Seminar 3

**Structures to Withstand Natural Disasters:
Experiences and Applications**

**Structures résistant aux catastrophes naturelles:
expériences et applications**

**Bauwerke zum Schutz gegen Naturkatastrophen:
Erfahrungen und Ausführungen**

Organizer: Johan Blaauwendraad,
The Netherlands
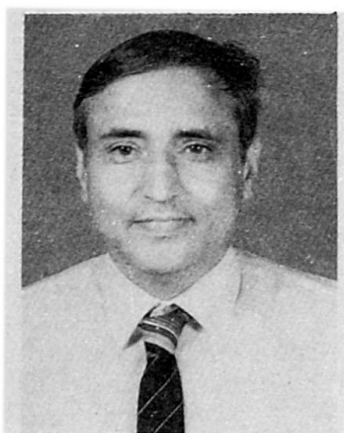
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Foundations for Deep Scour — Ganga Bridge at Varanasi

Protection contre les affouillements du pont de Varanasi

Kolkschutz der Varanasi-Brücke über den Ganges

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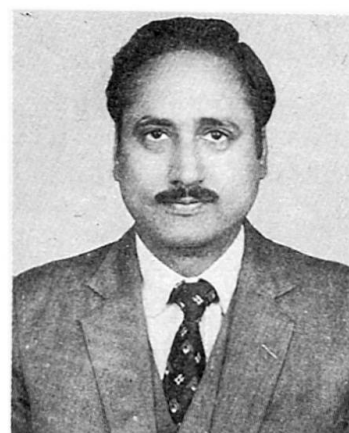
Dr. G.P. Saha, received his degree in Civil Eng. in 1959 from Calcutta Univ. and did Post-graduate studies at Leeds Univ. England. During Post-Graduation he worked on fatigue on concrete and analysis of bridges. Dr. Saha's subjects of specialisation are reinforced and prestressed concrete and types ;of structures are bridges and industrial structures.

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SUMMARY

Foundations for bridges crossing the great rivers of the Indian plains are designed to resist high floods, which cause large depths of general bed scour and local scour around them. The paper describes the salient features of analysis and design of the large diameter deep cylinder sunk by open dredging for the 920 m long monumental bridge project over the Ganga at Varanasi.

RÉSUMÉ

Les fondations des ponts enjambant les grands fleuves des plaines de l'Inde doivent être conçues pour résister aux grandes inondations, qui causent des affouillements profonds dans les lits soit à grande échelle, soit locale-ment autour des fondations de ponts. L'article décrit les particularités essentielles du projet et du calcul du puits cylindrique profondément foncé et de grand diamètre, réalisé par dragage ouvert pour assurer la protection du gigantesque pont de 920 m de portée, projeté pour la traversée du Gange à Varanasi.

ZUSAMMENFASSUNG

Die Grüundung der Brücken über die grossen Ströme der indischen Ebenen werden für extreme Hochwasser bemessen, die tiefe Auswaschungen des Flussbetts und Auskolkungen an den Pfeilern verursachen. Der Aufsatz beschreibt die Hauptpunkte der Berechnung und Bemessung der tiefen Brunnengründung, die mittels offener Baggerung abgesenkt wurde, für die imposante 920 m lange Varanasi-Brücke über den Ganges.

1. INTRODUCTION

High floods in the great rivers washing the alluvial plains of India recur frequently causing immense damage and distress. Flood plains are very wide. Bed and banks are erodible. For building highway crossings over these rivers, large investment is needed for river training into defined channels. Besides, bridge foundations have to be designed to withstand general bed scour and local scour around the foundation which are large.

2. PROJECT DETAILS

The new highway bridge under construction over the sacred waters of Ganga at Varanasi can serve as an outstanding example of Indian experience and practice in designing and building bridge foundations for very deep scour.

The width of the flood plains at the crossing spreads to 3 km. The design flood discharge is 46200 m³/s. The right bank is inerodible and the flood flow will be trained into 920 m channel by an earthen guide bund forming the left bank. The location and layout of the guide bund has been determined by hydraulic model studies (fig.1).

The subsoil strata in river bed are generally silty sand for considerable depth and dense sand at founding level. Very hard clayey silt/silty clay layers have also been encountered at various levels.

The bridge on National Highway 2 is 920.5m long with an approach viaduct 349.5m long (fig.2). It carries a four lane carriageway and 1.5m wide footpaths on either side.

The main bridge consists of 6 middle spans @ 131m each and two end spans of 65.75m each. The superstructure consists of prestressed concrete two cell box girder of depth varying from 2.4m to 8.7m built in cast-in-place cantilever construction and is provided with hinge at mid span. Reinforced concrete hollow piers and single cylindrical caissons have been adopted for foundations.

3. ESTIMATION OF SCOUR DEPTH

General scour is estimated as per formula given in IRC:78 [1]. Design scour depth around pier is taken as twice the general scour depth to account for the local scour.

As per the model studies conducted with 920m long bridge (having 9 spans-2 end spans of 57.5m each and 7 middle spans of 115m each with 7m wide piers) and left upstream eliptical guide bund of 460m length and 30m long straight guide bund on down stream, concentration of flow near the right bank was observed. Discharge through the right 6 bays was 85% whereas through the left 3 bays was only 15% of the total discharge. Velocities of flow varied from 3 m/s to 4.1 m/s.

For the purpose of design the river is divided into two scour zones - deep and shallow. Depth of general and design scour for the two zones are given in Table.1.

FIG. 1. SITE PLAN – MODEL STUDY

LEFT DATUM IN MODEL

APPROACH EMBANKMENT

GHATS

VARANASI TOWN

GANGA

D C B A

GUIDE BUND

RAMNAGAR FORT

RIVER

BRIDGE AXIS

RIGHT DATUM IN MODEL

1600 m U/S

EXISTING MALAVIYA RAIL CUM ROAD BRIDGE

VELOCITY ALONG APPROACH EMBANKMENT

LOCATION	Q =3240 m³/s	Q =46180 m³/s
A	0.81	NIL
B	0.63	0.63
C	—	0.41
D	—	—

VELOCITY ALONG LEFT GUIDE BUND

DISTENCE FROM ABUTMENT	Q =32400 m³/s	Q =46180 m³/s
1.8 m	2.03	2.28
2.70 m	2.85	3.29
3.60 m	3.08	4.67
4.20 m	3.35	5.05
4.60 m	2.69	4.36
NOSE	1.50	3.03

φ = DISCHARGE

PROPOSAL No. 3

1. TOTAL WATER WAY (L) — 920 m
2. U/S LEFT GUIDE BUND:
   460 m (0.5L) ELLIPTICAL CONFORMING TO THE ELLIPSE $\frac{x^2}{460^2} + \frac{y^2}{184^2} = 1$ FOLLOWED BY THE ARC OF RADIUS 184 m AND ANGLE 30°
3. D/S LEFT GUIDE BUND: 30 m STRAIGHT

FIG. 2 GENERAL ARRANGEMENT OF BRIDGE

26.50 m

920.50 m (MAIN BRIDGE)

349.50 m (VIADUCT) 14 EQ SPANS

65.75 131.50 131.50 131.50 131.50 131.50 131.50 65.75

ROCKER CUM SLIDING BRG.

P.S.C BOX GIRDER

CENTRAL HINGE

CAST IN SITU R C BOX GIRDER

HFL 75.60

LWL 60.50

CAST IN SITU R C BORED PILES

38.00 SCOUR

22.00

16.10 SCOUR

5.00

6.50

5.0

7.0

CIRCULAR WELL (TYP.)

A P1 P2 P3 P4 P5 P6 P7 P8

	SCOUR ZONE	
	deep	shallow
width	720m	200m
depth of general scour	29.75m	18.80m
design depth of scour	59.50m	37.60m

TABLE 1   DETAILS OF SCOUR ZONES

In deep scour zone the design depth of scour (59.5m) is unusually high even for Indian conditions and require special considerations in design and construction. Salient features of the design adopted for foundations in deep scour zone are presented below.

4. FOUNDATIONS

Single cylindrical caissons (wells) sunk by open dredging has been found to be optimal solution for almost all foundations of this type and the same have been adopted for Ganga bridge also.

4.1 Dimensions

From the consideration of design scour depth and minimum grip length specified in IRC:78 [1] minimum depth of foundation works out to 65.5m below the lowest water level. Diameter of well has been adopted as 13m and depth of well varies from 65.5m to 67m from stability considerations. Thickness of steining has been adopted as 2.5m from considerations of sinking effort required. Guidance has been taken from other works of similar nature executed in the past.
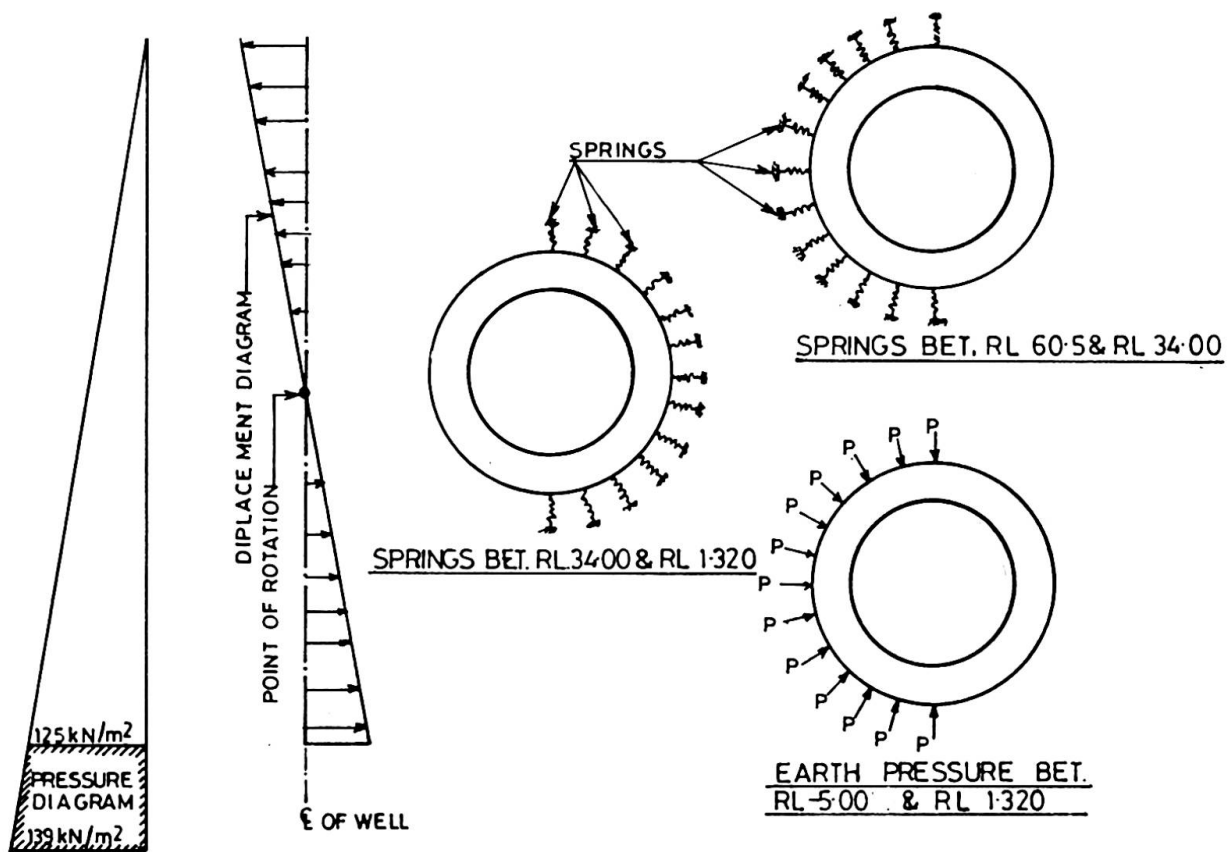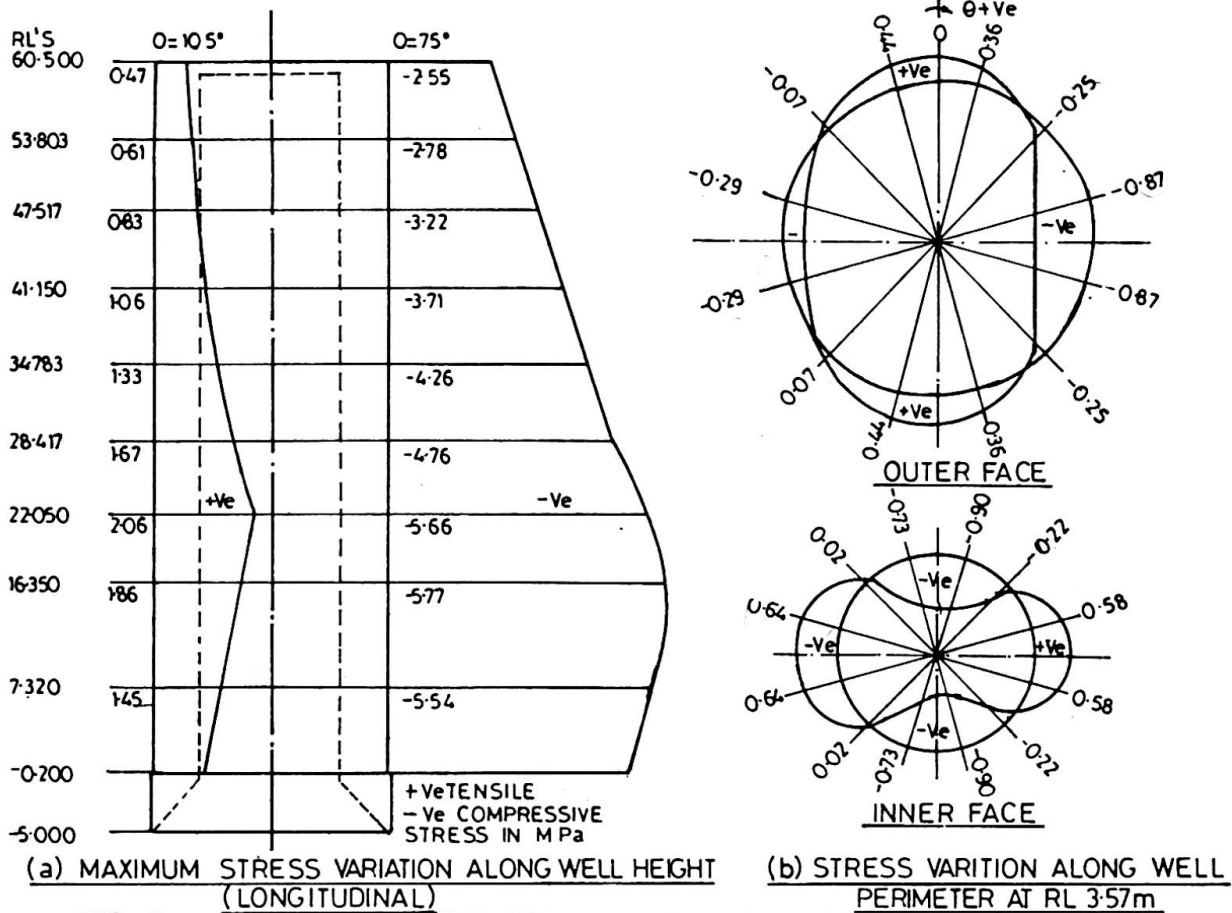
4.2 Analysis

4.2.1 Service Stage

Analysis for steining stresses has been done manually for all possible critical load cases and well steining has been designed for the worst load combination. Steining stresses have also been worked out using 3DFE analysis for the critical load cases.

For 3DFE analysis quadrilatral 4 noded plate bending elements have been adopted for well steining, bottom plug and well cap. Typical plot showing variation of longitudinal stress along the well height and variation of circumferencial stress along the well periphery at critical level are shown in fig.3 (a) and (b) respectively.

Longitudinal stresses obtained by 3DFE analysis compared fairly well with the results of linear analysis. Hoop stresses are found to be critical and require higher quantum of steel reinforcement than the minimum prescribed IRC:78.

4.2.2 Sinking Stage

During sinking operation apart from the normal phenomenon of tilt and shift associated with kentledge, the well foundations are also encountered with the sand blow which has caused failures to a number of bridge foundations and is the most critical condition.

(a) MAXIMUM STRESS VARIATION ALONG WELL HEIGHT (LONGITUDINAL)

(b) STRESS VARITION ALONG WELL PERIMETER AT RL 3·57m

FIG.-3 3 DF E ANALYSIS FOR WELL P-4 SERVICE STAGE

FIG.-4 SAND BLOW CONDITION·-APPLIED FORCES AND DISPLACEMENT MODEL

Sand blow is caused due to the difference in head of water between inside and outside the well. Though equilibrium condition is reached, in the process, the well steining is subjected to non-uniform earth pressure which give rise to transverse stresses in the steining. Detailed analysis by 3DFE method has been carried out for sand blow condition. Typical details of applied forces, displacement and springs of resistance around the periphery of well adopted in the 3DFE model for sand blow analysis are given in fig.4.

Adequacy of section has also been checked as per the model for earth pressure suggested by Santra and Ganguli [2].

5. INSTRUMENTATION

To assess the performance of the structure both during construction and in-service, extensive instrumentation scheme has been planned.

In the foundation earth pressure cells and reinforcement guages are being provided to measure the earth pressure and stress in reinforcement both longitudinal and hoop actually developed at various stages. Provision has also been made to measure inclination of wells with the help of inclinometer. Pressure cells provided around the periphery at various levels along the length of well may give sufficient data to arrive at the mathematical model of earth pressure for analysis. Similarly, reinforcement gauges may help in checking the mathematical model assumed in analysis and design. Sand filling normally done in the wells has been dispensed with to facilitate inspection of steining from inside the well with the help of under water video camera. A hole has been provided in the well cap for inspection purpose.

Similarly, extensive instrumentation scheme is being planned for piers and superstructure.

6. STATUS

Owner   : Union Government of India
Agency   : Uttar Pradesh State Public Works Department
Contractor   : M/s.Hindustan Construction Company Ltd., Bombay,
                  India.
Date of Commencement : December, 1988

Work on foundations is in progress. By July 1991 out of the 9 well foundation 7 have reached the founding level. It had been possible to sink the wells without much difficulty.

7. CONCLUSIONS AND RECOMMENDATIONS

It is possible to sink deep wells in alluvial soil. Analysis should be done for both longitudinal and hoop stresses especially when diameter of well is more than 10m.

REFERENCES

1. IRC:78-1983 Standard Specification and Code of Practice for Road Bridges, Section VII Foundations and Substructure.

2. Santra A. and Ganguli R.C., Design of Well Steining against Sand Blowing Condition During Sinking. Cement and Concrete, Vol 8, 1968.
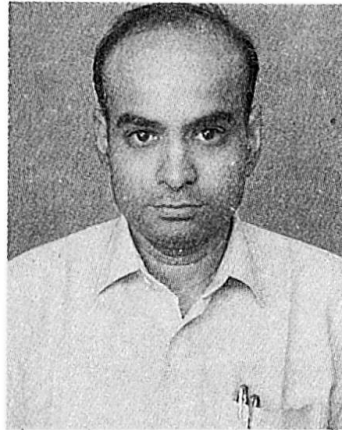
Engineered Cyclone Shelter for Disaster Prevention

Prévention des dégâts de cyclones par abris adéquats

Ingenieurmässig konstruierte Schutzbauten gegen Wirbelstürme

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Santhakumar received his B.E. Civil and M.Sc. (Struc.) from college of Eng. Guindy. He was a commonwealth Scholar at the Univ. of Canterbury, Christchurch, New Zealand where he obtained his Ph.D. For 25 years he has been teaching and guiding research at Anna University.

SUMMARY
The paper describes a technical study on the behaviour of cyclone shelter models in the wind tunnel. Two different types of shelter have been considered. The superior performance of these shelters has been highlighted.

RÉSUMÉ
L'article présente une étude technique comparative sur le comportement de deux modèles d'abris prévus contre les cyclones et soumis à des essais en soufflerie. Il met en relief les résistances performantes de telles constructions.

ZUSAMMENFASSUNG
Zwei Typen von Schutzbauten gegen Wirbelstürme wurden vergleichenden Tests in einem Windtunnel unterzogen. Die vorliegende technische Studie bespricht die überragende Widerstandsfähigkeit dieser Konstruktionen.

