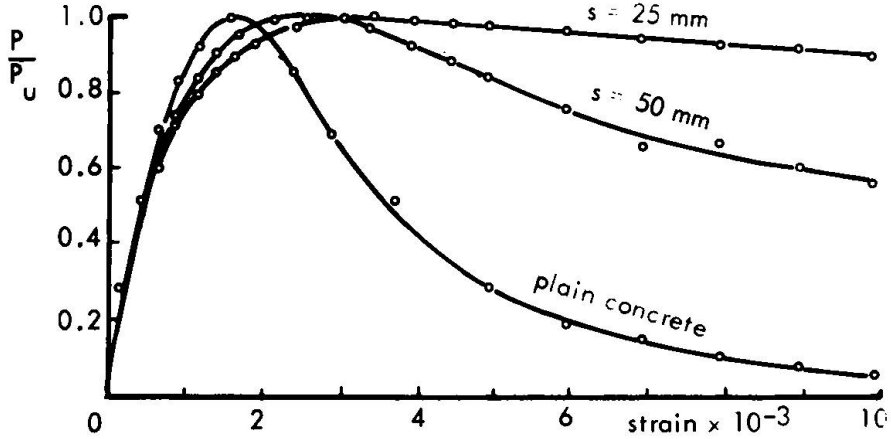


Fig. 2. Stress-strain relationship for plain and confined concrete tested at a strain rate $\dot{\epsilon}$ = 0.32 mm/mm/sec.

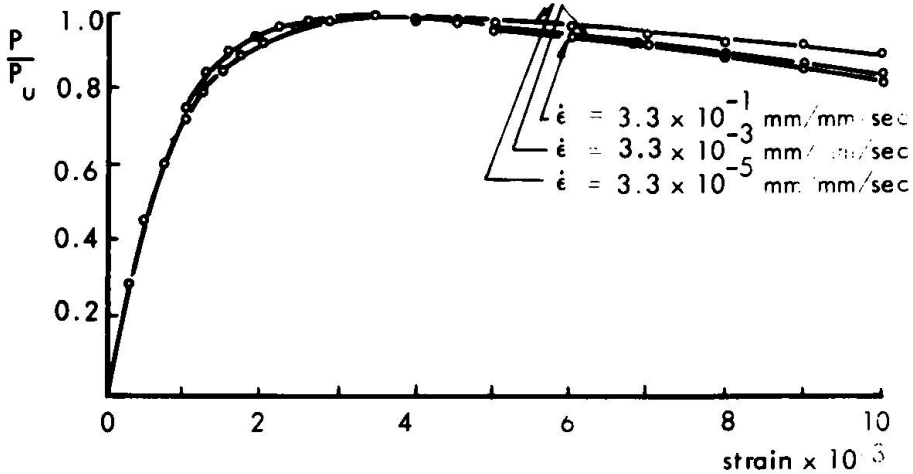


Fig. 3. Stress-strain relationship of confined concrete (spacing of ties 25 mm) subjected to different loading rates

2.3. Confined concrete under dynamic load

The research on transversely reinforced concrete under dynamic load was done during the past five years at the University of Calgary [3] [4] [5]. The main purpose of the research was to investigate the response of confined concrete to various loading rates particularly to extremely high loading rates such as those which occur under catastrophic conditions, for instance in earthquakes and blastwaves. Knowing the response of axially loaded specimens to different strain rates it is possible to establish the response of confined concrete to eccentric loading [6].

The experimental program involved 72 prismatic specimens of dimensions 150 x 150 x 600 mm and the following parameters were investigated : strain rate; type of stirrup (square spirals or discrete ties) and the spacing. The concrete strength was kept constant at about 20 N/mm^2. 24 specimens were not reinforced laterally. The 5 mm plain lateral bars were spaced at 25, 50 or 100 mm and had a yield strength of 248 N/mm^2. The concrete cover was 12.5 mm. The tests were executed in an MTS electrohydraulic closed loop system which allowed a maximum strain rate of 0.33 mm/mm/sec. to be applied to the 600 mm long specimens.

A few typical results are presented in Figs. 2 to 4 : The effect of the transverse reinforcement is shown in Fig. 2. From Fig. 3 it is evident that for confined concrete, the strain rate does not have a significant effect on the shape of the stress-strain diagram. The magnitude of the maximum stress, however, is clearly dependent on the rate of strain as is shown in Fig. 4. From this plot it can also be seen that the effect of the confinement on the strength increase is small compared to the effect of the straining rate.

The results of Ref. (3) have been evaluated by Tadros [6] to incorporate the strain rate in the stress-strain relationship of concrete given by Eq. (1)[1]. To this end the value of the modulus of elasticity E_c was modified according to the following equation:

$$E_c^{(t)} = (1.06 - 0.03 \log_{10} T)\, E_c \qquad (7)$$

where

T = time in minutes to produce a strain of 1000×10^{-6} in the specimen.

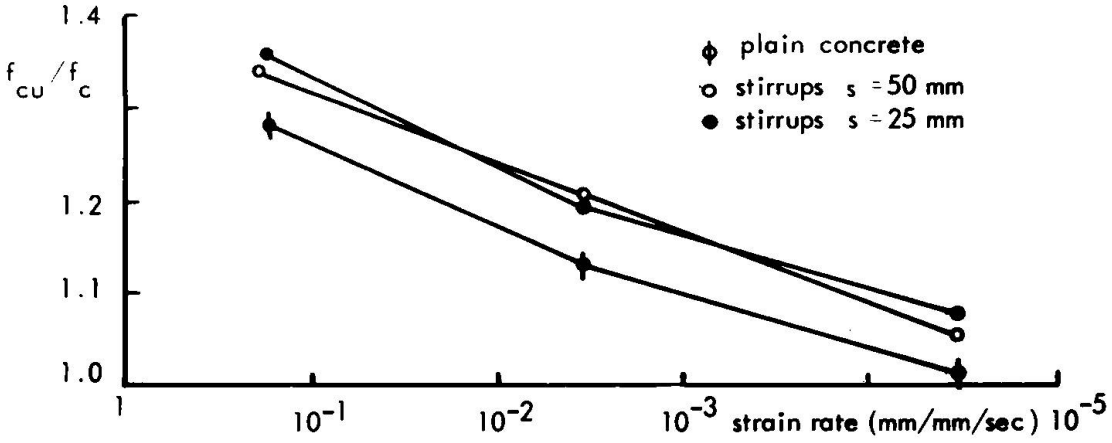
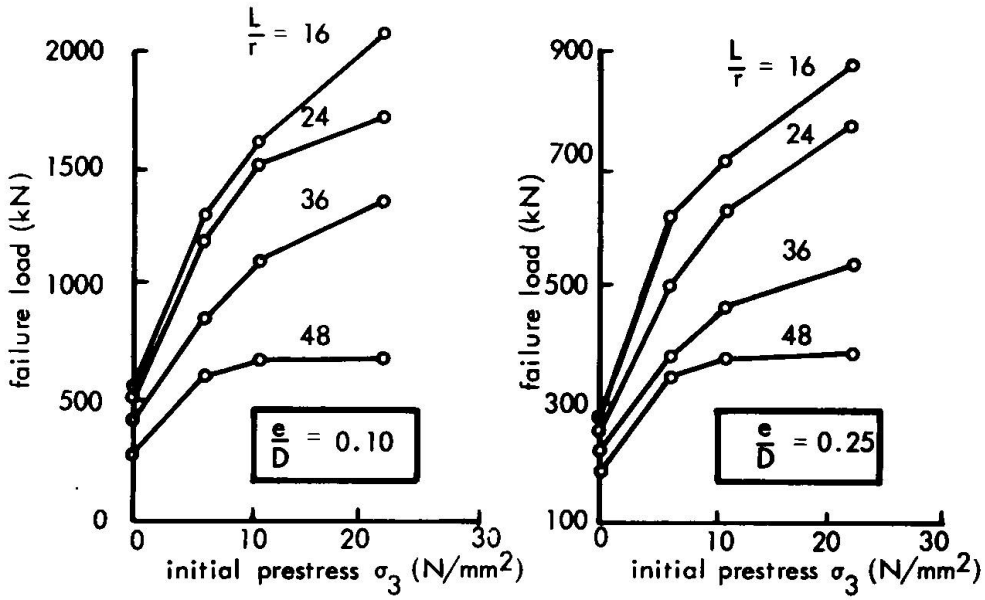


Fig. 4. Relationship between the ratio of failure stress to concrete strength and the strain rate



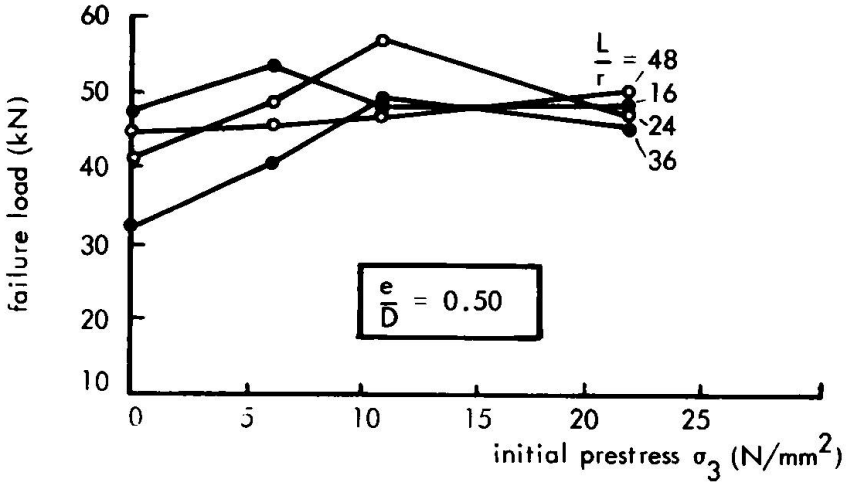


Fig. 5. Variation of failure load with lateral prestress

The value of ϵ_o had to be modified for different strain rates as follows :

$$\epsilon_o = [0.41 + 0.25 \log_{10} T + 1.14 (6.9 f'_c)^{0.24}] \times 10^{-3} \qquad (8)$$

where f'_c is in N/mm^2.

In addition, Tadros [6] introduced the same time factor as in Eq. (7) to take account of the effect of strain rate on concrete strength :

$$k_t = (1.06 - 0.03 \log_{10} T) \qquad (9)$$

With these strain-rate dependent parameters good correlation between the experimental and mathematical representation was achieved.

The mathematically formulated stress-strain diagram was used by Tadros to predict the plastic hinge rotation of members subjected to axial load and / or bending moments; again good results were obtained.

2.4. Laterally prestressed short and long columns

Gardner and his co-workers [7] attempted to take advantage of the increased concrete strength under triaxial compression by applying a lateral prestress to concrete columns.

The lateral prestress was applied by wrapping prestressing wires around the column under tension. The lateral prestress was varied between 6.2 and 22.1 N/mm^2 by changing the pitch of the wrappings. Other parameters investigated were the slenderness ratios of the columns and the eccentricity e of the applied load between e/D = 0 and 0.50, where D is the diameter of the column. The results of this investigation can be summarized as follows :

(1) The load carrying capacity of short concrete columns can be increased substantially by prestressing the column by wrapping it with wires under tension.

(2) The failure of the columns is governed by the maximum lateral stress developed in the prestressing wires.

(3) Short columns with small eccentricities ($e/D < 0.5$) will fail by crushing of the concrete and rupture of the wires simultaneously ($\bar{\sigma}_3 = \sigma'_3$) x)

(4) Columns with large eccentricities ($e/D > 0.5$) or large slenderness ratios will fail before the tensile strength of the wires is reached. ($\sigma_3 \leq \bar{\sigma}_3 \leq \sigma'_3$)

(5) In slender column with large eccentricities ($e/D \geq 0.5$) the transverse prestressing has no beneficial effect. A number of characteristic results are depicted in Fig. 5.

(6) The capacity of cylindrical columns with lateral prestressing can be expressed by the following empirical equation :

$$P_{ult} = A'_c \left\{ 0.85 f'_c + 11.6 \left[1 - 0.715 \left(\frac{\bar{\sigma}_3}{f'_c}\right)^{0.2} \right] \left[1.04 - 0.0056 \frac{L}{r} \right] \left(1 - 0.66 \frac{e}{D}\right) \bar{\sigma}_3 \right\}$$

$$\left\{ 1.042 - 0.0057 \frac{L}{r} \right\} \left\{ 1 - 0.35 \frac{e}{D} \right\} \qquad (10)$$

where $A'_c = \frac{D^2}{4} - 2e\sqrt{\frac{D^2}{4} - e^2} - \frac{D^2}{2} \sin^{-1}\left(\frac{2e}{D}\right)$

for $0 \leq \frac{e}{D} \leq 0.5$

σ_3 = initial stress due to lateral prestressing

σ'_3 = prestress corresponding to the rupture of the wires

$\bar{\sigma}_3$ = final lateral concrete stress at concrete failure lies between initial lateral prestress σ_3 and the value corresponding to the tensile strength of the wires, σ'_3

L = length of column

r = radius of gyration

e = eccentricity

D = diameter of concrete column

---

x) Symbols defined below

2.5. Concrete filled pipe columns

A comprehensive research program on concrete filled steel pipes has been carried out by Gardner and his co-workers [8] [9] [10]. The purpose of this research program was to improve the relevant ACI (American Concrete Institute) and NBC (National Building Code of Canada) design equations, and to find out whether the steel pipe provided lateral restraint to the concrete so that use could be made of the triaxial state of stress in the concrete core.

It was found that the confinement of the concrete by the steel tube does not exist in the initial stages of loading because of the larger Poisson's ratio of steel. In the later stages, particularly towards failure, the lateral deformations of the concrete catch up and hoop stresses in the steel develop which means that the cylindrical steel shell is under a biaxial state of stress resulting from the axial stress $\sigma_{s\ell}$ and the tangential stress σ_t. Assuming that the steel is in a condition of yield when the concrete fails, two extreme bounds are estimated assuming either $\sigma_{s\ell}$ or σ_t' equal σ_y, the yield stress. With $\sigma_{s\ell} = \sigma_y$ (i.e. $\sigma_t = 0$) the capacity of a short column is

$$P_1 = A_c f_c' + A_s \sigma_y \qquad (11)$$

where A_c und A_s are the concrete and steel areas, respectively.

If the steel exerts a circum ferential stress $\sigma_t = \sigma_y$ (i.e. $\sigma_{s\ell} = 0$), the column capacity is

$$P_2 = A_c f_c' + \frac{k}{2} A_s \sigma_y \qquad (12)$$

where k is an empirical factor expressing the effect of lateral stresses on the strength of the concrete, normally k = 4.

By assuming that the maximum shear stress theory holds, i.e. $\sigma_t + \sigma_{s\ell} = \sigma_y$ the column capacity is

$$P_3 = A_c f_c' + \frac{k}{2} A_s \sigma_t + A_s \sigma_{s\ell} .$$

From the experiments it was found that the steel tube yielded first in longitudinal direction, but at failure the steel had yielded also in the tangential direction.

The capacity of the short columns was best predicted by P_3; P_1 was always much lower and P_2 higher than the experimental values.

For long columns it was found that the tangent modulus approach, normally used to predict the buckling load of steel columns, leads to unsafe results because estimating the tangent modulus from a uniaxial test leads to a higher value than that of a column where the steel is biaxially stressed; the presence of the hoop stress reduces the magnitude of the axial stress at which yield occurs. Near yield the magnitude of the tangent modulus decreases rapidly so that a small hoop stress can result in a large reduction of the tangent modulus.

In reference (8) interaction curves are presented for various axial loads and moments; in addition suggestion for design procedures are given.

3. Multiaxial creep, strength and fatigue behaviour of plain concrete

3.1. Creep of plain concrete

It is necessary in the analysis of structures for the effect of creep to have reasonable assumptions of material behaviour in mathematical form. In steel and other metals it is customary to assume that the response to volumetric stresses is elastic and that creep is entirely deviatoric - in other words that the volume does not change as a result of creep. Early work showed that this assumption is not valid for concrete but several investigations subsequently showed conflicting results. Difficulties in experimentation were encountered because of the very small strains to be measured and because of platen effects at the interface between the concrete and the loading device (11). It became clear, however, that the Poisson's ratio for creep was between zero and the static elastic value.

The moisture condition of the concrete is a most important variable in considering the axial compliance and it is to be expected that it will influence the lateral compliance as well. For sealed concrete the assumption of a constant Poisson's ratio equal to the static elastic value is reasonable as shown in the strain predictions in Figs. 6 to 8 from the work by Jordaan and Illston [12] [13]. For cases where the concrete is not sealed, a direct method for three-dimensional analysis is discussed in a personal contribution [14] in which the effect of drying on the analysis is outlined. It is shown that, even though it is not exact, the assumption of a constant Poisson's ratio is reasonable even for drying concrete. The complications involved in using a more sophisticated assumption are not justified by the additional accuracy obtained, except perhaps in exceptional cases.

3.2. Strength and fatigue under triaxial stresses

A series of static tests in which strengths and deformations of plain concrete were determined at the University of Western Ontario has been described by Gardner [15]. Twenty eight specimens were tested in a triaxial cell and the results were compared to an approximate theory based on the strength and deformational characteristics of a packing of praticles under triaxial stress. Reasonable agreement was found at lower stresses (less than 80 percent of ultimate); with a modification the theory reduces to the Richart-Brandtzaeg-Brown formula

$$\sigma_{1\,max} = k_3\,\sigma_3 + k_4$$

in which $\sigma_{1\,max}$ = ultimate axial stress on cylinder with lateral confining pressure σ_3 and k_3, k_4 = constants.

An investigation was undertaken at the University of Calgary [16] to determine whether the fatigue life of concrete is dependent on the state of stress of the concrete. Cylindrical specimens were subjected to an constant radial confining pressure and axial fatigue loadings. The 102 mm diameter by 305 mm cylindrical specimens were sealed and the loading age was five weeks.

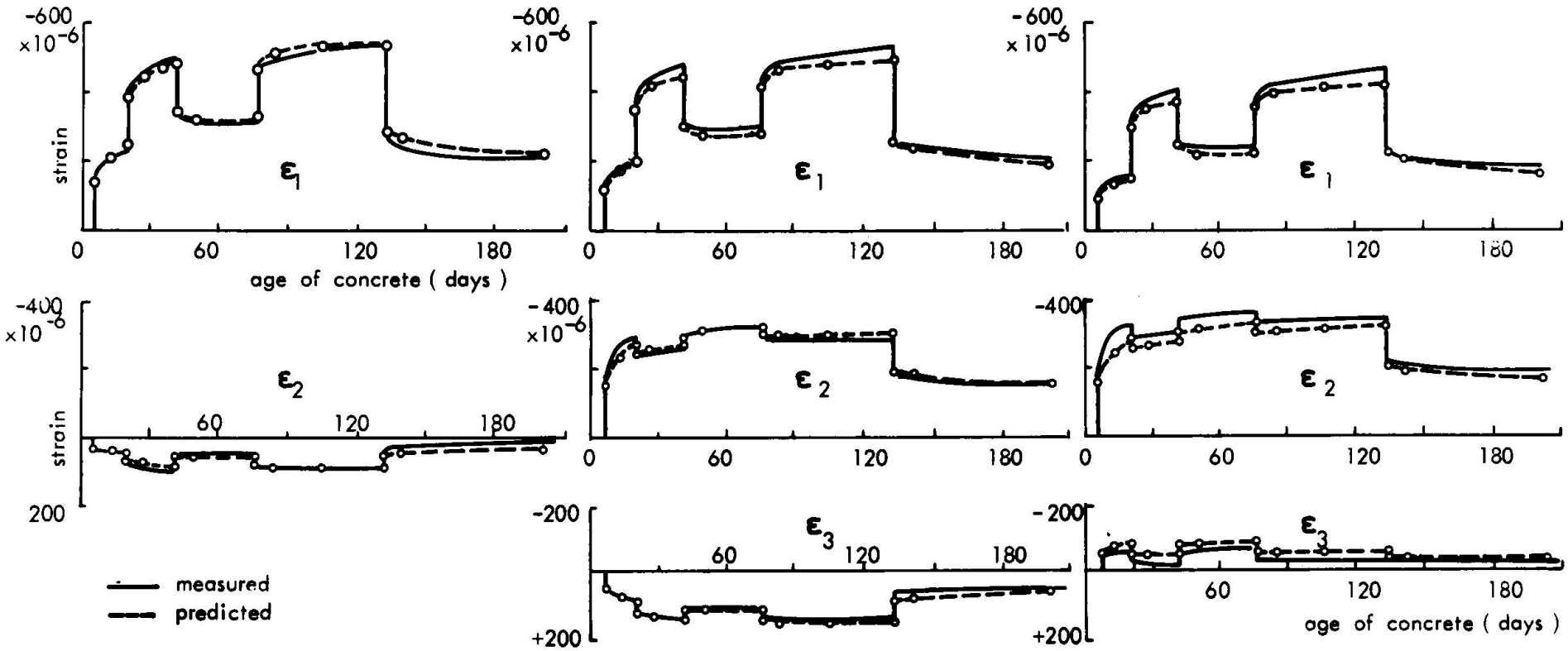


Fig. 6. Strain vs. time plot for uniaxial loading

Fig. 7. Strain vs. time plot for biaxial loading

Fig. 8. Strain vs. time plot for triaxial loading

The tests were performed in a triaxial cell in which volumetric pressure was applied by means of hydraulic fluid while the axial load on the cylinders was applied mechanically by an MTS electrohydraulic loading system.

The static triaxial strengths were determined in static tests at the same confining pressure as the corresponding fatigue test; the minimum stress in the fatigue test was 20% of the static value while maxima of 80 %, 85 % and 90 % were included in the test series. The frequency of load application was in general kept constant at 60 cycles per minute.

Results of the tests are summarised in Table 1; analysis shows that at the 90 % stress level there was no significant effect on the fatigue life, while at the 80 % level a significant improvement resulted from the imposition of lateral confining pressure. The report suggests that the fatigue behaviour is affected by the change in the local state of stress at crack tips, thus suppressing the propagation of cracks. The state of stress is therefore an important variable in the study of fatigue of concrete.

