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SESSION 4

Operating Methods

Méthodes de renforcement

Methoden der Ausführung

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Reconstruction des structures du Dôme de Milan

Wiederaufbau der Tragstruktur des Mailänder Doms

Rebuilding of the Supporting Structure of the Dome de Milan

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Carlo Ferrari da Passano, né en 1917, ingénieur civil, a exercé une vaste activité dans le domaine des constructions et de la restauration des monuments. Depuis 1965 il est l'Architecte de la V.Fabrique de la Cathédrale de Milan.



Guido Oberti, né en 1907, a été professeur de constructions civiles aux Polytechniques de Milan et de Turin. Directeur puis Président Honoraire de l'ISMES. Membre d'honneur de plusieurs Associations Internationales. Ingénieur conseil dans le domaine des ponts et grands barrages.

RESUME

La contribution explique les raisons et le but de la reconstruction des colonnes centrales soutenant le «tiburium» du Dôme. Après elle se déroule en deux parties. La première expose les essais, effectués à l'aide d'un modèle élastique et de différents modèles physiques poussés jusqu'à la rupture. Les résultats les plus importants sont présentés. La deuxième partie donne des renseignements sur l'oeuvre de reconstruction choisie, actuellement en cours, et expose le système adopté sur le chantier pour le contrôle automatique des déformations et des contraintes.

ZUSAMMENFASSUNG

Der Beitrag zeigt die Gründe und den Zweck des Wiederaufbaus der Zentralsäulen, die die Domkuppel stützen, auf. Dann teilt sich der Beitrag in zwei Teile. Im ersten werden elastische Modellversuche des Zentralteils und der angrenzenden Seitenschiffe sowie Versuche an grossmassstäblichen Stützen bis zum Bruch beschrieben und die wichtigsten Ergebnisse erläutert. Der zweite Teil beschreibt die jetzigen Aufbauarbeiten sowie die installierten automatischen Verformungs- und Spannungskontrollen.

SUMMARY

This paper first explains the reasons and the aim of the rebuilding of the central columns which support the cupola of the Cathedral. It is then divided into two parts. The first part sets forth tests carried out, both with the aid of an elastic model of the Cathedral's central section including the cupola and relevant naves, and by using various large-scale models of the columns, tested to failure. The second part provides information on the rebuilding work being carried out, and briefly describes the system used on site for the automatic control of strains and stresses.

1. CONSIDERATIONS PRELIMINAIRES

Le Dôme de Milan se lève au centre de la ville sur un terrain où, déjà à la époque romaine, on avait établi d' importants bâtiments civils et religieux. Le commencement de la construction, qui remonte au 1386, est lié aux événements politiques de cette époque en Italie et, en particulier, à Milan, lorsqu' avec Jean Galeazzo Visconti en 1385 l' on atteint l' unification du Duché dont les visées égémoniques s' étendirent presque à toute l' Italie septentrionale entrainant et poussant dans la participation à cette réalisation le clergé, dans la personne de l' Archevêque Antonio da Saluzzo, et tout le peuple de Milan.

Le style du monument est inspiré au gothique de l' Europe centrale qui, à cause de nouvelles particulières élaborations introduites par l' art lombard en phase de projet et d' exécution, en a fait un exemplaire unique.

Les cinq siècles qui ont été nécessaires pour achever cette Cathédrale, eurent de remarquables conséquences soit stylistiques que statiques, qui encore aujourd' hui sont évidentes et d' une extrème actualité.

Les moments les plus significatifs furent le bâtiment de la grande coupole du Tiburium en 1500, la réalisation de la grande flèche en 1774 et l' achèvement de la façade en 1813.

Le matériel qui aurait dû être employé pour la construction, selon le projet originaire, était en brique et il fût, au contraire, remplacé pour toutes les parties en vue, soit internes que externes du monument, par le marbre de Candoglia qui est un carbonate de chaux au 98% de structure cristalline et avec d' excellentes caractéristiques chimico-physiques: résistance à la rupture (par compression simple) supérieure a 1.000 Kg/cm^2; module élastique moyen 500.000 Kg/cm^2.

Les fondations du Dôme, constituées par plinthes isolés pour les piliers de la grande nef, du transept et de l' abside et par structures continues périmétrales, ont été posées au limite du niveau de l' "aves", à ce temps là entre les 7-8 mètres au dessous du plan de campagne sur un terrain alluvial, formé de gravier et de sable avec des infiltrations de vase et de couches de vase sableux et argilleux jusqu' à la profondeur de 120 mètres, au dessous on a des terrains lacustres et fluvio-lacustres de l' ancien quaternaire.

Les principaux éléments statiques de la construction sont représentés par les grandes colonnes, les quatre du "Tiburium", et les piliers secondaires, objet de l' intervention statique (Fig. 1 et 2).

La composition des piliers est hétérogène et constituée par une couronne périmétrale en marbre de Candoglia, avec huit nervures grandes qui s' alternent avec quatre petites, et par un noyau intérieur, formé par des couches non pas liées avec celles du marbre, et qui a une épaisseur assez variable, composé de briques, de pierres de "Serizzo" et d' écailles de marbre, liées ensemble avec du mortier de chaux et de sable.

Le rapport entre les surfaces des deux types de la section horizontale des piliers est presque égale. Le diamètre moyen des colonnes du Tiburium, qui soutiennent un poids de 3.200 t, double des autres, résulte de m 2,90 contre m 2,45 des piliers secondaires. La valeur de la charge transmise au terrain

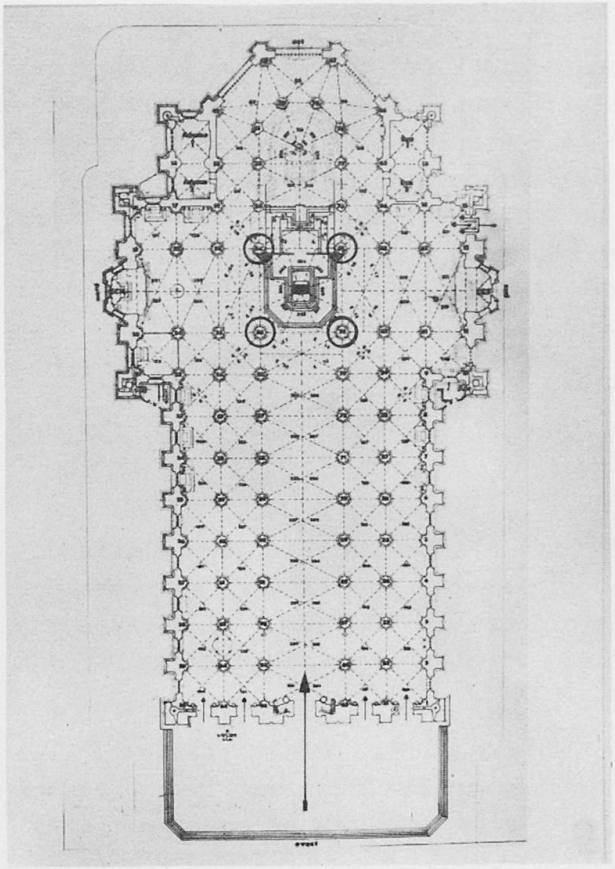


Fig. 1 Plan de la Cathédrale et indication des 4 piliers du Tiburium

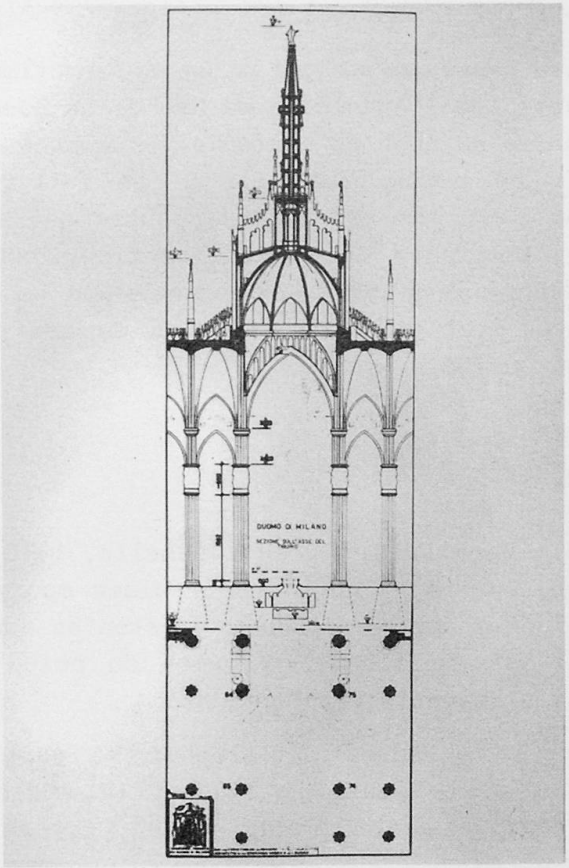


Fig. 2 Section verticale suivant le axe principal du Tiburium

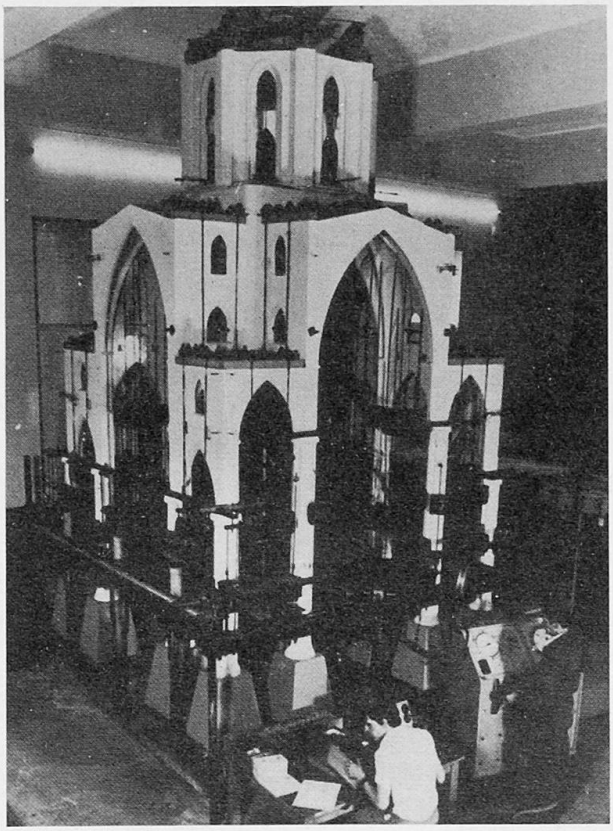


Fig.3 Le modèle élastique,à échelle 1:15, vue d'ensemble,prêt pour les essais

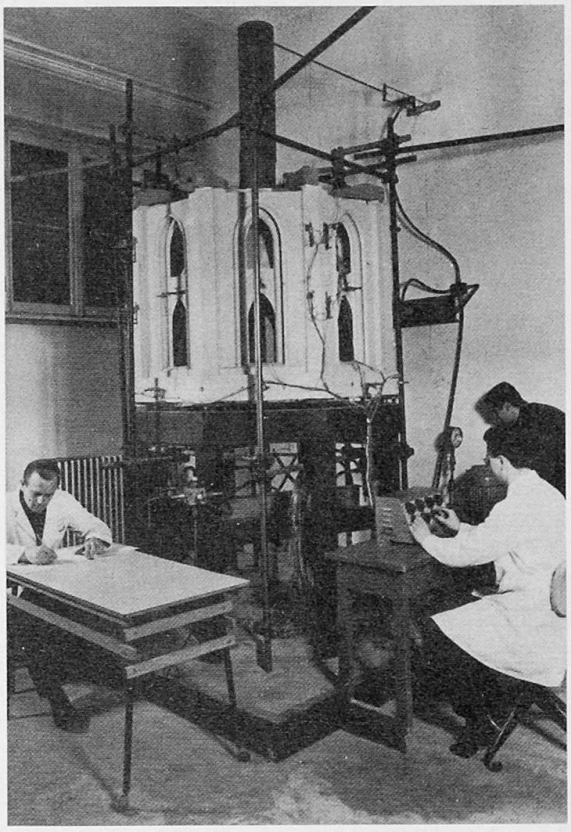


Fig.4 Le modèle élastique,partie supérieur,prêt pour les essais

par les fondations des piliers principaux est de 7,5 Kg/cm^2.

Les causes principales des déformations apparues ont été des lésions très graves dans les éléments en marbre de la couronne extérieure, ainsi qu' il a été nécessaire en 1969 de soutenir les quatre colonnes centrales avec un gainage provisoire en béton armé afin d' en éviter l' écroulement. Les causes des dégâts, on peut les reconnaître, outre que dans l' ancienneté du monument et dans les vicissitudes de la construction, dans les évidents affaissements différentiels des piliers dûs à la remarquable baisse de la nappe phréatique (25 m en vingt ans) et aux perturbations du trafic, souterrain et de surface, présent tout autour du monument.

2. LES ESSAIS SUR MODELES

a) Un modèle à grande échelle, 1:15, bâti avec différents matériaux élastiques appropriés (liés avec résines époxys) fut exécuté, chez ISMES, pour évaluer l' état des contraintes existentes dans les piliers du Tiburium, qui soutiennent la grande flèche, à cause du poids propre, à la tension des chaînes et aux affaissements des fondations.

On a reproduit entièrement la partie centrale du Dôme, c' est-à-dire: les 4 piliers centraux et les 12 adjacents soutenant le Tiburium, les 4 voûtes relatives à la grande nef et, encore, les 4 voûtes des nefs secondaires (Fig. 3 et 4).

Le poids propre, soit des structures supportées par le Tiburium et soit des autres voûtes reproduites, a été réalisé avec l' emploi de ressorts dynamométriques en "caoutchouc" (Fig. 5), ancrés, à l' aide de tirants verticaux, au modèle en haut et à une plaque horizontale en bas. Cette plaque, réalisée avec des poutres en acier assez rigide, pouvait être déplacée verticalement à l' aide de vérins hydrauliques pour donner, dans un seul coup, la charge correspondante au poids propre.

A la base de chaque pilier on a réalisé, en celluloïd, un bloc simulant les caractéristiques géométriques et de déformabilité élastique, prévues pour les fondations réelles (Fig. 6).

On a conduit des essais, avec un large emploi d' extensomètres électriques, pour evaluer les déformations et donc les contraintes, en domaine élastique, dues au poids et celles qui étaient conséquentes à la tension exercée par les quatre chaînes placées aux naissances des grands arcs en Serizzo soutenant la voûte du Tiburium (*).

En outre il a été examiné l' effet de l' affaissement des fondations, soit des piliers principaux soit de ceux des nefs latérales, sur la charge verticale supportée par chacun de ces piliers, avec l' emploi de spéciales cellules de mesure connexées à une centrale automatique pour la relévation des données.

(*) Le comportement élastique, linéaire, du modèle a été contrôlé au moyen d' essais préliminaires.

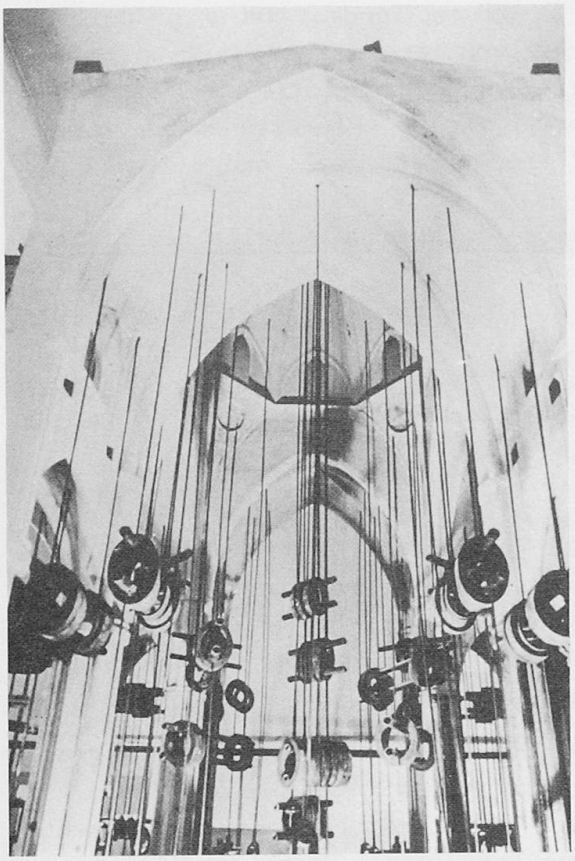


Fig. 5 Dispositif de charge (au poids propre) du modèle élastique

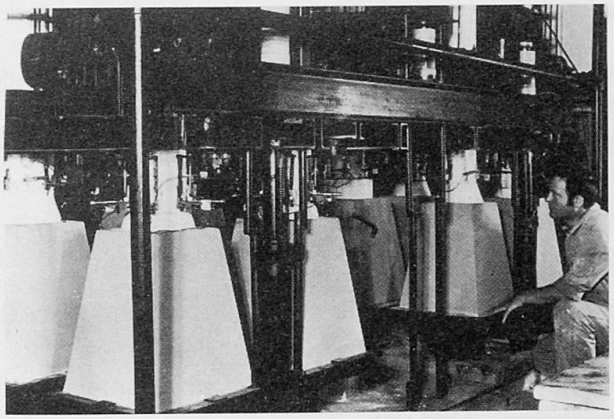


Fig. 6 Particularités des fondations du modèle

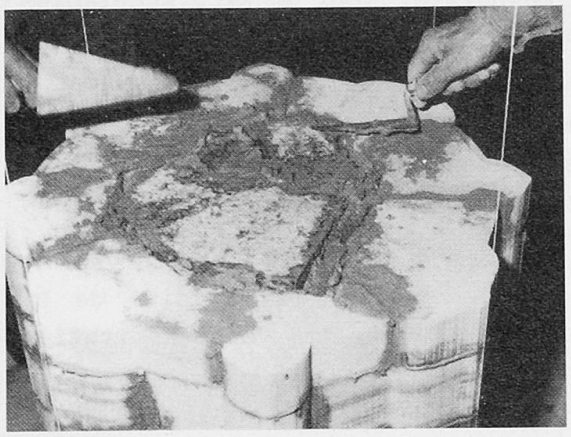


Fig. 7 Assemblage des blocs constituant les grands modèles

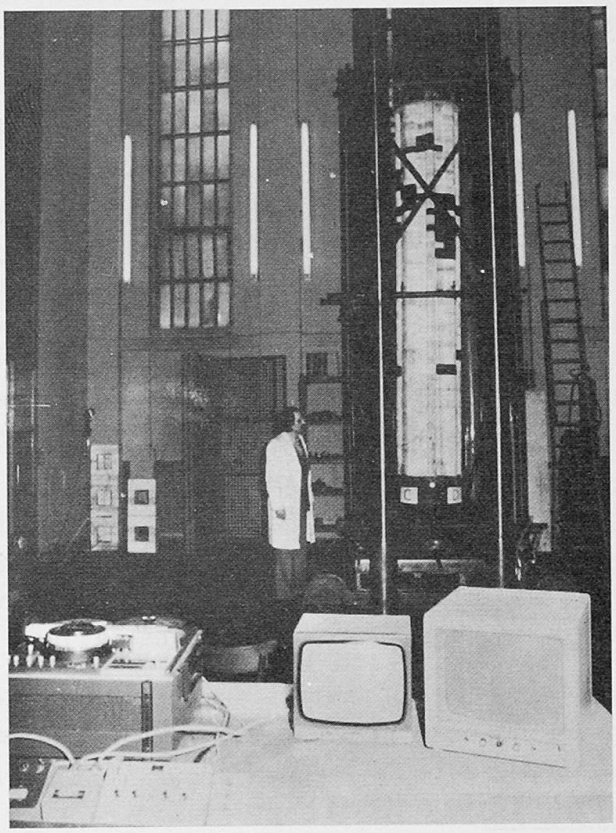


Fig. 8 Modèle d' une des colonnes sous la presse de 2.000 t

Les conditions les plus défavorables sont apparues lorsqu' on a produit le affaissement d' un des piliers principaux qui soutiennent le Tiburium. Avec un abaissement correspondant à un centimètre on a enregistré une réduction d' environ un tiers de la charge supportée par le pilier intéressé et une augmentation correspondante dans les piliers qui l' entouraient de l' ordre de 15%.

b) Les essais à rupture avec l' emploi de modèles à grande échelle des piliers principaux du Tiburium

Ces essais, qui ont pris presque dix années, ont été commis à ISMES de Bergamo par le Dr.Ing. C. Ferrari da Passano, dirigeant la Ven. Fabrique de la Cathédrale, suivant les conseils d' une Commission d' experts nommée par le Préfet de Milan.

Tous les modèles représentaient fidèlement, à grande échelle (1:4,7 environ), un des quatre principaux piliers soutenant le Tiburium du Dôme. Ces piliers, ayant la hauteur de 24 m environ, ont été bâtis par tranches horizontales avec l' assemblage soigné de blocs en marbre de Candoglia à l' extérieur et remplis au moyen d' autres blocs en Serizzo et, quelquefois, en briques ou en pierres, constitutants le noyau central moins résistant du pilier (Fig. 7).

La construction des modèles, chez ISMES, a été réalisée avec le plus grand soin, soit en ce qui concerne le choix des matériaux (marbre de Candoglia et granite de Serizzo) soit dans l' emploi des mortiers spéciaux, ciments et chaux, employés pour l' assemblage des blocs entre-eux. Comme dans la réalité, les joints horizontaux présentaient une légère rastrémation vers l' intérieur qui a eu un reflet remarquable sur l' apparition des premières fissures et des cassures à la surface extérieure des piliers.

Tous les essais ont été achevés au moyen de la grande machine verticale (presse Losenhausen de 2.000 t et de 6,5 m de portée utile) de ISMES.

Les premières expériences ont eu lieu sur un modèle de 6 mètres (Fig. 8), correspondant à la partie de la colonne qui a au réel (comprenant le chapiteau) une hauteur totale de 27 m. On a contrôlé l' effet de l' excentricité de la charge de compression axiale suivant les données à disposition (presque 10 cm au réel), et on a poussé l' essai jusqu' à la rupture finale. Avant cela on a constaté l' apparition graduelle des fissures et des détachements à la surface extérieure des blocs, en marbre de Candoglia, tout à fait semblables à ceux qui se présentaient sur les piliers du Dôme.

Après on a exécuté un grand nombre d' autres essais sur modèles de piliers, réduits à 2 mètres de hauteur, ayant surtout le but de contrôler soit l' effet (remarquable) de la composition des mortiers adoptés pour former les joints entre les blocs, soit l' efficacité des structures en acier conçues pour renforcer - en voie provisoire - les piliers endommagés.

Avec l' aide d' autres modèles on a voulu aussi vérifier la validité de la insertion de nouveaux blocs (toujours en marbre de Candoglia) en substitution des vieux plus ou moins fissurés (*), scellés avec un bon mortier en ciment.

(*) Obtenus avec surcharges préalables.

On doit remarquer que cette opération a été achevée avec le maintien de la charge correspondante au poids propre agissant sur le pilier. L' augmentation obtenue dans la valeur de la charge de rupture résulta, en général, liée au volume des blocs remplacés: avec une substitution (maximale) de l' ordre du 40% cette valeur a atteint 100% (Fig. 9).

Encore, on a essayé l' emploi d' une technique spéciale (étudiée par l' Ing. Ferrari) d' intervention globale pour la reconstruction totale des piliers centraux du Tiburium, avec le remplacement, commençant de bas en haut et par phases successives, de toutes les couches des blocs en marbre de Candoglia atteignant (à la fin) des résultats extraordinaires puisque l' on arrive à une charge de rupture 10 fois la valeur de la charge supportée normalement par les colonnes "in situ".

Enfin on a essayé la solution finale, actuellement en oeuvre, dont nous dirons après, avec l' emploi de deux modèles bâtis exactement les mêmes et soumis à essais de préfissuration simulant les conditions réelles des prototypes. Après, un des modèles a été tenu comme contrôle et l' autre, par contre, a été soumis à la substitution des blocs endommagés, toujours sous l' action de la charge normale, suivant la technique prévue par l' Ing. Ferrari. Ensuite les deux modèles ont été soumis aux essais à outrance poussés jusqu' à la rupture, et on a obtenu par comparison la valeur de la marge de sécurité qu' on pourra prévoir lorsque l' oeuvre de reconstruction sera achevée.

3. METHODE D' INTERVENTION

Le but de l' opération consiste dans le déplacement en secteurs, en commençant de haut en bas, du gainage provisoire en béton armé (Fig. 10) et, en même temps, dans la consolidation statique des piliers en remplaçant les parties de marbre lézardées de la couronne circulaire, qui représentent presque le 60% de leur développement total en surface, avec d' autres éléments nouveux, toujours en marbre, introduits de façon telle à résulter immédiatement collaborants avec la structure originale, c' est-à-dire en absence de déformations appréciables.

En 1975 il avait été projété, approuvé et expérimenté (sur le modèle à échelle 1:4,7) un système d' intervention globale, qui prévoyait la substitution totale des piliers par trois phases d' intervention successives, en ce cas réalisées en commençant de bas en haut. Les résultats atteints par ce système spécial, comme l' on a déjà dit, ont été exceptionnels, mais - à cause de différentes raisons - ce projet de remarquable intérêt, même général, n' a pas été réalisé, et l' on a préféré un autre projet (celui aussi étudié par l' Ing. Ferrari), qui propose de nouveau la méthodologie déjà adoptée pour la restauration des seize piliers du transpet, de l' abside et la grande nef. Il s' agit d' interventions partielles soit pour extension que pour profondité, plus ou moins vastes, au moyen desquelles on enlève les parties lézardées en les remplaçant (par un procédé spécial) avec d' autres éléments neufs, toujours en marbre de Candoglia, qui deviennent immédiatement éléments portants.

La différence opérationelle des deux projets est représentée par le différent sens de l' exécution; de bas en haut dans le premier cas, s' agissant de substi_

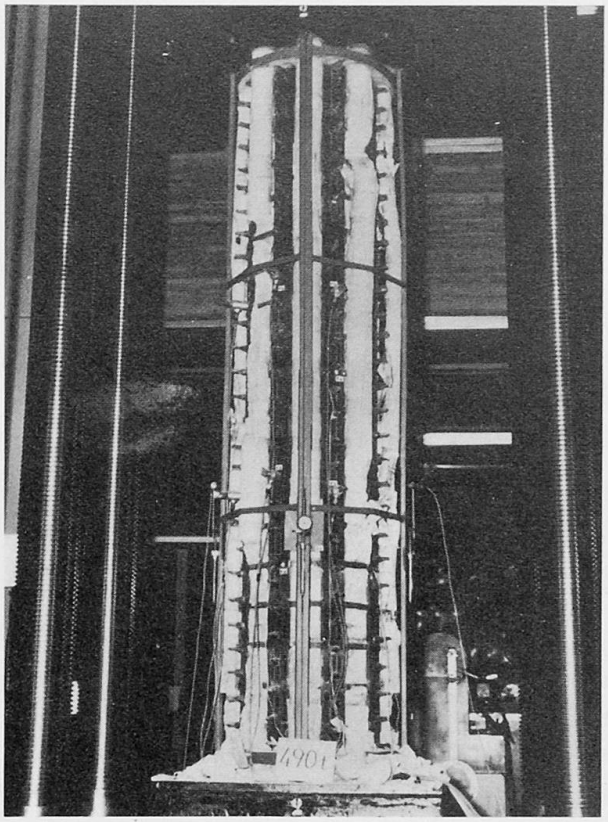


Fig. 9 Modèle de hauteur réduite (3 m) après la rupture. On voit l'appareillage de mesure.

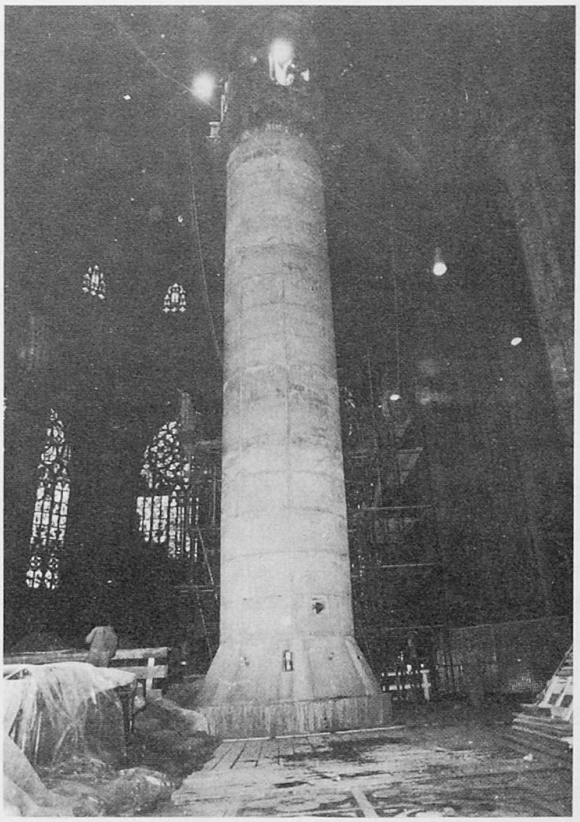


Fig. 10 Gainage provisoire en béton armé appliqué à un des piliers.

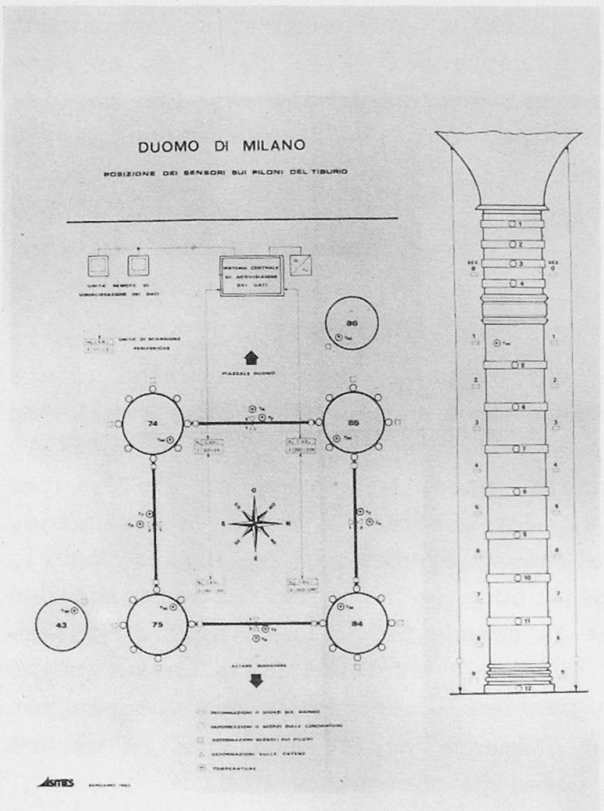


Fig. 11 Plan des instruments électriques appliqués à une des colonnes

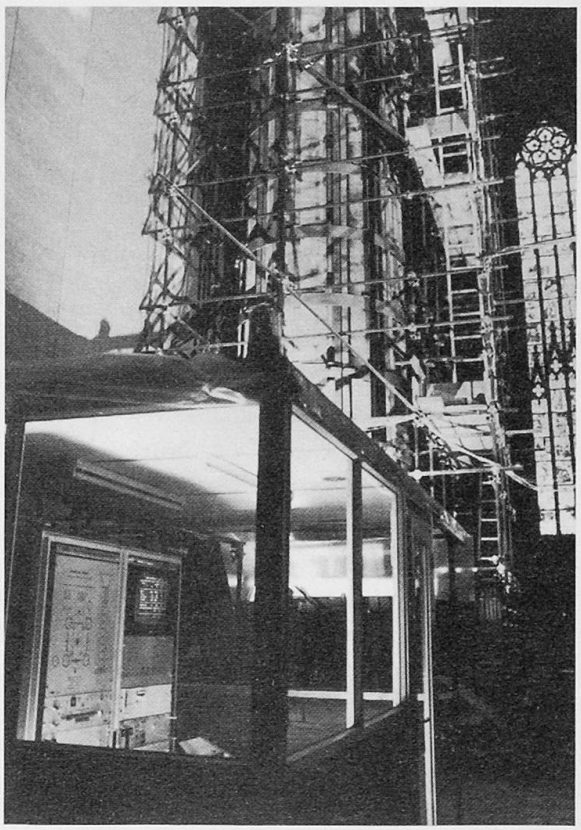


Fig. 12 Centrale automatique pour le contrôle continu et vision partielle du chantier

tution totale, exécutée par éléments verticaux; de haut en bas, dans le cas présent, s' agissant d' une intervention partiale, même si ça intéresse outre le 60% du développement total des piliers; en outre le gainage en béton armé, pour être actif comme étayage jusqu' à la fin des travaux, doit être enlevé par secteurs, en commençant de la partie supérieure du pilier. Pour cela la intervention de restauration statique prévoit, dans le cas en objet, un cerclage préliminaire métallique de sécurité autour du gainage, déjà en place, de façon d' en substituer sa fonction dès qu' il est compromis par les tailles, qui sont exécutées par éléments de m 2,50, à peu près, de hauteur et de m 0,50 de largeur.

Une fois enlevé par secteurs le gainage en béton armé, l' on fait la taille à sec par des courounnes "vidiam" refroidies par de l' air comprimé et, au même temps, par l' aspiration de la poussière des blocs de marbre lézardés et des parties internes composées, ainsi que l' on a déjà dit, de matériaux hétérogènes.

Ayant donc rectifié les plans soit horizontaux que verticaux des cavités ainsi obtenues, on applique une couche continue de deux millimètres de mortier de béton mélangé avec de la poudre de quartz et silice et l' on introduit dans les dîtes cavités, ainsi réalisées dans les piliers, des nouveux éléments de marbre façonnés avec des logements soit radiaux que circulaires, par un forçage sur le dernier de ces nouveux blocs de marbre, qui en pratique résultent dédoublés et taillés à coin (dans les deux parties) de façon à rendre le nouveau complexe structure portante ainsi que la structure originale, avec l' avantage de diminuer notamment les affaissements.

Pour le contrôle opératif pendant la restauration statique on a fait les suivantes mesures (Fig. 11):

a) variation de la longueur globale de chaque pilier, du chapiteau au plinthe, par l' emploi d' un extensomètre micrométrique et par un extensomètre à variation d' inductance à lecture continue, centralisée et enregistrée;

b) déformations locales dans les zones d' intervention par l' emploi d' un calibre extensométrique;

c) déformations locales (et contraintes) sur le marbre, cerclages en fer, variations de longueur des chaînes et de la température (de l' air, du marbre et du fer) avec l' emploi des mesures continues centralisées (Fig. 12).

Les données obtenues jusqu' à présent ont été analysées avec soin et on a constaté la grande influence des effets thermiques sur les déformations locales, avec des écarts notables sur les contraintes prévues. Par contre les fléchissements et le affaissements ont été assez réduits et ils ont confirmé la validité du procédé appliqué.

4. CONCLUSIONS

Suivant le but de ce Colloque, les Auteurs ont essayé de donner aux Partici-

pants, intéressés aux aspects techniques de la restauration des grands monuments historiques, un aperçu de ce qu' on a exécuté pour sauver le Dôme de Milan, donnant en particulier quelques données sur l' application des modèles physiques et sur les méthodes poursuivies pour le renforcement des piliers soutenant le Tiburium du Dôme. Enfin on a brièvement décrit les buts des installations de contrôle employées pendant l' exécution des travaux, encore en cours et qu' on compte d' achever d' ici en 1985.

Post-Seismic Strengthening of Churches

Renforcement d'églises après des séismes

Verstärkung von Kirchen nach Erdbebenbeanspruchung

Ramiro SOFRONIE
Doctor in Civil Engineering
Bucharest, Romania



Ramiro Sofronie, born 1936, graduated as civil engineer in 1958, and received his doctoral degree in 1970. Professional activities: 3 years on construction sites, 10 years in design and 12 years teaching Strength of Materials and Theory of Structures at the Faculty. Researches in suspension bridges and tall buildings.

SUMMARY

The paper refers to churches of three-cusped plane wide-spread in the Carpatho-Danubian-Pontic area, where tectonic earthquakes frequently occur. Firstly it is shown that centuries ago two antiseismic shapes were conceived, one in Moldavia by means of buttresses and another in Wallachia by overwidening the ante-naves. As a diagnosis method it is suggested that the results of lateral bending and general torsion analyses be combined with the map of seismic damages. As therapy for the damaged churches, some strengthening techniques with R.C. coatings, belts and/or buttresses are recommended.

RESUME

L'article se réfère aux églises au plan trilobé largement répandues dans la région des Carpates et du Danube où ont lieu de fréquents tremblements de terre. Aux siècles passés deux configurations antisismiques étaient conçues, l'une en Moldavie avec des contreforts et l'autre en Wallachie surélargissant les pronaos. Pour le diagnostic, il est suggéré de combiner les résultats d'analyse de la flexion latérale et torsion générale avec la carte des dommages sismiques. Des techniques de renforcement avec des couvertures, ceintures et/ou contreforts de béton armé sont proposées pour la réparation de ces églises.

ZUSAMMENFASSUNG

Der Bericht bezieht sich auf Kirchen mit Dreispitz-Grundriss die im Karpaten-Donau-Gebiet, wo häufig tektonische Erdbeben auftreten, weit verbreitet sind. Zuerst wird gezeigt, dass vor Jahrhunderten zwei antiseismische Formen ausgedacht wurden: eine mit Strebenpfeilern in der Moldau und eine mit einer Vorschifferweiterung in der Wallachei. Als Diagnosemethode wird vorgeschlagen die seitliche Biegebeanspruchung und die allgemeine Torsionsbeanspruchung mit der Karte der seismischen Schäden zu verbinden. Für die beschädigten Kirchen werden Verstärkungen mit Stahlbetonverkleidungen, Gurten und/oder Strebenpfeilern empfohlen.

INTRODUCTION

The oldest monumental buildings preserved in the Carpatho-Danubian-Pontic area are churches. For centuries they were the most representative creations of ecclesiastic and monumental architecture. Erected with stone or brick masonry these Eastern Churches of Balkan-Byzantine style were always an evidence of the level of technical knowledge and artistic refinement reached during their epoch. They also reflect the foreign influences on the autochthonous art of building.

Unfortunately, strong tectonic earthquakes frequently occur in this area. The main focus being located in the Carpathian curvature and at a depth of about 150 km, it influences the whole area. The Eastern churches of three-cusped plane seeming to show an intrinsic sensitivity to earthquake actions. In the course of time some of them were completely destroyed. Others survived being, however, more or less damaged. Often by strengthening parts of the original work were altered or even definitely sacrificed.

The aim of this paper is to show that history's lesson was, however, useful. The structural systems were gradually adapted to the local seismicity of the ground and even some unwritten safety rules were conceived. Further is shown how a simple technique of analysis can be used as a diagnosis tool of the damages caused by earthquakes. Finally some strengthening techniques are presented.

1. STRUCTURAL SYSTEMS

Centuries ago, for strategical reasons, human settlements were located in hilly areas. Among the first Eastern churches of those early times are the Trinity Church, without any steeple, of Siret in Moldavia, built in 1376 (Fig.1) and the church, with a steeple devoted to Pantokrator, of Cozia Monastery - Wallachia, built in 1388 (Fig.2). Both are well preserved until today as at their emplacement the ground seismicity is low.

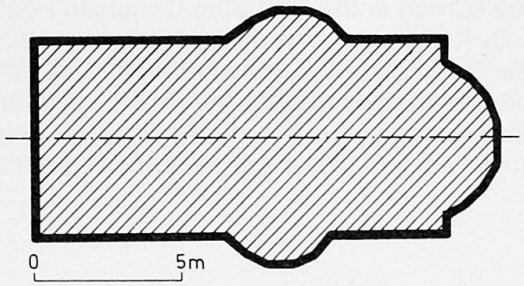


FIG. 1 SIRET 1376

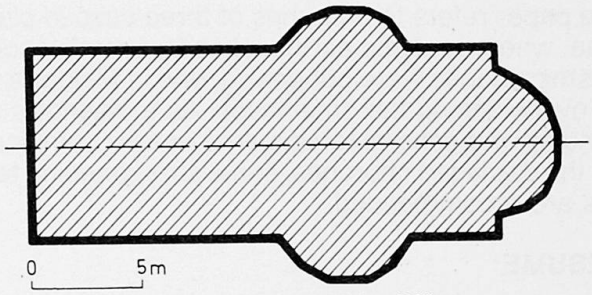


FIG. 2 COZIA 1388

Later, when the danger of invaders decreased and the settlements gradually enlarged, they were moved towards the plains where by agricultural means the increased needs of food were easier satisfied. Naturally, in this process the shape and size of Eastern churches knew some changements. They were developed on larger areas, increased in height and enriched by steeples and ornamentations. But simultaneously, due to the higher ground seismicity the danger of damages became imminent.

The three-cusped plane was also endowed with lateral buttresses. The first such shaped church was St. Procopie Church at Milişăuţi, built in 1487. Beginning with that year all Moldavian churches were provided with lateral buttresses. Originally they had a prevailingly ornamental function and were probably adopted under the influence of Transylvania's Romanesque and Gothic architecture. Generally, lateral

buttresses were not connected to the lateral strengthening parts of the church-body, but contributed to some extent to discharge the heavy masonry walls into the foundation ground. However, in Wallachia lateral buttresses were seldom adopted.

Alternatively, the three-cusped plane was developed by a slight widening of the ante-nave. This structural measure was first adopted to the so-called town churches, built in Moldavia between 1490 and 1496. The aim of this widening was apparently to enlarge the inner space of the ante-nave for the believers. But along the longitudinal walls of the widened ante-naves lateral buttresses were omitted, which means that it was considered their function was taken over by the lateral walls appearing as a consequence of that widening. Instead, lateral buttresses were maintained in front of the altar apses of the same churches.

Finally, beginning with the sixteenth century the churches of three-cusped plane are endowed with pairs or even several steeples. The main steeple is further devoted to Pantokrator while one of the additional steeples got a belfry function, being also used, as observation point against fires or invaders. It should, however, be mentioned that the steeples of Moldavian Churches are essentially different from the Wallachian ones. In Moldavia, under Gothic influence, steeples were disposed in a single line, following the church axis, being conceived like castle towers with few and narrow openings as windows. On the contrary in Wallachia, under Byzantine influence, steeples were arranged in both directions of the church, being erected with wider openings as windows and slender columns [1].

2. SEISMIC DAMAGES

According to the first written documents, between 1471 and 1846, on the Carpatho-Danubian-Pontic territory over 60 earthquakes were felt, which means more than 16 in one century. Fortunately, only a few were violent as those of Nov. 8th, 1620, June 11th, 1738 and Nov. 26th, 1802. During our century earthquakes were recorded in 1903, 1912, 1929 and 1938, but the strongest occurred on Nov. 10th, 1940 and March 4th, 1977 [2].

As concerns the damages caused by earthquakes, first there should be mentioned the steeples. As a rule the masonry columns of the steeples are horizontally sheared at their bottom and top. The steeples of Wallachian churches yielded easier to shearing forces than those of Moldavian churches. Consequently, now, in Bucharest one church out of three has false, wooden steeples.

Apse walls of the nave and altar were also severely damaged. The typical damages consist in vertical cracks when these curved or polygonal walls are completely closed, and in 45° inclined cracks when there are e.g. openings for windows. The same two types of cracks have been developed in the straight walls of the ante-naves, especially when they were not braced at their tops.

The semi-circular arches as integral parts of the surrounding walls, designed to narrow the vaulted space and to support the cupola or steeples have also been damaged by earthquakes. Generally, the cracks appear at the arch crown as well as at the quarter of the free spans. Such damages are often caused by the ties mounted too eccentrically. A faulty foundation also allows damages to the apse walls and transverse arches. This is the particular case of churches rebuilt in a new masonry style over ancient foundations of wooden churches burnt or stone churches destroyed by earthquakes [3].

There were, however, churches of three-cusped plane which lasted for centuries without to be damaged by earthquakes. It has been observed that in certain rather restricted areas churches of about the same size, being erected in the same period and with a comparable kind of foundations and brick-work behaved quite differently. Some of them were dramatically damaged while others remained as if untouched. The explanation of this different behaviour seems to be in the variety of the adopted shapes. Therefore, it could be assumed that since long ago certain antiseismic shapes were more or less consciously searched for. But no written rules or documents were preserved so far. They remained as professional secrets. Instead, an old legend about Wallachian church builders is going on.

3. MASTER MANOLE'S LEGEND

In the former Wallachian capital of Curtea de Argesh, the Prince Neagoe Basarab (1512-1521) founded the church of Argesh Monastery. This church was consecrated on August 15th, 1517 and became immediately famous due to its outstanding architecture. According to some documents this church replaced the ancient metropolitan church of three-cusped plane, like Cozia church, founded by Dracula's father. It seems that a good part of the old foundations were preserved, namely that of the altar apse and that of the ante-nave walls sustaining now the twelve stone columns. The church body and its four steeples are entirely achieved of stone masonry, and the monument has no precedent as architectural conception and development in plane and space. That is why a legend has been embroidered around this church.

According to the legend Voivode Negru entrusted a team of ten masons under the leadership of Master Manole to implement somewhere on the Argesh River, on the site of an ancient church "broken down and unstrengthened", an unequaled church. They started to work carefully, but beginning with a certain instant the walls they erected during the day tumbled down during the night. The accidents recurred till they decided to immolate Master Manole's wife. Only thus the walls were stopped to fall, and the church could be finished, lasting over centuries [4].

Viewing Master Manole's legend from the angle of construction art it is easy to conclude that of all possible actions, able to cause the repeated collapse of the masonry walls, there could be nothing else than earthquakes. Indeed, tectonic earthquakes, typical to the Carpatho-Danubian area, occur frequently during the night, are recurring and present a particular aggression against curved walls and arches or vaults of masonry. Otherwise, it is very likely that the ancient metropolitan church, mentioned in the legend, has also been destroyed by earthquakes. As concerns the immolation consequence, it consists in the overwidening of the ante-nave and thus the church has been anti-seismically shaped. It was an inspired solution because for 466 years, from 1517 to 1983 the church behaved perfectly in earthquakes (Fig. 3).

The three-cusped plane with an enlarged ante-nave of the Argesh Monastery Church became a model for the following Wallachian churches. Indeed, all churches which preserved rigurously the shape of this model behaved excellently in earthquakes, although some of them, namely those in Bucharest, were located on a ground of a higher seismicity degree. Thus the anti-seismic qualities of this shape were definitely proved.

Master Manole's legend carried as far as Moldavia the fame of the Argesh Monastery Church. However, that shape was not at all adopted there, although the ground

seismicity is the same. Moldavian ecclesiastic architecture followed its traditional style of three-cusped plane with buttresses of Gothic influence. As anti-seismic protection the church bodies were somewhat longitudinally extended while the lateral development of nave apses was slightly reduced. In 1639, on this principle was built the famous Church of Three Hierarchs Monastery founded in Jassy by Prince Vasile Lupu (Fig. 4). A faithful copy of this church was immediately tested under more severe seismic conditions on the Wallachian ground and it behaved splendidly. Therefore, with the aid of lateral buttresses an alternate anti-seismic shape of three-cusped plane was achieved.

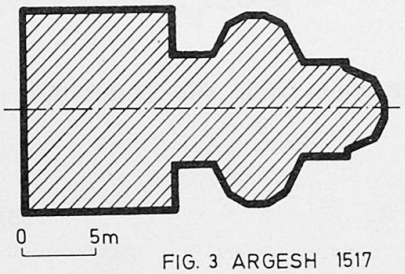


FIG. 3 ARGESH 1517

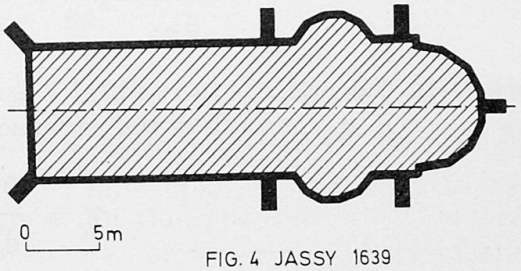


FIG. 4 JASSY 1639

4. DIAGNOSIS OF SEISMIC DAMAGES

The resultant of inertial forces developed by the church body during an earthquake is a horizontal force S proportional with the church weight W and it is applied at the centre of mass C.M. (Fig. 5). In Bucharest for example.

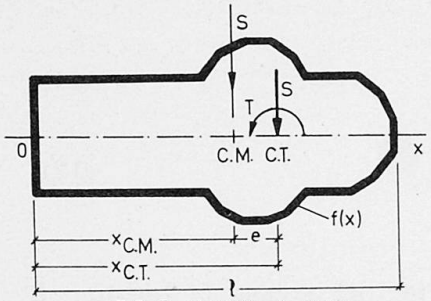


FIG. 5 SEISMIC FORCES

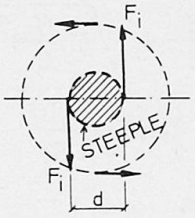


FIG. 7

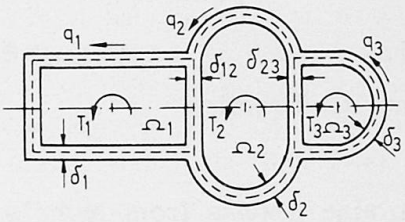


FIG. 6 TORSION MODEL

$$S = 0.09W. \tag{1}$$

By reducing this force to the centre of twist C.T. one obtains the torque

$$T = Se, \tag{2}$$

where

$$e = \frac{\int_0^{\ell} f^3 x\,dx}{\int_0^{\ell} f^3\,dx} - \frac{\int_0^{\ell} fx\,dx}{\int_0^{\ell} f\,dx}. \tag{3}$$

Therefore during earthquakes a church is subjected to a phenomenon of general torsion and to another of lateral bending. Generally, when $e = T/S > 0.10\ell$ prevails torsion, and when $e < 0.01\ell$ prevails bending.

Stresses in torsion could be determined assuming the horizontal section of the church as a hollowed section (Fig. 6). Then, by virtute of Bredt's theory

$$2q_1\Omega_1 + 2q_2\Omega_2 + 3q_3\Omega_3 = T, \tag{4}$$

and

$$\frac{q_1}{\delta_1} s_1 + \frac{q_1 - q_2}{\delta_{12}} s_{12} = 2G\Theta\Omega_1, \tag{5}$$

$$-\frac{q_1 - q_2}{\delta_{12}} s_{12} + \frac{q_2}{\delta_2} s_2 + \frac{q_2 - q_3}{\delta_{23}} s_{23} = 2G\Theta\Omega_2, \tag{6}$$

$$-\frac{q_2 - q_3}{\delta_{23}} s_{23} + \frac{q_3}{\delta_3} s_3 = 2G\Theta\Omega_3, \tag{7}$$

where Θ is the specific torsion. By solving this system one finds the shearing flows q_1, q_2, q_3, and then the stresses

$$\tau_i = \frac{q_i}{\delta_i}, \quad i = 1, 2, 3. \tag{8}$$

Stresses in bending could be determined by Navier's theory

$$\sigma = \frac{M}{I_x} y \tag{9}$$

and that of Jourawski

$$\tau = \frac{S}{\delta I_x} \int_A y dA. \tag{10}$$

Further one finds the principal stress of tension

$$\sigma_1 = \frac{\sigma}{2} + \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2} \tag{11}$$

and its direction

$$\tan\alpha = \frac{\tau}{\sigma_1} \tag{12}$$

as also results from Mohr's circle.

It is possible now to construct the trajectories of the principal stress of tension σ_1, once separately for torsion and bending and then superposed. By comparing these trajectories with the map of existing cracks one can establish with a satisfactory accuracy which of the phenomena : general torsion, lateral bending or both are responsible for seismic damages.

Finally, the shearing stresses in steeples will be

$$\tau = 1.4 \frac{S_i + F_i}{nA}, \tag{13}$$

where S_i is the shearing force given by steeple bending and F_i introduces the influence of general torsion of the church being, cf. Fig. 7,

$$F_i = \frac{T}{d}, \tag{14}$$

while n is the number of columns, A their cross-section and 1.4 a nonuniformity coefficient. The value thus obtained is compared with the masonry shearing strength usually of 0.10 to 0.30 N/mm^2.

As concerns the two anti-seismic shapes discussed above, now it is obvious that in the case of the Wallachian solution (Fig. 3) by overwidening the ante-nave increased

the torsional rigidity of the church, while in the case of the Moldavian solution (Fig.4), by extending the church length the torsion vanished, and the lateral bending was prevented by transverse walls in combination with buttresses.

5. THERAPY OF SEISMIC DAMAGES

As long as the diagnosis of damages in proved to be true any strengthening, local or of the whole church, should satisfy some rules as follows :

- To preserve as much as possible the original shape, architectural ornaments, as paintings, as well as the initial structural system and its mechanism of discharging.

- To put in value the existing strength reserves of masonry walls, columns, arches or vaults, as well as of the connections between them and the foundation ground.

- To secure an intimate and permanent co-operation between the old material and that added for strengthening.

- To restore the continuity of all sections in such a way that the flow of stresses be everywhere as uniform as possible.

- By strengthening to provide the prescribed anti-seismic protection so that the church should never be damaged again.

There are no repairing and strengthening solutions equally valid for all churches of three-cusped plane. The basic principles established for masonry buildings should be applied from case to case according to the specific shape and size of each church. However, as long as only R.C. is used as a strengthening material, there are three strengthening techniques.

One of them, and perhaps the best, consists in covering the damaged surfaces with reinforced coats of mortar cement. Certainly, the coatings are carried out only on the outer sides of the walls. Such coatings can be also applied on steeple columns between window recesses. The best solution is to achieve continuous and closed reinforced coats. In front of the inlet corners and architectural ornaments the coats should be framed by R.C. cores well clamped in the masonry walls.

Another technique consists in bracing the church body with R.C. belts clamped at the upper part of masonry walls. The bracing will be carried out only at the outer and convex sides of the walls. In addition, the bracing belts should be closed around the church body. For a three-cusped plane that means to carry out two R.C. belts intersecting in four points. If the inner structure of the church does not allow such a development at the four inlet corners, R.C. cores should be applied. The outer belts could also be combined with inner tie-beams of the arches or vaults. Yet, it has to be mentioned that tie-beams are efficient only if they are perfectly centred as to the vertical symmetry planes of the arches or vaults.

Finally, the mechanism of discharging, as well as the interaction between foundation and ground could be improved by R.C. buttresses. Some corrections of the development in plane of foundations can also be achieved on this occasion. However, the solution assumes first to ensure a good connection with the strengthening elements of the church body and secondly an intimate contact with the foundation ground. In addition it must be remembered that the function of buttresses remains limited to lateral, they do not help against torsion. All these strengthening techniques were applied in Romania after the strong earthquake occurred on March 4th, 1977. They proved able to satisfy the five above mentioned basic rules, being not too difficult

as manual labour and rather convenient as time and money [5].

CONCLUSIONS

1. During earthquakes Eastern churches are subjected to shearing phenomena. The shearing is caused by lateral bending and general torsion. Therefore the behaviour of these churches in earthquakes depends only on their shape. It is a matter of geometry before to be one of strength.

2. Since beginning to use masonry, the builders were concerned with the antiseismic shaping of Eastern churches. Fortunately, about five centuries ago there were found two such shapes. One in Moldavia, by means of buttresses and the other in Wallachia by overwidening the ante-naves. Both anti-seismic shapes were proved in the course of time by facing strong earthquakes without being damaged.

3. The legend of Master Manole could be looked at as an ancient technical code. Indeed, it expresses in a stylized form, first the danger of earthquakes for the most representative buildings of that epoch, the churches of three-cusped plane, and then it suggests the existence of an anti-seismic solution. The solution consists in an appropriate shaping and is unique. As a matter of fact, similar legends can be found in all seismic areas around the world.

4. The diagnosis of damages in monumental works as Eastern churches is a difficult and responsible task. Such buildings cannot be tested on reduced scale models, nor on a natural scale. Also the analysis models are approximate enough. Therefore, on only combined methods like that of finite-element used by ISMES-Italy or the one suggested in this paper could be more helpful for a correct diagnosis.

5. The therapy by the three techniques of strengthening does not eliminate the intrinsic sensitivity to earthquakes of Eastern churches. The single radical solution would be to change their shape, which practically is not possible. As concerns the new materials, different from concrete, produced by modern technology, they should be very judiciously used and only after a thorough checking up. Otherwise, irreparable losses could result. In fact, the therapy technique must be adopted on the basis of a very serious, competitive examination.

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Bucharest, March 15th, 1983

Dommages séismiques et réparation de la cathédrale de Guatémala

Der Dom zu Guatemala - Erdbebenbeschädigung und Instandsetzung

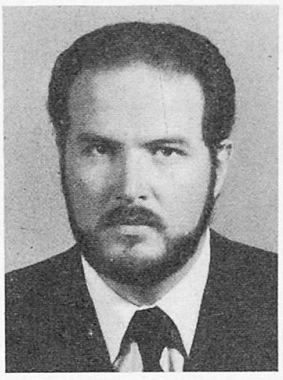
Cathedral of Guatemala - Seismic Damage and Restoration

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RESUME

Endommagée par le violent séisme de 1976, la Cathédrale Métropolitaine de Guatémala vient d'être réparée. Datant de l'époque coloniale, maintes fois frappée par des secousses mais toujours remise en état, la cathédrale comportait des reconstructions en béton armé du début du siècle. En 1976, ces masses de béton ont bien résisté mais ont endommagé le reste et ont gêné les travaux. L'article décrit la structure, les dommages subis et les réparations qui ont fait appel à des techniques anciennes et modernes. La précontrainte a été largement employée.

ZUSAMMENFASSUNG

Der metropolitanische Dom zu Guatemala ist jetzt nach dem schweren Erdbeben von 1976 repariert worden. Aus der Kolonialzeit stammend, mehrmals durch Erdbeben beschädigt, jedoch immer wieder instandgesetzt, enthält der Dom gewisse Stahlbetonrekonstruktionen aus dem Anfang dieses Jahrhunderts. 1976 haben sich diese Bauteile gut bewährt, den restlichen Teil des Domes jedoch beschädigt und die Reparaturarbeiten erschwert. Der Artikel beschreibt die Baukonstruktion, die Beschädigungen und die Instandsetzung für die sowohl alte als auch moderne Baumetoden benötigt wurden. Es ist weitgehend Vorspannung verwendet worden.

SUMMARY

The Metropolitan Cathedral of Guatemala which was damaged by the violent 1976 earthquake, has now been repaired. The Cathedral which dates from the colonial period, and which has been hit several times by earthquakes but each time restored, comprised certain reconstructions in reinforced concrete dating from the beginning of this century. In 1976 the concrete parts resisted well but caused damage to the rest and hampered the works. The article describes the structure, the damage and the repair works which have required both ancient and modern techniques. Prestressing has been applied extensively.

1. INTRODUCTION

La Cathédrale Métropolitaine de Guatemala a été fortement endommagée par le violent séisme qui frappa le pays le 4 février 1976. Dans la ville de Guatemala l'intensité était de 6 à 7 selon l'échelle de Richter.[1]

L'histoire de la cathédrale remonte à deux siècles. En 1773 ANTIGUA GUATEMALA, capitale de la Capitanie d'Amérique Centrale, fut détruite par tremblement de terre et la capitale fut transférée à quelques 60 km jusqu'à la vallée de NUEVA GUATEMALA, réputée moins assujettie aux séismes, d'où nécessité d'une nouvelle cathédrale. Un projet en style néoclassique fut établi par MARCOS IBANEZ et approuvé par le roi d'Espagne en 1776. La construction commença vers 1783, elle était juste terminée quand, en 1821, le Guatemala accéda à l'indépendance.

Le projet initial était assez bien adapté aux conditions séismiques du pays prévoyant une construction compacte avec des tours trapues, mais les plans furent modifiés en cours de construction pour donner à la nouvelle cathédrale un aspect baroque, modifiant hauteurs et volumes au détriment de la tenue aux séismes.[2]

Depuis sa construction, la cathédrale a subi plusieurs tremblements, à peu près un tous les soixante ans. Les destructions ont souvent été importantes, mais jamais totales et après chaque cataclysme la cathédrale a été remise en état en la modifiant le moins possible.

2. DESCRIPTION DE LA STRUCTURE

La cathédrale se compose de trois nefs longitudinales bordées de chapelles latérales et d'un transept qui se termine en chapelles principales. La croisée entre la nef principale et le transept est surmontée d'un dôme.

La façade principale orientée vers l'Ouest comprend un portique servant d'entrée principale, un fronton, deux tours latérales et murs d'union.

La cathédrale est sise sur un terrain qui descend d'Ouest en Est, ce qui a permis l'établissement d'une crypte sous la moitié postérieure du bâtiment, avec une entrée directe de la rue.



Fig. 1 Cathédrale. Vue d'ensemble 1980.

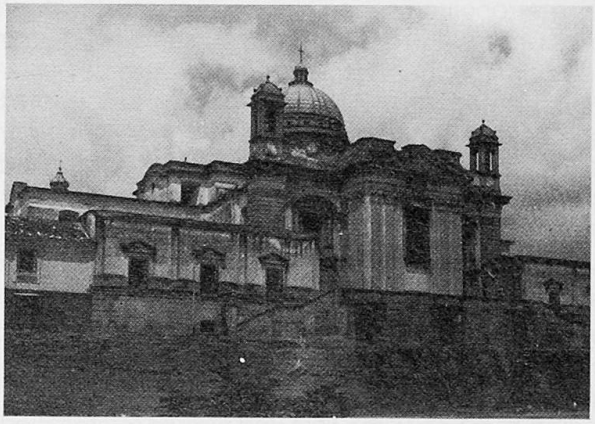


Fig. 2 Façade arrière 1978.

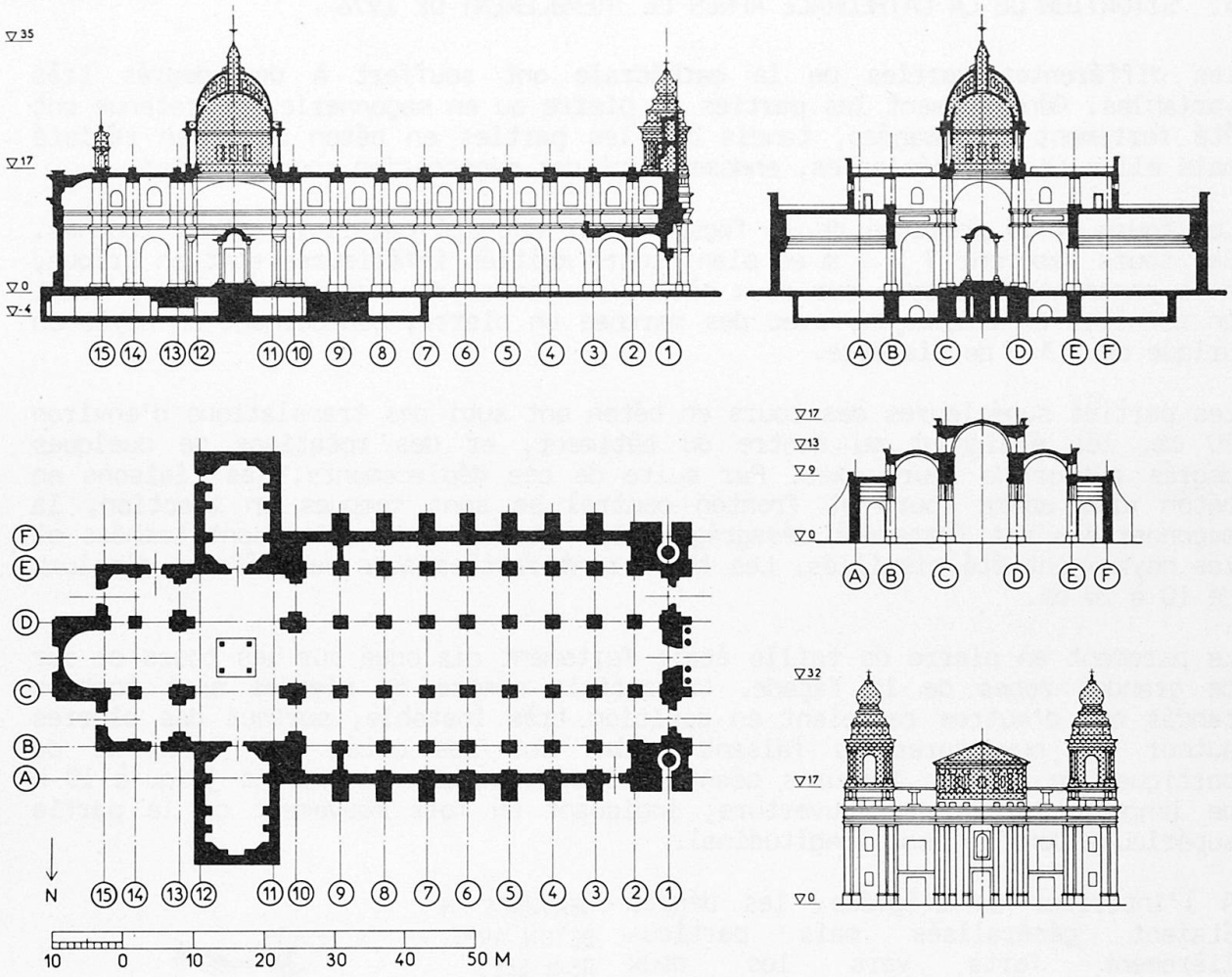


Fig. 3 Cathédrale Métropolitaine de Guatémala. Plan, sections et façade.

A juger par des puits qui ont été ouverts pendant la campagne de réparation, les murs et piliers reposent sur des semelles de quelques mètres d'épaisseur, constitués par des murs en retour, en pierre ou en brique, avec remplissage de pierres ou débris. Le terrain de fondation n'a pas donné lieu à inquiétude.

Les éléments constitutifs du bâtiment sont essentiellement des murs et piliers, arches et voutes en maçonnerie de brique, recouverts, selon les endroits, par des parements en pierre ou par des enduits de mortier ou de platre.

La cathédrale comprend certaines parties en béton armé qui reposent sur les maçonneries et qui ont été réalisées en remplacement d'autres effondrées sous le séisme de 1917-18. Il s'agit notamment des moitiés supérieures des grandes tours et du fronton de la façade principale, du dôme y compris ses murs et colonnade porteurs, des petits clochers sur la façade arrière et de certaines voutes derrière la façade et près de la coupole. Il existe également un réseau de nervures en béton armé, placées sur les toits ou incrustées dans les murs. Ces nervures renforcent les nefs longitudinales et transversale et relient les tours à l'ensemble.

La cathédrale est entourée d'une série de bâtiments de faible hauteur abritant archevêché, sacristies, écoles etc, dont certains sont contigus à la cathédrale.

3. SITUATION DE LA CATHEDRALE APRES LE TREMBLEMENT DE 1976

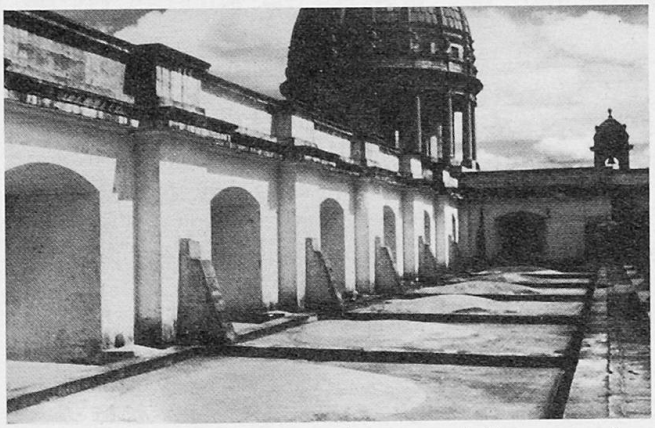
Les différentes parties de la cathédrale ont souffert à des degrés très variables. Généralement les parties en pierre ou en maçonnerie non retenue ont été fortement endommagées, tandis que les parties en béton ont bien résisté mais elles se sont déplacées, endommageant des maçonneries sous-jacentes.

Les tours et le portique de la façade Ouest ont été particulièrement touchés. Ces tours mesurent 7 x 7 m en plan, leurs moitiés inférieures sont en brique, avec parement de pierre sur deux côtés. Au centre de chaque tour, il y avait un escalier en colimaçon, avec des marches en pierre, contournant un noyau en brique de 2.3 m de diamètre.