1. GENERAL

Safeguarding life and property from the destructive effects of cyclones is a major worldwide problem. India being a peninsula with 5000 Km coast, is often threatend by this natural hazard. Every year the cyclones damage crops, distroy human lives, cattle and structures. Safe design of structures against wind require a reasonably accurate estimate of loads as well as a knowledge of structural behaviour when subjected to these loads. As a part of disaster prevention plan, a program of building cyclone shelters for housing the people who are rendered homeless has been taken up at sites shown in Fig.1.
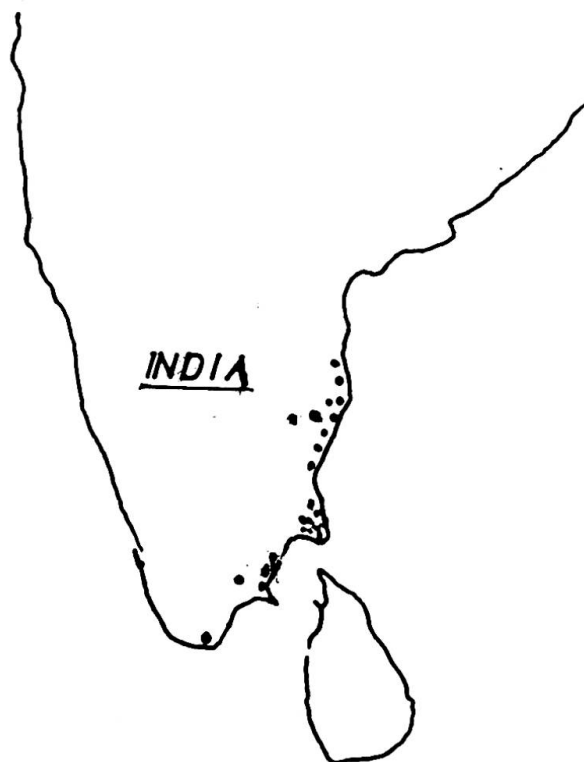

Fig.1 Distribution of Cyclone shelters

2. TYPE OF STRUCTURES

The cyclone shelter(Type I) as shown in Fig.2, is of circular plan form so that it offers least resistance to wind force from any dorection. The diameter of the shelter is 15 metres. The height is 7.5 metres. It has two floors. The shelters are erected on a raised ground to prevent flood waters reaching the floor.

In another type design, the bath and washing were provided outside the main floor area as shown in Fig.3.In this type, the symmetry of the structure is broken by the inclusion of a projection over a 60° plan form. The second type (typeII) of shelter has also been studied.
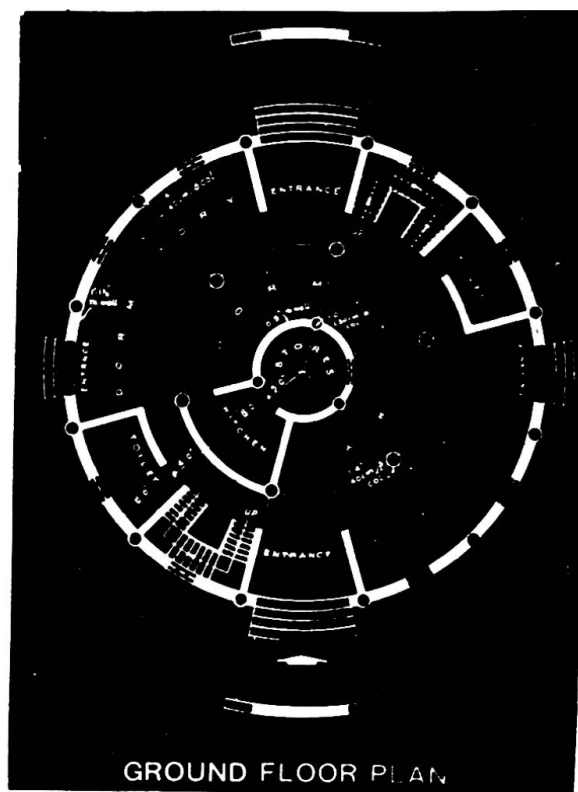
GROUND FLOOR PLAN

Fig.2 Type I shelter

Toilet

Fig.3 Type II shelter

3. MODEL TESTING

Model test in a wind tunnel has been conducted to estimate the variation of pressures along the walls and roofs of the cyclone shelters and thus find the aerodynamic forces and moments. The subsonic wind tunnel situated at Anna University was used for testing. The shape of the cyclone shelter has been faithfully reproduced. The windows, ventilators and openings have been scaled down to a model scale of 1/50. Since the Reynolds Number of the structure and the model are well over 10^6, it is reasonable to assume that the force coefficients of the prototype and model are directly related. Structural similarity between the model and prototype is not

attempted as only a static model was tested. Monometers and
pitot tubes have been used to measure the wind pressure and
wind speed respectively. A typical model is shown in Fig.4.
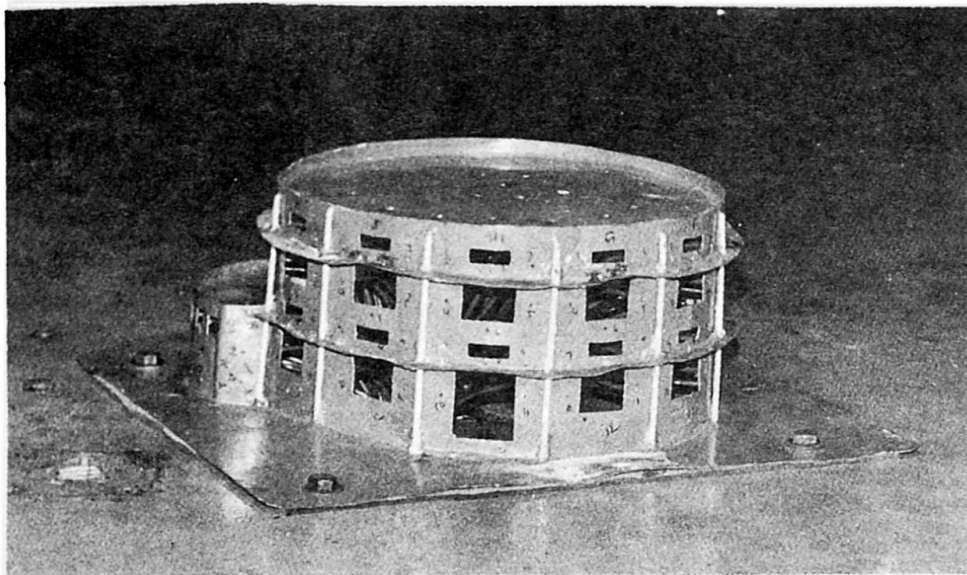
Fig.4 A typical model

Type II was tested for nine different postions as shown in
Fig.5.
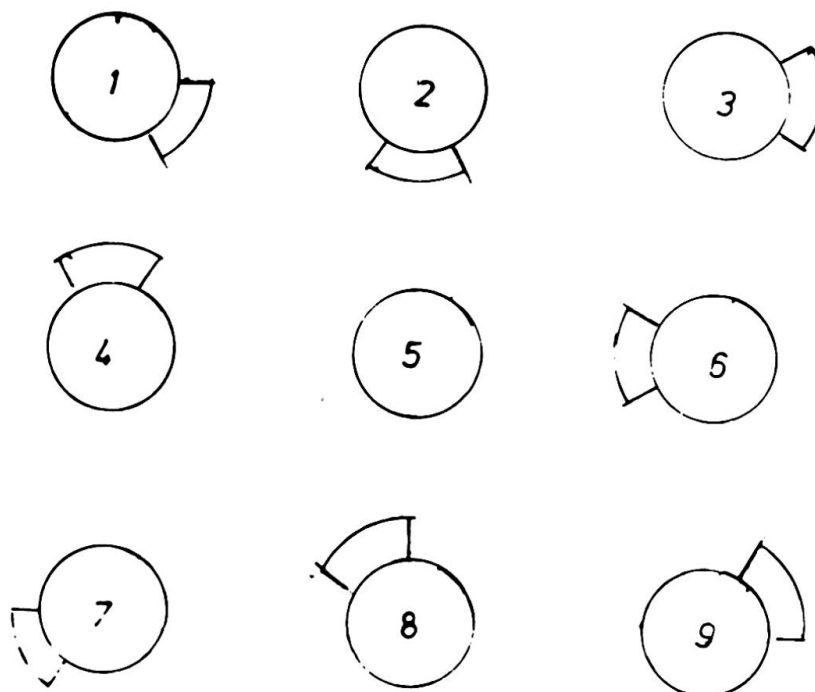
Fig.5 Positions during testing

4. WIND LOADS ON THE STRUCTURE

From the model tests pressure coefficient C_L at different wind speeds for various positions of models and different permiabilities were obtained.The flow lines obtained during testing of both type I and Type II models are shown in Fig.6 and Fig. 7 respectively.

Fig.6 Flow lines for type I
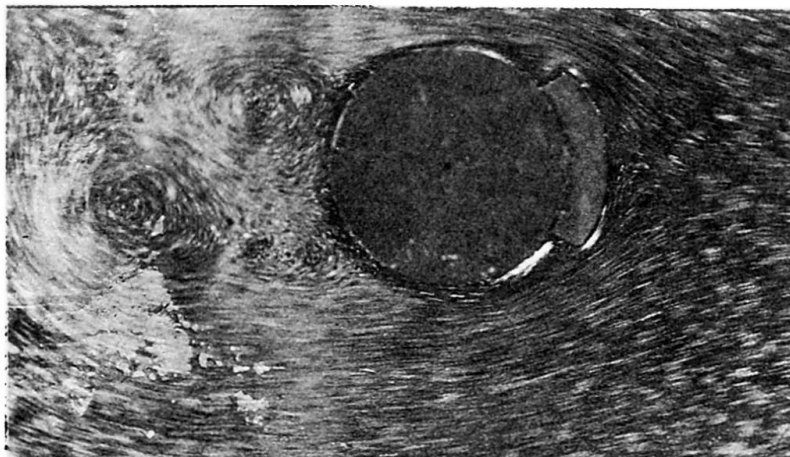
Fig.7 Flow lines for type II

5.CONCLUSION

The calculations show that present structure put up along the coast of India are safe from usual cyclones that form in Bay of Bangal for wind speeds upto 150 Kmph.

Type II has to resist three times the overturning moment as Type I. though both are safe.

REFERENCES

1. SANTHAKUMAR A.R.,PADMANABHAN K and SUBRAMANIAN P.S., Final project report on Evaluation of cyclone shelters, August, 1982.

2. Antidisater planning for cyclones and flood affected areas of Tamilnadu, Directorate of Town and Country Planning Document, 1978..

Vibration Control of Structures by Tuned Liquid Column Dampers

Amortissement de vibrations des constructions par colonnes de liquide

Tilgung von Bauwerkschwingungen mittels Flüssigkeitssäulen

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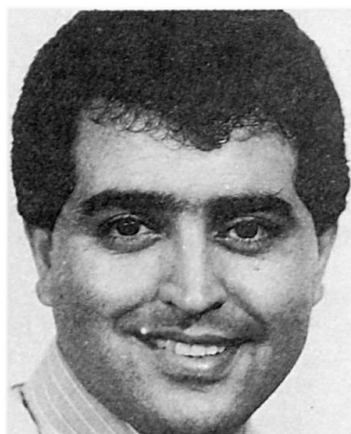Bijan Samali, born 1955, received his Doctor of Science degree in structural dynamics from George Washington University. His research interests and activities are mainly in the area of active and passive control of structural vibrations caused by wind and earthquakes. He has been involved with a number of high-rise building projects in a consulting capacity.

K.C.S Kwok, born 1950, received his Ph.D from Monash University in 1978. His research interests lie in structural dynamics and wind engineering. He has published extensively in Engineering Journals. He has been responsible for wind tunnel investigations of a great number of building structures in Australia and overseas.

D. Tapner, born 1938, received his structural engineering degree from the Institute of Technology, Sydney, Australia. He is currently the engineer in charge of the Structures Laboratory of the School of Civil Engineering at UTS. He is involved with a number of research and testing programmes performed in the Structures Laboratory.

SUMMARY
The effectiveness of Tuned Liquid Column Dampers (TLCD) in suppressing wind and earthquake induced vibrations of structures is investigated both analytically and experimentally. Random vibration analysis of a 40-storey building subjected to earthquake loading confirms the effectiveness of the system in reducing building response. The experimental work investigated the effectiveness of the proposed damper system for a single degree of freedom structure in terms of additional damping displayed by the structure when equipped with the TLCD system. The study also included a parametric study of the TLCD system resulting in an optional system.

RÉSUMÉ
En se basant sur l'utilisation d'amortisseurs à colonne de liquide à effet direct, l'article examine, tant du point de vue analytique qu'expérimental, la suppression efficace des vibrations induites dans les structures par les effets du vent ou des tremblements de terre. Le calcul des vibrations aléatoires produites dans un bâtiment de 40 étages par des secousses sismiques confirme l'efficience de ce système d'amortissement, par suite de la réduction de la réponse de l'ouvrage. L'étude expérimentale a porté sur une structure à un seul degré de liberté, en fonction d'un amortissement additionnel enregistré par la structure équipée du système d'amortisseurs à colonne de liquide à effet direct.

ZUSAMMENFASSUNG
Die Wirksamkeit von Tilgern auf der Basis abgestimmter Flüssigkeitssäulen wird analytisch und experimentall für wind- und erdbebeninduzierte Bauwerkschwingungen untersucht. Die stochastische Schwingungsanalyse für ein vierzig-stöckiges Gebäude unter Erdbebeneinwirkung bestätigt die Wirksamkeit auf die Bauwerksantwort. Die experimentellen Studien umfassten die erreichbare Zusatzdämpfung in Systemen mit einem Freiheitsgrad und die optimale Abstimmung eines solchen Tilgers.

1. INTRODUCTION

Control of excessive vibrations of tall buildings and other structures subjected to severe environmental loads such as wind and earthquake is of particular importance to structural engineers and has received considerable attention in recent years.

Tuned Mass Dampers (TMD) and visco-elastic dampers have been found effective in reducing the response of structures subjected to dynamic loads [1-4]. Recently liquid dampers such as sloshing dampers and Tuned Liquid Column Dampers (TLCD) are found effective in suppressing structural motions with definite advantages over other damping devices including low cost, easy handling and virtually no maintenance requirements. In this paper the effectiveness of the TLCD system in suppressing building vibrations is investigated both analytically and experimentally.

In the TLCD system the vibration energy of the structure is dissipated through the motion of liquid mass in a tube-like container, the restoring force due to the gravity acting upon the liquid and the damping effect as a result of loss of hydraulic pressure due to orifices installed inside the container.

The TLCD system is a versatile and simple system. The shape of the container tube is arbitrary and can assume any shape to suit its accommodation in buildings and towers. The vibration frequency of the liquid is only a function of the length of the liquid inside the tube. This would allow the system to be tuned (or retuned) to the frequency of the structure by simply altering the length of the liquid. Changing the damper frequency for tuned mass damper systems is rather difficult as it involves the change of damper mass or its spring stiffness, or the length of the supporting cables in the case of a pendulum type TMD.

TLCD systems are most suited for tall buildings and towers for which a water tank is usually installed for fire fighting or water supply. The proposed idea is to design the afore-said water tank as a TLCD system and hence absorb some of the vibration energy induced by wind and earthquake excitations. For this reason inclusion of a TLCD system does not impart a cost or extra weight penalty as has been the case for some TMD systems. A typical liquid column tube is shown in Fig.1.

2. ANALYTICAL FINDINGS

The effectiveness of the system has been investigated analytically following the random vibration analyses of three typical tall buildings subjected to earthquake excitations with an intensity equivalent to that of the Housner's average response spectra. The analysis uses the governing equation of motion for TLCD systems given by Sakai, et al [5] as

$$\rho A L \ddot{x} + 0.5\,\rho A \xi\, |\dot{x}|\, \dot{x} + 2\rho A g x = -\rho A B \ddot{y} \qquad (1)$$

in which y is the displacement of the tube, x is the elevation change of the liquid and $\rho, L, B,$ and A are, respectively, the density, length of the liquid, the width, and cross-sectional area of the tube. ξ is defined as the coefficient of headloss (constant) governed by the opening ratio of the orifice(s) and g is the acceleration due to gravity. The natural frequency ω_1 and the natural period T_1 of the liquid column tube of Fig.1 are given in the following equations:

$$\omega_1 = \sqrt{2g/L} \qquad ; \qquad T_1 = 2\pi\sqrt{L/2g} \qquad (2)$$

As seen ..from Eq. 1, the damping term in the fundamental equation of tuned liquid damper is non-linear. This is treated by an equivalent linearization technique as shown by Xu, et al [6]. The equivalent linear equation corresponding to Eq.1 may be written as

$$\rho A L \ddot{x} + 2\rho A C_p \dot{x} + 2\rho A g x = -\rho A B \ddot{y}_n$$
(3)

in which C_p is the equivalent damping coefficient which is related to the coefficient of headloss as well as the velocity of the liquid column.

In this paper the effectiveness of the TLCD system is demonstrated on a 40-storey building equipped with a TLCD system with first natural vibration frequency of 0.17 Hz. The mass of the water is 2 % of that of the building. The required water column length for optimum tuning was found to be 18.7 meters. The required cross-sectional area of the tube is then 55.1 m^2. Maximum damping would be achieved when the ratio of the horizontal portion of the liquid to its total length is maximised in static position. Hence, 90 percent of the total liquid length was positioned horizontally. It is also found that maximum response reductions are achieved when the tuning ratio, ie, the ratio of the liquid column frequency to the frequency of the structure, is about 0.95. This ratio however is a function of mass ratio as well as the orifice opening ratio. Fig.2 illustrates the effectiveness of the TLCD system in reducing the top floor displacement of the forty storey building when subjected to earthquake loading with an intensity equivalent to that of the Housner's average response spectra. Fig. 2 also shows the sensitivity of the system to tuning ratio and the opening ratio of the orifice installed inside the tube. It is observed that a TLCD system with a tuning ratio of about 95 percent, and an orifice opening ratio of 46 percent is capable of reducing the building response by about 25 percent. The ordinate in Fig.2 represents the response ratio which is the response of the building with the TLCD system to the same response quantity in the absence of the damper system.
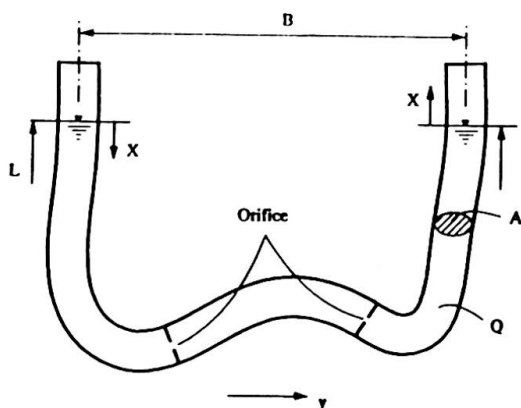
Fig. 1 A typical liquid column tube
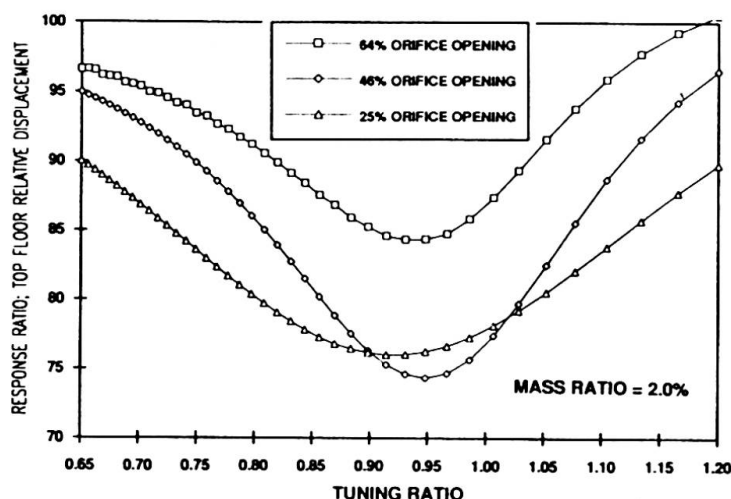
Fig. 2 Response of the 40-storey building equipped with TLCD system

3. EXPERIMENTAL WORK

In order to verify the effectiveness of the proposed TLCD system and to identify the parameters which affect its performance, a series of free vibration tests were performed in the structures laboratory of the School of Civil Engineering at the University of Technology, Sydney. The structure was modelled as a single degree of freedom system possessing a natural period of vibration of 3 seconds. The effect of orifice opening ratio and mass ratio was

investigated. The effectiveness of the system was measured in terms of system
overall damping ratio ie system damping as percentage of the critical value.