TABLE 1 RESULTS OF FATIGUE TESTS

Lateral pressure, N/mm^2 (psi)	0		6.9 (1,000)			13.8 (2,000)	
Maximum stress level as a proportion of the static strength	0.80	0.90	0.80	0.80	0.90	0.80	0.90
Average number of cycles sustained, N	818	28	90,280 15,000)xx	1,675 (57,140)+	158	(16,750)x	19.2
Average of $\log_{10}N$	2.71	1.38	4.92 (4.18)	2.90 (4.76)	1.83	(4.19)	1.22

x Five run-out specimens
xx Five run-out specimens
+ One run-out specimen

4. Analytical and Experimental Analysis of Three Dimensional Concrete Structures

4.1. Three-Dimensional Finite Element Analysis of Concrete Structures

The finite element method is being used more and more extensively in the analysis of three-dimensional structures such as arch dams, or pressure vessels for nuclear power stations. In a paper entitled "Analysis of Arch Dams Using a Space Frame Model" Skjolingstad and Cheung [17] examine the stresses in an arch dam using space frame analogy and compare the results with those obtained from a coarse and fine mesh finite element analysis.

In the analysis the isoparametric hexahedron finite element with 20 nodal points has been used. This element has been proven among other by Skjolingstad [18] to be one of the most accurate and convenient means of analysis for three-dimensional solids. The deflections and stresses for the space frame were obtained utilizing the STRUDL-1 computer program. The doubly curved arch dam of Fig. 9 is used for comparison. Fig. 10 shows the location of the horizontal and vertical members for the space frame analysis and Fig. 11 shows the division of the dam into 32 and 15 elements respectively. From the comparison of the result in Fig. 12 it is evident that the space frame gives in most cases a reasonable estimate of the magnitude of the stresses, which is sufficiently accurate for a preliminary design of a dam. For the final design , however, a fine mesh 3 D finite element analysis is recommended by the authors.

4.2. Photoelastic Analysis

In addition to analytical methods, photoelastic methods to study the three-dimensional behaviour of structures have been used in Canada. A paper by Jones and Mantle [19] on "Three-Dimensional Photoelastic Analysis of a Diamond-Head Buttress Dam" describes a study on a model of the Squaw Rapids Power Development on the Saskatchewan River. The dam (see Fig. 13) was analyzed for water pressure and earth fill at the upstream face and for earth fill from elevation 295 m down at the downstream face. The frozen-stress technique was adopted to obtain an indication

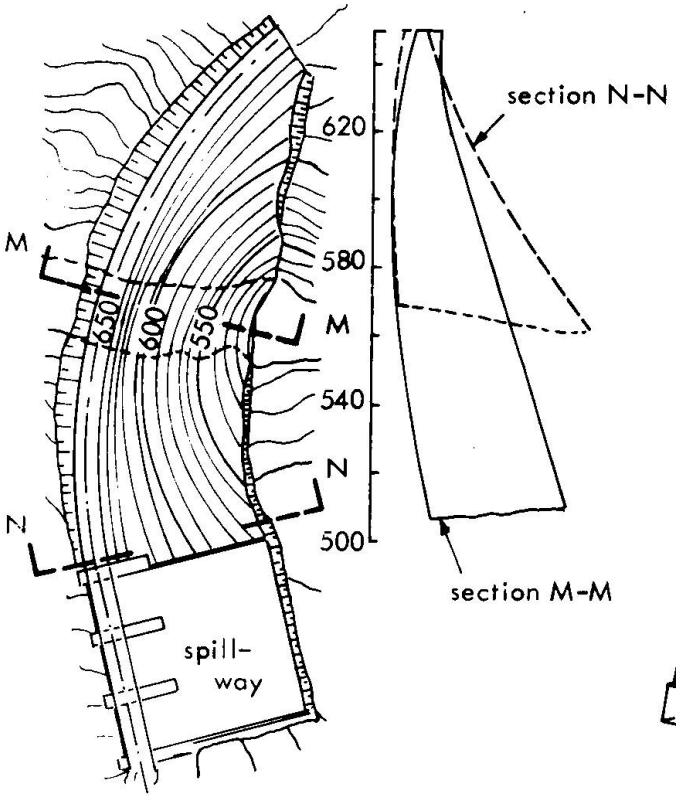


Fig. 9. Plan and cross section of the arch dam used for analysis

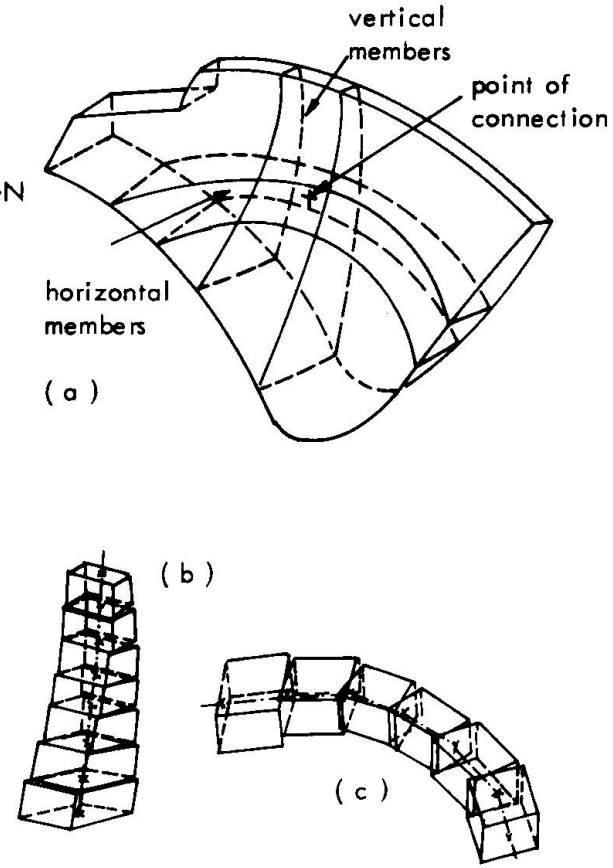


Fig. 10. (a) The arch dam showing the location of (b) typical vertical and (c) typical horizontal members

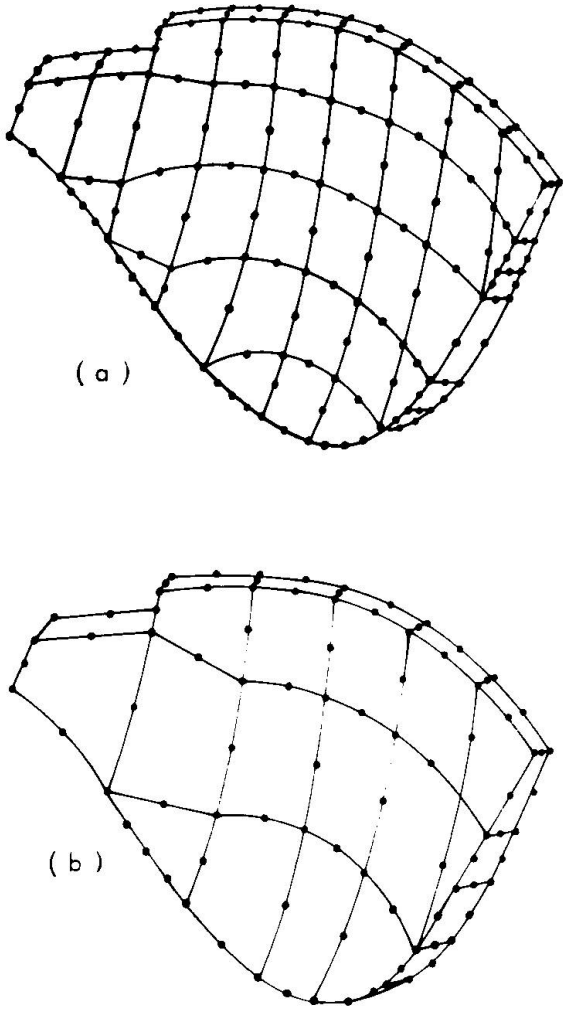


Fig. 11. (a) Fine and (b) coarse finite element mesh

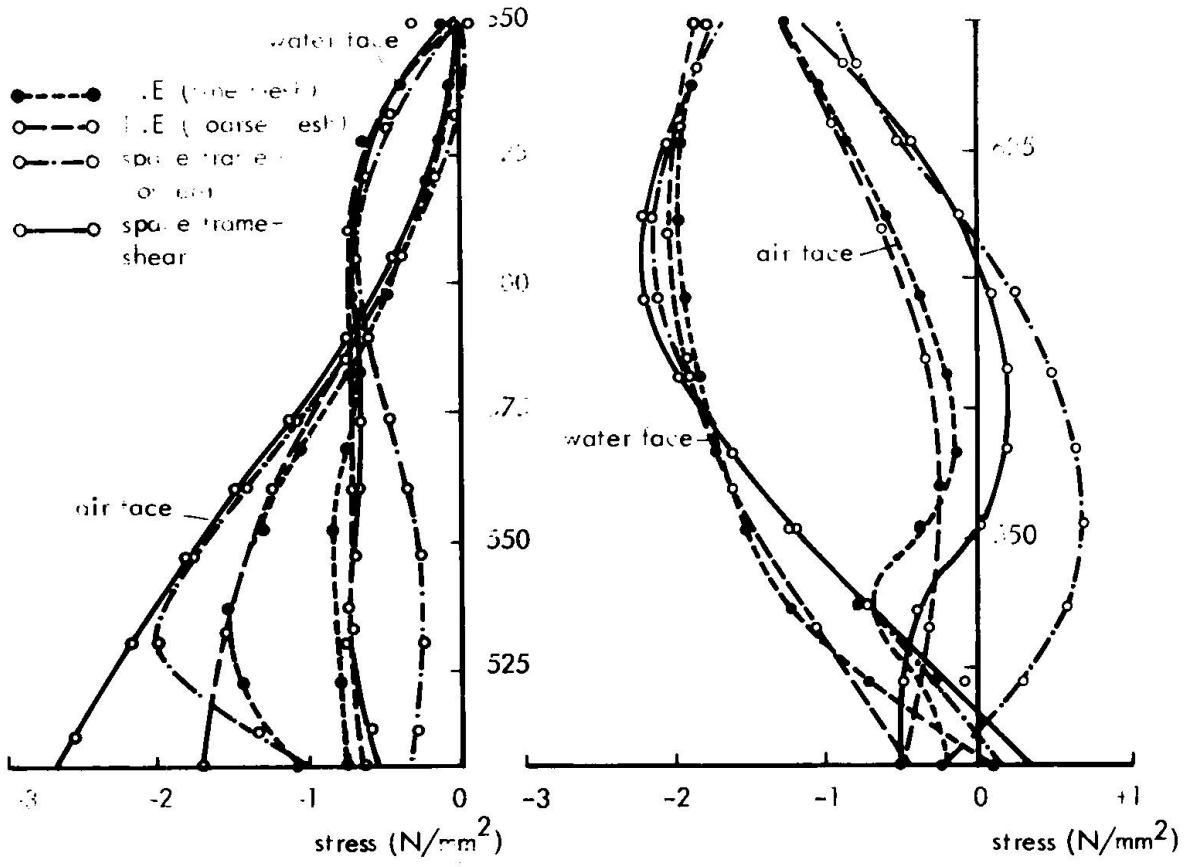


Fig. 12. (a) Vertical stresses at section A and (b) hoop stresses at section A-A

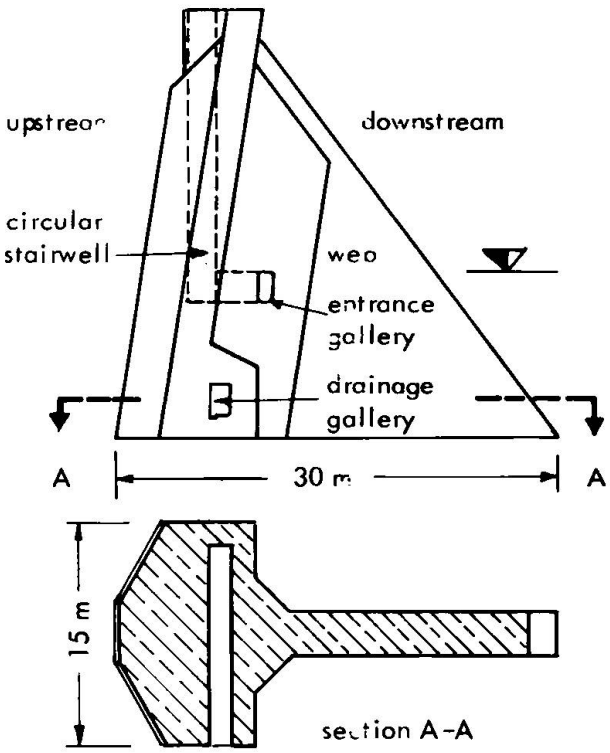


Fig. 13. Unit of buttress - head dam

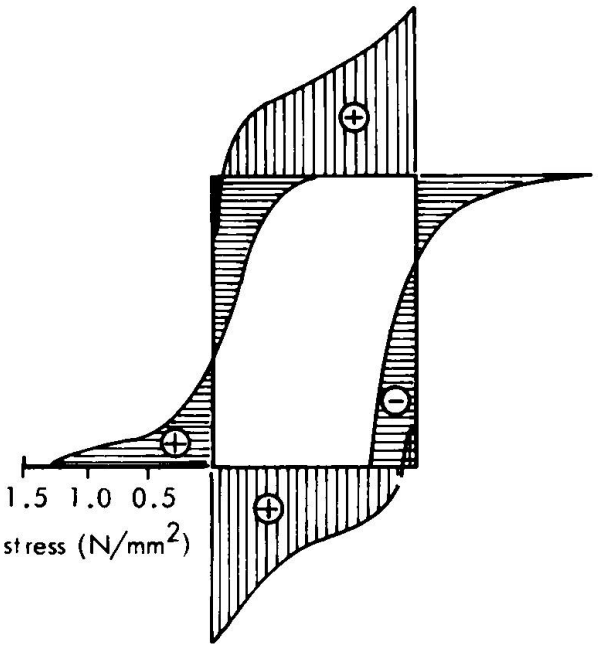


Fig. 14. Tangential stress distribution around gallery

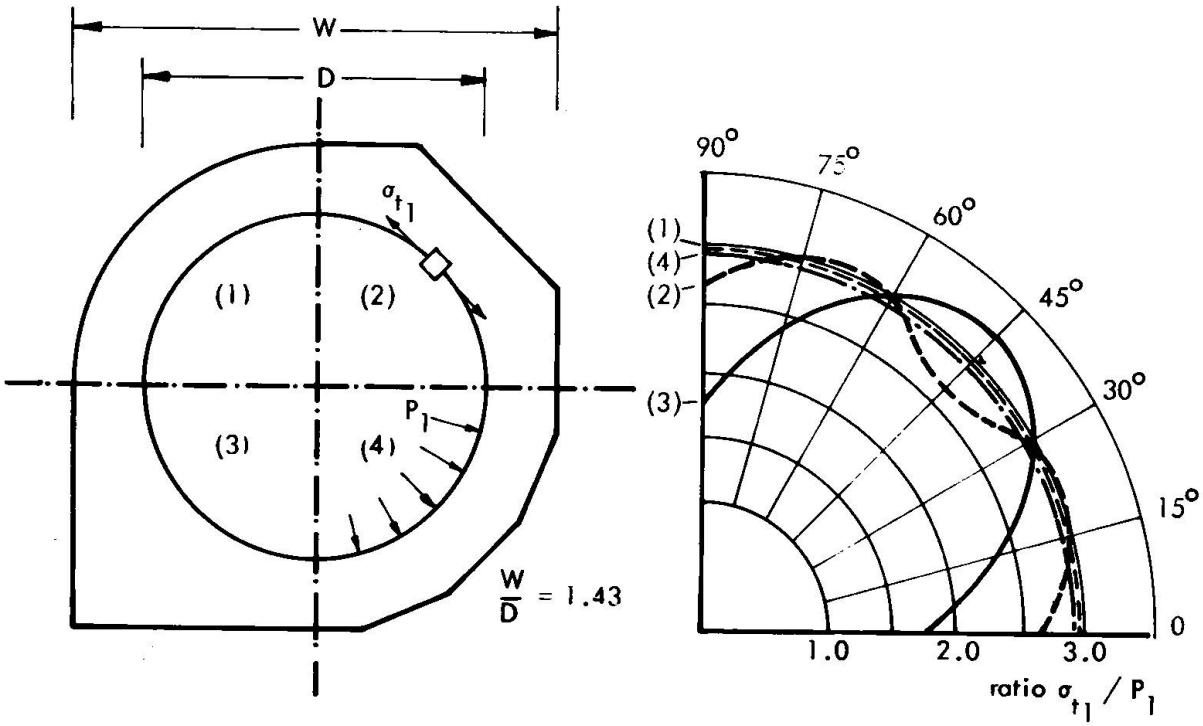


Fig. 15. Cross section and distribution of tangential stress at inside boundary for different cylinders :
(1) Thick cylinder (2) Oktagon (3) Square (4) 16 sided polygon

of the stress that might be expected in the buttresses, especially in the regions of the stairwells and the galleries.

The results of the photoelastic analysis did not show up any extreme stress concentrations and proved the original design to be adequate. The final design of the structure called for reinforcement to be placed around the stairwell and the galleries and for fillets to be provided at the corner of the galleries in order to provide for the high tensile stresses (see Fig. 14).

In another photoelastic study [20] the stresses in conduits with cylindrical internal surfaces and non-cylindrical external boundaries were determined. The external shapes considered were square and polygonal, and curves of stress distributions were presented to make rapid design possible. A typical stress pattern is shown in Fig. 15.

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References :

(1) Sargin, M.:
"Stress-Strain Relationships for Concrete and the Analysis of Structural Concrete Sections."
Study No. 4, Solid Mechanics Division, University of Waterloo, Waterloo, Ontario, Canada 1971, pp. 167

(2) Sargin, M., S.K. Gosh and V.K. Handa :
"Effects of Lateral Reinforcement upon the Strength and Deformation Properties of Concrete."
Magazine of Concrete Research, Vol. 23, No. 75-76, June-September 1971, pp. 99-110

(3) Kowalczyk, R. and W.H. Dilger:
"Strength and Behaviour of Concrete Under High Strain Rates."
Polish Academy Nauk, Report No. 44, 1971, pp.80

(4) Kowalczyk, R. and W.H. Dilger:
"Ductility of Plain and Reinforced Concrete."
Polish Engineering Archiv, August 1972, pp. 72

(5) Dilger, W.H., R. Kowalczyk and R. Koch:
"Strength and Behaviour of Confined Concrete Subjected to Different Strain Dates."
Submitted for Publication, pp.18

(6) Tadros, G.S.:
"Plastic Rotation of Reinforced Concrete Members Subjected to Bending, Axial Load and Shear."
Ph.D. Thesis, University of Calgary, 1970, pp.298

(7) Gardner, N.J., R.M. Godse and T.T. Wong:
"Experimental Investigation in the Use of Laterally Prestressed Concrete Columns."
RILEM, International Symposium Buenos Aires, Argentinia, Sept. 1971, pp. 169-193

(8) Gardner, N.J.:
"Design of Pipe Columns."
Transactions of the Engineering Institute of Canada, Vol.13, No. 3.,4. March 1970, pp. 6

(9) Gardner, N.J. and E.R. Jacobson:
"Structural Behaviour of Concrete Filled Steel Tubes."
ACI Journal, July 1967, pp. 404-414

(10) Gardner, N.J.:
"Use of Spiral Welded Steel Tubes in Pipe Columns."
ACI Journal, November 1968, pp. 937-942

(11) Illston, J.M, I.J.Jordaan and L.J.Parrott:
"The Measurement on Estimation of the Strain in Concrete Under Multiaxial Stress."
The Deformation and the Rupture of Solids subjected to Multiaxial Stresses, RILEM International Symposium, Cannes, 1972, pp. 113-129

(12) Jordaan, I.J. and J.M.Illston:
"The Creep of Sealed Concrete Under Multiaxial Compressive Stress."
Magazine of Concrete Research Vol. 21, No. 69, December 1969

(13) Jordaan, I.J. and J.M. Illston:
"Time-Dependent Strains in Sealed Concrete Under Systems of Variable Multiaxial Stress."
Magazine of Concrete Research Vol. 23, No. 75-76, June-September 1971

(14) Jordaan, I.J.:
"Analysis of Creep in Concrete Structures under General States of Stress."
Personal Contribution, IABSE Seminar on Concrete Structures subjected to Triaxial Stresses, Bergamo, 1974

(15) Gardner, N.J.:
"Triaxial Behaviour of Concrete."
Journal of the American Concrete Institute, Vol. 66, 1969, pp. 136-146

(16) Takhar, S.S., I.J. Jordaan and B.R.Gamble:
"Fatigue of Concrete under Lateral Confining Pressure."
American Concrete Institute Special Publication (in press)

(17) Skjolingstad, L. and Y.K.Cheung:
"Analysis of Arch Dams Using a Space-Frame Model."
Water Power, Febr. 1973

(18) Skjolingstad, L.:
"Three-Dimensional Stress Analysis and Field Problems by the Finite Element Method."
M.Sc.Thesis, The University of Calgary, 1970

(19) Jones, C.R. and J.B.Mantle:
"Three-Dimensional Photoelastic Analysis of a Diamond-Head Buttress Dam."
The Engineering Journal, September 1961,
The Journal of the Engineering Institute of Canada

(20) North, W.P.T. and J.B. Mantle:
"Photoelastic Study of Stresses in Hydrostatically Loaded Cylinders with Noncircular External Boundaries."
Experimental Mechanics, March 1962

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SEMINAR on:
«CONCRETE STRUCTURES SUBJECTED TO TRIAXIAL STRESSES»
17th-19th MAY, 1974 - ISMES - BERGAMO (ITALY)

Development and application of advanced computer analyses in canadian concrete bridge design

Dévéloppement et application des méthodes avancées d'analyse de calcul pour le 'design' des ponts en béton au Canada

Entwicklung der theoretischen Analyse der endlichen Element und ihre Anwendung fuer den Entwurf von Betonbruecken in Kanada

GREGORY MALCOLM McNEICE
Associate Professor Solid Mechanics Division University of Waterloo
Waterloo, Ontario - Canada

1. INTRODUCTION

The ever-increasing demands by both the motoring public and the commercial transport community for increased traffic networks in Canada, have led recently to the development of bridge structures which no longer resemble conventional designs. In high density urban areas, especially in the capital cities, the spider-web of expressways, parkways and interchanges result in multi-level stacking of overpasses. These present particular problems since column spacing becomes irregular both in the longitudinal and transverse directions. In many cases this has led to single columns placed on the center line of multiple span curved decks. In the case of multi-level overpasses, depth of the superstructure is an important consideration. Although box girder design will allow longer spans and therefore less support congestion, it also leads to higher overall elevation of approach embankments which leads to further cost and other considerations. The increased vehicle speeds lead to more stringent requirements on geometrical layout with respect to both planar curvature and superelevation. User acceptance of these structures is directly dependent on the quality of ride, the ease with which one can enter and exit as well as the aesthetic characteristics of the structure when viewed from under, over or some distance away. The present awareness of our environment and the public demand for government preservation of it, has undoubtedly reached the eyes and ears of the designer. He no longer can view his structure primarily on the basis of strength and serviceability. Improved quality of ride especially at high speeds requires continuous span structures in order to avoid construction and expansion joint disturbances which can be quite annoying to those thousands of motorists who commute daily by automobile.