Les parties supérieures des tours en béton ont subi des translations d'environ 20 cm, les éloignant du centre du bâtiment, et des rotations de quelques degrés autour de leurs axes. Par suite de ces déplacements, les liaisons en béton armé entre tours et fronton central se sont rompues en traction, la maçonnerie s'est fortement désagrégée, les marches d'escalier sont tombées et les noyaux ont été cisaillés. Les tours sont restées avec des défauts d'aplomb de 10 à 20 cm.

Le parement en pierre de taille était fortement disloqué sur les tours et sur de grandes zones de la façade. Un certain nombre de pierres sont tombées tandis que d'autres restaient en position très instable, surtout des pierres autour des ouvertures ou faisant angle. Sur les côtés des tours et du portique, de grandes fissures béantes se sont ouvertes mesurant jusqu'à 10 m de longueur et 15 cm d'ouverture, indiquant un fort mouvement de la partie supérieure dans le sens longitudinal.

A l'intérieur de l'église, les dégats étaient généralisés mais particulièrement forts vers les deux extrémités. Il y avait des fissures longitudinales dans les voutes, des fissures radiales en arches, des fissures transversales tendant à séparer les extrémités du reste du bâtiment, des fissures circulaires autour des petites coupoles et des fissures en X sur les façades des piliers séparant les nefs.



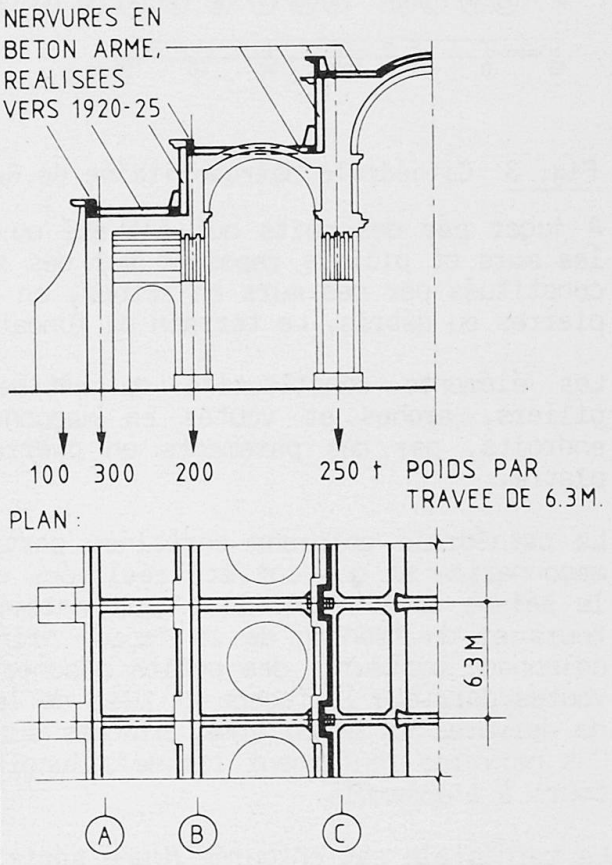


Fig. 4-5 Renforts exist. en béton armé. Charges verticales d'une travée.

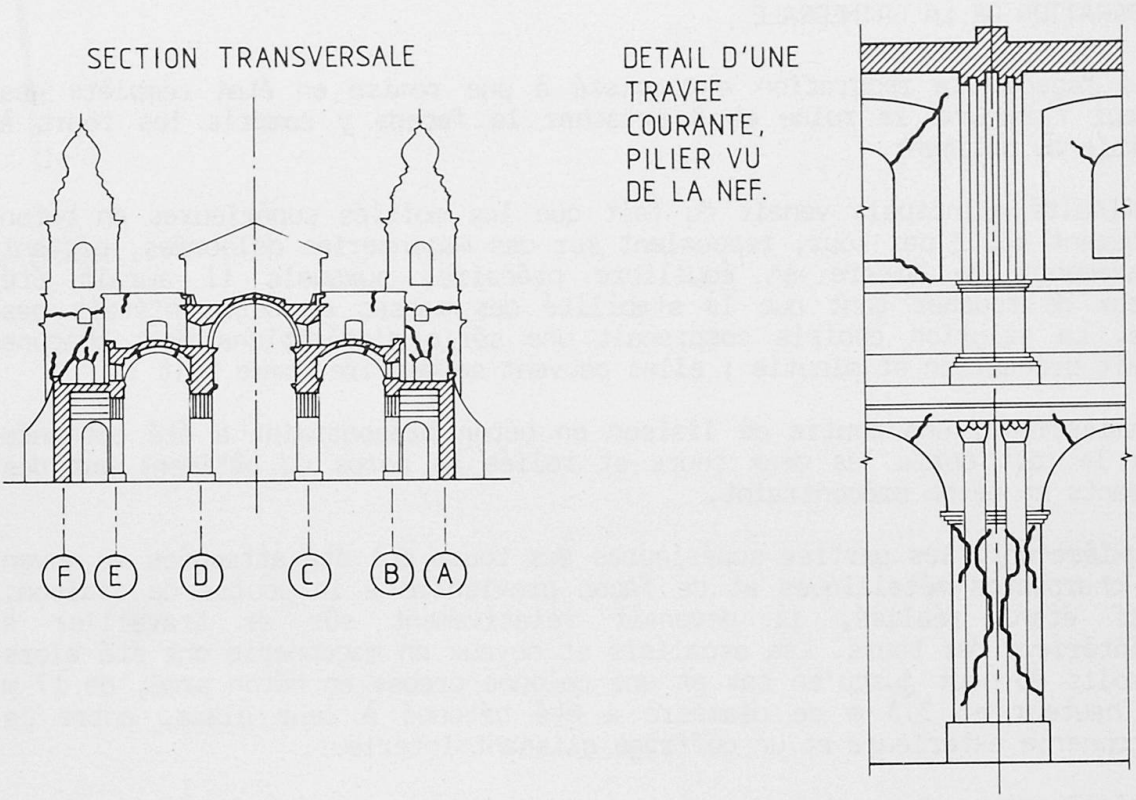


Fig. 6 Fissures typiques provoquées par le séisme de 1976.

Le dôme était intact mais tous ses supports étaient cisaillés indiquant des balancements dans les deux directions.

La façade arrière présentait des dégats semblables à ceux de la façade principale. Les deux petits clochers se sont éloignés du centre du bâtiment, leurs liaisons en béton armé ont été déchirées et la maçonnerie sous-jacente s'était désagrégée.



Fig. 7-8-9 Dégâts 1976. Respectivement tour Nord, tour Sud, supports du dôme.

4. REPARATION DE LA CATHEDRALE

Pour la façade, la réparation a consisté à une remise en état complète des tours qui frisaient la ruine et à attacher la façade y compris les tours à l'ensemble du bâtiment.

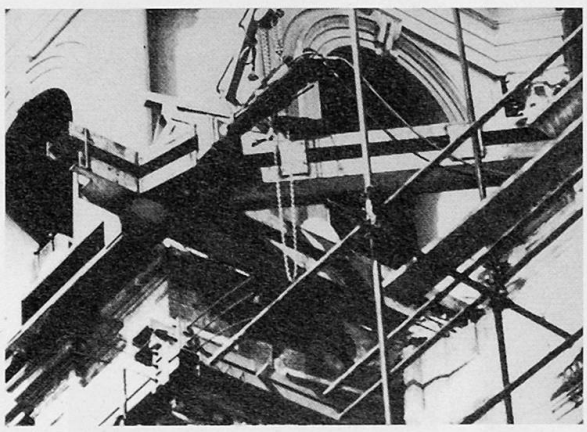
La difficulté principale venait du fait que les moitiés supérieures en béton armé, pesant 400 t par tour, reposaient sur des maçonneries délabrées, portant des parements de pierre en équilibre précaire, auxquels il aurait été hasardeux de toucher tant que la stabilité des masses de béton n'était pas assurée. La solution choisie comprenait une série d'opérations dont chacune demandait précaution et minutie ; elles peuvent se décrire comme suit :

- Premièrement, une poutre de liaison en béton précontraint a été bétonnée sur le toit entre les deux tours et reliée au corps du bâtiment par des tirants en béton précontraint.

- Deuxièmement, les parties supérieures des tours ont été attachées au moyen de charpentes métalliques et de façon provisoire à la poutre de liaison. Ceci étant réalisé, il devenait relativement sûr de travailler à l'intérieur des tours. Les escaliers et noyaux en maçonnerie ont été alors démolis du haut jusqu'en bas et une colonne creuse en béton armé, de 17 m de hauteur et 3.5 m de diamètre a été bétonné à leur place, entre la maçonnerie extérieure et un coffrage glissant intérieur.

- Troisièmement, les parties supérieures des tours ont été fixées de façon permanente à la poutre de liaison, ce qui demandait un travail minutieux en sous-oeuvre, comprenant démolition partielle de maçonnerie, mise en place de gaines et armatures, bétonnage, enfilage et mise en tension de câbles ancrés dans la poutre de liaison.

Une fois que les parties supérieures des tours étaient bien attachées à la poutre de liaison, la seconde phase de la réhabilitation des tours pouvait commencer. Elle comprenait la dépose du parement de pierre, l'exécution en surface de la maçonnerie d'une ossature légère en béton armé destinée à la retenir, l'ancrage de l'ossature à la colonne creuse par barres d'acier dur,



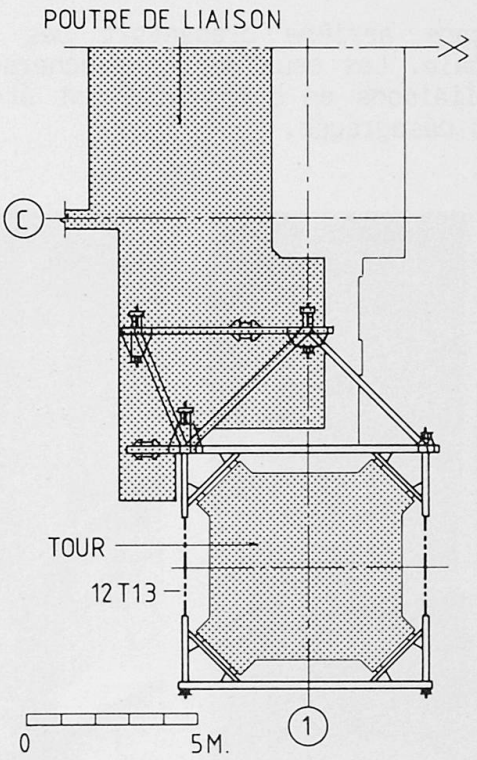


Fig. 10-11 Fixation provisoire des tours à l'aide de charpentes métalliques.

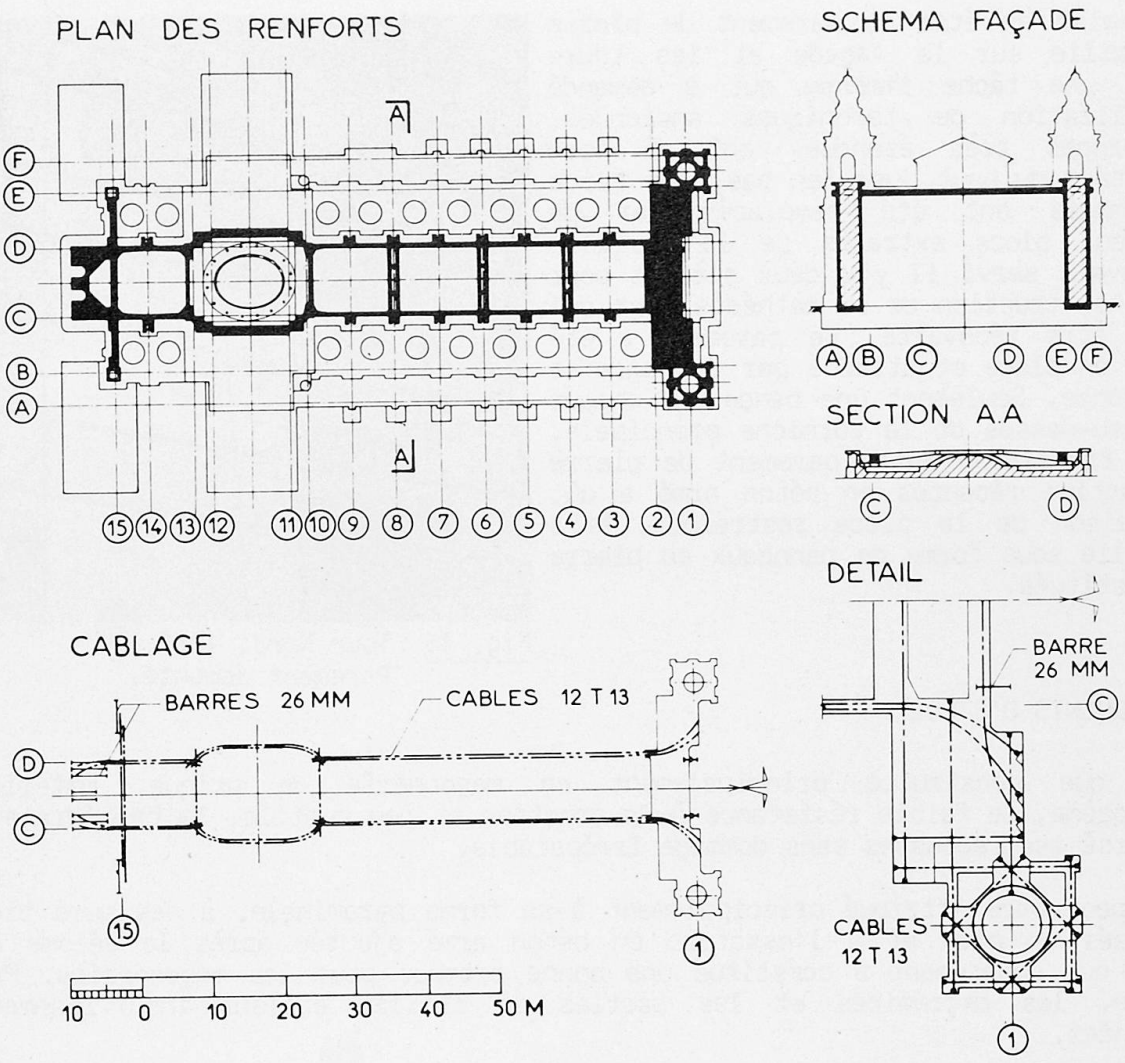


Fig. 12 Renforts en béton précontraint, réalisés en 1977-80.

la remise en place du parement de pierre et l'injection des vides au mortier de ciment sous basse pression.

Le dôme était en bon état mais ses supports devaient être remplacés ou réparés. Afin de se libérer de leur charge pendant les travaux, une charpente métallique a été montée sur le toit permettant le levage du dôme au moyen de 16 paires de vérins plats. Le dôme a été levé en avril 78 et par cette opération son poids a été déterminé à 510 T recoupant l'estimation à 10% près. Ensuite les murs cisaillés ont été démolis par groupes et refaits, les colonnes ont été réparées et le dôme de nouveau assis dessus par vidange de boîtes à sable.

Les réparations de la façade arrière comprenaient la continuation des tirants sur le toit et leur ancrage au moyen de barres d'acier dur fixées à une traverse en béton incrustée dans la façade à un niveau d'environ deux tiers de sa hauteur. Les petits clochers ont été attachés aux tirants par une poutre de liaison.

Les réparations à l'intérieur de l'église ont consisté en la réparation des voutes, arches, murs et piliers comprenant des travaux de pierre et de maçonnerie très étendus et le renouvellement d'enduits rendant à l'église son aspect esthétiquement agréable et rassurant.

La remise en état du parement de pierre de taille sur la façade et les tours était une tâche immense qui a demandé l'application de techniques anciennes. Des zones très étendues ont dû être démontées du haut jusqu'en bas, les blocs endommagés ont été remplacés par de nouveaux blocs extraits de la carrière qui avait servi il y a deux siècles pour la cons- truction de la cathédrale et qui a dû être réouverte. Le parement a été remis en place et attaché par des agrafes de bronze. Seulement une bande sur chaque tour au-dessus de la corniche principale, à la transition entre parement de pierre et parties récentes en béton armé a dû, en raison de la place restreinte, être rétablie sous forme de panneaux en pierre reconstituée.

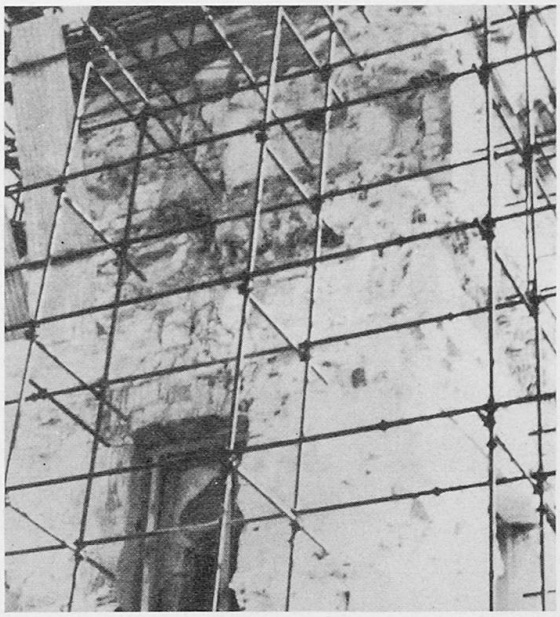


Fig. 13 Tour Nord, 1979. Parement démonté.

5. ELEMENTS D'ETUDE

Bien que construite principalement en maçonnerie de brique, matériau hétérogène, de faible résistance à la traction et peu ductile, la cathédrale a traversé les secousses sans dommage irréparable.

Ceci peut être attribué principalement à sa forme pyramidale, à des murs bien disposés en plan et à l'ossature en béton armé ajoutée après le séisme de 1917, qui apparement a constitué une bonne retenue pour les maçonneries. Par contre, les extrémités et les parties en saillie étaient insuffisamment attachées.

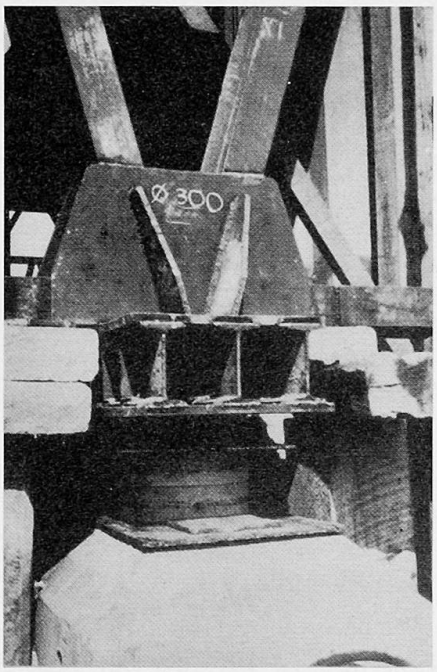


Fig. 14 Levage du dôme par vérins plats.

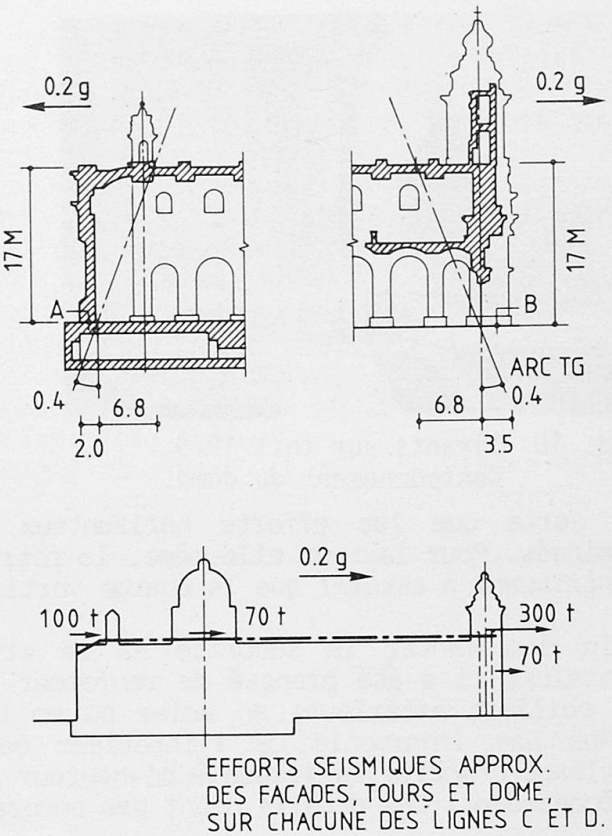
L'idée directive de la réparation a été de respecter l'authenticité du bâtiment existant et d'en constituer un corps continu à l'intérieur duquel la transmission des charges verticales et horizontales pouvait être suivie. La cathédrale devait rester une structure poids et même si on était amené à incorporer du béton armé ou précontraint, les charges verticales devaient toujours transiter par les maçonneries afin de permettre la transmission des charges horizon- tales par mobilisation du frottement interne. Par contre, les efforts de précontrainte, le cas échéant, ne devaient pas être transmis directement par les maçonneries car l'absence de vides ne pouvait pas être garantie.

Pour la maçonnerie, nous avons supposé une résistance à la traction pratiquement nulle et un coefficient de frottement de l'ordre de 0.4, donc plus qu'il ne faut, dans des cas simples, pour transmettre un impact de 0.2 g.

Considérant des murs en maçonnerie soumis aux impacts horizontaux, deux cas typiques se distinguent. Un mur soumis à l'impact transversal risque de se renverser tandis que soumis à l'impact longitudinal, un coin triangulaire à chaque extrémité a tendance à se dégager, vu que la cohésion du mur n'est pas assurée. Dans le premier cas, la stabilité peut être obtenue en ancrant le mur à un niveau approprié et en vérifiant les sections en flexion composée. Dans le deuxième cas, la stabilité peut s'obtenir à l'aide de tirants qui assurent le monolithisme d'un coin suffisamment important pour être en équilibre.

Ces principes ont conduit à placer sur le toit de l'église en direction longitudinale, des tirants qui pour des raisons de sécurité vont d'une extrémité à l'autre. Ces tirants sont ancrés aux deux bouts et en des points intermédiaires. Pour le dimensionnement, et vérifiant de façon classique les équations de projection et de moment, nous avons considéré l'impact venant des masses situées au delà des lignes inclinées d'arc tg 0.4 avec la verticale commençant suffisamment à l'intérieur du mur pour permettre la transmission de la contrainte de compression correspondante (fig.15).

Dans le sens transversal, la bonne tenue du bâtiment pendant le dernier séisme a permis de conclure qu'il n'avait pas besoin de renforcement supplémentaire, sauf au niveau de la façade.
Les tours présentaient un problème à part, vu qu'en tout cas il était nécessaire d'attacher les parties supérieures pour pouvoir travailler en dessous. En cas de séisme transversal, les tours, reliées entre elles par la poutre de liaison, seront soutenues par le mur de façade. En cas de séisme longitudinal pendant les travaux, tout l'impact des tours serait transmis aux murs longitudinaux et donc aux piliers par l'intermédiaire des tirants tandis qu'une fois les travaux terminés, les maçonneries des tours sont attachées aux colonnes creuses et contribueront à résister au renversement des tours



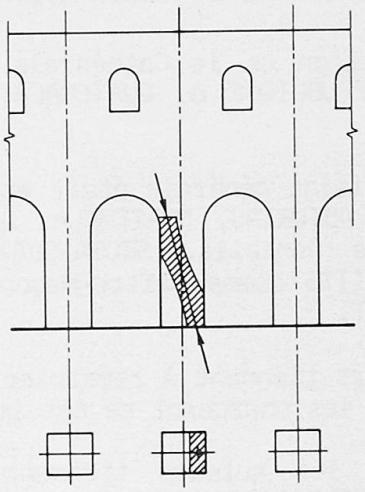


Fig. 15 Principes adoptés pour le renfort et schéma de transmission des charges.



Fig. 16 Tirants sur toit 1979. Contournement du dôme.



Fig. 17 Réfection du parement, 19/9.

de sorte que les efforts horizontaux transmis aux piliers s'en trouvent diminués. Pour la tour elle-même, le retrait et le fluage de la colonne creuse contribuent à assurer que la charge verticale passe par les maçonneries.

Afin d'augmenter la securité de la structure vis-à-vis des charges horizontales, il a été proposé de renforcer les piliers de la grande nef au moyen de colliers extérieurs en acier ou en bronze, d'une forme telle qu'elle ne rompe pas l'harmonie de l'intérieur de la cathédrale. De plus, il a été envisagé d'ancrer la façade à mi-hauteur par des tirants assez courts en béton précontraint. Ces détails n'ont pas encore trouvé leur solution définitive.

6. INTERVENANTS ET REMERCIEMENTS

La réparation de la Cathédrale Métropolitaine a été réalisée de 1977 à 1980 par la REPUBLIQUE du GUATEMALA, Ministère des Travaux Publics, comme Maître d'ouvrage.

La supervision générale était assurée par le Ministère. L'étude a été réalisée par José ASTURIAS, GUATEMALA, Architecte et EUROPE ETUDES, FRANCE et MEXICO Ingénieurs Conseils. SECSA-GUATEMALA était Entrepreneur Général avec John HALL-HIBBITTS comme Maître-Maçon. La précontrainte a été réalisée en système FREYSSINET.

Les auteurs tiennent à remercier les Autorités de la confiance qu'elles leur a montré en les chargeant de cet important travail.

En plus, les auteurs tiennent à exprimer leur gratitude à son Eminence CARDINAL MARIO CASARIEGO, Archevêque de GUATEMALA pour le profond intérêt avec lequel il a toujours suivi les travaux.

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College and Cathedral Restorations

Restauration d'une cathédrale et d'un collège

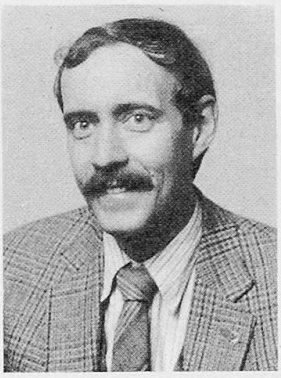
Die Restauration einer Kathedrale und eines Universitätsgebäudes

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SUMMARY

This paper is concerned with the work performed by the architects in the conservation of University College (1856, rebuilt in 1890) and the Cathedral Church of St. James (designed 1850-1868). Both buildings are located in Toronto. These are continuous conservation projects requiring restoration and renovation techniques. Both buildings have problems dating back to their beginning, problems of age and modern problems caused by a city environment.

RESUME

L'article traite de la conservation de l'University College (1856, reconstruit en 1890) et la Cathédrale de St. James (1850-1868). Les deux bâtiments sont à Toronto, Canada. Les deux projets sont des conservations continues exigeant les deux techniques de la restauration et de la rénovation. Les deux bâtiments ont présenté des problèmes dès le début de leur construction ainsi que des problèmes d'âge, et des problèmes modernes dûs à l'environnement urbain.

ZUSAMMENFASSUNG

Der Beitrag beschäftigt sich mit den Architektur-Arbeiten die im Zusammenhang mit der Restauration eines Universitätsgebäudes (1856, Neuerstellung 1890) und der St. James-Kathedrale (1850-1868) geleistet wurden. Beide Gebäude stehen in Toronto und befinden sich in einem ständigen Restaurationsprozess, um ihre Erhaltung zu gewährleisten. Die Instandstellungsprobleme bestehen seit der Errichtung der Gebäude und haben sich durch zeitliche Einflüsse und Umweltabänderungen verstärkt.

The two buildings discussed in this paper were designed by F. W. Cumberland, a most distinguished 19th Century Canadian Architect and an outstanding Toronto citizen. Frederic William Cumberland was an engineer by training and profession and had come to Canada in the mid 1840's from England where he had worked as an assistant Engineer in the building of the Great West Railway under the renowned engineer I. K. Brunel. Cumberland carried on two jobs, one as chief engineer for the new Toronto to Barrie railway and that of architect. In both jobs he excelled. University College was his last major building and on its completion he devoted all his time to the railway and to the public service, being elected to the first Legislature of Ontario in 1867.

In 1849 a fire had destroyed the original Church of St. James and Cumberland was appointed the architect to design the new building. Cumberland's associates for this building were G. Storm and Henry Langley and it opened for services in 1852 but the spire was not completed until 1874. The style is Early English Gothic and Cumberland's concept was based on the great fourteenth century parish churches of East Anglia and the Cotswolds. The chief material is yellow brick made from local clay with the stone trim and facings of imported Ohio sandstone. The main body of the church is 200 feet long by 84 feet high by 98 feet wide at the side porches. The spire, 324 feet high, is the tallest in Canada.

In 1978 we were asked to survey the Cathedral and to prepare a proposal for its renovation and restoration. Our proposal recognized that any work to be done must fall into one of two categories. Firstly, existing conditions or emergency repairs, which, if not done within a short period of time would adversely affect the structure and fabric of the building and secondly work of redecoration and general maintenance which would improve the appearance and aesthetics of the interior spaces but which could be postponed for several years.

One of the major problems we faced was a high level of humidity in the church. This humidity we concluded was caused by the high water table around the building, expecially in the spring and no proper floor slab in the basement side aisles and in the crypt. Moisture was being absorbed through the walls below grade. Above grade the brick had been cleaned a few years ago and the silicone had lost its effectiveness. This high humidity caused condensation damage to all surfaces, particularly walls and roofs. In the walls condensation resulted in plaster damage and flaking paint. Vapour migration to points of exhaust such as the nave roof, the tower, and the spire had caused timbers to decay and stone work to spall. In the roof, wood sheathing had decayed and the fasteners holding down the copper roofing had little or no strength which caused a serious problem during a windstorm in 1978.

An additional cause of the humidity, we learned, was a defective steam main. It finally ruptured outside the building but since it was installed in an 18" diameter barrel drain it allowed the steam to erupt in the Vestry of the church causing a fair amount of damage. The lesson to be learned from this experience is that if you are having difficulty controlling the humidity in an old building which depends for its heating on steam, check the steam mains very carefully. The breaking down of steam mains through corrosion both inside and outside the line can cause serious problems.

We cannot be sure we have discovered all sources of the dampness. Because of the age of the church most records of sanitary and storm drain locations have been lost. We proposed that the exterior walls be waterproofed down to the footings and as the work proceeded we were able to examine the condition of the storm drainage system. This proved to be a major contributor to the ground water problem as the drains were thoroughly plugged with decades of debris.

Additional corrective measures included a new slab with vapour barrier inside the church and, where the exterior walls were plastered inside we used a waterproofing type of plaster.

After a heavy windstorm resulted in damage to the spire and the roof it was obvious something urgent had to be done and we proceeded to make recommendations on the renovation of the building. The weathervane on the spire had been blown over and the decayed wooden base had to be replaced. On the downwind side of the roof the copper flashings and battens had opened at the seams and copper roof pans had been sucked out under heavy wind pressure. Scaffolding to repair the spire and roof was a major expense but it was not until scaffolding was in place that we could closely inspect the damage and make repairs.

Repairs to the damaged part of the roof were straightforward. The original copper was in excellent condition with very few cuts or holes. The roofer was able to reuse all of the flat sheets, replacing only the copper battens and some flashings.

The rotten roof deck was completely replaced using preservative treated wood. While the deck was removed we took the opportunity to insulate the roof and install roof vents. A vapour barrier was a major problem and there was some dispute as to whether an effective vapour barrier could be installed. It was subsequently left out. In monitoring the performance of this solution we have found a noticeable degree of condensation on the underside of the wood deck. Consequently, as we replace the remaining nave and side aisle roofs we intend to install a reinforced vapour barrier and to further improve ventilation of the installation.

Having completed the emergency roof and spire repair work we then undertook a structural analysis of the remainder of the spire. In collaboration with Morrison Hershfield, our structural engineers, we discovered that the steel beams supporting the bell chamber were severely corroded and that the wooden framing of the spire moved excessively under heavy wind loads. We had no alternative but to advise the owners of the danger that existed in the tower and spire. The 20 ton loading from the bells and the noise and vibration from ringing the bells presented a structural deficiency we could not ignore. We then proceeded to do a measured drawing of the spire and in collaboration with the engineers we designed a system of steel tie rods and gusset plates to stabilize a structure that had moved over 12 inches to the east and 8 inches to the south (under winds in excess of 50 m.p.h.) and put in a new steel frame from which we supported the bells. It still deflects slightly in the wind but is now tied into the masonry fabric and is no longer a threat to human safety.

To support the bells a new steel frame structure was built into the

masonry above the existing bell frame which was then tied to the bell support structure with hanger rods with rubber pads to dampen the vibration.

From the bell support structure we ran vertical tie rods up into the wooden spire following the line of each of the 8 corner posts. In this manner the overturning moment of the spire could be transmitted down into the solid masonry base.

The remainder of the wooden spire framing was "stitched" together with gusset plates at every framing intersection and with cross bolts through the laminated corner posts.

It was a very complicated procedure further complicated by the irregularity to be found in a very old octagonal structure.

After repairing the roof and alleviating the humidity problem, we could resolve other problems in the restoration of this building. We have completely cleaned and repainted the interior, revealing enriched cornices and other detail. We have also protected the stained glass windows and improved the building energy efficiency. This work was broken down as follows:

1. Strengthening and stabilizing the spire.

2. Draining the crypt and resolving rising dampness.

3. Repairing outer stonework, tuck pointing and installation of storm drainage.

4. Sprinkler system in Cathedral for fire protection.

5. Upgrading of interior lighting.

6. Renovation of west nave and side aisle roofs.

7. Interior decorating, repairs and cleaning.

8. Fundraising and incidentals.

We have been working on this project for four years now and we have done much to restore a building which had been ravaged by time and a climate too severe for a building more suited to the temperate English cotswolds.

The second building I would like to talk about is University College, a building which is now a National as well as a Provincial Historic Monument. It is generally considered a fine example of Gothic-Romanesque architecture and its only peer in Canada is the Houses of Parliament in Ottawa.

The fabric consists of stone and brick walls, wood framed floors, slate roofing and copper flashing. Internally it has two superb timber roofed halls and three staircases of a richness and quality of craftsmanship not to be found even in the older universities of Great Britain.

Historical and Architectural Significance

University College is one of the very few pre-Confederation buildings of historical and architectural significance still remaining in Toronto. It was built between 1856 and 1859, when Toronto was the capital of Canada, and the then Governor-General, Sir Edmund Walker Head, played a large part not only in promoting the project as a whole but also in formulating many aspects of the architectural design. Historically, the construction of University College marked a decisive stage in the evolution of "the university question", which engaged so much of the attention of legislators in Canada under the Act of Union from 1841 to 1867. The controversy still continued but now it circled about a new focus.

The mere existence of the building did much to ensure the future of the University of Toronto and to shape it, in due course, into a federal pattern, which was later to be followed elsewhere in Canada and the Commonwealth. The architectural interest and merit of the building has been widely recognized from the day it was opened down to the present. As soon as it was completed, the building became one of the great showpieces of Toronto and, indeed, of Canada, and important visitors, such as the Fathers of Confederation, were invariably taken to see it. Formal recognition of the building's historical and architectural importance was granted in 1971 when it was declared a national historic monument by the Historic Sites and Monuments Board of Canada.

Use and Overuse

The building is far from being merely a monument. No building in the University of Toronto has been used more heavily over the years than has University College. No building is being used more today. For more than thirty years after it was completed in 1859, it was the University's only building. Even after other buildings had been added to the campus, it was still referred to as the "Main Building" and continued to house all the administrative offices of the University until 1922. It still stands as a symbol of the University of Toronto, although it is now used primarily by the more than 2,000 students of University College. But it is also used for many other University purposes as well - for lectures to students in other divisions of the University, for examinations, for instruction in the School of Graduate Studies, for evening classes offered by the Department of Extension, for classes in the Summer School. It was the very heavy use that the building had received over the years, and is still receiving, that accounted more than anything else for the very poor condition that most of it was in 1971. It is only necessary to see the thousands of students who pour through the corridors in any ordinary day in term-time to understand why the old wooden floors were in such lamentable shape. From its normal resources the University had done what it could to patch and repair and to deal with emergency situations. The electrical and mechanical services in the buiding were hopelessly out of date, the lighting poor, ventilation nonexistent, the fire-hazard was frightening, the interior colours and finishes were worn, dull, and depressing.

The Urgency of Renovations

The urgency for such a large-scale programme of renovation had been

demonstrated by a number of things that came to light during investigations of the fabric. It had been revealed, for example, that the entire slate roof, together with the gutters and downpipes, had deteriorated through age to a point where dangerous damage was being caused by rain to the masonry walls and interior finishes, so that the roofing had to be replaced with the least possible delay. Another ominous sign manifested itself earlier. One spring day in 1967 when West Hall was being used as a robing-room for Convocation a large patch of plaster fell in one of the lecture-rooms beneath it. Investigation disclosed that the original beams that had supported West Hall since 1859 had twisted and loosened so that the whole of the floor of the Hall might well have collapsed under the weight of any large crowd. These particular wooden beams were replaced with steel. But no one could be sure that there were not similar situations elsewhere in the building. At about the same time, when part of the Cloister Wing had been stripped down to the original masonry, it became obvious that the whole fabric of that part of the building had been so weakened by alterations made over the decades to adapt it to changing uses that there was imminent risk of the bearing-walls giving way.

Planning

In 1966 the University appointed the late Professor Eric Arthur as the architectural consultant for the old main building of University College. Since then he and our firm of architects with whom he was associated, Wilson Newton Roberts Duncan made a thorough survey of the building, drew up plans for a comprehensive programme of renovation, and submitted recommendations about how the programme should be phased and carried out. All these plans and recommendations were carefully studied by officers of the Department of Physical Plant of the University and received their full approval. The objectives that the architects set for themselves, after consultation with the Principal of the College, in planning the work of renovation might be set down roughly as follows:

1. to reduce to a minimum the fire-hazard, while maintaing so far as possible the historic atmosphere and architectural distinction of the building;

2. to make the most efficient use of available space;

3. by minor structural additions, to recover most of the space that will be lost through changes in the interior lay-out to reduce the fire hazard;

4. to improve the services in the building, bringing them up to the present-day standards;

5. to make the facilities available to staff and students (including washrooms, which were then sorely inadequate) conform more closely to what is expected nowadays;

6. to protect and preserve a unique historic building for another hundred years of useful life.

Ontario Fire Marshal's Regulations

For a building built between 1856 and 1859 to be renovated successfully and satisfactorily it was necessary to seek some elasticity in the application of the regulations of the Ontario Fire Marshal and his colleagues in other jurisdictions.

The architects prepared a brief to the Ontario Fire Marshal outlining their proposals and the concessions they required. After substantial negotiations an agreement was reached whereby fire safety could be assured without destroying the quality of the building or making renewal impossible or impossibly costly.

In the subsequent renovations we replaced wood floors with concrete and steel, installed new staircases, installed new firedoors and a sprinkler and alarm system.

Other changes include new and efficient electrical and mechanical systems.

To blend these modern (and frequently drastic) changes into the fabric of the old building became the major problem.

The concrete floors now have new oak parquet floors in place of the old oak strip floors. While not the same, these floors restore the colour and atmosphere of the original without the sound.

The ceilings were dropped to accommodate the electrical and mechanical equipment. The original high ceilings are a blessing, but even so it was a struggle getting all the equipment in.

Where new doors were required the trim and panelling would be duplicated. If the door was a fire door, the hollow metal door would be panelled with wood and stained. They are usually indistinguishable from the originals.

Always we tried to maintain the original wainscotting. To conform with the fire requirements we would strip the original finish and then would stain and refinish with a fire retardant varnish. Many samples of the finish are required each time before a suitable colour can be obtained. The fire retardant finishes have a different character from the original shellac or varnish finishes. When new wood is present the new finish seldom matches that on the old wood. The glossiness and transparent depth of the finish is important and can be affected by the order and manner in which gloss and satin (or satin and gloss) coats are applied.

The new staircases were another challenge. For fire exits, we could not use wood and duplicate the other staircases. We turned to stone for the treads and wrought iron, for the balustrade and used an oak handrail. The doors go through where windows or doors originally were, thus allowing us to preserve original trim and stonework. Where a window had to be closed up we left it exposed and recessed the masonry slightly (leaving it exposed) thus leaving the history of the building visible.

Outside the new brick colour is a compromise between the existing uncleaned brick colour and the colour of the brick if it were clean. Result: it blends in very well.

Mechanical & Electrical

The mechanical and electrical requirements necessitated some drastic measures. Because of the size of the building the college has three mechanical rooms. For two of these rooms there was insufficient space. The solution: bury them. The college is surrounded by earth berms. We selected areas where flagstone terraces or patios already existed, dug them up, built the mechanical rooms and replaced the terraces almost exactly as they were before.

In one case, access to the mechanical room was required from two separate areas. We managed this by trenching under the basement floor and covering the resulting trenches with access covers.

New Uses, Adaptations

Much of the basement area of the college consisted of dark, dank and avoidable seminar rooms, storage rooms, electrical rooms and even inaccessible rooms. A vast resource!

The whole was made habitable by dampproofing the exterior walls and enlarging the area wells outside. Coupled with improved lighting and ventilation and by improved access we have just about added a new floor to the college.

Some seminar rooms were modified for audio visual uses by means of adding wood shutters to the inside of the windows and new wiring in the floors and the walls. By this means, we obtained two additional language laboratories, two new theatre-type lecture rooms, plus various seminar rooms and offices.

Conclusion

Throughout, our objective has been conservation wherever possible. We restore where parts are badly damaged. And we recycle, or modify, when modern needs and requirements demand it. Always within the context of, and in sympathy with the old building.

It has been possible in each case to preserve the character of the old building and to unobtrusively improve its efficiency, safety and comfort.

Preserving the Original Static Scheme in the Consolidation of Old Buildings

Sauvegarde de l'équilibre existant dans la consolidation de vieux bâtiments

Bewahrung des vorhandenen Zustandes bei der Konsolidierung alter Gebäude

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F. Lizzi originated in 1952 the «palo radice» (root pile), first example of micropile, initially used for underpinning and afterwards for solving difficult foundation problems. At about the same time he introduced the «reinforced masonry» technique for the consolidation of upper structures.

SUMMARY

There is no reason to consider a «living reality» such as an existing building, even though it may be in a precarious static condition, as an inert material to call to new life by the introduction of artificial forces. The possibility of increasing the safety factor of the building, within the framework and with full respect to the present equilibrium, should be the guiding principle for a correct static restoration work. According to this essential principle some original techniques are presented.

RESUME

Un bâtiment, même en mauvais état du point de vue statique, reste une «réalité vivante»: il n'y a aucune raison pour le considérer comme une matière inerte à raviver, moyennant des forces externes, et qui vont altérer l'équilibre actuel. Un travail de restauration, dans le but d'améliorer la stabilité d'un vieux bâtiment doit être basé sur le respect absolu de cet équilibre. Sur la base de cette philosophie ont été développées des techniques d'intervention, couramment utilisées.

ZUSAMMENFASSUNG

Ein Gebäude ist immer eine lebende Wirklichkeit, auch wenn es nicht mehr standfest ist. Es besteht kein Grund es als totes Material zu betrachten, das man durch äussere Einwirkungen, die den gegenwärtigen natürlichen Zustand verändern könnten, ins Leben zurückrufen kann. Ein Konsolidierungsprojekt, das dazu bestimmt ist die Stabilität eines alten Gebäudes zu verbessern, muss den vorhandenen natürlichen Zustand beachten. Auf dieser Grundphilosophie sind Interventionstechniken entwickelt worden, die dargestellt sind.

INTRODUCTION

This paper deals with some aspects of the consolidation of old masonry constructions.
The static equilibrium of such constructions is based essentially on the compressive strength of the masonry, as well as on the overall structural continuity of the building. The ability to resist tensile forces, peculiar to modern reinforced concrete structures, is completely lacking in old masonry fabrics.
The genius of the Old Architects of the Past was, therefore, fully engaged in absorbing horizontal thrusts, transmitted by arches, vaults etc., through original schemes, in which vertical additional loads would prevent the development of tensile stresses in the masonry.
Only in exceptional cases they resorted to "tie-backs", perhaps reluctantly, because such a device could not be considered on a satisfactory technical level; furthermore it was, generally, an unsightly addition from the aesthetic point of view.

1. THE CRACKS IN AN OLD BUILDING.

The symptoms of a possible state of crisis of the static equilibrium of an old building are the cracks, which appear in the structures as a clear evidence of the lack of tensile strength of the masonry.
At any rate the presence of fissures in a building does not necessarily imply a state of danger: old masonry buildings enjoyed, during the construction period and for some time after, of states of plasticity which allowed, generally, a compensation for some minor unforeseen deformations in the foundation soil, as well as in the upper structure.
In these evolutive phases, the intervention of Nature was instrumental in redistributing the loads and, when necessary, in opening natural joints (cracks) where the continuity of the masonry was not sufficient for absorbing tensile stresses. Generally, in these cases, the reduction or the elimination, dictated by Nature, of the redundant bonds peculiar to old constructions, are to be considered a relief more than a damage.

2. THE STATIC CRISIS IN OLD FABRICS.

When, on the contrary, there is clear evidence, in an old building, that the existing cracks are progressing and, furthermore, that some new fissures are appearing, then a state of danger has to be presumed.
At any rate, also in these critical cases, it should be remembered that the structure is still in equilibrium; its Safety Factor defined, as customary, as the ratio of the stabilizing forces to the disturbing forces, may be approaching the minimum value of one but, for the time being, it is not less than one. There exists a precious safety margin not to be wasted.

2.1 The importance of the tensile resistance in the masonry.

The cracks in the masonry are the most conspicuous evidence of a state of crisis for the structure; but they are only the effects, consequent to several

different causes. Therefore the most important step to be taken for the static restoration of an old building is the elimination of the original disturbing causes.
That is not enough; there are secondary adjustment movements to be feared, and the possibility that the original causes may act again (e.g. seismic events).
All the above causes entail the development of tensile forces which cannot be resisted by the masonry; therefore the latter needs an appropriate reinforcement.
Finally, there are cases in which the essential and only cause of the crisis is a general weakness of the masonry, which requires the introduction of reinforcing elements to absorb tensile stresses.
In conclusion, no serious restoration works can be carried out without a correct reinforcement of the building to counteract possible tensile forces.

3. ESSENTIAL REQUIREMENTS FOR A CONSOLIDATION WORK: THE "QUICK RESPONSE".

A consolidation work must be technically adequate, long lasting and, what is most, when completed, it has to give a "quick response" to any further displacement of the structure, however small.
The very restricted residual limits of "time" and of "deformations", available for the restoration works to be carried out and to start being operative, must be the subject of very careful and responsible attention from the Designer.
Also in cases where propping structures are to be provided for, the allowable limits of time and of deformations, both for the struts and for the building, have to be considered. All of that within the framework of the present and future equilibrium of the fabric.

4. THE CONSOLIDATION OF OLD MASONRY BUILDINGS: DIFFERENT APPROACHES

The consolidation of old buildings can be carried out according to two different approaches:
- "Active intervention" by which additional external forces are introduced in the structure and in the subsoil, under the form of constraints.
- "Passive intervention", based on reinforcing elements introduced in the soil and in the structure, without any constraints, in order to ameliorate the existing state of equilibrium, increasing the Safety Factor.

5. THE "ACTIVE INTERVENTION".

The active intervention, with the introduction of external prestressing, is intended to partially counteract the acting forces and, possibly, in some cases to reverse the deformations.
The active intervention is dictated by the fear that further deformations could be fatal to the overall stability of the construction.
This is the case of:
- underpinning by means of precast or cast-in-place piles forced against the upper structures;
- introduction of states of stress in the subsoil immediately below foundation level, in order to accelerate its consolidation;

- introduction in the masonry of tie-backs, steel bars and/or cables to be prestressed in order to provide concentrated tensile strength, where necessary.

Practically, according to this type of intervention (active), a new additional system of forces is superimposed to the present static scheme, considered no longer sufficient for guaranteing the stability of the building.
The soil and the masonry are partially surcharged beyond the present levels: this is a risk which cannot be underestimated by the modern designer, entrusted with a restoration work.
As a matter of fact, one should not forget that the problem of consolidating an old building is completely different from the problem of a new construction:

- In modern constructions, the materials are well known as far as their strength and elasticity are concerned; they are capable of accumulating a well calculated elastic reserve (as with prestressing); their response to the acting forces, in terms of unit stresses and deformations, can be very reliably assessed using the most recent calculation methods.
- On the contrary, the problem of consolidating an old building is completely different: the materials are heterogeneous and brittle; the forces/deformations ratio is practically unknown because the deformations have already taken place; their response to new forces cannot be reliably assessed. Also, in case of extensive grouting of the masonry, the possible presence of weak spots due to inadequacies of the treatment can reduce or nullify any local prestressing intervention. The only most important data to consider is that the building is standing, hence it is in a state of natural equilibrium, however precarious; any change to its static scheme entails a very serious unwarranted risk.
  In a consolidation work "what" the Engineer wants to do must be subordinate to "how" he wants to do it.

6. "PASSIVE INTERVENTION"

For all the above, it seems more logical to consider the present state of equilibrium of the building as the starting point for the consolidation works; the present Safety Factor is preserved and should be increased by the new restoration works.
According to this philosophy, in a passive intervention the consolidating works are aimed at providing additional resources for a "living" structure, within the framework of the existing state of equilibrium, without modifying its static scheme and, what is more, without worsening the present critical state of stress, both in the masonry and in the subsoil.
On the other hand, the requirement of giving a "quick response" to any further displacement of the structure must not be forgotten.
The essential features and the means for a passive intervention are described in the following paragraphs, both for foundation strengthening (underpinning) and for masonry reinforcement, where the possibility of giving a "quick response" is duly stressed.

6.1 Underpinning by means of "pali radice" (root piles).

This special type of pile (micropile) was developed by the Author of the present paper about thirty years ago; its use for underpinning, as well as for other

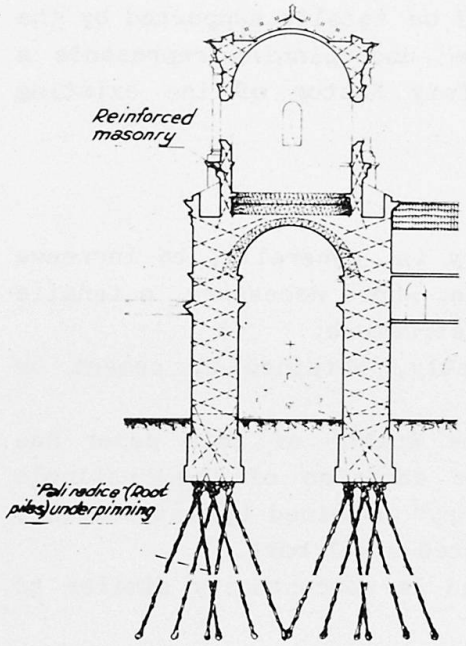


Fig. 1 - Typical strengthening of an old building.

engineering purposes, is described and discussed at length in technical literature.

A typical underpinning by means of root piles is shown in Fig. 1; where the "reinforced masonry" technique for the upper structure is also indicated.

For the purpose of the present paper, the two essential characteristics of a "palo radice" are recalled:

- The pile is drilled "through" the existing masonry; therefore, it is automatically connected to the upper structure and becomes operative as soon as the concrete has set ("quick response" in terms of time).
- The "palo radice" is essentially a friction pile and therefore its settlement under load is very reduced ("quick response" in terms of deformations).

Fig. 2 summarizes how the "quick response" problem is solved with a "pali radice" underpinning.

Chart a) shows the assumed evolution of the foundation settlement with time; this diagram could be obtained from the study of the previous movements of the building; "m" is the value of settlement corresponding to the period of time considered still available for carrying out the underpinning (point T).

In presence of provisional struts, the value "m" (generally no more than a few millimetres) can be taken as the value for which the propping structures are assumed to become fully loaded.

Chart b) shows the Load/Settlement curve for the selected "palo radice". This diagram can be drawn on the basis of previous experience or derived by direct, unexpensive, load tests. The working load P to be entrusted to the single pile of the future underpinning is the value corresponding to settlement "m", as indicated in diagram a).

For all the above, a "pali radice" underpinning does not introduce any alteration in the existing overall equilibrium of the structure and, at the same time, it can readily become operative and give a gradually increasing support to

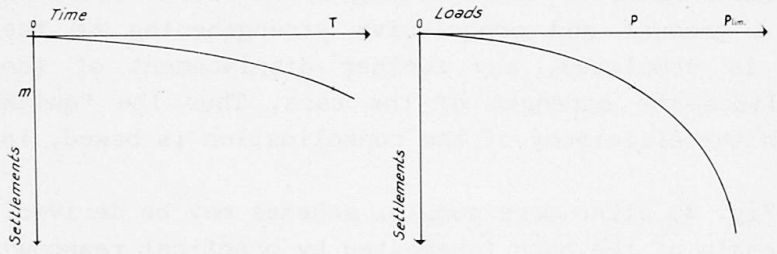


Fig. 2 - The problem of the "quick response".

the existing foundation.
The transfer of the loads from the building to the "pali radice" underpinning takes place gradually, according to the further settlement of the structure, if any. It is possible that the settlement will stop well before reaching the maximum value of "m", for which the building would be totally supported by the new foundation; that means that the "pali radice" underpinning represents a substantial addition to the fully preserved Safety Factor of the existing foundation.

6.2 "Reinforced masonry".

The purpose of the consolidation of an old masonry is, generally, to increase its resistance to compressive forces and to provide, where necessary, a tensile strength which cannot be displayed by the original structure.
The increase of compressive strength is, currently, obtained by cement or chemical grouting.
As regards the resistance to tensile forces, the Author of this paper has fostered, over the past thirty years or so, the adoption of the "reticolo cementato" which is some sort of "reinforced masonry" obtained by introducing, in the very thickness of the wall, a series of grouted steel bars.
"Reinforced masonry", therefore, is inspired by and is conceptually similar to reinforced concrete.
The basic element of "reinforced masonry" is a steel bar (2÷3 metres long), cement or resin grouted in small holes, drilled to appropriate slopes through the masonry wall, from face to face. The grout (injected at very low pressure) provides the necessary bond between the bars and the surrounding masonry.
A series of such bars, adequately overlapped, provides the masonry with the required tensile strength; therefore, positions, length and number of reinforcing bars must be designed according to the tensile forces to be resisted, just like for reinforced concrete.
Fig. 3 shows, in an horizontal cross-section of a wall, an elementary scheme of a series of bars (the so called "overlapping scissors"); the scheme, formed by two rows (at different levels) of slanting bars, supplies a chaining for resisting a tensile force T; it is worth noting that the reinforcement can be extended to any length of wall.
The efficiency of the chaining is based on the steel/masonry adherence as can be easily checked by unexpensive in-situ tests. As a rule, a tensile force is not entrusted to a single chaining: normally, the walls are engaged for a substantial height by several series of "overlapping scissors", adequately staggered.
It is to be stressed that the construction of such a bolting system in the masonry does not entail any disturbance of the existing static conditions; on the contrary, it represents a gradual and progressive strengthening of the structure. When the bolting is completed, any further displacement of the masonry, however small, mobilizes the strength of the bars. Thus the "quick response" requirement, on which the efficiency of the consolidation is based, is perfectly fulfilled.
From the elementary scheme of Fig. 3, other more complex schemes may be derived. Due to the relatively short length of the bars (suggested by practical reasons) the boltings can be positioned so as to suit any resistance scheme, with great flexibility; no substantial alteration of the external appearance of the



Fig. 3 - The "overlapping scissors".

structures is involved.

6.2.1 Examples of "reinforced masonry".

Fig. 4 shows the horizontal cross-section of a reinforced masonry cross-wall. The reinforcement is intended to provide resistance to Traction as well as to Bending Moment, which can affect the wall. The shadowed lines indicate the resistance paths on which the reinforcement can be relied upon.

Fig. 5 shows the reinforcement for the upper wall of a colonade. The reinforcement is intended to form some sort of "reinforced masonry" long beam, overhanging the colonade, in order to compensate out-of-balance effects which can be generated by possible differential settlements of the columns.
The dotted lines represent, schematically, tensile chainings formed by several series of "overlapping scissors".
The complete network forms a reinforced beam, with a double horizontal steel reinforcement (sections A-A and B-B) and the slanting bars C-C as stirrups.
Also in this case, no disturbance is introduced in the equilibrium of the structure which is gradually reinforced and, at the end, behaves like a proper beam.

Another example is the reinforcement of a column shown in Fig. 6: the bars are arranged according to a double helix, tangent to an ideal internal core; the complete network provides a reinforcement both in vertical and in horizontal sections, like in a reinforced concrete column provided with vertical reinforcement and stirrups.
The work is to be carried out step by step with good workmanship, having care not to create any disturbance, even if temporary.
If necessary, it is advisable to provide for a provisional strut formed by cables fastened around the column under light tensioning.

6.3 "Reinforced masonry" and the earthquakes.

One of the characteristics of "reinforced masonry" is the possibility of spreading the reinforcement over large wall surfaces instead of concentrating it on single areas or elements. This enables a reinforced wall to display an overall tensile strength.
Consequently, the energy introduced by an earthquake can be dissipated without a substantial loss of strength, because the continuity of the masonry can be preserved also in case of diffused cracking. Practically the ideal knots formed at the crossings of the bars behave like "plastic hinges" which keep the masonry together under any unfavourable emergency.

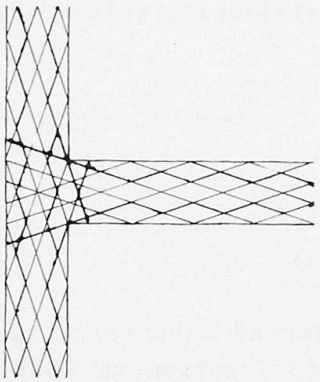


Fig. 4. Reinforced cross-wall.

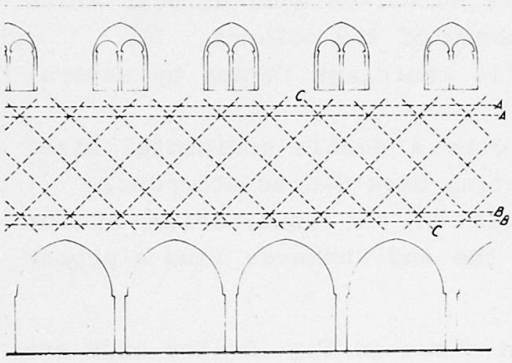


Fig. 5. Reinforcement of a colonade upper wall

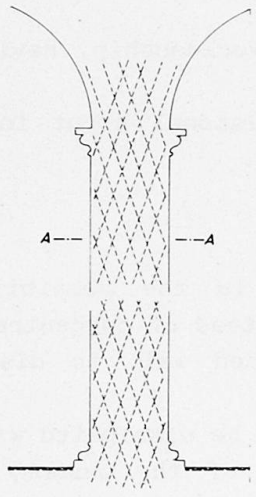


Fig. 6. Column reinforcement.

CONCLUSIONS.

Additional stresses, introduced in the subsoil and/or in the upper masonry of a building in critical static conditions, are to be feared, in the Author's opinion, more than further slight foundation settlements and/or upper structures deformations; the displacements can be accepted or, alternatively, prevented by temporary struts.

A correct consolidation work should be carried out in full respect of the existing static equilibrium, aiming to increase the Safety Factor of the building. This may be best obtained by the "passive intervention" approach. Experience gained by the Author, who was responsible for the static restoration of several hundreds Old Buildings and Monuments in various parts of the world, indicates that both the "pali radice" underpinning and the "reinforced masonry" technique are reliable and efficient tools for the preservation and strengthening of old fabrics.

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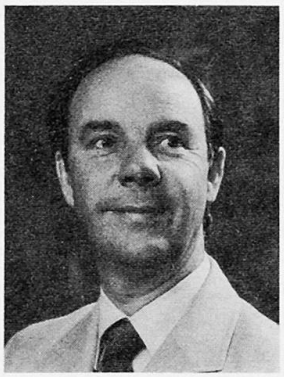
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Relevelling, Raising and Re-siting Historic Buildings

Soulèvement et déplacement de monuments historiques

Heben und Verschieben von historischen Gebäuden

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J. Pryke, born 1928, joined his family business, Pynford Limited, in 1952, as an Engineer after graduating from Trinity College, Cambridge. He has had 30 years' experience in underpinning schemes, foundation and structural design, moving buildings, structural alterations and restoration.

SUMMARY

The Pynford stooling method has proved the key to a simple, safe and systematic method of underpinning a wide variety of brick and stone masonry buildings. The paper describes a selection of underpinning, jacking and moving projects, including raising a 14th century building, moving an 18th century building, both in the U.K., and raising and moving a building in Norway. Proposals for lifting the Leaning Tower of Pisa up 2.25 m and for preserving historic city squares by raising the surrounding buildings are discussed.

RESUME

La méthode Pynford d'étançonnement s'est avérée un moyen simple, sûr et systématique pour étayer une grande variété d'édifices en briques et en pierre. L'article décrit une sélection de projets pour étayer, soulever et déplacer des bâtiments, tel que le soulèvement d'un bâtiment du XIVème siècle, le déplacement d'un édifice du XVIIIème siècle tous les deux au Royaume-Uni, et le déplacement d'un édifice en Norvège. Des propositions pour redresser la tour de Pise et pour préserver des places dans des cités historiques en soulevant les édifices environnants sont discutées.

ZUSAMMENFASSUNG

Die Pynford-Stützmethode ist ein einfaches und sicheres Mittel für die Unterfangung verschiedenster Backstein- und Blocksteingebäude. Der Beitrag schildert eine Auswahl von Unterfangungen, Hebungen und Verschiebungen von Gebäuden, einschliesslich das Heben eines Gebäudes aus dem 14. Jahrhundert, das Verschieben eines Gebäudes aus dem 18. Jahrhundert, beides in Grossbritannien, sowie das Verschieben eines Gebäudes in Norwegen. Vorschläge zur Hebung des Schiefen Turmes in Pisa um 2.25 m und zur Erhaltung der historischen Plätze durch das Heben der umliegenden Gebäude werden diskutiert.

"RELEVELLING, RAISING AND RESITING HISTORIC BUILDINGS"

PRESERVING OUR BUILT HERITAGE. 1. If we are to preserve the wonderful heritage of our older buildings and cities we must not only maintain the fabric by continual care and repair but also maintain their useful occupation. Major expenditure is needed if the foundations fail, or if sites sink or water levels rise causing repeated flooding. Our major cities also need to be modernised if they are not to be deserted and alternatives to demolition and reconstruction are needed. If there is a high risk of earthquakes then important masonry buildings may also need the insertion of structural frameworks with minimum disturbance of the internal appearance.

2. The cost of major structural alterations and repair is a combination of the direct cost of the structural work and the indirect cost of temporary works, protection and reinstatement. It is most important to develop methods that reduce both direct and indirect costs. This paper describes applications of the Pynford stool method of constructing reinforced concrete beams within or under loadbearing brick or stone masonry walls. The Pynford method does not require needles or other temporary external shoring, and it has been used to underpin over 10,000 homes and many larger buildings. The method has also made possible and economically viable many projects for lifting buildings small and large and for moving them to new sites. Thus roads can be widened and car parking or further accommodation can be provided beneath older buildings with reasonable economy and without wholesale demolition and reconstruction. A current project in Norway, described briefly later in this paper, is particularly important. A building is being moved back to facilitate road widening and raised to provide an additional level for parking. The work is a privately financed development project. It has been made commercially viable by the combination of a number of techniques, each of which have lowered costs.

3. The construction of frameworks of reinforced concrete beams beneath walls of loadbearing brick or stone masonry, using a simple, safe and systematic method, is the essential key if projects for raising and moving major buildings are to be completed at predictable cost and on programme. Once the frame and permanent supports beneath it are constructed any building may be lifted. The amount of damage will then depend upon the precision with which the jacks are controlled. With suitable equipment differentials of less than 5 mm can be maintained during lifting and moving even if the supports yield differentially. This will eliminate significant damage. Heavy structures require hydraulic jacking systems to lift them and sliding means such as p.t.f.e. on stainless steel, or grease skates running on accurately constructed tracks to move them. Smaller buildings or monuments may be lifted and moved by crane (Fig. 1) and road trailer.

UNDERPINNING. 4. Underpinning is required when existing foundations are failing or when proposed projects will weaken or remove the support that they provide. If existing foundations are too small then it may be practicable to enlarge them to provide a greater support area and reduced ground pressures. This was the method adopted to stabilise the central tower of York Minster, (Ref. 1). Pynford has proposed that the Leaning Tower of Pisa should be stabilised by enlarging the foundations. It is most usual, however, to provide new and deeper foundations when existing ones fail. Unless a firm stratum can be found within, say, 5 m of ground level, or less if the water table is high, it is cheaper to construct piles beside walls than to dig piers beneath, particularly in non-cohesive soils with a high water table. However, capping and supporting costs are reduced if support is provided directly beneath the loads, and much disturbance and reinstatement cost is eliminated if access is restricted to one side only of the wall to be underpinned. This can be done using the Pynford stooling method coupled with hand excavated piers or jacked in piling.

THE PYNFORD STOOLING SYSTEM. 5. The Pynford stooling system was an important step forward in technique. It eliminated needles and props on either side of



Fig.1. Crane lifting building.

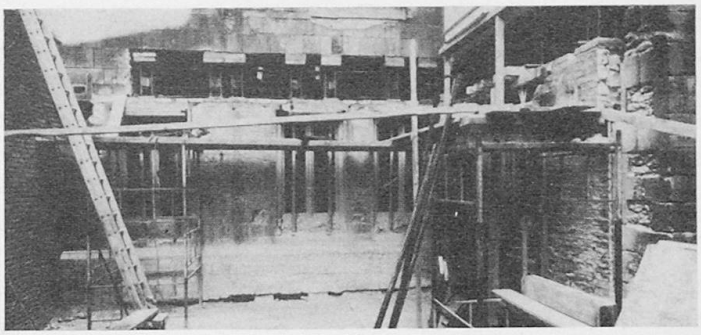


Fig.2. Stools at Trinity College.

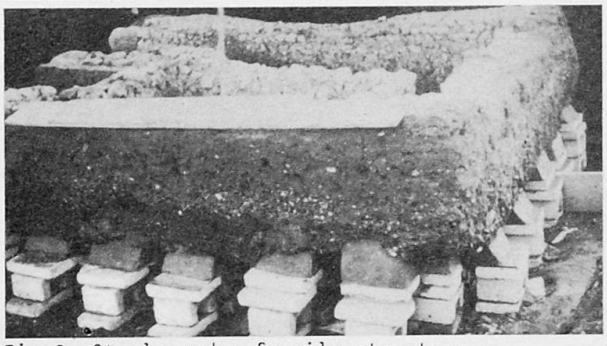


Fig.3. Stools under fragile structure.

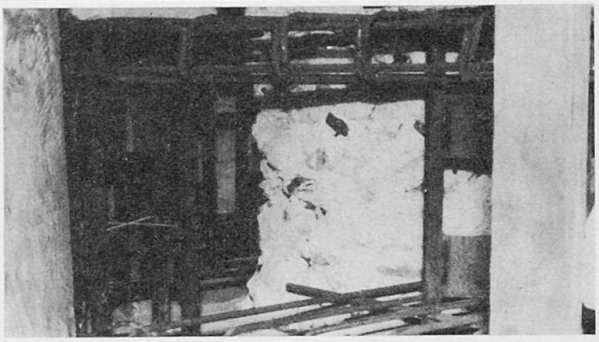


Fig.4. Stools in Winchester columns.