3.1 Test Set-up

In order to study the effects of the above-said parameters in relation to the
performance of the TLCD system, a pendulum with variable mass was chosen as the
single degree of freedom structure. A fixed volume of water passing through a
tube, fitted with a variable orifice, was located at the centre of mass of the
pendulum. The water was to pass through a 50 mm diameter tube with 90% of the
total volume of water contained within a horizontal length of this tube and the
remaining 10% in the vertical arms. Tuning ratios of 110%, 100% and 90% were
tested. To simulate different water/pendulum mass ratios, the mass of the
pendulum was varied for each tuning to represent different mass ratios of
1%,2%,3% and 4%, respectively. The initial amplitude of the pendulum and the
decay, together with the velocity and quantity of water flow in the pipe, was
measured.

The length of the pendulum was calculated using Eq. 4.

$$T = 2\pi \sqrt{(L/g)} \qquad\qquad (4)$$

For the required period of oscillation of 3 seconds, the pendulum length, L,
was therefore calculated as 2.24 metres. A 2 x 2 point suspension system has
been used.

The height of the water in the vertical arms of the damper was determined by
measuring the pressure within a closed vertical tube connected to a pressure
tapping mounted close to the end of the horizontal section of the pipe. The
velocity of the water flow was determined based on the slope of the
pressure/time graph.

As the mass of the pendulum had to vary over a considerable range, and
simultaneously support the horizontal length of the water tube, which varied in
length for each tuning range, it was decided that two parallel lengths of
square hollow steel tubing, (each 55 x 55 x 3.9 mm) connected by welded steel
brackets across the bottom flanges, would be structurally adequate to both
support the applied extra masses and maintain the masses and water tube at a
co-planar centre of gravity.

The mass of the two 5 metre lengths of steel tube, together with the PVC 50 mm
bore tubing, was 65 kg. Steel saddles, designed to rest on the top of the
tubes, and fitted with cylindrical solid rod outrigger arms, provided
additional mass to the frame to bring the mass of the total unloaded pendulum
to 100 kg. The eight steel saddles were each capable of supporting six 22.5 kg
lifting weights, thus providing a theoretical upper limit of 1180 kg.
The centre section of the damper pipe was fitted with a double flange piece
machined to accept flat pre-drilled plates with apertures of 100%, 80%, 60%,
40% and 20% of the pipe cross-sectional area. Fig. 3 illustrates the test set-
up.

4. TEST RESULTS AND CONCLUSIONS

Free vibration test results for mass ratios of 1,2,3, and 4%; tuning ratios of
100, 90, and 110% and orifice opening ratios of 20,40,60,80, and 100% were
analysed in order to obtain the optimum damping level of the system.   The
recorded amplitudes of the nth and the n+mth vibration cycles were fed into a
computer program from which system damping was obtained by calculating log-
decrements. The results for 100 percent tuning ratio is tabulated in Table 1
and also illustrated in Fig. 4. The results indicate that the optimum tuning
ratio is around 100%. The system damping ratio for 90 and 110% tuning ratios

are generally less when compared with the results obtained for 100% tuning
ratio.   It is also observed that for 90 and 110% tuning the system damping
increases monotonically with reducing orifice opening ratio.

Table 1 and Fig.4 clearly indicate that for 100% tuning ratio the largest
system damping is achieved when the orifice opening ratio is 40%.   This is true
for all mass ratios.   It was anticipated that the optimum opening ratio would
be somewhere between 0 and 100 %, as the former represents full closure and the
latter full openness, both of which were known to be non-optimal.

The results further indicate that for orifice opening ratios in the range of
100 to 60%, a larger mass ratio would not necessarily mean more damping.
Furthermore, in this range the system damping is not sensitive to mass ratio
and choosing a large mass ratio would not increase the system damping greatly.
However, for the orifice opening ratios of 40 and 20%, a larger
damper/structure mass ratio is associated with a larger system damping.

Test results clearly demonstrated the effectiveness of TLCD systems in
suppressing vibration energy.   The system is most efficient with tuning and
orifice opening ratios of 100% and 40%, respectively. For these optimum values,
the effectiveness of the system increases with increasing mass ratio.   It must
be realized however that there is a practical limit to mass ratio which is
governed by architectural and other design constraints.   In these experiments
an optimal TLCD system with a mass ratio of only 2% increased the system
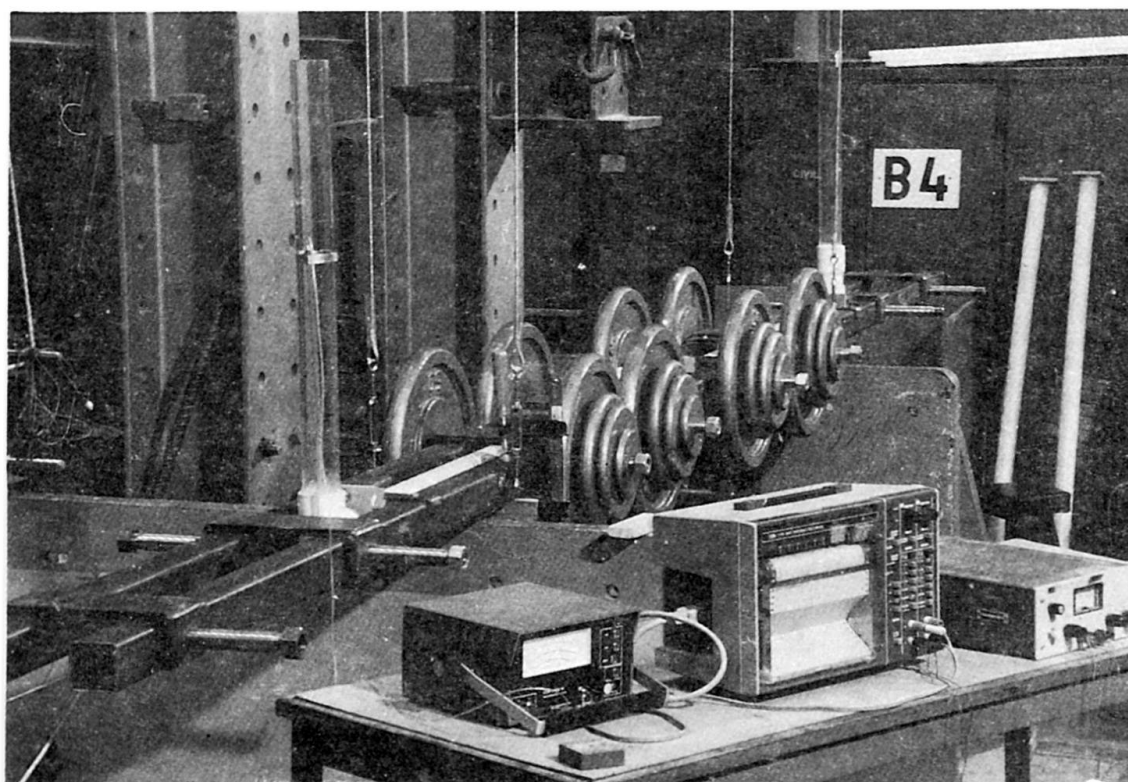damping by about 4% ie from 0.8% to 4.9% of critical damping.

Fig.3 Test set-up

		No TLCD	Orifice Opening Ratio				
			100%	80%	60%	40%	20%
Mass Ratio	1%	0.6	2.4	2.6	3.0	3.3	1.7
	2%	0.8	2.1	2.4	2.8	4.9	3.4
	3%	1.2	3.3	3.5	3.8	6.6	5.4
	4%	1.3	2.7	2.8	3.9	8.5	6.7

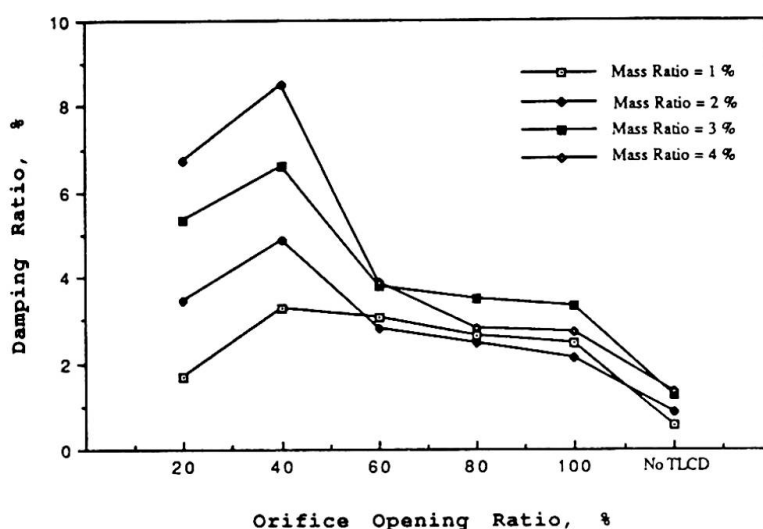Table 1 Damping ratios of the system for 100% tuning

Fig. 4 Variation of damping ratio with orifice opening
and mass ratio for 100% tuning

REFERENCES

1. VICKERY B.J. and DAVENPORT A.G., An investigation of the behaviour in wind of the proposed Centrepoint Tower, in Sydney, Australia. Engineering Science Report No. BLWT-1-70, Univ. of Western Ontario, 1970.

2 PETERSON N.R, Design of large scale tuned mass dampers. ASCE Convention and Exposition, Boston, Mass., preprint 3578, 1979.

3. WEISNER K.B., Tuned mass dampers to reduce building wind motion. ASCE Convention and Exposition, Boston, Mass., preprint 3510, 1979.

4. TANAKA H. and MAK C.Y., Effect of tuned mass dampers on wind-induced response of tall buildings. J. of Wind Eng. and Industrial Aerodynamics, 14, pp. 357-368, 1983.

5. SAKAI F., TAKAEDA S. and TAMAKI T., Tuned liquid column dampers - new type device for suppression of building vibrations. Proc. of International Conference on High-rise Buildings, Vol. 2, Nanjing, China, 1989.

6. XU Y.L., SAMALI B. and KWOK K.C.S., The effect of tuned mass dampers and liquid dampers on wind-induced response of tall/slender structures. Research report No. R621 School of Civil and Mining Engineering, University of Sydney, Sydney , NSW, Australia, 1990.
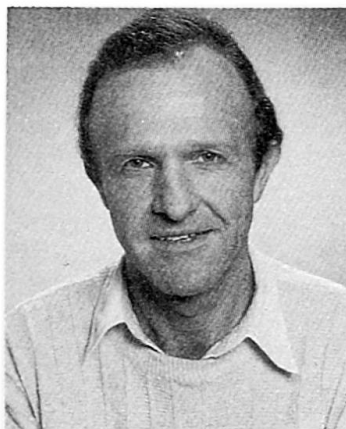
Anchors — an Efficient Means for Disaster Resisting Structures

Tirants pour protection des bâtiments contre les catastrophes naturelles

Anker für Schutzbauwerkee gegen Naturkatasrophen

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SUMMARY

Prestressed ground anchors have gained wide acceptance. Beyond the usual applications the anchoring technology is an ideal measure in connection with impacts caused by natural disasters. From the manifold applications this article describes how anchors were used to enhance the stability of buildings against typhoon wind loadings, how they increase the stability of dams and how easy and effective they can be used for gallery protection against rock fall.

RÉSUMÉ

Les tirants d'ancrage précontraints ont fait leurs preuves partout et dans bien des domaines. Outre les applications habituelles, la technique d'ancrage représente une mesure idéale de protection complémentaire contre les effets des catastrophes naturelles. Parmi les nombreuses applications possibles, cet article décrit comment l'utilisation d'ancrages peut assurer la stabilité des bâtiments contre l'action de vents extrêmes, comment ils peuvent augmenter la stabilité des barrages et avec quelle simplicité et efficacité ils peuvent être utilisés dans la protection des galeries contre les chutes de pierres.

ZUSAMMENFASSUNG

Vorgespannte Bodenanker haben sich überall und auf allen Gebieten bewährt. Neben den üblichen Anwendungen ist die Ankertechnik eine ideale Massnahme im Zusammenhang mit Schutzbauwerken gegen Naturkatasrophen. Von den vielfältigen Anwendungen beschreibt der Artikel, wie Anker die Stabilität von Talspesren erhöhen, und wie einfach und wirksam sie im Zusammenhang mit Steinschlaggalerien eingesetzt wurden.

1. INTRODUCTION

Prestressed ground anchors have gained wide acceptance and have established a permanent place in civil engineering practice. They are economical, easy and quick to install and, if appropriately designed using the latest state-of-the-art technology they are a reliable means for transferring forces to rock or soil, whether temporarily or permanently.

The most common uses for anchors today are for anchoring excavations and retaining walls, securing slopes and fractured rock zones and for stabilizing caverns. They are equally suitable for use in resisting uplift, anchoring concentrated forces and testing piles.

2. STATE-OF-THE-ART ANCHOR TECHNOLOGY

Within the scope of this paper it's not possible to fully treat today's state-of-the-art anchor technology or anchor recommendations and the reader is referred to relevant FIP publications ([1], [2]). It's an acknowledged fact that design and planning of anchors call for experience and know how and that the installation phase requires the skilled and qualified labour of the specialized anchor contractor. Modern anchors can be detensioned and restressed in a controlled way at any time during the life of a structure, a feature which should be decided and specified in the planning or design stage in advance of the anchor installation. The same applies where anchors are to be monitored after installation.

The necessity of adequate corrosion protection cannot be over emphasized. As a general rule permanent anchors should always be protected. The protective system should totally exclude the atmosphere from the tension member by completely encapsulating it within an impervious sheath. High density polyethylene is very often used as it meets all corrosion protection requirements and - in most countries - is readily available and cheap. The quality and integrity, however, greatly depend on the workmanship of key interfaces e.g. anchor head/free length and free length/tendon bond length.

3. DISASTER RESISTING STRUCTURES

Beyond the traditional applications mentioned above the anchor technology has for quite some time been used in connection with structures which either have to be secured against the effects of natural disasters, or which protect the environment from the effects of natural disaster, or which prevent the effects of natural disaster to take place at all. In all these cases anchors play a key role and it's therefore very important that the proper function of these anchors is monitored and that the anchor force can be adjusted.

4. EXAMPLES FROM PRACTICE

To illustrate the effectiveness of anchors in connection with disaster resisting structures some major and outstanding structures are described in the following. From the manifold applications two examples show how anchors assure the stability of buildings against wind, three on how they are used to strengthen dams against exceptionally high water levels, floods and earthquake movements and two on protective measures against rock fall.

4.1 Anchors to enhance the stability of buildings

Two of the most spectacular buildings in Hong Kong, the Hong Kong and Shanghai Bank and the Bank of China owe their stability against typhoon wind loading (design wind velocity of 250 km/h) and ground buoyancy effects to permanent rock anchors.

Figure 1: Hong Kong and Shanghai Bank

The spectacular appearance of the Hong Kong and Shanghai Bank (Fig.1) was achieved by hanging the structure from 8 towers. Each tower comprises 4 braced steel tubular masts. In addition to the primary caissons forming the immediate foundation for these towers, the structure loads are also distributed to a series of secondary caissons to complete the substructure of the building.
To provide stability against wind and ground buoyancy effects from the high water table a total of 3?? permanent, restressable and monitorable anchors, ranging in working load from 650 to 2100 kN had to be installed. The maximum anchor length is 32 m. The design life of the anchors is 100 years.

The Bank of China which rises to a height of 315 m with 70 floors is presently the tallest building in Hong Kong (Fig. 2). The structure is reinforced concrete and steel framed with a composite metal tray and reinforced concrete floor system. The structural concept consists of a space truss which acts to carry both the vertical loads as well as the wind forces.

The substructure is comprised of a 1.0 m thick diaphragm wall, four massive 9 m diameter caissons located under the four main columns at the corners of the superstructure and a number of smaller caissons which support the super-structure. Due to its relatively light construction and height, permanent prestressed rock anchors were introduced into the foundation to provide both overturning resistance and to improve lateral shear capacity. A total of 77 rock anchors were located through the diaphragm wall and another 50 around the caissons. The caisson anchors had the additional task of providing restraint against hydrostatic uplift. (Fig. 3)
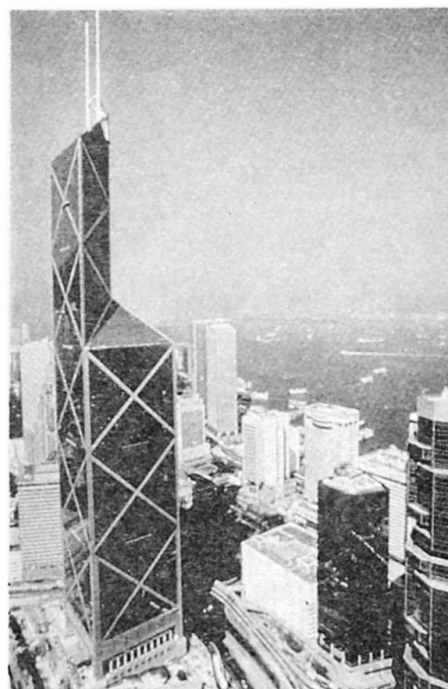
Figure 2: The Bank of China

Three different sizes of VSL permanent anchors were used in the works:

Type and Unit	Number of 0.5" strands	Number of Anchors	Ultimate capacity (kN)	Working load range (kN)	Longest net tendon length (m)	Bond Length (m)
EG 5-21	21	50	3,906	from 461 to 1,953	53.30	7.0
EG 5-35	35·	63	9,275	from 2,164 to 4,638	70.25	10.5
EG 5-42	42	14	11,130	5,513 and 5,565	62.70	10.7

With a design life of 100 years, it was essential that the material and design concept for the anchor was fully verified by stringent testing prior to commencement of the actual works.

Figure 3: Schematic Section through Substructre and Foundation

After being homed the anchors were grouted both internally and externally. Once the grout had attained 30 N/mm², the anchors were stressed with a multistrand jack. In its final state a protective ca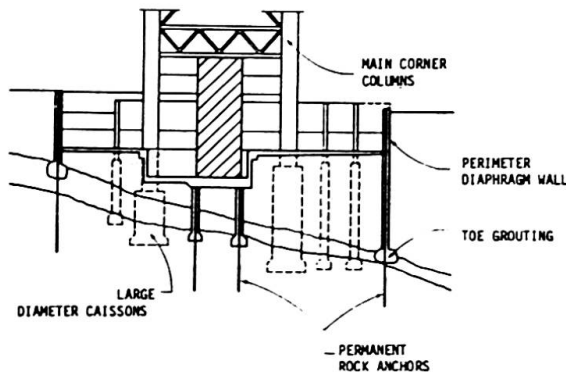p filled with grease was fitten to the anchor head. Over its design life the load in the anchor can be checked periodically with a load cell and if necessary adjustments to the anchor load can be made.

Prestressed anchors provide an economic and time saving solution for anchoring such structures. The alternative would have been only by introducing massive concrete foundations to increase the self weight or the excavation of multiple deep rock caissons to mobilize the rock.

4.2 Anchors to increase the stability of dams

Some old concrete dams often no longer satisfy modern safety requirements in regard to stability at exceptionally high water levels, floods and earthquake movements. On the other hand dams sometimes need to be strengthened and rehabilitated, either because they are old and thus have suffered cracking in course of time or because earthquakes or landslides have impaired the integrity of the structure.

Anchors used to increase dam safety owe their continuous success to a variety of important and unique advantages:

- they don't interfere with normal reservoir storage and can be installed at short notice;
- they don't adversely affect the overall appearance of the dam as they don't require extra space;
- they are adaptable in length for rock and soil conditions;
- they are monitorable and adjustable with regard to their working force;
- and last but not least it's known from experience that anchors are more economical than alternative solutions.