Although there are many other factors, in addition to the above that influence the choice of bridge structure, it is a fact that new and improved designs are required to meet the demands of the Canadian people. These designs usually involved irregular geometry, very long spans and

irregularly spaced isolated supports. In addition to the long slender structures, there are increasing demands for exceptionally wide shorter span slabs with skewed end supports and skewed support lines with isolated columns. In many cases the width of the slab exceeds the span. Comparing slab type designs with conventional girder design leads one to question the validity of using approximate analysis techniques which are based on simplified behaviour. The tendency of many bridge designers who have wide experience in conventional girder slab design is to extend the simplified concepts of beam theory to structures that undoubtedly behave as plates. This has led to some embarrassment on occasions when the resulting structure cracked extensively, deflected beyond acceptable amounts and shifted off its supports. Not only is there a need for better understanding of mechanical behaviour but the effects of creep, shrinkage and differential temperature are very important items in the design of continuous slab type bridges.

In Canada, as in many other countries, academics who research methods of structural analysis and design are usually well versed in the theoretical aspects of two and three dimensional solutions of various boundary value problems. The same is not true for chief designers nor those directly under them. Consequently, it is not unreasonable to find some engineers applying inadequate theories to certain aspects of bridge design. This paper presents a report on what academics across the country have been doing to supply the designer with more appropriate tools for cases in which he must use them and to indicate implementation of various developments to the design office.

In collecting the data for this paper I was fortunate to meet with some of the researchers, both in the western and eastern regions of the country. Since, to the best of my knowledge, the most significant work has come from the University of Calgary in Western Canada, McGill University in Eastern Canada and the University of Waterloo in Central Canada, I have chosen to report the efforts in a similar regional order. These centers that have been mentioned are not the only places where bridge research is continuing, but rather, they are the establishments at which the advanced stress analysis systems are being developed. For purposes of reporting the regions are illustrated in Figure 1.

2.0 WESTERN REGION

The major contribution from western Canada was kindled by Professor Y.K. Cheung who co-authored the first textbook on the Finite Element method while lecturing at the University of Swansea in Great Britain. Upon his arrival at the University of Calgary, he set out to extend his Finite Strip approach to the analysis of slabs [1]. A specific application to bridges was made early in 1969 [2]. The finite strip approach has been the central theme of most of the computer analyses development at Calgary between 1969 and 1972. A brief outline of the method follows, after which a number of applications to bridges will be summarized.

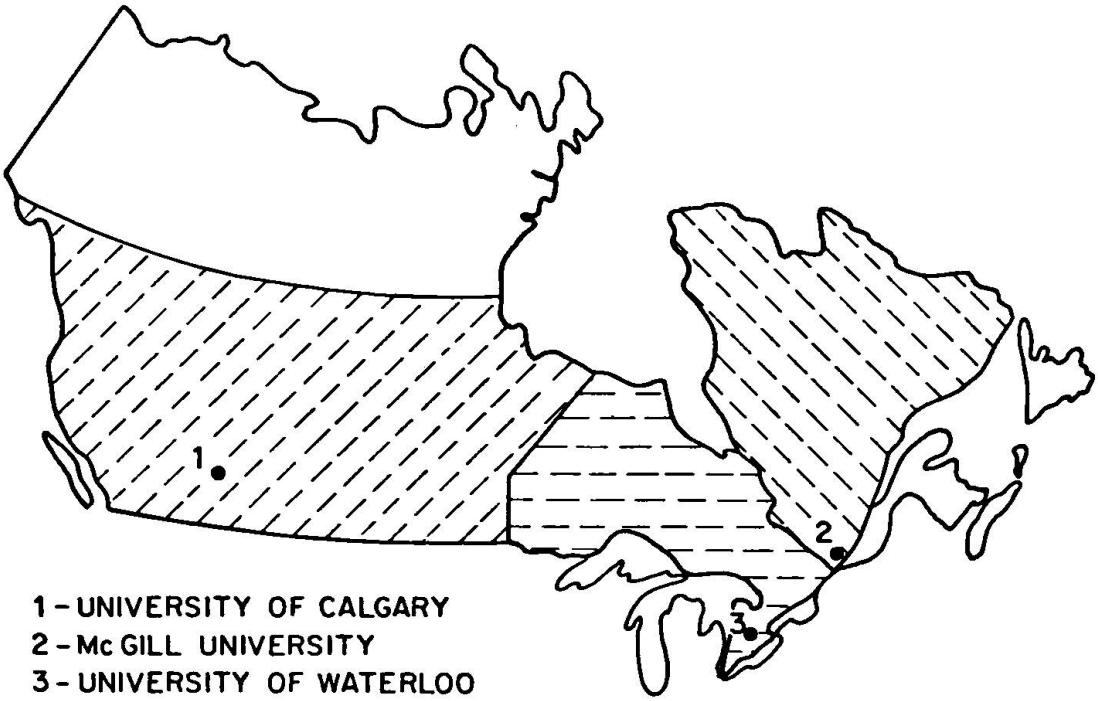


Figure 1 - Research Centers in Regional Subdivisions

2.1 Finite Strip Approach to Finite Elements

In the finite strip approach, the structure is considered to be divided into continuous strips in one direction. Each of these strips is given some approximate response function. Next the force-displacement or stiffness properties are derived and the strips are connected together such as to satisfy equilibrium and compatibility to a certain degree. To illustrate more clearly, consider the two cell box girder in Figure 2.

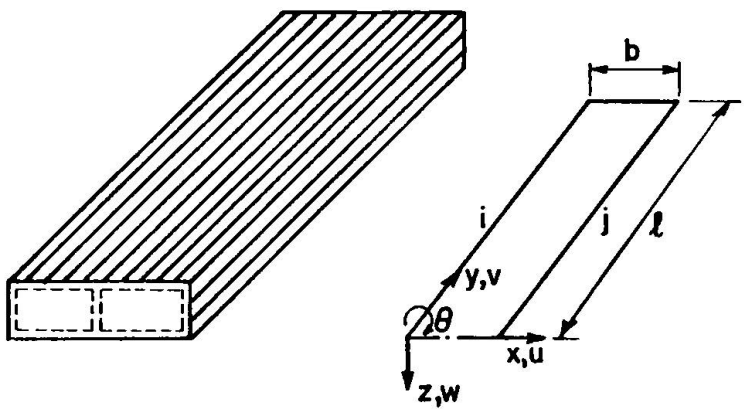


Figure 2 - Typical Finite Strips for Box Girder

The box girder, simply supported, is assumed to be divided into a number of strips in the longitudinal direction in both the webs and flanges. The properties of each strip are considered constant over that strip. Thickness variation can be approximated by using a number of constant thickness strips. Each strip will contain both in-plane or membrane stresses and bending stresses. Since small deflection theory is assumed, these two basic types of stress are considered to be uncoupled, allowing strip behaviour for membrane and bending to be dealt with separately.

The stiffness characteristics for in-plane response are derived by first selecting the end conditions for the strips. For example, the displacement and stress conditions at the ends of the strip can be selected as

$$u = 0 \qquad \sigma_y = 0 \tag{1}$$

for $y = 0$ and $y = \ell$. At $x = 0$ and $x = b$, the nodal line displacements are respectively,

$$u = u_i, \quad v = v_i \qquad \text{and} \qquad u = u_j, \quad v = v_j \tag{2}$$

The strip displacement functions which will satisfy these conditions can be given by a Fourier series such as,

$$u = \sum_{m=1,2,\ldots r} [(1 - \tfrac{x}{b})u_{im} + (\tfrac{x}{b})u_{jm}] \sin \frac{m\pi}{\ell} y$$

$$v = \sum_{m=1,2,\ldots r} [(1 - \tfrac{x}{b})v_{im} + (\tfrac{x}{b})v_{jm}] \cos \frac{m\pi}{\ell} y \tag{3}$$

In Equations 3, u_{im} and v_{im} are displacement parameters of the mth term at the nodal lines. Proceeding with the two dimensional case and using the virtual work principle, one can derive the strip stiffness matrix. A full account of this derivation is given in reference [2].

For the case of strip bending behaviour, the strip end conditions, if simply supported are

$$w = 0 \qquad M_y = 0 \tag{4}$$

for $y = 0$ and $y = \ell$. The displacement functions which satisfy these conditions are

$$w = \sum_{m=1,2,\ldots r} [(1 - \frac{3x^2}{b^2} + \frac{2x^3}{b^3})w_{im} + (x - \frac{2x^2}{b} + \frac{x^3}{b^2})\theta_{im}$$

$$+ (\frac{3x^2}{b^2} - \frac{2x^3}{b^3})w_{jm} + (\frac{x^3}{b^2} - \frac{x^2}{b})\theta_{jm}] \sin \frac{m\pi}{\ell} y \tag{5}$$

These functions can be used to establish the bending stiffness matrix for the strip. The applied loads on each strip are also resolved in terms of Fourier series. The structure is assembled by specifying common nodal line displacements between strips for proper compatability in any one plane. Also, equilibrium of nodal line forces and any applied loads must be established. Since the end conditions are automatically satisfied at the outset, the final set of assembled force-displacement equations can be solved to determine the nodal line displacements. Implied herein, of course, is the fact that a definite number of series terms (m = r) have been used. The more terms, the better will be the approximation. Once nodal line displacements are known, calculation of membrane stresses and internal bending moments is a straightforward procedure. One disadvantage of the strip model as outlined in reference [2] is the fact that vertical and horizontal strips are not fully compatible with respect to rotations along their intersection. One of the major advantages of the finite strip approach is that analyses can be performed on medium-sized computers quite readily. For application to continuous span structures, some difficulty was encountered when longitudinal strips were used. Consequently, for continuous spans, transverse strips were chosen. An example of this can be seen in reference [3] in which Cheung, Cheung and Reddy carried out frequency analysis of continuous span bridges. One example was a two span isotropic bridge of constant thickness shown in Figure 3. Figure 4 illustrates the thickness idealization that was used on a second example containing a haunch over the interior support. Comparison of frequencies and various mode shapes with known analytical results illustrated the good accuracy provided by the strip approach [3]. In addition to Professor Cheung's involvement in the development of the finite strip method, Professor Ghali became interested in the method since he had carried out a number of physical model studies of bridges and wished to compare these results with analytical values. After having co-authored a paper involving continuous span finite strip analysis [4], Ghali eventually changed the approach to continuous span analysis. In reference [4] superposition of two simply supported cases was used to model continuity of the structure over the interior support. Ghali's approach [5] was to first analyze each span as simply supported, determine the rotation and then superpose couples to produce the necessary compatibility. Of major importance in reference [5] was the introduction of the trigonometric basic functions into the transverse deflection approximation. That is,

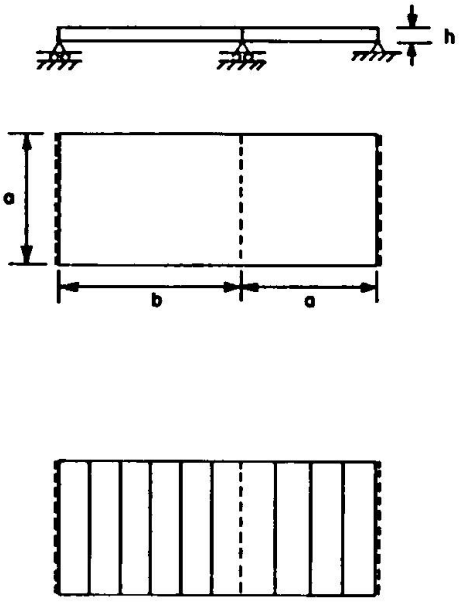


Figure 3 - Transverse Strips for Continuous Span Analysis

$$w = \sum_{m=1}^{r} f(x)\, Y_m \tag{6}$$

in which $f(x)$ is a polynomial and Y_m the basic function required to satisfy the strip end conditions in the y direction (see Figure 5).

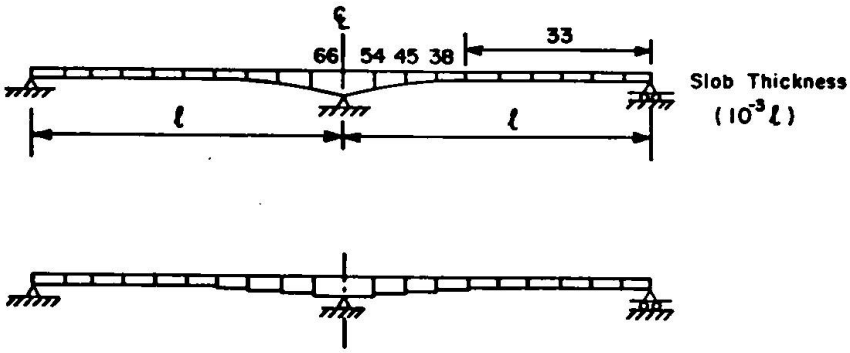


Figure 4 - Actual and Idealized Thickness Variation

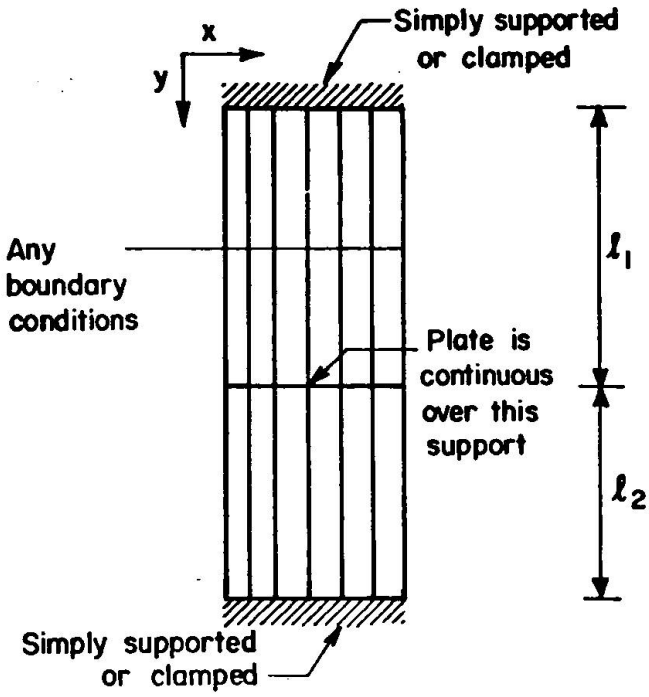


Figure 5 - Longitudinal Strips for Continuous Span Analysis

The orthogonal properties of the trigonometric basic functions allows the total number of equations to be uncoupled in such a manner that a number of smaller sets of equations can be solved. This is true with these basic functions for both simple supports and clamped conditions. The major disadvantage found in the work of reference [5] was that the connecting moment method of superposition required many terms in the series in order to obtain good accuracy. In an attempt to reduce the computational costs due to a high number of terms, an attempt was made to develop an extrapolation technique in order to predict results on the basis of those obtained using only a few terms. As outlined in reference [6], the finite strip approach using basic functions that have orthogonal properties consists basically of adding weighted solutions, one for each term in the selected trigonometric series. This evolves from the fact that the loading on the strip must be approximated also by a series approach. The solutions obtained from the uncoupled sets of equations are directly dependent on the type of load and

the functions chosen to approximate them. From the contributions so established, the deflection at any one point can be plotted versus the number of terms. If a polynomial curve is passed through these values, it can be used to extrapolate results beyond that obtained using the terms chosen. This procedure certainly increases the possibilities of the finite strip approach.

Since many box girder bridges have skewed supports, Brown and Ghali [7] developed parallelogram strips having a fifth order polynomial in the transverse direction. The results reported indicate that good accuracy is obtained by using a small number of equations. However, only solid skew slabs were analyzed and no box girders were attempted. Although this work and the previously outlined finite strip developments have clearly defined advantages for certain types of plate analyses, the method is not convenient for irregular bridges. That is, in cases where diaphragms are used, or in which solid sections exist over support regions and in end anchorage zones. Also, there does not appear to have been any attempts to analyze slabs with isolated column supports, a common case in modern day bridge complexes.

2.2 Special Parallelogram and Quadrilateral Elements

Realizing the shortcomings of the finite strip method for general purpose bridge analysis, Sisodiya, Cheung and Ghali [8] developed a parallelogram finite element especially designed to model bending deformation in the webs of box girders. A second element used for the deck membrane response is a quadrilateral element and is presented in the same paper. In general, the element uses curvilinear coordinates, similar to the linear isoparametric elements [9]. However, for membrane behaviour, three degrees of displacement freedom were developed. That is, the in-plane displacements of u and v as well as a rotation θ_z in the plane of the element. This rotation is of fundamental importance in providing rotation compatibility at nodes that are common to two elements which are inclined to one another. These freedoms combined with the transverse displacement w and the bending rotations θ_x and θ_y constitute the required minimum number of degrees of freedom for point continuity. For bending response, the authors selected the Dawe element for use with the parallelogram element and two of the Bazeley et al. triangular elements for the quadrilateral element [9].

The parallelogram element is illustrated in Figure 6. The displacement v is assumed to vary as a cubic in ξ and linear in η.

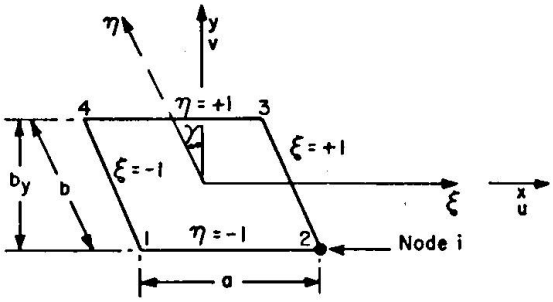
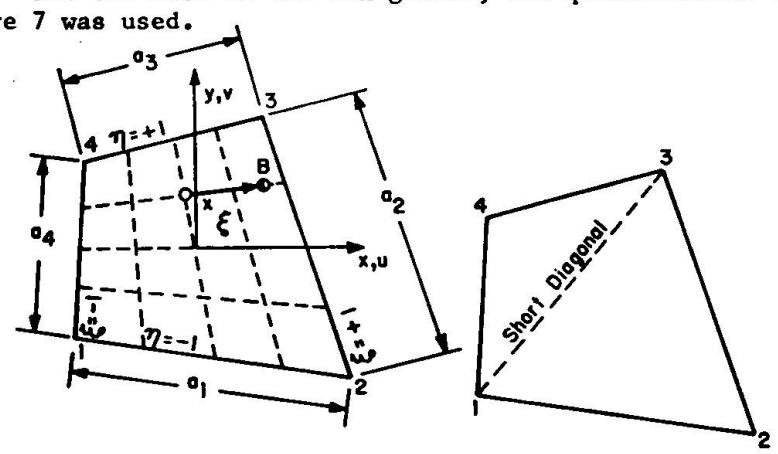


Figure 6 - Special Parallelogram Finite Element

The rotation normal to the element plane is simply $\theta_z = \partial v/\partial x$. The resulting element response is obviously biased for direct application to web deformation. The cubic response in v allows the bending deformation

to take place in the plane of the web perpendicular to the ξ axis. However, v need only be linear in the vertical direction. The displacement u is quadratic in both ξ and η allowing a linear distribution of bending stress in the plane of the web as would be required.

For the deck of the box girder, the quadrilateral element of Figure 7 was used.



(a) Coordinates in the element

(b) Plate bending from two triangles

Figure 7 - Special Quadrilateral Finite Elements

For this element, u was chosen as a linear function of ξ and η with v cubic in both. This allows the element to be used as a web element if a variable thickness box girder is analyzed. A number of tests were performed using both elements, the results of which indicated that both elements model standard beam type problems very well. For skewed beam dimensions, the parallelogram element was clearly superior. The quadrilateral element had to be used in cases of haunched beams, of course. In these cases the results seemed quite good, sufficient for normal engineering practice. Application of these elements was made to the two span skewed box girder in Figure 8. Additional applications were also presented in reference [8].

A further application to single and double cell skewed box girders with diaphragms is presented in reference [10]. The results of that study showed that end diaphragms in skewed bridges produced undesirable effects on reaction distributions. It was also concluded that transverse stiffeners have little use and in some cases have a harmful effect. An interesting application of these special elements was also made to a skew vault bridge [11]. There, excellent results were produced when compared to other type of finite element models.