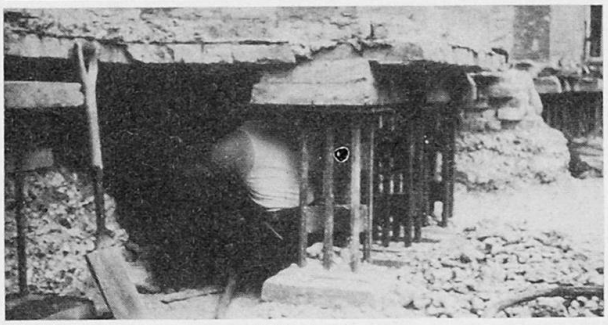


Fig.5. Stools under 400 T pier

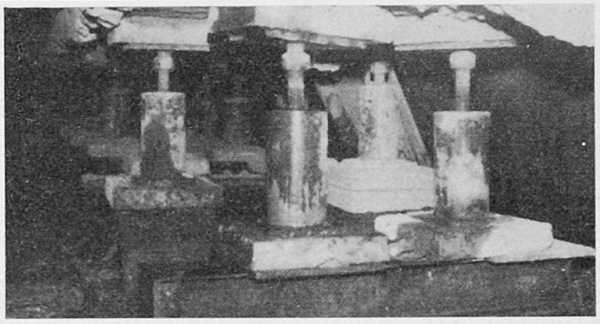


Fig.6. Pre-loading timber piles.

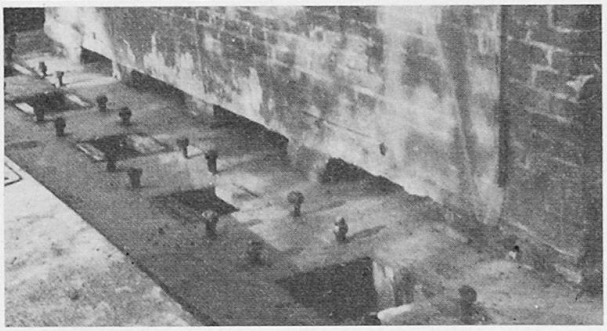


Fig.7. Underpinning Dutch church.

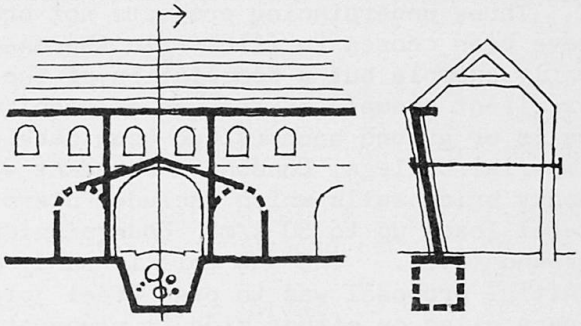


Fig.8. RC frame in mediaeval wall.



Fig.12. Steel frame under 17th Cent. mansion.



Fig.13. 2500 T being moved - Norway.

the wall by using the existing foundation, or the wall that is to be removed, to support a permanent prop, the Pynford stool, which replaces the needle. The method has been described on a number of occasions by the author and others. (Refs. 2, 3 and 4). It is very safe and offers a controlled method of cutting into loadbearing masonry no matter how weak. As well as the intrinsic safety of the method, the stooling system has many practical advantages. This is demonstrated by the increasing use of the stooling method of cast in-situ underpinning beam construction world-wide.

6. Masonry with a low bearing capacity per unit area is replaced with steel struts which can carry the same load in a very much smaller area. (Fig.2). If the wall is thick, two or more stools may be placed in a row through the wall thickness. If the wall is fragile it is only necessary to cut partially into the wall before the first stool can be fixed so that the weakest walls can be "stooled up". In extreme cases it may even be necessary to clamp the wall above and below the stooling position. An important advantage of the stooling system is that it is very easy to increase the number of supports at little extra cost. In very fragile structures the size of the plates and the spacing of the stools may be adjusted to provide virtually continuous support. (Fig. 3). It is thus possible to construct continuous reinforced concrete beams or slabs less than 1.0 m deep within existing walls, no matter how thick, without cutting away more than, say, $0.3m^2$ at any one time. For smaller highly stressed piers, this area can be reduced to as little as $0.1m^2$. Older stone masonry walls are often formed with two outer skins of cut and bedded stonework with a weak rubble infill which often contains considerable voids. On one project, strengthening the columns of Winchester Cathedral, R.C. pads were constructed in the core of the column without propping the arches or removing the stone facing (Fig.4). Because the Pynford stools are permanently cast into the new R.C. beams, the load the stools carry is directly transferred into the new beam. This eliminates another cause of the damage that it is customary to expect during underpinning operations. It is easy to widen the beam by inserting further stools under projections such as chimney breasts or piers. Fully continuous beams can be constructed, although it is important to consider the overall stability of the structure if a significant length of the wall is to be cut away. Care must always be taken to provide adequate lateral as well as vertical support with any shoring system.

7. Three underpinning projects not previously described in the technical press have been chosen to illustrate the use of Pynford beams. The first is a very early example but a description of the work has not been published and it is an excellent example of the use of stooling. The work was carried out before slurry walls or ground anchors had been developed. To begin the redevelopment of Imperial College, London, a basement was required 11 m deep up to the face of heavy brick walls which included piers carrying up to 400 tonnes and foundation level loads up to 50 t/m. Underpinning was required to a depth of 13m below ground level. The sub soil is sandy gravel to a depth of 8 m over clay. The initial proposal was to pass steel joists through the piers supported on temporary piles on either side in preparation for the underpinning. This had two disadvantages. Firstly, the large holes required could not be safely cut through the piers; secondly, needling would have created considerable internal disturbance. Pynford were originally called in to solve the first problem by forming a reinforced concrete bearing pad through the piers which would cantilever on either side on to the needles. This has been done on a number of occasions, a recent example being the A-Kerk in Amsterdam. To solve both problems Pynford proposed a beam below basement level which would act as a needle running on the line of the wall to support the piers. (Fig.5). When excavation was commenced it was discovered that the water table was 3 m above the gravel clay interface, not 0.6 m as expected. In underpinning work it is vital that ground is not lost during excavation as it is providing the support for the rest of the structure. The

author invented and patented an excavation shield for this project which enables piers to be hand dug in waterlogged sand below water table without loss of fines. The Pynford shield has proved invaluable for many subsequent underpinning projects.

8. The Town Hall, Dundee, was suffering subsidence for the common reason that drainage had lowered the water table beneath the building and timber piles supporting the walls were going rotten immediately beneath the foundation. New piles were sunk on either side of the wall. Pynford beams were constructed at foundation level beneath the wall. The timber piles were cut off below water table, treated, pre-loaded by jacking and pinned in place using stools (Fig. 6), to provide temporary support whilst the underpinning was constructed, and permanent support at a reduced load. A combination of new bored R.C. piles and repaired pre-loaded existing piles offered significant cost savings on the project.

9. Many other examples of the use of Pynford stools may be cited. The very old Dutch brick church at Eenum, near Groningen, is typical of many early churches in The Netherlands. Early settlements were built on artificial mounds to protect them from winter flooding and heavy buildings constructed on these mounds frequently suffer considerable subsidence. The church tower had developed a significant inclination and broken away from the nave. The tower was underpinned with a projecting R.C. slab pierced with holes around the perimeter through which de Waal piles were jacked down to provide pre-tested support. (Fig. 7). At the Kings Gate, Winchester, an R.C. frame was inserted within the flint faced, rubble cored, North wall of this unique church over a mediaeval gateway without disturbing the inner wall face. (Fig. 8). At the Old Library at Shrewsbury, rotting timber beams were removed and horizontal R.C. ring beams and vertical R.C. panels were constructed in the inner loadbearing brick face without disturbing the mediaeval stone facing of the building, (Figs. 9, 10 and 11). In this case the stones were drilled and tied in from the inner face to the new R.C. work with resin bonded stainless steel ties.

JACKING AND MOVING. 10. The earliest Pynford jacking projects were designed to straighten out buildings distorted by subsidence and used large numbers of closely spaced jacks. It soon proved far more economical to relevel the sunken corner of a building than to demolish and rebuild it. This also has the considerable aesthetic advantage that the existing materials are retained. Buildings severely distorted by mining subsidence may tilt at inclinations exceeding 1 in 20.

Such properties are uninhabitable. Such tilting is often accompanied by minor distortion although, surprisingly, the distortion is often less than that caused by subsidence due, for example, to shrinkable clays, where differential subsidence of less than 10 cm will often cause far more damage than tilting due to mining subsidence of 60 cm. It is distortion of the structure that causes cracking and damage. The classic example of tilting without significant cracking is, of course, the Leaning Tower of Pisa. Tilted low-rise domestic dwelling houses can be relevelled and restored for less than two-thirds of the rebuilding cost.

11. Having established a reputation in the UK for both underpinning and jacking enquiries were received for moving buildings. Before large and fragile structures such as buildings can be raised and moved the first requirement is that they be supported by a secure framework or platform. (Figs. 1, 12). Major jacking and moving projects completed in the UK using the methods that have been described include underpinning and raising the 1000 tonne 14th C Shambles in Manchester 1.5 m as part of a major city centre redevelopment project and moving the 800 tonne 18th C Old Academy in Warrington 15 m to release land for road widening.

12. The sloping, derelict, Market Street site in Central Manchester, U.K. contained an important building, the oldest in the city, the partially timber

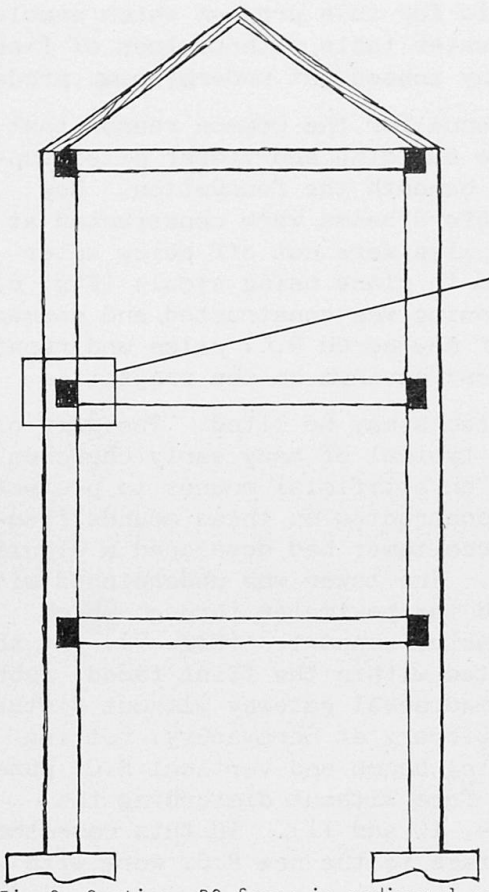


Fig.9. Section: RC frame in mediaeval walls.

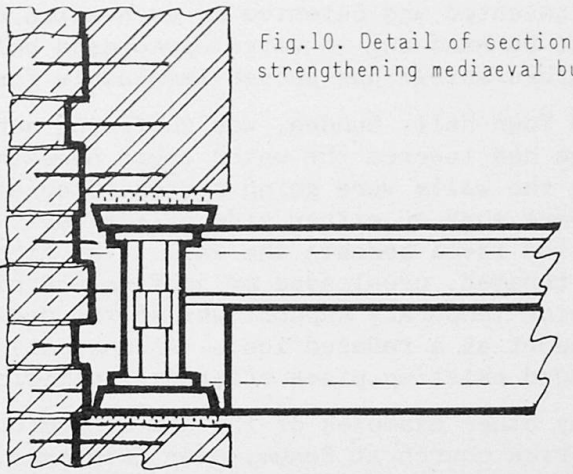


Fig.10. Detail of section: strengthening mediaeval building.

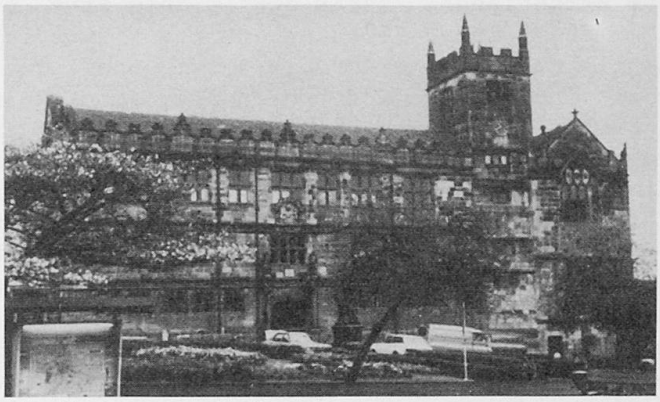


Fig.11. Shrewsbury Library.

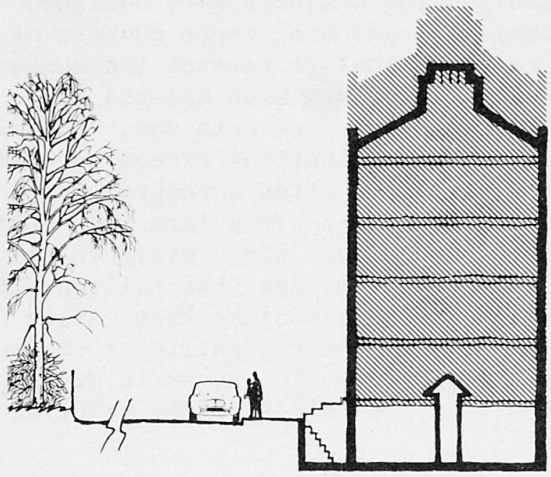


Fig.14. Typical city square.

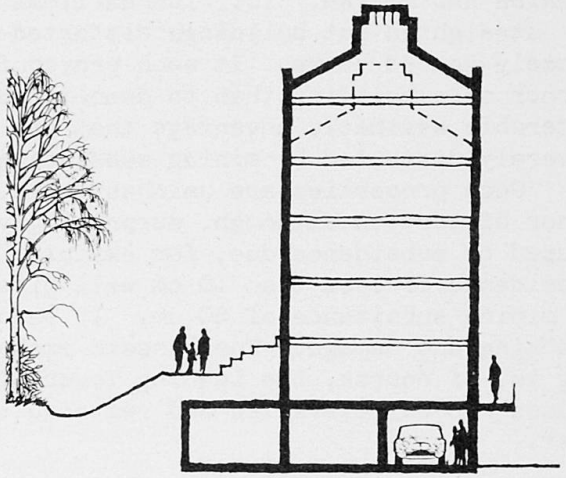


Fig.15. Parking with rear access under building.

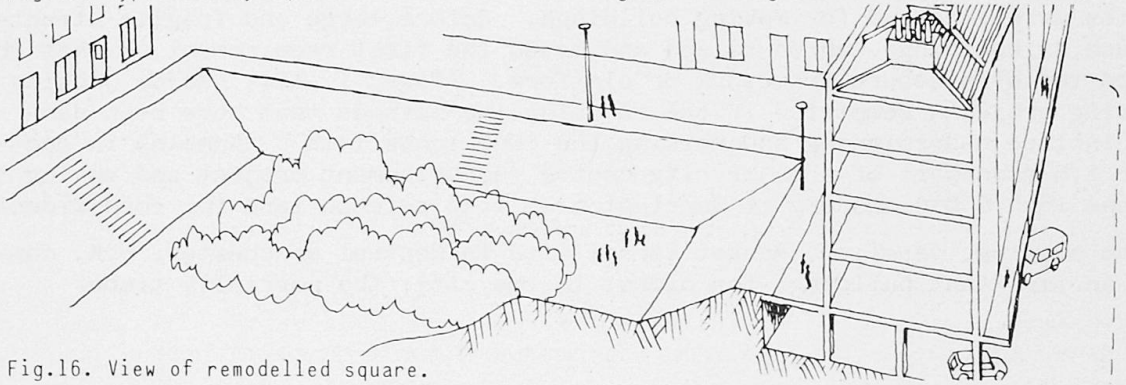


Fig.16. View of remodelled square.

framed and partially brick, Grade II listed, 14th C Shambles, over 30 m long and up to 4 storeys high. The proposed new development included a level pedestrianised area 1.5 m above the ground floor of the Shambles. The whole site was also to be excavated 7 m for two levels of underground access and parking with the entrance ramp passing underneath the Shambles. Pynford detailed and constructed an R.C. frame under the building, dug eight supporting piers 9 m down to the underlying rock and jacked the building up 1.5 m from these to the required new level. The jacking was controlled to within $\pm$ 3 mm to prevent cracking and limit stresses in the R.C. frame during the lift. The whole structure being lifted weighed 1000 tonnes. (Ref. 5).

13. The early 18th C. brick Old Academy in Warrington, UK, stood on the site of a proposed road widening scheme at the north end of the Mersey Bridge in the town centre. It is particularly important for it was the first purpose built non-conformist academy for higher education in the UK. It had been condemned as unsafe. To begin the work, cracked brickwork was injected with epoxy resin and flat steel bars were bonded on. An R.C. frame was then constructed just below ground floor level. Beneath this three further beams were formed to provide a running track and extended onto the new site and linked with cross beams and an R.C. slab to form an inverted cellular raft foundation. Jacks were then fitted in the 400 mm gap left between the upper and lower beams to lift the building 100 mm off the existing foundations. Grease skates running on thin steel sheets were then set under each jack and the building was winched to the new site. The move took about 20 minutes. (Ref.6)

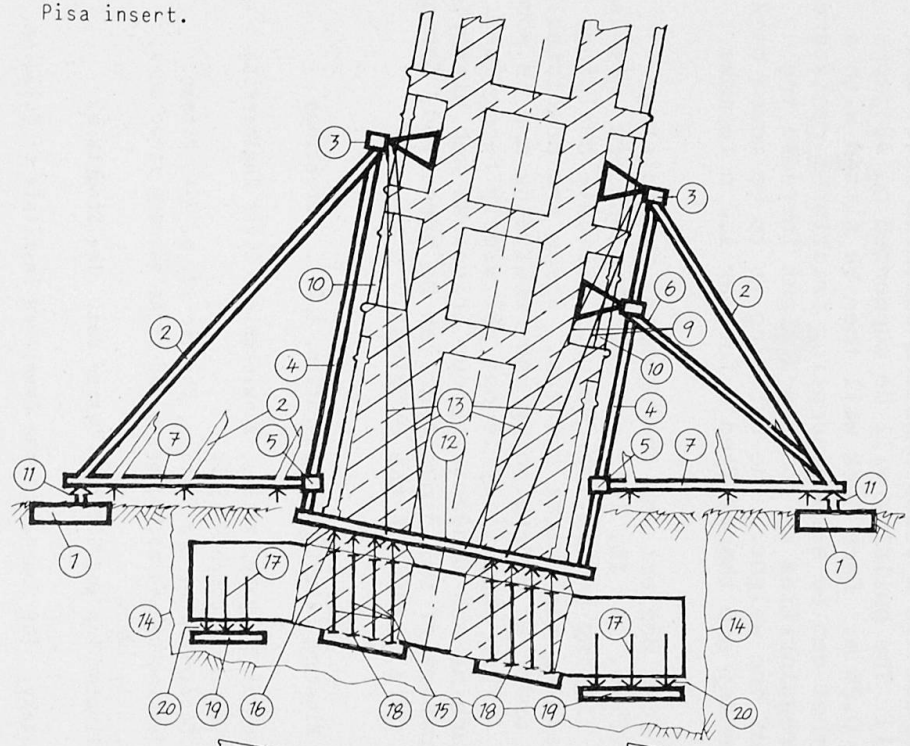
14. At the time of writing Pynford is engaged upon lifting and moving a 2500 tonne building in Norway. (Fig. 13). It is 11 m x 24 m on plan with basement walls and four upper floors that are early examples of reinforced concrete. The walls above basement level are brickwork 3 storeys high and the steep timber framed roof contains the two topmost levels of this 6 storey building. An R.C. raft 0.7 m thick has been constructed 1.2 m below existing basement level and extended 6 m beyond the rear wall. The building will be supported on 42 jacks arranged in 12 groups and lifted 0.25 m. Each jack will then be fitted with a 75 tonne grease skate, a device that can be slid generating frictional resistance of about 0.1% although slight irregularities on the track might increase the restraining forces to over 0.5% of the weight. The building is to be moved back 5 m to release land for road widening and then lifted a further 1.2 m to make space for a basement car park.

15. The insert illustrating the Pisa project and the proposal for raising buildings around a city square, (Figs. 14, 15, 16), are self explanatory. It is safe and practicable to consider lifting and moving the most delicate buildings such as churches or palaces containing many stone columns. Costs are predictable and are likely to be of the same order as underpinning and rebuilding and in some cases may be significantly less. This capability is not yet widely recognised. It may alter thinking about replanning and modernising our priceless and historic city centres, and increase the numbers of buildings that can be preserved, for a reconstructed building is only a pale shadow of the original.

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Pisa insert.



1. Construct a ring of shoring foundation (1)

2. Erect raking struts (2) supporting a compression ring (3) from which hangers (4) support a tension ring (5) provide additional struts (6) on low side.

3. Tie the bases of the raking struts with ties (7) to the tension ring and pack between this ring and the drum of masonry (8) forming the base of the Tower. Provide secondary struts (9) between the tower columns (10) at compression ring level and below.

4. Complete the shoring by preloading the temporary foundations with jacks (11) to support the Tower and transfer pressure from the overloaded low side of the foundation to the understressed high side. It is now safe to cut into the Tower foundations.

5. To complete the temporary support construct an RC slab (12) approx 0.75m thick beneath the Tower walls just below original ground level using the Pynford stooling system. Link this to the tension ring (5) with extensions of the ties (4) and with tension cables (13) attached to the compression ring.

6. Stress the cables (13) and increase pressure in the jacks (11) to transfer approx 5000 tonnes to the shoring foundations (1). Note that the Tower is lifted at base level in a steel cradle attached to an RC Base Slab constructed approx 3.5m above the underside of the existing foundations.

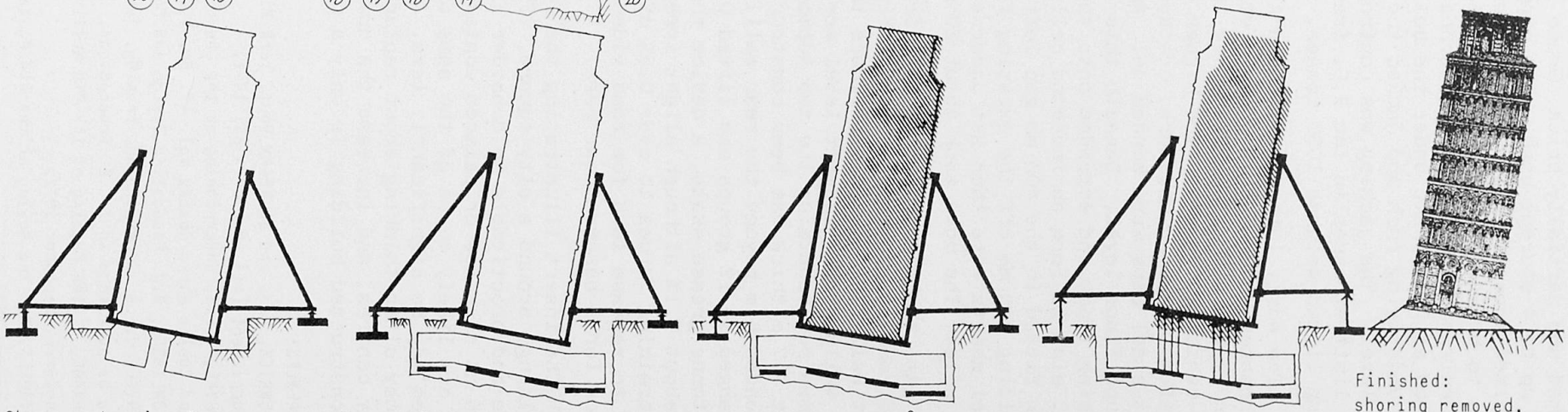
7. It is now safe to construct a new foundation raft approx 3 mm thick to replace the existing foundations and reduce ground pressures to a safe load.

8. Form an impervious ground curtain (14) around the Tower and underpin in sections approx 3 m wide propping the slab (12) with struts (15) pre-loaded with jacks (16).

9. The new raft (17) would be of cellular construction bearing partly upon the heavily compressed subsoil (18) beneath the existing foundations and a new RC foundation ring (19) constructed upon the uncompressed soil surrounding the Tower (10). Use jacks (11) and (16) to reduce the inclination of the Tower approx. 2.25 m to ground level as originally constructed, pack up and remove jacks (16).

12. Using secondary jacks (20) load the foundation ring (19) until consolidation becomes minimal, at the same time, reducing the load in jacks (11) to transfer all the Tower weight to the new foundation raft.

13. Grout the cables (8) to link the Tower to the new foundation. Remove shoring and make good the paving surrounding the Tower.



Shore up Leaning Tower.

Form raft.

Reduce inclination 1^{0}

Raise 2.25m

Finished:
shoring removed,
paved up to Tower.

Strengthening and/or Repairing of Existing Structures

Renforcement et/ou réparation de structures en béton armé

Verstärkung und/oder Reparatur von bestehenden Bauwerken

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SUMMARY

Three cases of reinforced concrete structures presenting unusual problems to the engineer are presented:

— strengthening of a loading dock to meet the requirements of a massive increase of live load
— reversal of the construction sequence of building when it was not possible to proceed in the normal manner: supporting columns poured after the slab
— repair of a severely damaged reinforced concrete column of an eight storey building.

RESUME

Trois cas de structures en béton armé présentant des problèmes peu communs sont présentés:

— renforcement d'un quai de garage pour quintupler la capacité portante
— inversion des étapes de construction par suite d'obstacles inattendus: des poteaux ont du être coulés après le plancher qu'ils devaient supporter
— réparation d'un poteau sévèrement endommagé d'un édifice de 8 étages.

ZUSAMMENFASSUNG

Drei Fälle von Stahlbetonbauwerken, die dem Ingenieur besondere Probleme stellen:

— Verstärkung einer Verladerampe für eine massive Erhöhung der Nutzlast
— Umkehrung der Reihenfolge im Bauablauf: Giessen der tragenden Stützen nach der Erstellung der Platte
— Reparatur einer stark geschädigten Stahlbetonstütze eines Gebäudes mit acht Stockwerken.

1. INTRODUCTION

The professional Engineer is often confronted with problems different from the usual one of conceiving and designing a new structure.

An outline is here presented of three such cases successfully solved:
- Increasing the load capacity of an existing structure;
- Reversal of the construction sequence of building when, due to special circumstances, it was not possible to proceed in the normal manner;
- The repair of a severely damaged reinforced concrete ground floor column of an eight storey building.

2. STRENGTHENING OF A REINFORCED CONCRETE STRUCTURE FOR MUCH HIGHER THAN DESIGN LOADS

2.1 Problem Description

On completion of a reinforced concrete structure for a loading dock, the management of the Company for which it had been designed and built, realised that the previously stipulated live load of 5 tonne trucks was far from sufficient. The owner decided that the loading dock should support the traffic of 25 tonne trucks.

The first impression was that such a massive increase in design load could only be met by demolition and reconstruction of the area in question. However this was ruled out due to excessive cost and the time factor.

2.2 Planning

Rankine and Hill was commissioned to produce a scheme for the strengthening of the recently finished structure in order to support the new live load.

The basic requirements to be met were:
- reasonable cost
- limited time for the execution of the works
- if possible, no propping to be used under the existing loading dock in order to allow other urgent activities in this area.

All these conditions were satisfied.

2.3 Existing Structure

The existing structure was a flat plate reinforced concrete floor with drop panels supported on concrete column, the slab thickness varying from 250mm to 400mm, to achieve falls to sumps. The structure was analysed by computer firstly for dead load only and then for live load. (Fig. 1). The live load consisted of trucks (D.M.R. H10 loading, equivalent to 10 tonne trucks) passing on the slab, two trucks to each dock. This loading pattern resulted in excessive overstress. However, the slab as constructed was adequate for trucks of up to 8 tonne (an acceptable overstress was indicated for this condition).

2.4 Design Procedure

The approach to the problem showed that it was necessary to keep a close correlation between design and execution in order to ensure that the design assumptions were effectively translated into practice.

To increase the capacity of the slab it was proposed to add 110mm of reinforced concrete on top of the existing slab, keyed to the existing slab by scabbling the contact surface and using a wet to dry epoxy. Special attention was given to this detail to insure a sufficient bond to resist the horizontal sliding shear (Fig. 2).

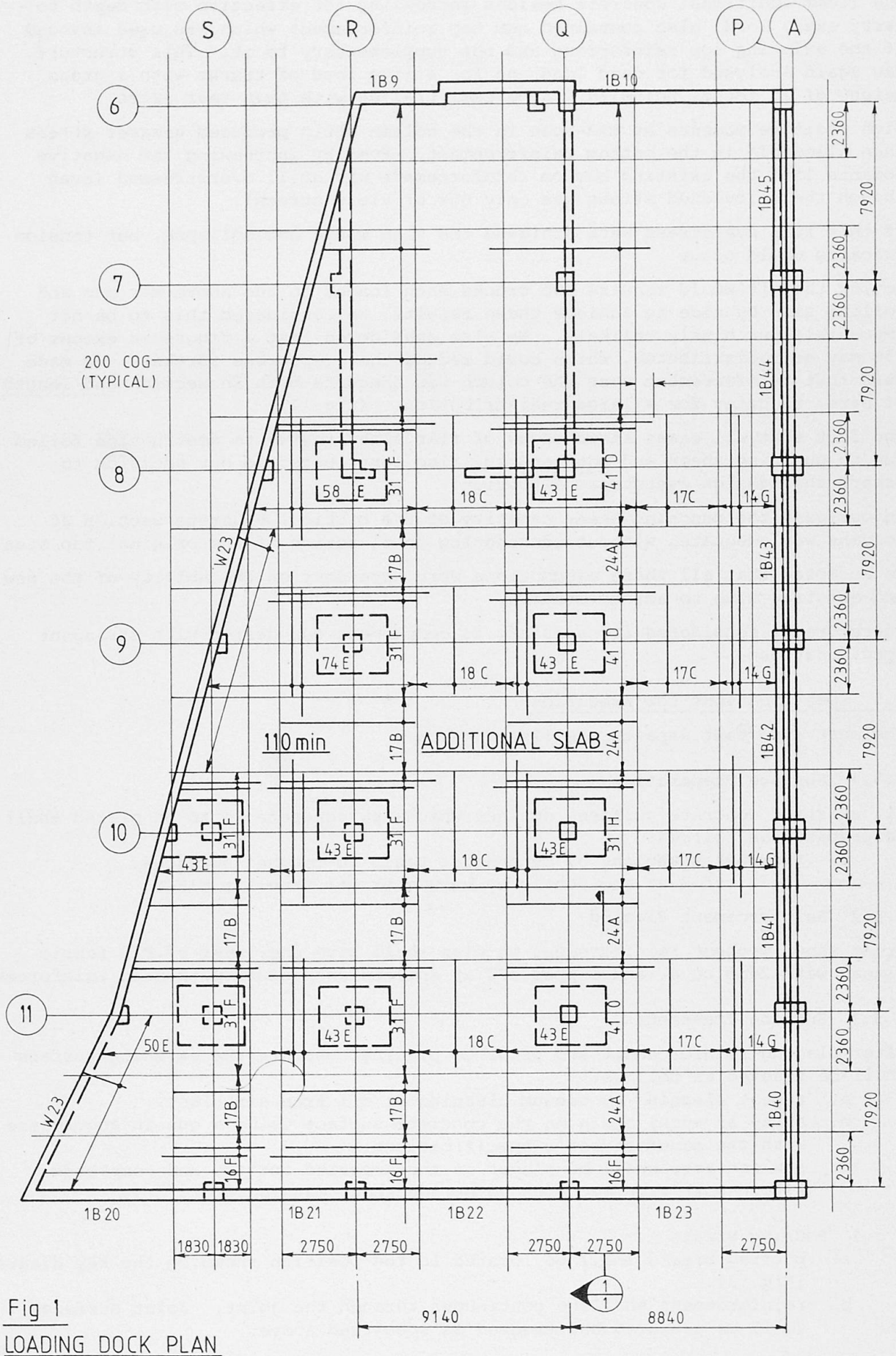


Fig 1
LOADING DOCK PLAN

The 110mm additional concrete besides increasing the effective slab depth to carry extra load, also contained new top reinforcement which was used instead of the existing top reinforcing and not supplementary to it. This structure was again analysed for dead load and for a live load of trucks with a gross weight of 25 tonnes being an H type semi trailer with twin rear axles.

High positive moments at mid-span in the column strip produced greater stress than allowable in the bottom reinforcement. Even by increasing the negative moments 15%, the existing bottom reinforcement was still overstressed (even though the calculated stress was only 65% of yield stress).

If this full overstress were achieved the slab would not collapse, but tension cracking would occur.

Noting that it would require two trucks each loaded to the above maximum and rolling side by side to achieve these results, we considered this to be not impossible but highly unlikely. We also considered that a figure in excess of 15% may be redistributed, which could reduce the overstress further. We made sure that reinforcement over the column was adequate both in section and length of bars, to cater for a large redistribution. (Fig. 1).

The fact that all cases known to us of flat slabs tested to destruction failed due to punching shear and not bending, also contributed to our decision to accept this design overstress situation.

In our case the punching shear capacity of the critical concrete section at columns was adequate, without considering dowel action of the original top steel.

We accepted that all these assumptions were dependent on the ability of the new and existing slab to act together.

Therefore we considered it necessary to complement our design with stringent specifications.

2.5 Specifications for Execution

The most important aspects are listed below.

2.5.1 Surface Preparation

All existing concrete surfaces against which new concrete is to be placed shall be prepared as follows:

a. thorough scabbling by mechanical tools to roughen surfaces.
b. clean off dust and debris by vacuum or oil free air blast.

2.5.2 Reinforcement Placing

Areas shown without reinforcement on plan shall have one layer of F72 fabric placed with 25mm cover and lapped 300 at splices and joints with main reinforcement.

2.5.3 Surface Treatment

After placing reinforcement and prior to placing concrete the existing surface shall be treated as follows:

a. repeat cleaning by vacuum cleaning or oil free air blast.
b. spray an epoxy resin on the concrete surface using a gun in accordance with the manufacturer's specifications.
c. new concrete shall be placed on the prepared surface and completed within work time recommended by the epoxy manufacturer.

2.5.4 Pouring Breaks

a. pouring breaks shall be located in the position shown on the key diagram [Fig. 3].
b. reinforcement shall be continuous through the joint. Joint surfaces shall be prepared and treated as specified above.

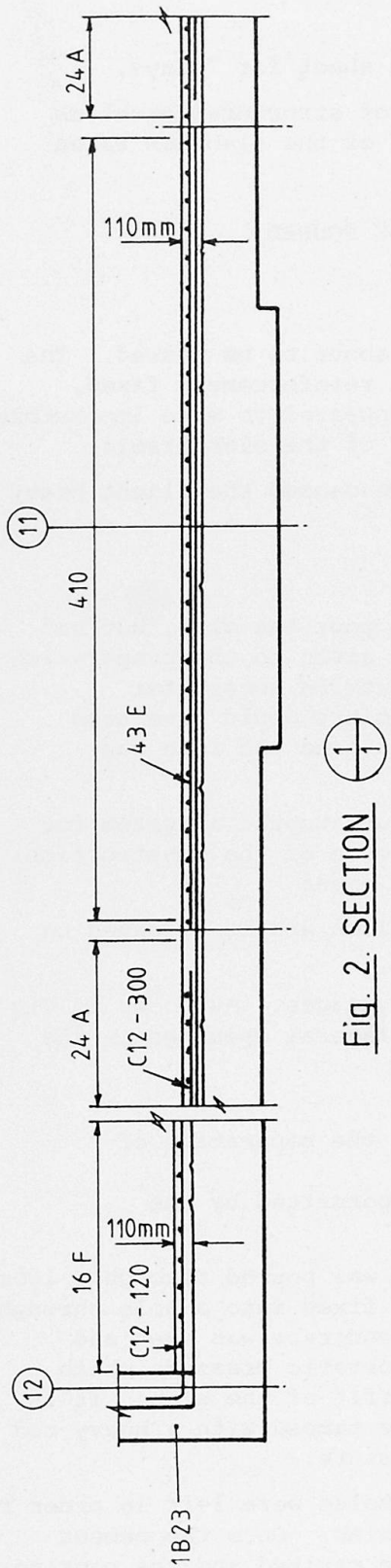


Fig. 2. SECTION 1/1

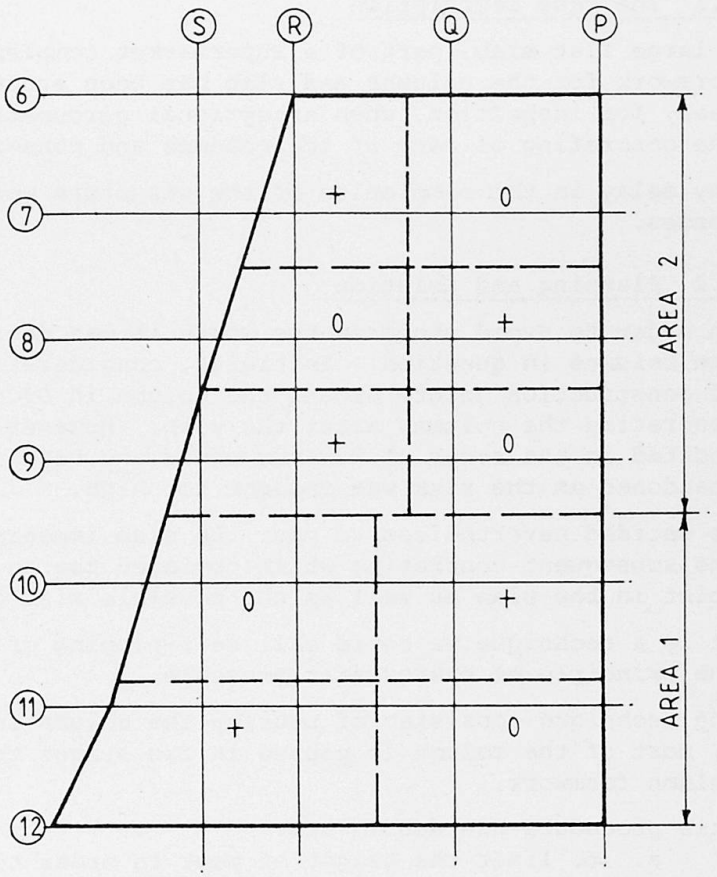


Fig 3 KEY DIAGRAM
(Pouring Sequences)

2.5.5 Curing

Curing was stipulated to be by covering with polythene sheet for 7 days.

After seven years of intensive use there are no signs of structural problems and contrary to expectations no cracking of the soffit of the slab has taken place.

3. THE SOLUTION TO A SITUATION WHERE COLUMNS HAD TO BE POURED AFTER AN EXTENSIVE SLAB

3.1 The Case Description

A large flat slab, part of a Supermarket complex, was about to be poured. The formwork for the columns and slab has been erected and reinforcement fixed, ready for inspection, when exceptional circumstances appeared to make impossible the concreting of many of the columns and consequently of the slab itself.

Any delay in the completion of the structure would have caused the client heavy losses.

3.2 Planning and Solution

In order to avoid stopping the works it was decided to pour the slab, but not the columns in question. Initially, consideration was given to the preparation of construction joints around the column in order to provide access for concreting the columns after the slab. However such joints would have been located in the zones of maximum shear and bending stress and the idea was abandoned as the risk was thought too high.

We decided nevertheless to pour the slab immediately but adopted a system for the subsequent concreting which obviated the inconvenience of the construction joint in the slab as well as the possible risk of settlement.

It is a technique we could call self-pinning of the column and it is based on the principle of hydrostatic pressure.

The technique consisted of pouring the column in three stages. As shown in Fig. 4, most of the column is poured in two stages through lateral openings in the column formwork.

This procedure has double aim:

a. to limit the height of pour in order to avoid the separation of aggregates.
b. to allow after each pour as long a period as permitted by the circumstances, for shrinkage to take place.

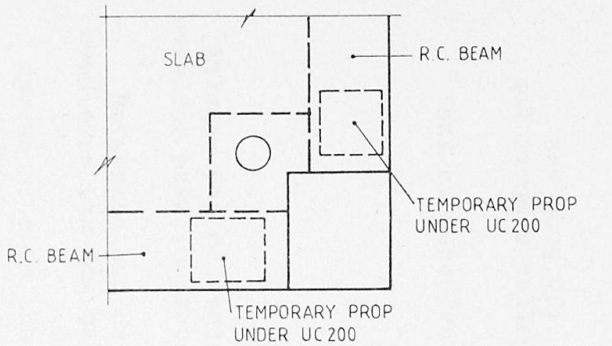
The third and shortest part, at the top of the column, was poured through a 100mm diameter pipe projecting some 900mm above the slab and fixed into a hole through it (Fig. 4). For this third and last pour non-shrink concrete was used and poured to the top of the pipe forming a column of hydrostatic pressure which produced an upward pressure of concrete against the soffit of the slab. It is recommended that the wet concrete in the pipe should be tamped with a heavy rod during the pouring in order to increase the upward pressure.

At the very top of the column's formwork several 20mm holes were left in order to avoid the formation of air pockets between column and slab. Once the cement slurry started to emerge through these holes, they were caulked and the pouring of the pipe finished as described before.

Twenty four hours later, the pipe and concrete above the slab were removed. After the usual time for curing, the whole area was stripped.

No settlement of the structure has been observed.

FIG. 6



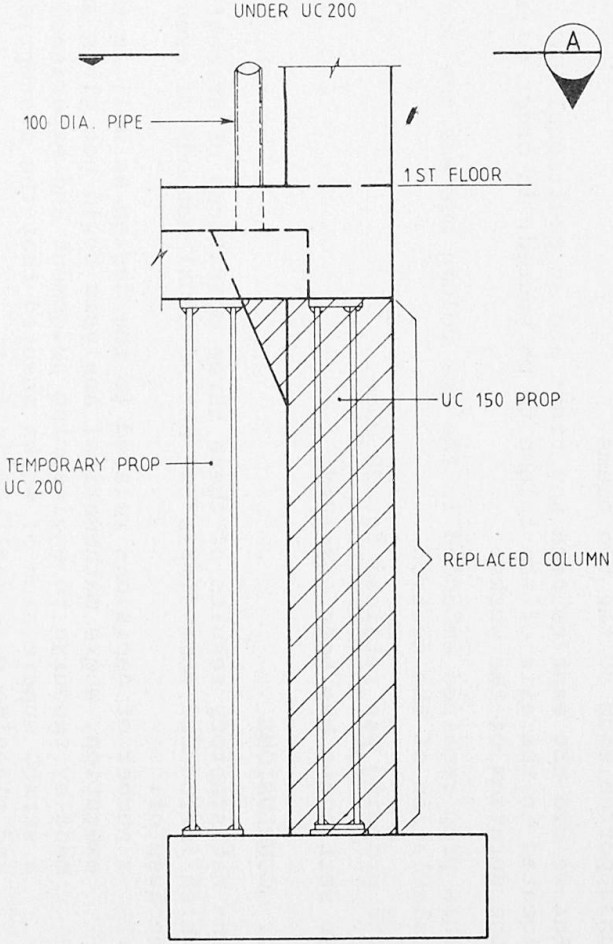


FIG. 5

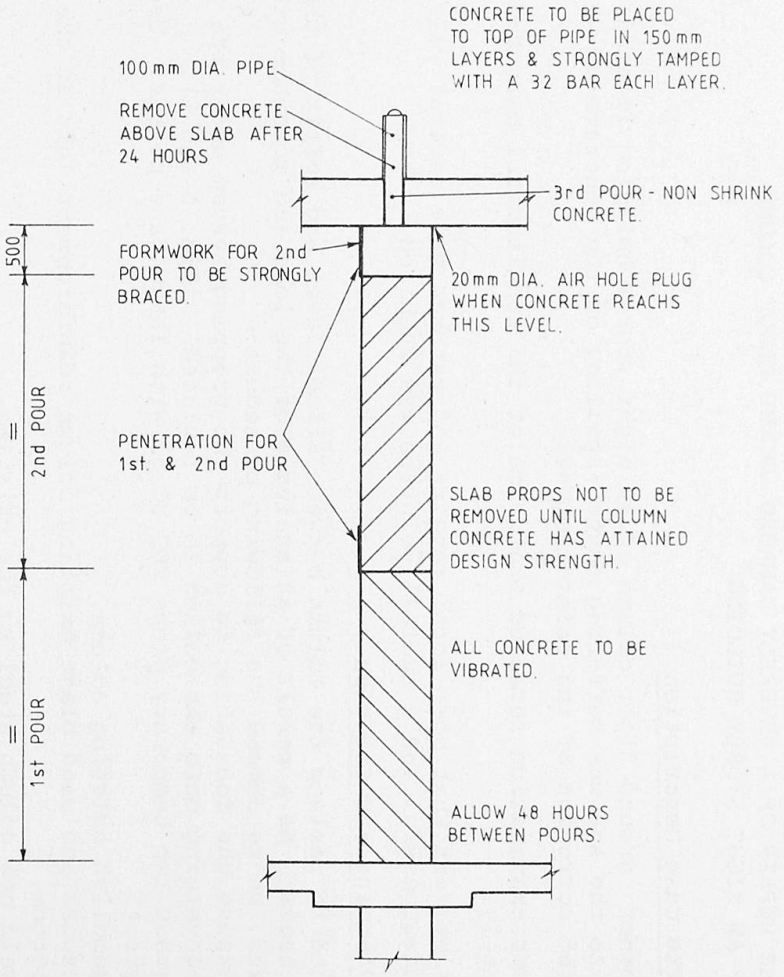


Fig. 4

4. THE REPAIR OF A SEVERELY DAMAGED GROUND FLOOR COLUMN OF AN EIGHT STOREY BUILDING

4.1 The Case Description

The corner ground floor column of an eight storey building located in Newcastle close to the sea was suffering severe spalling of the concrete as well as advanced corrosion of the reinforcement.

A closer examination revealed that some of the longitudinal bars had rusted completely.

It was evident that some local repairs and patches made several years before had been unsuccessful and a radical solution was required.

4.2 Planning and Solution

We opted to replace the column between street level and soffit of the first floor beams. As a result of an analysis of the load and possible risks involved, we recommend the following procedure:

- expose the foundation to support the propping system under the two beams converging onto the column to be replaced. (Fig. 5 and 6).
- erect two temporary props 200 UC 52 with large base plates. Use expanding grout.
- demolish existing column.
- expose and sand blast existing column reinforcement left at the top and the bottom.
- weld new column steel to the existing.
- replace column.

The replacement of the column was carried out using the same technique described in Chapter 3, with additional precautions.

The calculated load of the corner column over the 1st floor could have produced an excessive shear in the converging beams. The real situation, due to the exterior brick walls in all panels up to the eighth floor and roof was reassuring, because it was reasoned that these panels would act as vertical shear walls and not significant load would flow through the column on the provisionally unsupported ends of the two beams.

But we did not want to take any risks and an additional U.C. 150 prop was located in the axis of the column to be poured, in order to offer support for the duration of the works.

This prop remained embedded in the new column but was not considered in the calculation of the section.

The propping was retained for 28 days.

No settlement has been observed.

5. CONCLUSIONS

The satisfactory results of these three operations of strengthening and repairs, which introduced some simple but not so usual methods of execution, were achieved because of:

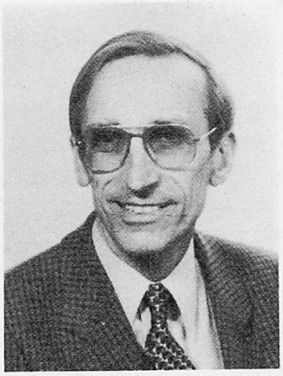
- a number of decisions related to the design as well as to the stages of execution, where mathematical analysis could not given an answer, have been made by recourse to engineering judgement and experience.
- a strict supervision of works assured that the assumptions made in design and planning were realised in fact.

Réparation de la couverture en voile mince d'un bâtiment

Instandsetzung eines Schalendaches

Repair of a Shell Roof Structure

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RESUME

La couverture en voile mince du bâtiment de l'émetteur d'Europe 1 au Felsberg (Sarre) a subi dès sa construction en 1954 un accident d'où sont résultées des dispositions assez originales pour la précontrainte. Une nouvelle précontrainte vient d'être effecutée, se substituant entièrement à la précontrainte d'origine, ce qui a permis de voir l'état de conservation des câbles et de montrer les infinies ressources de la précontrainte en matière de réparation.

ZUSAMMENFASSUNG

Schon bei der Errichtung des Schalendaches der Rundfunksendestation «Europe 1» auf dem Felsberg (Saarland) 1954 ereignete sich ein Missgeschick. Dies führte zu einer ziemlich eigenartigen Auslage der Vorspannung. Im Zusammenhang mit vor kurzem durchgeführten Reparaturarbeiten ist die Vorspannung völlig ausgewechselt worden, wobei der Zustand der Kabel genau erkennbar wurde. Die Arbeiten haben ferner die fast unbegrenzten Möglichkeiten der Vorspannung für Baureparaturen nochmals gezeigt.

SUMMARY

The shell roof of the broadcasting station Europe 1 building, on Felsberg suffered an accident during erection in 1954 which led to a quite unusual prestressing arrangement. During recent repair works new prestressing has been introduced in order to replace entirely the original. The replacement has fully disclosed the state of conservation of the original tendons and has once more demonstrated the versatility of prestressing for structural repairs.

Les émetteurs d'EUROPE N° 1 sont disposés dans un bâtiment couvert par un voile mince en béton précontraint, construit en 1954.
Une faute de conception, sur laquelle nous reviendrons plus loin, devait entraîner la déchirure du voile encore sur cintre, lors des opérations de précontrainte initiale, en juillet 1954 ... d'où une première réparation sous la direction d'Eugène FREYSSINET et d'Yves GUYON.

Depuis cette intervention, la couverture s'est bien comportée et sans doute n'aurait-elle pas encore attirée l'attention sans un accident survenu en 1980, à Berlin, sur un voile mince présentant quelque analogie avec celui du Felsberg. Ceci est à l'origine des travaux de réparation que nous allons décrire, sur une structure riche d'enseignements, car la réparation a permis de mesurer les limites des techniques utilisées en 1954.

Le bâtiment est constitué par une série de poteaux supportant une ceinture en béton armé à laquelle est fixé un voile de couverture en béton précontraint, de 5 cm d'épaisseur. Les poteaux limitent une aire en forme de coeur, inscrite dans un rectangle de 86.50 m x 46 m. Entre les poteaux, les parois sont uniquement constituées par des vitrages (fig.1).

La ligne moyenne de la ceinture coiffant les poteaux est contenue dans deux plans symétriques par rapport à l'axe et inclinés de 20 % sur l'horizontale : l'arête du dièdre formé par ces deux plans est inclinée, les niveaux de cette arête étant à une hauteur de 4.50 m à une extrémité de l'axe et de 9.50 m à l'autre. Dans l'ensemble, cette ligne moyenne a la forme d'un coeur tracé sur un plan qu'on aurait plié par son milieu.

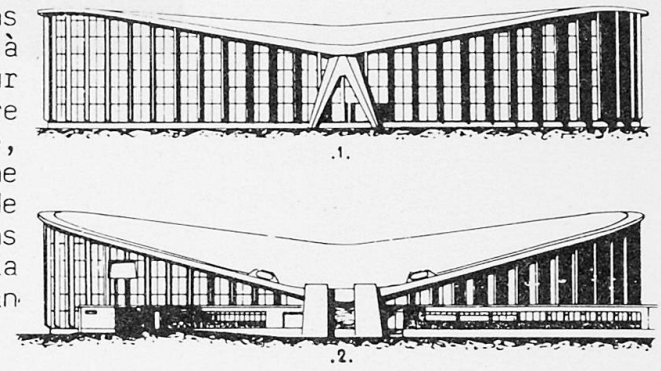


Fig.1 Façades du bâtiment

Le voile est précontraint par des câbles disposés perpendiculairement à son axe. La surface du voile est générée par des paraboles correspondant au tracé des câbles, tandis que les coupes parallèles à l'axe de symétrie du bâtiment présentent une courbure inversée très variable en raison de la forme de la ceinture.Les efforts divers s'exerçant sur cette ceinture sont équilibrés grâce à 6 tirants disposés en éventail à partir de la pointe du coeur (fig.2).

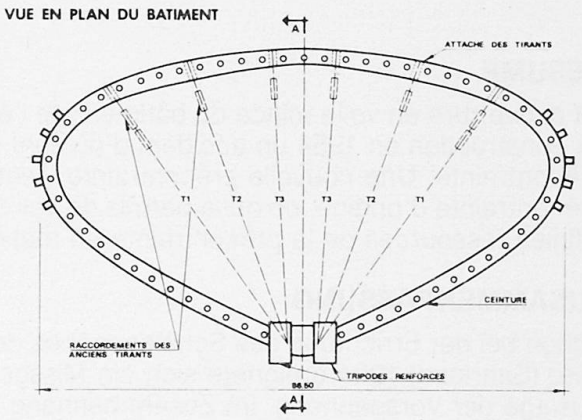


Fig.2 Disposition des tirants

1. ACCIDENT ET REPARATION DE 1954

Dans le projet initial, les tirants étaient passifs. De ce fait, lors du décintrement du voile par mise en précontrainte des câbles longitudinaux, le voile s'est déchiré parallèlement aux câbles, en raison de la déformation de la ceinture sous l'effet de la précontrainte : les tirants non tendus ne pouvant s'y opposer sans déformation excessive pour le voile.

Après cet accident, les tirants d'origine, constitués de 120 à 340 fils Ø 5, ont été raccordés à l'une de leurs extrémités par des épissures injectées au mortier, sur des câbles bouclés en U sur selles en béton. Celles-ci, prenant appui sur la ceinture, à l'extérieur du bâtiment, ont été écartées de 10 cm environ par l'action de 4 couches de vérins plats, afin de conférer la tension requise, de l'ordre de 500 MPa (pour des aciers de résistance à rupture 1400 MPa à cette époque). Rappelons qu'il s'agissait de câbles libres qui seraient donc soumis aux variations de contraintes consécutives aux variations de charges sur la couverture.

Quant aux câbles longitudinaux, des 12 Ø 5 à entr'axe moyen 0.30 m, on devait les inclure dans des nervures non prévues au projet initial, nervures obtenues par écartement des panneaux de fibres agglomérées sur lesquelles le voile était bétonné (une lère couche de panneaux jointifs assurant une isolation thermique générale, cachant entièrement le voile en béton). Le logement des câbles longitudinaux dans les nervures laissait la place dans le voile mince de 5 cm pour un ferraillage de répartition assez dense. Les panneaux de fibres étaient suspendus au voile par des épingles en acier galvanisé Ø 3 mm (fig.3).

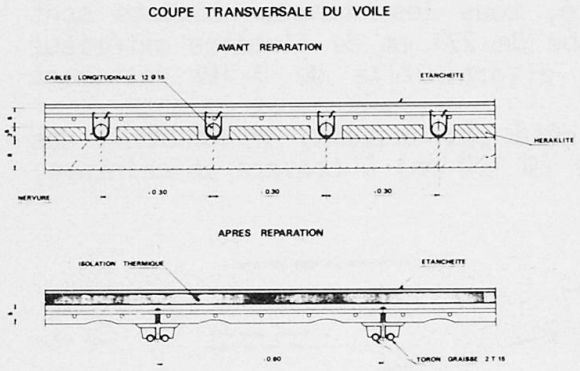


Fig. 3

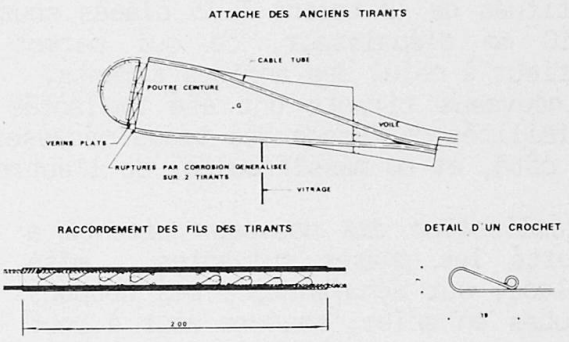


Fig.4

2. ETAT DE LA COUVERTURE EN 1980

Les premiers examens faits ont décelé une corrosion avancée avec rupture d'un certain nombre de fils d'un tirant, en partie basse du joint, côté selle de traction (fig.4). Par contre, quelques sondages faits en intrados de la voûte donnaient une vue rassurante des câbles longitudinaux, laissant à penser qu'il y aurait tout au plus à réinjecter ces câbles, opération maintenant supposée classique.

Toutefois, nous avons fait des investigations complémentaires nécessitant la dépose de quelques panneaux de fibres avant de fixer le détail des réparations, et voici ce que nous avons trouvé :
- corrosion avancée des épingles supportant les panneaux de fibres,
- aspect médiocre des nervures d'enrobage des câbles de précontrainte,
- corrosion des gaines apparentes en fond de nervure,
- corrosion superficielle mais assez généralisée des fils de précontrainte.

Par contre, le voile du béton s'est révélé très sain.
Le client ayant manifesté le désir d'une véritable "remise à neuf" de la structure, il a été décidé une substitution complète des câbles de précontrainte et des tirants, et l'exécution d'une protection thermique au-dessus du voile, afin de réduire les risques de condensation interne.

Tous ces travaux devaient se faire au-dessus des émetteurs en exploitation, avec des consignes strictes de sécurité imposant la mise en place d'un échafaudage général de protection et de travail, la sous-face de la voûte étant à une hauteur variable de 5 à 17 m.

3. SUBSTITUTION DES TIRANTS

Cette opération comportait nécessairement, de façon transitoire, des efforts complémentaires sur la couverture, faute de pouvoir synchroniser précontrainte des nouveaux tirants et suppression des anciens.
Aussi pour limiter les sollications parasites liées à cette phase des travaux, nous avons retenu des "tirants précontraints", nettement moins déformables que des tirants ordinaires, et pour réduire leur encombrement et leur poids, la section comprimée est celle d'un tube en acier rempli de béton.

Dans ce système, une partie T de l'effort de précontrainte P est appliquée à la ceinture, tandis que le tirant est comprimé par P-T : la répartition de la précontrainte dépendant de la flèche de construction du tirant, des raideurs relatives de la ceinture associées au voile, et du tirant, enfin de son poids propre équilibré par la réaction de courbure sous l'action de la traction T.

Par souci de simplification et de sécurité, tous les nouveaux tirants sont constitués de 19 torons T 15 placés sous tube de 273 mm de diamètre extérieur et 10 mm d'épaisseur, ce qui permet un effort utile de 3 MN nettement supérieur à celui des anciens tirants.
Les nouveaux tirants ont été implantés à côté des anciens, en fonction des possibilités de forage des trous nécessaires (Ø 100 mm) à travers la ceinture, d'un côté, et du massif culée, de l'autre.

La réalisation des nouveaux tirants a comporté les phases suivantes : mise en place, sur échafaudage, des coupons de tubes en acier, soudure bout à bout de ces tubes, enfilage de la gaine centrale placée sur des supports assurant sa bonne position, bétonnage par injection entre tube et gaine, enfilage des câbles, mise en tension par paire de façon sensiblement symétrique.
Pour les tirants les plus longs (1 et 6 longueur = 45 m), la flèche était de 51 cm à la construction, et 26 cm au décintrement, variation de flèche accompagnée d'une rotation importante, 2 %, à la liaison avec la ceinture. Déformation maximale en plan de la ceinture 3 cm.
Aussi a-t-on réalisé cette liaison sous forme d'articulation (fig.5). Les articulations métalliques devenues inutiles en service ont toutes été bloquées par bétonnage.
L'opération de démontage des anciens tirants, libres vis-à-vis de la structure, semblait présenter quelques risques inhérents à l'importance de l'énergie élastique accumulée.

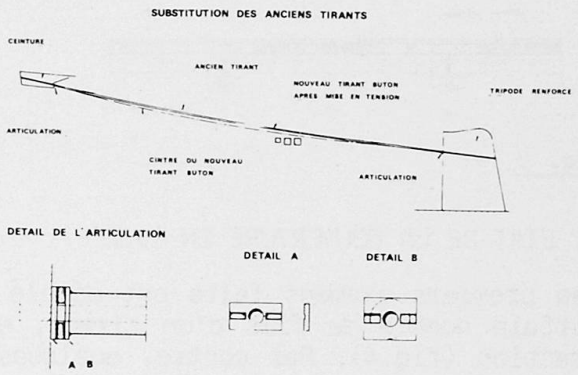


Fig.5

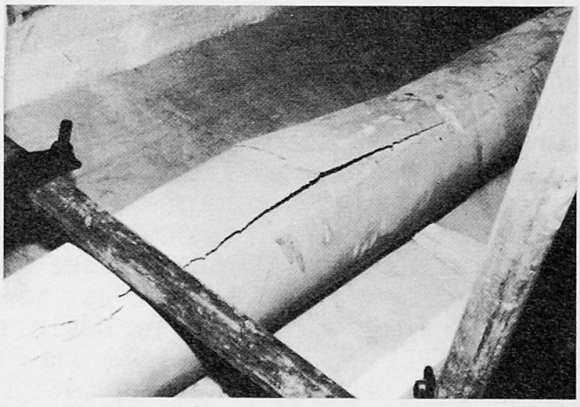


Fig.6 Eclatement du mortier à la coupe des tirants

En l'absence d'enrobage permettant des reports d'efforts entre fils, le tirant se comporte comme un paquet de fils pouvant être coupés successivement, et il suffit de "canaliser" le fouettement individuel des fils afin de protéger le personnel. Par contre, en cas de bonne adhérence des fils au mortier d'enrobage, celui-ci se met progressivement en compression par adhérence des fils coupés tandis qu'il y a surtension des autres fils. Deux instabilités peuvent apparaître : par compression, puis éclatement du mortier, ou par rupture simultanée du faisceau des fils non coupés.
Pour éviter ce dernier risque jugé le plus dangereux, nous avons dégagé le tirant de son mortier d'enrobage sur une longueur de 5 m, suffisante pour résorber si besoin par déformation plastique l'allongement élastique de 10 cm donné à la construction des tirants. La rétraction observée des premiers fils coupés, de l'ordre de 1,5 cm, correspond bien à la déformation élastique de ces fils sur la longueur libre. Cette rétraction est allée en croissant avec le report de compression par adhérence sur les parties latérales du tirant provoquant en fin de coupe un éclatement progressif du mortier (fig.6).

On a coupé les fils au disque de tronçonneuse et au chalumeau. Cette solution qui s'est avérée très pratique a été retenue après comparaison des 2 méthodes. La coupe des fils a permis de "retrouver" l'accourcissement élastique correspondant à l'allongement communiqué au tirant il y a 28 ans, à la précision des mesures.

4. SUBSTITUTION DES CABLES DU VOILE

La dépose, d'abord partielle de la 1ère couche de fibres agglomérées, montrait qu'on ne pouvait pas proposer une réparation valant neuf, et pouvant être garantie comme telle par les assurances, basée sur la pérénnité de la précontrainte ancienne, quand bien même aucune rupture n'était constatée sur ces câbles. En effet, les barrières de protection de ces câbles étaient devenues illusoires : (fig.7)
- l'enrobage des câbles était souvent pratiquement nul, ce qui ne se voyait pas sous les panneaux de fibres,
- ou avaient disparu : la gaine en feuillard mince aggrafé de 0.2 mm était réduite à l'état de dentelle, par corrosion,
- Quant à l'injection, elle était très fragmentaire, et il nous semblait impossible de rétablir une protection efficace sur des câbles soumis à une corrosion encore superficielle, mais assez généralisée, sauf zones correctement injectées.

Le mauvais état du béton constituant les nervures - plus semblable à un mortier qu'à du béton - nous incita, pour examen plus appronfondi, à couper une de ces nervures, avec son câble : la nervure, bétonnée en une 1ère phase entre les panneaux de fibres de la 1ère couche, était peu adhérente au voile lui-même, et pouvait assez facilement être détachée de façon progressive en profitant du plan de reprise.



Fig.7 Vue d'un câble bien injecté après dépose du 1er panneau de fibres

Comme au-dessus le voile béton lui-même était parfaitement sain, son épaisseur contrôlée par forage assez régulière, à savoir 5 cm ± 0.5 cm, nous avons réalisé la substitution totale de la précontrainte comme suit.

La force de calcul des anciens câbles 12 Ø 5 mm était F = 0.2 MN. Mais, pour des raisons diverses, dont essentiellement le frottement, leur "efficacité" était bien moindre, et les calculs refaits montraient qu'une précontrainte de 0.15 MN environ par nervure était bien suffisante, ce qu'on obtient actuellement avec un seul toron de 15 mm.

La substitution a consisté à déposer une nervure sur 4, puis à disposer en précontrainte externe sous la couverture 2 torons de 15 mm graissés sous gaine plastique aux emplacements disponibles, puis dépose de 2 anciennes nervures, remplacées par un nouvel ensemble de 2 T 15 sous l'une d'entre-elles, et enfin, dépose du dernier quart des anciennes nervures. En phase finale, nous avons maintenant un espacement de "bandes porteuses" sous la dalle double de l'ancien entr'axe, soit 2 x 0.30 m, compatible avec la résistance locale de la voûte.
Chaque bande est ainsi appuyée sur 2 torons T 15, que les réactions de courbure appuient sur le voile par l'intermédiaire de petites cales en béton de fibres régulièrement distribuées tous les 1.50 m, cette distribution ayant été retenue parce qu'il était impossible de créer un appui continu propre en intrados du voile, ne risquant pas de blesser la protection des torons (fig.3).
Une grande précision de pose a dû être recherchée afin d'obtenir un tracé effectivement régulier, tant en plan qu'en élévation : les déviations en plan risquant de créer des poussées transversales faisant glisser les cales, et les erreurs en élévation pouvant annuler la portance de certains patins.

Des forages ont dû être pratiqués dans les ceintures afin d'y ancrer les câbles porteurs au plus près possible des fibres extrêmes. Compte-tenu de la géométrie du voile, cette condition a nécessité, selon les cas, un seul forage avec ancrage en extrados, ou 2 forages avec une zone de raccordement en extrados de ceinture (fig.8).

Compte-tenu d'une géométrie à la fois complexe et mal définie, ceci a nécessité des relevés préalables aux forages, convertis sur ordinateur en instructions numériques avec sorties graphiques, câble par câble.

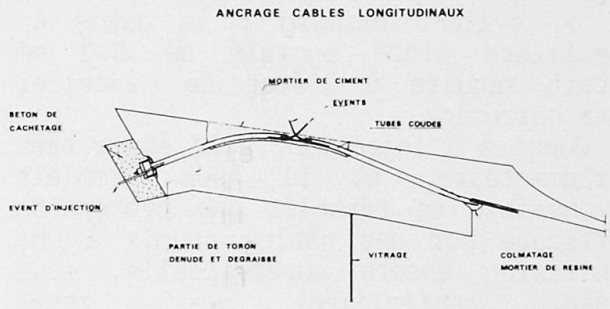


Fig.8

5. MODE DE CALCUL

Alors que la première réparation de 1954 avait fait appel à l'expérience des plus éminents ingénieurs de l'époque pour atteindre par des approches quelque peu intuitives l'état de la structure, la nouvelle étude a été conduite en utilisant les énormes possibilités de l'Informatique, d'abord pour les problèmes de géométrie déjà évoqués, mais aussi pour réétudier la stabilité de la coque mince de 5 cm, grâce à des calculs précis de déformabilité, tenant compte des effets du second ordre, et de quelconques défauts de géométrie.

Le modèle de calcul (fig.9) utilisant le programme HERCULE a donné des résultats bien vérifiés par les mesures in situ, partout où l'importance des déformations a permis ce type de contrôle, par exemple lors du décintrement des tirants.
Ces calculs ont montré que la structure était finalement très saine, malgré sa très grande légèreté, et justifiait pleinement la "remise à neuf" de la précontrainte.

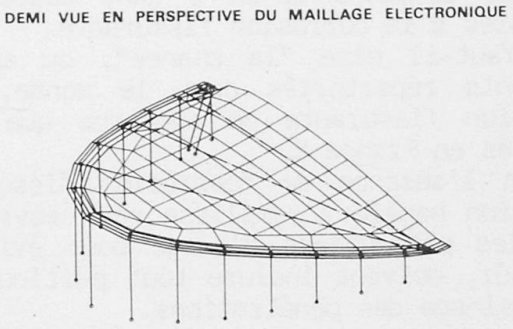


Fig.9

6. PERENNITE DE LA PRECONTRAINTE

Le premier accident, lors de la construction, avait conduit les intervenants de l'époque à "faire de leur mieux" lors de la réparation, en particulier au niveau des injections, opération que l'on savait déjà délicate. Aussi les tirants avaient été "bétonnés" en rigole en leur partie courante, interne au bâtiment : protection qui s'est révélée parfaitement efficace.