At Mullardock Dam in the Scottish Highlands (Fig.4) a sudden and alarming increase in leakage caused by unusual cracks in the dam face was discovered some years ago. A three dimensional finite element analysis showed that stresses concentrated towards the central buttress section where cracks as wide as 1,5 mm were found.
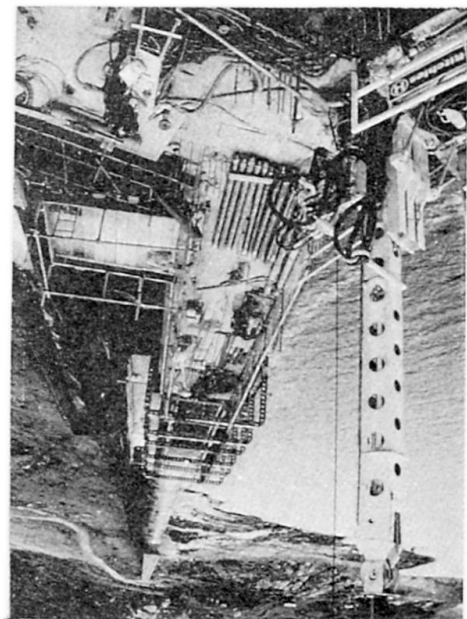

Figure 4: Anchoring at Mullardock Dam

Prestressed anchors proved to be the fastest and most economic solution to the problem and during 1990, 26 double corrosion protected, cement grouted anchors each with 37 no. 0.6" diameter dyform strands (ultimate capacity: 11,100 kN) and approx. 50 m long were assembled near the site, then airlifted to the crest of the dam by helicopter and directly homed into the drilling holes. On site acceptance tests with 3 load cycles followed for each anchor and finally left the anchors with a working load of 5,472 kN. A jumping load cell is used to monitor the behaviour, in particular the force consistency of the anchors.

The Sefid Rud Dam located approximately 200 km north-west of Tehran, Iran has undergone similar treatment. During June 1990 an earthquake of magnitude 7.3 struck the region between Zanjan and Rasth. The dam suffered extensive cracking at several locations, in particular horizontal cracks in the region 18 m below the crest. Rehabilitation work included anchoring of the upper part of the dam to the concrete body below the cracks by applying a load of about 100 MN to each of the 24 buttresses.

Installation of 234 VSL permanent rock anchors (including 40 monitoring anchors) each with an ultimate capacity of 14,078 kN and a working load of 8,447 kN began on March 3, 1991. The anchors each with 54 no. 0.6 " diameter strands have an inclination varying from 2° to 22° and are placed using a special homing device. Design bond length of the anchors is 12 m with an average overall anchor length of approximately 40 m. Stressing was completed on July 4, 1991.

Due to increased peak maximum flood levels the main wall of the Burrinjuck Dam (Fig. 5), located some 360 km south-west of Sydney, Australia was raised by a massive 13.2 m, which will result in a greatly increased spillway capacity. As for most similar cases in the past VSL permanent restressable anchors were used to secure the dam raising. The major rock anchoring work consists of installing 159 anchors each with 63 no. 0.6" diameter strands representing a minimum breaking load of 16,443 kN. The average anchoring force based on a wall crest length of 144 m and a design working load equal to 65 % of the rock anchor breaking load is 11,780 kN/m.
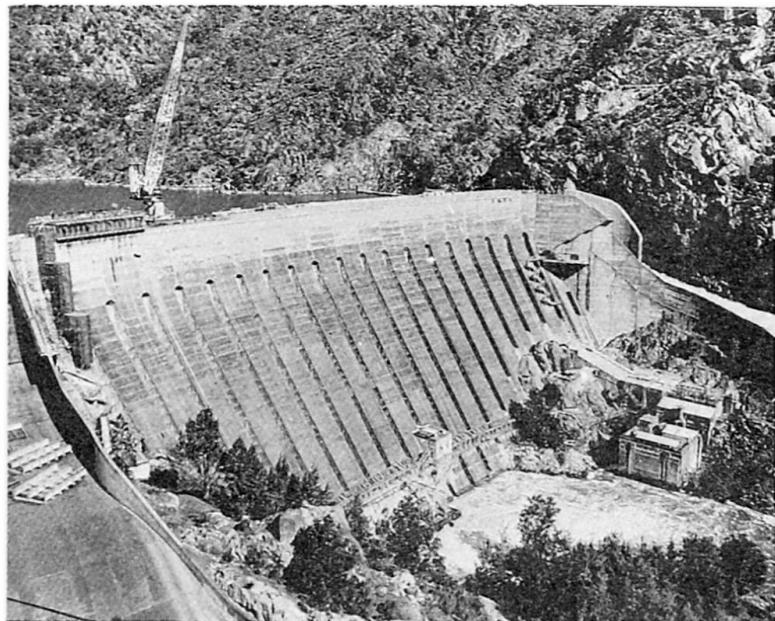
Figure 5: Burrinjuck Dam, Sydney

The anchors are arranged in two parallel rows along the crest in order to provide an acceptable spacing. To check on the performance of the anchor throughout its life the anchor design allows full load monitoring and load adjustment if required.

With a consumption of approximately 1,400 tonnes of high tensile strands the project represents one of the largest concentrations of anchoring force ever carried out in the world.

4.3 Anchors for gallery protection against rock fall and avalanches

The effects of erosion and the mechanical destruction of the rock caused by snow, ice, wind and rain very often leads to high risk of rock falls onto roads and railway lines. Various different types of rock fall and avalanche galleries have successfully been used to solve the problem. Two of them are located in Central Switzerland where rock anchors contributed to a very satisfactory solution. Near Hergiswil on Lake Lucerne the 160 m long gallery consists of unbraced free cantilevering slabs which are post-tensioned and anchored into the rock (Fig. 6). After installation of the anchors their stressing ends are provided with coupler heads to which the slab cables are connected. The anchors are 20 to 28 m long and were stressed to a working load of 3,314 kN representing 67,5 % of the breaking load.
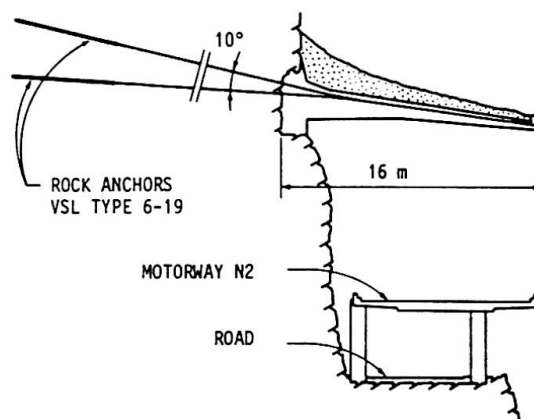
Figure 6: Rock fall gallery near Hergiswil, Lucerne

For the second gallery type (Axenstrasse, Central Switzerland Fig. 7) pre-fabricated elements were used to minimize the time during which the road had to be closed. The 1,300 m long main supporting structure is cantilevered out from the rock face and consists of upper main girders and lower struts. The upper foundation for the main girders was pressed directly against the rock face by two prestressed rock anchors every 5 m, each with a working load of 1,500 kN.
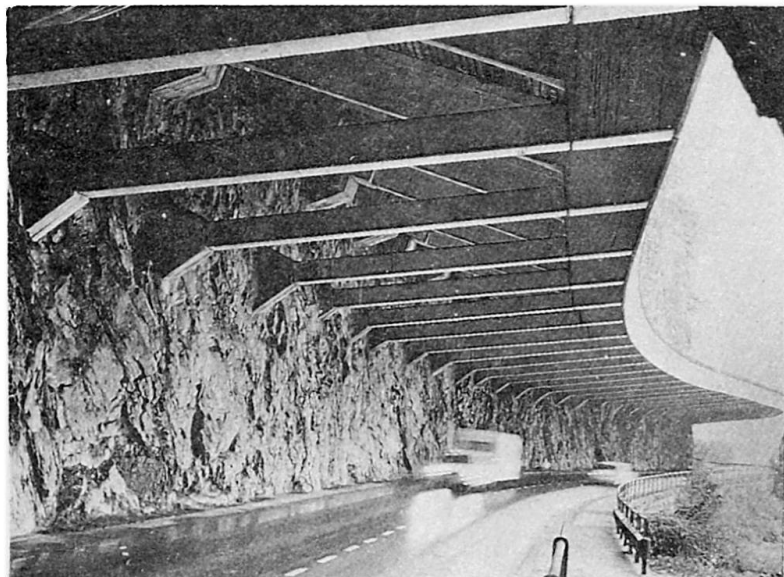
Figure 7: Axenstrasse, Central Switzerland

REFERENCES

[1] FIP, Fédération Internationale de la Précontrainte, State-of-the-art report "Corrosion and corrosion protection of prestressed ground anchorages", Thomas Telford 1986.

[2] FIP, Fédération Internationale de la Précontrainte, "Recommendation for the design and construction of prestressed ground anchorages", Thomas Telford 1982 (under revision at the time of writing this article)

Lagerung gefährlicher Gase und Flüssigkeiten in Großbehältern

Storage of Dangerous Gases and Liquids in Large Capacity Mounded Tanks

Stockage de gaz et liquides dangereux de grands réservoirs enterrés

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ZUSAMMENFASSUNG
Für eingeerdete Grossbehälter bis zu 8,50 m Durchmesser und bis zu 110 m Länge werden aus eigener Erfahrung Mitteilungen zur Ausführungsweise, Dimensionierung und zur Berechnung und Fertigung gemacht.

SUMMARY
From own experiences, information is given on the design, dimensioning and the calculation, and fabrication of big storage vessels up to a diameter of 8.50 m and a length of 110 m.

RÉSUMÉ
Nous donnons ces informations appuyées sur nos expériences personnelles quant à l'exécution, le dimensionnement, le calcul statique et la construction de grands réservoirs enterrés, dont le diamètre varie jusqu'à 8,50 m et la longueur jusqu'à 110 m.

1. TYPES OF FOUNDATION

For the foundation of big horizontal storage vessels for liquid gases with earth cover, the following types of foundation are possible:

a)   continuous reinforced concrete foundation

b)   reinforced concrete saddles,
        possibly with piled foundation

c)   sandbed storage

d)   sandbed storage combined with
        an angular retaining wall

e)   sandbed storage on a concrete rafter
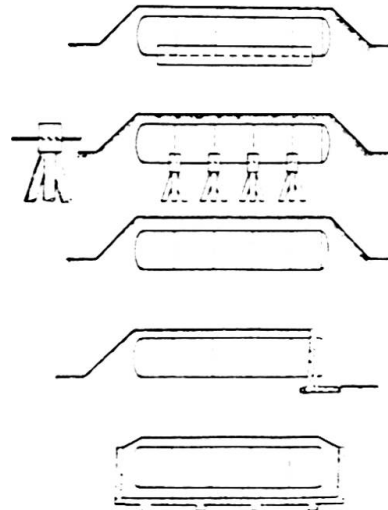
Fig. 1   Foundation methods

2. LOADING CASES

2.1 Summary of loading cases

During assemblage and transport of the vessel and installation into the sandbed, but particularly in the hydro testing and operational loading case, a variety of load components are to be considered:

Assemblage condition:	Dead weight
1st pressure test: (without earth cover)	Dead weight, water filling, test pressure differential settlements
Operation:	Dead weight, operational filling, design pressure, earth cover, traffic loads, differential settlements, temperature, friction, passive earth pressure on heads, vacuum, external explosion and earthquake (if applicable).
2nd pressure test: (with earth cover)	such as 1st test plus earth cover

2.2 Characteristic loading cases

The earth load ensued from the assumptions are presented in fig. 2. For the load assumption shown in fig. 2a, the radial component of the hydrostatic soil pressure on the shell has been taken into account. The figure 2b and 2c consider load increasing effects, which can result from settlements in the neighbourhood of saddle laid tanks (2b) or from arc action effects in the soil between tanks arranged closely together (2c).

$$p_s = -Q \frac{\pi^2 - 4(\pi - \psi_0)^2}{4R\pi(\pi - \psi_0)\cos\psi_0} \; ; \; \psi = 90° \; ; \; p_v = \frac{2Q}{2\pi R}$$

$$Q_0 = l_r \frac{\pi}{2} R(\gamma H + p_v)$$

$$Q_{t_a} = Q_0 \cdot Q_{1_a} \cdot \sqrt{2}$$

$$Q_{t_a} = l_r \left\{ \frac{\pi}{2} R(\gamma_t H + p_v) + \frac{2}{3} R^2 \gamma_t (\sqrt{2} - 1) \right\}$$

$$Q_{1_a} = \frac{(2 - \sqrt{2})}{3} R^2 \gamma_t \cdot l_r$$

$$Q_{t_b} = l_r \{ A_b \gamma_t + p_v \cdot D \}$$

$$Q_{1_b} = \frac{Q_{t_b} - Q_0}{\sqrt{2}}$$

$$Q_{t_c} = l_r \{ A_c \gamma_t + p_v (D + b) \}$$

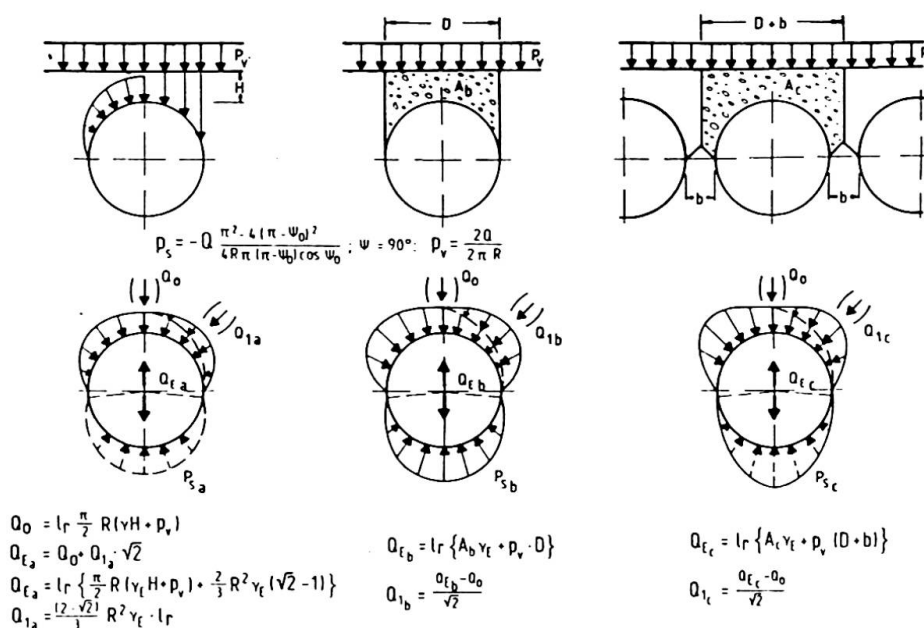$$Q_{1_c} = \frac{Q_{t_c} - Q_0}{\sqrt{2}}$$

Fig. 2   Load assumptions for earth surcharge

For the assessment of the behaviour of earth covered cylinders upon earthquake loads, earthquakes are usually prescribed by the appropriate authority.

According to fig. 3, the bearing reaction occurs on a smaller surface so that the local soil pressure as well as the circumferential reactions increase. For this loading case, however, lower safety is required or rather higher stresses are allowed (for example according to BS 5500: +25%).

R = resulting vertical load
e = ratio of horizontal acceleration and gravity
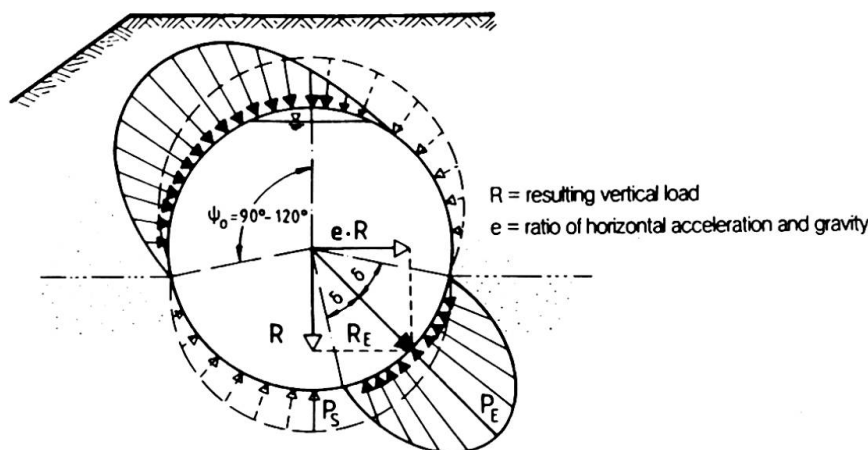
Fig. 3   Assumptions for earth covered load and bearing pressure in case of earthquake

3. STRESS INVESTIGATIONS

3.1 Elastic foundation

The determination of the reactions related to the tank's longitudinal axis should be realized according to the theory of beams on elastic foundation for the various loading cases, considering the characteristic subsoil data, where the most unfavourable distribution of the bedding module due to differential settlements should be taken into account.

The governing reactions in circumferential direction have to be calculated in the cross section of the highest bearing pressure (see fig. 4)

Fig. 4 Distribution of bending moment and bearing pressure for various bedding configurations

3.2 Stresses of secondary order caused by internal pressure

With internal pressure, the unstiffened shell area widens more compared to the ring stiffened region. The constraint developed in this way pro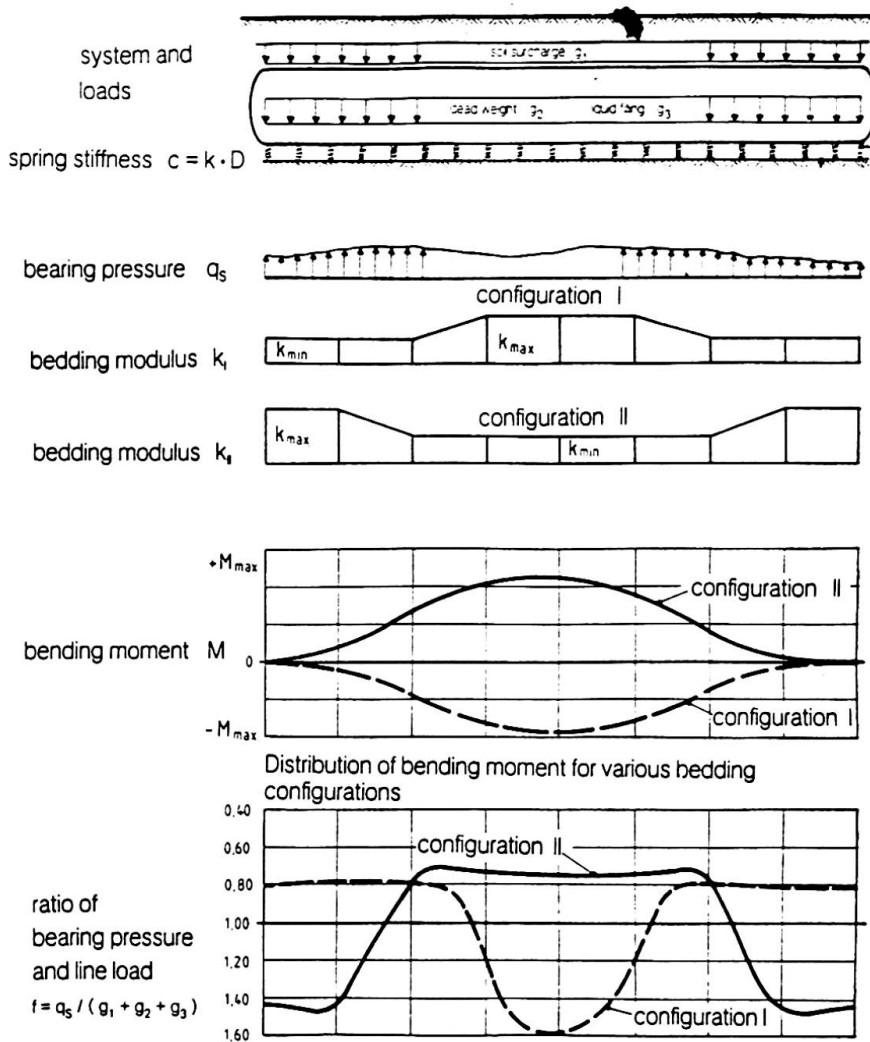duces bending moments in the tank's longitudinal direction. The calculation of this secondary stress can be realized for example according to [1]. To this particular type of stress peaks, reference is made in British Standards 5500, App. A, as so-called "self-limiting" stress, e.g. the stress peak is reduced after yielding takes place. For this reason, BS 5500 allows stresses up to the double yield strength of this particular case.