It should be stated here, as did the authors in reference [8], that these elements do not provide compatibility of displacements along common boundaries of all elements. This is since the x axis of two adjacent elements could have different directions in a mesh of quadrilaterals and the displacement functions for u and v are unsymmetrical with respect to the co-ordinates of each element. In addition, when parallelogram web elements are connected to quadrilateral deck elements, the bending and in-plane rotations do not conform to the same displacement functions and this leads to incompatibilities. Although the authors state that these

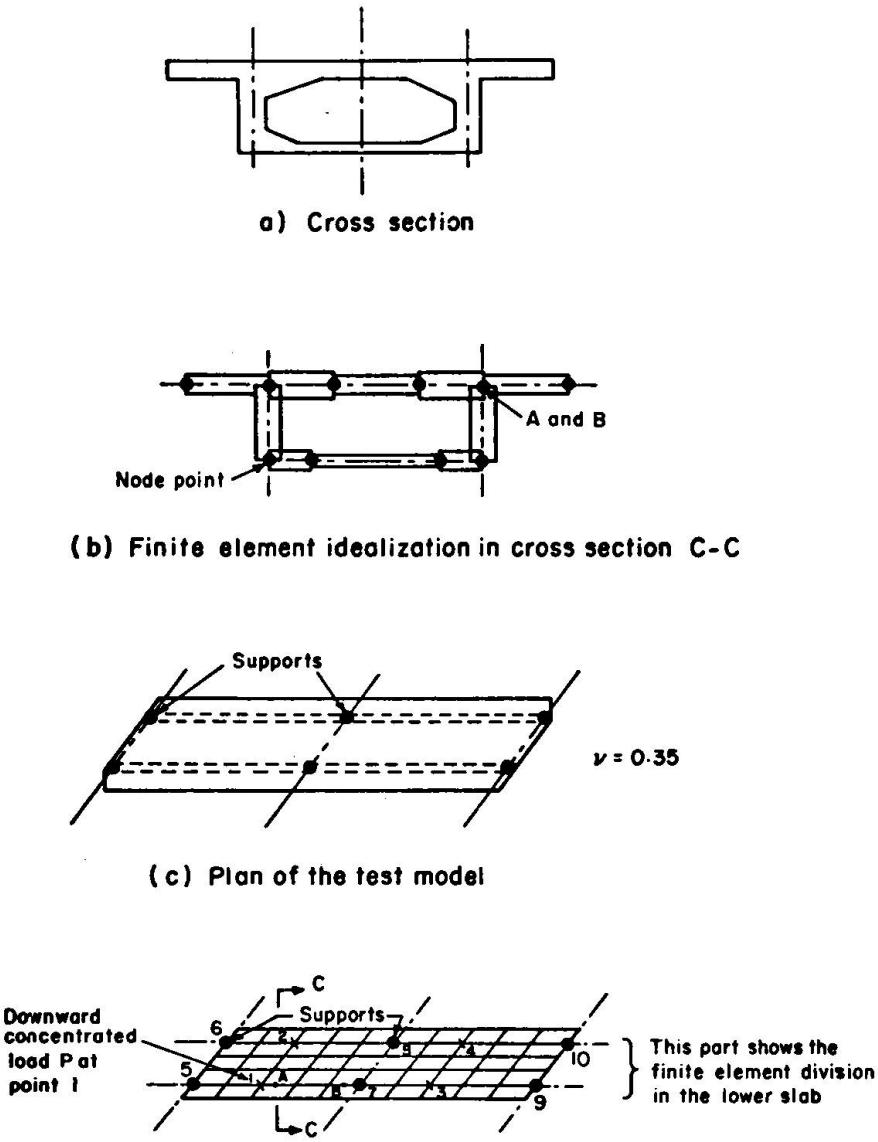


Figure 8 - Straight Skew Box Girder Model

effects are negligible, they lead to the requirement that an increased number of elements must be used for any one problem as compared to an analysis using fully compatible elements. Furthermore, if the designer wishes to view stresses in the regions of intersection between webs and slab or in the vicinity of the end anchorage zones, it is unlikely that much importance could be attached to these stresses. However, for general overall response, it is clear that these models provide substantially more information than simplified beam theory approaches.

To summarize the contribution from western Canada, perhaps it is fair to say that the work done at the University of Calgary under the direction of Professors Cheung and Ghali constitutes the major effort directed toward bridge analysis in western Canada using modern methods of structural analysis. To the best of my knowledge there has been no

significant application of this work to bridge designs produced by western provincial governments. Researchers at Calgary have provided results for individual consulting firms on certain bridge structures in the west. Although there is evidence of an increasing awareness of the finite element method in design offices all across the country, there is no clear effort being made by western provincial ministries to co-ordinate analytical capabilities or provide any means of using existing knowledge.

I should add here that after having completed his post-doctoral studies at Calgary, Sisodiya was given an assistant professorship at McGill University. This leads me to report on the contribution from eastern Canada.

3.0 EASTERN REGION

Evidence of early application of the finite element method to metal box girder bridges in Canada exists in reference [12], in which Mehrotra et al. at McGill University used triangular finite elements with linear membrane response and a modification of Tocher's bending element. The elements were used to model webs and flanges in a fairly straight forward manner. The major drawback of the model was the fact that compatibility of displacement was not maintained, requiring many elements for accurate results. More recent results will form the main discussion to follow.

3.1 Circular Curved Box Bridges

A new in-plane annular finite element has been developed by Fam and Turkstra [13]. The element is illustrated in Figure 9. A polar co-ordinate system is used with each element having four corner nodes, each with four degrees of freedom u, v, $\partial u/\partial\theta$ and $\partial v/\partial\theta$. The displacement functions are cubic in θ and linear in r. The generation of the element membrane stiffness matrix is straightforward and is given explicitly in [13]. Comparison of results for simple structures having series solutions showed that the annular element gave accurate results and could therefore be used with confidence. Practical application consisted of the analysis of the horizontally curved box girder bridge illustrated in Figure 10.

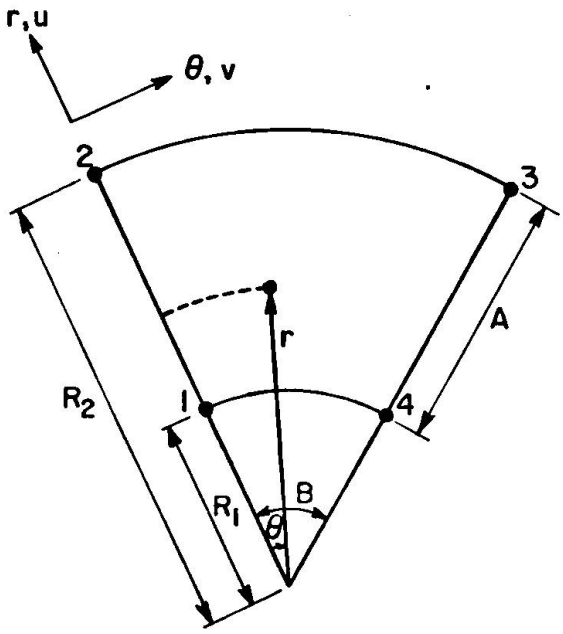


Figure 9 - Special Annular Element

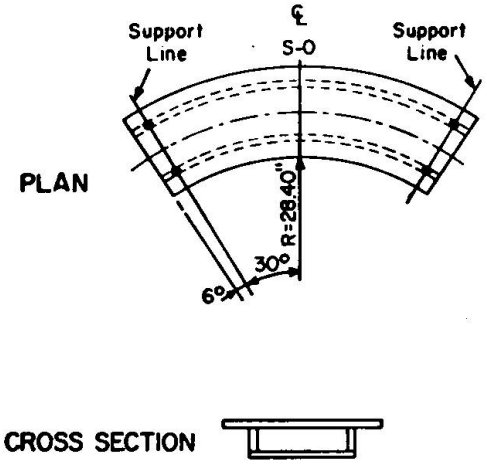


Figure 10 - Geometry of Box Girder Model

The in-plane behaviour of the top and bottom flanges were modelled using the above annular element. The bending behaviour of the flange elements was modelled using the Olson-Linberg annular bending element [14]. A new cylindrical element was used for the web regions. This latter element will be published in the near future. The particular bridge in Figure 10 was a plexiglass model which was tested and reported on by Aneja and Roll [15]. The superior results of the annular elements are clear from all the comparisons made between experimental, annular and regular flat type element results. One example is shown in Figure 11.

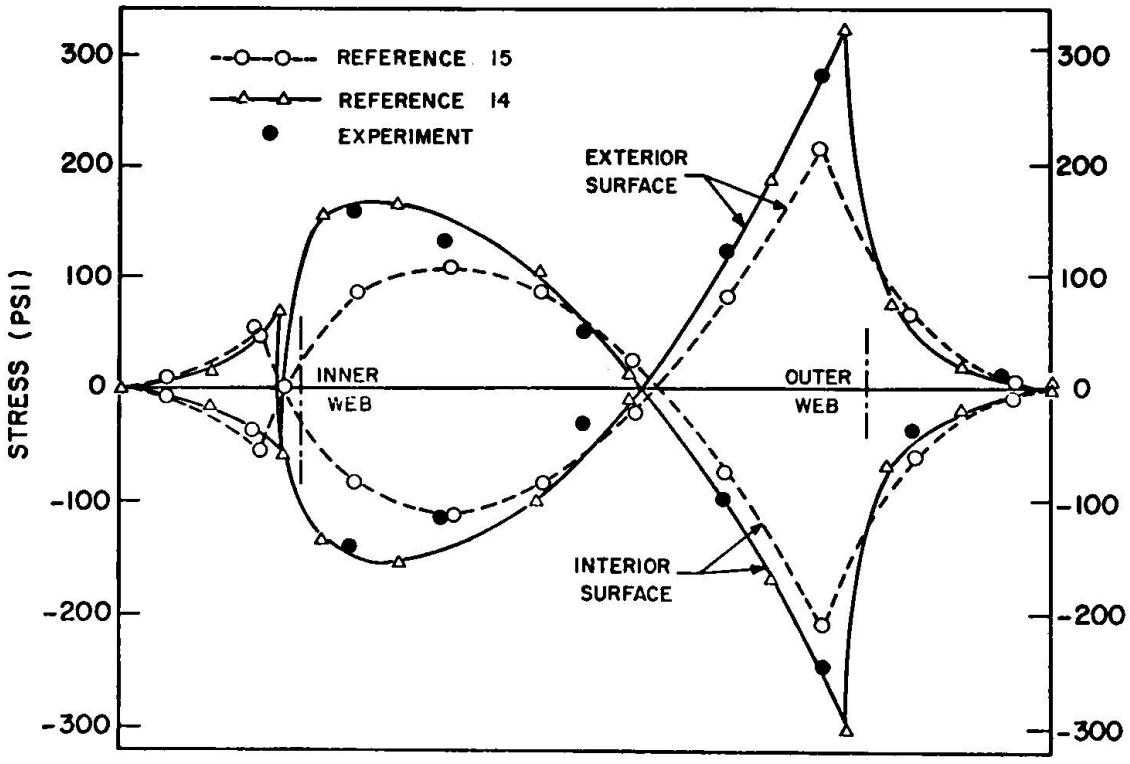


Figure 11 - Radial Stresses in Top Flange

These results indicate the importance of having the proper mathematical model for different types of box girder bridges.

3.2 Special Membrane Elements for Box Girder Bridges

Another approach to multi-cell box girder analysis was presented by Gurevich and Redwood [16]. In this analysis only membrane behaviour was used. In order to approximate the bending effects, fictitious transverse diaphragms were introduced. Therefore, the bending stresses in the model were assumed to be only membrane stresses in the top and bottom flanges of the boxes. No bending stresses were allowed through the thickness of any flange or web element. Although approximate, this approach can produce good results. For example, in cases in which all members, especially flanges, are thin and do not possess much bending strength, this model will perform adequately. However, for cases in which the flange thickness to span ratio approaches a medium thick plate, the flange bending stresses can be relatively high and this model will not likely provide adequate approximation. The authors used their model to analyze a five cell, 60° skew box girder which had previously been analyzed by usual finite elements [17]. The results are shown in Figure 12 and they are encouraging. Some of the elements developed for use on the webs are shown in Figure 13. The major drawback of a purely membrane type model is the complete lack of rotational compatibility, even at the nodes. The influence of such discontinuity between members can lead to suspicion of stress results at these joints. Also, fairly narrow cells must be available in order to allow the fictitious diaphragms to approximate the bending modes in the transverse span directions.

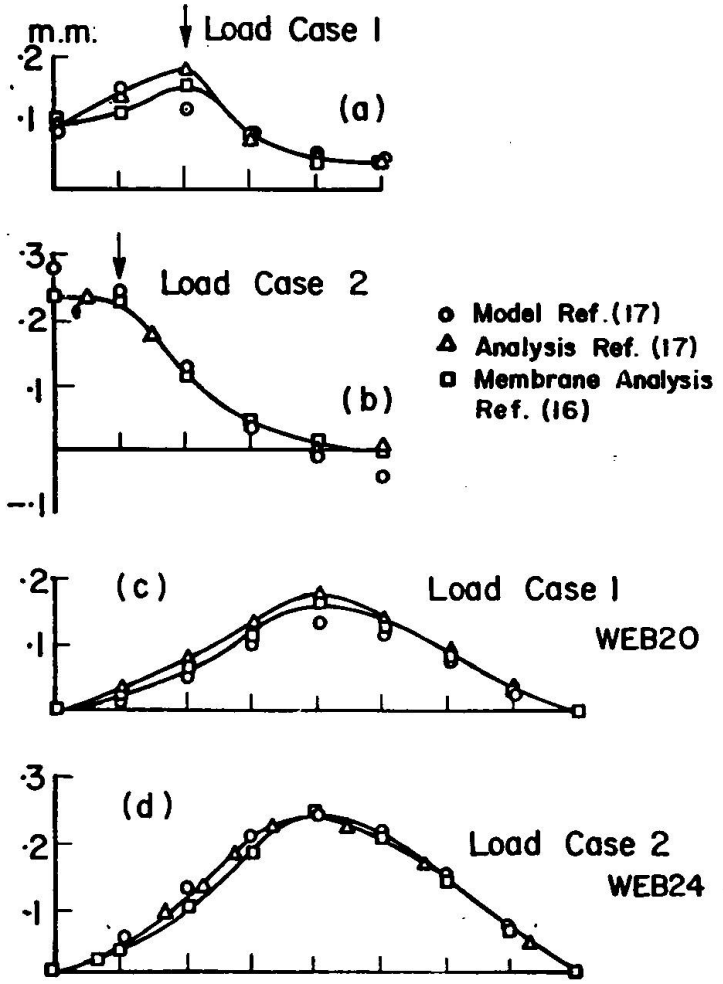


Figure 12 - (a) & (b) Centerline Deflection Point Loads on Ribs 2 & 3
(c) & (d) Loaded Rib Deflections

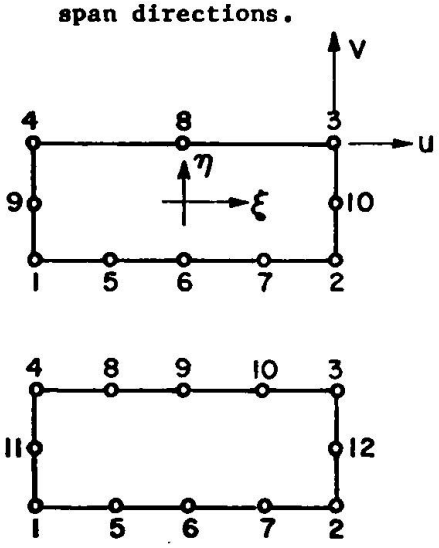


Figure 13 - Special Isoparametric Web Elements

Recently, Sisodiya at McGill University reviewed the state-of-the-art with respect to the analysis of box girder bridges [18]. Included in this review is the work of Scordelis at the University of California, U.S.A. and that by Cope at the University of Liverpool, England. The major point to be made is that most attempts have involved only 5 degrees of freedom at common nodes. This leads immediately to rotational incompatibility and for practical cases of stress analysis for which the designer does not have textbook solutions nor experiment to check the results from his computer, the effect of these discontinuities on the stresses will be impossible to assess. Indeed, the analyst cannot even perform a three solution convergence study of energy in order to assess the approximations. If, however, the 6 degrees of freedom (3 translations and 3 rotations) are used and the connecting elements are properly formed, certain types of box girder bridges will be fully compatible and the necessary numerical checks can be made. The models developed by Sisodiya include the 6 required degrees of freedom. Although computer costs are higher with use of more degrees of freedom, these costs are trivial compared to that taken to prepare the geometrical input data. Also, as computers and computer hardware systems become more efficient, these disadvantages become less important.

Summarizing the efforts from eastern Canada, there appears to be less direct bridge research being done than in the west. However, since Professor Sisodiya is now at McGill with the desire to further develop his work, I am sure that more developments will be coming from the eastern region.

Although I have devoted the discussion thus far to the analysis of box girder bridges, I should like to make note here of the substantial efforts made in the finite element field by the group at the National Research Council. In particular, the work done by Kosko, Cowper, Lindberg and Olson. Their research on higher order elements for shell, plate and membrane analyses has been given significant reference internationally. I should also like to mention the research by Professor Reddy of Memorial University. He has carried out analysis of various plates subjected to random excitations using the finite strip approach [19]. Once again, and to the best of my knowledge, there has not been any direct interaction between government design offices and researchers in eastern Canada; that is, with respect to coordination of efforts toward bridge research and design. I am aware, however, of such involvement in central Canada and it is to this that I address the remaining portion of this report.

4.0 CENTRAL CANADA

In the past, most Ontario highway bridges were designed such that the primary structural members were griders of either steel or prestressed concrete. For these structures, the well-known theories of beam bending and combined axial and flexural response were quite adequate in predicting prototype behaviour. However, when bridge slabs are used as the primary structural member, care must be exercised in using beam theory to check their design. The reason is that in many cases, such as in skewed slabs or in the vicinity or isolated supports, the longitudinal and/or transverse stresses are not the critical stresses. Instead, due to shear stresses the principal stresses far exceed the normal stresses.

In Ontario, the latest bridge design consists of voided prestressed slabs with multiple spans and in many cases, irregularly spaced single columns. For these slabs, beam theory provides a good estimate of longitudinal stresses but is inadequate in predicting transverse stress distributions which contain stress concentrations over the void regions. Also in the end anchorage zones, the stresses are very complex and of a three dimensional nature. Consequently, more refined methods of stress analysis are required for bridge slabs that behave as plates rather than as beams.

Recognizing the need for additional information concerning the behaviour of these structures, the Ministry of Transportation and Communications of Ontario, through the Structural Research Branch, undertook an extensive programme of study. It included analytical, numerical, physical and on-site investigations.

The particular bridge design currently used in Ontario contains voids with diameters as large as 80 percent of the depth of the slab. The effects of these voided regions, especially in a transverse direction are a major concern to the designer. The real structure is truly three dimensional in nature, since the concentration of stresses could occur over the voids along with a non-linear stress distribution across sections between voids. Since a full three-dimensional analysis would be too costly during the preliminary stages of design, a two dimensional model was chosen in order to assess its capability prior to any three dimensional analysis.

I will report firstly on the computer stress analysis system which uses two dimensional finite element uncoupled bending and in-plane stress analysis to analyze these bridges. This will be followed by a report on the three dimensional system currently being developed.

The programmes that make up the two dimensional Finite Element System are described briefly in the following. The system allows the engineer to perform the following types of stress analyses:

(a) Plane Stress
(b) Plane Strain
(c) Combined Bending and In-plane

4.1 Description of 2-D Finite Element Stress Analysis System

The two dimensional stress analysis system that was developed at the University of Waterloo is based on the displacement model of the finite element method and has been developed especially for the analysis of voided prestressed bridge slabs with typical cross-section as illustrated in Figure 14. However, the two main subsystems were constructed to work independently in order to provide additional stress analysis capability. The basic system flow chart is illustrated in Figure 15.

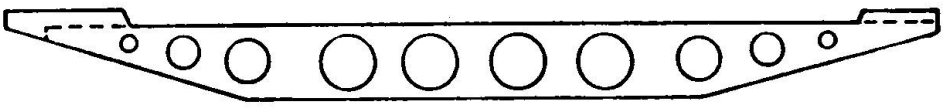


Figure 14 - Typical Cross Section with Circular Voids

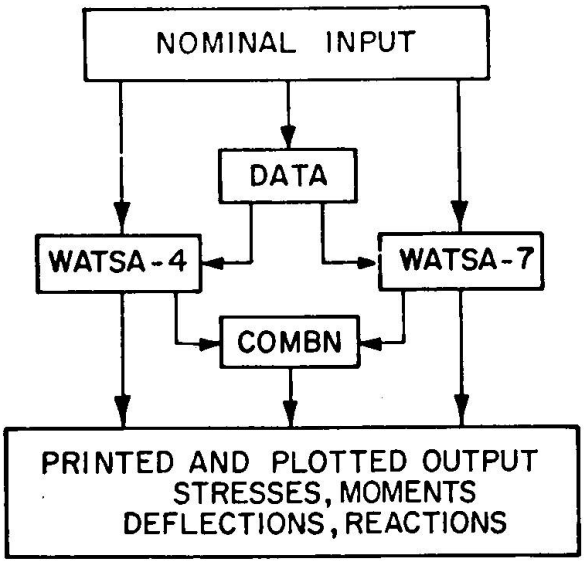


Figure 15 - Flow Chart of M.T.C. 2-D Finite Element System

Programme WATSA-4

This programme performs finite element plate bending analyses based on thin plate small deflection theory. The Clough-Felippa quadrilateral bending element [20] is used throughout. Material properties in each element can be orthotropic and with different orientations in each element. Individual element thicknesses can vary linearly over the element. Automatic element mesh generation is optional and the programme sorts identical elements such that unnecessary calculations are prevented.

The programme has a four level overlay structure requiring approximately 220K bytes core storage, 10 scratch files and allows the use of 1000 nodes and 900 elements. Plotting of element mesh as well as principal stresses are optionally available.

Programme WATSA-7

This programme performs finite element plane stress and plane strain analyses. There are two elements available, namely a quadrilateral isoparametric element [21] with or without a centroidal node. Material properties in each element can be orthotropic and with different orientations in each element. For the plane stress case, element thickness can vary linearly over the element. Automatic element mesh generation is optional and the programme sorts identical elements such that unnecessary calculations are prevented.

The programme has a three level overlay structure requiring approximately 220K bytes core storage, 9 scratch files and allows the use of 1000 nodes and 900 elements. Plotting of element mesh as well as principal stresses are optionally available.

Programme DATA

This programme creates the necessary input data required by WATSA-4 and/or WATSA-7 for typical voided prestressed slabs. The bridge is laid out in the usual manner by using a control line and a provincial coordinate system. Various measurements such as curb location, void positions, type and size of void, support region and skew angle as well as location of column supports must be provided as input to the programme. The user specifies the density of elements both longitudinally and in the transverse direction. The programme divides the slab into appropriate elements in the support regions as well as over the spans. These regions are matched, the supports are located, the moments of inertia and necessary cross-sectional areas are automatically calculated as input to the analysis programmes. In addition, a plot of the entire plan view of the bridge is provided in order to check on element subdivisions, etc. prior to analysis. This programme also orders and sorts elements into groups with similar geometry and material properties. It also calculates all dead load forces and prepares the loading to simulate prestressed effects for separate input to WATSA-4 and WATSA-7. The programme allows up to 8 spans and assumes that the cross section selected is constant throughout the length of the bridge. Both circular and rectangular voids can be positioned anywhere in the cross-sections.

Programme COMBN

This programme simply combines the stress and displacement results as produced from WATSA-4 and WATSA-7 and provides plots of top and bottom surface principal stresses. It allows any factored combination (up to 10 selections) of results from individual load cases, either from WATSA-4 and/or WATSA-7. The programme requires two direct access files and five scratch files.