Le premier examen n'avait fait apparaître qu'un défaut grave de protection des câbles d'un tirant à sa pénétration en intrados de la ceinture, au droit du joint de vérinage : absence d'injection en une zone exposée en façade, d'où corrosion ordinaire, mais généralisée, des fils en acier Ø 5 mm, avec rupture d'un certain nombre d'entre-eux.
La dépose complète des tirants a montré qu'à un moindre degré ce défaut existait sur d'autres tirants, mais toujours au même endroit, particulièrement exposé quant aux venues d'eau, à la limite basse du joint et de la traversée de ceinture mal injectée.

Quant aux câbles longitudinaux, leur injection avait été faite avec de multiples évents, facile à mettre dans un voile aussi mince, ce qui aurait dû suffire à assurer la qualité de l'injection.

Au bout de 28 ans, nous n'avons pas trouvé de fils rompus malgré une corrosion plus ou moins généralisée, et bien sûr l'absence de corrosion dans les zones injectées (fig.10).
L'absence d'injection locale, mais fréquente, est essentiellement due à la non-étanchéité des gaines en feuillard très mince utilisées à l'époque (0.2 mm), et à l'enrobage souvent quasi inexistant des gaines, au contact des panneaux de fibres entre lesquels elles avaient été disposées : des plaques de mortier très analogues au "coulis" d'injection ont été trouvées en effet un peu partout entre les 2 couches de panneaux de fibres, mortier qui certainement a contribué à éviter la chute des panneaux malgré la corrosion des épingles de suspension, mais qui n'était plus où on supposait l'avoir injecté ...

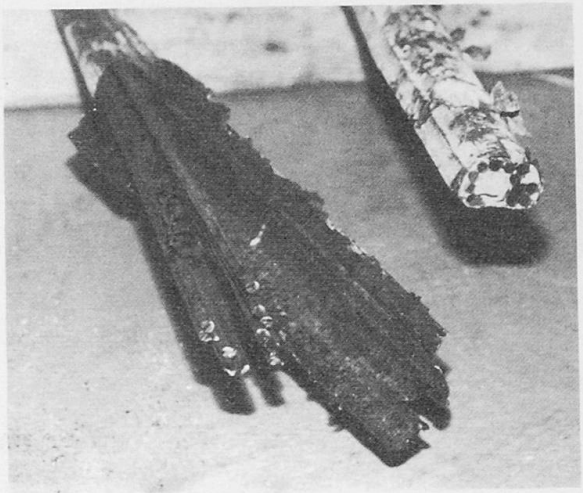


Fig.10 Etat des câbles 12 Ø 5 selon injection après 28 ans

On peut conclure qu'on a sans doute eu la chance d'avoir des aciers non sensibles à la corrosion fissurante.
Mais faut-il dire "la chance", ou au contraire, à la lumière de tous les incidents répertoriés dans le monde, dire que le phénomène inquiétant de corrosion fissurante ne concerne que certains aciers qui ont été très peu utilisés en France ?
Et, en l'absence de "corrosion fissurante", il faut du temps pour que la corrosion banale accomplisse son oeuvre destructive, du moins dès que l'on a pris les précautions d'usage pour éviter les venues d'eau, précautions qui, bien sûr, doivent inclure tout particulièrement l'examen des points singuliers au voisinage des pénétrations.

7. INTERVENANTS

Les travaux ont été confiés par EUROPE N° 1 à FREYSSINET INTERNATIONAL. Ils ont été réalisés par l'agence régionale de Nancy (STUP-FRANCE).
Le contrôle technique a été assuré conjointement par SOCOTEC et CEP (contrôle des études et des travaux), et par le Dr. Ing. Rudolf KUHN, de Sarrebrück, plus spécialement chargé de la sécurité du travail.
La réétude de la voûte a été faite par EUROPE-ETUDES-GECTI, en utilisant le programme aux éléments finis "Hercule", développé par SOCOTEC.

Je tiens à remercier l'ensemble de ces intervenants et les exploitants de la station pour leur participation active à la recherche des meilleures solutions, participation qui a permis de définir les méthodes, puis de les mettre en oeuvre en guère plus d'un an, l'ensemble des travaux ayant été faits au-dessus des émetteurs en exploitation.

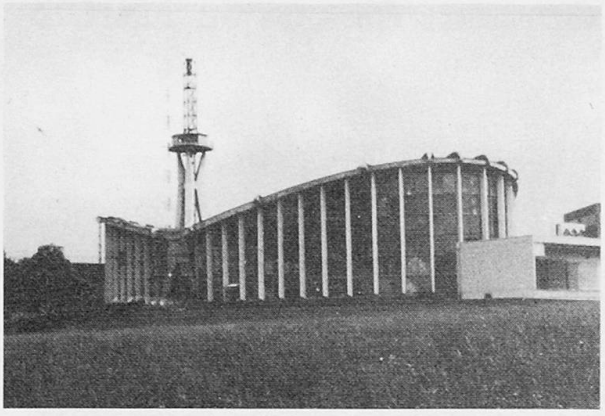


Fig.11 Vue extérieure

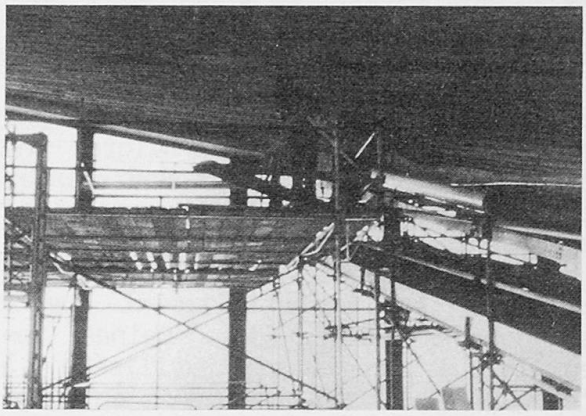


Fig.12 Vue intérieure partielle en cours de travaux

BIBLIOGRAPHIE

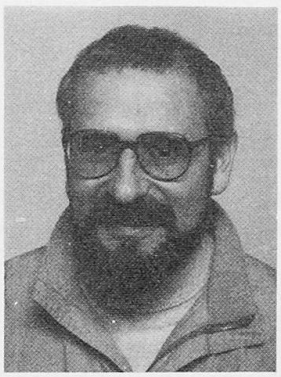
1. E. FREYSSINET, Incidents survenus au cours de la construction du bâtiment des émetteurs du FELSBERG (EUROPE N° 1) - TRAVAUX, février 1956.

Prestressing Cracked Masonry for Restoration

Renforcement de maçonnerie fissurée par des armatures précontraintes

Sicherung von gerissenem Mauerwerk durch Vorspannen

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Jürgen Haller received a diploma as civil engineer at the University of Karlsruhe in 1966. 8 years of practice in structural engineering were followed by 5 years of scientific research on prestressing masonry at the University of Karlsruhe. Jürgen Haller now works as a consulting engineer with emphasis on the restoration of old buildings.

SUMMARY

This article describes the aims and the technology of prestressing cracked masonry for restoration. A report follows about research conducted on the problem of imposing concentrated loads upon masonry and about long term measurements of the forces in built-in tendons and reinforcement bars. As a result, proposals are made for construction and calculation.

RESUME

Ce rapport décrit le but et la technologie du renforcement de maçonnerie fissurée par des armatures précontraintes. Des recherches sont présentées concernant le problème de forces concentrées dans la maçonnerie. De plus des mesures de longue durée sont faites pour surveiller les forces des armatures précontraintes et non précontraintes dans la maçonnerie. L'auteur présente les résultats de ses recherches et fait des propositions pour la construction et le calcul.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt zunächst Ziel und Technik der Sanierung von gerissenem Mauerwerk durch Vorspannen. Es folgt ein Bericht über Untersuchungen zum Problem der Einleitung konzentrierter Lasten in das Mauerwerk, und über Langzeitmessungen der Kräfte an Spannstäben und nicht vorgespannten Bewehrungsstäben, die im Mauerwerk mit Verbund eingebaut wurden. Als Ergebnis der Untersuchungen werden Vorschläge für das Konstruieren und Berechnen angegeben.

1. THE OBJECTIVE OF RESTORATION BY PRESTRESSING

Damages in historical and modern masonry buldings become visible as cracks in walls, arches and vaults. In many cases they just affect the appearance and the use of the building. But as soon as the central stone of a vault falls down or the shaft of a tower separates into several parts that move away from each other the stability is at stake.
If walls are thicker than 45 cm, which is usually only the case in historic buildings, it is possible to repair the cracked masonry by prestressing. The method leaves no visible marks on the outside. The substance of the masonry is preserved this way. It is strengthened internally to absorb future loads without further cracking.

2.TECHNOLOGY AND EFFECTS OF PRESTRESSING MASONRY

2.1 Technology of prestressing

First a hole is drilled lengthwise through the masonry, normally using drills 60 to 80 mm in diameter. A continous prestressing bar is inserted into the drillhole and anchored at both ends of the wall. A centre position in the hole is secured by distance pieces. Threaded rods can be used in handy, short pieces, an important advantage when working on a scaffold. They can be connected by couplers and be anchored at the end with a washer and steel plate. To transfer the prestressing force to the masonry an enlarged pressure area is needed. This is achieved by creating concrete cushions or steel anchor plates in pockets in the masonry that can be covered by plaster or by fair-faced masonry.

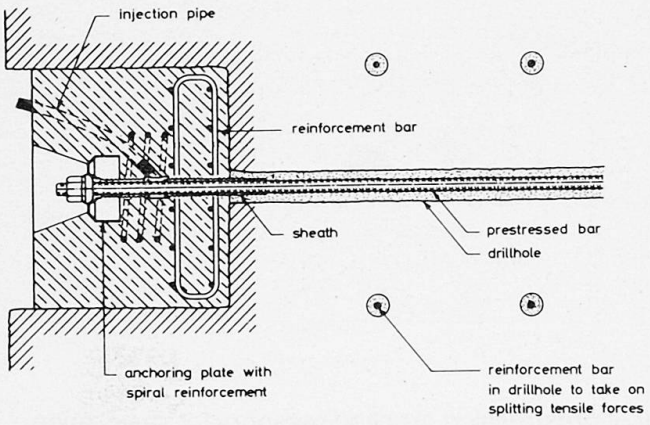
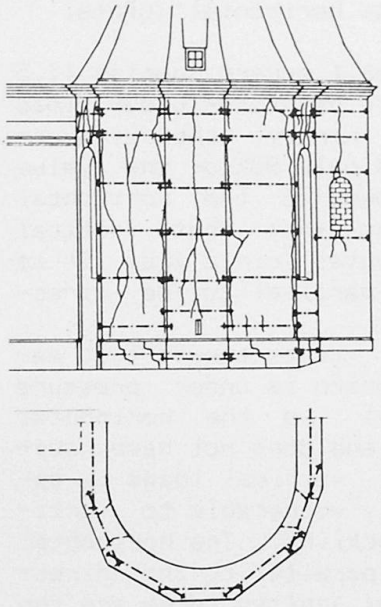


Fig.1 concrete cushion with anchorage

In order to pretension the wall the prestressing bar is anchored permanently at one end and tensioned at the other end by a hydraulic jack where it is then achored as well. To prevent the sensitive prestressing bar from corroding a cement-water paste is injected into the drillhole after pretensioning. The injection pressure ranging from 2.0 to 5.0 bar at the most, the cavities and cracks in the surrounding masonry are also filled. This way the wall receives a new mortar skeleton. More significant cracks and disturbances in the masonry bond, though, should be grouted before pretensioning.

2.2 Effects of prestressing

Cracks in masonry indicate that transverse tensile forces are active. If prestressing bars are arranged at a right angle to the direction of the cracks pressure is applied at this angle. This means that as long as the pretensioning pressure forces prevail further tensile forces can be absorbed. The constructional securing of crack damages (Fig.2) is not the only field of application for prestressing. It is also possible to improve the flow of forces and the transfer of loads within masonry by pretensioning. This can activate sheet action or bowstring action within walls to allow for openings or larger intervals between single supports (Fig. 3, 4).
In masonry that shows vertical or oblique-angled cracks prestressing bars are inserted horizontally. This case of prestressing parallel to the horizontal joints is by far the most frequent.



Prestressing at a right angle to the horizontal joints is applied much more seldom. It may be sensible to arrange prestressing bars eccentricly to correct off-centre load transfer if horizontal joints show one-sided gaps. Additionally, vertical pretensioning can increase the stability of walls that have too little load to withstand pressure from wind or to resist earthquakes.

It is a special advantage that prestressing bars have small cross sections and nevertheless can impose great forces. The types of steel used for this purpose have great material strength. Due to the spread of pressure behind the anchor plates each prestressing bar has an extensive area of action. Because of this, expenditure for drilling work can be kept to a minimum by using bars with a larger cross section and arranged at greater intervals.

Fig.2 constructional securing of crack damages

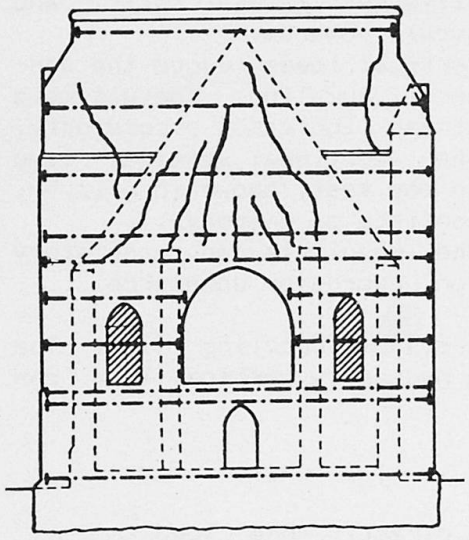


Fig.3 sheet action above an opening in a wall

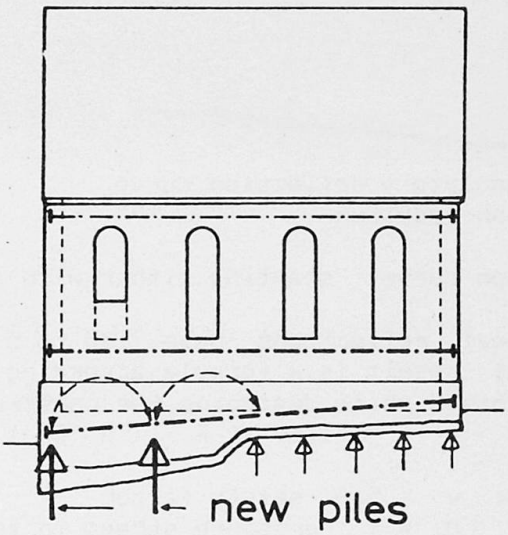


Fig.4 bowstring action

3. RESEARCH CONDUCTED ON PROBLEMS CONCERNING THE APPLICATION OF PRESTRESSING MASONRY

3.1 Imposing highly concentrated loads upon masonry

The size of prestressing forces is limited by the problems arising with imposing those concentrated loads upon masonry. Our knowledge of this problem is insufficient. The bearing capacity of masonry has been examined mainly for vertical load impact, that is at right angles to the horizontal joints. Prestressing, though, generally applies lateral pressure. This brought up the first question of interest for scientific research [1]: to clarify the behaviour of masonry under load applied parallel to the horizontal joints. To examine this, tests were conducted on samples of brickwork. The samples were made of solid brick masonry which was used very often in historic buildings.

3.1.1 The compressive strength of masonry parallel to the horizontal joints.

First of all the material strength was examined using small square samples 11,5 cm thick and 50 cm in lenght and height and comparing the behaviour under load parallel and perpendicular to the horizontal joints. Thereby, the ultimate stress achieved parallel to the horizontal joint reached only 80% of the value received for perpendicular load. This result probably owes to the horizontal joints that already split under low pressure and to the fact that vertical joints are generally filled with less mortar than horizontal joints are. It is significant in both cases that cracks run approximately parallel to the direction of the force applied.

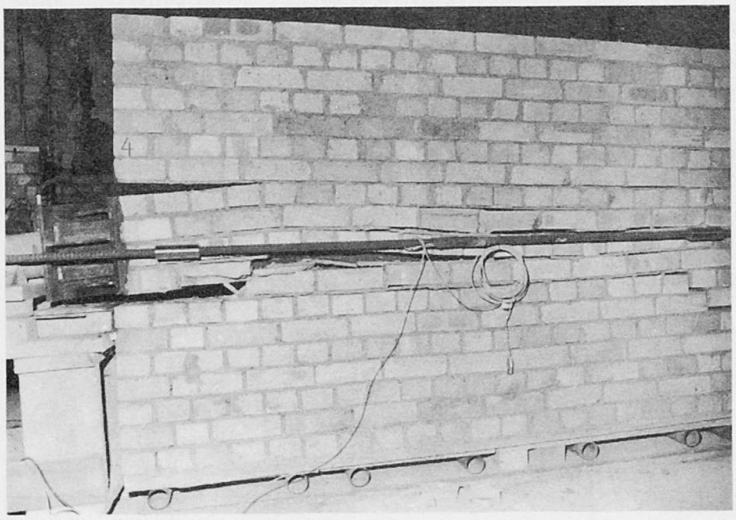


Fig.5 undulatory deflection curve upon buckling

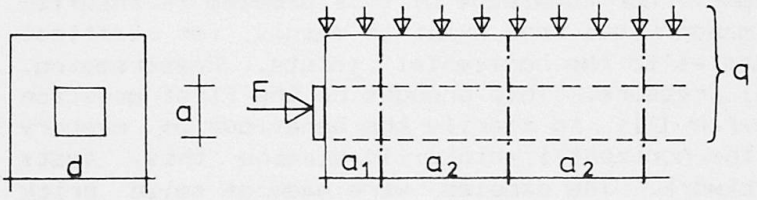
Further tests showed that masonry which is under pressure parallel to the horizontal joints and does not have sufficient vertical loads is especially vulnerable to vertical buckling. The horizontal joints parallel to the direction of applied force are the weak spots. They are torn apart fairly soon due to transverse tensile stress. The different layers split and buckle upwards.
Vertical loads reduce the danger of buckling. The ultimate stress increases accordingly. The additional vertical load in the test, shown in fig.5 , consists of masonry.
The result is an undulatory deflection curve, starting either with an undulation upwords or downwards.

Theoretical reflections have led to 5 possible cases of buckling [1] . The practical result is a formula according to which a necessary vertical load can be determined while designing the construction:

$$q = \nu \cdot 1{,}67 \cdot \sigma_{II} \cdot F \;/\; (h \cdot E_{II})$$

in which:
- $\nu = 5{,}0$ safety factor
- σ_{II} = pressure stress in those layers directly under load
- F = prestressing force
- h = layer height
- E_{II} = elastic modulus of masonry parallel to the horizontal joints

If the vertical load is not sufficient either the prestressing force or the pressure stress in those layers which are directly under pressure must be reduced accordingly. Another solution is to arrange reinforcement bars in the masonry wall crown (Fig. 6). Reinforcement bars are intensioned and are inserted into drillholes in masonry along with spacers and then grouted with a cement-water paste. A formula has been derived to determine the space between the bars which has to be smaller than the critical buckling length.



$$a_1 = 0{,}246 \cdot \sqrt[4]{\frac{h^3 \cdot E \cdot a \cdot d}{q}}$$

$$a_2 = 2{,}4 \cdot a_1$$

Fig. 6 intervals between vertical reinforcement rods at the masonry wall crown

3.1.2 Partial surface pressures parallel to the horizontal joints

The anchoring elements of prestressing bars apply force only to partial areas. Behind this locally highly concentrated transfer of force the pressure stress spreads out until, after a certain length of introduction, the masonry is compressed in its entirety. The spread of pressure causes splitting tensile forces approximately transverse to the direction of the applied force.

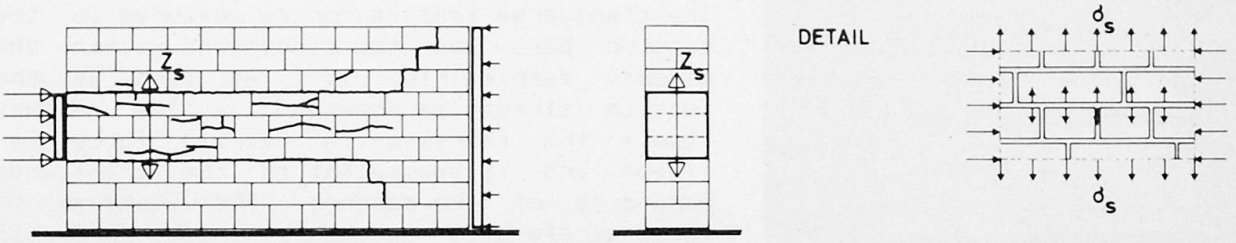


Fig. 7 splitting tensile forces perpendicular to the horizontal joints

Two cases must be distinguished when dealing with masonry, that is under load parallel to the horizontal joints: If the breadth of the pressure plate equals that of the masonry wall the splitting tensile forces act perpendicular to the horizontal joints and accelerate the process of splitting. The spread of pressure into peripheral zones is reduced until the layers directly under load remain to transfer the force alone. The masonry fails by showing horizontal cracks in the bricks (Fig.7).

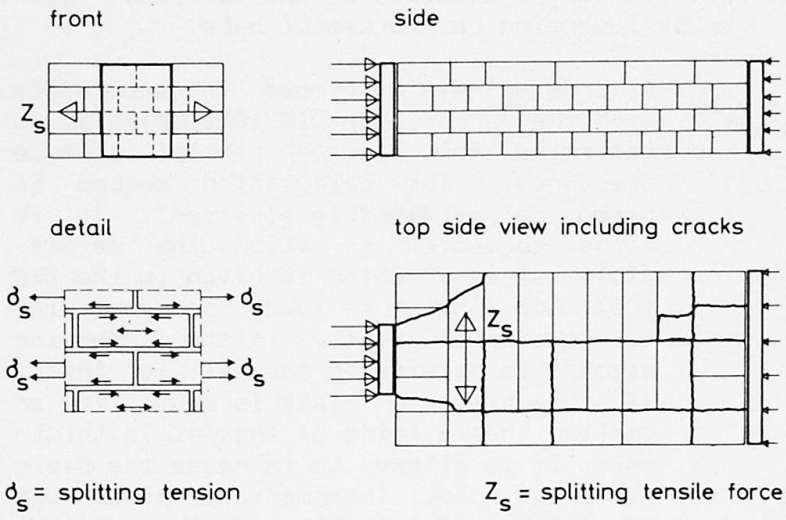


Fig.8 splitting tensile forces parallel to the horizontal joints

If the pressure plate is smaller than the wall is thick then pressure spreads transverse to the plane of the wall. The splitting tensile forces act perpendicular to the vertical longitudinal joints, which are covered, according to the rules of masonry bond, by bricks in the layers above and below. The partial surface pressures reached in the moment when first longitudinal cracks occur were considerably higher than achieved when pressure only spread out in the plane of the wall. As expected, the masonry failed by cracking along the longitudinal joints. In some cases the cracks run through bricks towards the longitudinal joints (Fig.8).

During practical restoration work on masonry, reinforcement bars are arranged transverse to the wall plane behind the anchorage. This prevents the masonry from splitting and cracking and also helps stabilize the area of force introduction by grouting before the prestressing force is applied (Fig. 1 and 15).
This procedure was simulated with a few masonry samples by inserting 6 tension bars and cross bars. In comparison with the tests without tension bars the resulting partial surface pressures at the moments of first cracking and failure were increased by about 30% . The cracks were distributed more evenly than without tension bars (Fig. 9). In further tests the tension bars were arranged vertically. This way upward buckling of courses of brick should be ruled out as

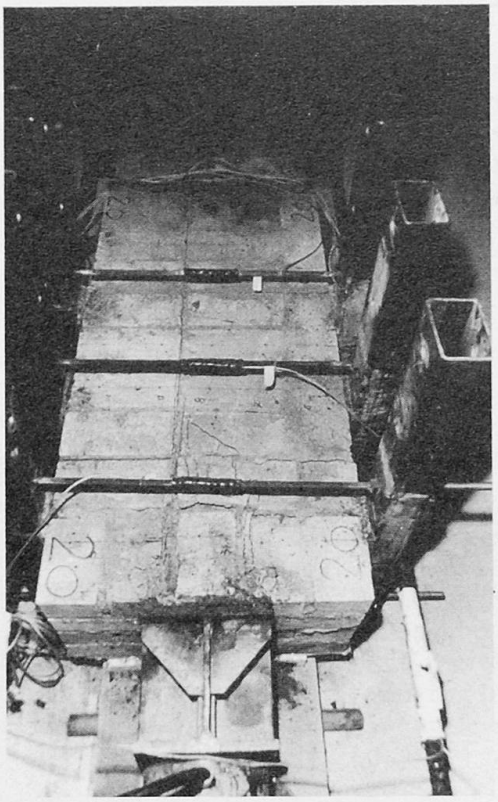


Fig.9 transverse horizontal tension bars

a cause of failure. The beginning of cracking in the horizontal joints was not influenced substantially by the tension bars. The stage of failure, though, was reached much later than with comparable samples without tension bars. The ultimate pressure was about double the value of similar samples without tension bars.

The transverse tensile forces measured in the tension bars show the conditions within the masonry sample (Fig. 10). As long as the tensile stress is absorbed by the masonry itself the increase in tensile force is linear and is equivalent to the transverse expanding of the masonry. The increase in force accelerates as soon as cracks appear in the horizontal joints and later in the bricks as well. The internal tensile stress in the masonry is increasingly transfered to the outside tensile bars. The ultimate forces in the bars reach about the size of calculated splitting tensile forces.

For designing, an adequate safety margin toward failure of the masonry is needed. Therefore it is possible to use the value of the calculated splitting tensile force and prove that it can be absorbed by the vertical load or by inserting reinforcement bars.

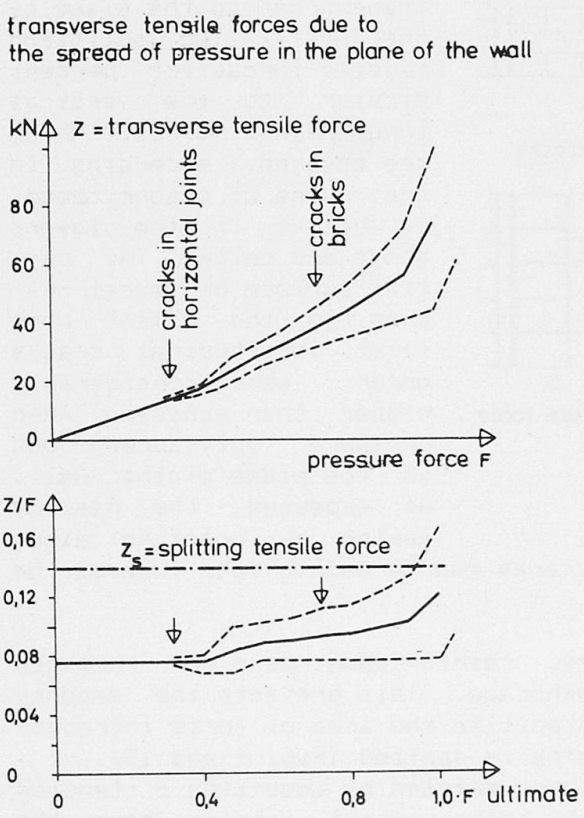


Fig.10 vertical transverse tensile force

A proposal was developed in accordance with the German code DIN 1053, page 1, to determine the size of partial surface pressures. The calculation method is based on "permissible stresses". It is being suggested to allow the permissible stress, which is given in the DIN 1053 and applies to loads perpendicular to horizontal joints, if the anchoring element is as wide as the wall is thick. If the pressure plate is equal to or smaller than a third of the wall's thickness, it is allowed to increase the basic value 1,5 fold. Intermediate values can be determined linearly. Most important was the safety towards cracking which totals 1.5 . The greater the distance to the anchoring elements the less pressure stress there is in the masonry. If the danger of cracking is minimal, it is sufficient to determine prestressing forces that will create an average lateral pressure stress of between 0.1 and 0.2 N/mm^2. In all other cases it is necessary to carry out statical calculating of the construction elements to determine the size and position of prestressing forces that are needed to definitely avoid tensile stress in the masonry.

3.2 Measuring the prestressing forces of built-in prestressing bars.

Besides dealing with the problems already mentioned concerning the introduction of prestressing forces into masonry the durability of applied tensioning forces had to be clarified. Experience with prestressed concrete shows that concrete shrinks when it hardens, it dries out, and it creeps under pressure, it shortens, whereby part of the applied tensioning force is lost. This is the same with masonry. Deformations occurring after prestressing lead to changes in the prestressing force. The deformation behaviour of modern types of masonry is generally known. The already determined data, though, cannot be applied to masonry of historic buildings which includes significant disturbances of its bond. To gather experience I conducted measurements on several restoration sites over a number of years. Thereby measurements were carried out during construction and afterwards on the built-in prestressing bars with the help of strain gauges.

3.2.1 The influence of the wall temperature

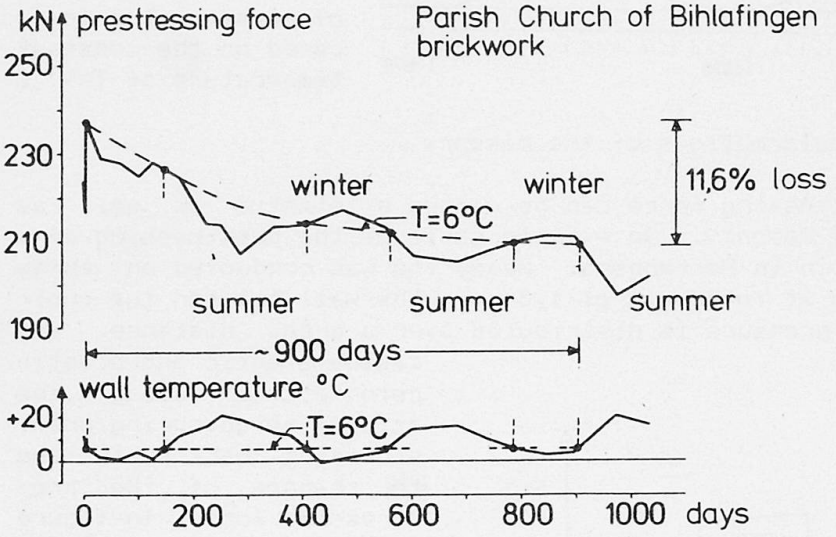


Fig.11 course of prestressing forces due to the wall temperature

Besides a falling tendency at the beginning the measurement curves show seasonal changes that are opposite to the course of the wall temperature shown below (Fig. 11). With increasing temperatures the force within the prestressing bar declines, whereas towards winter, when temperatures fall, the tensioning force grows, caused by the different coefficients of expansion α_T of steel and masonry. These coefficients could be determined from the measuring data.

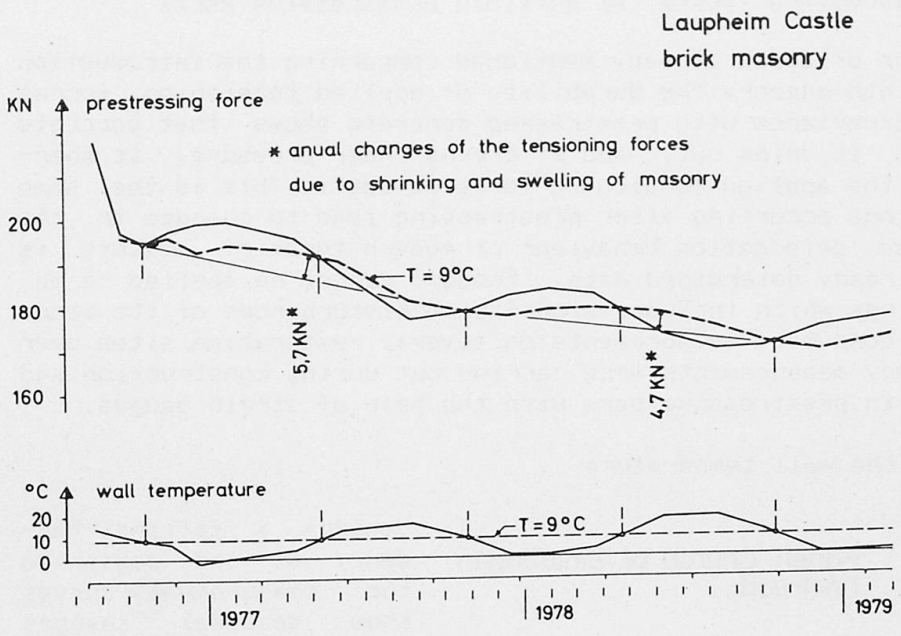
With $\alpha_T = 11.5 \times 10^{-6}$ of steel, the following values were determined:

for brick masonry $\alpha_T = 6{,}3 \cdot 10^{-6}$
and for sandstone masonry $\alpha_T = 7{,}7 \cdot 10^{-6}$

The long-term tendency of the curves becomes more obvious if points of the same wall temperature are connected in the curves shown by the dashed line in fig.11. This procedure eliminates another cause of seasonal changes, though, which shows only after the influence of temperature has been compensated by way of calculation for every single measuring spot as shown in fig. 12.

3.2.2 The influence of moisture in the wall

The curve, in which temperature changes have been compensated for, still shows seasonal swinging. The amplitude is smaller and the high and low points are displaced by about three months compared to the curve of wall temperature. Because the creep deformations occur mainly during the time of high summer temperatures some relation to the moisture content of the walls can be assumed. In summer the masonry dries out, creap is accelerated, in other words: if the creep of masonry happens continuously then the curve of creep is overlaid by an annual



swinging caused by swelling and shrinking of masonry. The annual changes due to temperature and moisture are insignificant for the determination of prestressing forces, because they do not lead to permanent losses.

Fig.12
the calculated course of prestressing force based on the constant temperature of T=9° C

3.2.3 Elastic and plastic deformations of the masonry

Permanent losses of prestressing force can be caused by elastic as well as plastic deformations of the masonry. An example could be the prestressing of a wall in the Collegiate Church in Herrenberg. Measuring was conducted on three prestressing bars, arranged at intervals of 1,3 m in the wall between the choir and the vestry. The applied pressure is distributed over a great distance and causes elastic and plastic deformations along the axis of neighbouring prestressing bars that lead to changes of the prestressing force. In figure 13 the bottom diagram shows 3 attempts to prestress the lower bar whereas the top diagram shows the changes in pretensioning force in the already stressed upper bar. The loss at the beginning is greater than during further burdening and relieving procedures. It consists of elastic deformations as well as of a portion of plastic shortening which effects the bar. The complete loss on the first day totalled 5.4% . These initial losses in pretensioning stress can be avoided by applying excess pressure to begin with or after the losses have occurred. It is not possible, though, to do the latter, if the drillhole has already been grouted, because of the bond between masonry and the bar.

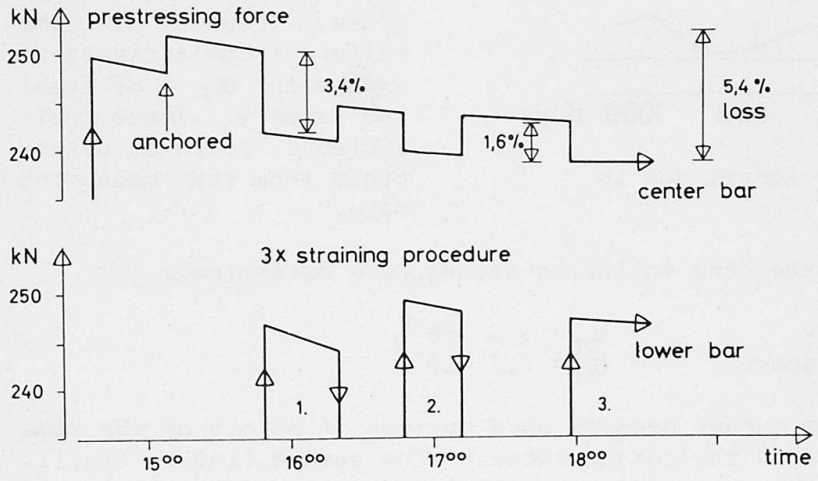


Fig.13 correlation between the bars during prestressing

The course of active tensioning force stabilizes after grouting. The plastic deformations of the masonry happen more slowly. There are only few measuring curves, though, that match the path of the creep curve to the same extent as the one in figure 12 showing the greatest at the beginning and smaller losses afterwards. The course of the curve is usually disturbed by effects resulting

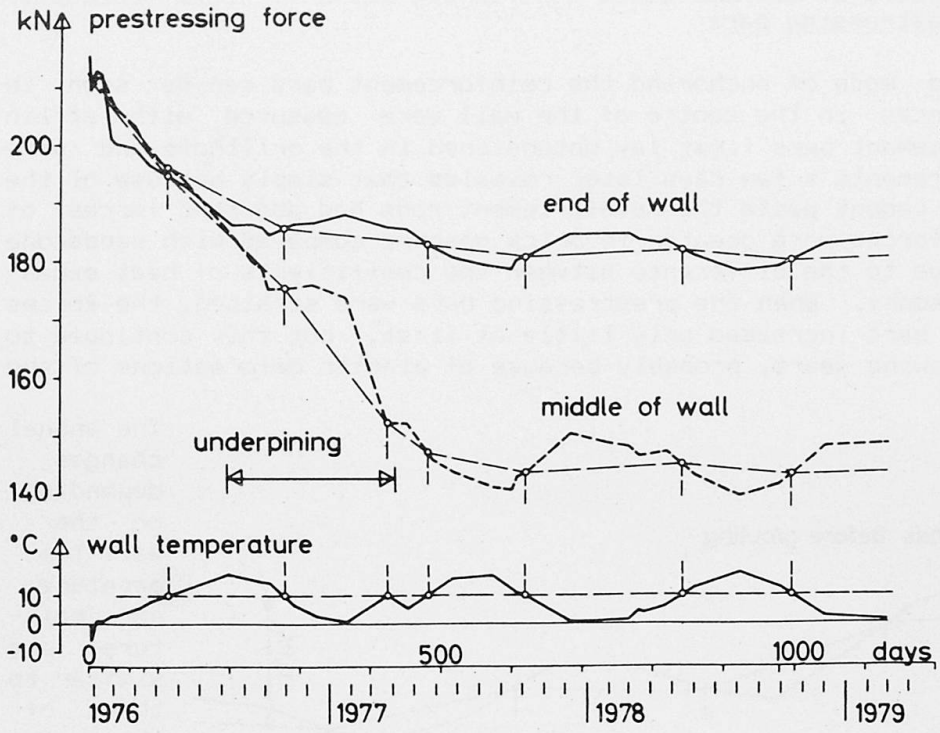


Fig.14 measuring curves showing great losses of prestressing force due to underpinning and excavating work

from the foundation, the rest of the building or from restoration work. Drilling in the masonry, work with a pneumatic hammer during excavating, settlement movements and underpinning cause spontaneous plastic deformations in the masonry that cannot be regarded as creep in the actual sense. The cause is not pressure stress, which leads to creep, but vibrations, that make the masonry give way. Figure 14 shows the most striking example measured.

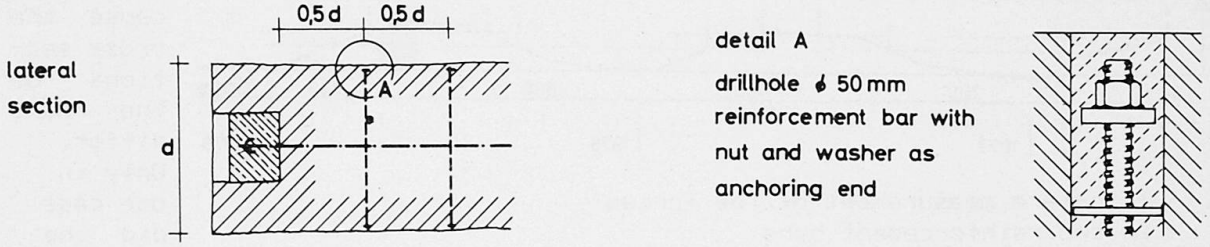


Fig.15 reinforcement rods in the zone of force introduction through prestressing bars

The size of losses of prestressing force usually remained under 15% . Because creep deformations happen over a long period of time, examinations conducted by Hage [2] were used to establish a prognosis on the final amount of creep. According to his results about 80-85% of all creep deformations occurred within 2 1/2 and 3 1/2 years. For dimensioning, coefficients of creep φ_∞ were derived from the measuring results. With their help the expected losses of tensioning stress ΔZ_K can be determined.

$$\Delta Z_K = \varphi_\infty \cdot \sigma_{M,V} \cdot A_V \cdot E_V / E_{II}$$

$\sigma_{M,V}$ = pressure stress in masonry due to prestressing
A_V = cross-sectional area of the prestressing bar
E_V = elastic modulus of the prestressing bar
E_{II} = elastic modulus of masonry parallel to the horizontal joint

As long as no other measuring results are available, the following can be assumed: φ_∞ = 1,3 till 1,6 and E_{II} = 1000 N/mm^2.

3.3 Measuring the forces of reinforcement bars in the zones of force introduction through prestressing bars

The arrangement and mode of anchoring the reinforcement bars can be seen in figure 15. The forces in the centre of the wall were measured with strain gauges. The reinforcement bars first lay untensioned in the drillhole and were then grouted. Measurements a few days later revealed that simply because of the setting heat of the cement paste the reinforcement rods had absorbed forces of up to 5.6 kN . The forces were greater in brick masonry compared with sandstone masonry. This is due to the difference between the coefficients of heat expansion of steel and masonry. When the prestressing bars were strained, the forces in the reinforcement bars increased only little at first, but they continued to grow during the following years, probably because of plactic deformations of the masonry.

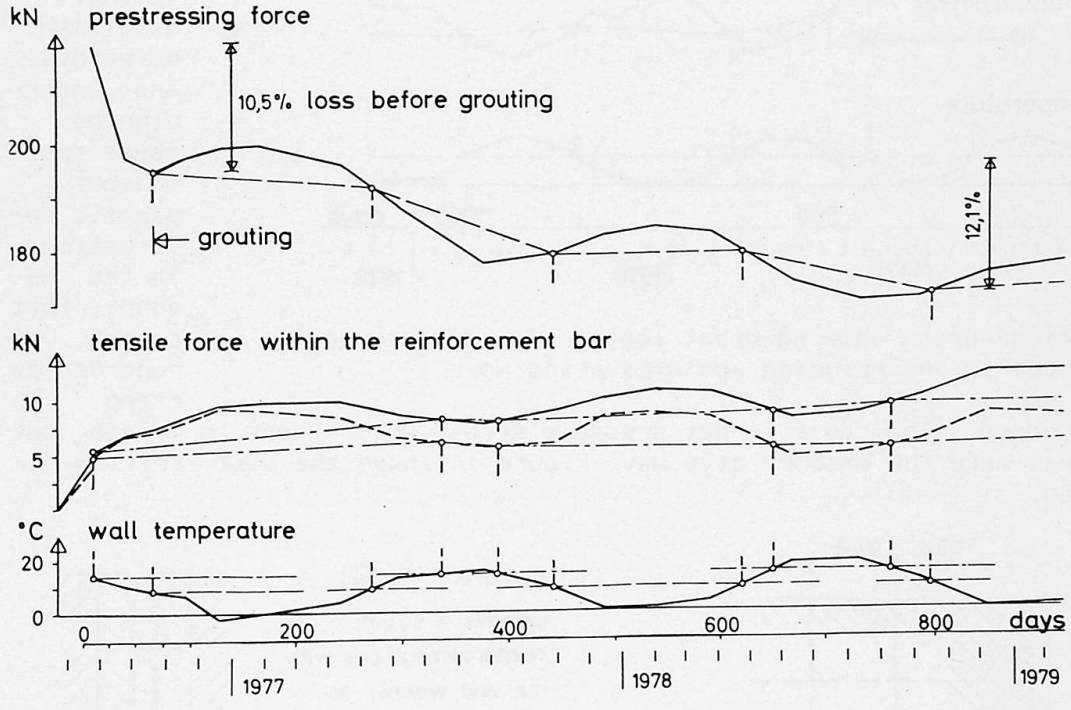


Fig.16 long-term measurement of the forces in two reinforcement bars

The annual changes depending on the wall temperature and moisture are similar to those of the prestressing bars exept that they are smaller in absolute value because the cross sections of the bars differ. Only in one case did the size of forces exceed the splitting tensile force calculated for the bar. Because of incertainties concerning the distribution of tensile stress within the masonry, it is suggested to dimension the reinforcement bars for 1.5 fold of the proportionate splitting force. For reasons concerning the securing of cracks the permissible stress for steel should be restricted to 120 N/mm^2 = 1200 kp/cm^2 according to the DIN 1053, irrespective of the steel quality.

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Repair Techniques of Reinforced Concrete Beam - Column Joints

Techniques de réparation de noeuds poutre-colonne en béton armé

Reparatur von Rahmenknoten aus Stahlbeton

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SUMMARY

Tests performed on specimens representing subassemblages of multi-storey r.c. frames are presented, and first results are discussed. Virgin specimens are stressed by means of cyclically imposed displacements. After repair, identical straining histories are applied. Some repair techniques, selected among the most commonly used, are examined. Results are discussed, with reference to a few parameters representative of strength and energy dissipation capacity.

RESUME

La communication présente les premiers résultats d'un programme de recherches sur des modèles structuraux de parties de portiques à plusieurs étages en béton armé (noeuds poutre-colonne). Les essais sont conduits sur les modèles originaux soumis à des sollicitations alternées et sur les mêmes modèles après réparation. Quelques techniques de réparation ont été examinées parmi celles couramment employées. Les résultats sont discutés en fonction des paramètres représentant la résistance et la capacité de dissipation.

ZUSAMMENFASSUNG

Es werden die Ergebnisse aus Versuchen an Probekörpern, die Knoten eines mehrstöckigen Stahlbetonrahmens darstellen, vorgestellt. Die Probekörper erfuhren vor und nach ihrer Verstärkung die gleiche Wechselbeanspruchung. Es wurden einige der üblichen Reparaturmethoden unteruscht. Die Ergebnisse werden hinsichtlich einiger für die Festigkeit und Energiedissipation bedeutungsvoller Parameter diskutiert.

AIM OF THE RESEARCH

Technical development as well as economy evolution of the last years have stressed the opportunity of retrofitting existing buildings, expecially those not designed to resist earthquakes, and salvaging the ones damaged in the struck areas.

The studies concerning the efficiency of the different repair and strengthening techniques are still, as yet, not so numerous neither so systematic as the researches concerning the behaviour of the virgin structures or structural elements. The present work is then aimed at acquiring an extensive knowledge of the efficiency of local repairs on r.c. elements.

The research has been promoted by C.N.R. (Italian National Research Council) and has been based on the cooperation among five laboratories of different universities (*). Gathering of research force sufficient for the task at hand was made possible in this way; moreover the coordination attains the results of homogeneity of experimental set-ups and techniques, as well as analysis and evaluation of the results.

EXPERIMENTAL PROGRAM

Damages in an ordinary r.c. frame subjected to earthquake have a tendency to take place in the joints and in the extremities of the surrounding members.

The present work is concerned with exterior joints only; the specimens are ideally obtained by extracting from a frame an element defined by two half columns and a length of a beam, thus obtaining a T-shaped element which is almost commonplace in similar tests (Fig. 1).

The characteristics that diversify the specimens tend to reproduce the situations really found in structures, not necessarily well designed and detailed. Sufficient as well as poor reinforcement percentages have been provided; simulation of lower or upper storey joints was obtained by vertically stressing the columns in two different degrees.

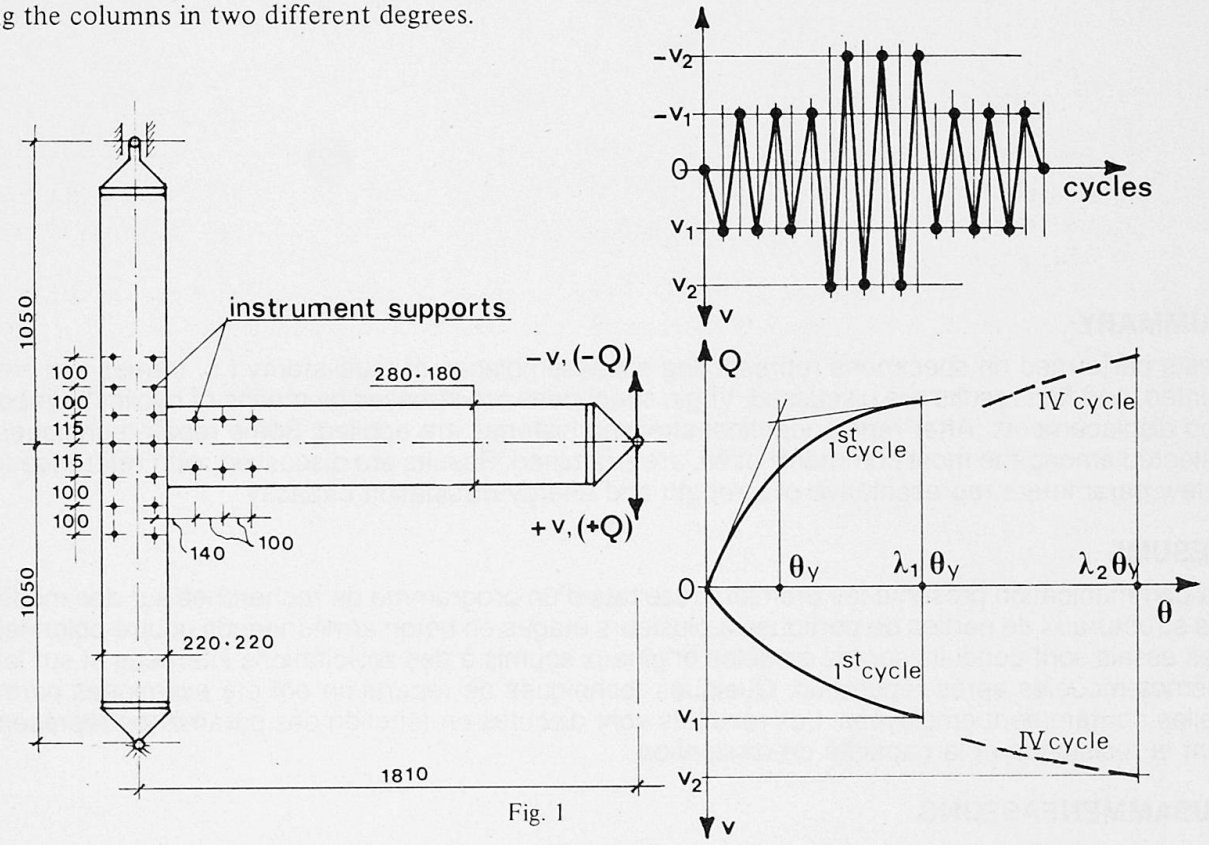


Fig. 1

(*) The laboratories that performed the tests (in the following they shall be indicated by the initials) are: Polytechnic of Turin - Engineering - (TO);- University of Florence - Architecture (Fla); - of Ancona (AN); - of Rome, Architecture (RMa); - Engineering (RMi).

Reinforcement was designed in order to concentrate damages either in the joint or in the member ends. Repair techniques must, of course, be adapted in accordance with the location and importance of damages.

As a consequence the specimens are classified on the basis:

I) of the section in which 'plastic hinges' are designed to occur, that is in the columns -**P**-; in the beam -**T**-; in the joint -**N**-.
II) of the transversal reinforcement, that is light -**R**- or strong -**F**-.
III) of the value of axial load in the column, that is light -**1**- or heavy -**2**-.

The specimens have been scaled down to 2/3 with respect to a typical actual structure; in this way the scale effect is practically negligible.

The tests are performed by cyclically applying a displacement $\pm v$ at the free end of the beam, by means of a hydraulic actuator operating statically.

The strain as well as the start and spreading of damages are of a level that can be found in actual structures struck by earthquake.

Preliminary tests suggested to assume the following straining procedure:

the governing factor is the strain (say the curvature ϑ_y in the critical section) at the knee of the diagram corresponding to the yielding of the reinforcement or at the appearance of other causes of non linearity. Straining is then increased until a curvature $\lambda_1 \cdot \vartheta_y$ is reached: deflection of the beam is recorded, at a value called v_1. Three cycles from $+ v_1$ to $- v_1$ are then imposed. The fourth cycle is again governed by the curvature, $\lambda_2 \cdot \vartheta_y$. The corresponding deflection v_2 is imposed alternately during cycles 4 to 6, then again $\pm v_1$ during cycles 7 to 9. Repaired specimens undergo the same sequence $\pm v_1$, $\pm v_2$, $\pm v_1$.

Instrumentation consists mainly in displacement meters fixed on rods deeply embedded in concrete, in order to avoid loss of readings due to spalling. Extensometric base lengths are about one half the depth of the column.

REPAIR TECHNIQUES

Repair techniques fall into two main categories. The first one, after removal of crushed concrete and cleaning of surfaces, consists in restoring the damaged segments by means of different materials and techniques with or without previous injection of epoxy resin in the cracks. Adding of new steel ties is applied in a few cases.

The second technique, after restoring by means of ordinary sand and cement mortar, consists in a strengthening of the joint by means of steel angles and plate lacings, which are prestressed in some cases (Fig. 2).

In such cases the angles, sand-blasted at the interior surface, are glued to the concrete by means of epoxy adhesive; after welding, lacings are heated to bright red temperature (~ 850 °C) in the middle segment (about 1/4 of the length), thus obtaining normal stresses of 120 **MPa** average value.

Restoring of damaged segments when the first technique is applied is obtained by means of: i) cement mortar with the addition of plasticizer non-shrink admixtures plastered or cast in formworks ii) epoxy mortar, plastered on surfaces.

Injection of cracks is obtained by pressure or vacuum procedure.

The different techniques applied are reported in the 7th column of Table 1.

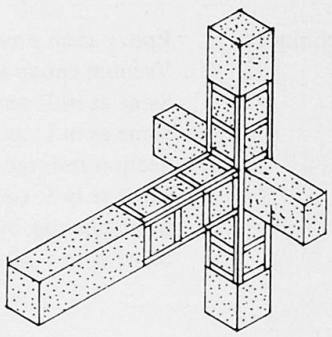


Fig. 2

Table 1

1	2	3	4	5	6	7	8	9	10	11
type	axial stress [MPa]	lab.	specimen identific.	v [mm]	damage level	repair technique	$\frac{\Sigma \lvert Q_r^{(i)} \rvert}{\Sigma \lvert Q_0^{(i)} \rvert}$	$\frac{\text{tot. } E_r}{\text{tot. } E_0}$	$\frac{Q_r^{(9)}/Q_r^{(1)}}{Q_0^{(9)}/Q_0^{(1)}}$	$\frac{E_r^{(9)}/E_r^{(1)}}{E_0^{(9)}/E_0^{(1)}}$
TT	4,19	TO	1 | 1 TT	40-73-40	A	1	0.98	0.71	1.12	1.63
		TO	2 | 2 TT	40-73-40	A	1	1.07	0.84	1.03	1.85
		FI a.	3 | 2 TT	54-75	D	6	1.13	0.79	–	–
		AN	4 | 1 TT	47-116	D	6	1.04	0.83	–	–
PF 1	4,19	TO	5 | 1 PF 1	59-86-59	B	3	1.15	0.91	1.22	1.83
		TO	6 | 2 PF 1	59-86-59	B	4	1.18	1.10	1.25	1.41
		RM a.	7 | 3 PF 1	35-70-35	A	1	1.03	0.89	1.22	1.58
PR 1	4,19	TO	8 | 1 PR 1	50-65-50	B	3	1.25	1.04	1.24	1.44
		FI a.	9 | 2 PR 1	27-54-27	A	2	1.03	0.98	1.21	1.59
		FI a.	10 | 4 PR 1	39-57-39	B	6	1.09	1.12	1.06	1.63
		RM i.	11 | 1 PR 1	48-62	C	7	1.03	1.06	–	–
		RM i.	12 | 2 PR 1	47-67	C	8	1.38	1.41	–	–
PF 2	8,38	TO	13 | 1 PF 2	57-85-57	B	3	1.22	1.08	1.18	2.00
PR 2	8,38	TO	14 | 1 PR 2	32-49-32	B	3	1.16	0.70	1.16	0.73
NF 1	4,19	RM i.	15 | 13 NF 1	75-117-75	C	7	0.91	1.16	2.49	1.86
		RM i.	16 | 14 NF 1	83-106-83	C	8	1.03	1.41	1.50	2.30

Damage level:
- A Crack opening up to 4 mm, moderate crushing of concrete cover.
- B Spalling of cover, moderate crushing of concrete core.
- C Extensive crushing of core.
- D Same as in C, with buckling of reinforcement bars.

Repair techniques:
1. Epoxy resin pressure injection.
2. Vacuum epoxy injection, with polyethylene foil jacket.
3. Same as in 1; section restored by means of epoxy concrete.
4. Same as in 1; section restored by means of cement concrete with admixtures.
5. Section restored by means of cement concrete with admixtures (no injection).
6. Same as in 5, ties added.
7. Strengthening by means of corner profiles and lacings.
8. Same as in 7; glued corners, heat-prestressed lacings.

RESULTS

In column 5 of table 1 displacements v_1 and v_2 are reported; in column 6 the damage level, as specified at the bottom of the table. Remaining columns from 8 to 11 contain indexes that have been chosen as representative of the efficiency of the adopted repair technique (suffix **o**, virgin specimen; suffix **r**, repaired specimen).

The index reported in column 8 evaluates the strength of the repaired specimens with respect to the one of the virgin ones. Namely such index is the ratio between the sum $\Sigma\ |Q_r^{(i)}|$ of the absolute values of the beam end responses $Q_r^{(i)}$ at the imposed displacements $\pm v_1$ and $\pm v_2$ of the repaired specimen, and the analogous sum $\Sigma\ |Q_0^{(i)}|$ of the responses $Q_0^{(i)}$ by the virgin specimen.

The column 9 contains the ratios between the cumulative energies dissipated in the whole series of applied cycles:

$$I_9 = \frac{\text{tot. } E_r}{\text{tot. } E_0} = \frac{\Sigma \int Q_r \, dv}{\Sigma \int Q_0 \, dv}$$

The index in column 10 gives an idea of the decaying of the strength along the applied straining history, as the fraction of the strength of the last cycle with respect to the strength of the first, $Q_r^{(9)}/Q_r^{(1)}$, in the repaired specimen divided by the same fraction as computed for the virgin one.

In column 11 an analogous index, this time concerning the energies dissipated during last and first cycles is reported.

It is of some interest to give more detailed information with regard to the variation of strength and dissipated energy during the straining history. In the Fig. 3 the values of the ratio between $Q_r^{(i)}$ and $Q_0^{(i)}$ are plotted against the number **i** of cycles for the most representative cases.

In Fig. 4 similar ratios are contained, concerning the energy dissipated in each cycle.

In Fig. 5 the values of $Q_r^{(i)}$ and $Q_0^{(i)}$ are plotted against **i**.

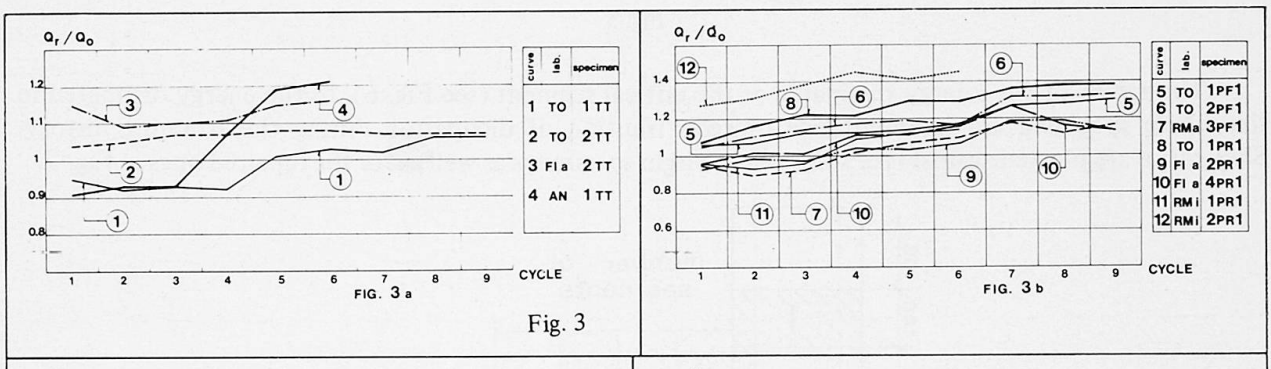


Fig. 3

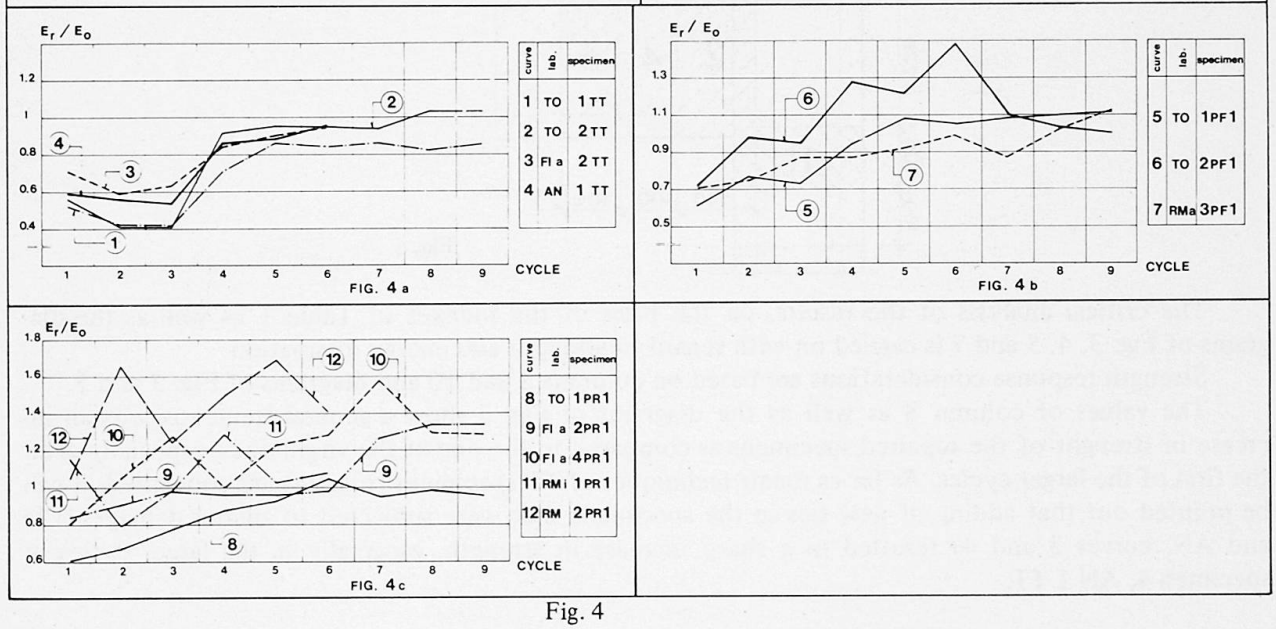


Fig. 4

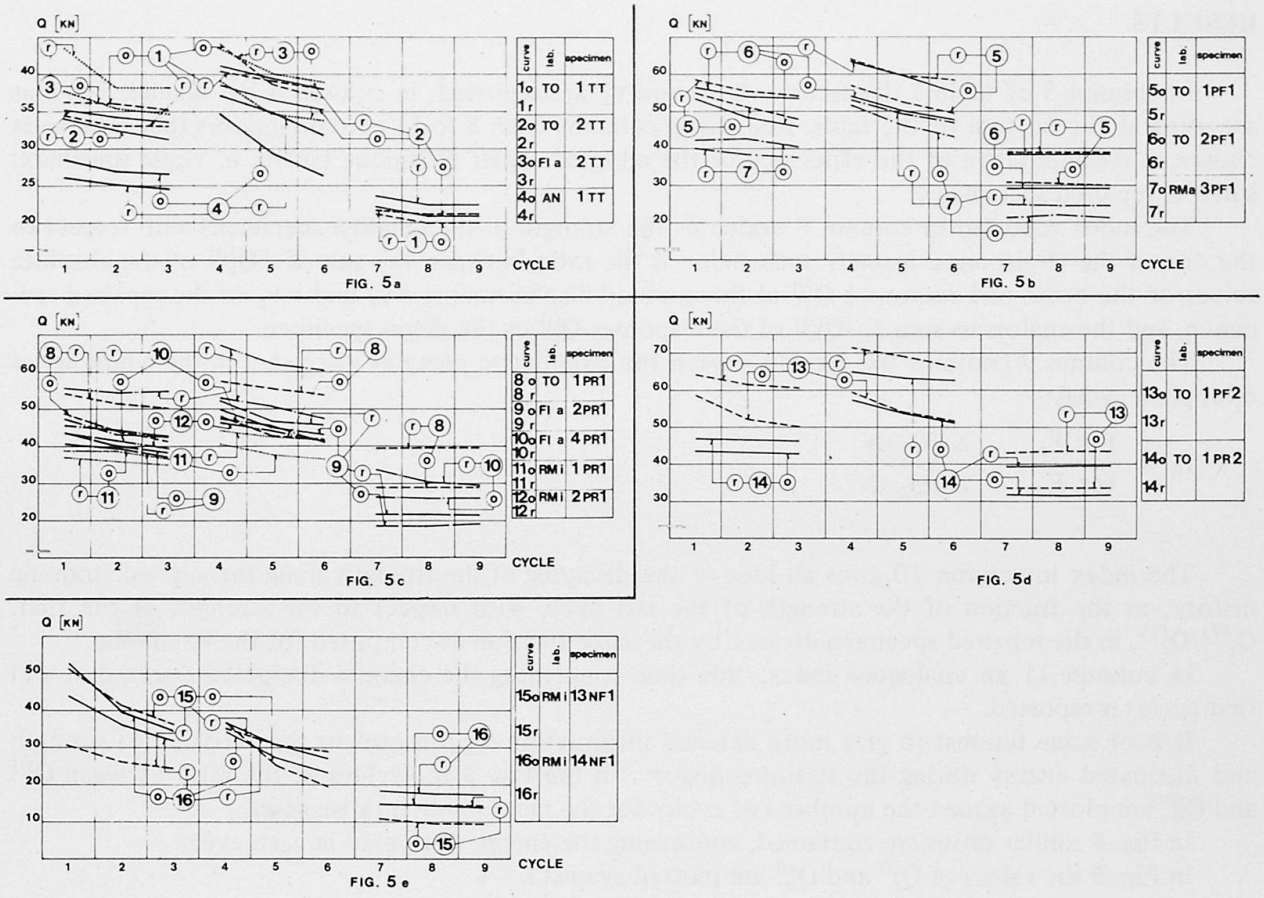


FIG. 5a

FIG. 5b

FIG. 5c

FIG. 5d

FIG. 5e

Fig. 5

The ratios of the energy dissipated at the critical segment (see Fig. 6), to the energy dissipated in the whole specimen are representative of redistribution of dissipation during the straining history. Such ratios are plotted against i for some of the virgin specimens as well as for the repaired ones in Fig. 7.

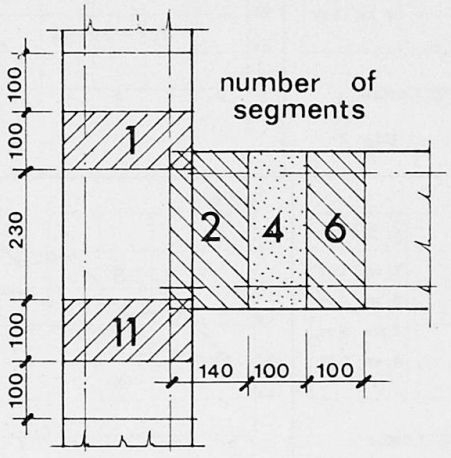


Fig. 6

The critical analysis of the results, on the basis of the indexes of Table 1 as well as the diagrams of Fig. 3, 4, 5 and 7 is carried on with regard to strength and energy dissipation.