4. INVESTIGATIONS INTO THE STABILITY

The external loadings from earth cover and bearing pressure, acting non-uniformly along the perimeter, have to be considered for buckling. Further, it can required to consider underpressure in the investigation, which might occur due to a possible malfunction under service or human error.

The longitudinal bending loads between individual saddles or from uneven settlements in a sandbed as well as axial compression forces from friction and earth pressure on the heads necessitate also an investigation for the longitudinal direction.

The corresponding investigations on buckling can be carried out according to the relevant pressure vessel codes.

5. MANUFACTURING AND ASSEMBLING METHODS

5.1 Manufacturing and assembling of the steel structure

The manufacturing and assembling of horizontal cylindrical tanks with big diameters is usually carried out as pure site fabrication.

The distribution of sheets typical for this process of fabrication is presented in fig. 5.

a) conventional pipe construction

b) centre split pipe method
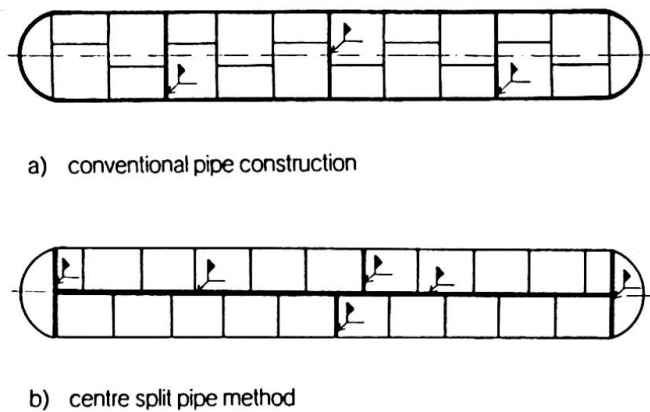
Fig. 5 Manufacturing methods

In order to reduce the work expenditure on site in favour of the fabrication in the workshop, the split pipe method has been selected for some projects. A typical course of the cuts is presented in fig. 6.

a) construction of single rings

b) connection of a pipe section

c) cutting of the cylinder into a centre split pipe

d) erection schedule

e) completed tank with inspection trenches

Fig. 6 Assembling course for the split pipe method

For the continuous position in a sandbed, accesses in the sandbed become necessary for fabrication and testing of the final seems (see fig. 6e).

After testing, these trenches cannot be compacted properly, and thus, form softer regions in the bedding. For this reason, possibly big units should be installed in order to limit the number of trenches. Recently, an

increasing number of tanks has been completely prefabricated in an assembly station on site or in the workshop.

A plan of the courses for the establishment of the earth cover is presented in fig. 7.
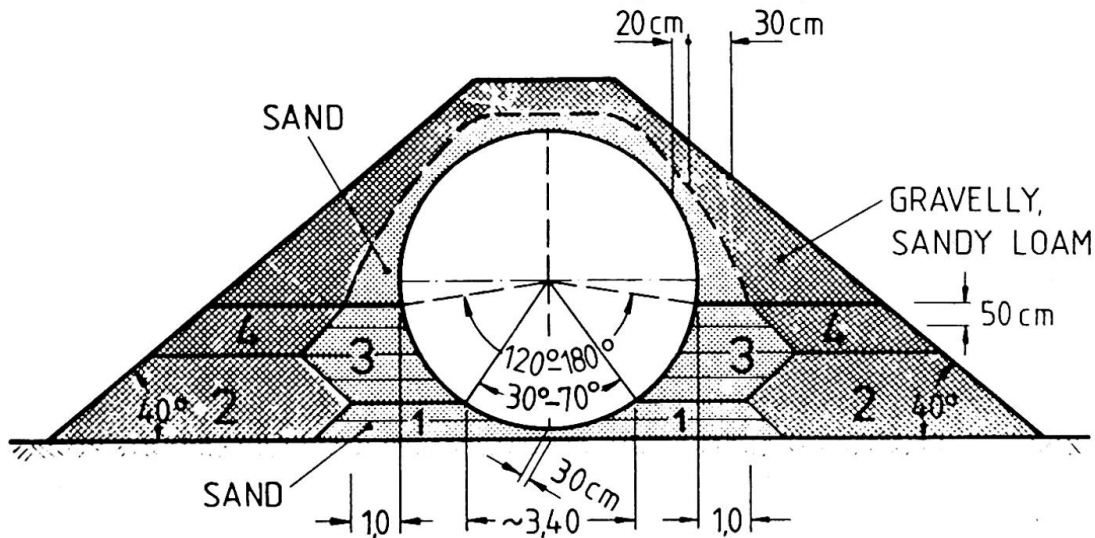
Fig. 7   Structure of layers for sandbed and earth cover
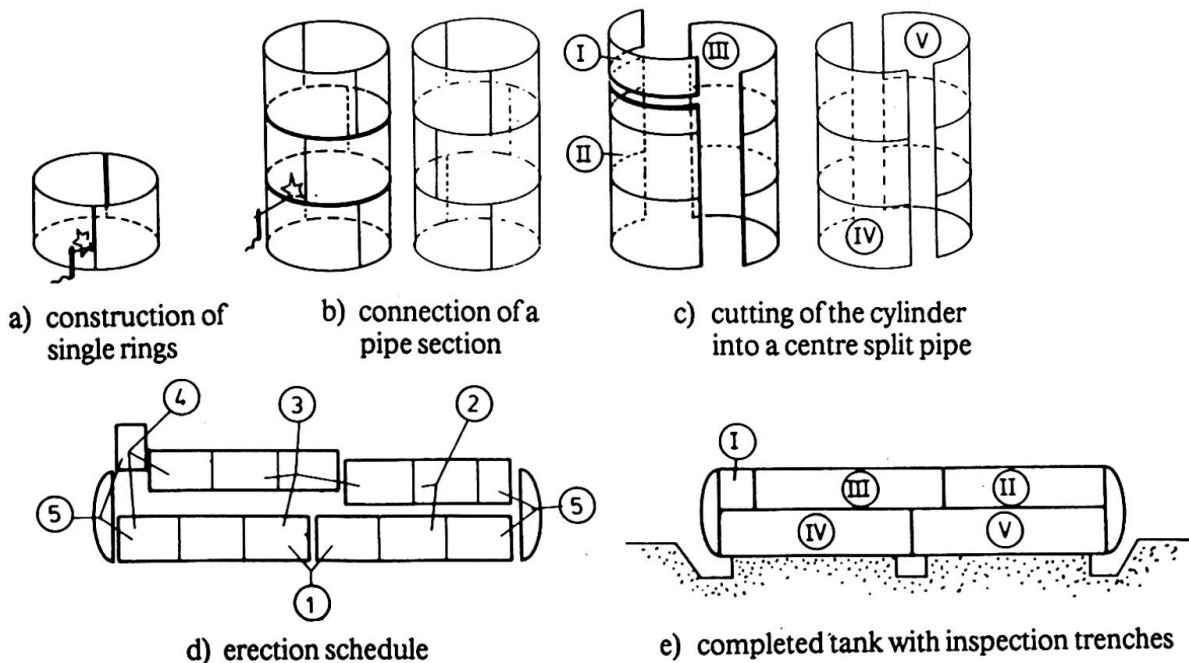
5.2 Quality assessment, performance test and control measures

When dimensioning new tanks, a welding factor of 1.0 is taken as a basis almost without exception. Therefore, each seam has to be subjected to 100% X-ray or ultrasonic proofs. These tests are done before the earth cover is installed. After a servicelife of 8-10 years, hydrotesting and weld inspections will be repeated (periodic testing).

Due to the settlement sensitivity of the tanks, which have partly a length of up to 100 m, it is recommended to monitor and record the settlement behaviour during hydrotesting and particularly under service in regular intervals.

6. REFERENCES

1.   TIMOSHENKO, S., Theory of Plates & Shells, New York

2.   MANG, F., Berechnung und Konstruktion ringversteifter Druckrohrleitungen. Springer Berlin, Heidelberg, New York 1966

3.   MANG, F., Zur Gestaltung und wirklichkeitsnahen Bemessung von geschweißten ringversteiften Behältern und Rohren. Technische Mitteilungen Heft 9, 1973, p. 414 - 417.

Innovative Strengthening of Structures for Earthquakes

Renforcement innovateur des structures exposées aux tremblements de terre

Innovative Verstärkung von Bauwerken für Erdbeben

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Hans Rudolf Ganz, born 1955, received his diploma in Civil Engineering and doctorate at the Swiss Federal Institute of Technology, Zurich. After working for VSL International Ltd. as a project engineer on the design of post-tensioned structures for several years, he is now Chief Technical Officer and Executive Vice President of the VSL Group.

SUMMARY

The paper describes new innovative methods for the strengthening of the lateral load resisting frame of a structure using the post-tensioning technique. After a brief review of general concepts, selected applications of unreinforced masonry and precast concrete buildings are presented. The paper concludes that post-tensioning is a very effective method for the strengthening of structures since it provides improved strength, improved cracking behaviour and makes optimum use of the existing structure.

RÉSUMÉ

Cet article décrit de nouvelles méthodes de renforcement de structures sollicitées par des charges horizontales en utilisant la technique de la précontrainte. Après un bref aperçu du concept géneral, l'article donne quelques applications dans le domaine de la maçonnerie non armée et dans celui de bâtiments en béton préfabriqué. Il y démontré que la précontrainte est un moyen efficace de renforcement de structures qui permet d'augmenter la capacité portante des éléments porteurs et, en outre, d'obtenir un meilleur comportement à la fissuration et une utilisation optimale de la structure existante.

ZUSAMMENFASSUNG

Der Artikel beschreibt neue Methoden zur Verstärkung der Tragstruktur von Bauwerken für Horizontallasten mit Hilfe von Vorspannung. Nach einem kurzen Ueberblick über das generelle Konzept der Verstärkungsmethode werden ausgewählte Anwendungen an unbewehrten Mauerwerks- und vorfabrizierten Betongebäuden illustriert. Es wird gezeigt, dass Vorspannung ein sehr effizientes Verfahren zur Verstärkung von Tragstrukturen ist, da sie neben erhöhter Tragfähigkeit ein verbessertes Risseverhalten bewirkt und optimalen Gebrauch der bestehenden Tragstruktur macht.

1.   INTRODUCTION

Earthquakes still belong to the most devastating types of disaster. They can cause a large number of fatalities and huge economical losses. Often, areas with old, unreinforced masonry structures will suffer major damage, [1, 2]. However, reinforced concrete buildings and engineering structures such as bridges may also be severely hit as demonstrated in the 1989 Loma Prieta Earthquake in California. While the design and detailing of new structures seems to provide an adequate level of safety for seismic loads, in general, this may not be true for older structures. A major percentage of buildings has even been built in a pre-code period, probably without consideration to lateral loads. To prevent excessive loss of live such structures need to be strengthened to provide a minimum acceptable level of safety. By definition, such minimum requirements will not provide the same level of protection as codes for new structures, in general, see e.g. [3].

This paper illustrates the potential for strengthening of structures using the post-tensioning technique. It focuses on the strengthening of the lateral load resisting frame of a structure for inadequate shear and flexural strength and/or ductility. fter a brief review of general concepts, selected applications of unreinforced masonry and concrete structures will be presented. Although important, upgrading of connections between structural elements, such as anchoring walls to diaphragms, will not be considered in this paper.

Typical approaches for the strengthening of the lateral load resisting frame of a structure include the addition of structural steel frames and the application of reinforced shotcrete to existing walls, respectively. While both approaches improve the strength of a structure they usually require major changes to the structure such as the provision of new foundations for the steel frame or added mass of the shotcrete, have a major visual impact on the appearance of the structure, or reduce the usable space in the structure. The proposed strengthening techniques with post-tensioning avoid these disadvantages. They make maximum use of the existing structural materials masonry or concrete and therefore, do not require new foundations, in general. Placing the post-tensioning tendons inside existing members minimizes the impact on the appearance and use of the structure.

2.   GENERAL CONCEPTS

The basic concept of the proposed method is to improve the strength and/or ductility of the vertical members of the lateral load resisting frame by introducing prestressed reinforcement along the vertical members. The prestressed reinforcement is designed to take all the tension forces associated with the expected lateral loads and/or displacements while compression forces are assigned to the existing masonry or concrete.

Particularly in older constructions, vertical members of the frame have been designed to carry mainly axial loads but only little flexure and shear due to lateral loads. This typically led to the use of unreinforced masonry or lightly reinforced concrete. For such unreinforced or under-reinforced members, the tensile strength of the material influences the behaviour considerably. Such members tend to show a low strength for lateral loads and, perhaps even worse, a brittle behaviour once the tensile strength is exceeded. Provision of a reinforcement

exceeding minimum requirements, i.e. to produce a strength exceeding cràcking load of the member, will considerably enhance the behaviour of the member both regarding strength and ductility. The different types of behaviour are illustrated in Fig. 1. Curves (1) and (2) in Fig. 1 show the behaviour of unreinforced and underreinforced members, respectively, while Curve (3) illustrates the improved behaviour of a member strengthened according to the proposed technique. It may be worth nothing that the favourable behaviour according to Curve (3), in Fig. 1 may be achieved by either bonded non-prestressed and prestressed reinforcement or unbonded prestressed reinforcement, and for both shear and flexural members, see [4].
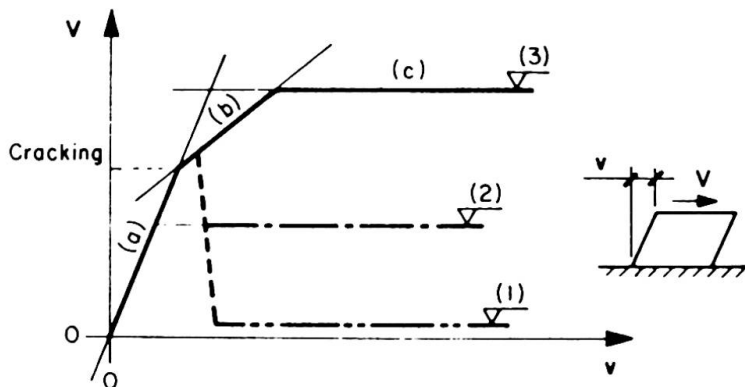

Fig. 1:
Adequat prestress improves strength and ductility of members.

To achieve a reasonable ductility of a member even without special confinement reinforcement the level of axial prestress and/or the yield force of the reinforcement shall be kept low, say below 25 % of the axial compressive strength of the member, [5]. This will provide displacement capacities of several times the nominal yield displacement of a member under cyclic load reversals.

3.   SELECTED APPLICATIONS

The following will give a short description of two examples in which post-tensioning has been used to strengthen structures or members. These examples are taken from work proposed and carried out by VSL Corporation, Campbell, CA, after the 1989 Loma Prieta Earthquake.

3.1   Unreinforced masonry, 2-storey building

The building, locally known as Brick Castle, is approximately 100 years old, Fig. 2. It consists of unreinforced clay brick masonry exterior walls, 200 mm thick, timber floors, partitions and roof, and a stone rubble foundation. The plan of the building is fairly irregular with lots of window and door openings and a particularly small total wall length on the South side, Fig. 3. During the Loma Prieta Earthquake the building had lost parts of masonry parapets, suffered considerably structural and non-structural cracking and as a consequence was red-tagged, i.e. habitants were no longer allowed to use the building. Structural cracking consisted mainly of diagonal shear cracks on the South side of the building, and vertical cracks along window and door openings.
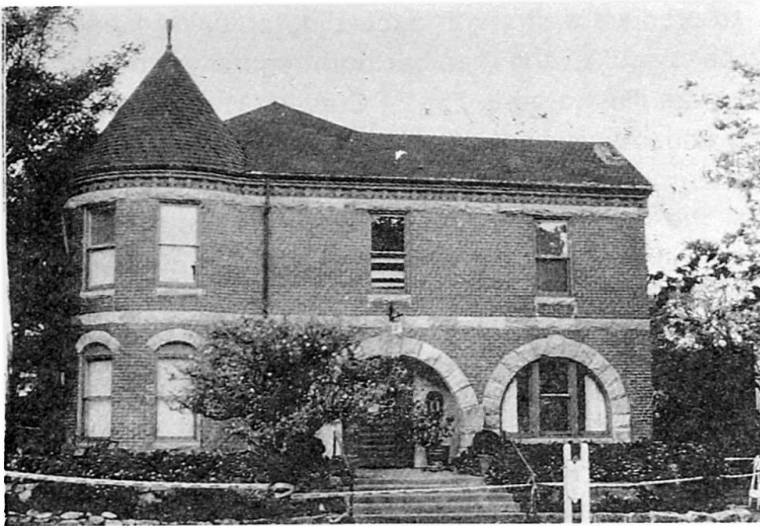
Fig. 2:
Brick Castle, Los Gatos, CA.

Structural repair work included grouting of cracks in the masonry walls, reconstruction of damaged parapets and the addition of continuous structural steel chords and anchors along the floor-wall connections to properly tie the walls to the floor diaphragms.

Preliminary in-situ testing indicated that the masonry was strong enough to allow the use of post-tensioning to increase the shear strength of the building. Consequently, vertical tendons were introduced in load-bearing walls, see Fig. 3, anchored in the stone rubble foundation and a continuous reinforced concrete beam, subsequently added at roof level. Tendons were placed in small slots at wall corners, thus tendon eccentricities were minimized and taken by cross walls. For long-term corrosion protection, monostrand tendons were used, i.e. individually greased and plastic sheathed strands. For fire protection and added mechanical protection, the slots with the monostrand tendons were grouted.

Repair and strengthening work started in fall 1990 and was completed first half 1991. Placing and stressing of monostrand tendons did not reveal any problems.
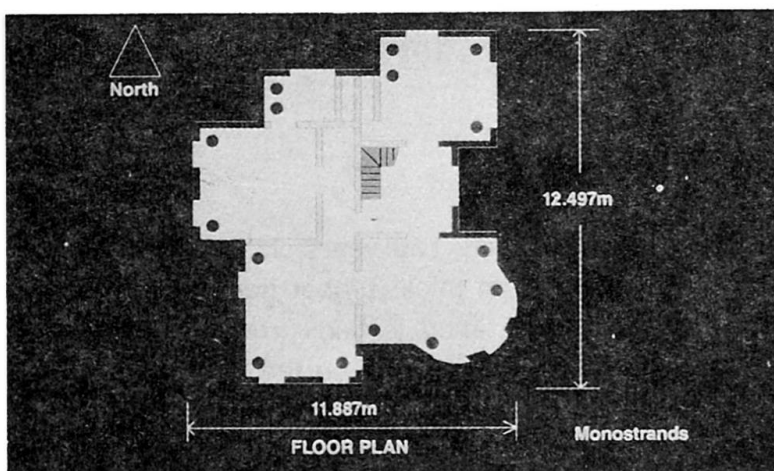

Fig. 3:
Floor plan of Brick Castle with tendon location.

3.2 Precast concrete, 7-storey building

This example considers an approximately 30 years old apartment building. It consists of a precast concrete one-way floor system supported by precast concrete wall panels, 140 mm thick, and an in-situ reinforced concrete mat foundation. In plan the building is of L-shape, each section having a substantial number of fairly regularly arranged shear walls. During the Loma Prieta Earthquake, walls panels in the ground and 2nd floor experienced severe cracking along the vertical edges. Removal of the concrete in these areas in selected wall panels, revealed two large bars, diameter 36 mm, along the panel edges without stirrup confinement. These bars were spliced at floor levels to adjacent panel reinforcement through eccentrically welded steel plates. Many of these connections had failed during the earthquake.

Since, for obvious detailing reasons, it was not possible to develop the strength of the provided main reinforcement, it was decided to remove it completely and to replace it with vertical post-tensioning. Post-tensioning tendons consisted of seven 0.5 in. strands, placed inside a thick-walled structural steel pipe. While the dead end of the tendons was in the foundation, the stressing end was at the fourth floor, in general. The steel pipe was anchored into the concrete wall panels and floors with dowel bars, diameter 16 mm, and bearing plates welded to the pipe at all floor levels, respectively, Fig. 4. After installation of pipe and strand, the wall edges were poured back with a pea gravel concrete, to their original dimensions. To minimize spalling of concrete cover in the future, a light reinforcing mesh and a special confinement reinforcement were placed along the pipe. After stressing of the tendons, the pipe was grouted to allow composite action of the prestressing and structural steel.