Each of the above programmes performs a separate task. As far as voided prestressed bridge slabs are concerned, only DATA pertains exclusively to these structures. Each of WATSA-4 and WATSA-7 can be used entirely separate from all the other programmes. Indeed, WATSA-7 can be used effectively for studying transverse behaviour by analysing a cross-sectional portion of the slab. It can also be used to study other structures ranging from steel gusset plates to underground concrete culverts. WATSA-4 can be used for analysis of any irregularly shaped slabs, manhole covers, concrete pavement slabs, etc.

Printed output can be selected from one or more of the following: displacements, bending and twisting moments, principal moments, normal and shear stresses, principal stresses. Stresses from various load cases can be combined. Plotted output includes the element mesh layout generated by the data preparation programme as well as plots of principal stresses on top and bottom surfaces of the bridge.

4.2 Example of Bridge Analysis

A series of bridge analyses has been reported elsewhere [22,23] as well as comparisons with experimental results [24]. In order to illustrate the type of visual output possible from the Finite Element System, the five span bridge of Figure 16 is presented herein. The particular loading case was that of dead load. The principal stresses at the centroid of each

element are presented such that the largest stress is directed in its proper orientation. The smaller stress is written beneath the largest. A glance at the stress plot gives the designer an idea of the distribution of stress around column heads and in the vicinity of cable anchorage zones.

The bridge in Figure 16 was fully subdivided automatically using DATA. Additional stress plots for each live load case, as well as for both top and bottom surfaces of the deck were produced during the one pass through the computer.

4.3 Applications to Various Problems

Gusset Plate Analysis (Plane Stress Analysis)

In plane stress analyses, stresses normal to the mid-plane of the plate are assumed to be zero while normal strain still occurs due to the Poisson effect. As an example of this type of analysis, the automatic mesh generation in WATSA-7 was used to produce the element mesh for the gusset plate in Figure 17. Nodes that were closest to the rivet holes were then selected as loaded points. Plotted output of principal stresses was selected for quick evaluation of the stress field.

Transverse Section Analysis (Plane Strain Analysis)

Plane strain analysis is used for structures that are of infinite extent in the direction perpendicular to its cross-section. Such is the case for a typical cross-section of a voided bridge slab. Figure 18 illustrates both a circular voided slab and a thick web box girder type. The element meshes seen here were custom made for these analyses using an electronic digital table. For complete manual preparation, they can be drawn to scale on graph paper and then the coordinates determined. It should be mentioned that this type of stress analysis performed on a section of the bridge gives a much better idea of the transverse behaviour of a slab design than by simply using a cantilever beam strip. Beam theory gives an erroneous distribution of stress in the vicinity of the void.

Effects of Shrinkage around Circular Voids

In order to assess the effects of shrinkage on stresses in the vicinity of the voids, a series of finite element analyses was carried out on a typical voided region. Shrinkage was approximated by introducing a temperature drop throughout the concrete. The restraint provided by the corrugated steel liner leads to surface tensile stresses which are very sensitive to the ratio of void diameter to slab thickness. Results of the analyses are summarized in Figure 19. As an example, a d/H ratio of 0.8 will give over 300 psi surface tensile stress over the void if a shrinkage strain of about 400 microstrain is selected.

These applications illustrate the varied use of the 2-D system and the analytical capability that now exists with the Ministry. Recently, the programmes have been used to study not only multi-lane skewed prestressed bridges, gusset plates and associated members, but also buried culverts having very large spans. The engineers in the Bridge Office are finding the system quite helpful in locating the principal stress regions of irregular members and wide slab bridges.

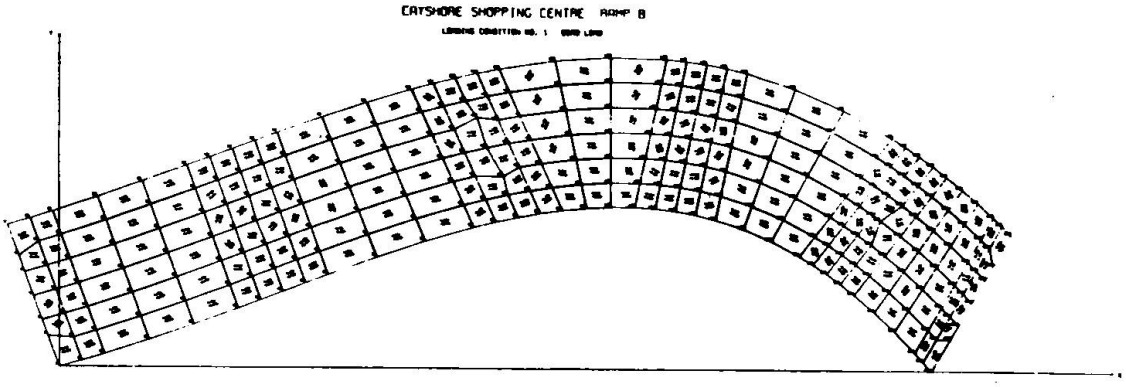


Figure 16 - Principal Stress Plot for a Curved Voided Bridge using WATSA-4

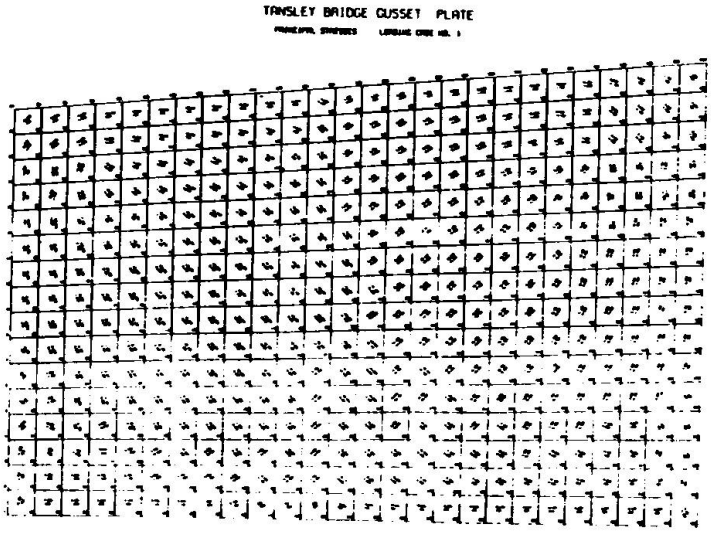


Figure 17 - Principal Stress Plot for Gusset Plate Analysis
Using WATSA-7 (Plane Stress)

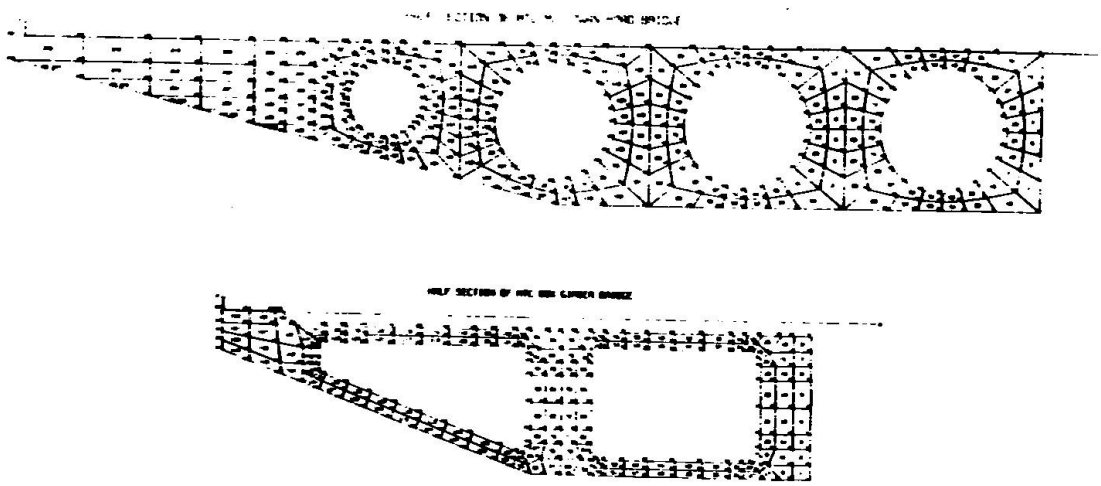


Figure 18 - Plots of Finite Element Meshed Used in Transverse Section Analysis Using WATSA-7 (Plane Strain)

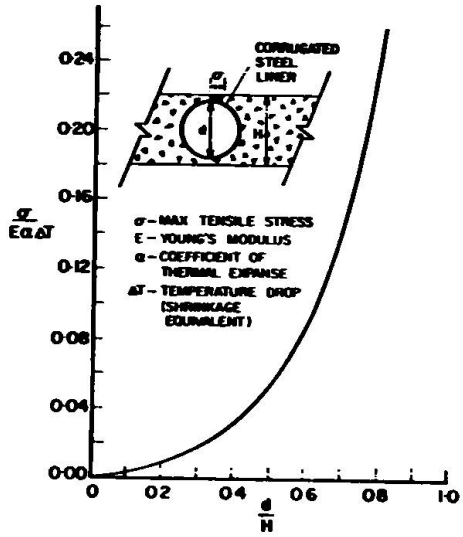


Figure 19 - Effect of Shrinkage Around Circular Voids

4.4 Experimental Model Investigations

In conjunction with the development of computer systems for bridge analysis, there are model studies being done on both rectangular and skew multi-span voided prestressed bridges at the University of Waterloo. These experiments are being conducted under the supervision of Professor Green. Through coordinated efforts, Professor Green and I are able to provide a set of computer-aided analysis systems along with physical verification. This determines both their capabilities and limitations. A three span 1/36 plexiglas bridge model with a rectangular planform is shown in Figure 20. The test results for this bridge were reported in reference [25]. Some of these results were compared with finite element analysis [24]. A second model of 1/24 scale 40° skew bridge was recently tested [26]. This model is shown in Figure 21, prior to testing. For completeness, I have included in the next section some of the results of reference [24].

4.5 Comparison between Numerical and Experimental Models

The primary purpose of the work reported in reference [24] was to assess the adequacy of the two-dimensional finite element system in analyzing the three dimensional stresses in the voided prestressed bridge slabs. Figure 22 contains a plan view of one half of the bridge, while Figures 23 through 27 indicate the accuracy obtained in predicting longitudinal and transverse stresses. The solid line is based on a neutral axis assumed to be at the mid-depth of the bridge cross-section. The dashed lines are finite element results based on a modified moment of inertia for the tapered exterior wings of the bridge. This is discussed in detail in reference [24]. From these results it is clear that where the stress state departs drastically from that assumed in two dimensional theory, a three dimensional model must be used.

The conclusion drawn in the original study [23] was that the two dimensional model was sufficiently accurate to predict stresses in regions other than the end anchorage zones where large concentrated cable forces caused complex three dimensional stress fields and in the outer wing regions where the assumption of the neutral axis cannot be accurately made. Also, in the transverse span direction, the stresses over the voids are not estimated properly. However, unless specific detail is required, the results from the two dimensional model can be appropriately factored in these regions and quite accurate estimates made. Although the 2-D system was more advanced than any similar system existing at the M.T.C. and could be used for most bridge designs, as well as a wide variety of other problems, it was the opinion of the engineers involved that a 3-D capability should be provided. Consequently, such a system was initiated and I should like to report on the status of this endeavour.

4.6 Outline of 3-D Finite Element Analysis System

The 3-D system consists of solid elements of the isoparametric family having quadratic displacement functions [27]. The introduction of a mixed displacement formulation [28] has also been added. This allows any combination of curved boundary elements to be used including those with 20 nodes along with others with any number less than 20. If 8 corner nodes are requested, then the element degenerates to a hexahedron with linear sides. Using these elements in combination will allow the higher order

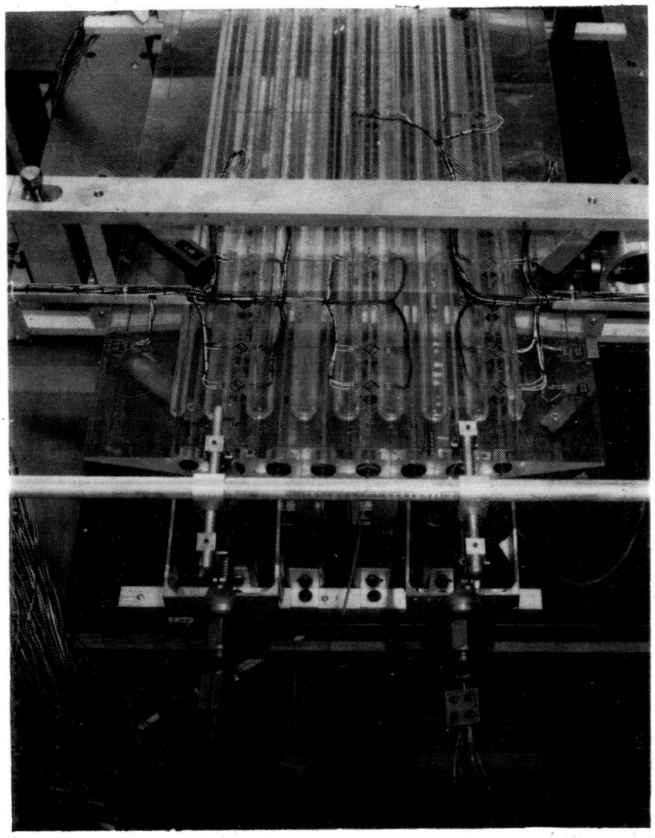


Figure 20 - Three Span Plexiglass Model of a Post-Tensioned Voided Bridge Slab (1/36 Scale)

Figure 21 - Two Span Plexiglass Model of a Post-Tensioned Voided Skew Bridge Slab (1/24)

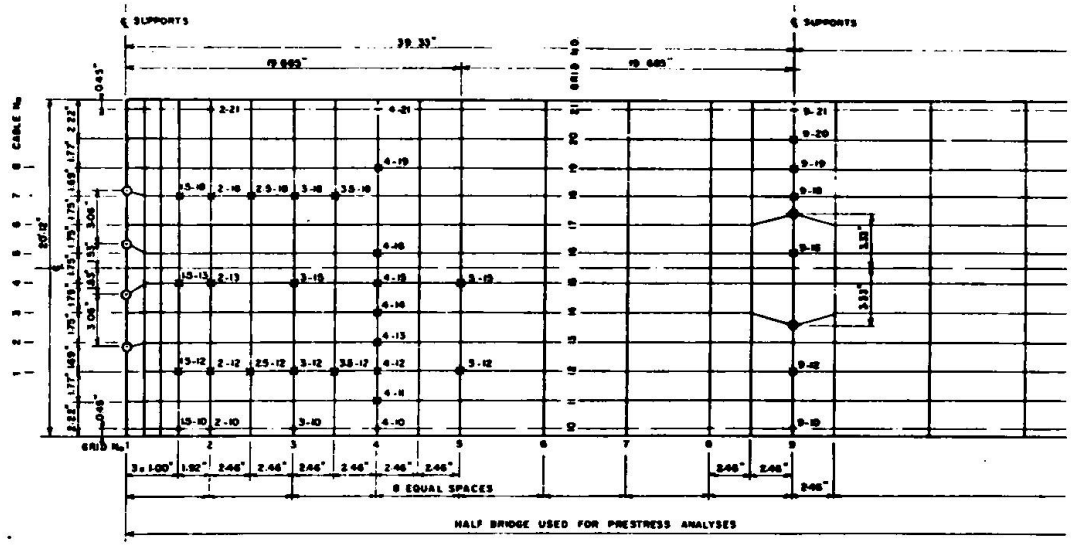


Figure 22 - One Half Plan View of 1/36 Scale Model Showing Gauge and Cable Locations

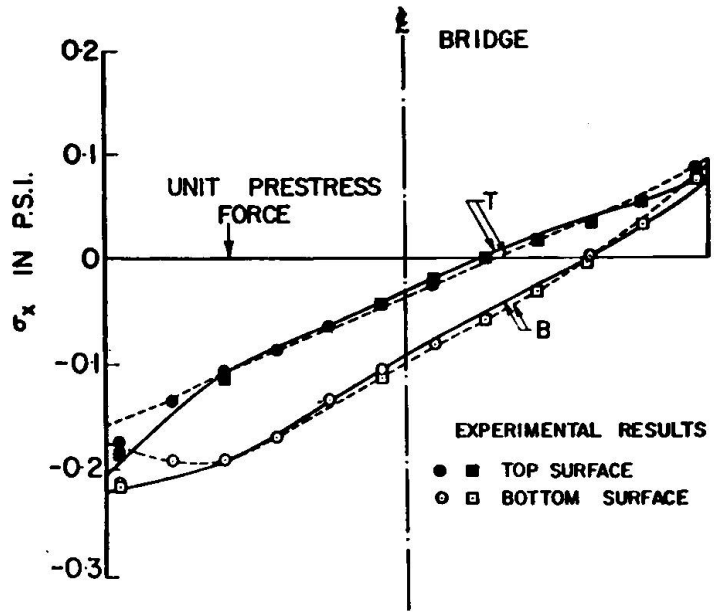


Figure 23 - Longitudinal Stresses Along Transverse Grid Line 4 Unit Prestress in Cable 1

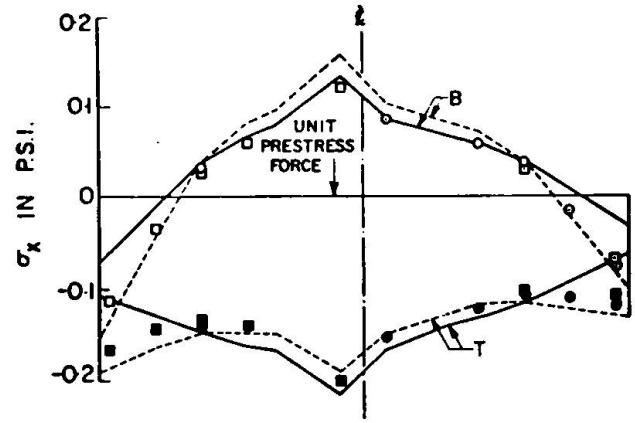


Figure 24 - Longitudinal Stresses Along Transverse Grid Line 4
Unit Prestress in Cable 4

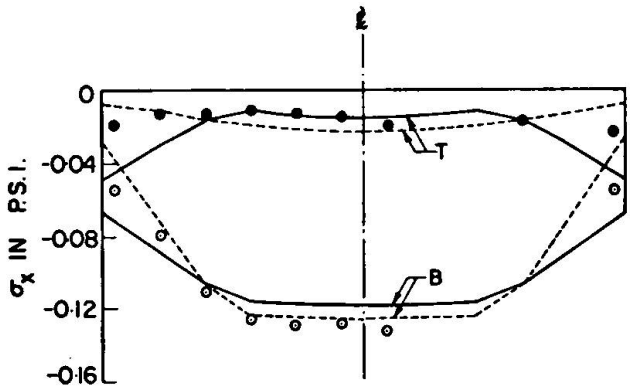


Figure 25 - Longitudinal Stresses Along Transverse Grid Line 4
All Cables Prestressed

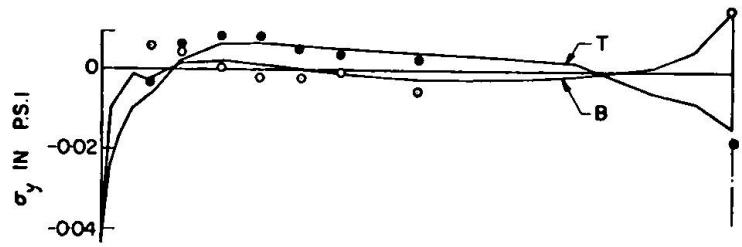


Figure 26 - Transverse Stresses Along Longitudinal Grid Line 12
All Cables Prestressed

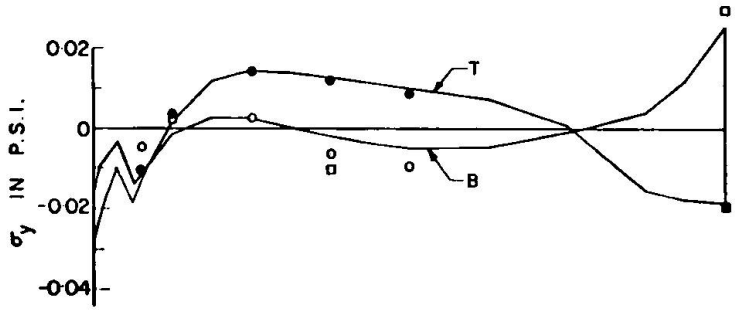


Figure 27 - Transverse Stresses Along Longitudinal Grid Line 15
All Cables Prestressed

quadratic elements to model high gradient strain areas as well as model the curved geometries better around voids, and the lower order elements to model less important areas of the structure. Such a gradation of elements combined with a non-banded equation solver allows much flexibility of analysis at reduced costs.

There are three major levels of activity associated with constructing computer based analyses systems. These comprise the data input level, the mathematical analysis level and the output level. For the 3-D system, the data input is very important since the bridge structure should be automatically described in a form available for analysis. The system being constructed at Waterloo has the capability of producing a complete bridge up to eight spans in length and having any cross-section. There can be both solid and voided regions throughout the length of the bridge. The only stipulation is that the geometry of the cross-section cannot change along the length of the bridge. Work is underway to allow variable width bridges with curved boundaries to be used. The system automatically calculates dead load and temperature and point loads can also be specified. Uniform loads can be positioned over any element and have any value. This is important in dealing with lane loads.

For the analysis section of the system, a large capacity out-of-core solver has been built using matrix partitioning schemes and sparse term storage. Substructuring is now being formulated since otherwise a complete bridge could not be attempted. Individual sections of the bridge can also be formed and analyzed separately in order to investigate local stresses without a full bridge analysis. Three principle stresses are produced at all nodes as well as the usual cartesian values.

The output options of the system will comprise a number of items. Computer plots of individual elements, sections of the bridge or the entire bridge can be selected and viewed from any chosen angle. Figures 28 and 29 are samples of three dimensional output. At the present time, no stress plotting has been developed. This will constitute the final aspect of the work.

Once again, experimental tests on plexiglas models are being done by Professor Green. So far, tests have been conducted on a transverse and a longitudinal section of a typical voided deck. These have been used to establish the element subdivision though the thickness of the slab and along the length of the span. Currently, model tests are being performed on a multi-cell box girder of scale 1/18. This will be used to assess both the 2-D and 3-D finite element systems results.

The system is scheduled to be completed late in 1974. This system, along with the 2-D system that is with the Ministry, should provide a substantial stress analysis capability for both the research and design offices. In this respect perhaps it is fair to say that the Ontario Ministry of Transportation and Communications through its research branch, is one of the most progressive governmental agencies doing structural research in Canada.