Strength response considerations are based on columns 8 and 10 and diagrams of Fig. 3 and 5.

The values of column 8 as well as the diagrams of Fig. 3 show a general trend towards an increase in strenght of the repaired specimens as compared to the one of the virgin ones, especially after the first of the larger cycles. As far as repair techniques of **TT** specimens (Fig. 3a) are concerned, it can be pointed out that adding of new ties in the specimens that were subjected to greater damages (FIa and AN, curves 3 and 4) resulted in a sharp increase in strength, especially in the larger cycles of specimen 4, AN 1 TT.

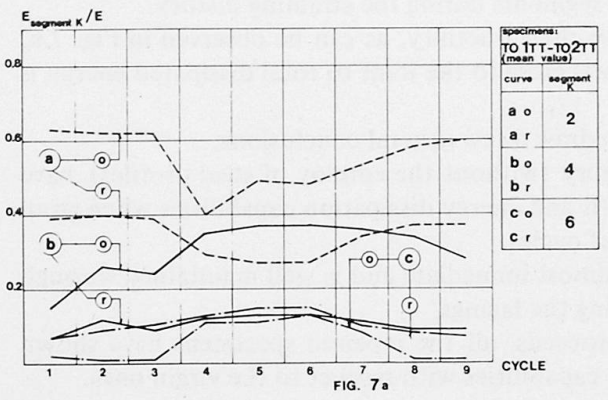


FIG. 7a

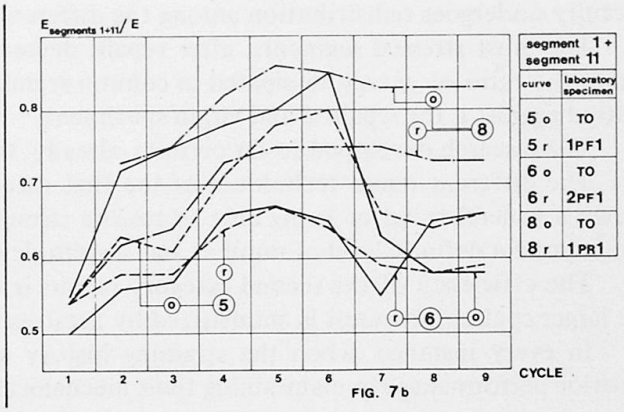


FIG. 7b

A good agreement can be found among results of tests on specimens **P**, type **F** and type **R** (Fig. 3b), with repairs of type 1, 3 and 4 (epoxy resin pressure injection, curves 5, 6, 7, 8) as well as of repair of type 2 (epoxy resin vacuum injection, curve 9), regardless of the material employed in restoring the section. Similarly, a good agreement exists between the results of tests on specimens repaired with steel profiles without prestressing, (curves 10 and 11, FIa and RMi). When the lacings are prestressed the response in strength is significantly improved (curve 12, RMi).

More information concerning the development of strength response can be desumed from Fig. 5, containing the readings of the $Q_0^{(i)}$ and $Q_r^{(i)}$ forces at the beam end, grouped in five different diagrams. The groups are composed of the results of the **TT**, **PF1**, **PF2** together with **PR2** (high axial stress on the column) and **NF1** specimens. For instance, the results concerning the **NF1** specimens repaired following the techniques of passive or prestressed lacings, show that the response of the repaired specimen is less than the response of the virgin one as far as the first three low amplitude cycles are concerned, while the opposite is true when the larger cycles are applied. Performance of the prestressed specimen is better in both phases.

The indexes contained in column 10, being always greater than unity, show that the repaired specimens have a better capacity of "overall maintenance" of strength than the virgin ones.

As far as energy dissipation is concerned, columns 9 and 11 as well as diagrams in Fig. 4 and 7 are provided for a critical analysis.

The values of the index in col. 9 together with the diagrams in Fig. 4 show a general trend of a lesser capacity of dissipating energy in the repaired specimens as compared to the one exhibited by the virgin ones; this is true in a higher degree during the first low-displacement cycles. When steel profiles are employed the opposite is true, the energy dissipated being larger along the whole straining history.

Considering the repairs carried on **TT** specimens (Fig. 4.a, curves 4 and 10), the adding of ties improves the capacity of dissipation since the very beginning.

For specimens tested at **TO** laboratory (curves 1 and 2, low level damage and epoxy injection of cracks), overall dissipated energies during the whole test of the repaired specimens were very close to the values of the virgin ones.

As far as specimens of the **P** type are concerned, with the exception of the ones repaired by means of steel profiles (curves 10, 11 and 12 of Fig. 4.c), all the others, regardless of the particular repair technique employed, show an increasing dissipation capacity of the repaired specimen with respect to the virgin one. As a consequence, the values of the index in column 11 are greater than unity.

A good agreement exists between the performances of repairs by means of non-prestressed steel profiles obtained at **FIa** and **RMi** laboratories, while the prestressed one show a superior performance even in terms of dissipation capacity.

More information concerning energy dissipation can be obtained examining the diagrams of Fig. 7.a, concerning the tests conducted at **TO** laboratory on **TT** specimens. It can easily be seen that the energy dissipated at the most stressed segments is greater in the virgin specimen than in the repaired one. The opposite in true when the less stressed segments are concerned. This fact means that the

ductility undergoes redistribution among the different segments during the straining history.

The most stressed segments, after repair, decrease their ductility, as can be observed in Fig. 7.b, where the ratio of energy dissipated in column segments close to the joint to total dissipated energy is plotted against i, for repaired and virgin specimens.

The research developed so far permits, already, to draw a few general conclusions.

The different repair techniques of the first category (without the employ of steel profiles), have shown a general trend of being able to restore strength and energy dissipation capabilities when strained beyond a definite level of number and amplitude of cycles.

The efficiency of the second category repairs is almost immediate and is well maintained through the larger cycles; this result is emphasized by prestressing the lacings.

In every instance, when the straining history proceeds, all the repaired specimens have shown superior performances in maintaining their mechanical capabilities with respect to the virgin ones.

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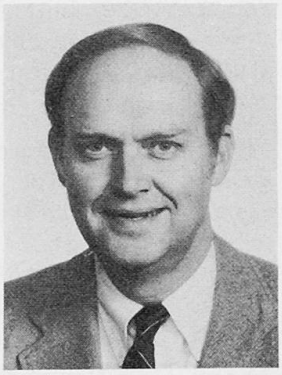
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Seismic Strengthening Procedures for Existing Structures

Renforcement de structures existantes contre les séismes

Erdbeben - Sanierungsmassnahmen für bestehende Bauwerke

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SUMMARY

Seismic strengthening of existing structures is a common form of rehabilitation in areas of the world where earthquakes occur. Suitable criteria for each project must be established. The quality of the strength provided is usually more important than the amount of the strength since ductile performance of structural members is essential for sound seismic resistance. Schemes must provide balanced strength and stiffness throughout the structure without discontinuities. Several examples are included to illustrate current practice in the United States.

RESUME

Le renforcement de structures existantes contre les séismes est une forme de réhabilitation commune dans les régions du monde affectées par des tremblements de terre. Il est d'abord nécessaire d'établir des critères spécifiques pour tout projet donné. La qualité de la résistance ainsi apportée est généralement plus importante que la quantité proprement dite, car la résistance appropriée aux chocs séismiques dépend essentiellement de la ductilité des éléments structuraux. Tous les projets de renforcement séismiques doivent conduire à un équilibre entre résistance et rigidité sans aucune discontinuité dans la structure considérée. Plusieurs exemples illustrent ce qui se fait actuellement aux Etats-Unis.

ZUSAMMENFASSUNG

Die Verstärkung bestehender Bauwerke gegen Erdbebeneinwirkungen ist eine übliche Sanierungsform in Gebieten mit Erdbebengefahr. Für jedes Projekt müssen geeignete Kriterien erstellt werden. Die Qualität der Verstärkungen ist meist wichtiger als deren Ausmass, da die duktile Verformungsfähigkeit der einzelnen Bauteile für den Widerstand gegen Erdbebeneinwirkungen ausschlaggebend ist. Der Plan muss eine ausgeglichene und lückenlose Verstärkung der Gesamtstruktur vorsehen. Verschiedene Beispiele neuerer Methoden aus den Vereinigten Staaten werden aufgeführt.

1. INTRODUCTION

Seismic strengthening of existing structures is the judicious modification of the structural properties in order to improve the performance in future earthquakes. The strengthening can be performed after a damaging earthquake, in which case it is that work aimed at improving the structural performance beyond the actual repair of damaged members. Seismic strengthening can also be performed on undamaged structures in anticipation of future seismic activity.

The economical design of structures to perform adequately in strong seismic ground shaking requires a design criteria which anticipates that member yield capacities will be exceeded in strong ground shaking. This requires members to be proportioned and detailed for acceptable ductile performance. Simply sizing members for strength based on frame analysis calculations will not result in suitable performance in a severe earthquake. Ductility must be provided throughout a seismic resistant structure.

Strengthening a structure for improved seismic performance is similar to strengthening a structure for gravity or wind loads but the criteria and philosophy are different. The criteria must address the anticipated seismicity and ground shaking at the site of the structure. The philosophy requires a thorough evaluation of relative strength and stiffness throughout the structure and the quality or ductility of the strength for inelastic performance and then the prudent modification of strength and stiffness to achieve ductile, well distributed and balanced lateral force resistance. Modification of strength and stiffness generally involves increasing both, although in some structures stiffness of some elements may be reduced to strengthen the structure. The most critical task in seismic strengthening is realistically evaluating the ductility or the ability of the structure to perform in the inelastic range.

2. INVESTIGATION AND EVALUATION

A thorough investigation of the existing structure must first be performed to verify existing conditions and member proportions. Original construction drawings, specifications and calculations are invaluable reference documents when available. They must be verified with actual field conditions and all modifications or alterations must be recorded. Non-structural elements which can affect structural performance such as infilled partition walls or partial height masonry walls must be noted for an evaluation of their affect on structural response.

If the structure has been damaged in an earthquake, the damage must be thoroughly recorded. Notes and photographic record of each damaged member are necessary not only to design repairs but also to evaluate performance in order to design strengthening measures.

An evaluation must then be performed to estimate how the structure will perform in an earthquake. Calculations must be prepared to evaluate the building's strength for gravity and lateral loads, considering the relative rigidity of all elements including non-structural elements which affect structural performance. Of greatest importance is a qualitative evaluation of the inherent ductility or quality of the strength within the lateral force resisting system and a comparison of this type of construction to the historic record of performance of similar structures in past earthquakes. Experimental research incorporating seismic based cyclic load histories can also assist in the evaluation, but the actual details of construction seldom duplicate those of the laboratory research. This evaluation involves considerable professional judgment by a structural engineer who is knowledgeable how buildings actually perform in earthquakes.

If the building was damaged in an earthquake, the engineer must carefully study the damaged structure and thoroughly understand why the damage occurred. He must determine the force resistant paths in the building and explain the reasons

why certain members sustained damage while other members were essentially undamaged. He must determine if the structure suffered due to discontinuities in strength or stiffness, due to torsional moments within the structure, due to hammering with adjacent structures or due to improper connections or details. He must consider the effects of non-structural elements such as infilled walls and appendages on the structural performance. He must determine if members failed due to shear, compression, flexure, bar anchorage, etc. This study and understanding of the seismic performance generates essential input for developing a suitable strengthening solution.

The decision to intervene or to strengthen the structure is based on this evaluation. It generally follows that the key factor governing the decision is the ductility or lack of ductility in the existing structure. For example, if the structure consists of reinforced concrete frames not specifically detailed for ductility for seismic forces and without shear walls, strengthening is appropriate as buildings of this type have historically exhibited poor performance and lack ductility.

3. CRITERIA FOR STRENGTHENING

Criteria for seismic strengthening projects can be simple or complex. The criteria selected must be suitable to accomplish the goal of the strengthening. The goal of the strengthening may be to comply with current Building Codes or Regulations because of an increased occupancy or retroactive compliance requirement; to voluntarily strengthen a weak-link in the structure; or to improve the overall seismic resistant performance, either to prevent potential structural collapse thereby protecting occupants or to minimize damage thereby protecting financial investment as well as occupants.

If the goal is to comply with Building Codes or Regulations, then those Codes or Regulations generally become the criteria, unless the engineer feels that a more severe criteria is appropriate and the owner agrees. If an evaluation of the existing building reveals that it has a "weak-link", such as a discontinuous shear wall, and it is decided to strengthen the building only to eliminate the "weak-link", then the lateral force design level consistent with the original design may form suitable criteria. If the goal is to strengthen the building voluntarily beyond regulations, then the engineer should select appropriate criteria. That criteria may be the current Building Code, an older edition of a Building Code with a lower lateral force design level combined with current code ductility requirements, or a criteria developed for the site based on soil conditions, and local and regional seismology. Regardless of which criteria is selected for design, the criteria must include requirements for adequate ductility in the structural members and their connections or joints.

4. DEVELOPMENT AND SELECTION OF STRENGTHENING SCHEMES

Once the decision to strengthen the building is reached, the engineer must conceive numerous different ways the building can be strengthened. These methods include adding reinforced concrete or masonry shear walls, adding structural steel diagonal bracing or new frames, adding reinforced concrete jackets to increase ductility and strength of concrete frames, adding reinforced infilled wall panels, or other suitable methods. Strengthening a building for seismic forces generally involves increasing its stiffness, although some strengthening solutions may involve the reverse when selected columns are short and stiff or other unbalances existing in the stiffness and strength of framing members. The strengthening solutions must be compatible with the functional usage of the building which will strongly influence the solution selected.

The proposed strengthening schemes must correct the observed or anticipated weak links in the lateral force resisting system. The schemes should provide uniform strength and stiffness distributions both over the height of the structure as well as throughout its plan. The effects of non-structural elements should be

considered and controlled either by isolation from the structural members or incorporation into the lateral force resisting system. Diaphragm strength and stiffness should be verified to assure that lateral forces can be properly distributed to new bracing elements, and new collectors, struts or ties added as appropriate. Of prime concern is insuring adequate ductility in the structure. Imagination and ingenuity should be exercised by the engineer while utilizing his professional experience in structural performance and construction techniques.

The alternative schemes need to be compared and their advantages and disadvantages weighed in order to select a preferred scheme. Items to be considered in this comparison include compatibility of the solution with the functional requirements of the building, feasibility of construction including availability of materials and experienced workmen, economical considerations, aesthetics and sociological considerations. Obviously, all schemes must comply with governmental regulations and the goals and criteria which were selected.

The selected solution for strengthening must then be thoroughly developed with all details established and clarified. The engineer must perform calculations and analysis to establish which members will sustain initial inelastic response as such response in columns may jeopardize the vertical support of gravity loads. The effects of added strengthening elements must be carefully evaluated to insure that they will not cause increased damage in future earthquakes. For example, frame buildings have been damaged in their first story in an earthquake and strengthened only in that story. The next earthquake has left the stiffened first story essentially undamaged but caused extensive damage in the unstrengthened upper floors. Such response should be anticipated and controlled within the initial strengthening program.

5. CONSTRUCTION EXECUTION

The drawings and specifications for seismic strengthening must be precise and detailed as strengthening elements will not function properly if all connections are not built in response to design requirements. This need is compounded as in all building strengthening projects as exact "as-built" conditions are not always accurately known until exposure during the construction work. Thus, it is frequently necessary to prepare details based on the best knowledge available and then modify them during construction when actual conditions are determined.

Due to this need to constantly monitor existing conditions as exposed and modify details to suit these "as-built" conditions, it is essential for strengthening projects that the design engineer be involved in the construction inspection process. The designer must periodically visit the structure during construction both to view exposed conditions for damage or distress as well as observe conditions to insure compliance of the construction work with the intent and assumptions of the design.

Strengthening projects also involve the use of new or seldom used construction materials and techniques which require precise specifications and instructions to the workmen as well as carefully conceived inspection procedures to insure that the construction work complies with the design intent. Many procedures involve fastening new materials to old existing materials with the quality of workmanship being a major factor affecting strength and performance. Thus, extensive field testing of connectors and fastenings is essential for a successful project.

6. EXAMPLES

6.1 Example No. 1

The existing building is a three-story reinforced concrete structure 58 meters square in plan, as shown in Figure 1. It is an office building. The First Floor is a slab on grade while the upper two floors are a concrete waffle slab

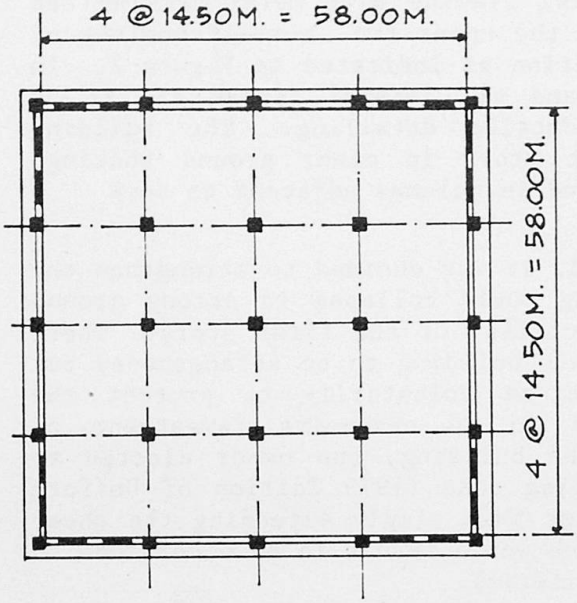


Figure 1. Typical plan of three-story Office Building in Example 1. Walls present only in upper two stories.

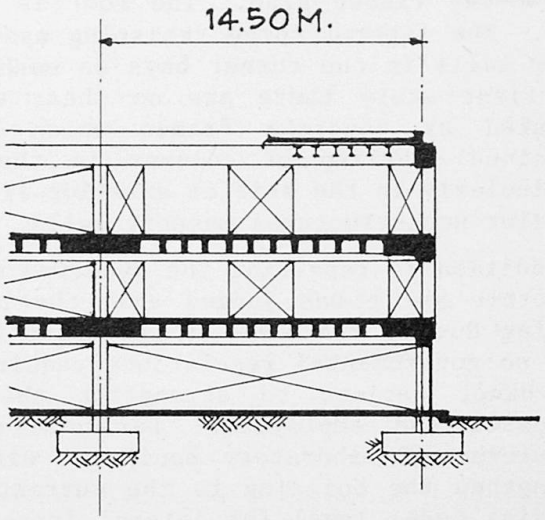


Figure 2. Partial elevation of exterior frame showing extent of existing walls in Example 1.

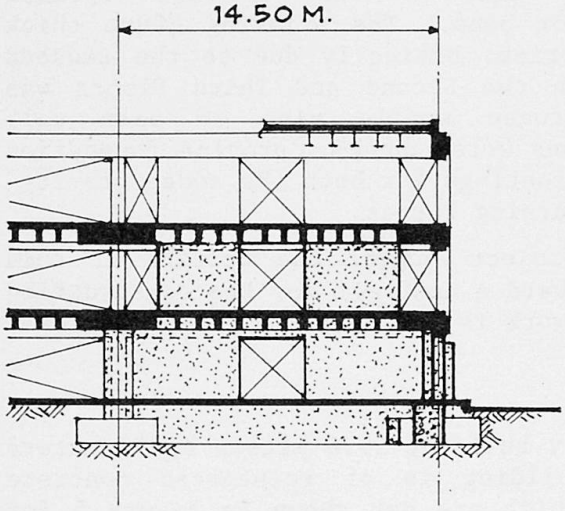


Figure 3. Partial elevation of strengthened wall system in Example 1.

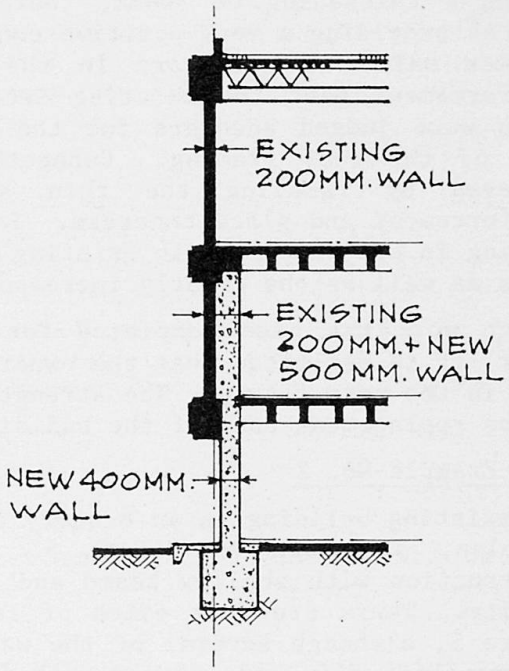


Figure 4. Section through strengthened wall in Example 1.

or two-way ribbed slab. The roof is of steel framing with metal decking and fill. The lateral force resisting system in the upper two stories consists of shear walls in the corner bays on each elevation as indicated in Figure 2. In the first story there are no shear walls and all lateral forces are to be resisted by concrete frames which lack ductile detailing. The building sustained spalling of columns in the first story in minor ground shaking, particularly in the stiffer exterior frames and in columns adjacent to some interior non-structural masonry walls.

In addition to repairing the observed damaged, it was decided to strengthen the structure as it was judged that the building could collapse in strong ground shaking due to the lack of stiffness and ductility of the first story. There were no governmental regulations requiring this building to be strengthened but the owner decided to strengthen the structure voluntarily to protect the occupants and employees. Furthermore, due to an extensive investment of computers and laboratory equipment within the building, the owner elected to strengthen the building to the current building code (1979 Edition of Uniform Building Code) level for lateral forces rather than simply extending the shear walls in the upper stories to the ground (which would result in an older code with lower lateral forces being the design criteria).

The strengthening solution is outlined in Figures 3 and 4. Shear walls 400mm thick were added in the First Floor beneath the existing 200mm thick shear walls in the upper stories. These walls were extended upward in the Second Floor by adding a thickening of 500mm, thereby both strengthening the Second Floor as well as providing a very positive connection between the new and old walls. The thicker wall was necessary in the Second Floor in order to pass vertical reinforcement past the existing Second Floor beam. The existing 200mm thick walls were judged adequate for the top stories, basically due to the reduced mass of the roof framing. Connection with the Second and Third Floors was achieved by removing the thin slab between waffle ribs to pass wall reinforcement and place concrete. Foundations were added to provide foundation bearing in conjunction with existing spread footings for both the added gravity loads as well as the greatly increased overturning forces.

Design work has been completed for this project and at the writing of this paper, it is estimated that the owner will award a contract for the construction work in the near future. The strengthening work is estimated to cost about 8% of the replacement cost of the building.

6.2 Example No. 2

The existing building is an 8-story dormitory building 57.6 meters by 17 meters in plan, as shown in Figure 5. The building is of reinforced concrete construction with shallow beams and slabs which are not shown in Figure 5 for clarity. There are four pairs of reinforced concrete shear walls as shown in Figure 5, although several of the walls are discontinuous in the Ground Floor. The two long sides are framed with 600mm square columns and beams or spandrels of reinforced concrete 2000mm deep extending from the window head in one story to the window sill in the story above. The columns are tied columns with ties at 450mm centers. The building has not been subjected to strong ground shaking but the owner has requested an evaluation of its potential seismic performance and recommendations for mitigation measures if appropriate.

The evaluation revealed that the building has sufficient shear walls in the north-south direction to adequately brace the building in that direction, but the discontinuous shear walls in the Ground Floor result in overturning forces being resisted by nominally tied columns which have traditionally exhibited a lack of ductility in earthquakes. In the east-west or longitudinal direction, the two exterior frames resist virtually all of the lateral forces due to their stiffness, and the deep spandrel beams are stronger than the columns which will force the inevitable inelastic response into columns. The nominally tied columns have virtually no ductility and their potential inelastic response will

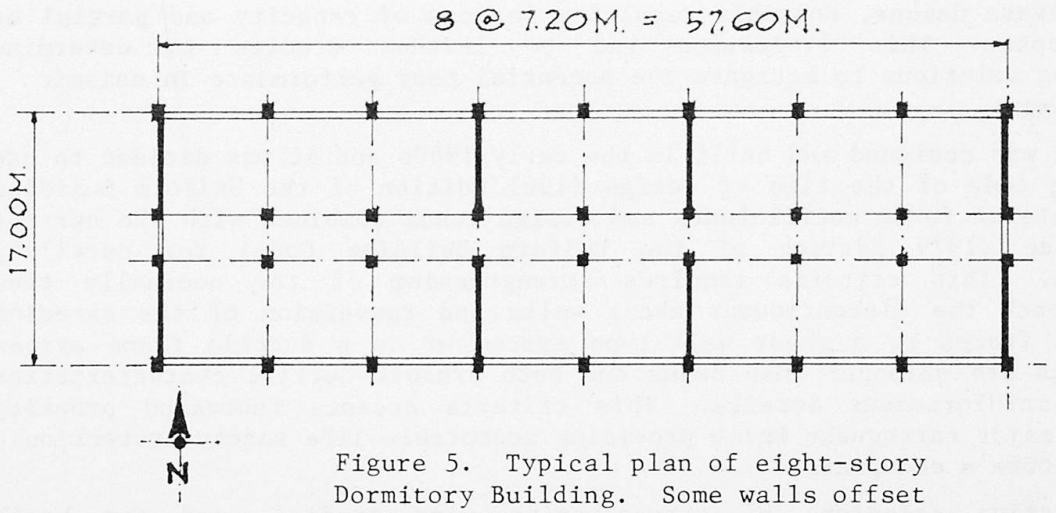


Figure 5. Typical plan of eight-story Dormitory Building. Some walls offset in Ground Floor.

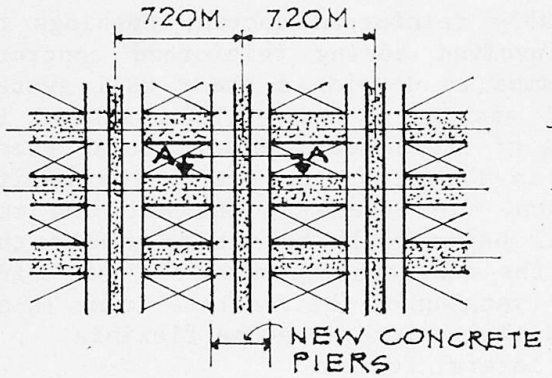


Figure 6. Scheme A strengthening adding concrete piers to existing columns to create shear wall bracing system on side walls.

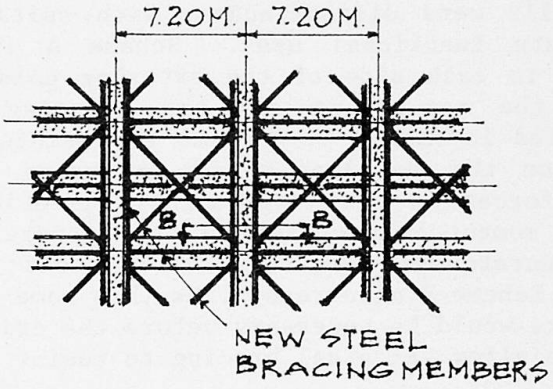
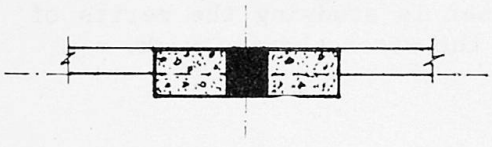
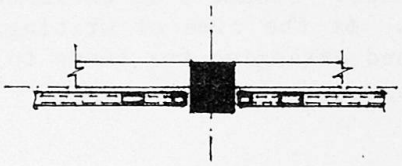


Figure 7. Scheme B strengthening adding steel bracing to existing frame on side walls.



Section A-A in Figure 6.



Section B-B in Figure 7.

result in severe damage, possibly resulting in loss of capacity and partial or total collapse. This evaluation led to further studies to determine strengthening solutions to mitigate the potential poor performance in seismic ground shaking.

The building was designed and built in the early 1960s and it was decided to use the building code of the time of design (1961 Edition of the Uniform Building Code) for lateral force coefficients and design loads combined with the current building code (1979 Edition of the Uniform Building Code) for ductility requirements. This criteria requires strengthening of the nominally tied columns beneath the discontinuous shear walls and conversion of the exterior longitudinal frames to a shear wall type system or to a ductile frame system where columns are stronger than beams and both exhibit ductile characteristics from their reinforcement details. This criteria accepts increased property damage in a major earthquake while providing acceptable life safety protection to the structure's occupants.

Many alternative solutions of strengthening were studied, and two basic solutions were presented to the owner for consideration. Both solutions included adding new reinforced concrete shear walls in the Ground Floor where shear walls were discontinuous, with suitably reinforced doorway openings to accommodate functional uses. Scheme A involved adding reinforced concrete sections to each side of the exterior columns to develop a shear wall system between the new piers and the existing spandrel beams. This scheme is illustrated in Figure 6. Scheme B consists of adding exposed structural steel bracing on the two long sides, as shown in Figure 7, to resist the entire lateral forces in the longitudinal direction. In Scheme B, the existing top negative moment reinforcing in the spandrel beams would be cut to weaken the stiff concrete exterior frame and protect the concrete columns from extensive damage. Scheme B also recognizes that some cracking of the concrete frame in an earthquake would be necessary before the existing concrete become flexible enough to allow the steel bracing to resist lateral forces.

Scheme A requires partial vacating of the building to allow work to proceed and results in a loss of windows for natural light. This scheme is probably more pleasant aesthetically and is estimated to cost about 35% of the building's replacement cost. Scheme B allows most of the work to be performed on the outside of the building without vacating the building except portions of the Ground Floor for the shear wall work. The exposed steel frame does not result in a loss of windows. Scheme B is estimated to cost about 25% of the building's replacement cost. At the time of writing, the owner is studying the merits of the two schemes and arranging for funds to finance the strengthening work.

Seismic Strengthening of Reinforced Concrete Buildings in Japan

Renforcement contre les séismes de bâtiments en béton armé au Japon

Seismische Verstärkung von Stahlbetonbauten in Japan

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SUMMARY

Available data on existing reinforced concrete buildings, strengthened against severe earthquakes as well as test data relevant to seismic strengthening of existing structures have been integrated and reviewed. The knowledge obtained from the review was described with respect to the effect of strengthening on the overall behaviour of structures, and to design criteria, seismic evaluation and construction techniques for the strengthening.

RESUME

Les données disponibles relatives à des bâtiments existants renforcés en béton armé contre les grands tremblements de terre, ainsi que les données d'essai applicables au renforcement contre les séismes ont été complétées et révisées. Les connaissances obtenues lors de cette révision sont décrites selon les effets des renforcements sur le comportement général des ouvrages, et selon les critères de conception, l'évaluation sismique et les techniques de construction pour le renforcement.

ZUSAMMENFASSUNG

Die vorhandenen Daten existierender, gegen starke Erdbeben verstärkter Stahlbetonbauten sowie Versuchsangaben über Verstärkungen für bestehende Bauten sind vervollständigt und übersprüft worden. Die aus der Ueberprüfung erhaltenen Erkenntnisse werden hinsichtlich der Wirksamkeit von Verstärkungen auf das Verhalten der Gesamtstruktur, der Bemessungskriterien, der seismischen Beurteilung und der Bauverfahren für Verstärkungen dargelegt.

1. INTRODUCTION

A number of reinforced concrete buildings, damaged by recent severe earthquakes in Japan, required extensive repair and also strengthening. The Tokachi-oki Earthquake of 1968 heavily damaged a large number of low-rise buildings. Some of these were strengthened by the addition of structural walls. Because of the lack of guidelines, the design as well as the construction for strengtheening was based on engineering judgement alone. This was practically the first experience for Japanese engineers to extensively strengthen existing buildings against severe earthquakes. The destructive 1978 Miyagiken-oki Earthquake was also followed by the strengthening of a number of buildings. However, in this case materials and techniques for construction were specifically selected and the design was based on experimental or analytical investigations or on guidelines, where these were available [1, 2].

Current studies have indicated that there is a wide scatter in the level of seismic resistance of existing buildings. It was found that a considerable number of low to medium-rise buildings, designed and constructed in accordance with previous building codes, may need strengthening. Consequently a number of buildings, considered hazardous, were strengthened or rebuilt [3 ~ 5]. While several experimental studies, relevant to the seismic strengthening of existing structures, have been conducted to establish guidelines for design and construction [6 ~ 13]. Thus the necessity for strengthening of hazardous buildings was recognized for some time, and as a consequence an advisory committee for the Japanese Ministry of Construction prepared design guidelines [14] in 1977. These design guidelines were intended to be used in conjunction with the method of evaluation of seismic safety of existing buildings, proposed by the same committee [15]. This method was described in some detail in references 16 and 18. In a large number of cases these guidelines have been used in Japan.

Available data of the design and construction for strengthened buildings as well as existing test data relevant to the strengthening have been integrated and reviewed by a committee chaired by the authors. The committee was organized in the Japan Concrete Institute on September 1981 to provide improved guidelines for seismic strengthening based on as much up-to-date information as possible. This paper describes the knowledge obtained from the review, as of March 1983, with emphasis on 1) structural effects of the strengthening, 2) design criteria and construction techniques for the strengthening and 3) seismic resistance of strengthened buildings. As the committee has been reviewing the integrated data, further results will be presented in other opportunities.

2. GENERAL PRINCIPLES OF SEISMIC STRENGTHENING

2.1 Aims of Seismic Strengthening

The aims of the strengthening are to provide (1) increased strength with respect to lateral loading, or (2) increased ductility, or (3) a proper combination of these two features. These concepts are illustrated in Fig. 1. To provide increased strength is considered as being the most promising approach in the strengthening of low to medium-rise buildings. Even if ductility is provided, increased strength is expected to reduce the magnitudes of inelastic displacements. It is considered to be also important to reduce eccentricities resulting often from the irregular distribution of stiffness within a story or throughout the entire structure.

2.2 Construction Techniques

Typical strengthening methods are assembled in the chart of Fig.1. Examples of construction techniques to meet both criteria for strengthening, the increased strength and increased ductility, are given in Fig.2. Generally new elements may be added to existing structures to give increased strength, or existing framing elements may be reinforced with new materials to improve their ductility. Infilled walls and side walls, shown in Fig.2(a), are cast-in-situ or precast wall elements which are attached to frames or to beams, as appropriate. In the process of increasing the ductility of existing columns, such as shown in Fig.2(b), the aims is to increase their shear strength. This is achieved by wrapping techniques shown in these illustrations. A narrow gap at the ends of the encasement is provided to avoid the undesired increase of the flexural strength of the member at that section.

For the systems shown in Fig.2(a), cast-in-situ or precast concrete is commonly used with the various connection techniques. Typical details for such connections are given in Fig.3. Careful attention must be paid to connections, because they will strongly affect the behavior of the strengthened structures. High pressure pumping of the fresh concrete or nonshrink material may be necessary to avoid the formation of cavities between the new and existing elements.

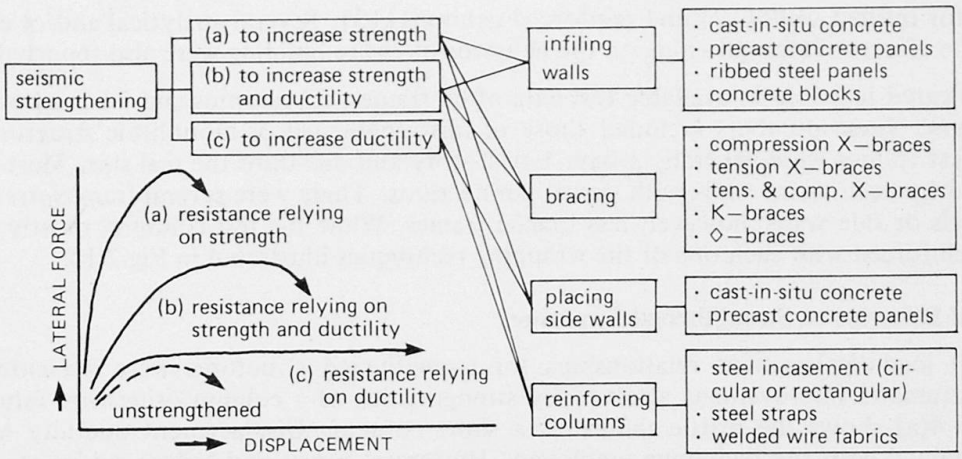


Fig. 1 Concepts of Seismic Strengthening and Strengthening Methods

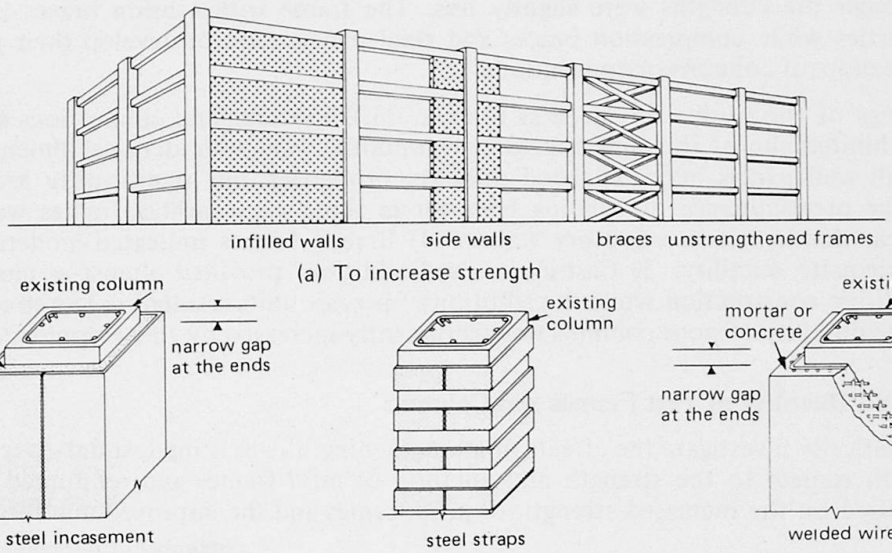


Fig. 2 Typical Construction Techniques for Seismic Strengthening

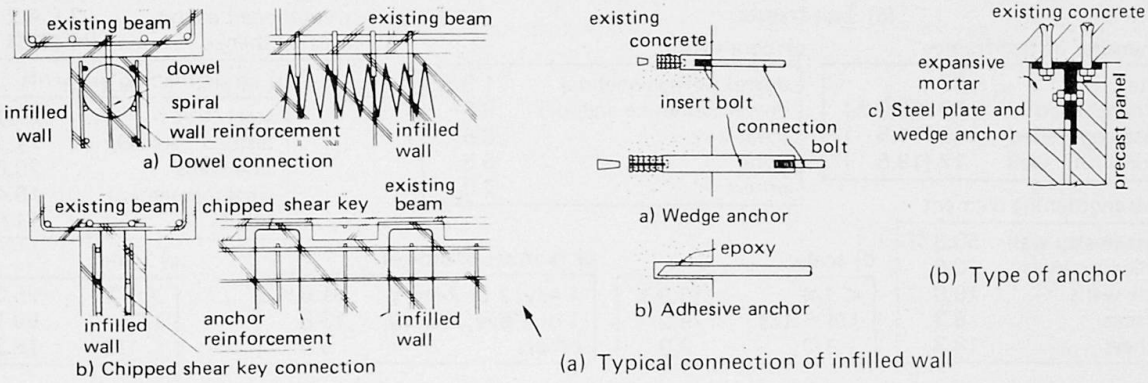


Fig. 3 Connection of Infilled Wall

3. REVIEW OF RESEARCH ON SEISMIC STRENGTHENING

3.1 Research on Seismic Strengthening and Existing Test Data

The number of strengthened structures that have been examined experimentally served as a background in formulating the design guidelines [14]. Since the proposal of the guidelines, further experimental studies have been conducted. The earliest tests, performed several years after the 1968 Tokachi-oki Earthquake, were aiming at the improvement of ductility of columns [6, 7] and at boosting the strength of frames by the addition of concrete walls [7, 8]. Subsequently one-story infilled frames with various connection details [7, 9, 11] and bracing systems [10, 11] were examined. Currently three-story frames, strengthened by infilling and bracing, were tested. Tests were also

conducted for infilled walls [12] and reinforced columns [13]. Several analytical and/or experimental studies of the effects of strengthening on the behavior of entire building were also reported.

As indicated in Table 1, available test data of 87 frames, 67 columns and 5 beams were collected for the review. These numbers included those of unstrengthened or monolithic structures for reference. The test frames were generally 1-bay, 1 or 2-story and one-third the real size. Most frames were strengthened by cast-in-situ walls with dowel connections. There were several frames strengthened by precast panels or side walls, however, less braced frames. While the test columns, mostly half the real size, were reinforced with each one of the wrapping techniques illustrated in Fig.2(b).

3.2 Typical Behavior of Strengthened Structures

Typical load-displacement relationships for strengthened structures were presented. Figure 4 shows the dramatic improvement attained by strengthening of a column using wire fabric wrapping [7]. Figure 4(a) shows the brittle failure of a short column. Displacement ductility larger than 6 could be attained with the technique employed. Hysteresis curves and failure modes of strengthened frames by various techniques [11] were presented in Fig. 5. Infilled walls behaved similarly to monolithic wall, though the strengths were slightly less. The frame with tension braces indicated good ductility properties while compression braces and steel panels did not develop their yield strengths because of the failure of columns or connections.

The findings of the studies are given as follows. 1) When adequate connections were provided, infill frames exhibited almost the same strength as monolithic wall with identical dimension. 2) Multiple precast infill wall panels indicated good ductility properties, but significantly less strength was attained. 3) The predominance of bending behavior in three-story infilled frames was observed in contrast to shear dominance in one-story frames. 4) Braced frames indicated moderate increase of strength and adequate ductility. 5) Cast-in-situ wall additions provided almost as much strength as identical monolithic construction while the addition of precast units resulted in less strength but more ductility. 6) The ductility of poor columns was significantly increased by the wrapping techniques.

3.3 Strength and Ductility of Test Frames and Columns

To systematically investigate the effects of strengthening, the existing test data were quantitatively reviewed with respect to the strength and ductility of infill frames and reinforced columns. The review was focused on the increased strength of infill frames and the improved ductility of reinforced

Table 1 Outline of Reviewed Test Data

(a) Test frames

a) number of test frames

total	87
strengthened	60 (69.0 %)
unstrengthened	10 (11.5)
monolithic wall	17 (19.5)

b) strengthening element

cast-in-situ wall	53.3 %
precast panels	10.0
side walls	10.0
braces	8.3
others	18.3

c) connection

dowel (wedge anchor)	71.9 %
dowel (adhesive anchor)	8.8
shear key	8.8
none	3.5
others	7.0

d) scale

< 1/4	13.8 %
1/4 ~ 1/3	78.2
> 1/3	8.0

e) number of stories

1-bay, 1 or 2-story	81.6 %
1 or 2-bay, 3-story	13.8
others	9.6

(b) Test columns and beams

a) number of test columns and beams

total	72
strengthened columns	45 (62.5 %)
strengthened beams	3 (4.2)
unstrengthened	24 (33.3)

b) strengthening elements

steel straps	27.1 %
welded wire fabrics	27.1
side walls	20.8
steel incasement	10.4
others	14.6

c) scale

> 1/2	25.0 %
1/2	59.7
< 1/2	15.3

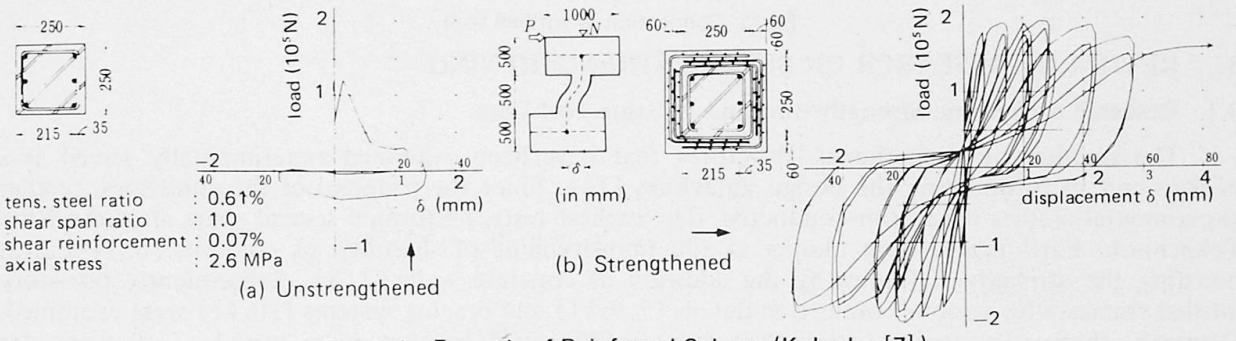


Fig. 4 An Example of Reinforced Column (Kokusho [7])

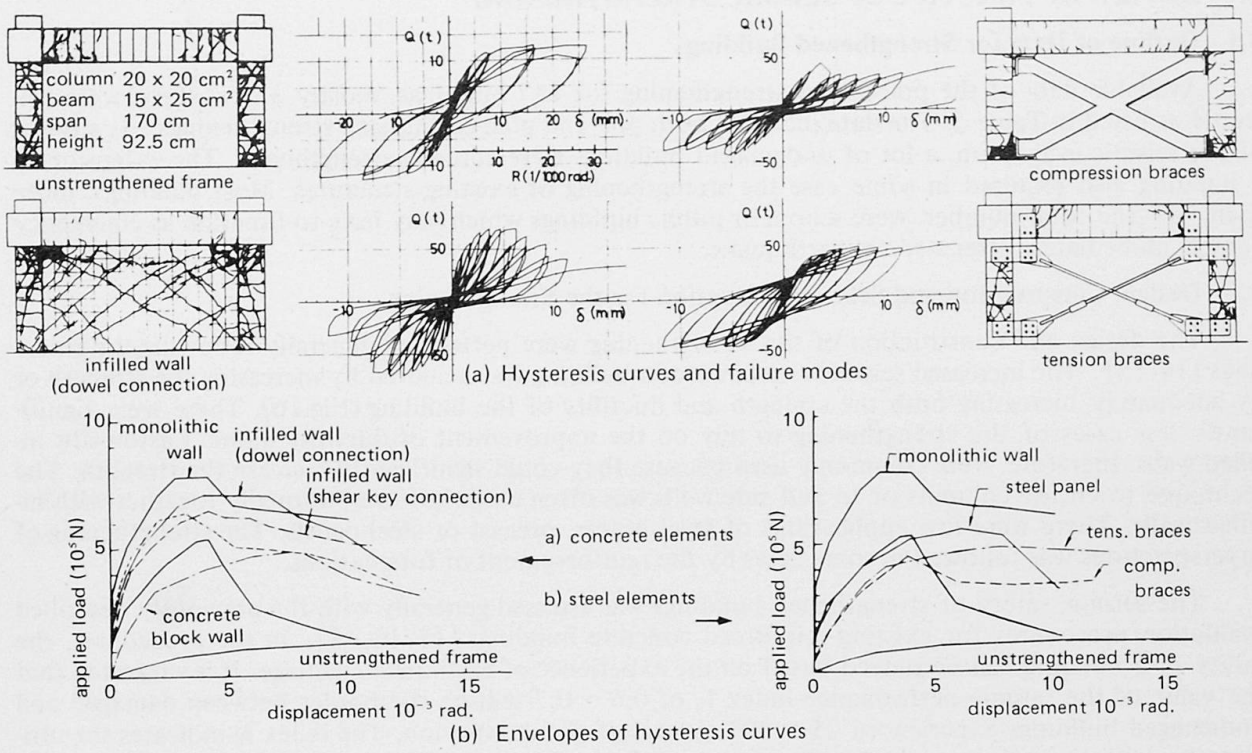


Fig. 5 An Example of Strengthened Frames (Sugano and Fujimura [11])

columns, respectiely. The selected frames for the review were those strengthened by infilling concrete wall and they included several frames of three-story or precast panels. Narrow slits were provided along columns in some of the infilled walls. The selected test columns were those reinforced with each one of the wrapping techniques in Fig.2(b). Figures 6 to 11 show comparisons of the obtained ultimate strength of infill frames with the strength of walls of monolithic construction or unstrengthened frames, and with the calculated strength by available procedures. The observed displacement ability of reinforced columns is presented in Fig.12 to 14.

The observed ultimate strength of cast-in-situ infill frames was in a wide range of 1.5 to 7.0 MPa in terms of the nominal shear stress of wall, and it clearly increased with the strength of connections (Figs.6 and 10). Their strength was 60 to 100% the strength of monolithic wall with identical boundary elements, and 2 to 5 times that of unstrengthened frames, respectively (Fig.7 and 8). These frames experienced similar displacement at the ultimate strength, 0.003 to 0.010 radian in terms of the slope of deflection, to that of monolithic walls (Fig.6). Another group of frames, those of three-story, with narrow gaps along columns and with multiple precast panels, indicated different feature because of the predominance of bending behavior. They obtained considerably less strength, however, significantly larger displacement ability than those of the above group.

Figures 9 and 11 presented the calculated shear strength as molithic wall [18] or that by the design guidelines [14]. The design strength of an infilled wall assembly in Fig.11 was given as the smaller of either; (1) the total shear strength of the wall panel and both columns, or (2) the total shear strength of one column and the connection along beam and the punching shear strength of the other column. The observed strength of cast-in-situ infill frame was 60 through 115% the calculated strength as monolithic wall. This indicated again that these walls could have as much strength as that of monolithic wall when adequate connections were provided. The above design procedure provided considerable margin of the strength for cast-in-situ infill frames (Fig. 11).

The significant improvement of ductility of columns was achieved by any one of the wrapping techniques (Fig.12 to 14). Displacement capacity larger than 0.02 rad. or displacement ductility larger than 3.5 could be attained. The increased displacement capacity was 1.5 through 7 or more times the capacity of unstrengthened columns. Moderate improvement of strength, resulting from the increased concrete section, was also attained by wire fabric wrapping and steel incasement. The design equation for the ductility factor [16,18] in Fig.14 provided considerable safe-side estimation.

4. REVIEW OF PRACTICE OF SEISMIC STRENGTHENING

4.1 Outline of Data for Strengthened Buildings

Available data of the practice of strengthening for 157 buildings, mostly 3 or 4-story, were collected as listed in Table 2. The data included both pre- and post-earthquake strengthenings. As a result of the seismic evaluation, a lot of undamaged buildings were actually strengthened. The extension of a building also required in some case the strengthening of existing structures. Most buildings, more than 70% the total number, were school or public buildings which may have to function as emergency centers immediately after a severe earthquake.

4.2 Design, Construction and Seismic Evaluation for the Strengthening

The design and construction of the strengthening were performed generally following the guidelines [14,15]. The increased seismic resistance of a building was achieved by increasing the strength or by adequately increasing both the strength and ductility of the building (Fig.16). There were significantly less cases of the strengthening to rely on the improvement of ductility alone. Cast-in-situ infilled walls, therefore, were commonly used because they could significantly increase the strength. The technique to encase columns or to add side walls was often used, however, normally together with infilled walls. There were few applications of steel braces, precast or steel panels. The strengthening of superstructures was followed in some cases by the reinforcement of foundations.

The seismic safety of strengthened buildings was assessed generally with the previously described evaluation procedures for existing reinforced concrete buildings [15, 16, 18]. In the procedures, the safety of a building can be judged based on the experience of earthquake damage. It is suggested that the value of the seismic performance index I_S of 0.6 ~ 0.7 will be the border between damaged and undamaged buildings experienced 25 ~ 30% g level of ground motion. The index I_S indicates the ultimate horizontal strength of the building in terms of the shear coefficient, or equavalent strength when

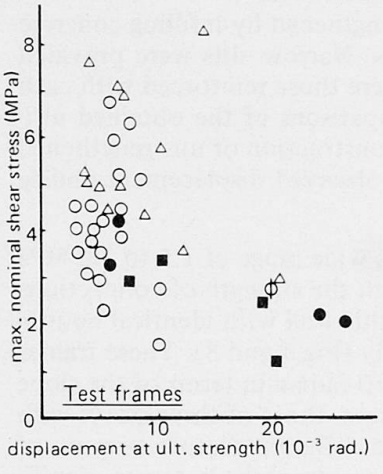


Fig. 6 Ultimate Strength and Displacement

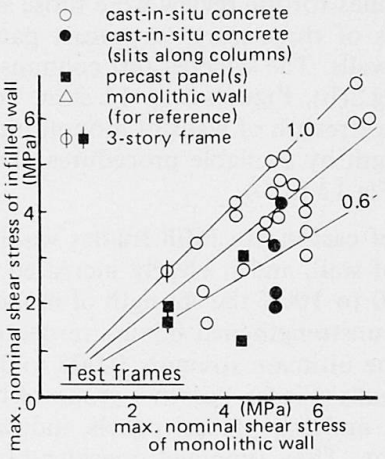


Fig. 7 Strengths of Infilled and Monolithic Walls

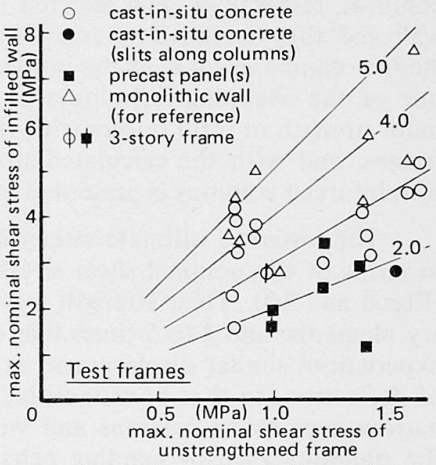


Fig. 8 Strengths of Wall and Unstrengthened Frame

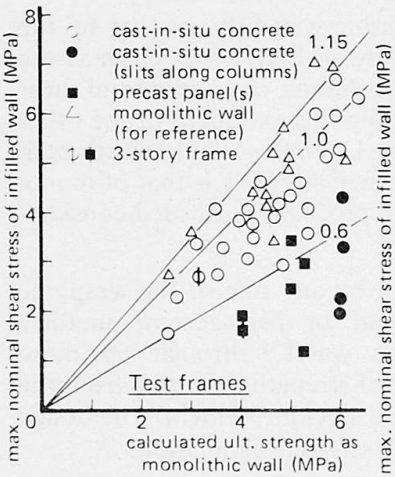


Fig. 9 Observed & Calculated Strengths of Wall

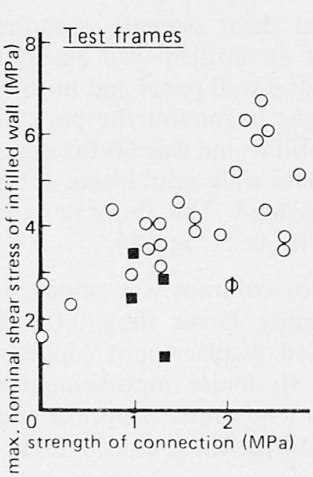
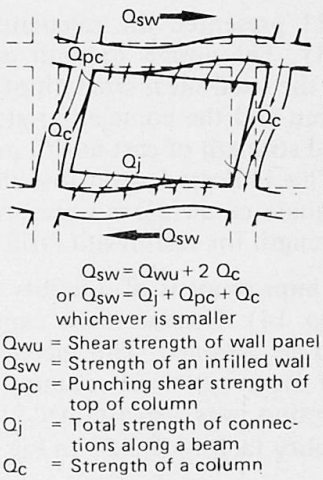


Fig. 10 Strengths of Infilled Wall and Connection



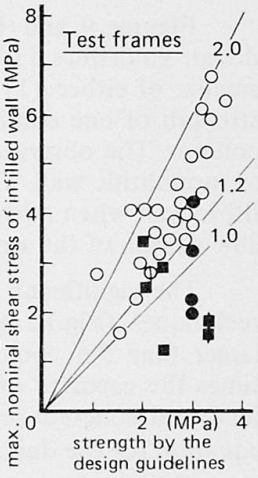


Fig. 11 Observed and Design Strengths of Infilled Wall

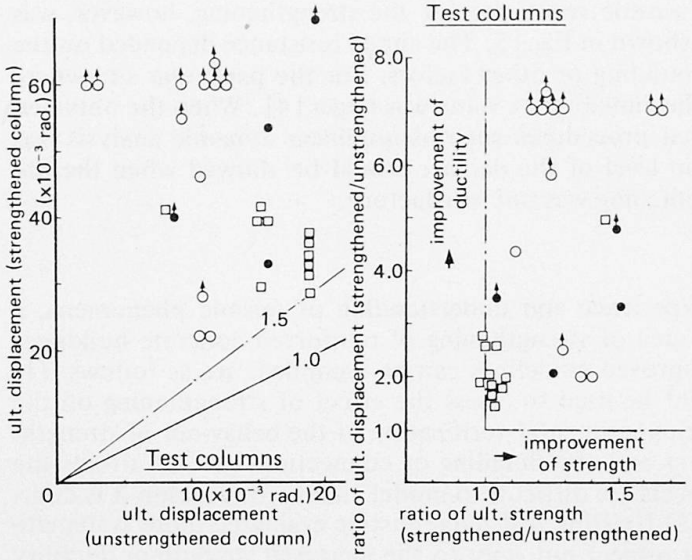


Fig. 12 Increased Displacement Capacity

Fig. 13 Effect of Strengthening

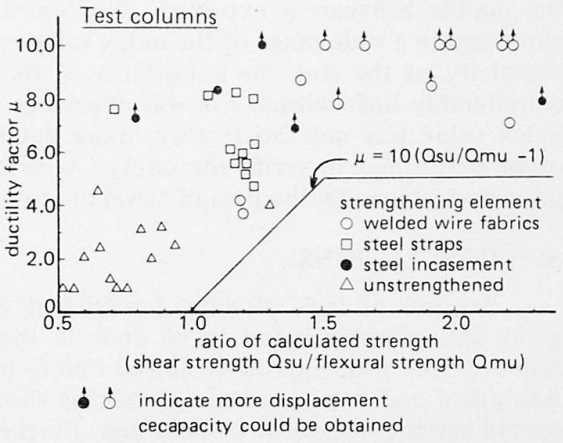


Fig. 14 Improvement of Ductility

Table 2 Outline of Reviewed Data of Strengthened Buildings

a) number of buildings

total	157
1 ~ 2-story	16 (10.2 %)
3-story	65 (41.4)
4-story	41 (26.1)
5 ~ 10-story	35 (22.3)

b) use of building

school	42.7 %
public office	28.7
private office	7.6
residential bldg	7.6
others	13.8

c) reason for strengthening

pre-earthquake evaluation	66.9 %
post-earthquake evaluation	17.8
others	15.3

d) strengthening element or method (290 cases)

cast-in-situ-concrete walls	130 (44.8 %)
reinforcement of columns	52 (17.9)
addition of side walls	43 (14.8)
reinforcement of foundation	11 (3.8)
reinforcement of beams	5 (1.7)
steel braces	3 (1.0)
others	46 (15.9)

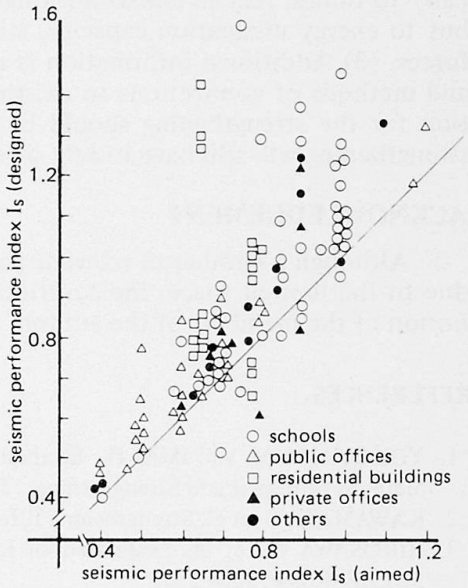


Fig. 15 Seismic Resistance of Strengthened Buildings

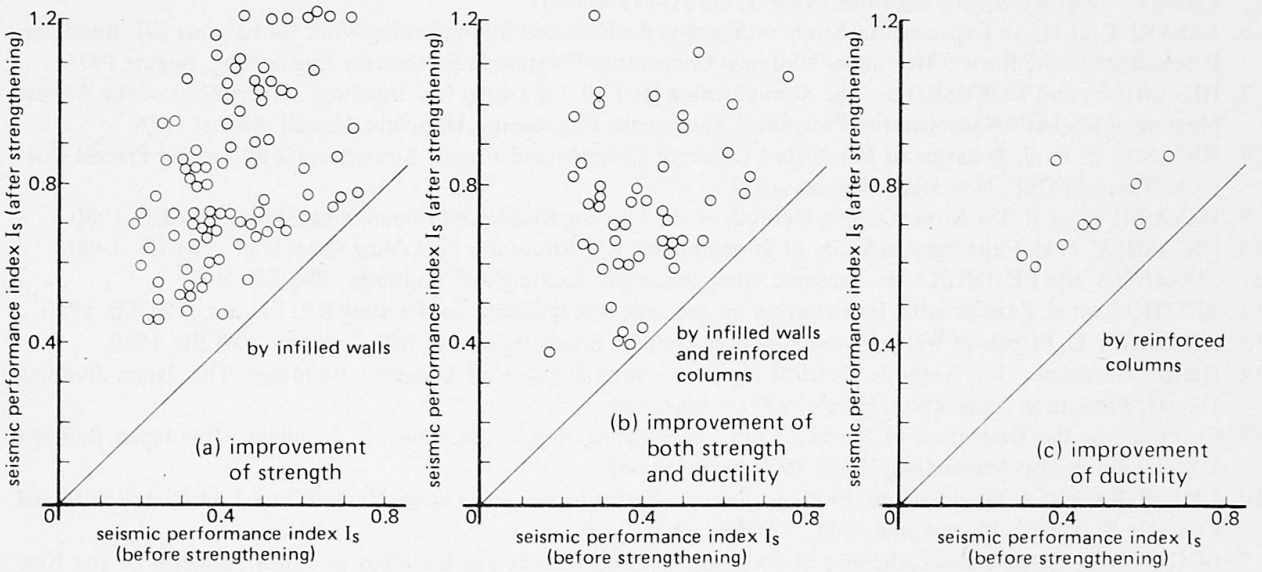


Fig. 16 Improvement of Seismic Resistance of Strengthened Buildings

the ductile behavior is expected. The aimed seismic resistance for the strengthening, however, was expressed in a wide range of the index value as shown in Fig.15. The aimed resistance depended on the seismicity of the site, the importance of the building or other factors. For the particular site where considerably high seismic risk was expected, the aimed index value was larger [4]. When the obtained index value was not satisfactory, more detailed procedures such as nonlinear dynamic analysis was often performed to verify the safety. A certain level of the damage would be allowed when the improved resistance by the possible level of strengthening was not satisfactory.

5. CONCLUSIONS

Because of lack of data, insufficient experience and understanding of seismic phenomena, a great deal of work is yet to be done in the area of strengthening of reinforced concrete buildings. Some of the problems to be solved before improved guidelines can be compiled, are as follows: (1) Analytical and experimental approaches should be used to assess the effect of strengthening on the overall seismic behaviour of buildings. Further experimental verification of the behaviour of strengthened sub-assemblages is required. Workmanship and the detailing of connections greatly affects the response of strengthened structures. These aspects are difficult to model and for this reason it is desirable to test large if not full scale specimens. (2) Existing test data must be evaluated more systematically to obtain relable global information with respect not only to the increased strength or ductility but to energy dissipation capacity, stiffness deterioration and potential strength to resist earthquake forces. (3) Additional information is required with respect to the use of precast and bracing elements and methods of connections to existing structural systems. (4) Available data for design and construction for the strengthening should be compiled for the benefit of potential users. The approach to strengthening will still have to rely on experience so gained and on judgement.

ACKNOWLEDGEMENT

Although a number of relevant papers, particularly those in Japanese, were not listed as references due to the limited space, the contribution of the authors of those papers is acknowledged. The contribution of the members of the authors' committee in the Japan Concrete Institute is also acknowledged.

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Repair and Strengthening of Reinforced Concrete Buildings

Réparation et renforcement de bâtiments en béton armé

Reparatur und Verstärkung von Stahlbetongebäuden

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SUMMARY

In 1978, a severe earthquake shook the Peninsula Izu located 120 km southwest of Tokyo, Japan. Several R/C school buildings were damaged. This paper presents the damage, the method of repair and strengthening, and the seismic capacity of the pre- and post-strengthened buildings.

RESUME

En 1978, un tremblement de terre violent s'est produit dans la péninsule d'Izu située à 120 km au sud-ouest de Tokyo, Japon. Quelques bâtiments d'école ont subi des dommages. L'article présente ces dommages, la méthode de réparation et de renforcement et la capacité sismique avant et après le renforcement.

ZUSAMMENFASSUNG

Im Jahr 1978 hat ein mächtiges Erdbeben die Halbinsel Isu, die 120 km südwestlich von Tokio liegt, heimgesucht. Viele Schulgebäude aus Stahlbeton wurden beschädigt. Der Beitrag beschreibt die Schäden, die angewendeten Methoden der Reparatur und der Verstärkung sowie den seismischen Widerstand der Gebäude vor und nach der Verstärkung.

1. INTRODUCTION

On January 14, 1978, a severe earthquake shook the Peninsula Izu located about 120 km southwest from Tokyo, Japan(Fig.1). The earthquake had a Richter magnitude of 7.0 and its epicenter was located about 15km from a town, Higashi-Izu cho. The maximum intensity at the town was V or more in JMA scale (VIII in Modified Mercalli scale)(Fig.2). In the area of the intensity of more than V, there were five reinforced concrete school buildings.
Among them, a part of one junior high school building was severely damaged, another junior high school building had a moderate damage, and the rest were slightly damaged. The severely damaged wing of one junior high school building was demolished and reconstructed, and another junior high school building was repaired and strengthenend after the earthquake.
Seismograph was not installed near the site, however, the peak ground acceleration was estimated about 20 percent of the gravity by the observation of the damage to the structures, the overturning of tomb stones, etc.,([1],[2]).
The primary purpose of this paper is to present the studies on 1) Damage to the reinforced concrete school buildings, 2) Structural analysis on seismic capacity of the school buildings based on the Guideline for the Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings([3]), 3) Method of repair and strengthening of the damaged school buildings, and 4) Precise structural analysis on the seismic capacity of one of the school buildings before and after strengthening.

2. DAMAGE TO THE REINFORCED CONCRETE SCHOOL BUILDINGS

A short discription about earthquake damage is shown in Table 1. Among them, the damage of Inatori junior high school building was most severe, while the frames could maintain structural performance after the earthquake. As shown in Fig.3, a major fault line was found across the ground. A number of cracks were found in columns, beams, walls and slabs. The damage was most significant in the slabs at the center core. The crack pattern showed that the east wing and the west wing moved in the opposite direction(Fig.4).
Atagawa junior high school building was one of the damaged buildings. A number of cracks were seen in columns, beams and walls. Both ends of the building were sunk several centimeters. By the excavation of ground, it was found that the footing beams at the west end were failed in shear. Seismic performance of other school buildings were quite well, while hair cracks in the structures, and damage to the non-structural elements were found.