Strengthening of this apartment building was done in 1990. Stressing of the multistrand tendons in the relatively thin wall panels did not reveal any problems even in cases where panels were stressed eccentrically, at one end only.
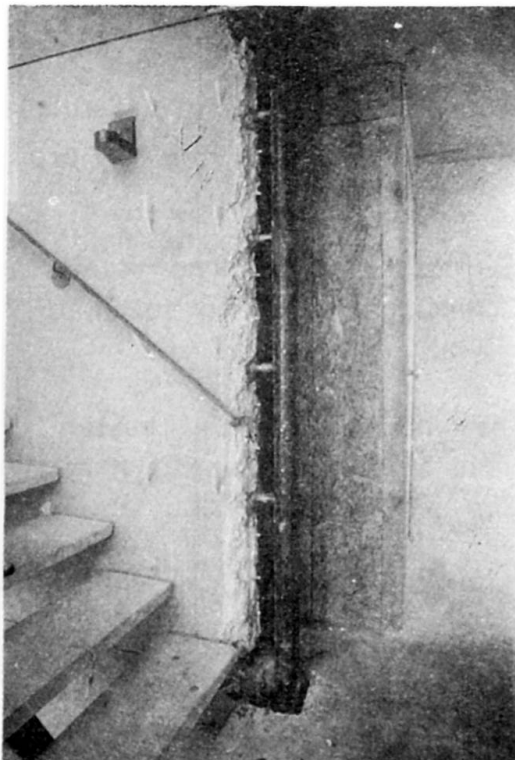

Fig. 4:
Post-tensioning tendon placed in structural steel pipe along panel edge.

4.   CONCLUSIONS

Strengthening of structures with post-tensioning tendons is most effective since it provides improved strength through a continuous reinforcement from anchorage to anchorage avoiding potential weakness at multiple laps of reinforcing bars. Further, the provided prestress improves the cracking behaviour of a structure and therefore, helps to reduce damage under low to moderate seismic loads.

The proposed strengthening technique makes optimum use of existing materials. Thus, major structural alterations such as the introduction of structural steel frames or the addition of new foundation can be avoided, in general.

While basically many types of structures can be strengthened with the proposed technique, it is best suited for larger structures made of reasonably strong masonry or concrete. Two selected examples have been described in this paper. Other examples such as the strengthening of an unreinforced masonry church and reinforced concrete columns are presently under consideration.

REFERENCES

[1]   "Loma Prieta Earthquake Reconnaissance Report", Earthquake Spectra, The Professional Journal of the Earthquake Engineering Research Institute, Supplement to Volume 6, El Cerito, California, 1990, 448 pp.

[2]   "Conference on the Newcastle Earthquake", Conference Proceedings, The Institution of Engineers, Australia, National Conference Publication No. 90/15, Barton ACT, 1990, 155 pp.

[3]   "Uniform Code for Building Conservation", Appendix Chapter 1 "Earthquake Hazard Reduction in Existing Unreinforced Masonry Buildings", International Conference of Building Officials, Whittier, California, 1987, pp. 21-33.

[4]   "Post-Tensioned Masonry Structures", VSL Report Series No. 2, VSL International Ltd., Berne, 1990, 35 pp.

[5]   H.R. Ganz, R. Guggisberg, J. Schwartz, B. Thürlimann, "Contributions to the Design of Masonry Walls", Institute for Structural Engineering, ETH Zurich, Report No. 168, Birkhäuser Publisher, Basel, Boston, Berlin, 1989, 37 pp.

Protection of Structures During Earthquakes

Protection des structures pendant les tremblements de terre

Vorrichtungen zum Erdbebenschutz von Tragwerken

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SUMMARY

Usefulness of application of devices for reducing earthquake induced forces on structures, especially bridges, is discussed in this paper. Use of devices which can be introduced at suitable points so as to reduce earthquake response is explained. Use of viscous dampers for this purpose has been explained in detail. Other vibration isolation systems have also been explained. Use of spring dashpot systems has been explained. Use of spring dashpot systems has been recommended as this is a superior system over other vibration isolation systems. The state of the art on damping devices for reducing earthquake forces on girder bridges has also been presented.

RÉSUMÉ

L'article examine l'utilité de dispositifs prévus en des points adéquats des ouvrages, en particulier dans les éléments structuraux de ponts, en vue de réduire les efforts induits par les secousses sismiques. Il explique également le mode d'action des amortisseurs visqueux ainsi que celui d'autres types d'appareillages antivibratiles, comme par exemple les combinaisons d'amortisseurs à air et à ressort. Ces derniers semblent fournir de meilleurs résultats que les autres dispositifs et il est recommandé de les envisager. Un synoptique de l'état actuel de la technique d'amortissement des vibrations est également fourni, tout spécialement pour les ponts à poutres.

ZUSAMMENFASSUNG

Der Beitrag diskutiert den Nutzen von Vorrichtungen, die insbesondere in Brückenbauwerken, die bei Erdbeben auftretenden Kräfte begrenzen sollen. Erklärt werden die Wirkungsweise viskoser Dämpfer und anderer Schwingungsisolatoren, wie Feder-Dämpfer-Kombinationen. Letztere scheinen anderen Systemen überlegen und sind besonders zu empfehlen. Für Balkenbrücken ergibt sich ein Überblick über den Stand der Dämpfungstechniken.

INTRODUCTION

When sufficient damping is not avilable by internal hysteresis of load bearing members an external device will have to be employed to attain additional damping. There is no limit to the damping that can be achieved by external damping devices. However space limitation, financial aspect along with feasibility and flexibility of incorporating the new devices has to be seen.

The oil damper permits slow movements without resistance but develops large resistance during earthquake motions introducing substantial damping thus reducing forces introduced in the system.

By introducing external device the response of structure can be reduced to tolerable limits. Not only structure but installations are also safeguarded.

MODEL TESTS

The author fabricated, tested, fitted and used oil dampers on simply supported bridge model 2.75 m span, 0.50 m pier height K= 5 kN/cm, W= 2.5 to 5 kN. The time period varied from 0.138 to 0.2 sec. The dampers were made up of piston moving in cylinder filled with silicone oil. The length of piston as well as its nominal diameter 2.54 cm, clearance 0.127 mm to 0.508 mm, C = 15 to 50 N-s/cm. The damping introduced in the system 5 to 10%. The model was subjected to free vibration tests, steady state tests and impulse load tests. During resonant testing it was observed that for damper capacity C/C_c greater than 0.20 complete locking occurs across roller bearing. During impulse load test half sine acceleration pulse 0.132 sec duration 0.7g to 0.75g was applied. Ratio of pulse duration to natural period 0.86. Acceleration response expressed as dynamic amplification went down from 1.59 (no damping) to 1.38 with dampers installed. The results matched very well with theory.

The recommendations to use dampers are based on effectiveness of device in all the three types of model tests.

BRIEF TECHNICAL REVIEW

Provision of Hydraulic Buffers to cushion longitudinal shocks as a protection against possible earthquake attack in designing Carquinez Cantilever Bridge dates back to 1927.

The U.S. National committee for Aeronautics in technical note in 1941 gave Damping characteristics of Dashpots. In the first case the piston was assumed coaxial and in second case piston assumed eccentric. Information is also avilable for circular piston in an elliptical cylinder. It was concluded that piston is normally eccentric in cylinder.

Product Engg staff report for Dashpots in 1956 claimed"synthesizing damping factor designs on paper instead of using models requiring tests and modification".

The publication of 1968 edition of code by Japanese Society of Civil Engineers gave details of many devices including use of oil dampers on bridges to distribute horizontal seismic force caused by earthquake.

Japanese practice in Seismic design of Prestressed Concrete Bridges has been described in detail in Prestressed Concrete Institute Journal of 1972 Details of dampers using oil and grease have been described. Another device described is in which "steel rod projected from girder inserted into hole filled with viscous material on top of substructure and special steel plate springs are placed between steel rod and concrete wall to absorb braking force at one support".

For the purpose of reducing seismic force on supporting structure of massive Prestressed Concrete Bridges Professor Leonhardt [1972] suggested "Limited movement between the bridge deck and the substructure. This was achieved by using modern rubber pot bearings fixed to substructure. The bridge deck can slide on teflon pads over chromated plates". Prof Leonhardt and his associates developed this method several years ago.

On earthquake resistance of bridges while describing first application of special earthquake proof devices like S.U. dampers etc in his book on Earthquake Engineering Professor S. Okamoto said " the idea of connecting bridge girder with oil damper is idea worthy of note".

In the Fifth World Conference on Earthquake Engineeing held in Rome in 1973 Influence of permitting limited rolling or sliding at both ends to distribute loads even on both piers on aseismic design of simply supported spans of bridges was again discussed. In the same conference the shock absorber called stopper and its effect on response of bridge to the earthquake was discussed. It distributed longitudinal horizontal force acting on girder during an earthquake evenly to movable support on continuous girder bridge,and reduced maximum value of response by energy absorption. These dampers were installed at the support of prestressed concrete girder Railway bridges. The effectiveness of above device was confirmed during Miyagi Earthquake in 1978 in Japan and reported in 7th WCEE held in Istanbul Turkey by Machida and others.

Reports indicate superiority of spring-dashpot system for the support of structures over other vibration isolation systems such as rubber pads for isolation from earthquake attack. Analytical reports are avilable to confirm advantage of spring dashpot vibration isolation system over Neoprene pads. According to Tezcan, Civi and Huffman "pads are reported to be a inadequate to provide vibration for vertical and rocking motion. The spring dashpot vibration isolation system was determined to be very effective in all Horizontal,vertical and rocking motion".

The patented device Extrusion energy absorber by W.H. Robinson suitable for protection of structures during on earthquake was used for protection a bridge. A number of tests were conducted on 20 kN x 2 cm stroke to 200 kN x 26 cm extrusion energy absorbers by W.H. Robinson and Greenbank.

Robinson, Longinow and Albert [1979] in the Design manual for Retrofitting bridges to withstand Earthquakes have described several retrofit concepts that can be applied to existing bridges "which will enhance the probability of survival of the structure when it is subjected to severe earthquake environment".

Degenkolb considered deficient spiral or ties in reinforced concrete columns to be great seismic weakness in bridges. He described two methods of retrofitting round columns which have inadequate ties or spirals."First is basically a turn-buckle which develops the strength of the reinforced steel and places initial prestress in the hoop. The second method consists of welding a steel shell round the existing column. The space between column and steel shell is filled with grout.

CONCLUSIONS

Use of techniques which increase damping in the system and help in reducing earthquake induced forces is recommended. The recommendation to use oil damper is based on rigrous experimental tests on a model. Damping resistances of 1000 kN.s/m per element are reported to have been measured.

ACKNOWLEDGEMENTS

REFERENCES

1. KAILASH CHANDRA, KRISHEN KUMAR, ARYA, A.S. (1982)" Reduction in Earthquake
    response of Bridges by Hydraulic Dampers", proceedings VII symposium on
    Earthquake Engg., University of Roorkee, Roorkee, INDIA (Nov 10-12,1982
    VOL 1) pp 303-308.

2. ARORA, KAILASH CHANDRA," Application of Damping Devices for Mitigating
    Effects of Earthquakes on Structures" world congress on Natural Hazard
    Reduction to be held in New Delhi India WCNHR/T-02/20.

3. LEONHARDT,F.(1972)"Improving the seismic safety of prestressed concrete
    Bridges", PCI Journal VOL 17, No.6, NOV-DEC 1972,pp 37-44.

4. ROBINSON,R.R., LONGINOW,A., and Albert, D.S,(1979) "SEISMIC Retrofit
    Measures for Highway Bridges" VOL 2 DESIGN MANUAL,FHWA-TS-79-217.

5. DEGENKOLB, O.H." Retroftting Highway Structures to Increase seismic Resist-
    ance" Highway structures- Life line Earthquake Engg pp 97-106.

Passive and Active Response Control of Buildings

Dispositifs de réponse active et passive des bâtiments

Passive und Aktive Erdbebeuisolation von Gebäuden

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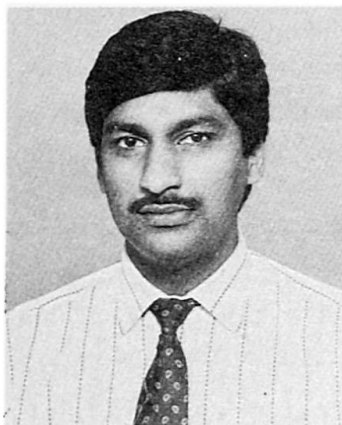
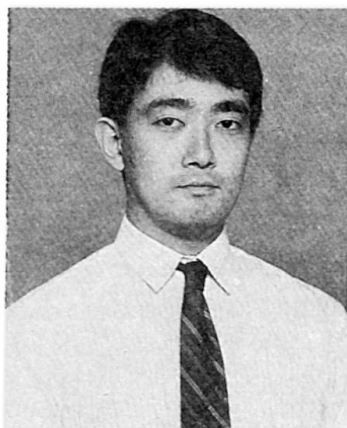Binod Bhartia, born 1962, obtained his B.Tech. at I.I.T. Kanpur, India, M.Tech. at Tokyo University, Japan, and Ph.D. at Princeton University, USA, in civil engineering. Since 1990 he is with Taisei Corporation working on active structural control problems.

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Ichiro Nagashima, born 1960, obtained his M.Tech at Kyoto University, Japan. For six years he was mainly involved in research and development of base isolation systems for Taisei Corporation. Since 1990 he is working on active structural control problems.

SUMMARY
A strategy is proposed to control the response of buildings to wind and earthquake loads. The strategy covers low to high rise buildings and identifies a variety of control devices and the range for their application. The control devices are divided broadly into passive and active types. Research activities are reviewed on actually used isolation type passive devices and mass damper type active devices.

RÉSUMÉ
L'article présente une stratégie poursuivie pour contrôler la réponse des bâtiments aux charges du vent et des tremblements de terre. Cette stratégie couvre les bâtiments bas et élevés et identifie une foule de dispositifs de contrôle et leurs possibilités d'application. Les dispositifs de contrôle sont divisés en types actif et passif. La présentation porte sur quelques dispositifs passifs du type à isolation actuellement utilisés et sur la recherche d'un dispositif actif du type amortissement de masse.

ZUSAMMENFASSUNG
Zur Kontrolle der Reaktion von Gebäuden auf Wind und Erdbeben gelangen unterschiedliche Strategien zur Anwendung. Diese Strategien umfassen sowohl hohe als auch niedrige Gebäude und eine Vielfalt von Kontrollgeräten und einen weiten Anwendungsbereich. Allgemein betrachtet sind die Kontrollgeräte in passive und aktive Typen eingeteilt. Wir erläutern einige gängige passive Ausstattungen des Isolationstyps und unsere Forschungsanstrengungen im Bereich der Massendämpfung des Type der aktiven Ausstattung.

1. OBJECT AND STRATEGY FOR RESPONSE CONTROL OF BUILDINGS

In recent years, a lot of work has been done on response control of civil engineering structures, such as buildings, towers, bridges and so on in Japan and the U.S. The purposes of response control are as follows.

1) To improve the safety of the building against severe natural hazard, such as strong earthquakes (maximum ground acceleration in the range of 300 gal to 400 gal), which the building may experience once in its lifetime.

2) To maintain the functions inside the building against medium level natural hazard, such as moderate earthquakes (maximum ground acceleration of 80-100 gal), which the building will experience several times in its lifetime.

3) To realize an enhanced comfort level for internal environment of the building subjected to wind or small earthquakes which will occur frequently in its daily life.

Research and development of several types of response control systems for buildings have been carried out as listed in Table 1. They can be classified into two types; passive type and active type. These control systems are applied according to the type of building and the control purposes mentioned above.

Here we introduce base isolation as an example of passive control systems and active mass damper as an example of active control systems.

2. BASE ISOLATION

2.1 Introduction

Base isolation is now a well-known anti-seismic strategy to improve the safety of buildings against strong earthquakes. In recent years more than 40 base isolated buildings have been built in Japan. It can be applied mainly for low to medium rise buildings. Period-lengthening type base isolation systems using elastomeric bearings are the most popular at present. However, since they have their own natural periods, they may cause resonance to earthquake motions with longer predominant periods.

A sliding-type base isolation system to reduce horizontal acceleration, which never resonates to any type of earthquake motion, has been developed at Technology Research Center. It has been named "TASS system" which stands for "TAisei Shake Suppression system"[1].

Table 1 Response Control Systems for Buildings by TAISEI CORPORARION

Building Height		Low	Middle	High
Period		Short	Medium	Long
		< 1sec	1 ~ 2 sec	2 sec <
Control Systems	Passive type	Base isolation (TASS system) ◄────►		
		◄──────── Floor isolation (TASS floor) ────────►		
				Damping System (DREAMY) ◄────►
	Active type		◄──── Active Mass Damper (AMD) ────►	
		Active Base isolation (VFB) ◄────►		
			◄──── Active Gyro - stabilizer (STREAM) ────►	

(a) At Rest   (b) During   (c) During
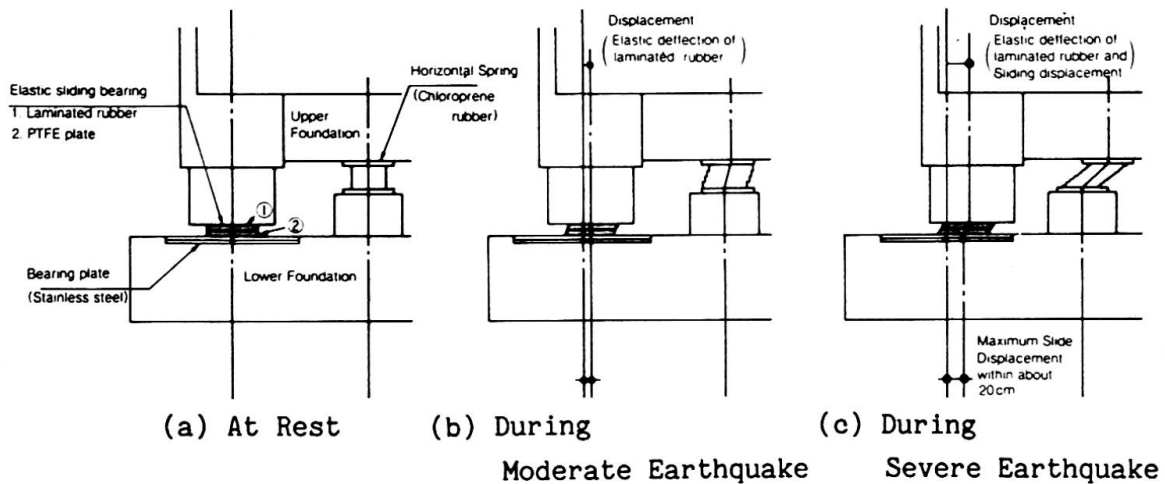   Moderate Earthquake   Severe Earthquake

Fig.1 Composition and Isolation Mechanism of TASS System

2.2 Sliding-type base isolation system

2.2.1 System Composition and its Superiority

The TASS system is essentially composed of sliding bearings, bearing plates and horizontal springs as shown in Fig.1(a). The sliding bearings and bearing plates support the vertical load of a superstructure and reduce the horizontal seismic force by sliding against severe earthquake motions. The horizontal springs restrain slide displacement with weak lateral stiffness. They sustain no long-term vertical load.

Two types of sliding bearings are prepared according to the required performance; one is a rigid type and the other one is an elastic type. Rigid sliding bearing is composed of a PTFE (Poly-Tetra Fluoro Ethrene) plate encased in a steel frame. The elastic sliding bearing is a laminated chloroprene rubber bearing with a PTFE plate attached to its bottom. This type of bearing deforms in shear at the rubber section even before sliding occurs. Accordingly an isolation effect can be expected under weak or moderate earthquakes as well as under severe earthquake motions. Behavior of the isolation devices under earthquake motions is schematically shown in Fig.1(b),(c).

The sliding-type base isolation system is supposed to have the following superiorities,
1) It never resonates to any type of excitation because the sliding mechanism has no natural period by itself unless an extremely strong restoring force overwhelming the friction is applied.
2) It stably supports superstructure because bearing devices do not deform excessively due to sliding. It is needless to say that the bearing plate should be wide enough.
3) It cripples the horizontal seismic force because no more than friction force is transmitted to the superstructure.