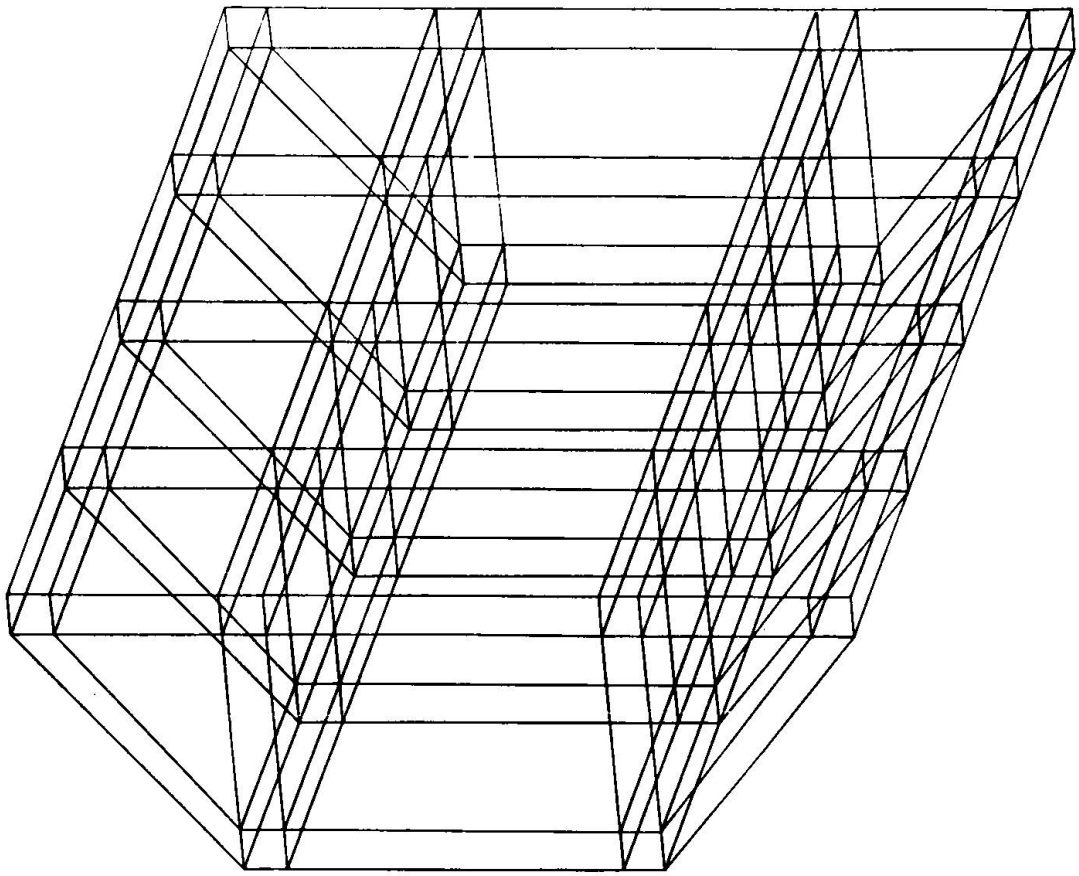


Figure 28 - Three Dimensional Plot of a Three Cell Box Girder Bridge

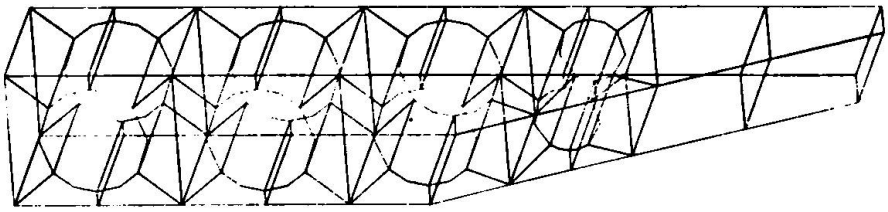


Figure 29 - Three Dimensional Plot of One Half of Transverse Section Test Model

5.0 CLOSURE

In this report I have attempted to summarize on a national basis, the efforts made by various researchers to provide methods of advanced structural and stress analyses for direct application to bridges. The regional subdivision selected was made arbitrarily and does not imply that other researchers in the areas subscribe to the regional work reported herein. Furthermore, the opinions expressed are entirely mine and I stand to be corrected if necessary.

From a national standpoint, there appears to have been substantial developments of computer stress analyses systems in the universities. However, coordination of efforts with potential users such as government design offices has not occurred except in Ontario. The potential now exists and hopefully other provinces will follow the Ontario example.

In closing I should like to thank everyone who has contributed to this report and especially those in the eastern and western regions who made my visits so pleasurable.

REFERENCES

1. Cheung, Y.K., "Finite Strip Method Analysis of Elastic Slabs", Journal of the Engineering Mechanics Division, A.S.C.E., Vol. 94, No. EM6, Proc. Paper 6267, December, 1968, pp. 1365-1378.

2. Cheung, Y.K., "Analysis of Box Girder Bridges by the Finite Strip Method", Proceedings of the Second International Symposium on Concrete Bridge Design, American Concrete Institute, Paper SP 26-15, April 1969, Chicago, Illinois, U.S.A.

3. Cheung, M.S., Cheung, Y.K. and Reddy, D.V., "Frequency Analysis of Certain Single and Continuous Span Bridges", Developments in Bridge Design and Construction, edited by K.C. Rockey, J.L. Bannister and H.R. Evans, Crosby Lockwood and Sons Ltd., London, pp. 188-199, 1971.

4. Cheung, M.S., Cheung, Y.K. and Ghali, A., "Analysis of Slab and Girder Bridges by the Finite Strip Method", Building Science, Vol. 5, Pergamon Press 1970, pp. 95-104.

5. Ghali, A. and Tadros, G.S., "On Finite Strip Analysis of Continuous Plates", Department of Civil Engineering Research Report No. CE72-11, March 1972, University of Calgary.

6. Tadros, G.S. and Ghali, A., "Convergence of Semi-Analytical Solution of Plates", Department of Civil Engineering Research Report, No. CE72-38, September 1972, University of Calgary.

7. Brown, T.G. and Ghali, A., "Semi-Analytic Solution of Skew Plates in Bending", Department of Civil Engineering Research Report No. CE72-46, December 1972, University of Calgary.

8. Sisodiya, R.G., Cheung, Y.K. and Ghali, A., "New Finite Element with Application to Box Girder Bridges", Paper No. 7479, Journal of ICE, London, 1972.

9. Zienkiewicz, O.C. and Cheung, Y.K., "Finite Element Method in Structural and Continuum Mechanics", New York, McGraw-Hill, 1967.

10. Sisodiya, R.G., Ghali, A. and Cheung, Y.K., "Diaphragms in Single and Double Cell Box Girder Bridges with Varying Angle of Skew", A.C.I. Journal, July, 1972, pp. 415-419.

11. Sisodiya, R.G., Cheung, Y.K. and Ghali, A., "Finite Element Analysis of Skew Vault Bridges", International Association for Bridge and Structural Engineering, Publications Vol. 32-II, Zurich, 1972.

12. Mehrotpa, B.L., Mufti, A.A. and Redwood, R.G., "Analysis of Three Dimensional Thin Walled Structures", Journal of A.S.C.E. Structural Division, Vol. 95, No. ST12, December 1969, pp. 2863-2872.

13. Fam, A. and Turkstra, C., "An Annular In-plane Finite Element", Proc. of the Specialty Conference on Finite Element Method in Civil Engineering, McGill University, June, 1972, pp. 1027-1047.

14. Olson, M.D. and Lindberg, G.M., "Annular and Circular Sector Finite Elements for Plate Bending", International Journal of Mechanical Sciences, Vol. 12, No. 1, Jan. 1970, pp. 17-34.

15. Aneja, I. and Roll, F., "Experimental and Analytical Investigation of a Horizontally Curved Box-Beam Highway Bridge Model", Paper SP26-16, Proceedings of the A.C.I. Annual Convention, Chicago, March 1969.

16. Gurevich, S. and Redwood, R.G., "Approximate Analysis of Multi-cell Skew Box Bridges", Proc. of the Speciality Conference on Finite Element Method in Civil Engineering, McGill University, June 1972, pp. 1027-1047.

17. Crisfield, M.A., "Finite Element Methods for the Analysis of Multicellular Structures", Proceedings of the ICE, Vol. 48, Paper No. 7373, March 1971, pp. 413-437.

18. Sisodiya, R.G. and Ghali, A., "Analysis of Box Girder Bridges of Arbitrary Shape", International Association of Bridge and Structural Engineering, Publications Vol. 33-I, Zurich 1973.

19. Sundararajan, C. and Reddy, D.V., "Response of Rectangular Orthotropic Plates to Random Excitations", Faculty of Engineering and Applied Science, Memorial University of Newfoundland, St. John's, Newfoundland, 1972 (private communication).

20. Clough, R.W. and Felippa, C.A., "A Refined Quadrilateral Element for Analysis of Plate Bending", Proceedings 2nd Conference on Matrix Methods in Structural Mechanics, Wright-Patterson Air Force Base, Ohio 1968.

21. Zienkiewicz, O.C., "The Finite Element Method in Engineering Science", McGraw-Hill, 1971.

22. McNeice, G.M. and Jofriet, J.C., "Finite Element Analysis of Bridge Slabs", Report submitted to the Ministry of Transportation and Communications, Downsview, Ontario, July, 1971.

23. McNeice, G.M. and Jofriet, J.C., "Finite Element Analysis of Post-tensioned Highway Bridges", Report submitted to the Ministry of Transportation and Communications, Downsview, Ontario, April 1972.

24. Jofriet, J.C., McNeice, G.M. and Csagoly, P., "Finite Element Analysis of Prestressed Concrete Voided Bridge Decks", PCI Journal, Volume 18, No. 3, May-June, 1973.

25. Green, R., "Prestressing Effects in Voided Slab Bridges", Report Submitted to the Ministry of Transportation and Communications, Downsview, Ontario, April 1972.

26. Green, R., "Skew Voided Slabs", Report Submitted to the Ministry of Transportation and Communications, Downsview, Ontario, April, 1972.

27. Ergatoudis, J.G., "Isoparametric Finite Element in Two and Three Dimensional Stress Analysis", Ph.D. Thesis, School of Engineering, Division of Civil Engineering, University College of Swansea, Great Britain, October, 1968.

28. McNeice, G.M. and Hunnisett, S.F., "Mixed Displacement Finite Element Analysis with Particular Application using Plane Stress Triangles", Journal of Strain Analysis, Vol. 7, No. 4, October 1972.

Summary

This report contains a summary of Canadian research efforts that are directed toward providing advanced analytical capabilities for the analyses and design of concrete bridges. Activities are reported on a regional basis arbitrarily selected as western, eastern and central. The corresponding centers from which most of the developments originate are respectively, the University of Calgary, McGill University and the University of Waterloo. Brief outlines of the theoretical aspects are provided in order to illustrate the varied approaches. Also, comparison with model studies are made wherever possible. With the exception of central Canada, most of the effort has been directed toward box girder analysis using two dimensional finite strip and finite element techniques. In Ontario both two and three dimensional computer systems are being developed. An account is given of a provincial government involvement in the creation of advanced stress and structural analyses systems.

Sommaire

Ce rapport contient un résumé des efforts de la recherche Canadienne dirigés vers l'avancement des capacités analytique de l'analyse et du 'design' des ponts en béton. Les activités sont rapportées sur une base régionale choisie arbitrairement comme celles de l'ouest, l'est, et le centre. Les endroits correspondante d'où originent la plupart des développements son L'Université de Calgary, L'Université McGill, et L'Université de Waterloo. De brefs sommaires des aspects théoriques sont fournis pour démontrer les diverses idées. De plus, des comparaisons avec des études en modèle sont faites chaque fois que c'est possible. A l'exception du centre du Canada, la plupart des efforts ont été dirigés vers l'analyse de poutres-caisson, en utilisant les techniques des bandes finies a deux dimensions et des clements finis. En Ontario, des systèmes de calcul à deux et trois dimensions ont été développés. On se doit de mentionner d'aide accordé par le gouvernement provincial pour la création de systèmes avancés d'analyse de tension et de construction.

Zusammenfassung

Dieser Artikel beinhaltet einen zusammenfassenden Bericht ueber die kanadische Forschungsarbeit auf dem Gebiet der Anwendung theoretischer und numerischer Analysen fuer den Entwurf von Betonbruecken. Die Arbiet stuetzt sich auf Forschungen an verschiedenen Universitaeten Kanadas; haupsaechlich University of Waterloo, University of Calgary and McGill University. Ein kurzer Uebersicht der theoretischen Aspekte ist beschrieben, um die vielseitige Method der Analyse klar zu zeigen. Wo es moeglich ist, sind auch Vergleiche mit den Resultaten aus den experimentellen Modelluntersuchungen gegeben. Die meisten Arbeiten betreffen die Anwendung von zweidimensionalen endlichen Elementen an Balkan. An der University of Waterloo werden auch dreidimensionale systemen entwickelt. Auszerdem ist auch eine Auffuehrung der praktischen Anwendung der Analyse angefuehrt.

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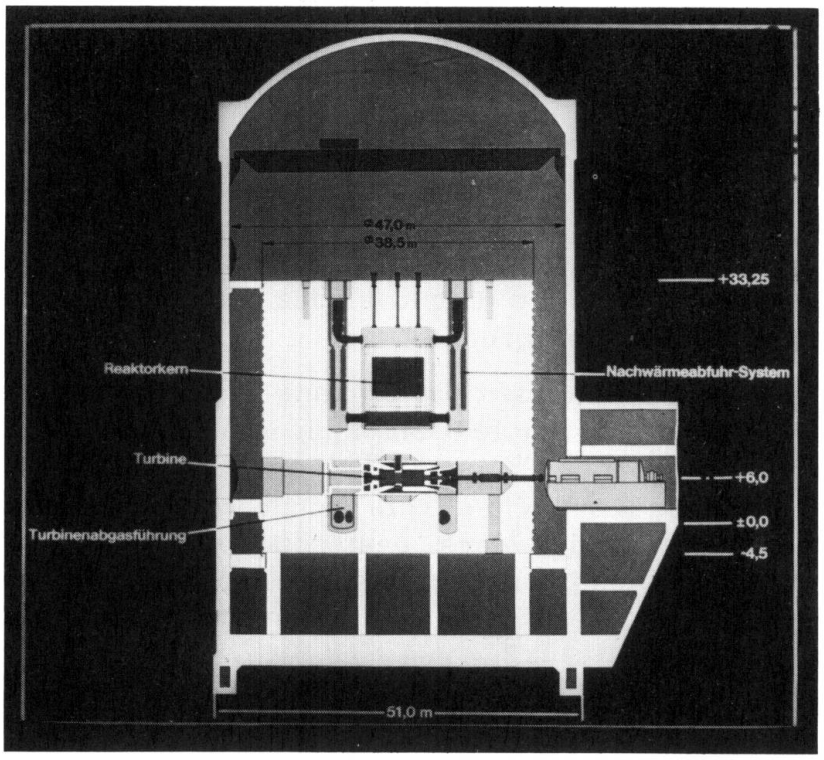
Rapports complémentaires
Ergänzende Berichte
Complementary Report

I-4 Prof. W. ZERNA

Ladies and gentlemen, the joint paper by Mr. Schnellenbach, Mr. Schimmelpfennig and myself gives a report on calculation and construction of concrete structures subjected to triaxial stresses with particular reference to prestressed concrete reactor pressure vessels. The authors report mainly on their work done at the university of Bochum in the Federal Republic of Germany, but nevertheless you may look upon this paper as a kind of national report, though some more activities in Germany have taken place. At this conference some more papers by authors from Germany will be presented. If you want to get more complete information on the work done in the Federal Republic of Germany in this particular field you'll have to consider these papers as well. In our paper we touch on to our mind, the most important problems which always have to be solved in the design of structures subjected to triaxial stresses, such as an example for prestressed concrete reactor pressure vessels. I may direct your attention particularly to the methods of prestressing described here. Also difficult construction problems concerning the liner are considered. We have described a particular calculation method called dynamic relaxation which you probably know. But it may be of some interest to you that we have given some information about experiences in the application of this method which has to compete with the very wellknown method of finite elements.



Of particular interest may be the chapter where the cracked state of concrete is considered and the chapter on the calculation of ultimate load behaviour. We have pointed out how to take account of complicated geometry conditions, of space and time dependent variations and non-linearity of material behaviour, and of the influence of variable temperatures. We think our knowledge on this type of structure has increased in the last years considerably.

To finish up my short remarks I may show you a slide not given in this paper. You will see the so-called HHT reactor where a gas turbine is used with a horizontal axis. If you consider the PCRV on this slide you will find that there really exist triaxial stress states. The investigation of this type of reactor has just started. Thank you.

I-6 Prof. E. FUMAGALLI

M. le Président, Mesdammes et Messieurs, dans mon rapport je traite la reproduction et l' expérimentation des structures massives par modèles statiques Dans ce domaine on doit considérer trois types différents de modèles:

1) Les modèles des conténiteurs nucléaires pour lesquels le poids propre ne représente pas une composante importante dans leur comportement statique. Ces modèles seront traités d' une manière spécifique dans un autre rapport.

2) Les modèles statiques traditionnels des barrages dans lesquels le massif rocheux d' appui intervient seulement pour assurer les conditions au contour de l' ouvrage. Dans ces modèles l' application de l' effet de poids propre est limitée au corp du barrage. L' installation est realisée par des câbles mis en tension par des ressorts étalonnés ou par une planchée mobile et des anneaux en gomme à grand allongement.

3) Les modèles géomécaniques utilisés pour le contrôle d' instabilité des massifs rocheux, particulièrement des falaises de fondations aux barrages en béton. Il s' agit éminemment de modèles à essayer à la rupture. Dans ces modèles on reproduit un schéma géologique avec les accidents et les discontinuités plus importantes qui peuvent intervenir dans l' instabilité du système rocheux.
Pour éviter l' application du poids propre par les câbles susdits (câbles qui représenteraient des liaisons inadmissibles en domaine de grandes déformations) on utilise des matériaux à haute déformabilité, c' est-à-dire qui disposent de caractéristiques mécaniques réduites à peu près dans le même rapport de l' échelle géométrique. En tel cas pour respecter les forces de masses, il est suffisant d' employer des matériaux qui disposent d' un poids propre égal à celui de la roche.

Brèvement pour fournir aux participants des idées sur cette technique, je vous présente les résultats sur le dernier modèle essayé à l' ISMES: il s' agit des essais sur le barrage de "Canelles" (Espagne). La fig. 1 représente le schéma reproduit dans le modèle à trois dimensions. Des trois systèmes de discontinuité représentés par les blocs, deux sont coïncidents; le troisième présente un gisement différent dans les deux falaises. Par l'expérimentation des deux modèles, on a vérifié en rive droite deux solutions de renforcement du rocher qui offre des appuis plutôt corticaux.

La première solution était représentée par une installation de câbles de précontrainte ayant le bût d' empaqueter les couches du rocher sub-verticales et de augmenter par conséquent les forces résistantes de frottement. La seconde était représentée par un contrefort en béton d' intégration aux courbes de niveau (fig. 2). Le contrefort était relié aux couches intérieures du rocher par des tunnels revêtus en béton précontraint.

Les essais ont démontré que la première solution, même si elle augmente la rigidité du système, n' empêche pas de brusques tassements de plasticité et le danger de ruptures fragiles superficielles.

La deuxième solution s' est démontrée plus efficace et moins. couteuse. Elle assure une résistance d'ensemble plus élevée et s'insère d'une manière harmonieuse dans l'équilibre statique naturel du système. En rive gauche le massif d'appui résulte plus volumineux mais plus déformable à cause de plus fréquentes discontinuités

Le cadre des déformations aurait été jugé de toute sécurité si par le dépouillement des résultats, après les essais à rupture du premier modèle, on n'aurait pas constaté que, à front des déformations éminemment réversibles dans la direction de la poussée transmise par le barrage, des déformations permanentes assez importantes s' étaient accumulées en direction presque normale aux précédentes (fig. 3). Parmi des cycles impulsifs à charge toujours plus élevée, des processus de disorganisation dans la roche s' étaient manifestés, donnant lieu à un effet d' ouverture en éventail des couches verticales et par conséquent à un danger de ces dernières en direction normale à l' axe de la vallée (fig. 4).

Par conséquent, dans le deuxième modèle on a reproduit des tunnels précontraints tout à fait semblables à ceux de la rive droite (fig. 5). Dans le paquet des couches verticales ainsi obtenu les essais ont démontré que si d'une part la déformabilité du système ne s' était pas réduite en mesure appréciable, par contre l' éffet d' une ouverture en éventail était totalement disparu.

Un tel comportement porte à une première considération: dans les systèmes discontinus, les déformations permanentes ne représentent pas, en principe, en pourcentage des déformations globales. Elles peuvent se dérouler d' une manière totalement indépendente aux déformations élastiques et obéir à des lois propres suggérées par la logique des discontinuités. C' est pour ça qu' elles peuvent même se présenter en direction tout à fait différente par rapport aux résultantes de charge. A ce sujet, on doit encore examiner le diagramme de déformation radiale mesurée à la clef du couronnement du modèle pendant les cycles à outrance (fig. 6). Le diagramme susdit documente comment la structure parvient à la rupture très lentement selon le cadre des fissures (figg. 7 et 8).

Une observation qui sort d' évidence c'est que les essais à outrance éxécutés par cycles impulsifs résultent plus lourds par rapport à des essais conduits par charge de longue durée (fig. 6). La vitesse moyenne de déformation qui en résulte est en effet plus élevée.

Si l' on passe à des considérations d' ordre général et interprétatif on observe qu' aux cycles alternés font suite des processus important de déformation différées. En plus les cycles ne sont pas réversibles et dénoncent des phénomènes d' hystérèse non négligeables.

Le modèle est encore en vision à l' ISMES et je reste à votre disposition pour des éventuels renseignements de détail.