3. STRUCTURAL ANALYSIS ON SEISMIC CAPACITY AND METHOD OF REPAIR AND STRENGTHENING

Just after the earthquake, a temporary therapy was done to the damaged school buildings. Then, the permanent therapy was employed under the detailed consideration of seismic performance of the buildings.
In the case of Inatori junior high school buildings, the center core was demolished immediately and reconstructed. The expansion joints were newly provided between the center core and both wings. The cracks in the west and east wing were repaired by epoxy injection. Cracks in other buildings were also repaired by epoxy injection.
After the temporary therapy, a detailed diagnosis was done on the seismic performance to future earthquake effect. A criteria for structural performance was to avoid collapse to the ground motion having an acceleration of 30 percent of the gravity, since a severe earthquake (so called Tokai Earthquake) is predicted at the west sea of the Izu Peninsula by seismologists.
Seismic performence of the damaged and undamaged school buildings were examined by the Guideline to Evaluate Seismic Capacity of Existing Medium and Low-Rise Reinforced Concrete Buildings in Japan([3],[4],[5]).
The overall method of the guideline consists of three different level procedures;

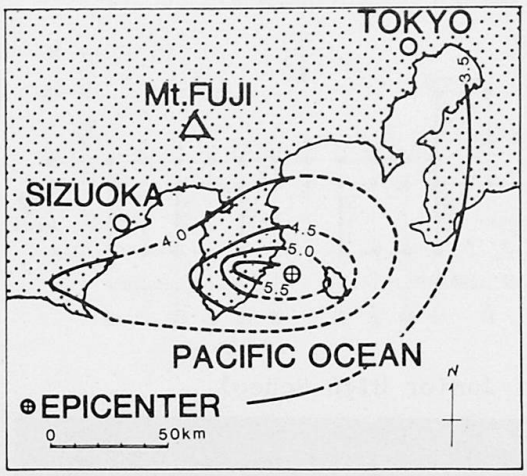


Fig. 1 Seismic intensity of 1978 Izu Oshima Kinkai Earthquake (Reference[1])

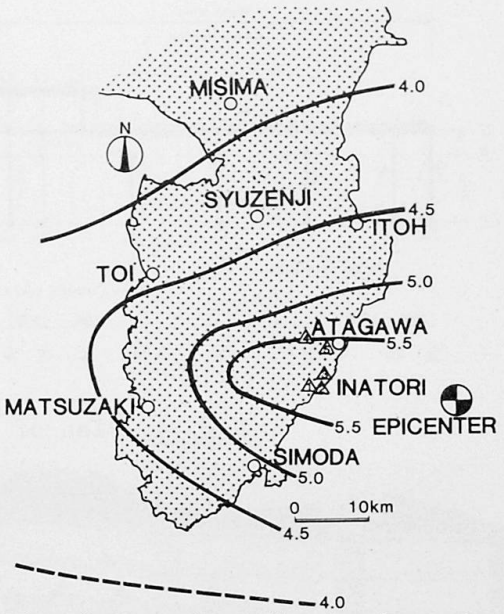


Fig. 2 Location of schools and seismic intensity([1])

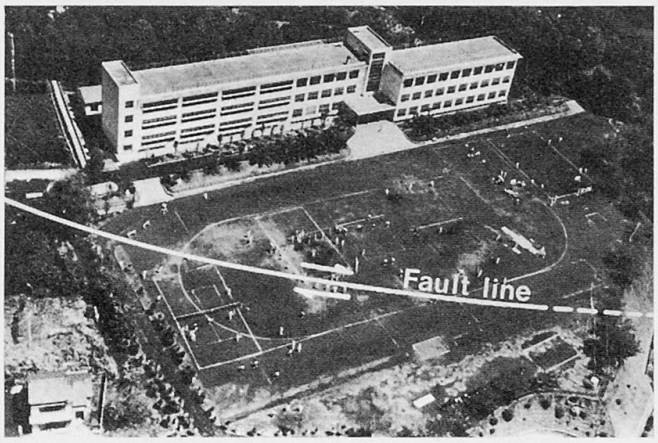
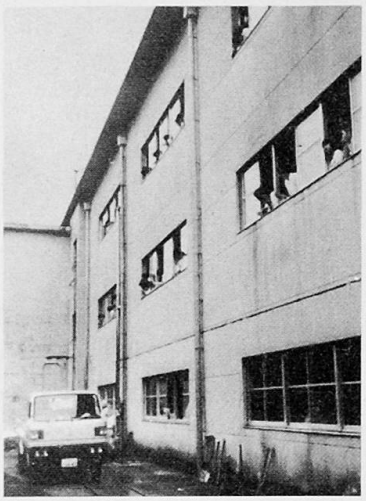


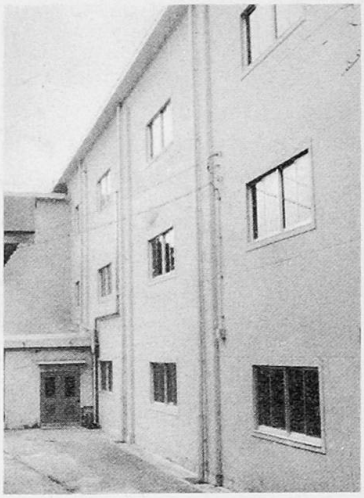
Fig. 3 Bird's eye view of Inatori Junior High School

ID Number in Fig. 2	Name of School	Location	Type of Structure Number of Stories	Year of Construction	Damage	Degree of Damage
1	Inatori Junior High	Higashi-Izu-cho Inatori	RC/3	1955	Broken in two parts at center core. Shear failure of some columns. Numerous cracks in columns, beams, walls and slabs. Surface breakes on ground.	midium damage (partially heavy damage)
2	Inatori Elementary	same as above	RC/4	1970	Hair cracks in walls and columns. Inclination of eaves of porch. Breakes of window glass. Fall of lighting implements.	slight damage
3	Inatori High	same as above	RC/4 (two buildings)	1965	Collision of expansion joint. Surface breaks on ground	slight damage
4	Atagawa Junior High	Higashi-Izu-cho Yaramoto	RC/3 penthouse	1962	See Fig. 4. Breaks of under-ground pipes.	midium damage
5	Atagawa Elementary	same as above	RC/3 and RC/4 (two buildings)	1975	Shear cracks in walls and some slabs	slight damage

Table 1 Discription of damage



a) Before strengthening



b) After strengthening

Fig. 5 Shear walls of Inatori Junior High School

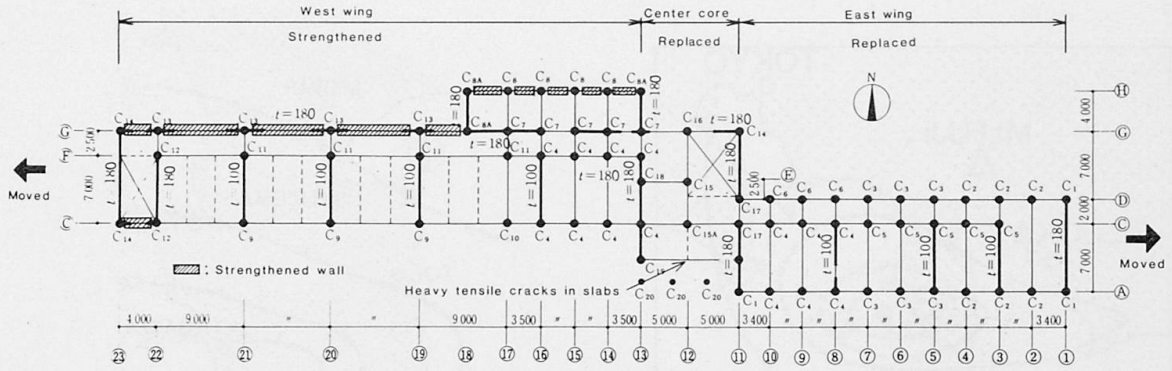
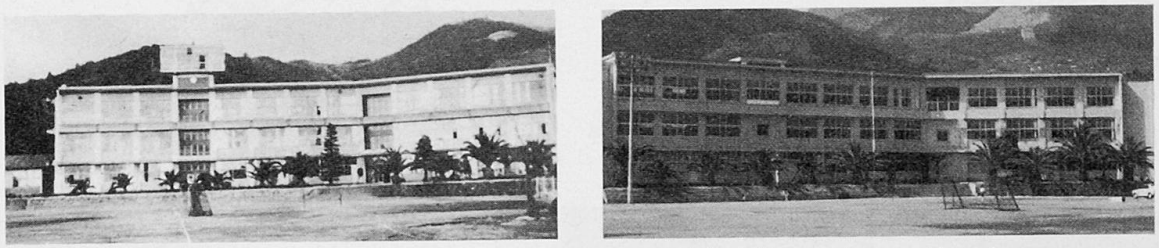
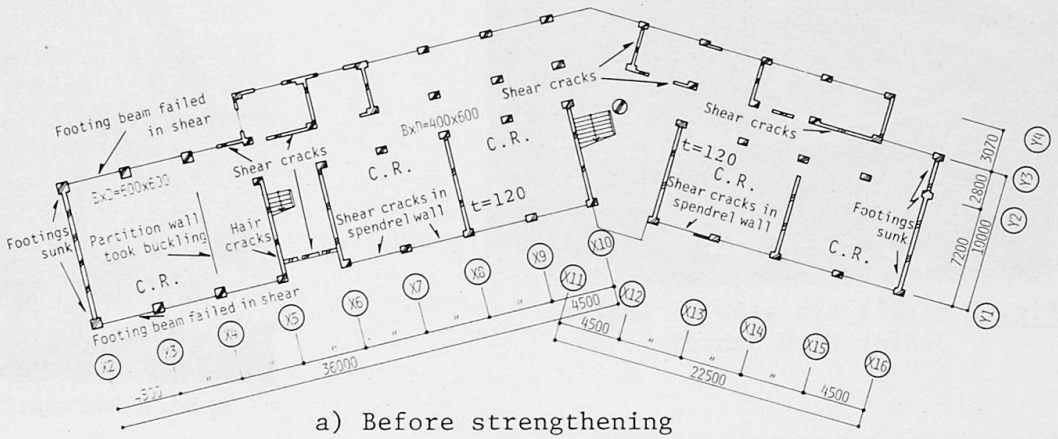


Fig. 4 Plan of Inatori Junior High School

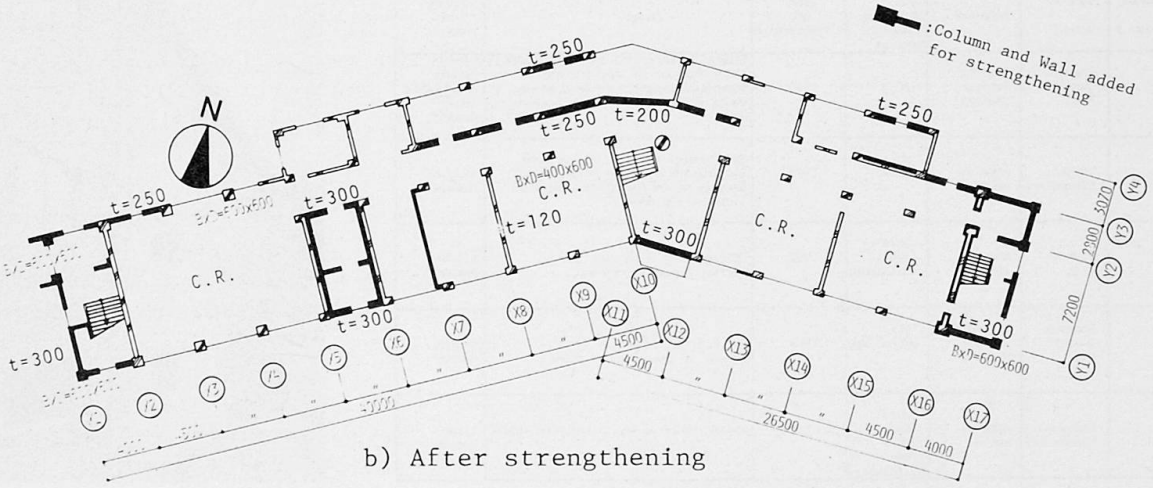


a) Before strengthening b) After strengthening

Fig. 6 Overall views of Atagawa Junior High School



a) Before strengthening



b) After strengthening

Fig. 7 Plan of Atagawa Junior High School

first, second and third level procedures. The first level procedure is the simplest, but most conservative of the three, while the basic concept is common for all three.

In the guideline, the unified seismic performance index of structure (Is) up to six stories is evaluated by the following equation at each story and to each direction;

$$Is = Eo \cdot G \cdot S_D \cdot T \quad \cdots\cdots\cdots\cdots (1)$$

where, Eo = basic structural index calculated by ultimate horizontal strength, ductility, number of story and story level considered

G = local geological index to modify the Eo-index

S_D = structural design index to modify the Eo-index due to the grade of the irregularity of the building shape and the distribution of stiffness

T = time index to modify the Eo-index due to the grade of the deterioration of strength and ductility

The Is-indices by second level procedure are shown in Table 2([2]). Since the seismic performance to the span direction is very high due to shear walls located between class rooms, the Is-indices to the logitudinal direction are shown in the table.

It is recognized that the grade of the Is-indices corresponds to the degree of damaged, except Inatori high school building. In the case of the high school, the building is considered to be shaken possibly to the span direction by the observation of the damage of other structures and the ground on and around the school complex.

By the results of earthquake response analysis to idealized structural models and the experience of earthquake damage, Is-index of about 0.6 is a border of damage and undamage to the ground motion having the acceleration of 25-30 percent of the gravity([3]-[6]).

Considering these analyses and the grade of damage, the following permanent therapy was adopted for two junior high school buildings : 1) Inatori Junior High School — The east wing was also demolished and replaced to new building in 1983. The west wing was strengthened by adding reinforced concrete shear walls in the existing frames(Fig.4,5). By this therapy, the Is-index of the west wing is estimated more than 1.0. 2) Atagawa Junior High School — This building was also strengthened in 1983 as shown in Fig.6,7,8. Reinforced concrete walls were added in the existing frames. Reinforced concrete cores were newly constructed at the both ends of the building and jointed to the existing part. The penthouse was taken off to reduce the building weight.

Detail of joints are shown in Fig.9, which are typical practices in Japan. By this therapy, the Is-index after strengthening is estimated more than 1.0.

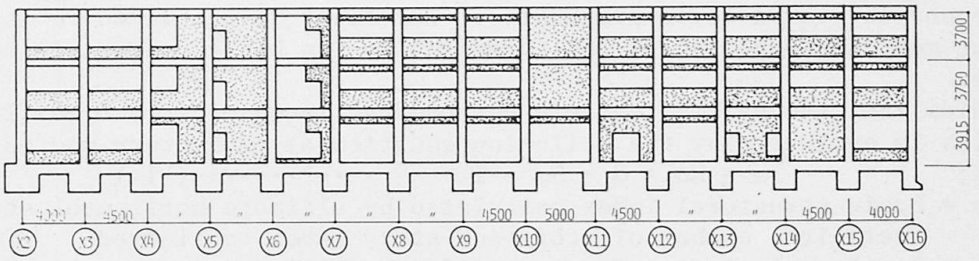
4. PRECISE ANALYSIS OF ATAGAWA JUNIOR HIGH SCHOOL BUILDING

In order to verify the siesmic performance estimated by the Guideline, a precise analysis of Atagawa junior high school building on the ultimate strength and failure mechanism to lateral force was done based on the non-linear behavior of the structural members([7]).

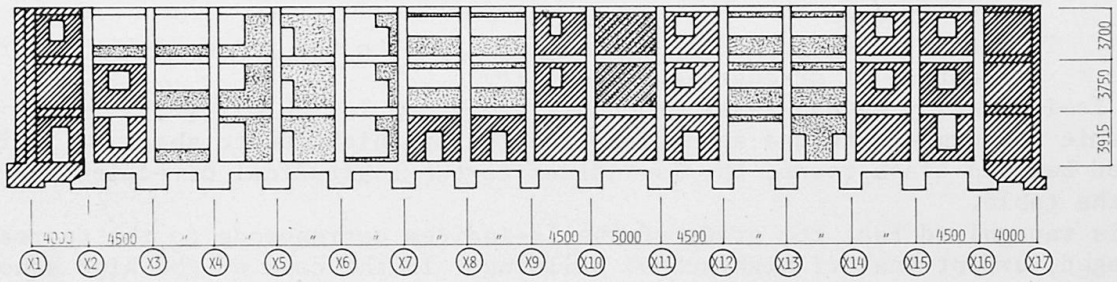
As shown in Fig.10, each member was idealized into beam model having rigid zones at both ends. In order to represent the non-linearity of member, one-component model consisting of series combination of flexural spring and shear spring was used. A degrading tri-linear type of hysteretic rule was used for flexural spring and an origin-oriented type for shear spring. At the bottom of the wall, bi-linear elastic spring was attached representing the rotation of footing. A bent of the frame lines was not considered.

Lateral force distribution along stories was assumed as inverse triangular type in terms of lateral force coefficient. Based on the assumptions mentioned above, lateral force-displacement relationship of each frame was calculated.

Base shear coefficent-story drift relationship at the first story is shown in Fig.11. At the stage of the displacement of 2cm, where shear failure occurs



a) Before strengthening



b) After strengthening

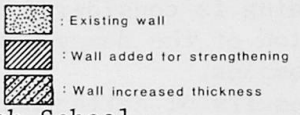
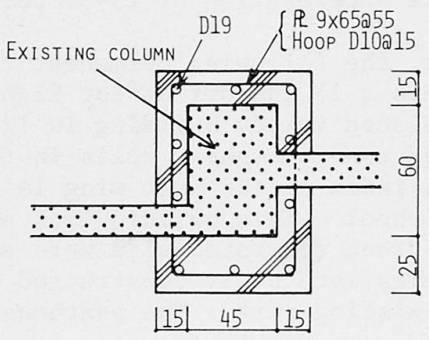
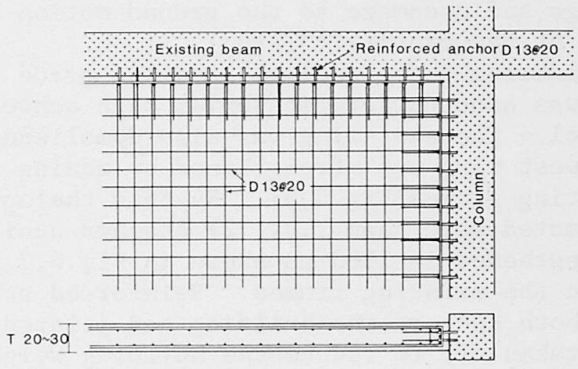


Fig. 8 Frame #Y3 of Atagawa Junior High School



a) Column

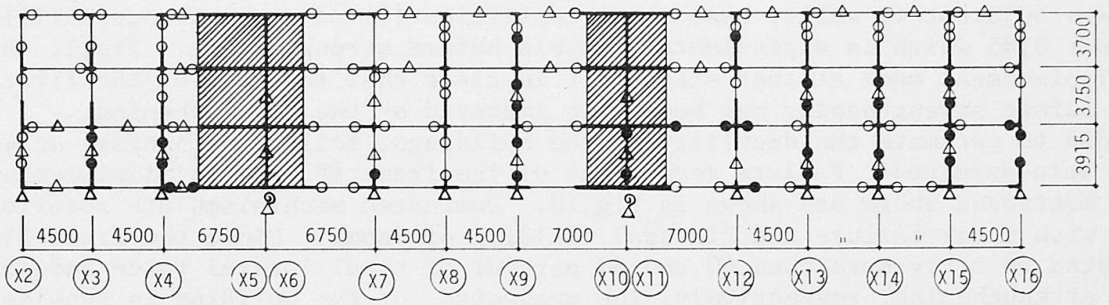


b) Wall

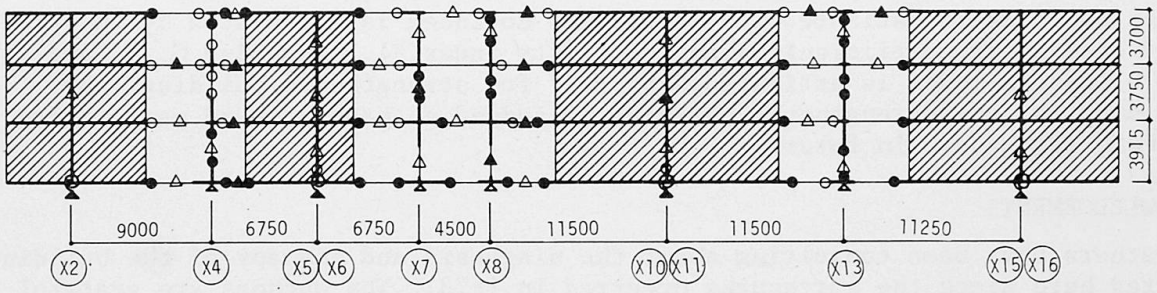
Fig. 9 Detail of joints

ID Number in Fig. 2	Name of School		I_S-indices at First Story
1	Inatori Junior High	East Wing	0.42
		West Wing	0.30
2	Inatori Elementary		0.67
3	Inatori High		0.35
4	Atagawa Junior High		0.43
5	Atagawa Elementary	North Building	0.61
		South Building	0.72

Table 2 I_S- indices by second procedure



a) Before strengthening



b) After strengthening

○ : Flexral crack
● : Frexral yielding
△ : Shear crack
▲ : Shear failure
: Rotational failure

Fig. 10 Failure mechanism of Atagawa Junior High School (δ_1=2 cm, R_1=1/200)

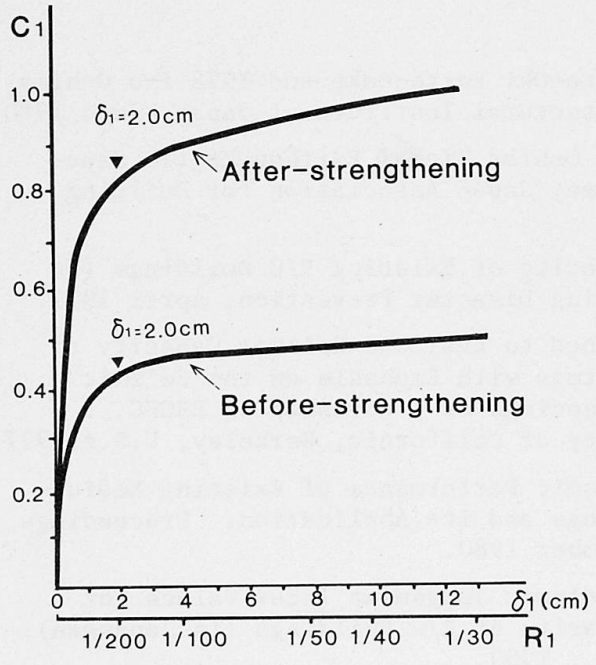


Fig. 11 Base shear coefficient - story drift relationship of Atagawa Junior High School (1st story)

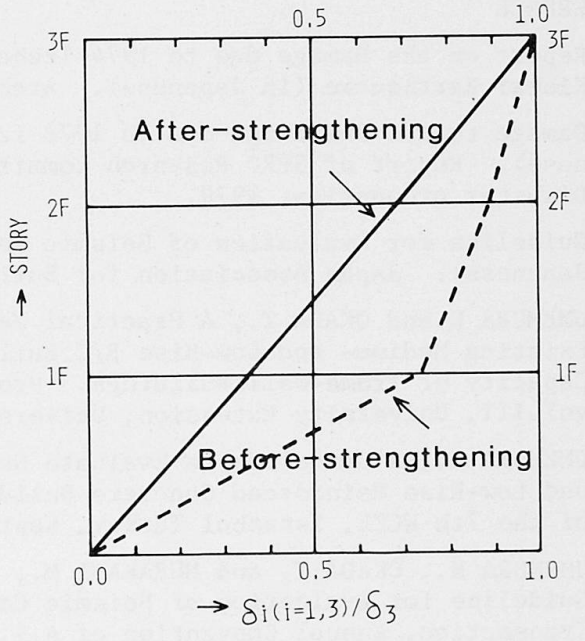


Fig. 12 Mode of displacement(δ_1=2 cm) of Atagawa Junior High School

in beams adjacent to walls, base shear coefficient(C) of strengthened building is about 0.85 which is approximately double before strengthening. Fig.12 shows the displacement mode at that stage. It is clear that the sway at the first story before strengthening has been also improved by the strengthening.
In order to estimate the ductility of the buildings, failure mechanisms of structures were examined. Failure mechanisms of the frame #Y3 at the displacement stage mentioned above are shown in Fig.10. Dominated mechanisms are rotation of walls with shear failure and flexural yielding of beams. Since the frame #Y3 is estimated to carry more than 40 and 60 percent of total lateral force before and after strengthening, respectively, the mechanism of the building is considered as same as the frame #Y3.
According to the third level procedure of the Guideline, a ductility index(F) of such building is estimated 1.0 to 1.5 and Eo-index is calculated by the product of base shear coefficient(C) and ductility index(F). Assuming G, S_D and T are 1.0, the Is-index is estimated 0.85-1.27 for strengthened building. By these studies, the strengthened building is judged to have a sound performance to severe earthquake in future.

ACKNOWLEDGEMENTS

The authors have been consulting about the diagnosis and therapy of the buildings reported here since the earthquake occurred in 1978. The authors are grateful to the staffs in the Higashiizu City Office and the Board of Education who have provided such oppotunity to them. The authors also wish to express their appreciation to Dr.M.Teshigawara, Ms.J.Moriyama, Mr.S.Horiuchi, Institute of Industrial Science, University of Tokyo, Mr.I.Kimura, Kimura Architects and Engineers, and the members of SPRC Committee(Chairman.Prof.H.Umemura, in Japan Building Disaster Prevention Association) who advised and helped in preparing this report.

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Adhérence acier-mortiers de résines

Haftung zwischen Stahl und Harzmörteln

Bond in Resin Mortars

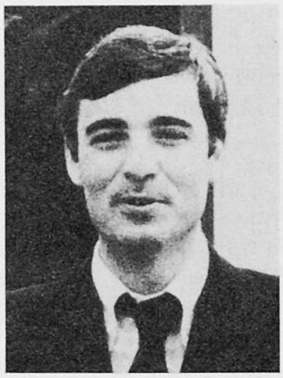
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RESUME

Des essais d'arrachement ont été effectués sur des barres d'acier enrobées dans des mortiers de résine époxy et polyester. Le dosage en résine doit rester faible, l'ancrage total est obtenu pour une longeur d'enrobage de quelques diamètres. La liaison est régie par des mécanismes analogues à ceux des bétons hydrauliques mais l'action des facteurs de résistance et leur importance relative ne sont pas les mêmes.

ZUSAMMENFASSUNG

Es wurden Ausziehversuche an Bewehrungsstählen, die in Epoxidharz- und Polyesterharzmörteln eingelegt waren, durchgeführt. Es wird gezeigt, dass die Dosierung der Harze schwach sein muss und dass eine totale Verankerungslänge von wenigen Stahldurchmessern genügt. Die Verbindung hängt von Mechanismen ab, die mit denjenigen im gewöhnlichen Beton vergleichbar sind. Jedoch sind die Haftfaktoren und deren relative Bedeutung nicht dieselben.

SUMMARY

Pull out tests have been performed on steel bars embedded in epoxy and polyester resin mortars. It appeared that the resin proportion must stay at a low level and that total anchorage is reached for an embedded length of some diameters only. It has been noticed that bond is ruled by the same mechanisms as those found for hydraulic concretes, but their relative influence has changed.

INTRODUCTION

La réparation et la transformation de bâtiments existants sont devenues des exigences nouvelles de la construction dans de nombreux pays. Dans ce domaine, les techniques actuelles font souvent appel à l'emploi de résines époxy ou polyester, lesquelles sont généralement associées à des granulats pour constituer des mortiers et bétons.

Ces produits sont d'un coût moins élevé que celui de la résine seule et conservent la plupart de ses qualités tout en réduisant ses principaux défauts comme le retrait ou le fluage [1][2][3].

Ainsi, il est techniquement et économiquement possible de :

- reconstituer la qualité perdue d'armatures plastifiées, par un placage d'acier sur le parement du béton [4][5] ;
- ragréer du béton localement ruiné en le remplaçant par du mortier de résine ;
- munir une construction ancienne d'aciers en attente permettant l'extension du bâtiment, la liaison acier-béton ancien étant assurée par du mortier de résine mis en place dans des forages préalablement exécutés.

On conçoit, cependant, que l'efficacité de telles techniques dépende pour une large part de la qualité de l'adhérence entre les différents matériaux en présence. Or, parmi les nombreuses études consacrées aux mortiers et bétons de résine, quelques unes seulement abordent le problème de l'adhérence et se cantonnent au domaine du béton plaqué.

C'est pourquoi les auteurs ont entrepris l'étude systématique des phénomènes qui régissent la liaison entre les ronds à béton et les mortiers de résine. Les principaux résultats obtenus sont présentés dans cette communication.

1. PRINCIPE DE L'EXPERIMENTATION

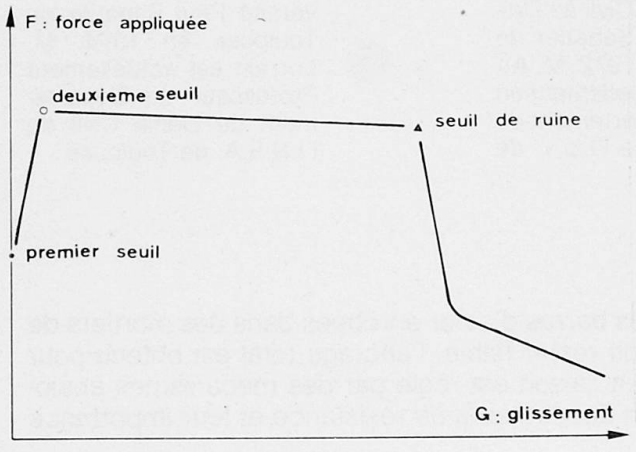


Fig. 1 Courbe type de glissement

Des capteurs de déplacements, montés sur l'éprouvette permettent de tracer, en continu, les courbes représentatives du glissement de l'extrémité non chargée de la barre, par rapport au béton.

Afin de faciliter l'exposé des résultats, une courbe de glissement type est présentée sur la figure 1. On distingue :

- un premier seuil de glissement ;
- un second seuil de glissement ;
- une phase de grands déplacements ;
- un seuil de ruine ;
- une phase de ruine.

2. RESULTATS

2.1 Influence du dosage pondéral en résine

Les essais ont été réalisés avec des ronds lisses en acier doux de 12 mm de diamètre, enrobés sur une longueur de 12 cm.

Dans l'ensemble, d'une résine à l'autre les performances sont comparables, les meilleures étant obtenues pour des valeurs moyennes du dosage.

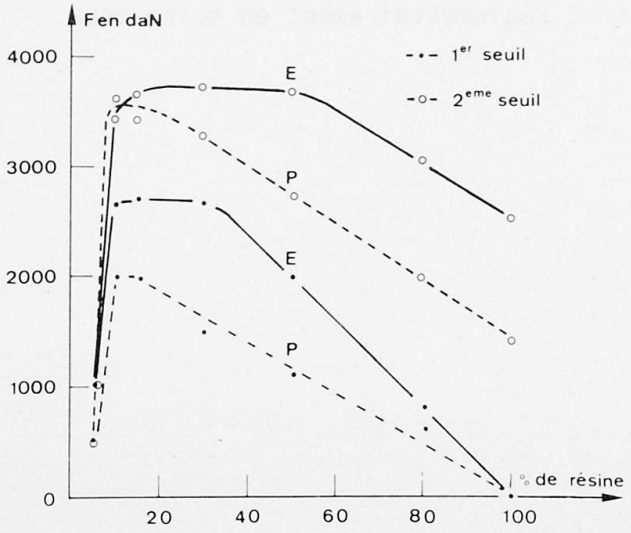


Fig. 2 Variation des résistances aux seuils de glissement en fonction du dosage en résine

La seule différence appréciable concerne le seuil de ruine qui montre une dégradation brutale de la liaison avec les mortiers de résine époxy pour une valeur faible du glissement.

L'influence du dosage en résine apparaît nettement sur les courbes de la figure 2 qui représentent les forces équilibrées aux premier et deuxième seuil en fonction du pourcentage de résine. Le maximum est obtenu dès 15 %, valeur commune aux deux séries dont les pouvoirs adhésifs sont pourtant différents.

Les plus mauvais résultats sont obtenus avec très peu ou beaucoup de liant. Logique dans le premier cas, ce résultat est plus surprenant dans le second même s'il s'explique par le retrait ou par la mauvaise qualité du moulage, qui augmentent avec le dosage en résine.

Il est donc établi qu'il est préférable d'opérer dans la pratique avec des mortiers raisonnablement dosés en résine. Nous avons retenu en conséquence un dosage pondéral de 15 % associé à une granulométrie continue de calcite de 0 à 5 mm.

2.2 Influence de la longueur d'enrobage

La longueur d'enrobage des armatures constitue, au même titre que le dosage, un paramètre essentiel de la technologie de mise en oeuvre des procédés liés à l'emploi des résines.

Nous avons effectué les essais avec des barres lisses, en acier doux puis mi-dur, dont les courbes de glissement sont plus faciles à interprêter que celles obtenues avec des barres crênelées à haute-adhérence. Nous avons fait varier les longueurs d'enrobage entre 2 et 25 cm.

a) Acier doux

Les ordres de grandeur des performances des deux types de matériaux sont voisins. Cela signifie que le passage d'une résine à l'autre n'entraîne pas de modification importante aux mécanismes en jeu.

On constate surtout que l'augmentation de la longueur d'enrobage entraîne une amélioration de la résistance à l'arrachement pour une longueur de 8 cm seulement. Au-delà, les efforts appliqués correspondent à la limite d'élasticité de l'acier dont les déformations transversales favorisent la dégradation de la liaison par décollement, les courbes de glissement ne se distinguent plus.

L'influence de la longueur d'enrobage se manifeste nettement sur les courbes de synthèse, de la figure 3. On peut ajouter que le meilleur pouvoir adhésif de la résine époxy multiplie par deux les performances obtenues au premier seuil avec la résine polyester. Au second seuil, les performances obtenues sont logiquement confondues car se sont les mécanismes de frottement qui sont alors prépondérants.

On retiendra de cette série d'essais que si la résistance à l'arrachement d'une barre en acier doux est fonction croissante de la longueur d'enrobage jusqu'à 5 fois le diamètre, elle est constante au-delà. Il est donc intéressant de prolonger ces expériences avec un acier de limite élastique plus élevée que celle de l'acier doux dont l'étude était cependant indispensable puisque les suspentes de

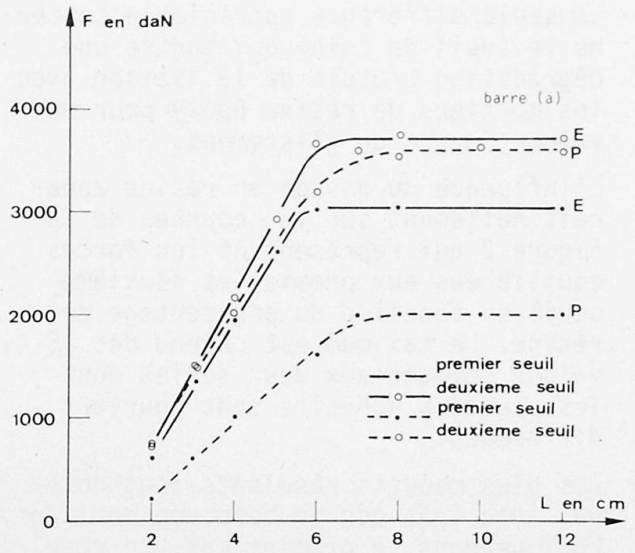


Fig. 3 Variations des résistances aux 1er et 2ème seuils de glissement en fonction de la longueur d'enrobage

manutention des pièces préfabriquées sont réglementairement en acier doux.

b) Acier lisse mi-dur

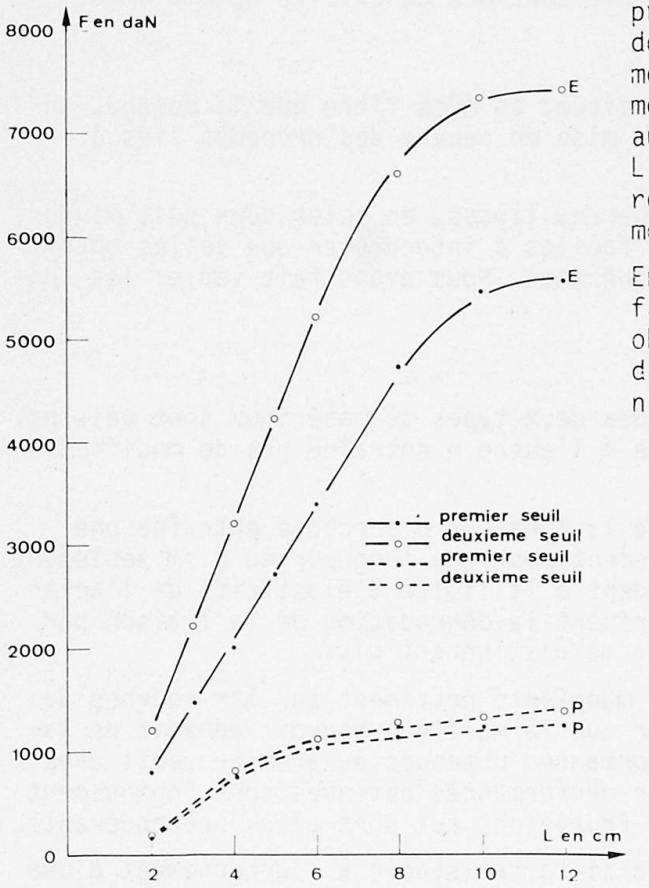


Fig. 4 Variations des résistances aux 1er et 2ème seuils de glissement en fonction de la longueur d'enrobage

Les essais conduits de façon analogue aux précédents donnent des ordres de grandeur des forces équilibrées considérablement modifiés : les performances diminuent de moitié avec la résine polyester, mais augmentent de 30 % avec la résine époxy. L'excellent pouvoir d'adhésivité de la résine époxy se manifeste particulièrement sur les courbes de la figure 4.

Enfin, c'est pour une longueur d'enrobage faible que les valeurs maximales sont obtenues. Ce résultat d'intérêt majeur dans la pratique est indépendant de la nature de la résine.

3. MISE EN EVIDENCE DES MECANISMES DE LA LIAISON ACIER-BETON

3.1 Méthode expérimentale

Les expériences ont été réalisées sur des aciers lisses et crénelés, graissés ou non, afin de mettre en évidence par comparaison des courbes de glissement, les effets respectifs du collage, des crénelures et du serrage.

La barre crénelée, dont la forme schématique est inspirée des travaux de REHM [6], est mise en oeuvre dans l'essai n° 1. La résistance à l'arrachement est alors due aux effets conjugués de serrage, collage et de l'action de butée de la crénelure.

L'essai n° 2 est réalisé avec une barre lisse ; la résistance à l'arrachement provient des effets de serrage et collage. Par soustraction, ces deux essais permettent donc d'isoler le rôle de la crénelure.

Les essais n° 3 et 4 diffèrent des essais n° 1 et 2 en ce que les barres sont enduites d'une pellicule de graisse interdisant tout collage.

3.2 Résultats

a) mortier de résine polyester

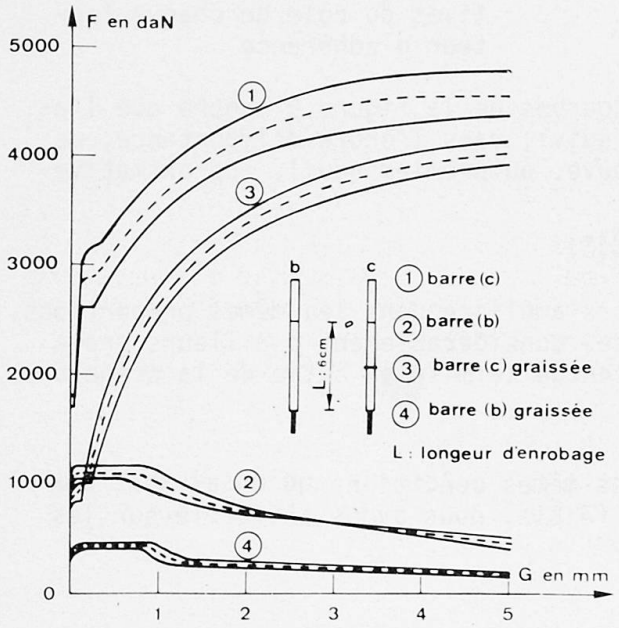


Fig. 5 Mortier de résine polyester. Courbes de glissement avec séparation des paramètres d'adhérence.

Les courbes de glissement présentées sur la figure 5, d'allure classique montrent clairement les effets des différents facteurs de l'adhérence :

* Avant glissement :

- serrage : frottement statique (S)
- collage : adhésivité (A)
- crénelure : effet de butée et cisaillement (C)

* Après glissement :

- serrage : frottement dynamique F(S)
- collage : frottement supplémentaire par imbrication des liaisons rompues F(A)
- crénelure : effet de compression cisaillement (CC) du mortier.

Les courbes de la figure 6, obtenues en combinant deux à deux les courbes moyennes de glissement, sont représentatives des rôles joués par chaque facteur d'adhérence :

① - ② ou ③ - ④ rôle de la crénelure

① - ③ ou ② - ④ rôle du collage

La figure 7 donne, pour l'ensemble des essais effectués, les courbes moyennes représentatives du rôle de chaque facteur d'adhérence.

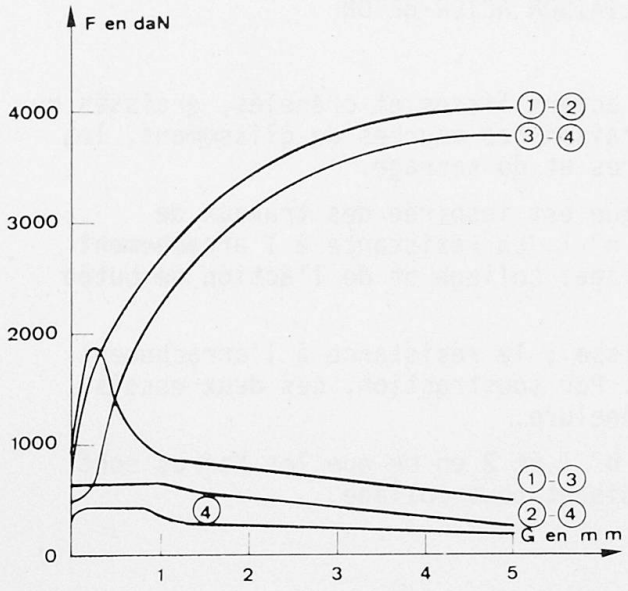


Fig. 6 Mortier de résine polyester. Rôle des facteurs d'adhérence

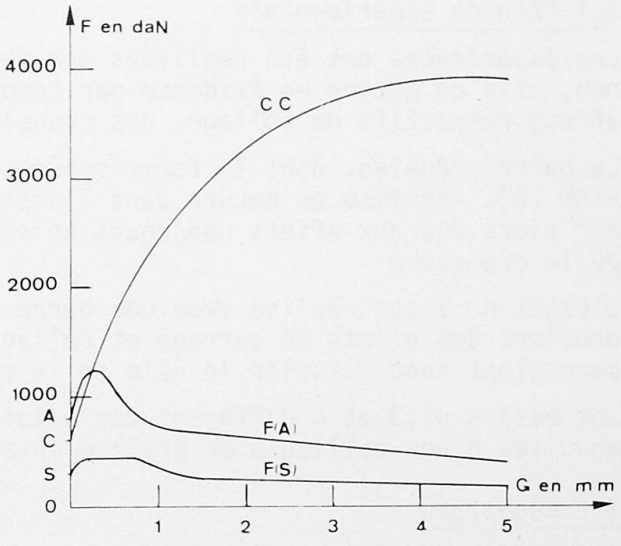


Fig. 7 Mortier de résine polyester. Courbes moyennes représentatives du rôle de chaque facteur d'adhérence

En première approximation, l'analyse des courbes de la figure 9 montre que l'adhésivité est le facteur le plus efficace suivi, dans l'ordre d'importance, de l'effet de crénelure et du serrage. On trouve, au premier seuil, approximativement :

$$(C) = 2(S) \; ; \; (A) = 3(S) = \frac{3}{2}(C)$$

Entre les deux seuils, chacun des facteurs s'améliore dans les mêmes proportions. Au-delà, seul l'effet de crénelure augmente, considérablement d'ailleurs grâce au déplacement relatif acier-béton qui accentue la mise en butée de la crénelure.

b) Mortier de résine époxy

Les courbes de glissement obtenues dans les mêmes conditions qu'en a) sont données sur la figure 8. La dispersion étant faible, nous avons travaillé sur les courbes moyennes tracées en pointillé.

On peut constater que :

- la courbe ④ (serrage (S) et F(S)) est la même que pour la résine polyester : le facteur (S) d'importance médiocre est donc indépendant de la nature de la résine ;
- les courbes ① et ② témoignent de l'excellente qualité de l'adhésivité de la résine époxy ;
- la résistance de la barre crénelée est inférieure, à partir du second seuil, à celle de la barre lisse.

Les valeurs de résistance attribuables à chaque facteur sont portées sur les graphes de la figure 9.

③ - ④ ou ① - ② pour le rôle de la crénelure ;

② - ④ ou ① - ③ pour le rôle du collage.

On note que les courbes représentatives du rôle d'un même facteur ne se superposent qu'après un glissement de l'ordre de 6 mm.

La crénelure d'une barre graissée améliore la résistance [③ - ④] alors que

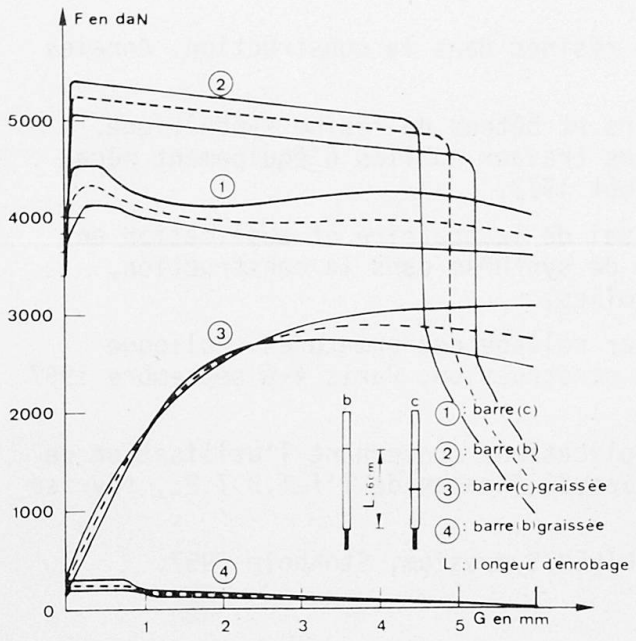


Fig. 8 Mortier de résine époxy. Courbes de glissement avec séparation des paramètres d'adhérence

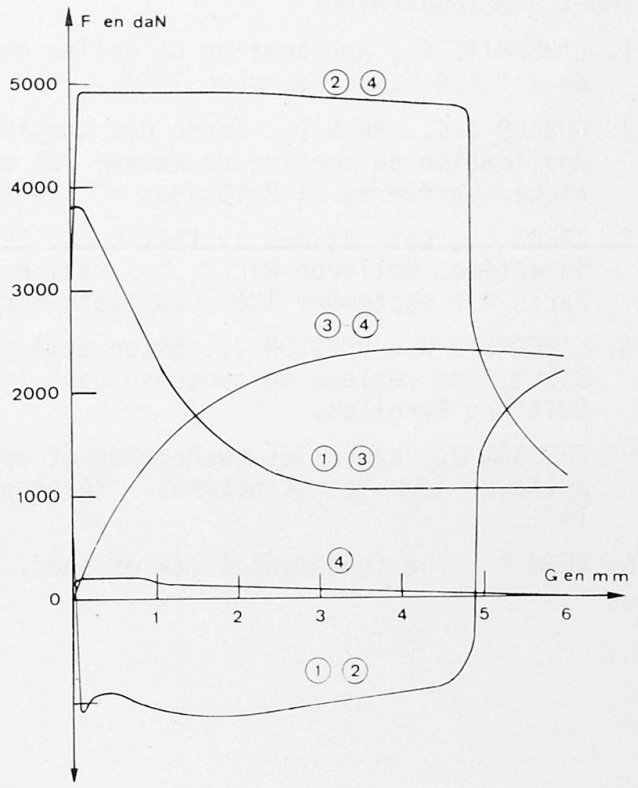


Fig. 9 Mortier de résine époxy. Rôles des facteurs d'adhérence

celle d'une barre adhérente la dégrade [① - ②]. De même le collage est plus performant avec des barres lisses [② - ④] qu'avec des barres crénelées [① - ③].

On remarque ici une grande différence de comportement par rapport au mortier de polyester, due pour une large part à l'excellent pouvoir adhésif de l'époxy.

Lorsque le collage se rompt, c'est nécessairement pour une valeur très élevée de la force d'arrachement. La brutale mise en charge du béton par la crénelure à une pression supérieure à la résistance locale de ce dernier provoque alors une ruine instantanée. En revanche, en l'absence d'adhérence, la crénelure sollicite et déforme progressivement le béton : elle améliore ainsi la résistance à l'arrachement (cf. figure 8 courbe n° 3). En définitive, le collage est de loin le facteur le plus efficace avec la résine époxy. La crénelure, qui a priori peut paraître néfaste, donne en fait une certaine ductilité à la dégradation de la liaison (disparition du seuil de ruine dans la plage de glissement étudiée).

CONCLUSIONS

Les expériences précédentes étaient destinées à mettre en évidence les rôles respectifs des facteurs de résistance de la liaison acier-bétons de résine. Elles ont été effectuées sur des matériaux obtenus en associant résine époxy ou résine polyester à une charge minérale.

Les résultats obtenus ont montré que, pour les matériaux étudiés :

- le serrage demeure un facteur relativement faible et apparamment indépendant de la nature de la résine ;
- le collage dû à l'adhésivité, modeste avec le polyester est considérable avec l'époxy ;
- la crénelure, dont l'efficacité dépend essentiellement de la valeur du glissement, est très importante avec le polyester et pratiquement pas avec l'époxy.

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Repair of Damaged Floor Slabs

Réparation de planchers endommagés

Instandsetzung von Gebäudedecken

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SUMMARY

The crack pattern and deflection of 48 damaged slabs in a 5-story reinforced concrete building with wide cracks and large deflections were measured. Two repair methods proposed were injection of epoxy resin into the cracks, and the casting of a concrete or mortar layer over the damaged slabs, depending on the degree of damage. Loading tests on one-way slab specimens similar to the actual slabs, and vibration tests on the actual slabs were carried out before and after repairing. The results of these tests show that the proposed methods of repair were very effective for the damaged slabs.

RESUME

Les fissures et les flèches de 48 planchers endommagés ont été mesurés. L'auteur propose deux méthodes de réparation, en fonction du dommage; l'une consiste à injecter de la résine d'époxy dans les fissures et l'autre, de superposer une couche de ciment ou de béton au-dessus du plancher endommagé. Les essais de charge sont réalisés sur des éprouvettes semblables. Des essais de vibration sur les planchers endommagés avant et après réparation sont effectués. Ceux-ci ont démontré l'efficacité des réparations proposées.

ZUSAMMENFASSUNG

Die Risse und die Durchbiegungen von 48 schadhaften Decken eines 5-stöckigen Stahlbetongebäudes wurden gemessen. Für die Reparatur wurde das Vergiessen der Risse mit Epoxidharz und das Aufbringen einer Beton- oder Mörtelschicht (je nach Beschädigung) vorgeschlagen. Statische Belastungsversuche bis zum Bruch an gleichartigen Platten sowie dynamische Versuche an den reparierten Decken wurden durchgeführt. Die Ergebnisse zeigten die Wirksamkeit der Reparaturmethoden.

1. INTRODUCTION

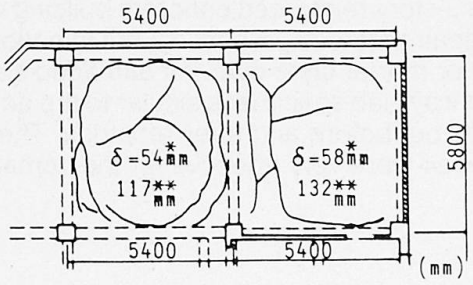
Recently, it has been a social problem that there were many floor slabs with wide cracks and large deflection under long-term load.[1),2)] The slabs of the building structures in Japan were generally composed of reinforced concrete two-way floor slabs monolithic with perimeter beams. In such slabs constructed before 4-5 years, many wide cracks at edges of the upper surface and at the center of the lower surface, and large deflection amounted to 2-6 cm have occured. Fig.1 and Fig.2 show the slabs with wide cracks and large deflection. These happened the residents of the buildings to be anxious about load capacity and to be inconvenient for living. The reasons of occuring such trouble of the slabs seemed to be bad quality of concrete, lack of curing times, lack of the slab thickness, lack of the effective depth of the slab by sinking of the upper reinforcements as construction, and overload during and after construction. Because of lack of publication and investigation about these troubles, the appropriate guides and methods to prevent such damages have been not established.

The objects of this study are to propose the methods to judge the damageness of the slabs and the methods to repair such damaged slabs and to discuss the effects by tests. In this study, the following investigation and tests were carried out.

1) measuring crack patterns and deflection of the damaged slabs in a apartment houses of reinforced concrete wall structure,
2) measuring the stiffness and strength of such slabs by loading tests and measuring the natural frequency by vibration tests to drop the steel ball,
3) proposing the repair methods to inject epoxy resin into cracks and to cast concrete on the damaged slabs,
4) tests on the one-way slab speciments damaged and repaired, whose span and depth were almost equal to the measured slabs,
5) vibration tests on the actual repaired slabs, in order to confirm the effects of repair.

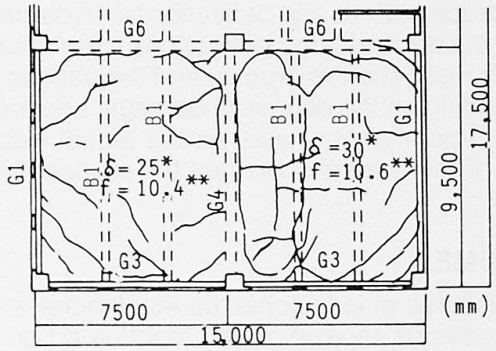
2. CRACK PATTERNS AND DEFLECTION OF THE DAMAGED SLABS IN ACTUAL BUILDING

Crack patterns and deflection of 48 damaged slabs of 16 flats in a actual 5-story apartment houses of reinforced concrete wall structure were measured.[5)] The plan of a flat is shown in Fig.3. The slabs consisted of three types, namely, SI (center to center span is 3.78 x 7.23 m), SII (3.78 x 6.39 m), SIII (3.78 x 3.78 m).



1) construction year :1961
2) girder section :350 x 700 mm
3) design depth :130 mm
4) natural frequency :
(12.5-13.0 Hz, before repair)
(18.2-20.5 Hz, after repair)
* : measured deflection at the center
**: measured depth

Fig.1 Square slabs with wide span[6)]



1) construction year : 1972
2) girder section(G1,G4);350 x 750 mm
(G3,G6);400 x 750 mm
3) beam section B1 ;300 x 650 mm
4) design depth ;120 mm
* : measured deflection at the center
**: measured frequency

Fig.2 Slabs with beams in a office Building[6)]

Figure 4 shows crack patterns of the most damaged slab. Cracks by constrain and shrinkage and by bending moment were mixed in the slab , but almost of wide cracks seemed to be by bending moment, judging from their occured place. There were many cracks whose width was more than 1.0 mm, and the width of a few cracks along the edges of short span reached 3.0 mm.

Table 1 shows the deflection at the center of SI and SII slabs and the total length of cracks at the upper and lower surfaces of the slabs. As the total length of the cracks increased, the deflection of the damaged slabs increased, and the deflection of two slabs were more than 4.0 cm.

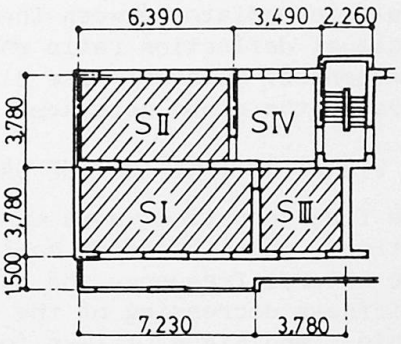
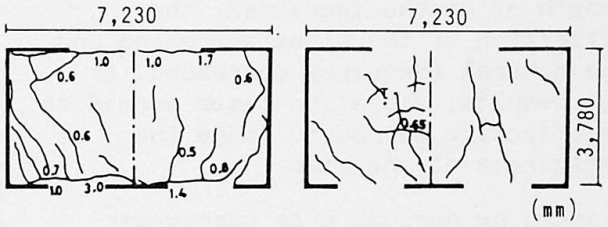


Fig.3 Plan of a flat

3.LOADING TESTS OF THE DAMAGED SLABS

In order to discuss the elastic behavior of the damaged slabs, loading tests were carried out on 4 pieces of SI slabs (3.78 x 7.23 m) in the houses named as 5D (story number is 5, flat number is D), 4D, 3A and 2C, whose damageness was differ from the others. These slabs were uniformly loaded by sacks of micro gravel (weigth is 30 kg) until the design live load (W_{LL}), and were unloaded after 4 hours. Residual deflection ratio, load-deflection curves and distribution of deflection were measured.



(a) Upper surface (b) Lower surface

Fig.4 Crack Patterns of the most damaged slab

Table.1 Measured results of the damaged slabs

story	flat number / type of slab		A SI	A SII	B SI	B SII	C SI	C SII	D SI	D SII
5	(1)	mm	98.0	95.0	106.0	—	116.0	—	108.0	109.0
	(2)	mm	39.0	21.0	26.3	16.0	19.5	4.5	48.3	17.5
	(3)	Hz	12.0	—	14.0	—	13.0	—	13.0	—
	(4)	m	137.0	112.0	106.0	79.0	71.0	79.0	105.0	102.0
	(5)	m	25.0	33.0	32.0	8.0	22.0	29.0	22.0	23.0
4	(1)	mm	107.0	—	106.0	104.0	120.0	—	—	—
	(2)	mm	15.0	11.0	27.5	15.5	35.0	16.0	43.3	27.0
	(3)	Hz	13.5	—	15.0	—	14.5	—	11.0	—
	(4)	m	35.0	38.0	36.0	27.0	29.0	23.0	26.0	20.0
	(5)	m	3.0	5.0	7.0	8.0	17.0	8.0	13.0	9.0
3	(1)	mm	100.0	—	102.0	100.0	115.0	—	—	—
	(2)	mm	11.5	8.0	27.5	28.5	8.5	2.5	14.8	6.5
	(3)	Hz	16.0	—	14.5	—	16.5	—	14.0	—
	(4)	m	27.0	20.0	32.0	21.0	30.0	25.0	22.0	8.0
	(5)	m	7.0	6.0	7.0	8.0	7.0	4.0	8.0	2.0
2	(1)	mm	107.0	—	100.0	—	117.0	—	113.0	100.0
	(2)	mm	21.5	10.0	25.8	17.0	24.3	12.5	31.0	20.5
	(3)	Hz	17.0	—	14.5	—	15.5	—	13.5	—
	(4)	m	37.0	30.0	62.0	46.0	17.0	71.0	81.0	77.0
1	(5)	m	7.0	0.0	10.0	6.0	6.0	3.0	7.0	3.0

(1) Slab depth
(2) Deflection at the center
(3) Natural frequency
(4) Total length of cracks on the upper surface
(5) Total length of cracks on the lower surface

Figure 5 shows the load-deflection curves of the slabs. The values of calculated stiffness with fixed edges and with supported edges were shown by the dotted lines, in this figure.The stiffness of the slab in 2C,which were damaged little, was intermediate between the both calculated values. But the stiffness and residual deflection ratio showed tendency to be larger, corresponding to their damageness. However, these slabs were confirmed to have enough load capacity, because the slabs were kept in elastic under design live load, as shown in Fig.5.

4. VIBRATION TESTS OF THE DAMAGED SLABS

The free vebration waves of the slabs were measured by the pick-up for vertical motions, when the steel ball falled on the slabs from a heigth of 28 cm. Reading the natural frequency and maximum amplitude of the slabs from those waves, the stiffness decreasing of the slab was discussed.[5] The tests results were shown in Table 1 and Fig.6. It was found that the natural frequency decreased as the deflection of the damaged slabs increased. It showed a similar tendency to the results of other's studies.[2),6)] Judging from Table 1, as the total length of cracks increased, the deflection of the slabs increased and the natural frequency decreased. As the results, vibration tests seemed to be effective method to judge the damageness of the slab.

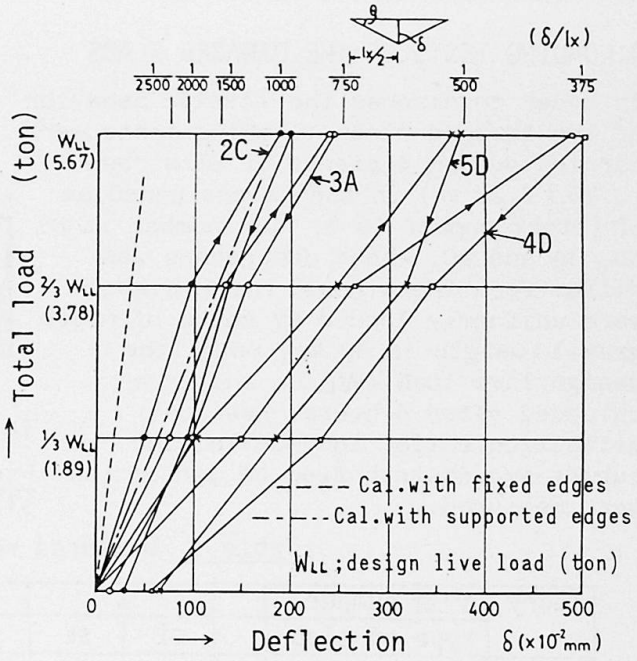


Fig.5 Loading tests results

5.TESTS ON ONE-WAY SLAB SPECIMENTS FOR STUDY ON THE REPAIR METHODS

16 one-way slab specimens of reinforced concrete, whose depth and span were similar to SI slab and whose width was 30 cm , were tested under short-term and long-term loading before and after repair by various methods, and the effects of the repair methods were discussed.[3),4)]

The following three repair methods were combinated to use, and the repair methods of each specimens were shown in Table 2 and 3.

1) injecting epoxy resin into cracks whose width were more than 0.3 mm,

2) casting normal concrete or light-weight concrete or mortal on the damaged slab,

3) reinforcing by deformed bars (D13) or wire mesh (3.2 mm diameter @ 20.0 cm) as casting concrete or mortal.

5.1 Tests under Short-Term Loading

Each specimen was loaded by many oil jacks as shown Fig.7 until the residual deflection reached the values shown in Table 2, and was unloaded after cracks occured. And the specimens were reloaded

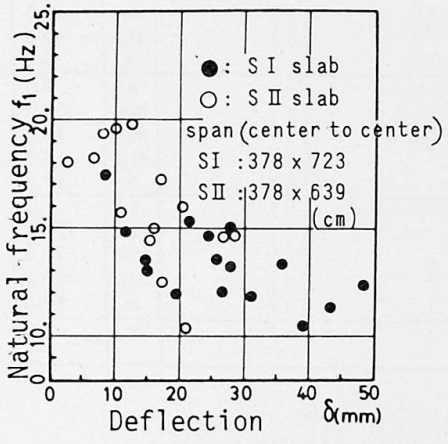


Fig.6 Vibration tests results

after repaired as shown in Fig.8. The load-deflection curves of the specimens were shown in Fig.9, and the natural frequency by vibration tests and the stiffness and deflection were shown in Table 2. The stiffness of the slabs repaired by injecting epoxy resin into cracks increased by 0.78-0.96 as that of the damaged slab.

Moreover, the stiffness, natural frequency and load capacity of the slabs repaired by casting concrete increased, as the depth of concrete increased. Especially, the stiffness, natural frequency and load capacity of the slabs repaired by casting concrete after injecting epoxy resin were by far more than those of the undamaged slabs. These repair methods seemed to be very effective for the damaged slabs.

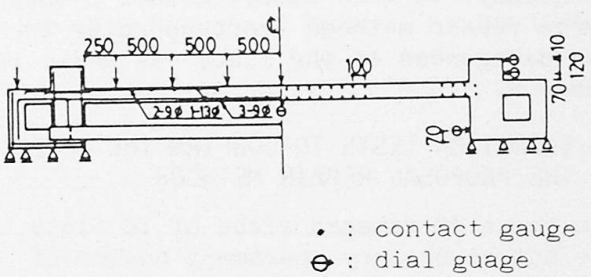


Fig.7 Specimen of short-term loading tests

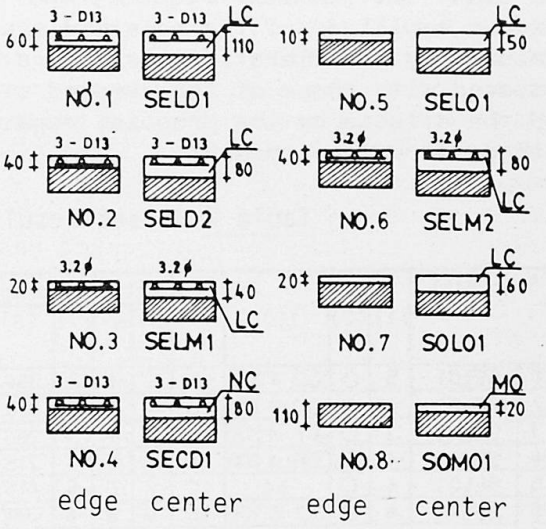


Fig.8 Sections of repaired slabs

5.2 Tests under Long-Term Loading

8 one-way slab specimens of similar section and arrangement as shown in Fig.10 were uniformly loaded steel plates until the residual deflection reached the values shown in Table 3, and they were unloaded after cracks occured. After repair shown in Fig.8, the specimens were loaded under long-term live load for design (w_{LL} = 180 kg/cm^2).

The long-term deflection increase of each specimens was shown in Fig.11. The deflection increase of all slabs were almost constant after 450 days. The deflection of No.13 slab repaired by injecting epoxy resin into cracks was 60 % of No.12 slab damaged and was almost equal to that of No.11 slab undamaged. As the depth of casting concrete increased, the degree of the deflection increase became smaller than that of the undamaged slab.

The repair method by casting concrete after injecting epoxy resin into cracks did not only increase the stiffness and load capacity of the damaged slabs under short-term loading, but also was very effective for decreasing the deflection under long-term loading.

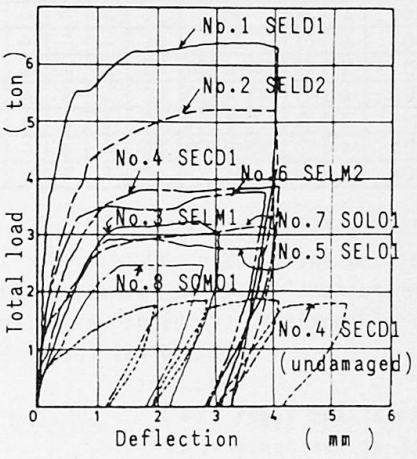


Fig.9 Load-deflection curves of the short-term loading tests

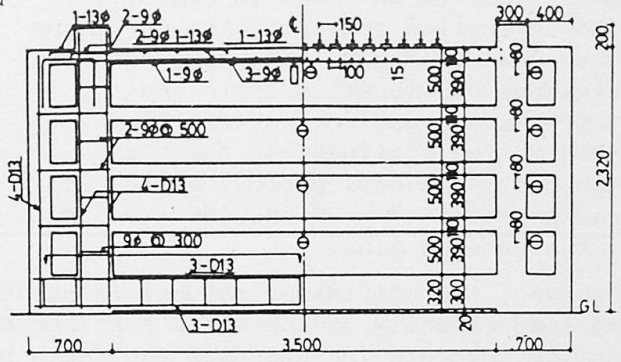
6. PROPOSAL REPAIR METHOD FOR THE DAMAGED SALBS

As the results of measuring the crack patterns and the deflection of damaged slabs, loading tests and vibration tests of the actual slabs, and loading tests of the one-way slab specimens, the damageness could be well judged by the deflection at the center of the slabs, and natural

frequency. In this study, author proposed three repair methods ,corresponding to the damageness of the slab, as shown in Table 4.