2.2.2 Implementation

Three buildings, utilizing this sliding-type base isolation system, have already been built in Japan. The first implementation is a laboratory building in the Technology Research Center, Taisei Corporation, in Yokohama Japan. It is

a reinforced concrete building with four stories. Its isolation devices set in the underground pit are shown in Photo 1. Elastic sliding bearings with diameters of 85 cm (maximum loading capacity 400 ton) and 75 cm (Maximum 300 ton) and with heights of 10 cm are placed at the bottom of the columns. Eight horinzontal springs of 35 cm in diameter and 15 cm height are placed at four corners.
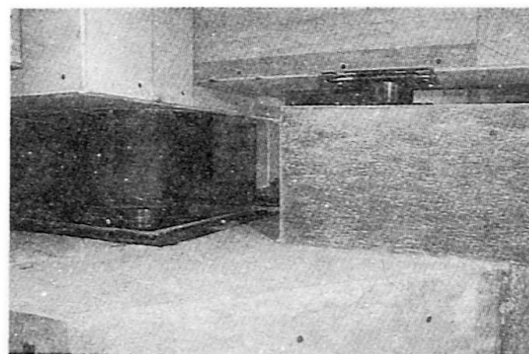
2.2.3 Earthquake Observation

Up to July 1991, since the completion of the building in July 1988, more than 100 earthquakes have been observed with the acceleration over 0.5 gal at GL-75m. The maximum ground acceleration was 55 gal observed in M5.7 earthquake Oct.14. 1989. No sliding has occurred yet. The amplification factors, which are the ratio of maximum acceleration at the roof to that of ground surface, are in the range 0.4 ~ 0.8 and mostly lie under 1.0, see Fig.2. The amplification factor of the isolated building is reduced to 1/2 ~ 1/9 that of the non-isolated building even in the stage without sliding.
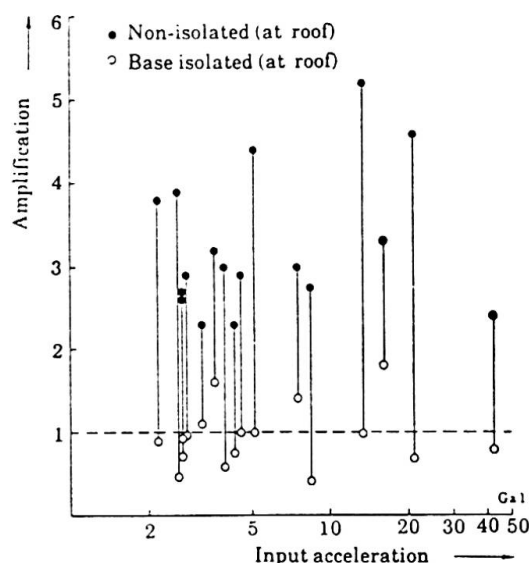
Photo 1 TASS System Actually Set

Fig.2 Ovserved Amplification of Acceleration Response.

3. Active Control Systems

3.1 Introduction

There are at least two main reasons why active structural control is a promising concept. One is that many passive control systems when augmented with even a small active control force perform much better; such systems are called hybrid systems. Another one is that active control provides the designer hitherto unavailable flexibility (and possibility) in designing buildings with enhanced comfort and safety; also, as buildings evolve with time because of structural degradations or changed performance requirements, the control system can be readjusted easily. Over the last 5-6 years a concerted effort has been made mainly in Japan and the US to active control for buildings. However, a control theory which is tailored to account for random, intermittent and transient load processes, and which can characterize the control performance more physically yet remains to emerge.

3.2 Active Mass Damper

Research activities at Technology Research Center on active mass dampers(AMD) are described, see Fig.4 for a schematic diagram of the control system. The additional mass at the top, usually 0.5-1.0% of the building mass,
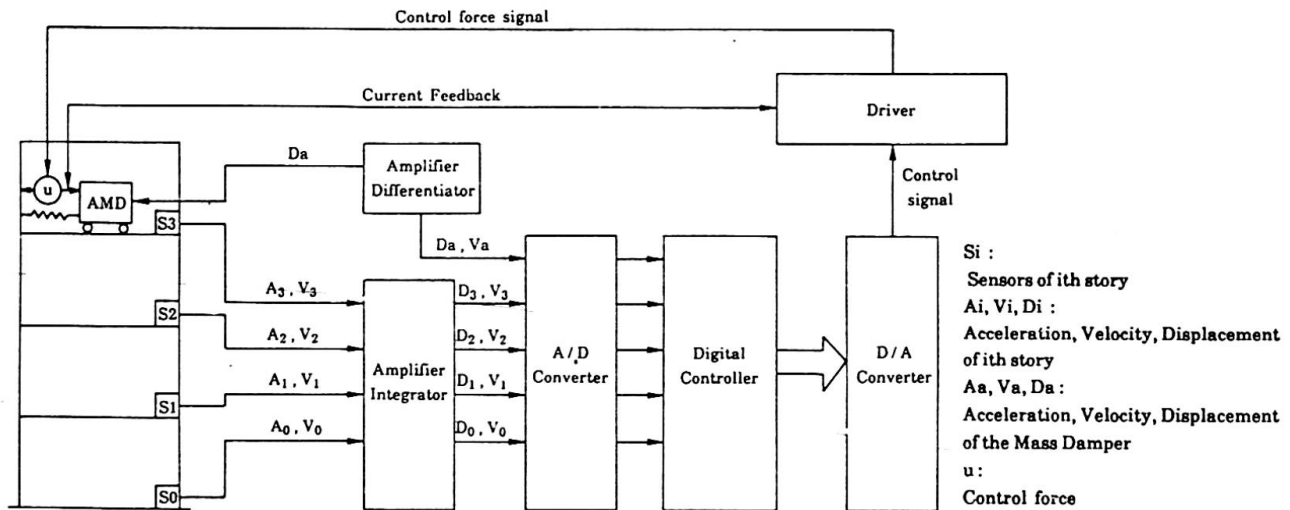
Fig.3 Active Mass Damper System

if tuned to one of the lower structural mode is called ATMD(and TMD when passive), otherwise simply AMD. The control force, u, which acts on the mass, thereby generating a reaction force on the building, is computed according to a control algorithm, pole allocation for example.

3.2.1 AMD vs ATMD

A lot of simulations were performed on the four story building model shown in Fig.4, with El Centro(NS), 1940, record of 50 gal peak acceleration as the input. With no control the first mode contributes almost 95% to the top story (peak) displacement. Assuming an additional mass of 1% of the effective first modal mass, with 6% damping, we studied if there exists a response reduction that separates the ranges of effective performances for ATMD and AMD. These simulations reveal that for small reduction, up to 25%, ATMD requires much smaller control force than AMD does, whereas for higher reductions AMD performs better[2].

In Figs.5(a)-(c) we show time hisotries of the earthquake input and the top story displacements with no control and with TMD; the TMD causes a slight reduction in the peak value 0.65 cm to 0.62 cm. We then used both ATMD and AMD to reduce the peak response value further to 0.47 cm(by 25%) and 0.31 cm(by 52%). For 25% reduction, ATMD required a peak control force of 14N, whereas AMD required 42N. In contrast, for 52% reduction, ATMD required a peak control force of 215N, whereas AMD required only 150N. Fig.5(d) shows the top story displacement for the case of 25% reduction with ATMD, and Fig.5(e) shows the same for the case of 52% reduction with AMD.

Through these simulations the following two conclusions are confirmed. The first is that a passive TMD supplied with even a small active control force may give much enhanced performance. The second is that for tall buildings if the first mode alone does not dominate the response, then an ATMD tuned to the first structural mode would be effective against wind and earthquake loads because then the same ATMD acts as an AMD for higher modes.
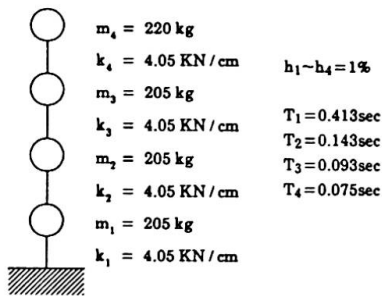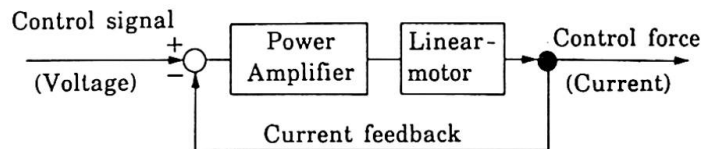
$m_4 = 220\,kg$
$k_4 = 4.05\,KN/cm$ $h_1 \sim h_4 = 1\%$
$m_3 = 205\,kg$
$k_3 = 4.05\,KN/cm$ $T_1 = 0.413\,sec$
                        $T_2 = 0.143\,sec$
$m_2 = 205\,kg$ $T_3 = 0.093\,sec$
$k_2 = 4.05\,KN/cm$ $T_4 = 0.075\,sec$
$m_1 = 205\,kg$
$k_1 = 4.05\,KN/cm$

Control signal
(Voltage) + — → Power Amplifier → Linear-motor → Control force (Current)
Current feedback

Fig.6 Block Diagram of the Driver

Fig.4 A Four Story Building Model

(c) Top story displacement with TMD   Max=0.624cm

(a) Input Earthquake   Max=50.0Gal

(d) Top story displacement with ATMD for 25% reduction   Max=0.469cm

(b) Top story displacement with no control   Max=0.647cm

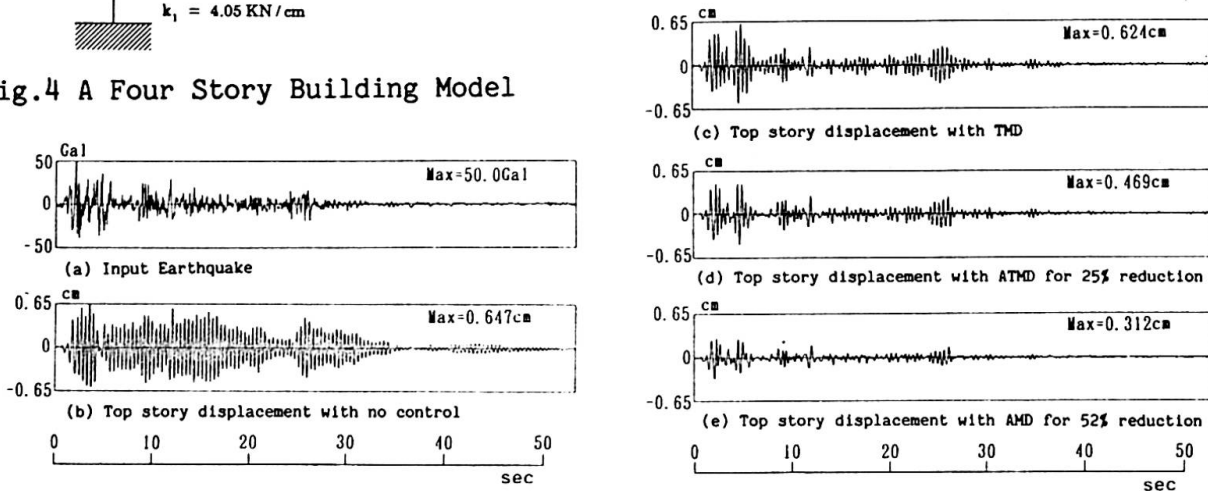(e) Top story displacement with AMD for 52% reduction   Max=0.312cm

Fig.5 Active Control of the 4 Story Building Model

3.2.2 Experimental study on AMD system

A series of experiments has been performed on the four story building model with both ATMD and AMD for the cases of full state feedback, output feedback, small reduction, large reduction, etc. The matching between the simulation and experimental results was very good(see [3]). A unique feature of our experiment was the use of a linear motor as an actuator, where the current feedback driver was used to generate the control force according to the control signal from the digital controller. The block diagram of the driver is shown in Fig.6. With high gain of the power amplifier, the time lag between the control signal and the control force tends to be zero; the measured time lag for this system is 0.9 msec. Thus, the continuous time control algorithm can be directly applied.

4. FUTURE SCOPE

As for the active mass damper we have established the feasibility through the model experiment. Our effort now forcuses on (a) development of poweful, high quality active control systems for highrise buildings, (b) studying robust output feedback control, and (c) assessing the reliability and the maintenance aspects of the control system.

REFERENCES

1. Kawamura S.,Hisano M.,Kitazawa K.,Nagashima I.,"Study on a Sliding-type Base Iolation System,",9th W.C.E.E., Vol.5, pp735-752,Aug 1988.
2. Bhartia B. and Nagashima I.,"A Study on Response Control of Buildings with Active Mass Damper,", Annual Meeting, A.I.J., pp1109-1110,1991.
3. Nagashima I.,Bhartia B.,Nishiyama S.,Kitazawa K.,"Experimental Study on Active Mass Damper System,", Proc. of Coloquium on Vibration Control, JSCE, Japan, July 1991.

Studies on Aseismic Measures for a Large Existent Sluice Lock

Protection antisismique sur une grande écluse existante

Nachträglicher Erdbebenschutz für eine grosse Schleuse

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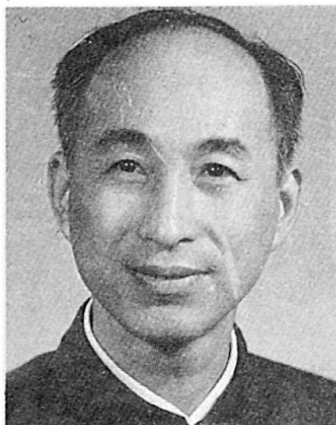
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SUMMARY

This paper presents a study on earthquake disaster reduction measures for a large sluice lock built during the late 1950s in Jiangsu, China and designed without seismic resistant capability. Field and laboratory dynamic tests were performed and a preliminary strengthening measure was taken. Field dynamic tests were performed again and a thorough three — dimensional numerical analysis taking into account structure-soil interaction and nonlinearity of the soil medium was made. Based on these studies, proposals and discussions for a better strengthening measure and rational analysis are put forward.

RÉSUMÉ

L'article présente une étude sur la limitation des effets des tremblements de terre pouvant agir sur une grande écluse à vannes, construite à la fin des années 50 sans aucune disposition antisismique. A la suite d'essais dynamiques effectués en laboratoire et sur place, l'écluse a été renforcée provisoirement, puis soumise à de nouveaux essais. Un calcul détaillé tridimensionnel a pris en compte l'interaction entre le sol et l'ouvrage, ainsi que la non-linéarité du matériau constituant le sol. En se basant sur les résultats acquis, les auteurs proposent de meilleures méthodes de renforcement et de calcul.

ZUSAMMENFASSUNG

Die folgende Studie betrifft die Erdbebenertüchtigung für eine grosse, schützenbetriebene Schleuse in Jiangsu, die Ende der fünfziger Jahre ohne jede Erdbebenvorkehrungen gebaut wurde. Nach dynamischen Labor- und Feldversuchen wurde sie provisorisch verstärkt und anschliessend von neuem getestet. Eine gründliche dreidimensionale Berechnung berücksichtigte die Boden-Bauwerk-Wechselwirkung und die ;Materialnichtlinearität des Bodens. Auf dieser Basis werden bessere Verstärkungsvorschläge und Berechnungsmethoden vorgeschlagen.

1. INTRODUCTION

Along with the occurrence of frequent strong earthquakes in China during the 1970s and the recognization of the probability of occurrence of very strong earthquakes near the lock site where the predicted intensity is IX to X in MM scale, a serial study aiming at strengthening the structure to raise its earthquake resistant capability of a large existent sluice lock located in the northen Jiangsu Province of China was carried out successively during the past 10 years. The lock which was built during the late 1950s designed at that time without consideration of seismic resistant capability is a reinforced concrete structure of 36 spans each 10m in length with a total length of 430m. Every 3 lock piers, one central pier and two side piers, together with the base slab forms a structural unit. The lock is situated on soil of medium stiffness with a predominant natural frequency around 3.3—3.9Hz. On the upper part of the lock, breast wall, highway bridge and operating bridge are constructed. The lock is a flood control and irrigation terminal of a large reservoir that controls an irrigation basin of over 0.2 million hectares in area. Therefore, the safety of the lock is a key problem in safeguarding district economic development and millions of lives.

During the late 1970s, field and laboratory dynamic tests and a 2—dimsnsional F. E. M. dynamic analysis were performed. Based on these studies, proposals for strengthening were put forward. Due to financial and some other reasons, a rather simple prelimnary strengthening measure was carried out. After the preliminary strengthening in the early 1980s, another field dynamic test was further carried out and a 3—dimensional F. E. M. dynamic analysis taking into consideration the structure—soil interaction and the nonlinearity of the soil medium was also performed. Further suggestions for strengthening and discussions on possible responses are made. In this paper, the abovementioned studies and analyses are presented.

2. FIELD AND LABORTORY DYNAMIC TESTS BEFORE PRELIMINARY STRENGTHENING

2.1 Field Dynamic Test

Resonant method was adopted for the field dynamic test with a 2t exciter. The fundamental natural frequency measured from the test is 4.8Hz for both the central pier and the side pier; the damping ratio is between 0.07~0.10 for the fundamental mode, the lateral flexural mode; and the amplification ratio of the pier top to the pier bottom is around 8~8.5. The second lowest frequency for lateral flexural mode measured is 17 Hz. The test result indicates that different piers of the structural unit vibrate simultaneously in phase as a unit although the breast wall and the piers are not rigidly connected and the central pier is thicker than the side piers.

2.2 Laboratory Dynamic Tests

2.2.1 Testing for Dynamic Characteristics of Different Structural Integrity

Plexiglass models in a scale of 1 to 50 were used in the testing for both the integrated structural unit and the single central pier. Sinuosoidal excitations applied in the testing

were carried out by both a small excitor and a medium—sized shaking table. In the testing for dynamic characteristics the following combinations of structural integrity were considered:

— (a), single central pier with a part of base slab only;

— (b), one integrated structural unit including 1 central pier, 2 side piers, the base slab and the breast wall between piers, but without superstructure;

— (c), one integrated structure unit with superstructure simply supported on the pier;

— (d), one integrated structure unit with superstructure semi—constrained to the pier;

— (e), same to (d), with 2 additional strengthening beams under the highway and the operating bridge between the pier;

— (f), same to (d), but with breast wall partially constrained to the pier.

The tested results are listed in Table 1, in which the results of field tests before preliminary strengthening are also listed for comparision. The values are all transformed into prototype according to the law of similitude. The first mode is the lateral flexural mode of the pier, the 2nd mode is the twisting mode of the pier, and 3rd mode is the second lateral flexural mode of the pier.

Item		Nat. Freq. (Hz)			Damping ratio 1st mode	Magnif. ratio top to bottom of pier
		1st mode	2nd mode	3rd mode		
Field test	Before strengthening	4.80–5.0		18.2–19.2	0.07–0.10	8.0–8.5
Model tests	Case a	4.0–4.1	8.7	16.6–19.3	0.027	
	Case b	4.1	6.2	10.5	0.074	
	Case c	3.9	8.5	14.8	0.189	7.2–8.4
	Case d	3.7	7.8	11.6		
	Case e	3.7	8.5	11.0		
	Case f	4.9	7.6	12.0	0.068	
3–D F.E. Analysis		4.88	5.04	6.10		

Table 1   Dynamic characteristics determined from the tests

2.2.2   Testing for the Interaction Effect Between Strurctural Units

Another model test for investigating the interaction effect between structural units was also performed. The tested results show that:

— the existence of side units do have some effect of reducing the responses although almost no effect for natural frequencies;

— a provision of stiff water proofing material in the expansion joint will have a beneficial effect in reducing the responses and increasing of fundamental natural frequency;

— a bigger I—section beam with a height of 4.20 m and flanges of 1 m, web of 0.30 m, in the downstream side between the piers can give more advantageous effect on the reduction of responses.

2.2.3   Analysis of the Tested Results

From the model tests it can be seen that:
— not too much differences in dynamic characteristics yield for various degree of integrity of the structure, the strengthening by only adding small connection beams under bridges will not be very effective;
— the natural frequencies measured for the case (f) in which breast walls are partially rigidly connected to the pier rise considerably and match the field test results, it seems that the actual connection between the breast wall and the pier approaches to the partially connected condition;
— a bigger beam similar to the breast wall under the highway bridge between the piers with similar constraint to the pier might not only reduce the twisting effect but also raise further the natural frequencies of the structure;
— stiffer waterproofing material in the expansion joint between the structural unit can also have some advantageous effect for the reduction of responses.

3.   NUMERICAL ANALYSES

3.1   Dynamic Equilibrium Equation of the System

The analysis is based on the following assumptions:
— the reinforced concrete lock structure is in linear state;
— the soil under the structure is a continum in linear or nonliear elastic state;
— the fluid is an ideal incompressible fluid.