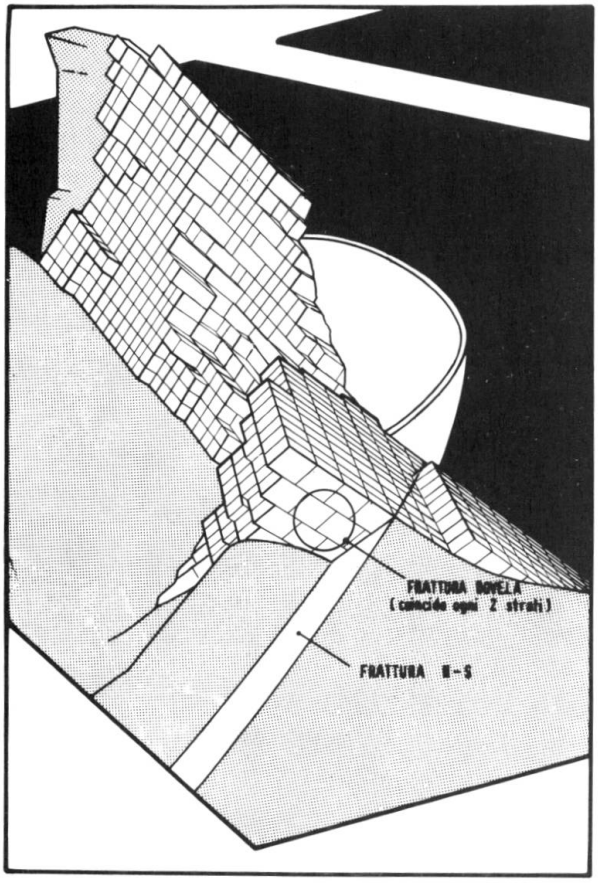


FIG. 1

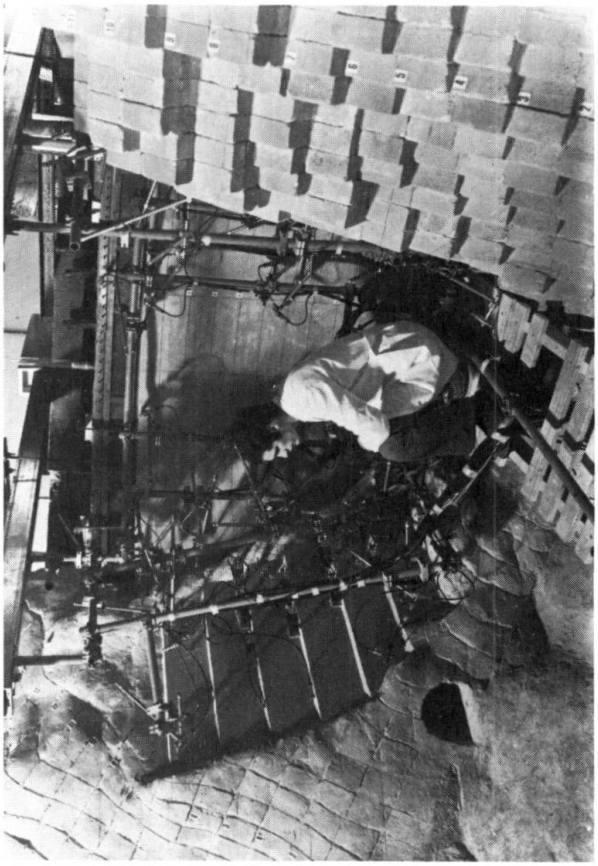


FIG. 2

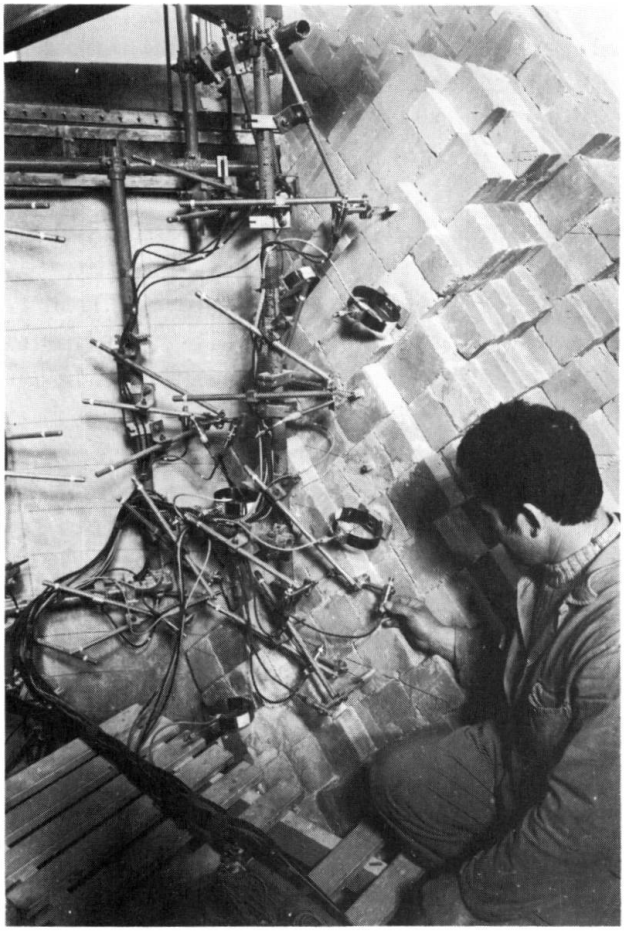


FIG. 5

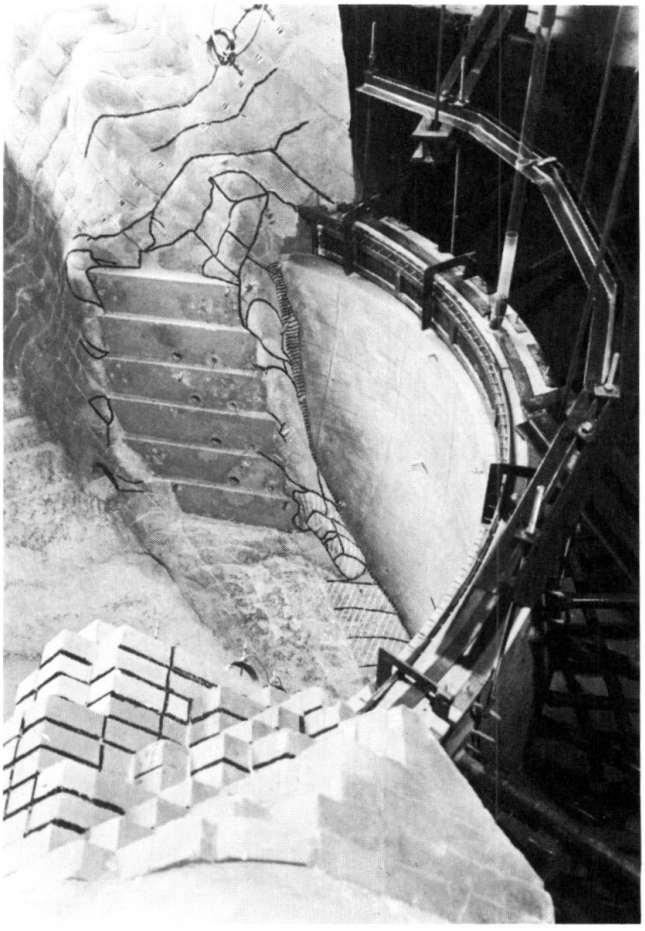


FIG. 8

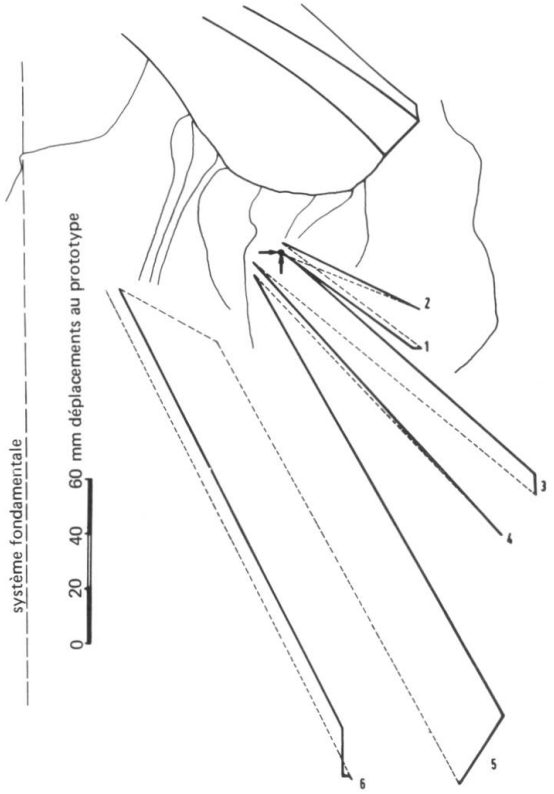


FIG. 3

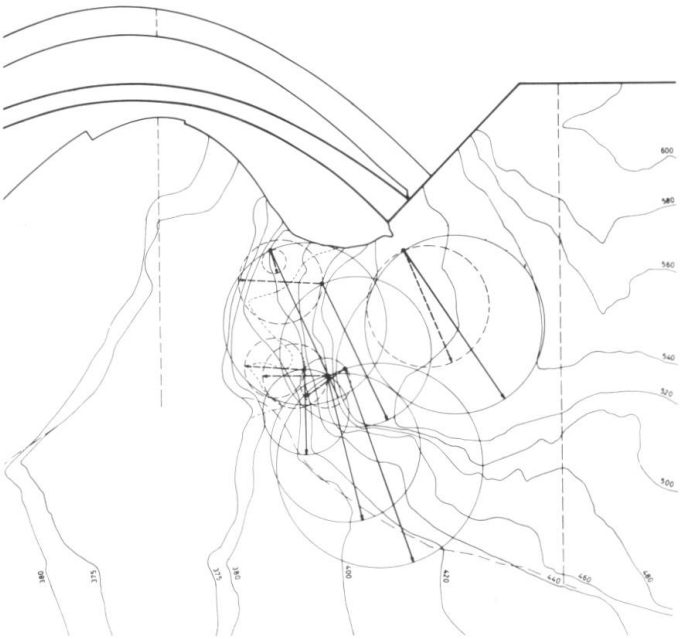


FIG. 4

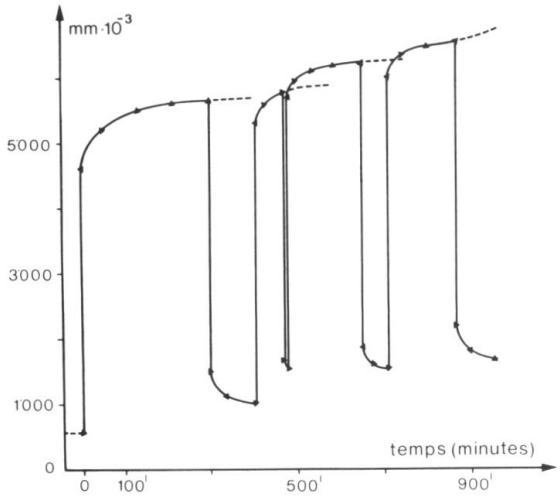


FIG. 6

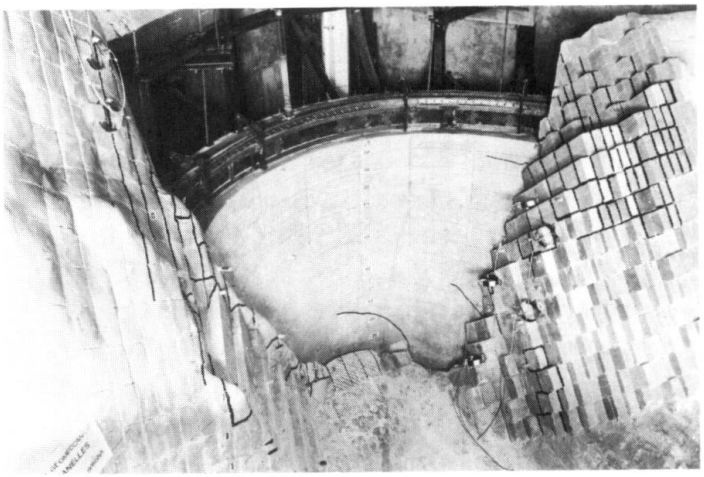


FIG. 7

Prof. M. COSANDEY

Vous avez vu que dans le programme il y a deux présidents. Alors nous allons dans cette deuxième partie exercer la participation M. le Prof. Finzi et moi. Alors c' est M. Finzi qui commence.

Prof. L. FINZI

There are two discussions concerning paper no. I-4 by Mr. Zerna. The first question is by Mr. Kawamata asking: Could you comment on the advantages and disadvantages of the dynamic relaxation method as a tool of design and comparison with the finite element method?

Could you explain your question more fully and then get the answer from the author.

Dr. S. KAWAMATA

I think the dynamic relaxation method and the finite element are representative tools for the design of PCPV today and both the methods are quite different, the dynamic relaxation taking esclusively the iterative form and using a finite difference system rather than a finite element system. What are the advantages and disadvantages of the dynamic relaxation as a practical tool of design.

Prof. W. ZERNA

Of course this is always a question to compare different methods.

But actually it is not always possible to compare, it depends on the problem. We have problems where one method or the other has advantages or disadvantages. If you use the one or the other method, sometimes it is also a matter of how you have started in developing programmes. And then you have the programmes and you improve them. Thus, your method has advantages compared with the other methods you do not know. Of course it also depends on the computer you use. We did some work on dynamic relaxation, Mr. Argyris used the finite elements method and we did some comparisons. We have found that for the problems we have considered so far there actually is not so great a difference that one could say, it is important to use one or the other method. There may be of course certain problems where you have advantages with the one or the other method, but on the whole it depends on so many parameters that you cannot give one single answer to that question.

Prof. L. FINZI

Thank you and now Mr. Schimmelpfennig, one of the co-authors of paper I-4, would like to add something to the paper.

Mr. K. SCHIMMELPFENNIG

Ladies and gentlemen, in the interval between sending the paper to the secretary and this seminar I got some more recent results with a little advance concerning compression fracture in triaxial loaded bodies, I think. Let me show you some of these results. The quality is not very good, but I hope that you can see the things that are important. I used for my calculation deep restrainted slabs which were tested by Taylor & Woodrow as published in Berlin last year by Dr. Garas. These are slabs with a diameter of 30 cm and a height of 7.5 cm, pretressed laterally with 57 Kg/cm^2 or 114 Kg/cm^2, respectively. The figures show radial sections of the slabs, and also the concentrical loading plate and annular bearing plate. The principal stresses in the radial planes are plotted for each computing mesh. In Fig. 1.1 it can be seen that tensile cracking starts near the edge of the loading plate. On Fig. 1.2 one can see that tensile cracking is going on in this region and also starts in the center of the slab and near the edge of the bearing.ring. In this stage compressive cracking starts at the edge of the loading plate. Fig. 1.3 shows a state when load is already decreasing. Tensile cracking has spread throughout the critical plane. Compressive cracking has enlarged and like in a central plug is going to be punched downwards. Where compressive cracking has taken place it is assumed that the material behaves isotropic, the shear modulus being reduced in a certain mode. One can say that the concrete there has gone into a granulated state, like coarse sand. Now some figures where the supporting ring is larger and thus the critical plane is vertical. Again in Fig. 2.1 you see the start of tensile cracking at the edges. In Fig. 2.2 tensile cracking has spread throughout the critical plane and compressive cracking starts at the edges. In the state shown in Fig. 2.3. compressive cracking has proceeded along the whole critical plane and this is already a state where load decreases.

All these results are not systematic up to now, but I hope that in the near future I shall be able to make progress herein. These effects of compressive cracking are very important when failure of structures subjected to triaxial stresses has to be examined. Thank you.

Ing. F. SCOTTO

Dr. Zerna, I was very much interested in the calculations that you have made in order to try to evaluate the effect of the creep on the PCPV structures. My first question is the following: you have tried to evaluate the deformation of the structure, due to creep in function of the time. This is one topic for the design of pressure vessels because we must know what happens to the penetrations during the life of the structure with special care for the operability of the control rods. Under this respect, I believe that in the area of the penetrations, at present, we are not in the condition of simulating the hundreths penetrations we have, and therefore to predict correctly the local deformations by means of calculations.

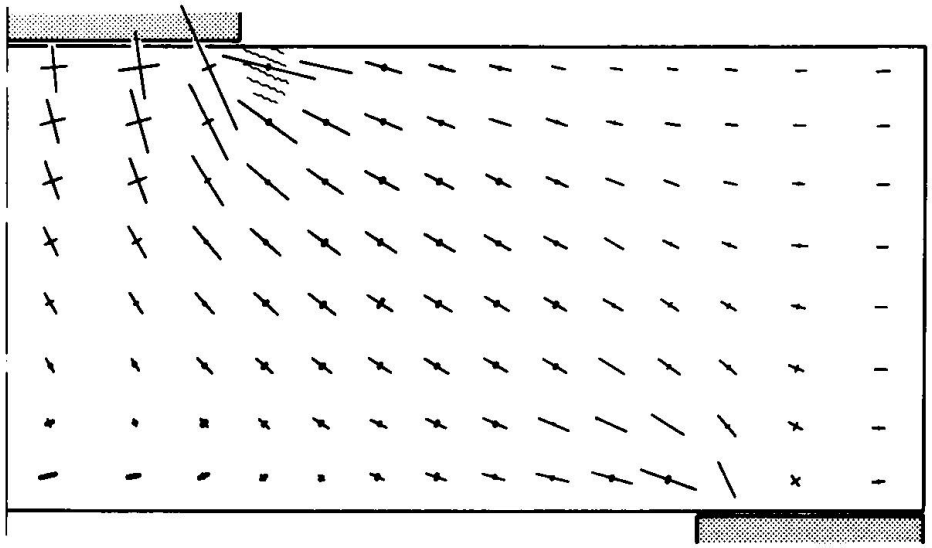


Fig. 1.1

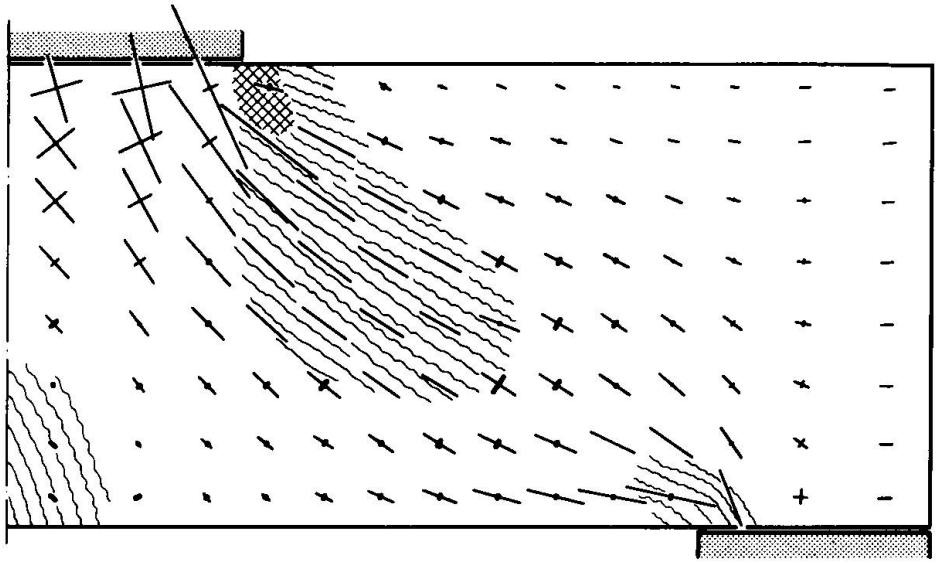


Fig. 1.2

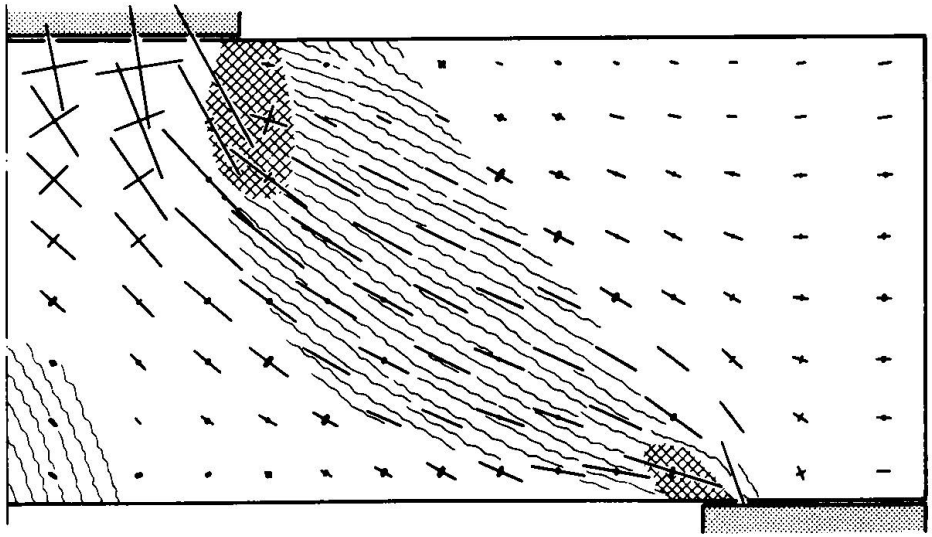


Fig. 1.3

Key for figures 1 and 2:

compressive principal stresses

tensile principal stresses

0 500 kp/cm^2

regions with tensile cracking, lines showing crack directions

regions with compressive cracking

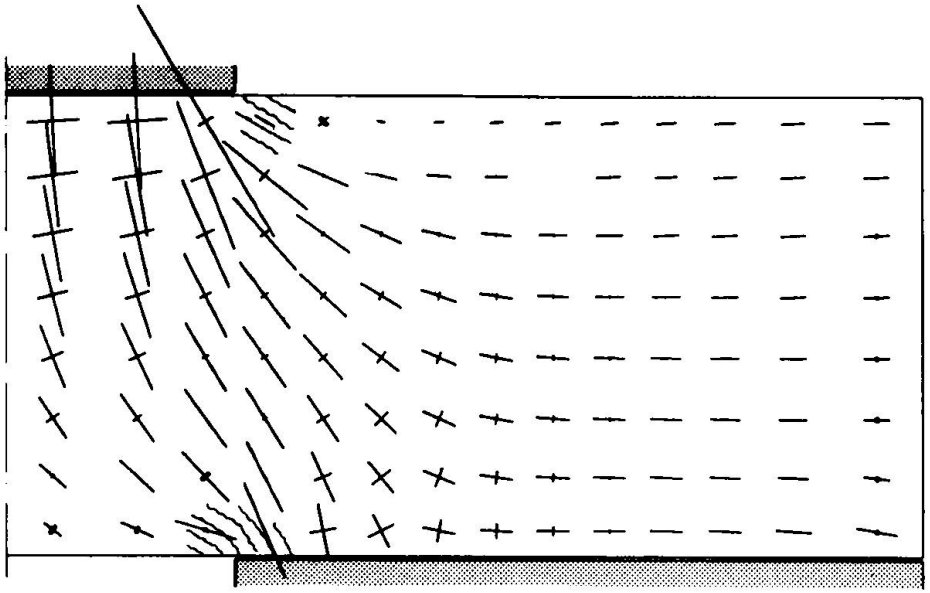


Fig. 2.1

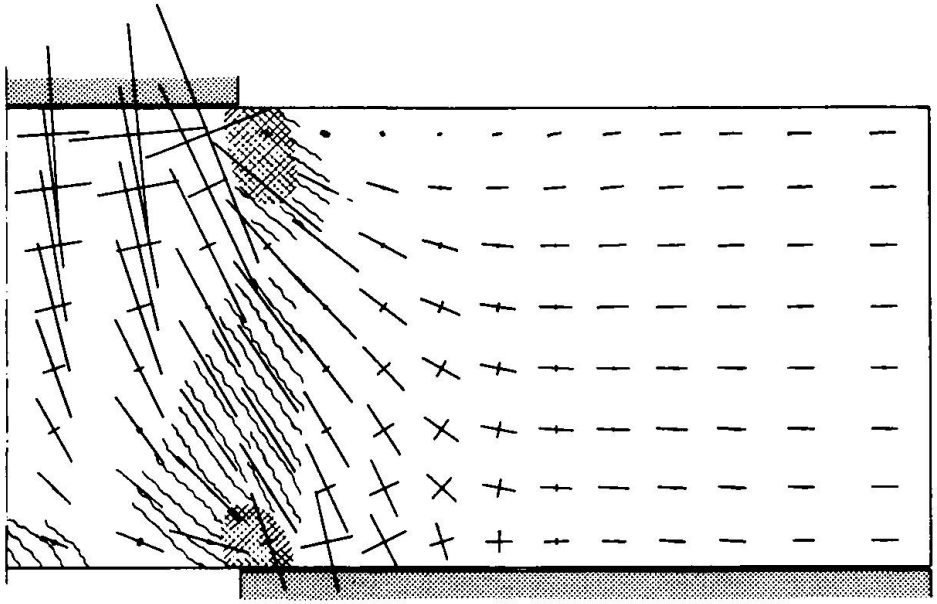


Fig. 2.2

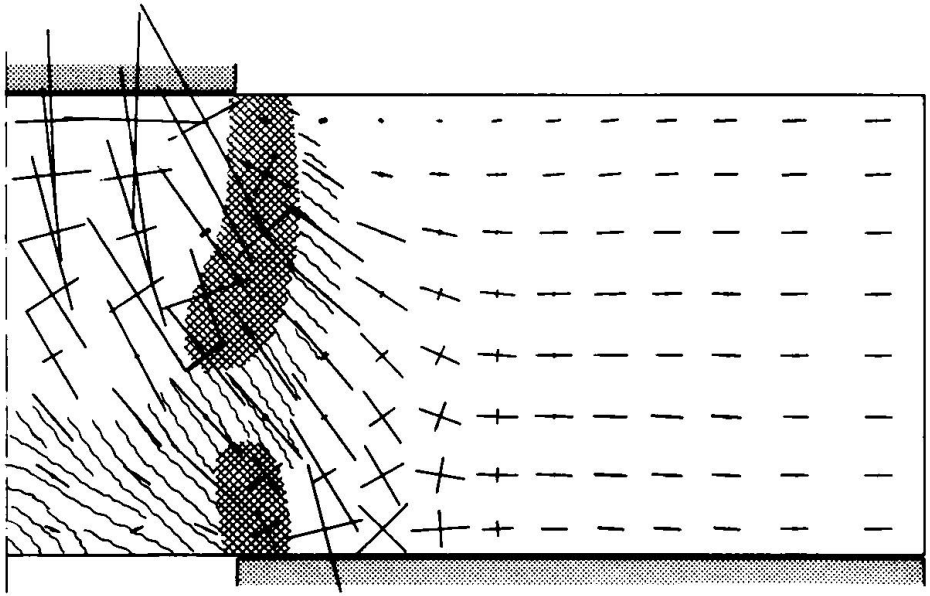


Fig. 2.3

Key for figures 1 and 2:

compressive principal stresses

tensile principal stresses

0 500 kp/cm²

regions with tensile cracking, lines showing crack directions

regions with compressive cracking

The second question is related to the governing laws to be selected for creep modelling. With reference to big gradients of temperature, and big gradients of stresses typical of the penetration area.

Now the point is, what is your opinion about these two topics, the possibility to evaluate the more probable deformative behaviour versus time, the reliability of the creep governing laws you have applied to.

And third, which is your opinion about the degree of safety of the concrete versus time because of the effect of creep. Thank you.

Prof. W. ZERNA

Thank you very much for your questions, Mr. Scotto. On the whole I think you are right, there are certain problems and I am afraid I cannot give you a completely satisfactory answer. We cannot calculate all these problems which have been mentioned in such a way that we actually simulate the physical behaviour. I think, however, that it is not necessary. What we want is nothing else than to have a design which is safe. There arises of course another problem in connection with the deformation, not only with the penetrations but also with the liner we have in the vessel, which is influenced by the deformations of the whole vessel and also of course in connection with the penetrations. As the design philosophy is today, we calculate, I may say, just what we can calculate, and then of course we have to introduce some factors of safety. A safety factor must be greater the worse our calculation is and it may be smaller if our calculation is improved. But I think on the whole that our calculation method does not matter, finite elements or another method, the dynamic relaxation. It gives us in any case a good idea about the magnitude of the stresses and deformations and so on, so that our design is such that we have a safety which is sufficient. I do not know if this answers your question completely, but I think that the point you have raised is too difficult to go into with all the details connected with it.

Prof. W. DILGER

I have a question about the final statement of Dr. Schimmelpfennig with regard to the compression cracking of concrete. I have been working on reinforced concrete problems particularly in shear for many years but I have never come across the term "compression cracking" in the literature. Do you mean the splitting of concrete due to concentrated axial forces or do you mean the cracking which is observed in concrete prior to compression failure? Cracking in concrete normally occurs due to tensile stresses and not due to compressive stresses.

Dr. K. SCHIMMELPFENNIG

My opinion is that if you consider principal stresses you cannot speak of shear failure because there are no shear stresses That is why I used this term. Let me add the following: cracking can be determined by a certain failure envelope, usually in the triaxial stress space like in Dr. Browne's report for example. Not all these cracking states on the failure envelope are tensile cracking states and you must define a certain limit between tensile cracking

and compressive cracking. Since it is not yet possible to get sufficient information from tests about this limit between tensile and compressive cracking I have done some parametric studies checking several assumptions in order to get a little step forward in describing cracking behaviour of concrete continua.

Dr. K.J. WILLAM

I would just like to follow up that previous question asking what kind of failure criterion is Dr. Schimmelpfennig using for describing concrete failure under triaxial conditions (and what kind of constitutive model is adopted in the case of cracking and shearing).

Dr. K. SCHIMMELPFENNIG

In former time we have done some work in our institute summarizing the results about this fact and we developed a failure surface which is very similar to that Dr. Browne showed in his slides. If you consider theoretical demands, it is not correct that this failure envelope has sharp edges, but if you treat the problems numerically that does not matter, I think. So it is good enough and moreover it is always on the safe side, as we have been able to demonstrate.

Prof. O.C. ZIENKIEWICZ

A further question for Dr. Schimmelpfennig. While I agree that a Mohr type envelope is a reasonable approximation for the ultimate behaviour, and indeed we have been using this ourselves, the problem remains, what happens to material after crushing has occured. When failure has been reached in concrete, the material becomes sand-like and loses its cohesion, nevertheless in a compressive environment it still possesses a certain degree of strenght. How does Dr. Schimmelpfennig deal with this matter?

Dr. K. SCHIMMELPFENNIG

I think that indeed it is very important to take care of the mode of decrease of stresses versus strains after failure. If the crushed region is near the surface of a body, then there will be no restraint and stresses immediately fall down. But if it is confined on all sides one must take suitable assumptions for the decrease of stress-strain curves. As I do not know any experimental results about this I have made an assumption on this subject with a decreasing curve like a parabole and I have checked different slopes. I think one can find by this way a reasonable solution. But you must indeed take account of this effect, you cannot calculate realistically if you consider an immediate fall down when the element is confined on all sides.

CHAIRMAN

Alors nous avons maintenant une question touchant le point 7, il s' agit d' une question de M. Mérot à M. Scotto. Comme M. Scotto est multilingue je pense que je peux poser la question en français, et je serai reconnaissant si

vous voudrez bien ensuite la traduire en anglais parce que vous prendrez la parole. A page 6 de votre communication le tableau montre une très grande économie de précontrainte dans la solution mince. Comment cela est-il possible, puisqu' il est connu que la section nécessaire de la précontrainte est déterminée dans un projet épais comme dans un projet mince par la sécurité par rapport à la rupture.

Ing. F. SCOTTO

I have shown in one slide , relating to the safety factor of the structure, that the thick and the thin solutions are, more or less, in the same conditions as far as the safety factor of the structure is concerned. In table 6 of my paper you can find the great difference in the quantity of the concrete between the thick and the thin solutions. The unit quantity of prestressing steel for cubic meter is more or less the same for both the solutions (of the order of 150 Kg/m^3). This is the reason for which we have this so big drop of the cost on this item due to the important reduction of the steel quantity. As it is known at present the cost of the prestressing is 2 $ per Kg in our country.

Prof. J. P. MEROT

Je voudrais bien d' abord féliciter le Dr. Scotto pour ses études qui sont très intéressantes et je ne voudrais pas qu' il croie que je suis seulement critique. Je suis d' accord avec lui sur beaucoup de points. Il est certain que avec les connaissances actuelles on peut faire des caissons avec des épaisseurs de béton plus petites. Il est certain qu' il sera possible de faire comme cela des économies et que c' est intéressant aussi pour les fondations, pour la résistance au séisme, pour beaucoup de choses comme l' a dit le Dr. Scotto. Il est certain aussi que quand les dimensions du caisson diminuent la longueur des armatures de précontrainte diminue et donc on fait certainement une économie. Ce qui m' étonne c' est la chose suivante: si je prends un exemple très simple, comme un caisson pour un réacteur graphite-gaz, comme on en a construits en France, la plus grande partie c' est la paroi latérale cylindrique. On détermine habituellement en France la section nécessaire des armatures de précontrainte (par exemple pour les armatures circonférentielles) en vérifiant qu' à rupture elles équilibreront l'effort dû à une certaine pression qui est la pression de service multipliée par un coefficient de sécurité qu' on peut discuter mais qui est imposé che nous à 2,5. A ce moment-là la section et donc le poids des armatures de précontrainte est déterminé. Et ensuite on détermine les autres paramètres, comme par exemple le gradient de température admissible pour que le béton résiste avec cette précontrainte déjà déterminée.

Si on prend, comme le fait le Dr. Scotto, une solution plus mince, les câbles vont être courts et donc si le Dr. Scotto me disait, j' économise 10-20-30% de la précontrainte, cela me paraîtrait tout à fait normal. Je crois qu' on ne peut pas économiser la moitié ou plus de la moitié de la précontrainte et je pense que le projet de référence sur lequel s' est basé le Dr. Scotto n' est peut-être pas très bien étudié.

Ing. F. SCOTTO

I thank Mr. Mérot for the appreciation to my work. It is true that the reference German design was a little conservative as far as the prestressing steel is concerned, because of the initial prestressing level of stresses allowed at that time by existing German codes ($\simeq$ 0.6 K_{UTSG} instead of 0.7 K_{UTSG}). But the fundamental of the cut down of the costs, that is of the prestressing steel (2 $ per Kg), for the "thin solution" lies in the fact that the prestressing steel is not sized in order to achieve a reference safety factor say 2.5 or 3. The prestressing steel is sized in order to achieve an overall compressive state, in the concrete structure, for the pressure and temperature corresponding to the design conditions. From our experience on small scale models, we know that with the conventional way of sizing the prestressing steel, as you have indicated, the real structural safety factor is more than 4 because of the complex interactional effects (contributions) of the concrete cable ducts etc.

In addition we have a general reduction in the length of the cables and as a matter of fact we have found that practically we have an average density of prestressing steel for thick or thin solution of the order of 150 Kg/m^3.

In others words, the cut down of the cost depends on the different design philosophy we have adopted on the basis of our experience acquired in testing several small scale models in the ultimate conditions.

Prof. L. FINZI

There is a question concerning paper I-9 presented by Mr. McNeice, the question comes from Prof. Baker: will the curved box girder bridges lose transverse stiffness by excentric liveload causing longitudinal yield lines? Is the dead live load ratio sufficient to prevent this? Under what circumstances could this happen?

Prof. G.M. MCNEICE

We had a little chat about this at coffee and unfortunately I cannot give you the answer, the reason being that the three dimensional analysis is what we really have to have in order to answer it. What we do in the finite elements analysis is to go through the elastic range until we get to the cracking stress level, then we change the stiffness of the element so as to approximately simulate the cracked stiffness. Then go through another stage whith that. Comparing those numbers with what we can get from a perspex model may give us some feel for the so-called shear racking across in a transverse section. It is a very great problem basically I think because when you are post-tensioning your webs, if you do not watch the contractor very closely for example, he will not follow the pattern that the designer has laid out unless you stand beside them. He does not want to move his jacking equipment as he is supposed to. He wants to leave it in one place as long as he can and do as much tensioning in that position as possible, before moving on to the next one. What you are doing then is of course lifting your structure in a biased manner and you are already introducing initial stresses into the system. Now we hope from the three dimensional analysis that we will be able to give some definitive statements concerning the construction practice, because if we do not know the initial stress possibilities it is very difficult to answer the final question at all. I cannot give you numbers unfortunately, perhaps next time.

Prof. L. FINZI

Now Mr. Scotto from ENEL would like to ask some of the authors that have presented papers here this morning what their position is as far as the influence of the gas in the concrete is concerned, influence on safety and so on. I think that Mr. Scotto could explain his question a little more fully.

Ing. F. SCOTTO

One of the topics we have to face to, in designing the PCPV, is the condition in which we have an incidental leakage of gas throughout the liner. In my opinion at present the realistic way to approach the problem is that we must try to avoid this incidental condition by means of a suitable purging system, but in any case, the question that I would like to put here in this specialized seminar, is if some authors can expand on this subject and, in particular, if some people has made any test in order to know which is the safety of a concrete subject to the combination of a triaxial state of stress and an internal gas pressurization.

Prof. W. ZERNA

I think the answer to this question depends very much on the philosophy you introduce into the question, what is meant by safety for such a vessel. I perhaps can say at the moment what we have done in Germany in this case. We have assumed that the gas penetrates through the liner and then gas pressure acts in the cracked concrete. Then it must be guaranteed that the vessel can still take all forces and stresses which occur in this case. In other words, I may introduce the wellknown term "pressurized crack". We calculate the pressurized crack, but that is only one way perhaps to meet this point. You could consider this of course in a quite different way, but actually this is more or less philosophy.

Prof. A. L. L. BAKER

I should like to ask Prof. Zerna about the heat effects on the cables, because in many desings the gas is at a very high temperature and if penetrating the cracks it would not take very long to heat the cables and weaken them very considerably. I cannot speak directly about tests that have been done in England, they have been carried out by the Atomic Energy Authority, but they did carry out tests on models by gas pressure as distinct from water pressure. The earlier tests were done with water pressure and I think as a result of that this question of gas in cracks was not studied sufficiently in the early days. But fortunately the safety factors of the early designs were pretty high and I do not think there is anything to worry about, particularly because the capacity of the safety valves is also high and therefore one gets release of pressure. One should do so long before pressure rises to a value which would start splitting the lining. But in the model tests which were done, immediately the lining split the whole vessel explodedor at least in most cases, and therefore it does look as though the limiting factor in a gas pressurized, gas cooled reactor is the splitting of the lining. As soon as the lining splits, cracks in the concrete widen and the gas pressure extends around the wall, the cables are heated at the same time and failure occurs. In the code of practice which has now been published for pressure vessel designs,

for gas cooled reactors in England we have special clauses for design against gas in cracks and we also have clauses in regard to the capacity of the safety valves, so that for the worst kind of accident which may occur, when the controls breakdown or fail, you can be sure that the safety valves will release the pressure before it can reach a value at which it will split the lining. There is a good chance that if one did in some extraordinary accident have a failure it would be due to splitting the lining, not the lining remaining intact and the cables breaking.

Prof. W. ZERNA

On the whole, Prof. Baker, I agree with what you have said. But here I think we have to distinguish between what actually can happen and what is only a merely hypothetical assumption. We have also introduced the term liner integrity, and that means that we want to know what the factor of safety is with respect to the liner. But we think that in reality the pressure can never increase up to such a critical level. We only assume such a case for calculation reasons. I think that is very important, because what actually can happen and what is only considered for calculation reasons is very often confused in the idea. I think also the experiments which have been done in this respect are wrong, because they have increased the pressure and they have shown an explosion of the vessel, but in reality the inside pressure of the gas is limited to a value according to the whole system of the reactor. It is impossible for it to increase and therefore we have do distinguish what we are calculating and what is the physical reality.

Prof. A.L.L. BAKER

There is one possibility of failure which the control rods and other safety devices will not cover and that is a fracture in a water header in the boiler system resulting in water turning into steam very rapidly. It has been calculated in certain cases that the pressure can rise so rapidly that the safety valves can scarcely deal with it, and the margin of safety is not as wide as one would like for that particular type of accident. Therefore one really needs to look at the gas in cracks position and make sure that one has got an adequate safeguard in case of excessive steam pressure due to a water header failing, and perhaps the safety valves being corroded.

Prof. A.D. ROSS

I think this seminar has touched on perhaps the most vital point - which Dr. Scotto raised - in the design of gas cooled reactors. It is true, as Prof. Baker said, that in our British Standard, which has now been published, we have required a minimum load factor for this pressurized crack condition, but we really do not know what the distribution of pressure along the crack might be and this is an important unknown. In my own mind I differentiate two cases of failure of the liner. The large split that I think Prof. Baker and others have talked about is one case, and I agree with Prof. Zerna, that this is really virtually impossible as I see it.

But what is always possible, is that there may be small leaks, allowing gas under pressure behind the liner and thereby possibly leading to pressurized cracks, the condition that we wish to avoid. I do not know what the answer is. Perhaps I might mention - although the work is not mine - that in England we are trying to discover, on an actual vessel, if a small gas flow could escape easily through the concrete structure or, if any cracks exist, whether they might be pressurized because of "choking". It is not much good examining small specimens for this purpose because there are construction joints and other leakage paths in the real structure along which the gas might escape. Hopefully in due course we might have a little more information on this point which I regard as being of primary importance.

Prof. J. P. MEROT

Je voudrais répondre au Dr. Scotto en lui disant ce qui se fait en France, bien que je pense que je ne lui apprendrai pas grand' chose. A ma connaissance il n' a pas été fait de tests sur l' effet de la pression dans le béton. Le Dr. Scotto sait fort bien que dans le règlement français il y a une prescription au sujet de l' établissement de la pression de gaz dans l' épaisseur de la paroi de béton. En fait cette prescription n' est pas gênante pour les caissons que nous avons étudiés jusqu' à maintenant, qui étaient des caissons pour les réacteurs graphite-gaz. On trouve que l' épaisseur des parois n' est pas assez forte pour que cette condition soit gênante.

Si on faisait des caissons pour des pressions beaucoup plus fortes, par exemple pour des réacteurs à eau bouillante ou surtout à eau pressurisée, le problème pourrait effectivement se poser et peut-être ferait-on des expériences à ce moment-là. Je crois surtout que la tendance en France serait de prévoir, comme l' a proposé le Dr. Scotto, un système de drainage derrière la peau. C' est cela qui nous parait à l' heure actuelle la meillure solution.

Je voudrais ajouter aussi qu' il ne faut pas oublier que ce risque de fuite est très peu probable parce que la peau d' étanchéité en acier est plus chaude que le béton. Ceci, le retrait du béton, l' effet de la précontrainte font que cette peau est comprimée, tout au moins en service, au moment où la pression de gaz existe. Par conséquent le risque de fuites est très faible.

CHAIRMAN

Nous n' avons plus de questions écrites, nous sommes arrivés théoriquement à l' heure où nous devons terminer pour ne pas influencer sur le début de la session de cet après-midi qui sera présidée par M. le Prof. Zerna, mais enfin si quelqu' un avait encore une question importante qu' il ne pense pas pouvoir poser dans les trois demi journées qui restent nous sommes encore prêts à lui donner la parole.

Prof. O. C. ZIENKIEWICZ

There are two points I would like to comment on. The first one concerns the matter of gas pressure in the body of concrete. This is a very similar problem to that dealt with frequently by engineers in the field of dams and problems there are referred to as uplift. I believe the question of dealing with concrete reactors under the possible internal pressure of gas per-

colating through the porous medium should be dealt on

A second point which I should like to comment upon is that raised in the very excellent paper by Professor Fumagalli. He finds there experimentally a progressive failure and continuing deformation after repeated cycles of the load. In numerical analysis this matter of cyclic failure or continuing deformation has not received sufficient attention mainly due to difficulties and cost of reproducing several cycles on a computer. We have recently studied one such problem and shown that deformation in a jointed rock mass can continue for many cycles if sliding conditions are reached at any stage of the analysis. I believe it is not economical to extend the numerical procedures further and some attention should be given to shake-down theorems such as for instance, used in steel structures to determine whether deformation will be progressive in a particular engineering case. Some extension of these theorems is needed for frictional materials.

CHAIRMAN

Alors je pense pouvoir clore cette session. J' aimerais remercier les auteurs des communications de cette première session. J' aimerais également remercier ceux qui ont participé à la discussion. Le rôle, précisément, de ces colloques avec un nombre de personnes réduit est de faciliter le dialogue entre les spécialistes et je crois pouvoir dire que cette discussion a été extrêmement fournie. Je ne suis pas un bon juge, puisque je ne suis pas un spécialiste du béton et des caissons de centrales nucléaires, mais enfin je crois que cette discussion a été extrêmement utile et très favorable et j' aimerais donc vous remercie pour y avoir participé.

J' aimerais également remercier les traducteurs, c' est toujours difficile de faire la traduction de choses techniques, d' autant plus qu' aujourd' hui nous avons demandé à chacun de faire vite pour que nous puissions terminer dans les délais. Merci mesdammes et messieurs pour votre participation.

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