7. VIBRATION TESTS TO CONFIRM THE EFFECTS OF THE PROPOSAL REPAIR METHODS

The above 48 damaged slabs of 16 flats in the actual 5-story apartment houses of reinforced concrete were repaired by the proposal methods. The repaired slabs consisted of three types, namely, SI,SII and SIII. The natural frequency and maximum amplitude of the repaired slabs measured by the vibration tests were compared with those of the damaged slabs, and the effects by the proposal repair methods were confirmed.[5)]



bar arrangement loading and measured points

Fig.10 Specimens of long-term loading tests

Table 2 Tests results under short-term loading

Specimens		(1)	(2)	(3)	(4)	Material property of concrete (5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13) WH	(14) f	(15) WRM	(16) fR	WRM/WH	fR/f
														ton	Hz	ton	Hz		
1	SELD1	5	○	D13 a 200	LC	6	11	3	354	1.4	0.32	1.65	7	3.09	26.4	7.17	44.0	2.32	1.67
2	SELD2	4	○	D13 a 200	LC	4	8	3	358	1.4	0.25	1.65		2.87	27.4	5.90	29.1	2.06	1.06
3	SELM1	2	○	3.2 @ 200	LC	2	4	3	286	1.4	0.21	1.60		2.62	25.9	3.64	28.2	1.39	1.09
4	SECD1	4	○	D13 a 200	NC	4	8	3	215	1.7	0.28	2.23	5	2.36	24.6	4.63	28.6	1.96	1.16
5	SELO1	4	○	×	LC	1	5	6	415	1.39	0.39	1.65		2.66	21.2	3.51	23.8	1.32	1.12
6	SELM2	4	○	3.2 @ 200	LC	4	8	6	388	1.34	0.36	1.65		2.46	20.4	4.45	26.3	1.81	1.29
7	SOLO1	4	×	×	LC	2	6	6	282	1.20	0.35	1.65		2.56	25.0	3.89	24.8	1.52	0.99
8	SOMO1	2	×	×	MO	0	2	6	425	2.09	0.32	—		2.51	23.8	3.03	21.4	1.21	0.90

Table 3 Tests results under long-term loading

Specimens		(1)	(2)	(3)	Material propaerty of concrete for repair (4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)		(13) WM (ton)	(14) (cm)	Natural frequency (17) f1	(18) f2	(19) f3	(20) f4
11	LO001	0	×	×	×	—	—	—	——	——	——	——		——	——	18.1	—	—	—
12	LO002	4	×	×	×	—	—	—	——	——	——	——		2.21	3.65	20.4	19.5	—	—
13	LE001	4	○	×	×	—	—	—	——	——	——	——		2.43	4.38	23.0	17.0	18.2	—
14	LELD1	5	○	D13-a 200	LC	6	11	4	363	1.36	0.34	1.65		2.93	6.63	25.5	17.4	20.1	39.8
15	LELD2	4	○	D13-a 200	LC	4	8	4	335	1.34	0.36	1.65		2.29	3.58	23.4	16.9	21.8	33.3
16	LELM1	2	○	3.2 @ 200	LC	2	4	4	——	——	——	——		1.77	1.97	23.6	15.4	21.0	27.3
17	LECD1	4	○	D13-a 200	NC	4	8	4	187	2.15	0.32	2.23		2.04	4.78	23.7	18.6	/	29.6
18	LELO1	4	○	×	LC	1	5	4	369	1.34	0.34	1.65		2.97	3.08	24.6	17.2	19.3	25.4

(1) Initial deflection (cm)
(2) Injecting epoxy resin ○;Yes, X;No
(3) Reinforcement (x: No reinforcement)
(4) Type of concrete LC : Light weight concrete
NC : Normal concrete
MO : Mortal
(5) Depth at the edges (cm)
(6) Depth at the center (cm)
(7) Curing term (week)
(8) Compressive strength of concrete (kg/cm^2)
(9) Initial elastic modulus of concrete ($\times 10^6 kg/cm^2$)
(10) Strain at peak stress (%)
(11) Specific gravity
(12) Numbers of loading point
(13) Maximum load capacity of damaged slab(ton)
(14) natural frequency of damaged slab (Hz)
(15) Maximum load capacity of repaired slab(ton)
(16) Natural frequency of repaired slab (Hz)
(17) Natural frequency of undamaged slab (Hz)
(18) Natural frequency of damaged slab (Hz)
(19) Natural frequency of slab repaired by injecting epoxy (Hz)
(20) Natural frequency of slab repaired by injecting epoxy and casting concrete (Hz)

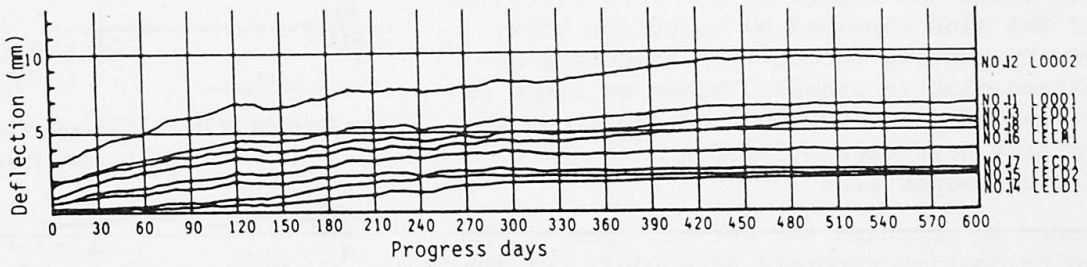


Fig.11 Long-term deflection increasing

Table 4 The Proposal repair methods

Damageness		methods
Deflection at the center δ (mm)	Natural frequency f (Hz)	
$\delta \leqq 15$	$f \geqq 15$	injecting epoxy resin into cracks whose width were more than 0.3 mm
$15 < \delta < 40$	$12 < f < 15$	casting mortal after injecting epoxy resin into cracks
$\delta \geqq 40$	$f \geqq 12$	casting concrete with reinforcing by wire mesh after injecting epoxy resin

Figure 12 shows the change of the natural frequency of the slabs repaired by injecting epoxy resin into cracks. The natural frequency of repaired slabs increased about 20-30 % than that of the damaged slabs. The measured results of the slabs repaired by casting mortal or concrete were shown in the dotted lines of Fig.12, and the natural frequency of such repaired slabs increased remarkably.

Figure 13 shows the relations between the natural frequency and maximum amplitude of the repaired slabs. The frequency increased and the amplitude decreased by injecting epoxy resin and still more by casting mortal or concrete.

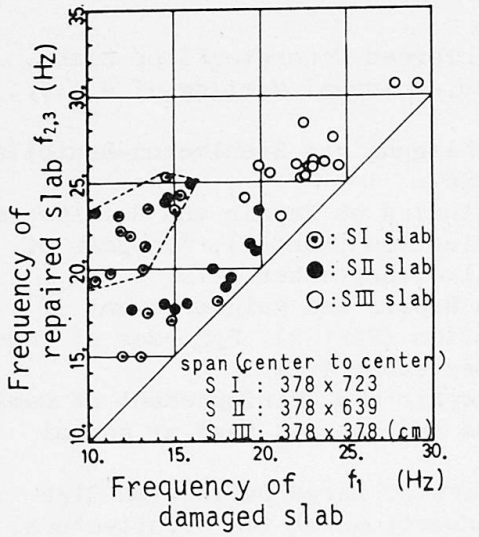


Fig.12 Increase of the Natural frequency of the slabs

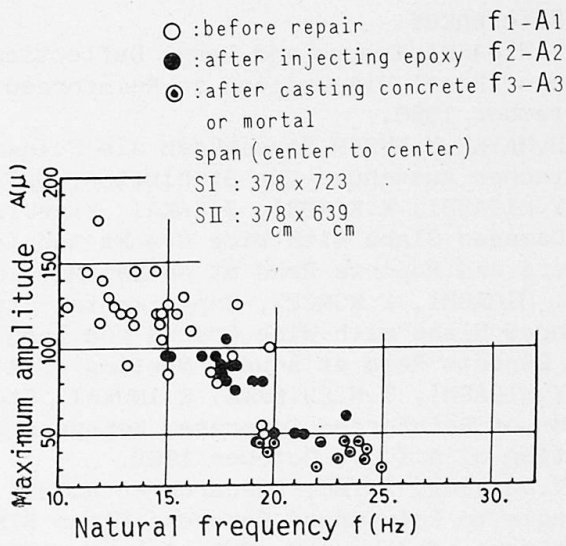


Fig.13 Relations between natural frequency and maximum amplitude

Figure 14 shows the change of the free vibration waves of the slab repaired by injecting epoxy resin and by casting mortal. The frequency of the repaired slab increased 2 times as large as that of the damaged slab, and the amplitude of repaired slab decreased 0.2 times as large as that of the damaged slab.

It happened to increase the seismic load of the building by casting concrete or mortal, but the increasing degree was less than 2 %, and it mattered little to consider the increase of the seismic load for design on this building

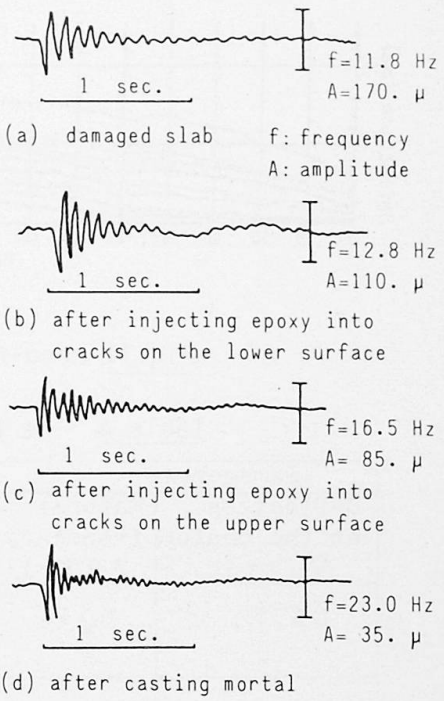


Fig.14 Waves changing of the repaired slab

8.CONCLUSION

1) As the measured results of the damaged floor slabs with wide cracks and large deflection, the stiffness by loading tests on the damaged slabs decreased and the natural frequency by vibration tests decreased, as the damageness of the slabs increased.
The damageness of the slabs could be judged from the deflection and natural frequency.
2) Author proposed the repair method by injecting epoxy resin into cracks and the strengthing methods by casting concrete or mortal on the damaged slab with or without reinforcing to increase the stiffness and load capacity, corresponding to the damageness of the slab.
3) The stiffness and load capacity of the one-way slab specimens repaired by injecting epoxy resin into cracks and by casting concrete or mortal on the surface were far more than those of the damgaed slabs and undamaged slabs.
The proposal repair methods seemed to be very effective for the damaged slabs.
4) As the results of vibration tests on the actual slabs, the natural frequency of the slabs repaired by injecting epoxy resin into crcaks and casting concrete or mortal were more than those of the damaged slabs.
It was confirmed that the proposal repair methods were very effective for the damaged slabs.

9. References

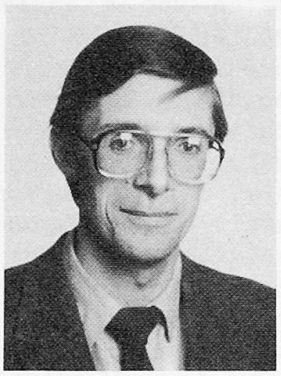
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Renforcement des maçonneries par injection de résine epoxy

Verstärkung von Mauerwerk durch Injektion von Epoxidharz

Reinforcement of Masonry by Injection of Epoxy Resin

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RESUME

Les résultats d'un programme de recherche sur le renforcement des maçonneries par injection de résine epoxy sont présentés. L'article expose la technique d'injection ainsi que les résultats obtenus pour une maçonnerie du 17ème siècle. Les possibilités de contrôle, basé sur l'analyse infrarouge, ont été étudiées.

ZUSAMMENFASSUNG

Die Resultate eines Untersuchungsprogramms über die strukturelle Renovation historischen Mauerwerks durch Injektion von Epoxidharz werden vorgestellt. Das Injektionsverfahren wird beschrieben, ebenso wie die mechanischen Charakteristiken eines injizierten Mauerwerks aus dem 17. Jahrhundert. Die Möglichkeiten einer Kontrollmethode, beruhend auf einer Infrarotanalyse injizierter Ziegel, werden angegeben.

SUMMARY

The results of a research program on the structural reinforcement of historic masonry by injection of epoxy resin are presented. The injection procedure is described, as well as the mechanical characteristics obtained for a 17th century masonry. The possibilities of a control method, based on infra-red spectroscopic analysis are indicated.

1. INTRODUCTION

Le développement d'une gamme très étendue de résines artificielles a donné lieu à des évaluations et des changements importants non seulement dans la technique de construction, mais aussi dans les techniques de conservation, de renovation et de restauration des structures de génie civil et des bâtiments historiques. Les résines artificielles sont appliquées en temps que barrière à la succion d'eau dans les murs, pour le durcissement superficiel des matériaux pierreux, le renforcement des structures en béton par collage de tôles en acier à la surface du béton, l'injection des fissures, etc.

Une technique nouvelle pour la restauration et le renforcement de maçonneries anciennes dégradées, est basée sur l'injection d'une résine epoxy dans la masse de la maçonnerie. En Belgique cette technique a déjà été appliquée avec un succès modéré dans quelques cas pratiques de restauration. Afin d'optimaliser le procédé d'injection et la technique d'exécution et d'améliorer les caractéristiques mécaniques des maçonneries injectées, un programme d'essais est en cours d'exécution au Laboratorium Reyntjens du Département des Constructions Civiles de la Katholieke Universiteit Leuven à Leuven, ceci en collaboration avec le Centre Scientifique et Technique de la Construction (C.S.T.C.) et la société Emergo Chemical Coating N.V. (E.C.C.). Ces recherches sont subventionnées par l'Institut Belge pour la promotion de la Recherche Scientifique dans l'Industrie et l'Agriculture (I.R.S.I.A.). A la suite des premiers résultats très encourageants, l'instauration d'un projet pilote à Herentals a été approuvée par le Ministère "Ministerie van Nederlandse Cultuur" de Belgique. Il s'agit d'un ancien hôpital avec chapelle du début du 17ème siècle (Fig. 1). Les murs porteurs de ce bâtiment nécessitent un renforcement structurel en vue de l'aménagement des anciens locaux en bureaux.

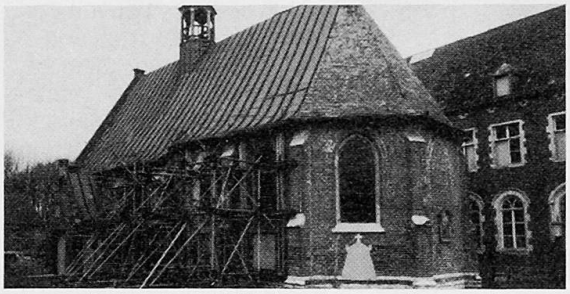


Fig. 1 Ancien hôpital et chapelle en annexe à Herentals (Belgique)

2. DESCRIPTION DU PROCEDE D'INJECTION

La résine d'injection est une résine epoxy à deux composants. L'injection se fait à l'aide d'une "pompe à deux composants". La résine et le durcisseur sont stockés dans deux réservoirs séparés. Ces composants sont aspirés, en proportion appropriée, et introduits dans la tête d'injection dans laquelle est réalisé un mélange "intime". Le début de la réaction et l'accroissement de la viscosité qui l'accompagne, sont retardés jusqu'au moment de l'injection dans la maçonnerie. La pression d'injection peut être contrôlée par l'operateur à l'aide d'un manomètre.

L'injection se fait d'un coté du mur. L'opération d'injection est entamée à la partie inférieure du mur via des trous forés, régulièrement espacés dans le sens horizontal et vertical. Les distances varient de 100 à 150 mm, en fonction de la perméabilité de la maçonnerie et de la viscosité de la résine. Des tubes en plastiques pour l'injection de la résine sont inserrés dans les trous et fixés par un ciment rapide (Fig. 2). Il est nécessaire d'imperméabiliser la face du mur, opposée à la face d'injection, afin d'éviter toute perte de résine. A des surfaces extérieures, l'application d'une barrière à base de ciment rapide n'est pas admis. Deux types de traitement ont été examinés pour assurer leur étanchéité : une impregnation par ester d'éthyl d'acide silicique et une impregnation aux silicates de sodium mélangés à une pâte de papier. Le premier traitement donne de meilleurs résultats, bien que quelques pertes locales de résine puissent se produire. Il est possible de remédier à ce problème par l'application de ciment rapide.

La porosité et la capacité d'absorption de la maçonnerie sont les caractéristiques déterminantes pour une injection. Les particules isolées de la maçonnerie se sont

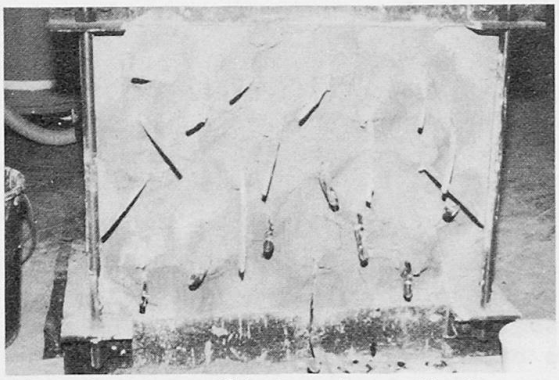


Fig. 2 Tubes d'injection, fixés par un ciment rapide.

agglomérées, grâce à une injection rigoureuse et donnent naissance à une masse pratiquement homogène qui présente des caractéristiques mécaniques accrues.

Après injection, la résine epoxy demeure encore fluide pendant un certain temps, dite "pot-life". Durant cette période la résine peut pénétrer dans les cavités et fissures par gravitation. Cet effet est d'autant plus important que la résine à un temps de durcissement plus long, ce qui est favorable pour une bonne impregnation et remplissage des pierres. Le degré de pénétration de la résine influence favorablement les caractéristiques mécaniques de la maçonnerie injectée. Ceci a été constaté par un essai sur chantier, où une zone de 1 m x 0,6 m au milieu d'un mur avait été injectée. Aux bords de cette zone la résine s'est répendue dans les zones non injectées. Les essais de compression sur des cylindres pris au centre de la zone injectée montraient des résultats bien plus élevés que pour les zones extérieures. On a même noté une augmentation de 50 % [2]. Une méthode pour la délimitation de la zone à injecter a été mise au point. La solution consiste à injecter de la résine isocyanate en bandes étroites, bornant la zone à injecter suivant des axes verticaux, et éventuellement suivant un axe inférieur si l'injection est entamé à un niveau intermédiaire. Dans certains cas cette technique permet de réaliser une économie importante. Vu le degré de renforcement réalisable avec une injection d'epoxy (des résistances à la compression élevées peuvent être atteintes), il ne sera pas toujours nécessaire d'injecter tout le massif de la maçonnerie. Une limitation à des zones réduites donne des résultats satisfaisants (Fig. 3). La largeur et la disposition de ces zones sont à determiner au moyen d'un calcul statique de la structure.

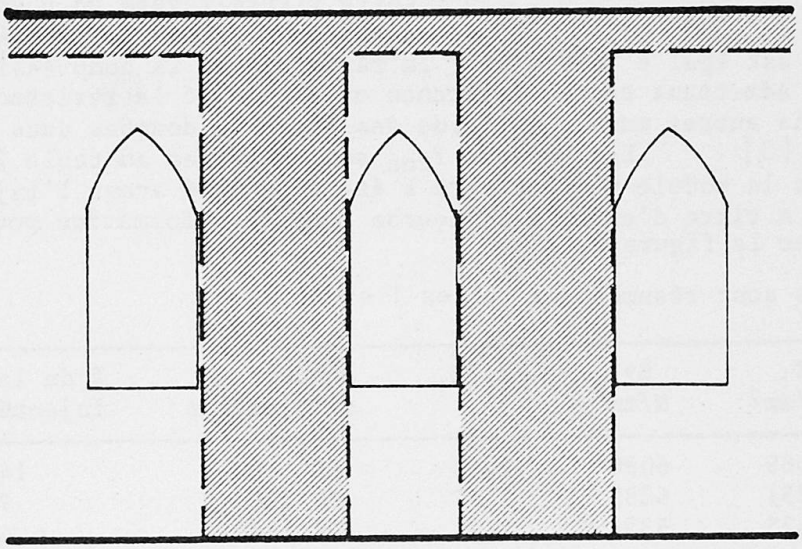


Fig. 3 Création d'une ossature par injection

3. EVALUATION ET CONTROLE

3.1. Généralités

L'application de résines artificielles pour des renforcements de structures exige un contrôle rigoureux de la qualité des produits avant et pendant leur mise en oeuvre. Seule une qualité constante et supérieure peut garantir un renforcement optimal. Ce contrôle est basé sur des essais d'identification par spectre infra-

rouge, l'équivalent d'epoxy, la densité, l'indice d'amine du durcisseur, la perte au feu, la viscosité, extraction de matières sèches [1].

Des essais préliminairs mécaniques et d'identification sont effectués simultanément. Si ces essais d'identification pendant la mise en oeuvre correspondent à ceux notés lors de l'identification, on admets que les caractéristiques mécaniques suivent les mêmes lois.

La détermination des caractéristiques mécaniques de la maçonnerie après injection est un problème délicat. Jusqu'à ce jour la méthode la plus efficace consiste à forer des carottes cylindriques aux endroits accessibles, c'est-à-dire aux embrasures. Afin d'obtenir des résultats représentatifs il est non seulement nécessaire de prélever un nombre considérable de carottes, mais un choix répandant aux lois de la statistique s'impose. Comme cette méthode s'avere parfois impraticable ou inadmissible, la recherche s'est dirigée vers un examen comparatif de tronçons de maçonnerie entiers d'une part, et vers l'analyse spectroscopique infrarouge de carottes de diamètre réduit et prélevées de ces mêmes tronçons d'autre part.

3.2. Essais de tronçons (Préparés sous forme de murets)

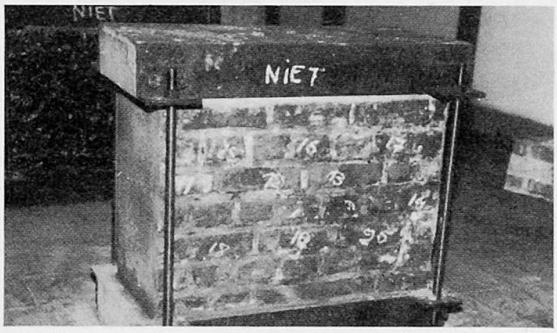


Fig. 4 Muret 2 avant l'injection.

Les résultats présentés sont obtenus sur la maçonnerie d'un bâtiment érigé au 17ème siècle près de Hasselt. Quatre murs ont été préparés pour les essais (Fig. 4).

La résistance à la compression de la maçonnerie non injectée est mesurée sur le muret n° 4. La courbe des déformations en fonction des tensions est donnée à la figure 5.

Cette figure révèle un phénomène important d'hystérésis dans le matériau. Le module d'élasticité est égal à 1570 N/mm^2; la résistance à la compression est égal à 11,5 N/mm^2. Admettant cette résistance on a calculé la résistance à la compression des trois autres murets à l'aide des formules données dans la norme NBN B24-301 [3]. Les valeurs f_{cuc} sont reprises au table 2. Aux trois autres murets le module d'élasticité a été déterminé avant l'injection (E_1) et après (E_2). A titre d'exemple la courbe tension-déformation pour le muret 1 est donnée par la figure 6.

Les résultats sont résumés aux tables 1 et 2.

Muret n°	E_1 N/mm^2	E_2 N/mm^2	E_2/E_1	produit d'injection	% de la résine injectée
1	2069	6090	2,94	PC 1380 T	14,3
2	2351	6285	2,67	PC 1380 S	7
3	2425	5351	2,21	PC 1380 S	4,5

Table 1 Module d'élasticité avant et après l'injection.

Ces résultats montrent que :
- une augmentation importante de la rigidité peut être réalisée par l'injection de résine epoxy;
- la résistance à la compression a augmentée de 60 à 80 pourcent vis-à-vis de la résistance initiale, même pour une maçonnerie encore en bon état;
- le comportement tension-déformation est devenu lineairement élastique jusqu'à des contraintes élevées (Fig. 6).

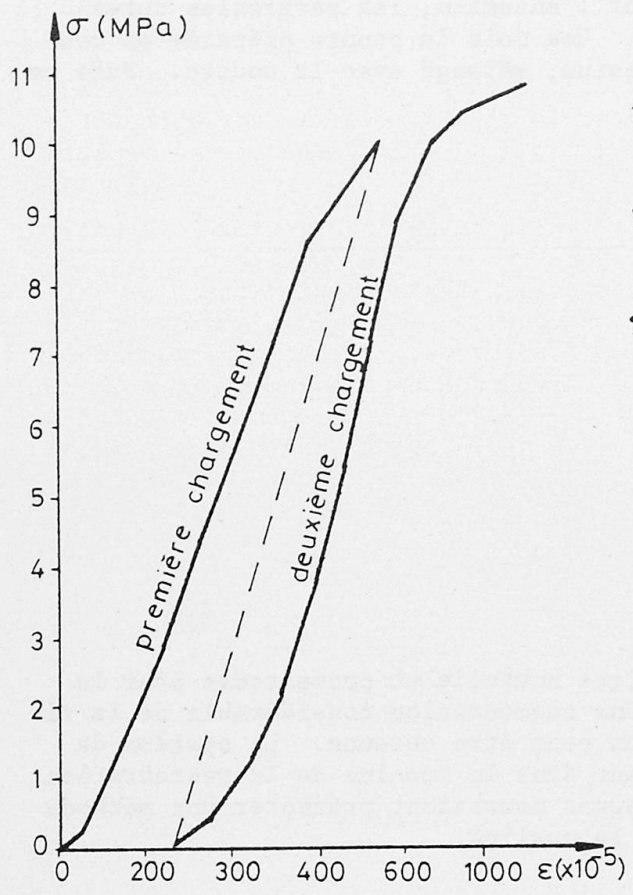


Fig. 5 Courbes tension-déformation pour muret 4

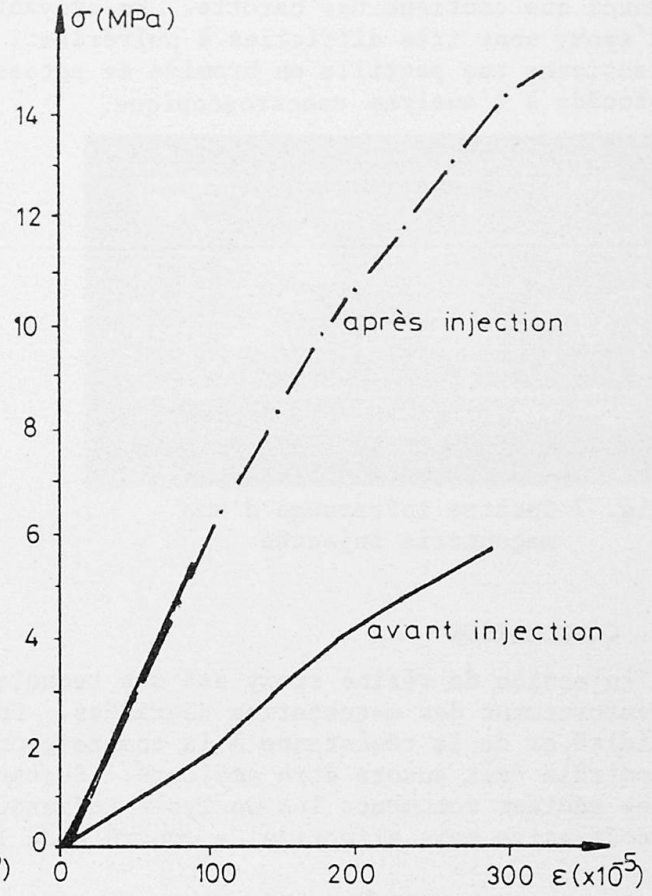


Fig. 6 Courbes tension-déformation pour muret 1

Muret n°	f_{cu} (mesurée) N/mm²	f_{cuc} (NBN B24-301) N/mm²	f_{cu}/f_{cuc}
1	15,35	8,87	1,73
2	16,54	8,86	1,87
3	16,32	10,01	1,63

Table 2 Résistance à la compression avant et après l'injection

3.3. Analyse par spectroscopie infrarouge

Des mesures de contrôle sur des tronçons de murs comme décrites ci-dessus seront rarement possible. Pour cette raison nous étudions les possibilités de l'analyse par spectroscopie infrarouge afin de déterminer les caractéristiques mécaniques de la maçonnerie injectée. Si nous réussissons à établir une relation entre les données du spectre infrarouge et la résistance, nous obtenons en même temps toutes les informations sur la résine injectée.

D'abord nous essayons de déterminer le pourcentage volumique de résine injectée dans la maçonnerie. Dans ce but une analyse quantitative du spectre est nécessaire : les raies caractéristiques d'absorption de la résine epoxy sont repérées, ainsi que la relation entre la surface intégrée par ces raies et le pourcentage d'epoxy, celui-ci étant le facteur qui détermine la résistance acquise. La figure 7 montre le spectre infrarouge d'une maçonnerie injectée.

L'élaboration de cette méthode nécessitera la résolution de multiples problèmes. D'abord l'acquisition d'un échantillon représentatif pour toute la maçonnerie : ceci est réalisable par le forage d'un nombre de carottes assez élevé et de diamètre réduit (16 mm). Un autre problème est le broyage en poudre de l'epoxy

durci que contient une carotte. En broyant l'ensemble, les particules dures d'epoxy sont très difficiles à pulvériser. Une fois la poudre préparée on confectionne une pastille en bromide de potassium, mélangé avec la poudre. Puis on procède à l'analyse spectroscopique.

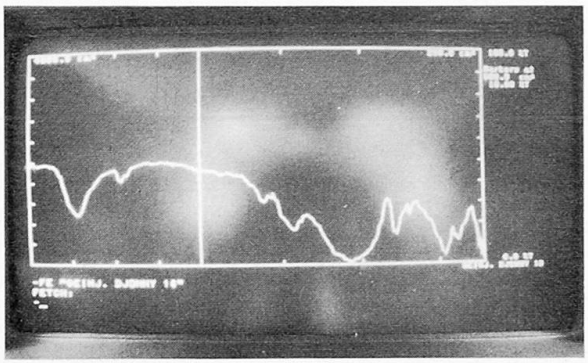


Fig. 7 Spectre infrarouge d'une maçonnerie injectée

4. CONCLUSIONS

L'injection de résine epoxy est une technique nouvelle et prometteuse pour le renforcement des maçonneries dégradées. Une augmentation considérable de la rigidité et de la résistance à la compression peut être obtenue. Le système de contrôle doit encore être amélioré. Surtout dans le domaine de la restauration des anciens monuments les analyses infrarouges pourraient présenter une méthode inoffensive mais effective de contrôle de la qualité.

REMERCIEMENTS

Les auteurs expriment leurs vifs remerciements à l'égard de la société E.C.C. et son directeur Mr. D. De Neef, pour le support logistique et matériel aux recherches, ainsi que l'Institut Belge pour la promotion de la Recherche Scientifique dans l'Industrie et l'Agriculture (I.R.S.I.A.) qui a subventionné les recherches. Nous remercions également MM. M. Opdebeeck et W. Smolders, qui ont effectué un grand nombre d'essais dans le cadre de leur travail de fin d'étude.

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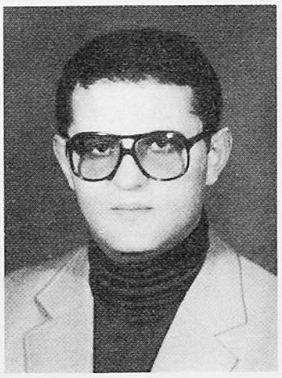
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Strengthening of a Prestressed Concrete Shell Structure

Renforcement d'une coque en béton précontraint

Verstärkung eines Spannbeton-Schalentragwerkes

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SUMMARY

In this paper, strengthening of a prestressed concrete shell in a chemical factory, in Cairo, is presented. The valley beams were strengthened by external prestressing cables anchored outside the building in precast end blocks. The shells were strengthened by casting 4 cm thick resin concrete on top of the existing shell.

RESUME

L'article présente le renforcement d'une coque en béton précontraint pour une usine de produits chimiques au Caire. Les poutres des extrémités ont été renforcés par das câbles extérieurs en précontrainte. Ces câbles ont été fixés par des blocs aux extrémités. Les coques on été renforcées par coulage de 4 cm de béton de résine sur la surface des coques existantes.

ZUSAMMENFASSUNG

Der Bericht behandelt die Verstärkung einer vorgespannten Schalenkonstruktion in einer chemischen Fabrik in Kairo. Die Schalenrandbalken wurden durch äussere Vorspannglieder verstärkt, die ausserhalb der alten Konstruktion in vorgefertigten Stahlbetonblöcken verankert wurden. Die Schalenhaut selbst wurde durch das Aufbringen einer 4 cm dicken Harz-Betonschicht verstärkt.

1. INTRODUCTION

In this paper strengthening and repair of a prestressed concrete shell is discribed. The building under consideration, which was constructed in the late forties, is an industrial building in a chemical factory in the area of greater cairo, Egypt.

The building is devided in to four identical parts separated by three expansion joints as shown in fig.1. Each part is composed of three cylindrical reinforced concrete shells with prestressed edge beams. Each shell is 10ms wide, 16ms long and 6cm thick. The intermediate edge, valley, beams were prestressed with three FREYSSINET cables 12 o 5 mm while the exterior beams were prestressed with only two cables of the same type.

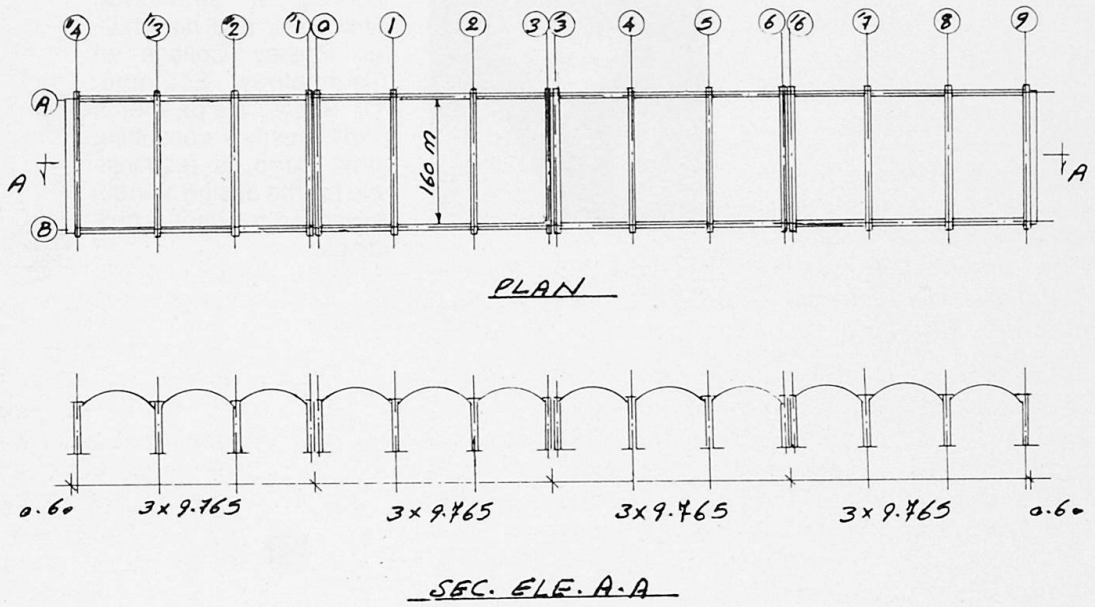


Fig. 1 General Layout

2. DAMAGES OBSERVED

2.1 Discription Of The Damage

In 1978 i.e. thirty years after construction, sudden collapse of the first part took place.

The factory manager requested a thorough investigation to the rest of the building in order to ensure the adequate safety. The following damages were recorded in the remaining three parts:

- In the webs of the prestressed beams - especially for those of the intermediate beams- several cracks spaced about 50cm between each other were noticed at mid-span. These cracks were extending vertically about 60cm from the bottom surface of the beams with crack widths varying from 0.5 to 3mm on both sides and on the bottom surface of the beams as shown in fig.2.
- In some localized areas of the shells, longitudinal and transversal cracks on the inside face and sometimes on the outside face were noticed. In these areas one can assume that the concrete is linked together along the cracks - which act as hinges- only by the reinforcing bars.

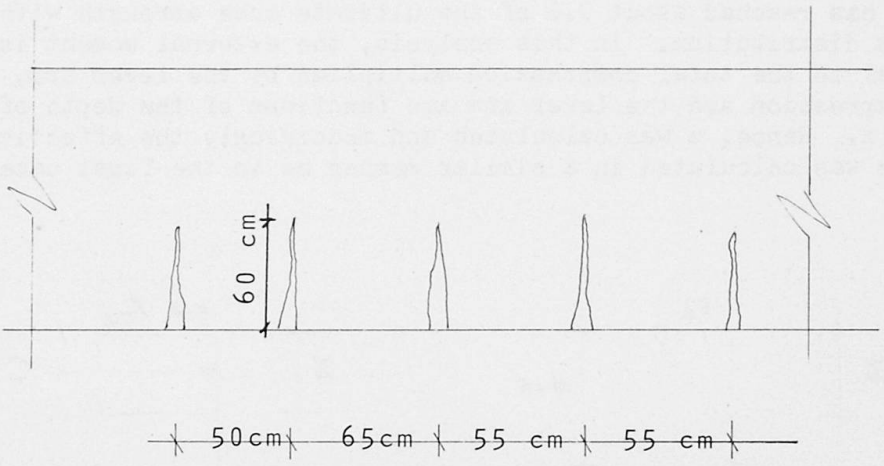


Fig. 2 Cracks At Mid Span Of Beams

2.2 Reasons For Damage

Generally in any building, damage can take place due to the action of one or more of several reasons such as overloading of any element, fires, unexpected deformations due to unknown soil conditions, accidental actions due to poor quality of construction, chemical and/or physical actions due to errors in specifications and design. However, for the building under consideration, the original design was checked and found to be safe.

The main reasons for the damages were the miss use and bad maintenance. A steel water tank of two cubic meters capacity was located on top of one of the shells. Plenty of pipe networks were also located on top of the shells. These pipes were not well connected and leakage of water and chemicals was noticed. The gutters over the valley beams, between adjacent shells, were found to be almost filled with mud which prevented proper drainage of water to take place. This mud and water added an extra load on the beams which was not taken in to account in the original design.

Due to the above mentioned reasons, the beams were cracked and the prestressing cables started to corrode. Accordingly, the effective prestressing forces were reduced considerably, followed by rupture of prestressing cables and sudden collapse.

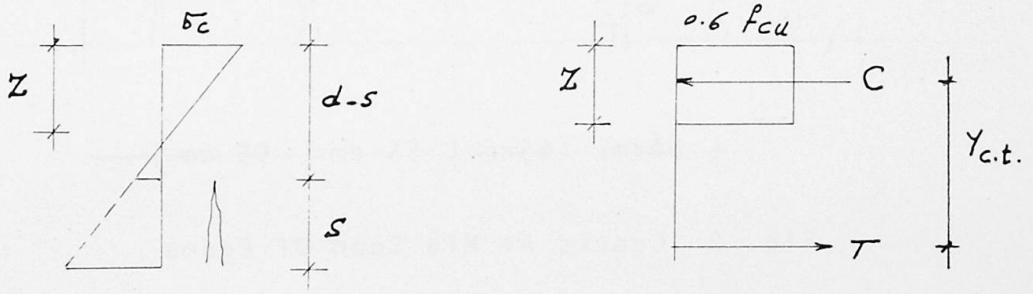
3. ANALYSIS OF THE STRUCTURAL BEHAVIOUR

It was necessary to estimate the remaining prestressing forces in the valley beams. In this respect certain bounding assumptions were made regarding the stress distribution across the critical section, i.e. section at mid span, and regarding the effective flange width to be considered in the analysis.

As far as the stress distribution is concerned, two limiting cases were considered as follows:

- In the first case, fig. 3a, the stress distribution was assumed to be linear. In this case it was assumed that the tensile stress at the level of the top end of the crack is equal to the tensile strength of the concrete which can be evaluated. The actual external moment applied on the section was calculated, i.e. it was known. Thus from first principals, the total compression was calculated which is equal to the total tension. The effective prestressing force was accordingly estimated.
- In the second case, fig. 3b, it was assumed that the compressive stress in

the concrete has reached about 0.6 of the ultimate cube strength with a non-linear stress distribution. In this analysis, the external moment is known which is equal to the total compression multiplied by the lever arm, $Y_{c.t.}$. Both the total compression and the lever arm are functions of the depth of the nutral axis, z. Hence, z was calculated and accordingly the effective prestressing force was calculated in a similar manner as in the first case.

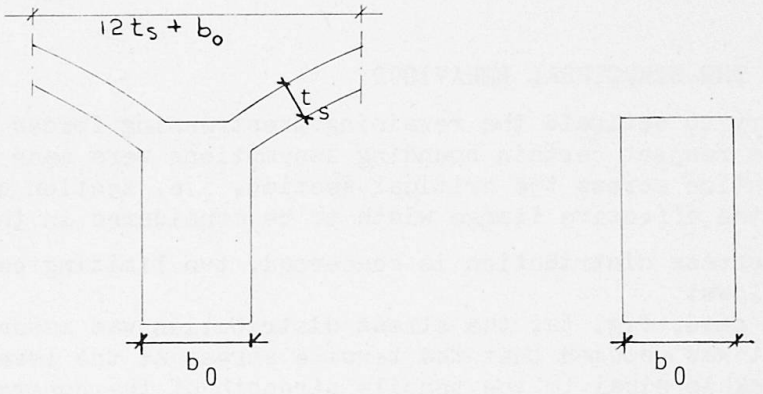


a) Linear stress distribution b) Equivalent rectangular stress block.

Fig. 3 Assumptions Of Stress Distribution

As far as the effective flange width is concerned, two cases were considered for each stress distribution case as follows:
- The beam was considered to be as a rectangular section without any flanges acting as shown in fig.4a. This case gave the heighest value of loss of prestressing.
- The beam was considered to have an effective flange width equal to the web bredth plus twelve times the shell thickness as shown in fig. 4b.

It should be noted that the problem under consideration is not that straight forward and some engineering judjment had to be made. From the crack pattern in the valley beams together with the above mentioned calculations, it was estimated that a maximum loss of about 40% in the original prestressing force has taken place.



b) Flange width $12t_s + b_0$ a) No flange acting

Fig. 4 Assumption Of Effective Flange Width Acting

4. SYSTEM ADOPTED FOR REPAIR

4.1 Beams

Two alternative methods for strengthening the beams were considered as follows:
- The first one by using ordinary reinforced concrete in jacketing the beams as shown in fig.5. The advantage of this method was that it does not need especial equipments. However, it needs the use of shuttering and especial casting technique in order to make sure that the fresh concrete has filled the shuttering completely. It necessitated breaking of the external walls in order enable casting the columns jackets which were required to support the jacketing of the beams.

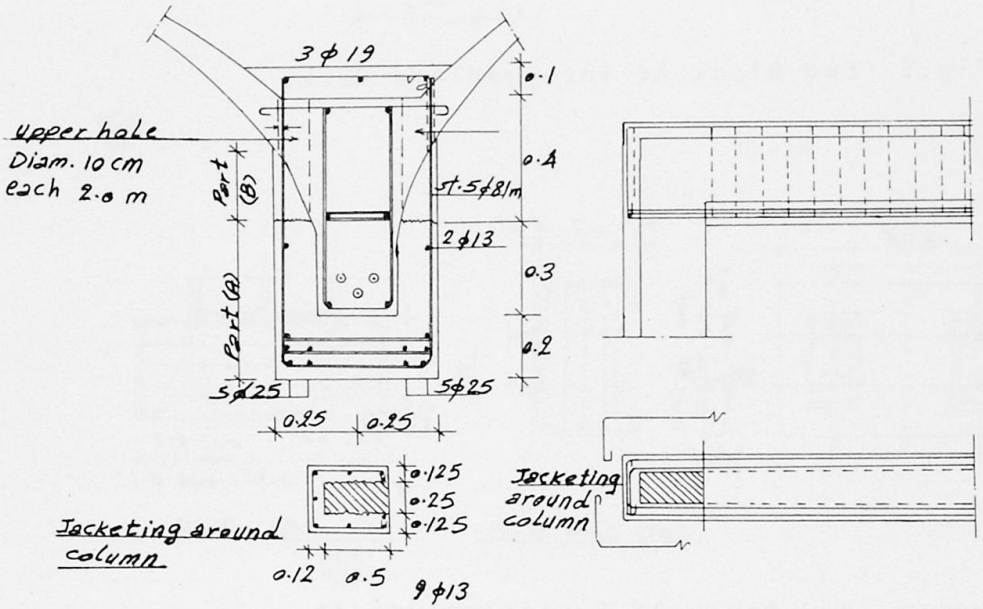


Fig. 5 First proposal for repair using ordinary reinforced concrete

- In the second method, the beams were to be strengthened by the use of external prestressing cables. These cables were to be anchord - outside the building - in precast concrete end blocks. Fig. 6 & 7 show the end blocks used for the intermediate beams and that used at the expansion joints. It should be noted that for the end blocks at expansion joints neoprene sheets were installed between the columns and the blocks in order to allow for expansion movements. Fig. 8,9&10 show the repair for the exterior beams, for the intermediate beams and for the beams at the expansion joints respectively. The advantage of this method over the first method is that it saves time, it is easy to perform, it does not add any more weight to the beams and it needs no shuttering, i.e. the repair can be carried out without the need of stopping the production of the factory. However, it needs especial prestressing equipments to be available.

It should be mentioned that in both proposed methods calculations were made for the following two limiting cases:
- The case of full old prestressing force acting.
- The case of 40% loss of old prestressing has taken place.

The second method was adopted and was carried out.

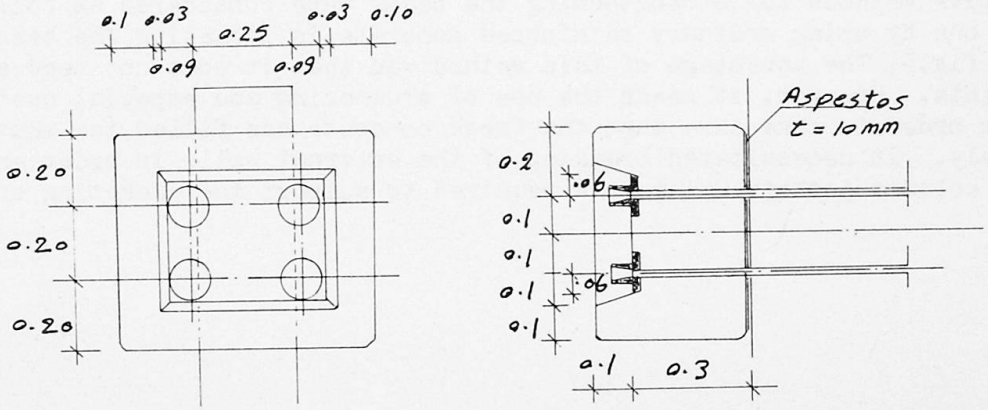


Fig.6 End Block At Intermediate Beams

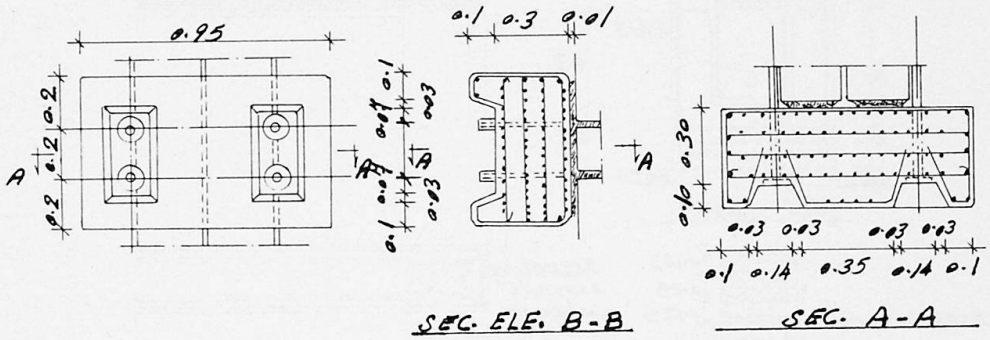


Fig.7 End Block At Expansion Joints

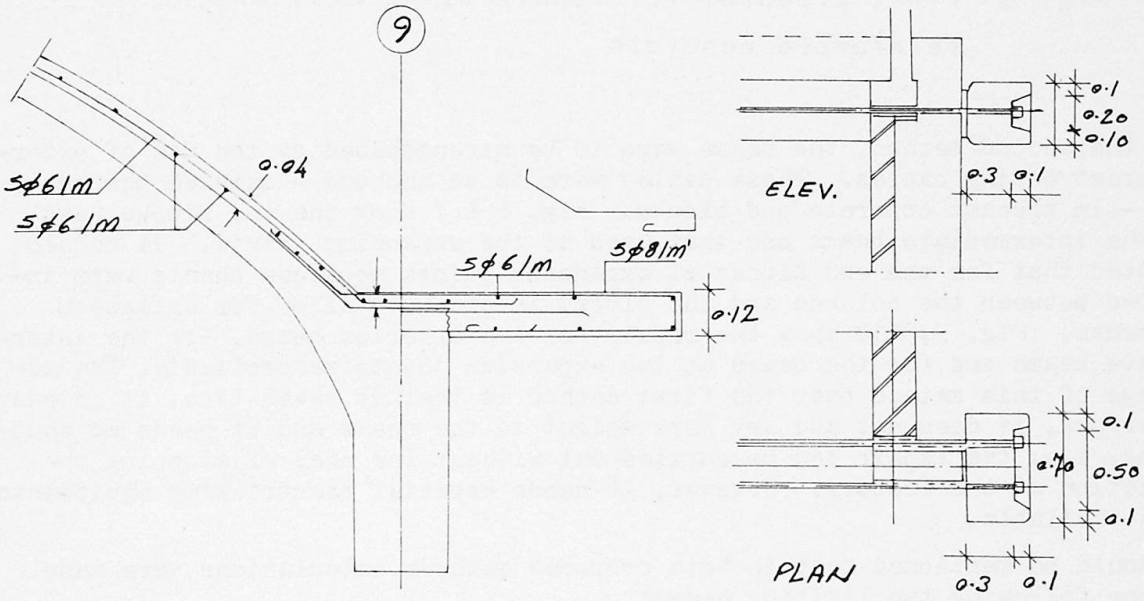


Fig.8 Repair Of Exterior Beams

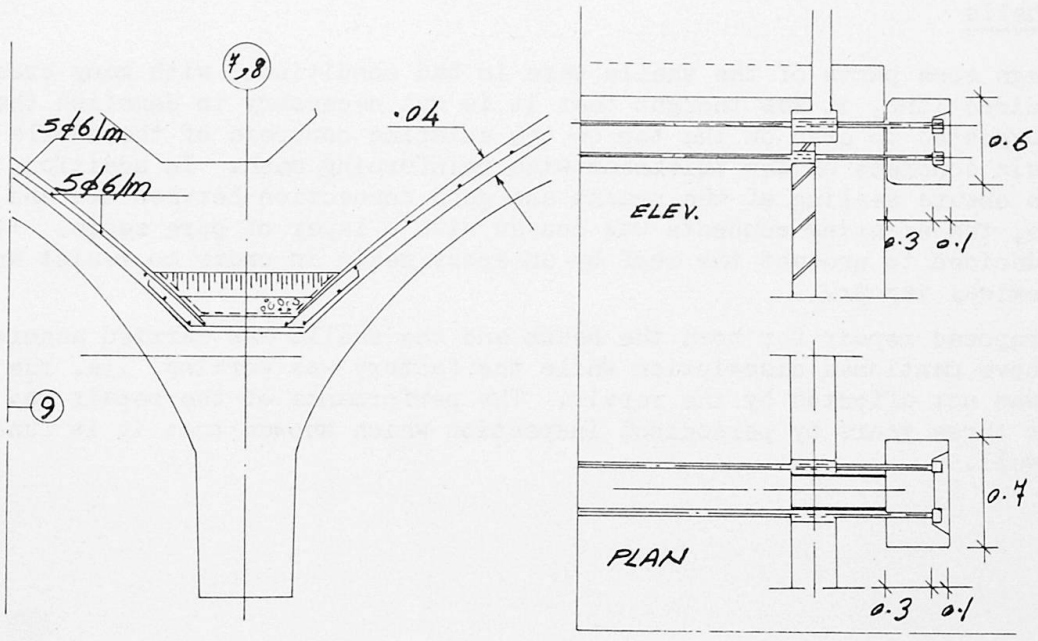


Fig.9 Repair Of Intermediate Beams.

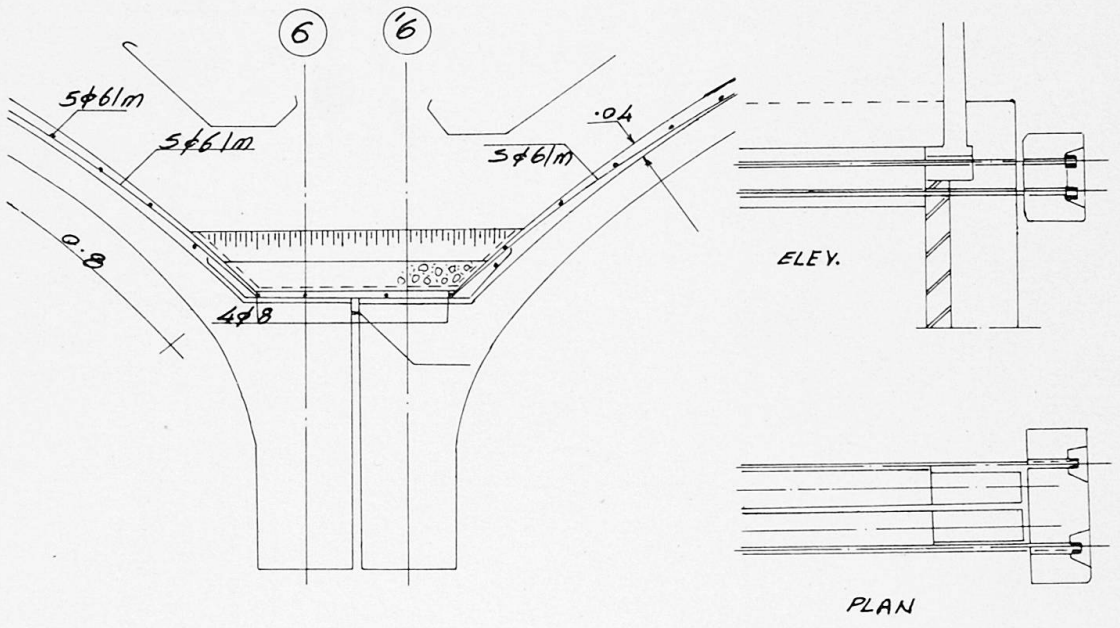


Fig.10 Repair Of Beams At Expansion Joints.

4.2 Shells

Although some parts of the shells were in bad conditions, with many cracks in both directions, it was thought that it is not necessary to demolish them. It was suggested to cast on the top on the existing concrete of the shella a layer of resin concrete of 4cm thickness with reinforcing mesh. In addition, in order to ensure sealing of the cracks and good connection between old and new concrete, the existing concrete was coated with a layer of pure resin. It was also decided to protect the roof by an epoxy resin in order to resist any water or chemical vapour.

The proposed repair for both the beams and the shells was carried according to the above mentioned discription while the factory was working, i.e. the production was not affected by the repair. The performance of the repair was recorded for three years by periodical inspection which showed that it is functioning very well.

Qualité des produits de réparation des structures en béton

Qualitätsbeurteilung von Ausbesserungsmaterialien für Betonbauten

Quality of repair products for concrete structures

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RESUME

Des organismes français ont étudié en commun le problème particulier de l'évaluation de la qualité des produits de réparation commercialisés. Ils ont rédigé des modes opératoires d'essais spécifiques et défini les modalités de mise en place d'une «Procédure d'évaluation de la qualité des produits de réparation des structures en béton». L'organisme coordinateur de la Procédure est le LCPC.

ZUSAMMENFASSUNG

Französische öffentliche Behörden haben gemeinsam das schwierige Problem der Bewertung der Qualität von Ausbesserungsmaterialien auf Kunstharzbasis, die auf dem Markt angeboten werden, bearbeitet. Es werden verschiedene spezifische Versuchsmethoden behandelt und Richtlinien für ein «Verfahren zur Qualitätsbewertung von kommerziellen Kunstharz-Ausbesserungsmaterialien für Betonbauten festgelegt. Das LCPC wurde mit der Leitung dieses Projektes beauftragt.

SUMMARY

Several French organisations have jointly studied the particular problem of estimating the quality of the repair products which are currently available. They have drafted specific operating instructions and defined the methods of setting up an «evaluation procedure for the quality of repair products for concrete structures». The projects coordinating organisation is the LCPC.

PREAMBULE

Tôt ou tard l'ingénieur qui doit résoudre le problème de réparation d'une structure en béton dégradée se trouve confronté à la question du choix du produit le mieux adapté à la méthode de réparation qui a été décidée. C'est alors que surgissent paradoxalement des problèmes d'insuffisance et de surabondance de textes ou documents lui permettant de guider son choix.

En effet pendant bien longtemps l'utilisateur s'est trouvé, d'une part, face à une absence pratiquement totale de recommandations ou de règlements lui permettant de connaître, à bon escient, les produits pouvant répondre aux exigences posées par les techniques de réparation nécessaires à l'ouvrage dégradé et d'autre part, face à une industrie chimique lui proposant un nombre de produits excessivement variés et diversifiés accompagnés d'une multitude de notices ou documents à caractère trop souvent publicitaire et parfois abusif.

Il s'en est suivi dans de nombreux cas des applications inadaptées, des emplois de produits inadéquats, suivis d'échecs plus ou moins graves dans la réparation effectuée.

Conscients de la gravité et de l'ampleur du problème les Organismes et Laboratoires se sont penchés sur cette question et ont effectué de nombreuses recherches sur les produits de réparation. En France, ces recherches se sont concrétisées, dans une première étape, dans la rédaction du "Guide pour le choix et l'application des produits de Réparation" publié par le Laboratoire Central des Ponts et Chaussées (LCPC) et le Service Etudes Techniques des Routes et Autoroutes (SETRA). Ce guide était destiné à permettre au Maître d'Oeuvre de s'orienter dans le dédale formé par la multitude de produits commercialisés. Il définit les principales familles existantes : liants hydrauliques liants mixtes c'est-à-dire association de liant hydraulique et polymère, les polymères thermoplastiques et les polymères thermodurcissables. Ces définitions sont accompagnées d'informations sur les types de réparations auxquelles s'adaptent au mieux les divers produits, leurs limites d'emploi ainsi que les exigences indispensables à chacune de leur application.

Cependant si ce guide permet à l'ingénieur d'accéder en toute connaissance de cause au langage (ou terminologie) employé par les producteurs, il ne peut pas le renseigner sur les performances exactes que présente un produit donné. Ainsi donc dans une deuxième étape, les organismes officiels et les représentants des utilisateurs ne sont unis pour mettre en place une Procédure d'Evaluation des Produits de Réparation.

OBJECTIFS DE LA PROCEDURE D'EVALUATION

Cette procédure qui s'applique aux produits destinés

- au collage du Béton frais sur Béton durci(Bf/Bd), du Béton durci sur Béton durci (Bd/Bd) (assemblage de voussoirs préfabriqués par exemple)
- aux réparations de surface du Béton
- à l'injection dans les fissures et cavités du béton ou des maçonneries
- aux scellements ou aux calages
- au collage de l'acier sur béton.

a deux buts essentiels :

1) Connaître, à partir d'une série de tests bien définis, l'efficacité et les caractéristiques des Produits de Réparation destinés aux applications précédemment citées.

2) Couvrir les exigences des utilisateurs ou organismes français suivants :

AEROPORT DE PARIS - Centre d'Etudes du Bâtiment et des Travaux Publics (CEBTP) - (COPREC) - Centre Scientifique et Technique du Bâtiment (CSTB) - Electricité de France (EDF) - Laboratoire Central des Ponts et Chaussées (LCPC) - Service d'Etudes Techniques des Routes et Autoroutes (SETRA) Société Nationale des Chemins de Fer Français (SNCF).

Ceci est important car, les résultats obtenus, dans les essais de la dite procédure, sont reconnus par l'ensemble des participants ce qui évite au producteur ayant soumisces produits à la procédure d'évaluation de recommencer sans cesse, au gré de chaque réparation, les mêmes essais ou des essais très analogues. Bien entendu, chaque organisme ou utilisateur se réserve le droit de faire subir au produit des essais spécifiques à une réparation bien déterminée.

DEFINITION DE LA PROCEDURE

Elle concerne les produits livrés avec tous leurs composants pré-dosés, elle ne couvre pas la mise en oeuvre des produits de réparation.

C'est à la demande du producteur et à ses frais que la procédure d'évaluation a lieu sur un produit donné. Cette demande doit être adressée, suivant des modalités bien définies, à l'Organisme Coordinateur de la Procédure, (en l'occurrence le LCPC), organisme qui assure le suivi des essais auprès des laboratoires habilités, et qui délivre le procès-verbal officiel d'essais. Les produits ayant subi la totalité des essais de la procédure figurent sur une liste officielle publiée dans le Bulletin de Liaison des Laboratoires des Ponts et Chaussées.

Le producteur s'engage, par ailleurs, à maintenir constante la formulation de son produit, ou à défaut à informer immédiatement l'organisme coordinateur, de toute modification apportée à la composition ou fabrication du produit figurant sur la liste.

STRUCTURATION DES ESSAIS

La procédure d'évaluation comprend les essais d'efficacité et les essais d'identification complète.

Afin de ne pas alourdir inutilement le travail des laboratoires habilités pour les essais, et les dépenses des producteurs, les essais sont effectués en trois étapes. A la fin de chacune d'elles, le demandeur peut, au vu des résultats obtenus, arrêter la procédure et retirer son produit.

La première étape comprend :

. Les essais d'identification rapide destinés à permettre au laboratoire d'essais de vérifier rapidement: qu'au cours de n'importe quelle livraison, pour la poursuite des essais de la procédure ou pour un chantier, le produit est bien le même et identique à celui qui subit, ou a subi, les essais d'identification complète.

. Les essais aux conditions limites d'utilisation très simples qui permettent, de mettre en évidence rapidement, le bien-fondé, ou mal fondé, de certaines caractéristiques annoncées par le producteur : thixotropie, température d'utilisation etc.. et d'orienter les essais d'efficacité en supprimant ceux pour lesquels le produit n'est pas adapté. Il va de soi que toute suppression de test prévu dans la procédure, se traduit, sur la liste officielle, par une observation, à caractère restrictif, sur l'emploi du produit.

. Les essais d'efficacité à l'état neuf

La deuxième étape concerne les essais d'efficacité à l'état vieilli. Les éprouvettes réparées avec le produit sont soumises à des conditions de vieillissement conventionnelles.

La troisième étape n'est abordée qu'à la demande du producteur sur les produits ayant conduit, dans les étapes précédentes, à des résultats estimés satisfaisants. Cette étape consiste à effectuer l'identification complète du produit.

Les résultats des essais d'efficacité concernent un échantillon prélevé dans certaines conditions. Leur représentativité est ainsi liée à celle de l'échantillon. Il importe donc de procéder à son identification afin de connaître ultérieurement si le produit livré sur le chantier correspond bien à celui qui a été testé au laboratoire, et si une modification quelconque d'efficacité, constatée in situ, peut être attribuée à un changement intervenant dans sa formulation.

Chaque application (collage du Béton frais sur Béton durci ou du Béton durci sur Béton durci ; injection dans les fissures, mortiers pour Réparation de surface, scellement et calage) fait l'objet d'une procédure distincte ; les points communs sont :

. le produit est toujours appliqué ou mis en oeuvre sur le support béton par le producteur.

. Il est toujours appliqué sur deux états de surface : sèche et humide, sauf contre indication restrictive du producteur.

. La surface du béton support est sciée sauf quelques exceptions près liées à la nature de l'essai. La surface sciée a été choisie car à l'heure actuelle c'est la seule qui nous a paru la plus répétitive et reproductible, dans un laboratoire et entre laboratoires.

ESSAIS PRINCIPAUX

Il faut signaler avant tout que l'ensemble des essais de la procédure font l'objet soit de normes, soit de modes opératoires officiels bien définis.

1) Essais d'efficacité à l'état neuf et de vieillissement conventionnel

1.1) Produits de collage Bf/Bd et Bd/Bd

Les principaux essais sont ceux du collage structural de Bf ou Bd lorsque la surface d'application est verticale ou horizontale.
On trouve ainsi :

a) L'essai de traction directe sur cylindre 16/32cm scié en son milieu suivant un plan médian perpendiculaire à la direction des génératrices (Fig.1) et reconstitué soit par apport de béton frais, soit par collage d'un demi-cylindre de béton durci.

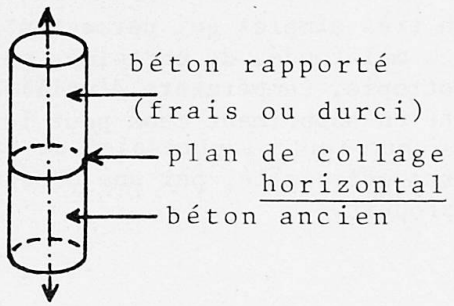


Figure 1 : Essai de traction directe sur cylindre scié et reconstitue suivant un plan médian perpendiculaire à la direction des génératrices.

b) L'essai de compression cisaillement sur prisme 10 x 10 x 30 cm scié suivant un plan oblique faisant un angle de 30° suivant l'axe longitudinal (Fig.4) et reconstitué soit par apport de béton frais, soit par collage d'un prisme scié à 30° de béton durci.

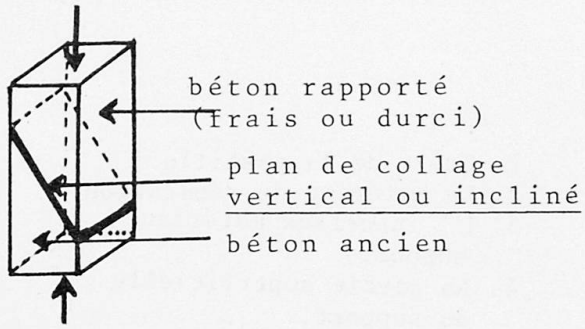


Figure 2 : Essai de compression sur prisme scié et recollé suivant un plan oblique faisant un angle de 30° avec l'axe longitudinal de l'éprouvette.

Les éprouvettes sont soumises aux essais de traction directe ou de compression-cisaillement 28 jours après leur reconstitution par collage. Ces essais sont effectués à 20°C mais peuvent être réalisés à la demande du producteur soit à + 5°C dans le cas des formulations pour l'hiver, soit à + 35°C pour tester une formulation pour l'été.

L'efficacité du produit est jugée en fonction de la forme de rupture des éprouvettes.

Les cas suivants peuvent se présenter :

- rupture en plein béton rapporté ou ancien pour une contrainte analogue ou supérieure à celle du béton témoin,
- rupture au joint de collage ou semi décollement pour une contrainte supérieure de 20 à 25 % à celle du béton témoin.

  Ce sont les deux cas où le produit est le plus efficace.

- rupture mixte béton-joint de colle, ou décollement pour des contraintes analogues à celles du béton témoin.

  On peut alors estimer que le produit ne peut être utilisé que sous certaines réserves et pour des réparations faiblement sollicitées.

- décollement du béton pour des contraintes inférieures à celles du béton témoin : c'est le cas le plus défavorable, le produit ne convient pas pour ce genre de réparation.

Le vieillissement est mesuré par un essai de fluage sous charge permanente.

Il faut noter le cas particulier des produits destinés à l'Assemblage de Voussoirs préfabriqués des ponts construits par encorbellement, pour ces produits les essais sont en cours d'étude et revêtiront un caractère spécifique.

1.2) Mortiers de réparation de surface

Les essais proposés visent à tester :

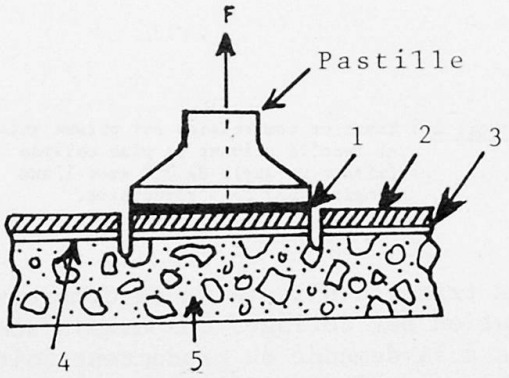
- d'une part l'adhérence au support en fonction, de l'état d'humidité de ce dernier (surface sèche ou humide),
- d'autre part, l'influence d'une arrivée d'eau sur l'adhérence, la perméabilité et les caractéristiques mécaniques du mortier de ragréage.