Under earthquakes, the dynamic equilibrium equation of the system can be written as:

$$[M] \{\ddot{x}\} + [C] \{\dot{x}\} + [K] \{x\} = - [M] [I] \{\ddot{x}_g\} \qquad (1)$$

where $\{x\}$, $\{\dot{x}\}$, $\{\ddot{x}\}$ are the displacement, velocity and acceleration vectors of the system respectively; $[M]$, $[C]$ and $[K]$ are the mass, damping and stiffness matrices of the system; $\{\ddot{x}_g\}$ is the input acceleration vector at the rock base; and $[I]$ is a unit transformation matrix. Natural frequencies and modal shapes of the system are first solved from the eigen equation. Proportional damping is assumed using the tested results.

3.2   3—Dimensional F. E. Dynamic Analysis

A 3—dimensional   finite element dynamic analysis is carried out, taking into account the structure—soil interaction and the nonliearity of the soil medium. In the analysis, the pier, the base slab and the superstructures are discretized into plate elements, the soil medium is discretized into solid elements and the fluid is discretized into 3—dimensional fluid elements. ADINA program is adopted in the analysis, and for the time response analysis, Newmark $-\beta$ method is adopted.
From the analyses using various values of modulus of deformation of the soil medium and various ranges of soil medium with and without soil mass being considered, it can be seen that for the case taking $Es = 80$ MPa with the range of soil medium equal to 40 $\times 36 \times 12$ m for one structural unit and the mass of soil medium not considered the

fundamental frequency is 4.88 Hz, closest to that from the field test as also shown in Table 1. However, for higher modes, the differences are large as in the space analysis natural frequencies of the soil medium and other structural elements are all included. Earthquake responses are evaluated taking into consideration the structure—soil interaction for linear and nonlinear soil medium. In the elastic condition, $Es = 80.0$ MPa is taken, and for the nonlinear condition, stress —strain relationship suggested by Hardin and Drneivich is adopted with slight modification with initial $Eso = 80.0$ MPa. EI Centro acceleration record with peak value regulated to 0.50 g is used for the input at the bottom of medium in the direction of cross current. The evaluated results for some key points are listed in Table 2.

Item	Point	Location	Analytical condition		
			Linear med.	Nonlinear med.	Rigid base
Max. diaplacement (cm)	10	Pier top mid.	1.13	1.61	0.618
	15	Pier top downstr.	1.11	1.60	1.067
	16	Pier top upstr.	1.14	1.61	0.847
Max. acce. (g)	10	Pier top mid.	0.810	0.765	0.444
	15	Pier top	0.891	0.759	0.369
	16	Pier top upstr.	0.824	0.765	0.572
Max. normal base stress (MPa)	1	Pier base upstr.	0.801	0.937	1.031
	4	Pier base middle	0.565	0.548	3.281
	7	Pier base downstr.	1.060	1.081	4.520

Table 2 Comparison of analytical responses for linear and nonlinear soil medium

From Table 2, it can be noticed that normal stresses at the base of pier are greatly reduced when deformation of soil medium is taken into account especially at the toe to only about 25% of the values for rigid base condition, although the maxium displacements and accelerations at top of pier both increased considerablly. This might be due to stress redistribution between the pier and the base slab as well as to the deformation of base slab in compliance to the deformation of soil medium. It can also be noticed that differences are not large between the values evaluated for linear and nonlinear soil medium.

4. STRENGTHENING MEASURES

4.1 Preliminary Strengthening Measure and Field Test After Preliminary Strengthening

Due to financial and other causes, a preliminary simple strengthening measure was taken. It includes 2 relatively small reinforced concrete T beams between the piers un-

der the highway bridge and 1 even smaller steel truss beam under the operating bridge. After the preliminary strengthening, field test was again performed. It is found that the change of dynamic characteristics is insignificant, however the reduction of responses is noticable. The fundamental frequency raises from 4.8 to 4.90–5.15 Hz, the second mode frequency which is of the twisting mode of the pier and had not been identified in the previous field test is 8.4–8.6 Hz, and the third mode frequency which is the second flexural mode of the pier is 17.4–18.0 Hz. The damping ratios determined are between 0.06 to 0.112 for the first mode, 0.041–0.069 for the 2nd mode and 0.037 to 0.074 for the third mode. The displacement amplitudes reduce from 33% to 56% for various points tested. It seems that the preliminary strengthening yields a certain degree of effectiveness although not very ideal. Therefore a proposal for further strengthening is put forward.

4.2　Proposal for Further Strengthening

From the field tests, model tests and the numerical analysis made, a proposal for further strengthening is put forward:

A large connecting reinforced concrete beam similar to the breast wall at the upstream side of the lock in the downstream between the piers close to the haunches and partially rigidly connected to the pier for each structural unit is suggested to replace the present small connecting beam.

The larger beam will not only reduce significantly the twisting effect of the pier at the downstream side of the lock but will also significantly enlarge the difference between the fundamental natural frequency of the lock and the predominant frequency of the soil medium and thus reduce the magnification effect. Stiffer waterproofing material placed in the gaps between structural units might be benefiical to increasing damping ratio and reducing responses.

5.　CONCLUDING REMARKS

Through the serial study, a comprehensive concept of the dynamic characteristics and earthquake resistant capability of the sluice lock is obtained, and a proposal is put forward for further strengthening.

As to the analysis, it is suggested that the lock, being a complicated structure, should be analyzed 3–dimensionally taking into consideration of structure–soil interaction if rational seismic responses are to be obtained, in which the soil medium can be taken to be elastic under earthquakes of medium intensity but better to be nonelastic if the intensity of earthquakes is large.

REFERENCES

1. Dai, G. Y., Wu, C. S., et al., Aseismic Testing Studies on Zhang Shan Sluice Lock, NHRI Report, Nov. 1980. (in Chinese)
2. GAO, M., MA, G. Y., Seismic Analysis of Sluice Lock Taking into Consideration of Structure–Soil Interaction and Method of Reliability Analysis, NHRI Report, Dec. 1989.

Prediction of Structural Damage by Using Expert Systems Technology

Prèdiction des dommages structuraux par systèmes experts

Schadenprognose für Stahlbetonbauten mittels Expertensystemen

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SUMMARY
This work deals with the development of a data base for the damage level evaluation of reinforced concrete structures affected by severe fire. This data base was coupled with a bayesian engined expert system, called EECC (Engineering Evaluation of Critical Conditions). Several hypotheses and evidences were considered and codified into the data base. Some illustrative examples are presented and discussed.

RÉSUMÉ
La présente étude porte sur le développement d'une base de données destinée à évaluer les dommages subis par les constructions en béton armé, à la suite d'incendies de grande envergure. Cette base de données a été reliée à un système expert appelé EECC (Engineering evaluation of critical conditions). Elle comporte un grand nombre d'hypothèses et de facteurs d'influence sélectionnés à cet effet. Les auteurs présentent en outre quelques exemples d'application.

ZUSAMMENFASSUNG
Die vorliegende Arbeit befasst sich mit der Entwicklung einer Datenbasis zur qualitativen Schätzung von Schäden in Stahlbetonbauten nach Bränden. Die Datenbasis wurde mit Expertensystem EECC (Engineering Evaluation of Critical Conditions) verbunden. Verschiedene Hypothesen, Grundannahmen und Einflussfaktoren wurden für die Datenbasis ausgewählt und eingefügt. Einige Beispiele werden diskutiert.

1. INTRODUCTION

Figure 1 illustrates the typical architecture of an expert system. As it can be seen, the kernel of the system is made up by the knowledge base and the inference engine.

The knowledge base essentially collects the "knowledge" of the computer, in terms of production rules, frame structures, rets, etc. [1,2,4]. Of course, this knowledge can be codified with probabilities associated to the production rules, which is the most often case in engineering in view of the probabilistic nature of the engineering processes.

Figure 1. Typical Expert System Architecture

The inference engine peforms the "reasoning" of the system, by combining the production rules to produce "intelligent" responses. Expert systems technology has experimented an exponential development in the last few years. In the case of engineering, a great number of expert systems has been developed to assist the specialists in the realization of non-trivial works, thus giving an improvement of the human performance. A large list of expert systems in many engineering fields are briefly described in refs. [2,3].

This work deals with the development of knowledge bases for the damage level evaluation of reinforced concrete structures affected by severe fire, as well as the risk prediction in slope unstability problems. These knowledge bases were coupled with an expert system (Engineering Evaluation of Critical Conditions EECC). Also, three modules to help the user were developed: a) a justification module, which is able to explain (at the user request) the reasoning path followed by the bayesian inference engine, b) an on-line help module, which provides the user with a more detailed explanation of technical terms appearing in the computer questions and c) a suggested reparations module, which gives the inmediate actions that must be taken according to the damage level detected.
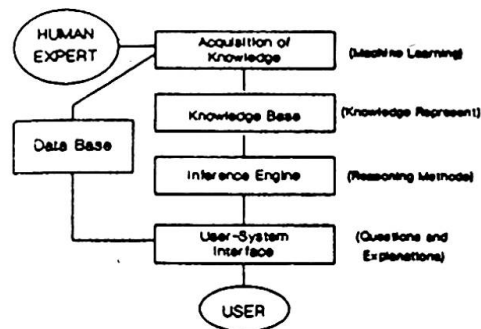
2. THE BAYESIAN APPROACH

The Bayes theorem has a singular importance when dealing with phenomema involving probabilistic information. Some examples are engineering design, damage assessment, prediction of attributes, etc.. In such cases, information is available from several sources, namely: experimental tests, engineer's experience, visual inspection, etc.. In what follows, we will briefly review the basic ideas of the Bayes theorem. Let U be the universe comprising a set of a mutually exclusive events Hi and Ej another event which also belongs to U. The conditional probability for the presence of event Ej, assumed that event Hi is present, is:

$$P(Hi:Ej) = P(Hi\&Ej)/P(Ej) \qquad (1)$$

where

P(Hi&Ej) = probability for the ocurrence of both events simultaneously

From (1) it can be obtained

$$P(Hi:Ej) = P(Ej:Hi) * P(Hi) \tag{2}$$

Thus, Bayes theorem could be written as follows:

$$P(Hi:Ej) = P(Ej:Hi) * P(Hi)/P(Ej) \tag{3}$$

In the present analysis, Hi should be interpreted as an hypothesis, while Ej must be viewed as a piece of evidence:

P(Hi)   = probability "a priori" for the presence of hypothesis Hi

P(Hi:Ej)= probability "a posteriori" for the presence of Hi, updated with the information of evidence Ej

P(Ej:Hi)= conditional probability for the presence of Ej, assumed the presence of hypothesis Hi.

3.   BRIEF DESCRIPTION OF THE EXPERT SYSTEM

The expert system developed during this research, named Engineering Evaluation in Critical Conditions (EECC Ver 2.1), is a computational system written in Pascal. It uses the Bayes theorem for performing the inference process, related to both prediction and diagnosis of real-world environments.

The inference engine was defined separately from the knowledge base. It is able to process "a priori" information, which is supplied mainly by human experts as well as "a posteriori" information, which is provided by the expert system user.

In this work, the "a priori" information was clasiffied, for the sake of clarity, into three main items: beams, columns and slabs. This strategy allowed the treatment of different structural components subjected to the same level of fire severity. The wide spectrum of practical damage levels in structures subjected to fire was discretized into four main categories, as follows:

* Slightly affected
* Moderately affected
* Strongly damaged
* Severely damaged

On the other hand, three modules for helping the user in the interpretation of the program performance, were developed and coupled with the system. The justification module explains the user the path followed by the system in the actual inference process. This justification process is based upon a permanent evaluation of the effect caused by the user answers on the probability values of the hypotheses currently under analysis.

Two sets of linguistic variables were created, in order to facilitate the user-computer communication. The former assigns the numerical user-answer a linguistic variable, which is used latter in the justification report. The second set was coupled to a mathematical function which evaluates the shift in the probability values caused by the user-answer.

The on-line help module allows the user to get a more detailed information regarding the current question prompted by the computer, since the questions in the knowledge base were formulated in a very synthetic manner and by using technical language.
The corrective module herein implemented was designed with the aim to provide the user the immediate required actions that must be undertaken according to the diagnostic given by the system. It must be emphasized that the corrective module is not intended to replace neither the human expert appreciation nor the need of experimental tests; it was just conceived to give some basic actions at a first glance.

4. ILLUSTRATIVE EXAMPLE

The knowledge base developed here was extensively tested and modified taken into acount the suggestions made by the human experts. Also, some critical situations were considered, giving satisfactory results.
The present example pressuposes the existence of a reinforced concrete beam affected by a severe and long-term fire. As a consequence, the beam shows strong symptons of structural damage.
The following text reproduces the dialog user-computer, when the above described example was analysed.

Please, answer the questions according to the scale shown below:

AFFIRMATIVE	NEGATIVE
5: No doubts	-5: Categorical
4: Strongly	-4: Firmly
3: Reservedly	-3: Moderately
2: Weakely	-2: Feeble
1: Wondering	-1: Uncertain

0: Total uncertainty

EECC: Were there non-structural smelted steel observed ?
User: 5
EECC: Did the concrete remain undamaged ?
User: -5
EECC: Were there cracks observed ?
User: 5
EECC: Did the firemen extinguish the fire ?
User: 5

(As the dialog progress, the system incorporates more and more "a posteriori" information and, after some more questions, it reaches and announces its most likely conclusion.)

EECC: Was loss of frieze observed ?
User: 5
EECC: Did you observe loss of adherence between steel and concrete
User: 5

DIAGNOSIS: The beam is SEVERELY DAMAGED with a probability of 99%

In order to perform the assessment of system reliability, there were runned on the computer three typical examples, corresponding to the three structural components currently available in the program.

It is necessary to demostrate that the system do not "jump" around intermediate local situations and that the questions prompted by the computer are made in a logical manner, similar as a human expert may do. An ilogical sequence of the questions probably will reduce the confiability of the user in the system. The way selected to obtain the results shown in figures 2,3 and 4 is as follows: all the computer questions were answered with the same level of confidence. So, four probability values were obtained for each analysis. Figures 2,3 and 4 illustrates the responses of the system for a beam, an slab and a column, respectively. It must be remarked that the system always exhibited a conservative behavior in the presence of very uncertain situations. Also, the system showed an adequate convergence towards the limits expected in known situations. These encouraging facts estimulated the refinement and improvement of the knowledge bases. This process is currently being done, either by reformulation of the evidences and its associated hypotheses or by numerical modification of the conditional probability values. In both strategies, the presence of the human expert is an unavoidable requirement if one expect to get success.

EECC Behavior
BEAM Evaluation

Figure 2

EECC Behavior
SLAB Evaluation

Figure 3.

EECC Behavior
COLUMN Evaluation

Figure 4.

5. CONCLUDING REMARKS

A knowlegde base for the evaluation of structural damage caused by severe fire on reinforced concrete structures has been developed. This knowledge base allows the user to know the damage level on either beams, columns or slabs of a certain concrete structure, that has been exposed to a long-term fire. As it is well-known, a wide variety of factors do affect a structural component when it is exposed to fire and, therefore, the evaluation of the current strength and confiability of such a component becomes an engineering subject of the most importance and not easy to be performed.
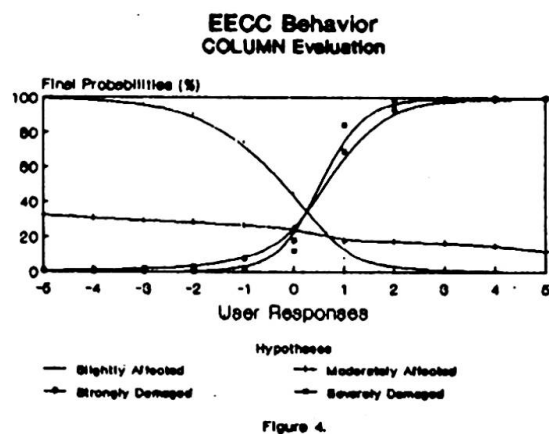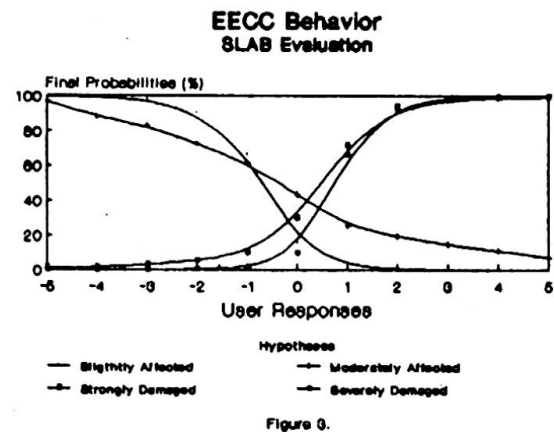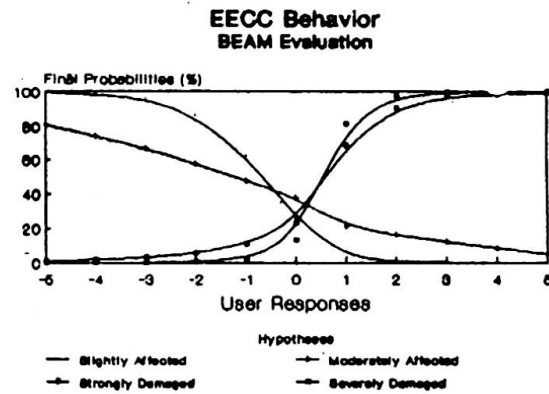
On the other hand, by reasons of space limitations, it was not possible to describe some preliminary results obtained when the expert system EECC was applied to the determination of security factors and risk assessment in soil slope unstability. However, it can be said that we have obtained a very encouraging results. Many practical soil engineering situations dealing with soil slopes were analysed with the computer, and it was observed that in more than 70% of the cases, the computer's prediction agreed with the human expert ones.

The complementary modules of EECC, i.e. justification, explanation and suggested reparations here developed, have substantially improved the behavior and performance of the expert system, as well as the user-computer interaction. Justification module explains the user the current reasoning path by using some sets of linguistic variables developed during this research. In this sense, the system is able to tinge his answers, so giving a really flexibility and a quite friendly behavior.

The on-line help module facilitate the user understanding of technical terms, which usually appears in specific engineering fields. Moreover, the powerful linkage of this two modules, help and justification, has shown that the system could be used as a tutorial for engineering students, provided that they have been appropiately trained in the subject.

The module of suggested reparations has also shown its usefulness. This module complements the diagnostic announced by the system, by providirg some preliminar and unavoidable reparations, associated to the damage level, which must be performed on the structural component.

The cornerstone matter when dealing with expert systems development and implementation is usually the validation of the system. The EECC system, and its knowledge bases, have displayed an "smooth" behavior, without showing "jumping" around certain local points and clearly identifying limit situations. Moreover, the system produced diagnostics which agreed with human experts approximately over 80% of the cases treated herein.

7. REFERENCES

[1] HAYES-ROTH, F., WATERMAN, D. AND LENANT, D. (Eds. 1983), "Building Expert Systems", Addison Wesley, New York.

[2] WATERMAN, D. (1986), "A guide to Expert Systems", Addison Wesley, USA.

[3] BARR, A. AND FEIGENBAUM, E. (Eds. 1982), "The Handbook of Artificial Intelligence", Vols. 1 and 2, Pitman Books, London.

[4] NILSSON, N. (1980), "Principles of Artificial Inteligence", Tioga Publishing Co.

[5] CERROLAZA, M., GOMEZ, B. AND ALARCON, E. (1988) "Expert Systems for Quality Prediction in Structural Engineering", 13th IABSE Congress, Helsinki, Finland.

[6] MANEIRO, T. AND TATOLI, M. (1990) "Expert System for the Damage Level Evaluation of Reinforced Concrete Structures Affected By Severe Fire", UCV, Caracas, Venezuela. (Spanish)

[7] CERROLAZA, M., MANEIRO, T., TATOLI,M. AND VALBUENA, H. (1991) "Expert Systems for Evaluation of Engineering Criticals Situations", II FACAM Congress, Valparaiso, Chile. (Spanish)

[8] DUDA, R.O, HART, P. AND NILSON, N. (1976), "Subjetive Bayesian Methods for rulebased inference systems", in Proc. Nat. Comp. Conference, New York, 7-10, June 1976.