Ces derniers essais intéressent principalement les mortiers contenant du ciment. Ils consistent à effectuer des essais de perméabilité à l'eau sous gradient hydraulique et des essais de compression flexion sur éprouvettes normalisées 4 x 4 x 16 cm.

a) Adhérence sur dalle

Le mortier de ragréage est appliqué sur une épaisseur de 1 cm, avec ou sans primaire d'accrochage, suivant les directives du producteur, sur une dalle de béton durci de dimensions 30 x 30 x 6 cm .

28 jours après l'application, on procède à six essais de traction directe type chapes minces d'étanchéité (Fig. 3).



1. Colle de la pastille
2. Le matériau de réparation
3. L' interface matériau support.
4. La partie superficielle du support.
5. La partie saine du support.

Figure 3 : Schéma du dispositif de traction.

L'adhérence est jugée satisfaisante lorsque la rupture a lieu dans le béton.

b) Essais de flexion centrée sur éprouvette réparée en fibre tendue

L'essai consiste à reconstituer à l'aide du mortier de ragréage une éprouvette prismatique 10 x 10 x 40 cm présentant une partie évidée, le long d'une de ses faces latérales, sur 1 cm de profondeur et 20 cm de longueur (Fig. 4). On mesure 28 jours après le ragréage la résistance à la flexion centrée de l'éprouvette, la partie ragréée étant en fibre tendue.

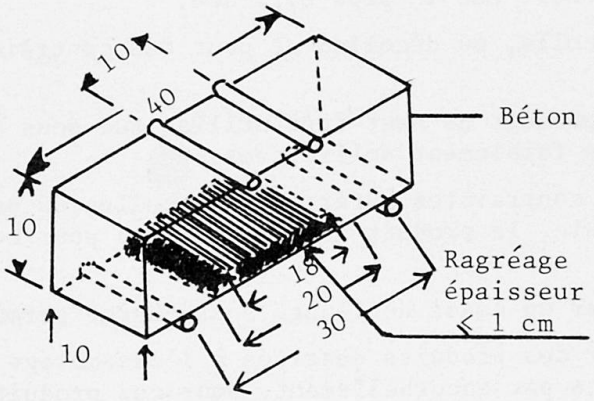


Figure 4 : Essai de flexion centrée sur éprouvette prismatique évidée puis ragréée en fibre tendue.

La figure 5 montre les divers types de rupture qui peuvent se présenter.

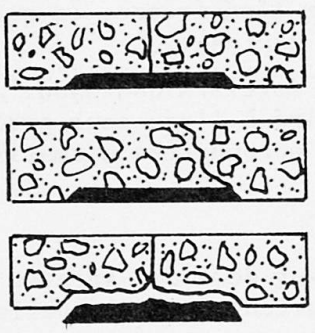


Figure 5 : Les différents modes de rupture.

Les tests à l'état vieilli consistent à mesurer l'adhérence sur dalle après des cycles de variation rapide de température : 100 cycles de 6 heures (-25°C, + 55°C) ainsi qu'après des essais de gel dégel sur prisme réparé sur la fibre tendue.

1.3) Produits de réparation interne des structures en béton (injection)

L'injection au Laboratoire consiste à faire pénétrer sous pression (0,075 MPa) de bas en haut d'une colonne remplie de sable calibré, le produit destiné aux injections.

Le pouvoir de pénétration de celui-ci est déterminé par le temps qu'il met à atteindre dans la colonne de sable les niveaux 5,10,15, 20, 25 cm.

La figure 6 montre les différentes courbes de pénétration obtenues avec divers produits.

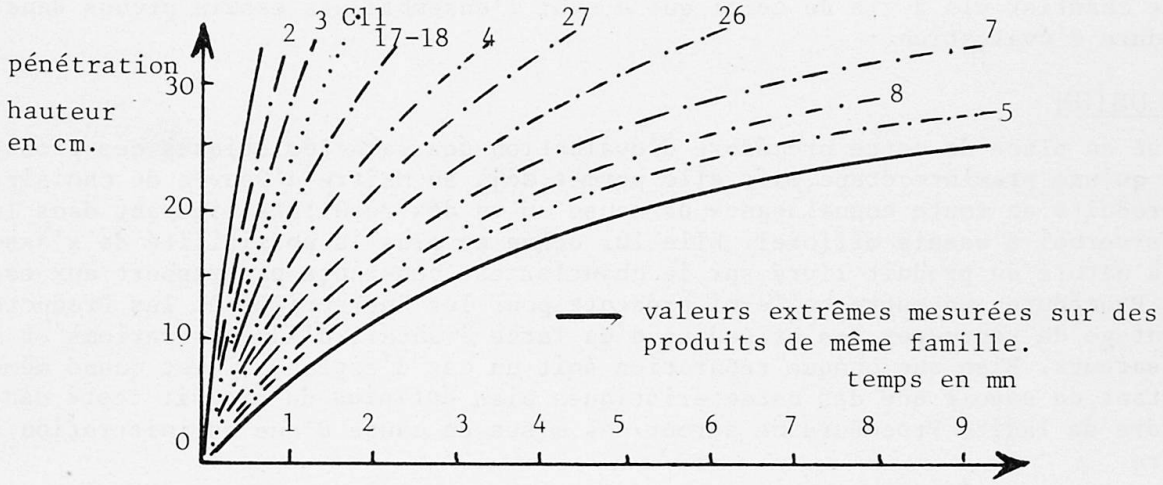


Figure 6: Courbes d'injectabilité à la colonne de sable sec à 20°C

Les applications sur chantier ont montré que les produits, dont la hauteur de pénétration est de 30 cm après un temps d'injection compris entre 0 et 2 minutes, n'ont pas présenté de difficulté à être injectés dans des fissures de largeur inférieure à 0,8 mm. Il est à noter que le produit n° 11 à base de ciment, et qui se trouve placé dans ce groupe, a pu être injecté avec succès dans des fissures d'une ouverture de 0,5 mm environ.

On peut aussi connaître avec ce test les possibilités d'injectabilité des produits à température variable et en milieu humide. Il suffit pour cela de placer la colonne de sable et le dispositif d'injection dans une enceinte climatisée, ou de faire circuler de l'eau dans la colonne de sable avant l'injection du produit à étudier.

Cet essai permet par ailleurs, de connaître l'adhérence du produit d'injection. Pour cela on laisse durcir la colonne de sable après remplissage par le produit pendant 28 jours et on effectue des essais de fendage (essais brésiliens) sur des cylindres d'élancement 2 obtenus par tronçonnage de ladite colonne.

Les essais de vieillissement conventionnel consistent à mesurer l'adhérence d'éprouvettes 4 x 4 x 16 cm après cycles thermiques ainsi que de fatigue mécanique.

2) Essais d'identification complète

Les essais sont destinés à étudier les caractéristiques

. des divers composants, on distingue dans ce cas les liquides et pâtes, ainsi que les poudres et granulats.

. du mélange des composants à l'état non durci

. du produit durci.

Certains de ces essais revêtent un caractère confidentiel et par conséquent,sont exécutés uniquement par le LCPC. C'est le cas en particulier des analyses des divers composants des produits polymères qui fournissent,le Spectre Infra-Rouge, les dosages des fonctions réactives, l'analyse chimique et granulaire des charges, ainsi que les analyses minérale et minéralogique, le dosage en liant actif etc.. des produits hydrauliques.

Dans le cas des mélanges, on trouve la DPU, le retrait, les temps de prise, la fluidité etc.. ces essais ne revêtent pas en général un caractère confidentiel.Il en est de même pour les mesures des résistances mécaniques, la dilatation thermique, les essais potentiocinétiques etc.. effectués sur les mélanges durcis.

Ces essais permettent en cas d'incident de connaître la conformité du produit livré sur le chantier vis à vis de celui qui a subi l'ensemble des essais prévus dans la procédure d'évaluation.

CONCLUSION

La mise en place de cette procédure d'évaluation des caractéristiques des produits n'est qu'une première étape,mais elle permet déjà au Maître d'Oeuvre de choisir les produits en toute connaissance de cause au vu des résultats figurant dans le procès-verbal d'essais officiel. Elle lui donne en plus la possibilité de s'assurer que la nature du produit livré sur le chantier est constante par rapport aux essais de la Procédure. En outre celle-ci présente pour les Entreprises et les Producteurs l'avantage de regrouper les exigences d'un large éventail d'Administrations et d' Utilisateurs. Bien que chaque réparation soit un cas d'espèce, il est quand même important de savoir que des caractéristiques bien définies du produit testé dans le cadre de ladite Procédure ne seront pas mises en cause d'une administration à l'autre.

Reste maintenant à mettre en place dans un proche avenir la procédure de qualification de ces produits. Dans ce but on étudie actuellement les diverses spécifications à fixer et qui doivent permettre de franchir cette nouvelle étape.

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Strengthening of Steel Buildings Against Earthquakes

Renforcement de constructions métalliques contre les séismes

Verstärkung von Stahlbauten hinsichtlich Erdbebenbeanspruchung

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SUMMARY

The basic concept of earthquake resistance capacity, upon which the seismic design regulations in the current Building Code and the Diagnosis and Therapy Standard were developed, is summarized. The principle of the methods for strengthening and repairing steel buildings in line with this concept is discussed.

RESUME

Le rapport décrit le principe fondamental de la capacité de la résistance aux séismes des constructions, sur lequel ont été developpés les règles de calcul dans les Réglements de construction récents et les recommandations pour le diagnostic et la thérapie. Le principe des méthodes de renforcement et de réparation des constructions métalliques, selon cette conception, est illustré.

ZUSAMMENFASSUNG

Der fundamentale Begriff der Erdbebensicherheit, auf welchem die Bestimmungen für erdbebensicheres Bauen in den gegenwärtigen Bauvorschriften, Diagnose- und Therapierichtlinien fussen, wird zusammengefasst. Der Grundsatz wie Stahlbauten verstärkt und wiederhergestellt werden können, wird diskutiert.

1. INTRODUCTION

In 1978 two strong earthquakes hit the middle part of Japan. Especially the second brought the considerable damage to the east of old Sendai City, the largest city of the northern Japan. Investigation on damaged steel buildings was carried out in the limited area (approximately 4 Km x 4 Km) where the damage was concentrated[1]. The numbers of houses in the investigation area are shown in Table 1. There were many steel office buildings and warehouses in this commercial and industrial region. Accordingly, considerable percentage of structures suffered by the earthquake.

Reinforced concrete buildings and wooden houses besides steel structures suffered considerably. This fact might accelerate the revision process of the seismic design regulations in Building Code which were already revised in 1981. The revised regulations had been developed on the basis of the new design concept that the earthquake resistance capacity is proportional to the plastic strain energy absorption. At almost the same time a diagnosis and therapy standard was prepared by us and other researchers under the sponsorship of the Japanese Government. The standard was developed on the same concept. This paper describes firstly the basic concept of the seismic design regulations and the diagnosis standard, and then the principle of the therapy, namely, the methods for strengthening and repairing existing buildings.

Total	Steel houses			R.concrete	Wooden	Others
	1-story	2-story and higher	LGS*			
2432	861	412	150	299	697	13
100%	35.4	16.9	6.2	12.3	28.7	0.5

* Light gaged steel

Table 1 The numbers of buildings and houses in the investigted area

2. SEISMIC DESIGN CODE AND DIAGNOSIS-THERAPY STANDARD

2.1 Basic Concept of Earthquake Resistance Capacity

In the earthquake resistant building design, there must be two classes of limit states: Namely, (a) the serviceability limit state for a moderate intensity earthquake and (b) the ultimate limit state for a major earthquake. Precise discussions have been done on these two classes of limit states by one of the authors[2]. Here, only a brief discussion on the ultimate limit state is repeated, because the establishment of such a limit state is necessarily required to set a standpoint for strengthening and repairing the existing building structures.

In a major earthquake which is unlikely to occur within the life time of a structure, the structure may be permitted to undergo considerable structural damage. However, the collapse of the structure must be avoided for safety of human life. The following design criterion can be accepted to evaluate the structural safety of a steel building.

The energy E_d which contributes to the structural damage is the sum of the cumulative plastic strain energy (the dissipated strain energy) W_p and the elastic energy W_e;

$$E_d = W_e + W_p \qquad (1)$$

and can be approximately expressed as follows after Housner[3],

$$E_d = \frac{1}{2} MS_v^2 \qquad (2)$$

in which S_v = spectral pseudo-velocity response for undamped system and M = total mass of the structure. As a simple example, further derivation of the above equations will be done on a portal frame as shown in Fig. 1. In the case that the frame is yielded and undergoes considerable plastic deformation up to δm as shown in Fig.2, Eqs. 1 and 2 can be expressed as follows;

$$\frac{1}{2} Mg^2\omega^2C^2(1+4\eta) = \frac{1}{2} MS_v^2 \qquad (3)$$

in which g = the acceleration of gravity, ω = the natural circular frequency, C = the yield shear coefficient and η = the cumulative ductility ratio defined as $\eta = \delta m/\delta y - 1$. After the vibration theory,

$$C = \frac{1}{\sqrt{1+4\eta}} \left(\frac{S_a}{g}\right) \qquad (4)$$

where S_a = the spectral acceleration response. Eq. 4 gives the relation between the yield shear coefficient C and the associate ductility ratio η.

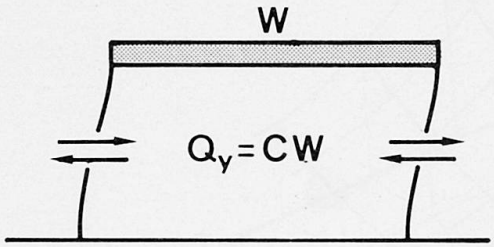


Fig.1 A portal frame

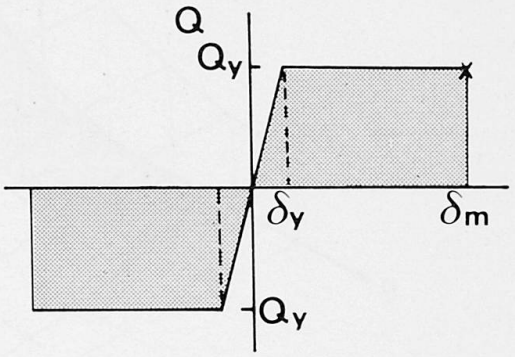


Fig.2 Story shear vs. sidesway relation

2.2 Seismic Design Regulations in Current Code

The seismic design regulations revised recently were established on the same basic concept as discussed above. From Eq. 4, the required yield shear coefficient C_1 is expressed as

$$C_1 \geq D_s(S_a/g)_D \ , \ D_s = \frac{1}{\sqrt{1+\alpha\eta_1}} \qquad (5)$$

in which $(S_a/g)_D$ = the design spectral acceleration index. The term D_S represents the structural characteristics of the frame and controlled by the required ductility ratio η_1. The coefficient α depends on the distribution of the stiffness and the story shear strength along the height in multi-story frames. The design spectral accumulation index $(S_a/g)_D$, a function of the natural period T, is expressed in the Code as

$$
\begin{aligned}
(S_a/g)_D &= 1 && ; \quad T < T_c \\
(S_a/g)_D &= 1 - 0.2(T/T_c - 1)^2 && ; \quad T_c \leq T < 2T_c \\
(S_a/g)_D &= 1.6T_c/T && ; \quad 2T_c \leq T
\end{aligned} \tag{6}
$$

in which T_c = the specific period depending on the soil condition at the site. In a graphical way, the index can be represented by the curve in the C-T plane of Fig. 3. The required ductility ratio η_1 is dependent on the shapes of skeleton members. The possible ductility ratios must be evaluated for the width-to-thickness ratios of section shapes, the slenderness ratios of laterally unsupported members and the slenderness ratios of brace members in the braced frames. In the regulations of the Code the maximum value of the required yield shear coefficient C_1 is determined 0.5 for steel structure. In other words it is concluded that steel frames are expected to have some ductility. C_1 in Eq. 5 can be considered eventually a function of T and η_1 . Then the value of C_1 moves on a surface of the spreading roof shown in Fig. 3. According to the revised regulation of the Building Code, the design yield shear coefficient must be on or above the surface.

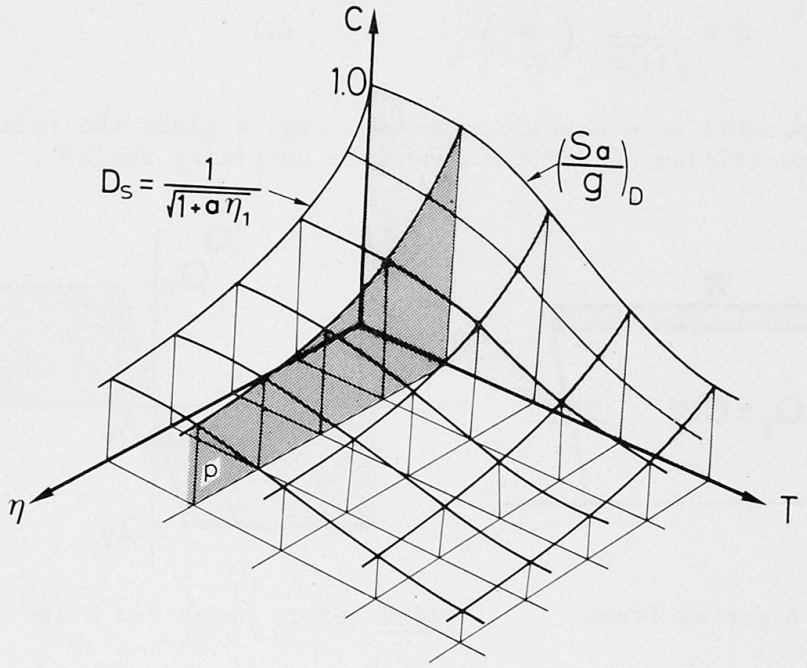


Fig.3 The same energy absorption level in the C-η-T space

2.3 Diagnosis and Therapy Standards of Existing Buildings

2.3.1 Diagnosis Standard

In parallel to the revision of the Building Code, a diagnosis and therapy standard for existing building was prepared. The existing buildings which were designed according to the old Building Code must be re-examined from the new standpoint of seismic design. The Diagnosis Standard consists of the regulations to calculate the existing yield shear coefficient and the regulations to evaluate the ductility ratio from the member sizes and shapes of inspected frames. In some existing building frames there can be seen the considerable shortage in strength and ductility. It is caused by a lack of consideration about the plastic behavior of structural members. Especially, the strength of joints is often insufficient to provide the required plastic deformation of connected members.

2.3.2 A Result of Damage Investigations

From the investigation results on the damaged steel buildings, the estimated maximum story shear coefficient α_u (the ultimate strength/the sustained weight) is plotted against the corresponding natural period of the building as shown in Fig. 4. The damage grades classified in Fig. 4 are:

(a) 0 = No damage, (b) I = No structural damage,
(c) II = Lightly damaged, (d) III = Moderately damaged,
(e) IV = Heavily damaged, (f) V = Collapse or Seriously damaged.

The details of the damage analysis are reported in the literature[1]. Hence, further discussion is eliminated here. However, it must be stressed that in the shorter natural period range, higher shear coefficients are required to reduce the structural damage.

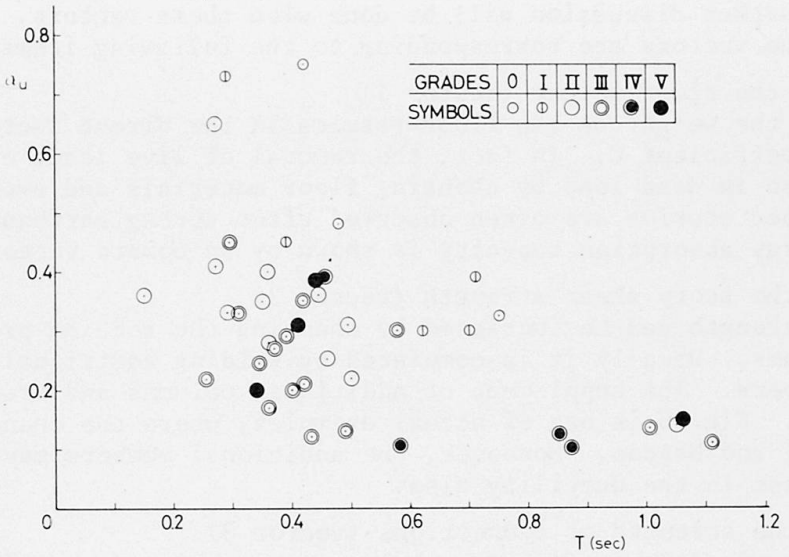


Fig.4 α_u-T relaton of damaged buldings

2.3.3. Strengthening-Therapy Standard

In the case that the insufficiency in the strength and the ductility exists, the story shear coefficient C is plotted under the roof in Fig. 3. By the energy absorption concept similar to that described in the preceding article, there can be considered another roof on which C, η and T are related at the same energy absorption level.

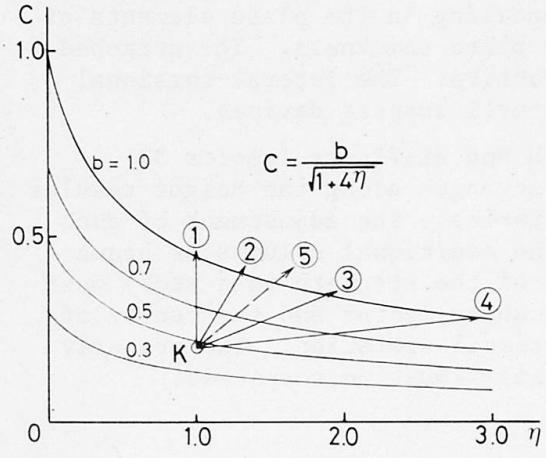


Fig.5 The same energy absorption levels in the C-η plane

Also the laminated roofs can exist for various levels of the energy absorption. Fig. 5 shows the section of such a structure made by the cutting plane denoted as p in Fig. 3. The solid curve represents the section of the roof required by the Building Code.

By help of the methods suggested in the Diagnosis Standard, the existing C and η of an inspected frame can be estimated. Such a set of (η, C) represents a point like K in Fig. 5. The curve of b = 0.5, passed near point K in the figure, shows energy absorption level considerably lower than the design level. Therefore, the point K must

be moved into the region above the solid curve by means of several methods for strengthening. The details of the proposed methods will be discussed in the next.

3. METHODS FOR STRENGTHENING AND REPAIRING

There are various methods for strengthening and repairing. Some of them are efficient in increasing the strength, namely, the yield shear coefficient C. Some are efficient in increasing the cumulative ductility ratio η or both of C and η. The amount of increase in C and η can be graphically expressed by a vector in the C-η-T space. The vectors in Fig. 5 represent the projections of such vectors. Regardless of the change in the natural periods of remedied structures, the further discussion will be done with these vectors. The numerals beside the vectors are corresponding to the following items.

(1) Reduction in the floor weight (vector 1)
The reduction in the weight on the floor results in the direct increase in the yield shear coefficient C. In fact, the removal of live loads on the floor, the decrease in dead load by changing floor materials and even the elimination of upper stories are often observed after strong earthquakes. The change of the energy absorption capacity is shown by an upward vector 1.

(2) Increase in the story shear strength (vector 2)
The story shear strength can be increased by changing the section properties of columns and beams. Usually it is completed by welding additional cover plates to the members. The supplement of additional columns and brace members is most effective. Fig. 6 is one of actual examples, where the channel members are added to steel rod braces. Moreover, the additional members may sometimes produce the increase in the ductility also.

(3) Increase in the strength of connections (vector 3)
The sufficient plastic deformation of members can be achieved, provided that the connections between the members are strong enough to maintain the member forces without fracture even in the plastic range. During the earthquake excitation the collapse of a structure is often initiated by the fracture of the connections due to careless design procedure and/or poor workmanship. Replacing and revising the weak connections result in the increase in the strength and the ductility.

(4) Improvement of member ductility (vector 4)
The member ductility is improved to avoid the local buckling and the lateral-torsional buckling in members. The local buckling in the plate elements of sections can be prevented by increasing the plate thickness. The attached coverplates as shown in Fig. 7 are also effective. The lateral-torsional buckling is prevented by properly placed lateral support devices.

(5) Adjustment of unbalance in the strength and stiffness (vector 5)
The excessive unbalance in the story shear strength along the height results in the damage concentration to the weaker stories. The adjustment of such an unbalance can be attained by supplying the additional columns or brace members. However, the excessive supplement of the strength to a story must be avoided. Disagreement between the story shear center and the center of sidesway stiffness in the plan causes a torsional vibration. The properly placed brace members can reduce such torsional vibration responses.

4. CONCLUDING REMARKS

In Japan a great number of steel buildings have been built for last several decades. The number increased suddenly in 1970's. It is obvious by the

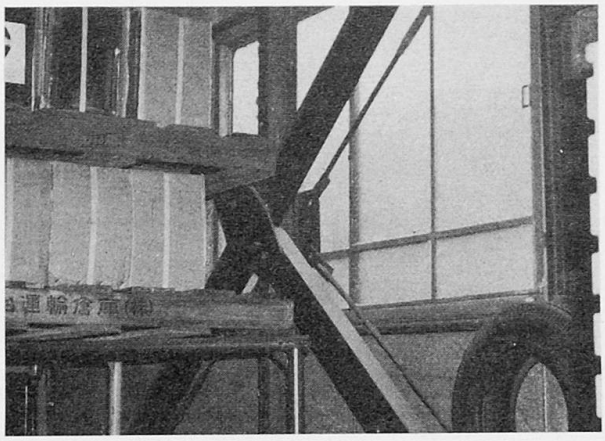


Fig.6 Channel braces added

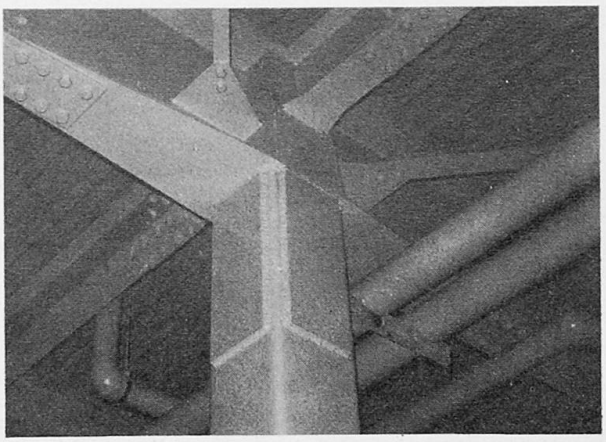
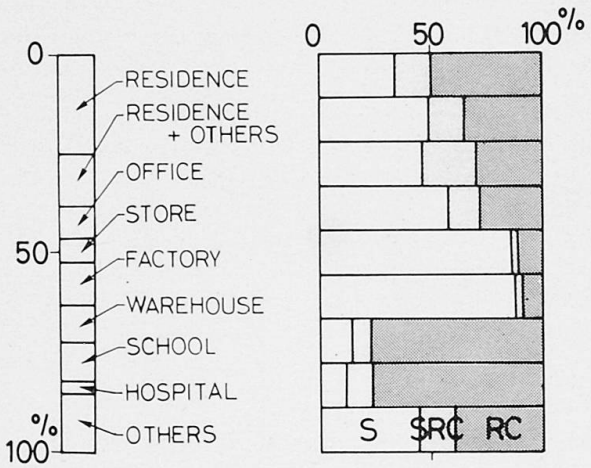


Fig.7 Coverplates welded

statistics of the annual total of newly constructed floor area in 1979 that steel structures are widely used for factories, warehouses and stores as shown in Fig. 8. Moreover, low rise buildings are often made of steel frames as shown in Fig. 9. Therefore, the inspection of earthquake resistance capacity of existing buildings and the strengthening become very important. In fact a municipal government set force their plans for strengthening houses along the Standard described in this paper. Some of them have been already examined and will be strengthened in the suitable methods.



S ; steel, SRC; mixed, RC; reinforced concrete

Fig.8 Usage of three types of structures (right) in different types of buildings (left)

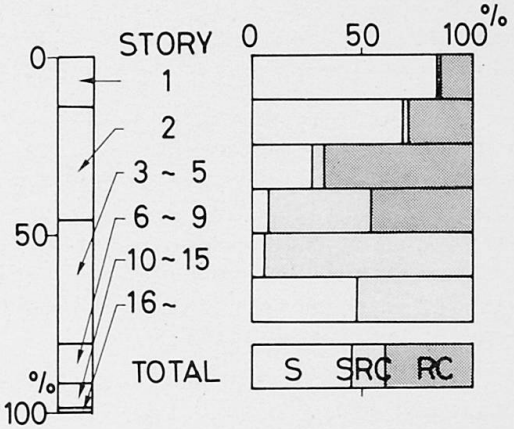


Fig.9 Usage of three types of structures (right) in different story buildings (left)

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Repair Works on the Main Pavilion at Tohdaiji Temple

Restauration du temple de Tohdaiji, Japon

Reparaturarbeiten am Hauptgebäude des Tohdaiji-Tempels

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SUMMARY

The world's largest timber building, located at the Tohdaiji temple in Nara, Japan, underwent large scale repairs in the period 1905 to 1915 and again in the 1970s. The first major repair work might be especially interesting to structural and architectural engineers from the view-point of strengthening of building structures, because there have been a number of new ideas and practical applications used in the strengthening of decayed structural members and the joints of the building.

RESUME

Le pavillon central du temple de Tohdaiji, situé à Nara au Japon, est la plus grande construction en bois dans le monde. Il a été restauré dans son ensemble durant la période 1905-1915 et à nouveau entre 1970-1980. Cette première réalisation de la restauration en grande dimension pourrait être spécialement intéressante pour les ingénieurs du point de vue du renforcement des structures de bâtiments. De nombreuses idées nouvelles et des applications pratiques ont été réalisées pour le renforcement d'éléments structuraux en piteux état.

ZUSAMMENFASSUNG

Das grösste Holzgebäude der Welt ist das Hauptgebäude des Tohdaiji-Tempels in Nara, Japan. Es wurde während eines längeren Zeitabschnittes, von 1905 bis 1915, und erneut in den 70er Jahren repariert. Für Bauingenieure dürfte die erste grosse Reparatur-die Verstärkung der Gebäudestruktur - besonders interessant sein. Hier kam eine Anzahl neuer Ideen und praktischer Verfahren für die Instandstellung vermoderter Bauteile und Verbindungsstellen zur Anwendung

1. INTRODUCTORY REVIEW OF THE PAVILION

The Tohdaiji temple was constructed for the first time in 751 A.D., in the middle of the so-called "Nara" period (710-784 A.D.). Facilities for the temple such as the main pavilion, the lecture hall, the dinning hall, priests' residences, the grand gate, the inner gates, corridors, the bell towers, etc., were recorded to have been constructed during the decade following the year of 751.

The records have shown us that the original pavilion building has covered an area spanning 97.2 m in the east-west direction, and 61.8 m in the north-south direction. The height of the main building was said to be 47.5 m above the stone platform which alone was 2.1 m higher than the ground level. The diameters of the columns of the building were 1.1 m at the bottom and about 0.9 m at the top.

The original buildings of the Tohdaiji temple were burnt in 1180 A.D. due to a battle fire. Later on, the buildings were reconstructed, but they were again burnt out in 1567 due to the same reason. The secondly reconstructed main pavilion, in the period starting from the end of the 17th century to the early 18th century (1708 A.D.), is the one that we can see at the present time. In the event of this reconstruction, the scale of the building was considerably diminished. Nevertheless, the main pavilion is still the world's largest timber structure that we can ever see in the 20th century.

Japanese wooden buildings, particularly large-size buildings such as shrines and temples are considered to require roof replacement once in 50-70 years and overhauling for repairs once in 200-300 years, although there are some differences in the intervals depending on roofing materials and types of structure.

Buildings such as shrines and temples have long overhanging eaves, and the weight of the roofs with heavy tiles is transmitted to the pillars of the buildings, so that the heavy load is borne by various kinds of structural members. Timber, which is the main structural material of these buildings, gradually changes its shape and dimensions, is bent and twisted and becomes thinner in a long time, mainly due to the following two reasons.

One of the reasons is that, because of the large size of such buildings, working stress on structural members is large and therefore, in a long time, deformations grow gradually, leading to the so-called creep phenomenon.

The other is the deterioration of wood which proceeds both interiory and externally caused by the environment surrounding the buildings, such as the penetration of rain water and dew, the repeated shrinking and expansion of wood by the changes in temperature and humidity, the weathering of the surface of wood caused by the dirt blown to them by wind, damage done by insects and termites, propagation of bacteria and various types of polution by birds and animals.

Wear of and damage to rafters, extended beams, columns and beams, particularly at their cut ends, have been observed in almost all cases of overhauling for repairs of wooden buildings so far carried out in Japan. As it is required to restore these damaged parts to their original state, a number of specific methods of reinforcement have been suggested and executed.

2.LARGE-SCALE REPAIR ON THE MAIN PAVILION CARRIED OUT IN THE PERIOD OF 1905-15

About 80 years ago, it was observed that the roof of the pavilion hang, its eaves had become wavy, and the large beams inside sagged down more than 30 cm. The reinforcing posts set up in the four corners of the pavilion to support the upper and lower roofs, barely prevented the building from collapse, and structurally the entire pavilion was in a very dangerous state. (Fig.1)

The so-called great repair work of the Meiji era (overhauling for repairs) on the main pavilion was carried out from 1905 through 1915. For this purpose,

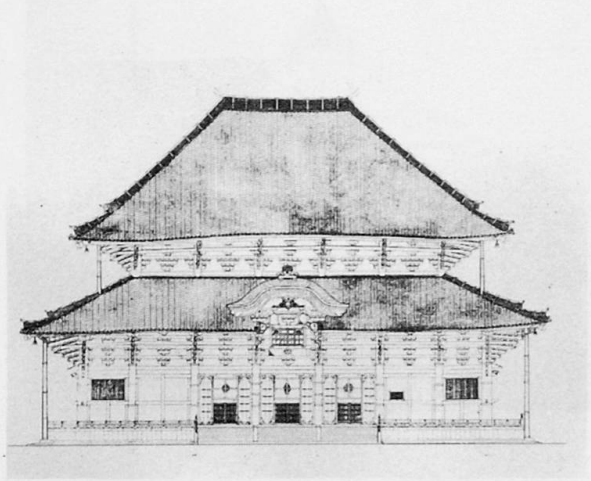


Fig.1 Front elevation in 1905

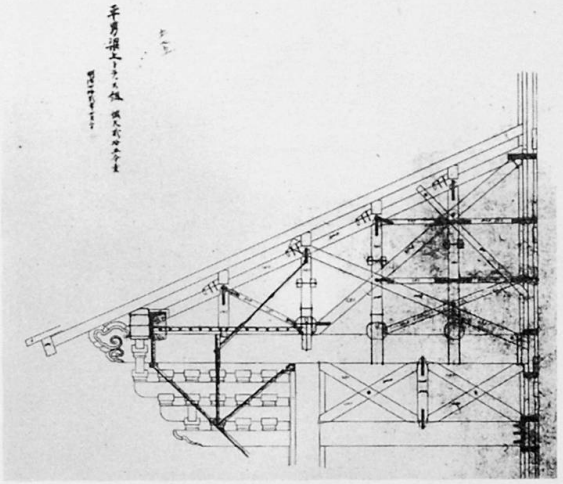


Fig.2 Reinforcement with steel

construction techniques and materials were actively introduced into Japan from European countries, marking an epoch in the overhauling and repairing of wooden buildings. This repair work was characterized by the following features.

1. Tiles on the ridges of the roofs were fixed with mortar using Portland cement which was imported into Japan for the first time in the 1900s.

2. Flat steel bars were used for the protection and preservation of the square framings of eaves and brackets. (Fig.2)

3. Rafters and extended beams, which are structural members to support the upper and lower roofs, were reinforced by using steel shapes and thick steel plates.

4. Steel frames made of rolled I-beams and channels, together with plates and angles, were embedded in the central parts of the cross-sections of wooden columns to enable them to withstand the bending moments on the columns. (Fig.3)

5. Hot-formed rivets were used to fasten the steel materials.

6. Double Warren steel trusses were provided under the existing huge timber beams in order to strengthen the frame for the large roofs and to enable them to

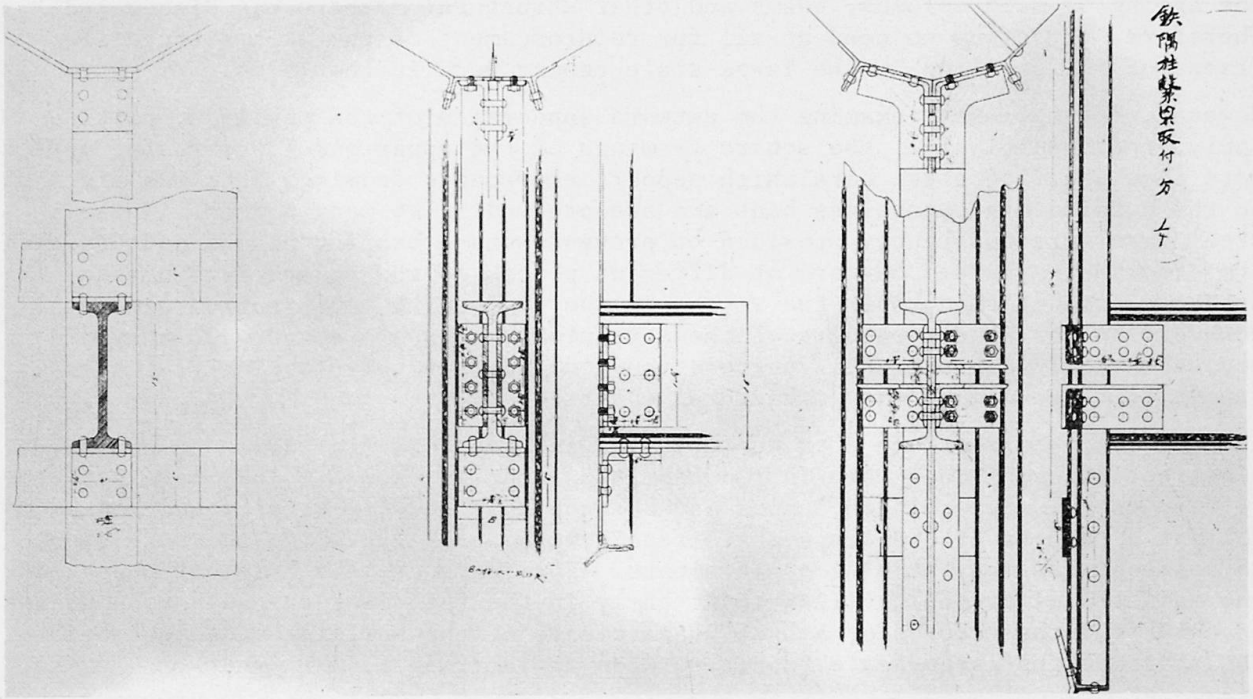


Fig.3 Details of the reinforcement of the wooden columns

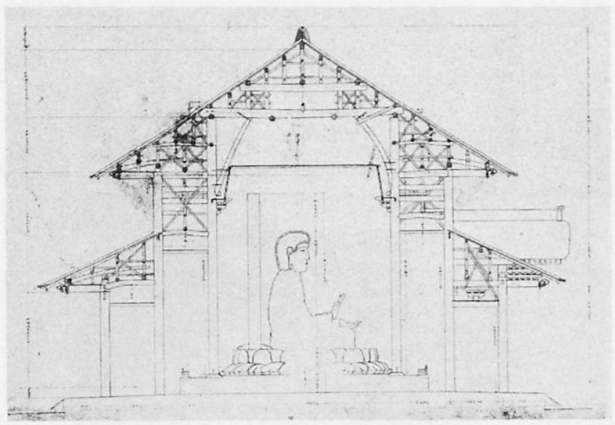


Fig.4 Deformed structural members

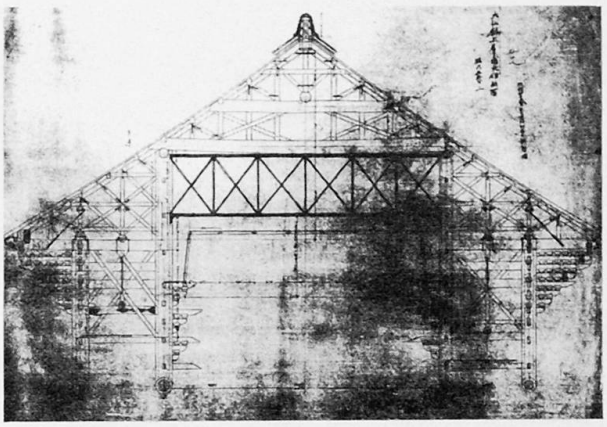


Fig.5 Double Warren truss inserted

bear the loads of the roofs, and further, the ceiling of the main pavilion was hung from these trusses. (Figs.4 and 5)

7. Wooden X-bracings were provided to the hidden parts of the building to reinforce the structural assembly, and the *"Kasugai"* clamps of the traditional Japanese style, bolts and nuts, and other steel materials were used in many places.

Timber trusses using a large number of bolts, nuts and other metal pieces that utilized the so-called new techniques in timber constructions developped in Germany and Britain in the beginning of the 20th century, were employed extensively for the wooden frames, for the main beams as well as the sloping beams (gathered beams) at the ridges of the pavilion.

It must be said that the large-scale repair work on the pavilion of Tohdaiji temple was successful in sufficiently strengthening this gigantic wooden building structurally. This repair work made it possible to remove the roof-supporting posts in the four corners of the building. Since then, the pavilion has withstood exposure to rain and snow for more than 70 years without its outward appearance being seriously damaged. When the author investigated the state of the structural frames of the pavilion 15 years ago, hardly any warping due to the ageing of main columns, beams and other structural members was discovered. Therefore, there was no need at all for reinforcement of the main structural frames of the pavilion in the large-scale repair work in the 1970s.

However, if we closely examine the outward appearance of the pavilion, particularly, the conditions of the square framings of the upper and lower roofs, we note that the flat steel bars which support the square framings from two sides on the outside are themselves bent and are protruding at many places. These steel bars were apparently provided to prevent square bearing blocks and bracket arms from being pushed outward at different places as the square framings and brackets are deformed under the weight of the roofs with long protruding eaves. However, so far as the results of these reinforcements are concerned, they did not necessarily achieve their purpose, and today these ugly-looking flat bars are still there without any structural contribution.

In the early years of the 20th century, new informations and knowledge on the construction techniques were introduced into Japan in excess. They were certainly very useful, but certain losses and unfavourable results were caused unexpectedly as a results of the large-scale repair work on the pavilion in those days, in spite of its many shining achievements. For instance, the original shape of the main frames of the pavilion built early in the 18th century can now be guessed only from the records of actual measurements of the building made just before the start of the large-scale repair work on it in 1905.

3. STRUCTURAL DESIGN OF THE DOUBLE WARREN TRUSSES

The double Warren trusses were designed, either in 1908 or in 1909, by a Japanese engineer who was in charge of the preparatory measurements and structural design so as to strengthen the main frames of the pavilion for overhauling. A sketch of the trusses, with the span of about 23.6 m and with the height of 4.83 m, is shown in Fig.6. Each chord member of the truss is consisted of two channels assembled side by side, with their legs protruding outward. At the inside web surface of the steel channels finished with the anti-corrosive paint, we can recognize a mark of SHELTON STEEL, CO. Also, we can find the mark of LANARKSHIRE STEEL CO., LTD, SCOTLAND at the web surface of other channels being used for the reinforcement of the roof structures of the building.

The top of the channels of the upper chords of the double Warren trusses are tightened together with a cover plate, a half inch (about 13 mm) thick and of 19 inches (48.3 cm) in the breadth. At the bottom of the upper chord members and at the top as well as the bottom of the lower chord members, there are tie plates of 12 by 19 inches (30 x 48.3 cm) and a half inch in the thickness, spaced at a pitch ranging between 90 cm and one meter. As the diagonal and vertical members of the trusses, there are used various structural steel shapes of T-5" x 3" x $^1/_2$", T-4" x 3" x $^1/_2$" and L-4" x 3" x $^1/_2$", together with flat steel bars of 4" x $^1/_2$", depending upon the working stresses on the members as calculated.

It may be interesting for structural engineers to see how the working stress on the members of the statically indeterminate trusses under the given vertical loads was obtained in that period of 75 years ago. As shown in Fig.6, there are the results obtained from graphical solutions of a Maxwell diagram for two trusses illustrated in the middle of the figure. It should be noticed that these two are the statically determinate trusses, and that each of them was looking like an elementary system of the double Warren trusses to be solved.

The result of the graphical solutions shows us that the Japanese engineer must have applied design vertical loads on both the elementary systems. After comparing the axial forces calculated for the members of the two statically determinate trusses with each other, he has adopted the larger value in either one of them as the stress for choosing cross-sections of the truss members.

It is well known nowadays that a high-speed electronic computer is useful to solve this type of statically indeterminate structures by basing upon the matrix methods. However, in the old days, the statically indeterminate structures consisted of the material which follows Hooke's law were treated in consideration of the strain energy, the principle of least work and the theorem of Castigliano, provided that the small displacements due to deformation can be neglected in discussing the action of the forces.

In a textbook of the strength of materials [1], there is a description that the principle of least work was stated first by F. Menabrea in his article, "Nouveau principe sur la distribution de tensions dans le systèmes élastiques", published in 1858, and that a complete proof of the principle was given by Castigliano, who

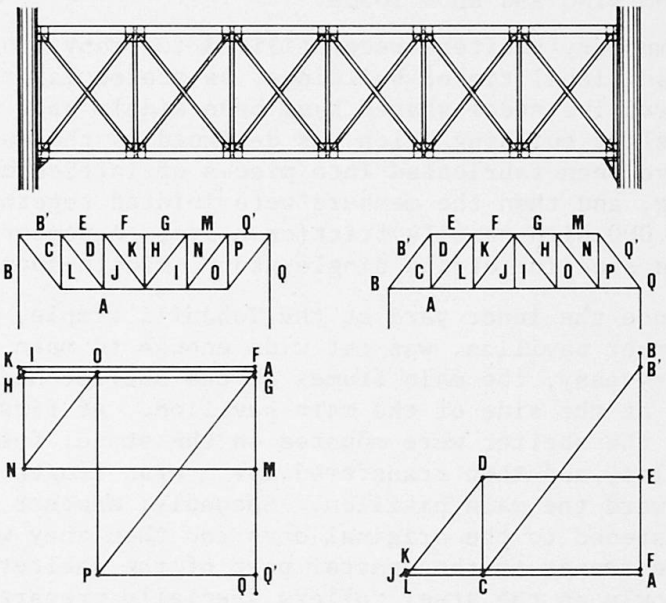


Fig.6 The truss and the results of solution

made this principle the fundamental methods of solution of statically indeterminate systems in the period of 1875-79. Also, the textbook tells us that "for an English translation of Castigliano's work see E.S. Andrews, *Elastic Stresses in Structures*, London, 1919."

If we think of the fact that in the early 1900s there was very little information in Japan on the Italian language papers in this specific field of building technology, we may conclude that the Japanese engineer who designed the double Warren trusses was not familiar with the Castigliano's work when he has engaged himself in the grand repair work of the pavilion of Tohdaiji temple. Of course, the introduction of the useful slope-deflection method [2] into Japan was a couple of years later than the grand repair work.

Thus, we can say now that the engineer has tried his own approach of obtaining the approximate values of the working stress on his statically indeterminate structure, without consulting the theories of structures being developped in the European countries at that time. Nevertheless, the author has proved a number of years ago that his results were not inadequate but his values were different from the exact values by only 10.7 % toward the safe side.

4. LARGE-SCALE REPAIR ON THE MAIN PAVILION CARRIED OUT IN THE 1970S

In 1969, the Tohdaiji temple and the Cultural Properties Protection Commission in the Nara Prefectural Office have established a joint committee, in which the author has been involved, for the survey and inspection of the roofs as well as the frame of the main pavilion. The investigation was carried out for the successive two years, and the scope and orientations of the renovation projects was decided. The following directions then have been drawn from the results of the preliminary survey:

1. Columns, beams and girders of the main pavilion are still in good condition, therefore, repair is not necessary but,

2. All the tiles shall be taken out completely for further inspection, and then the shingles and rafters shall be examined and repaired if corrosions will be observed, and

3. In order to make the renovation possible and easier, a new temporary shelter covering the pavilion will be indispensable. This shelter will render the renovation possible and safe even during rainy days and will protect the pavilion from wind and snow loads.

Temporary shelters were utilized for many projects of repairing the large-scale traditional timber buildings, as stated later. This time, high-tensile, anti-corrosive steel shapes have been widely used as the principal material of the shelter building which was designed by the author. The wide flange steel shapes have been fabricated into pieces of lattice members of the frames by shop welding, and then the members were jointed together at site by using a total of about 33,000 high-tensile friction bolts, A number of track cranes were employed for the erection of the single-story, single-spanned steel frames. (Fig.7)

Since the inner yard at the Tohdaiji temple, surrounding the world's largest timber pavilion, was not wide enough to make the erection of the steel shelter very easy, the main frames of the shelter had to be assembled on a stage prepared at the side of the main pavilion. At first, three frames of the central part of the shelter were mounted on the stage, fastened each other with the friction bolts, and then transfered for a span length of 5.4 m in the east-west direction toward the main pavilion. Secondly, another frame was erected on the stage and fastened to the original ones and then they were moved again for 5.4 m. Thus, the frames of the central part of the shelter building have been moved very slowly on the steel rollers specially prepared. After the frames were settled down at the designated position, they were anchored to the foundation, and then

the frames at both sides of the shelter were erected.

Thus, the main pavilion was covered by the steel shelter of about 86 m in the east-west direction, and of about 79 m in the north-south direction measuring from the center lines of its corner columns. The exterior columns were spaced at 5.4 m around the perimeter of the building. The height of the shelter was about 55 m. The shelter had 16 spans in the longer direction whereas 15 spans along the shorter direction.

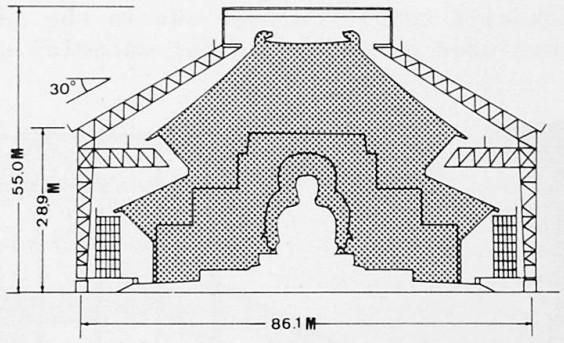


Fig.7 The steel shelter used in 1974-78

The roof construction of the shelter consisted of a layer of very thin steel plates, corrugated and painted on both surfaces. The corrugated plates were further constrained by flat steel bars so as to prevent them from being blown off. Two inch (5 cm) deep steel purlins of cold-rolled light-gauge shapes were placed at a pitch of approximately 60 cm. The purlins were spanning about 1.8 m between the supporting beams.

The side walls of the shelter were covered with cotton sheets and metal meshes. The wind resisting system consisted of rigidly connected frames interacting with diagonal braces. Virtually, every available column was thus mobilized to contribute to the lateral-force-resisting system.

An extremely comprehensive wind investigation, involving meteorological studies and wind tunnel testing, was carried out. The latter was conducted at the Disaster Prevention Research Institute, Kyoto University. A pressure model was used to define the pressures acting on the faces of the shelter. The wind climate of the building location was determined in terms of wind frequencies by direction and speed as well as velocity variations with height together with the turbulence structure.

In addition to the dead and live loads, as well as snow and wind loads, the shelter has carried moving loads of travelling cranes. Their capacity of two tons each carried the roof tiles and rafters of the main pavilion. One must be surprised to hear that the size of a wooden rafter is of about 30 x 30 cm in the cross-section and it is approximately 10 m long, weighing roughly 450 kilograms. The travelling cranes were supported by the cantilever trusses in the shelter, the tip of which were surrounding the eaves of the upper roof of the main pavilion.

5. CONCLUSIONS

In doing repair work on traditional wooden buildings, it is necessary to preserve their original appearance as much as possible, and it is extremely effective for this purpose to utilize steel members as auxiliary ones, wherever necessary, in order to remove structural defects due to ageing. However, there is a danger that the timber in contact with corroded steel members is deteriorated by rust so that it is indispensable to pay sufficient attention to prevent this. It happens very often that modern rolled steel nails swell as they get rusty and that cracks are caused in wooden members into which such nails were driven. In view of this, the use of forged nails made from iron sand by the traditional Japanese manufacturing method appears to be more suitable for the permanent preservation of wooden members.

Furthermore, there are many cases in which wooden buildings dismantled for repair are covered with temporary shelters. Fig.8 is the drawing of the temporary

shelter built up for the large-scale repair work on the main pavilion of the Tohdaiji temple carried out in the period of 1905-15, showing us that the logs were used as the principal material and that how the shelter was assembled.

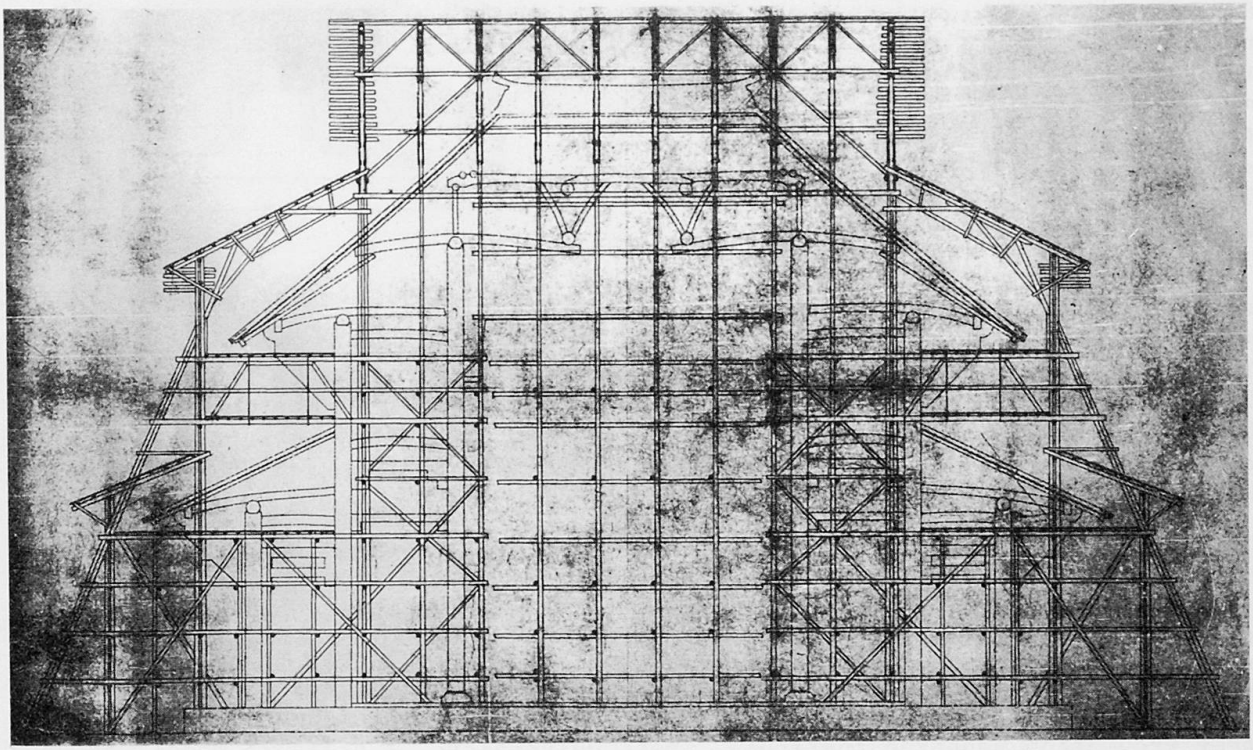


Fig.8 Section of the temporary shelter used for the project in 1905-15

In recent years, temporary shelters consisting of steel frames have been used extensively because of the shortage of timber resources for construction work and also of the regulations concerning the safety and hygiene of workers involved. In the latter case, it is easy to install overhead travelling cranes and other lifting mechanisms in the temporary shelter, which can serve not only to increase the safety of workers at the site but also to secure higher efficiency in handling the materials. We are very proud to say that the grand repair work on the main pavilion of the Tohdaiji temple has been completed satisfactorily in October 1980 without any accidents.

6. ACKNOWLEDGEMENT

The author wishes to express his gratitude to the engineers at the Cultural Properties Protection Commission in the Nara Prefectural Office for their support and cooperation.

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Strengthening of Wooden Ceiling Constructions

Renforcement de constructions avec des plafonds en bois

Verstärkung von Holzdeckenkonstruktionen

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Born 1923, graduated at the Technical UNI Bratislava, CSSR. Designed more than 1000 constructions of important buildings in steel, concrete and wood, and realised many reconstructions of historical objects. Specialist in hanging roofs.

SUMMARY

The loading capacity and the stiffness of wooden ceiling constructions in historical buildings is usually not sufficient for modern demands. A method for strengthening of such wooden ceilings is recommended by coupling the wooden beams with an additional reinforced-concrete slab over them. After this strengthening the loading capacity and the stiffness increases cca 4x.

RESUME

La limite de charge et la solidité de constructions avec des plafonds en bois, dans les bâtiments historiques, ne sont en général pas suffisantes pour les besoins modernes. Le rapport propose un procédé de couplage de poutres en bois avec une dalle en béton armé supplémentaire posée dessus. Par ce renforcement la limite de charge et la solidité est à peu près quadruplée.

ZUSAMMENFASSUNG

Die Tragfähigkeit und die Durchbiegungen von Holzdecken in historischen Gebäuden entsprechen heutzutage nicht den modernen Anforderungen. Eine neue Methode zur Verstärkung solcher Holzdecken durch Verbund der Holzbalken mit einer zusätzlich auf sie betonierten Betonplatte wird vorgestellt. Durch diese Verstärkung verbessert sich die Tragfähigkeit und die Durchbiegung um einen Faktor vier.

1. INTRODUCTION

Wooden ceilings in historical buildings have usually considerable elastic and permanent deflections, so that they are not sufficient for modern demands. A method for increasing the loading capacity and the stiffness of such ceilings is to change the wooden beams by other materials such as steel or reinforced-concrete constructions. It is a pitty that the wooden construction which served for several hundred years has to go lost. The change of ceiling construction is complicated and expensive.

In ČSSR we found a method for strengthening of such wooden ceilings by coupling the wooden beams with additional reinforced-concrete slab made over them. After this strengthening the loading capacity and the stiffness encreases cca 4 x .

2. TYPES OF WOODEN CEILING CONSTRUCTIONS

It is possible to strengthen several types of wooden ceilings. Fig. 1 and Fig. 2. Fig. 3 and 4.

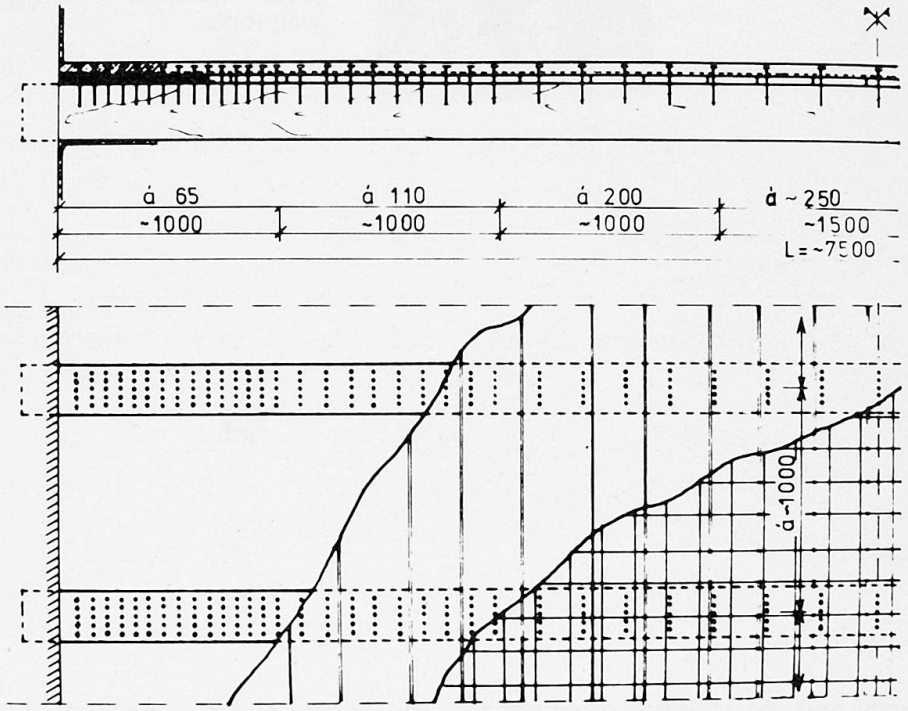


Fig. 1. Wooden beam ceiling construction with the beams in distances.

The figure 1 shows a wooden beam ceiling construction with the beams in distances of cca 1 m. Over the beams there is usually a wooden floor made of boards.

The figure 2 shows the cross section of such ceiling, after the strengthening.

In figure 3 are the wooden beams tightly together, and in figure 4 there is the same ceiling under strengthening operation. The surface of the old wooden ceiling gets coupling elements such as steel nails drived in distances fixed by statical analysis. The nails stick out of the surface cca 40 mm.

3. EXEMPLES OF STRENGTHENED BUILDINGS

The new strengthening method was used the first time for the restruction of a historical building "The good shepherd" in Bratislava, ČSSR in 1960. Since that time this house serves as a watch museum. Fig. 5.

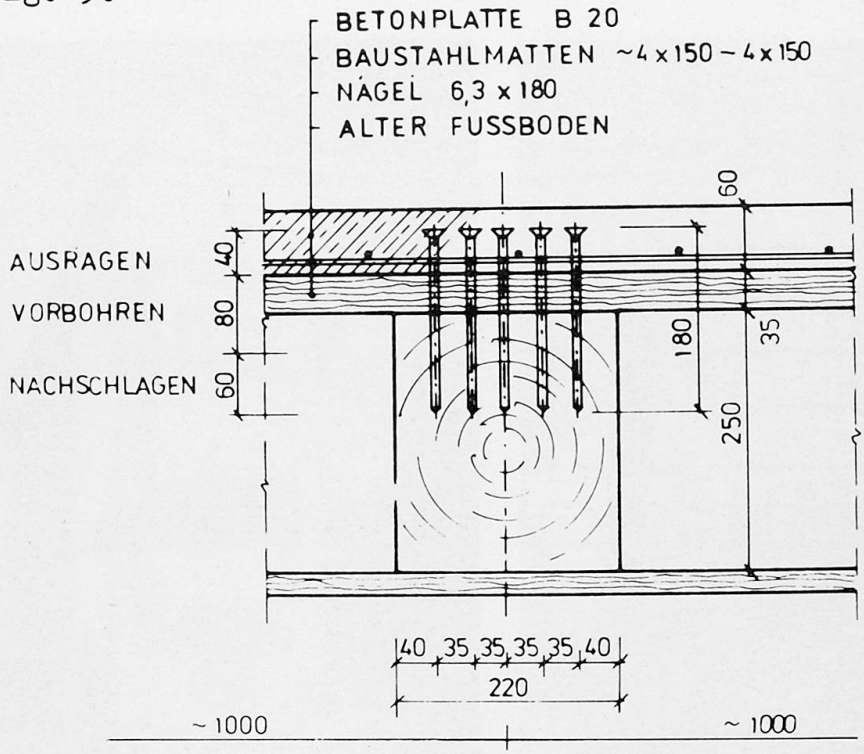


Fig. 2. The cross section of a beam ceiling after strengthening.

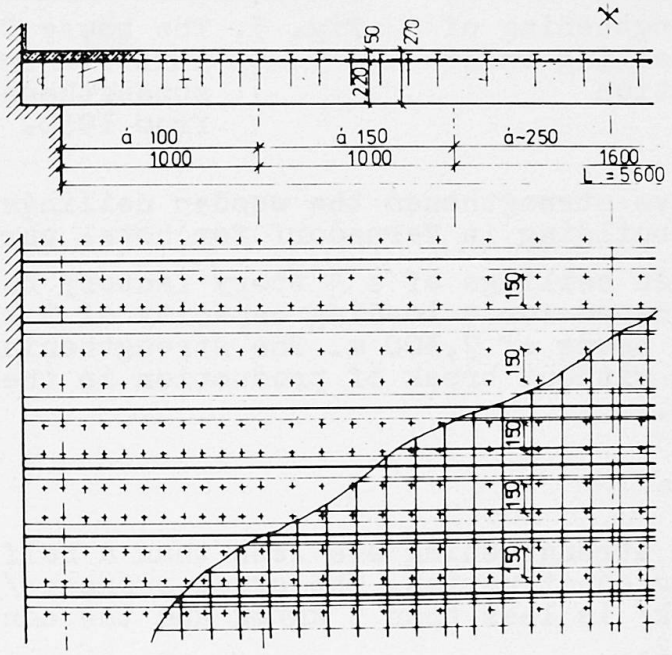


Fig. 3. The wooden beams tightly together - the ground plan.

An other wooden ceiling has been strengthened in 1965 fo the Slovak National Council in Bratislava. The loading capacity of the

old ceilings was not sufficient for assembly rooms with 5 kN/m2. The control measurements after the strengthening has shown that the stiffness and the loading capacity encreased cca 4 x.



Fig. 4. The strengthening of a wooden ceiling under construction



Fig. 5. The house "The GOOD SHEPHERD" in Bratislava with strengthened ceilings from 1960.

In 1968-69 we have strengthened the wooden ceilings of an old 4 story factory building in Varnsdorf for hotel purposes.

In 1972 the wooden ceilings of a 4 story factory building in Krnov were strengthened for a loading capacity of 5 kN/m2 and for a span of wooden beams of 7,300 m. The strengthening was made as a winter work without break of production in the other romms of this building.

4. ECONOMY

The costs of the strengthening are less than a half of the costs for a new ceiling construction. The steel economy /the nails and the steel network/ is less than 5 kg/m2 and the concrete economy is cca 0,06 m3/m2.

In addition to the strengthening we can book an other advantage in the increased protection of the wooden ceiling against fire from the upper surface of the floor.

5. THE STRENGTHENING METHOD

After cleaning of the ceiling surface and after the chemical protection of wood by penetration it is necessary to rough drill the holes for the nails in the upper surface of the wooden ceiling. The depth of the holes has to be more than a half lenght of the nails drived in.

For the rough drilling we use a drilling template made of a thin perforated steel sheet. Fig. 6. The perforation of the template is made in distances ordered by the statical analysis.



Fig. 6. Rough drilling with a drilling template

The nails are drived in by an electric percussion drill with a special inserted tool head in order to let the nails stick out of the surface with 40 mm. Fig. 4. Over the nails we lay down the wire net and the concrete slab can be made.

6. THE LOADING TESTS RESULTS

In order to control the loading capacity and the stiffness of the strengthened wooden ceiling in Krnov there was made a loading test with following results:

The span of the wooden ceiling ..... L = 7,300 m
The calculated load ..... q = 5,00 kN/m2
The maximum testing load ..... q' = 5,69 kN/m2
The tested deflection at q' = 5,69 kN/m2 ..... f' = 18 mm
" " " " q = 5,00 kN/m2 ..... f = 16 mm

$$f = 16 \text{ mm} = \frac{1}{455} L < \frac{1}{350} L$$

The calculated deflection of the wooden beams .. f" = 97 mm !

The stiffnes increased to ..... $n = 97 : 16 = 6{,}1$ x

The permanent deflection after the loading test $f_p = 2$ mm

7. THE LOADING TEST OF THE COUPLING EFFECT

In order to control the coupling effect between wood, nails and concrete slab we have made an other loading test. Fig. 7. The results of this test are in fig. 8 and the deformation of the nails can be seen on fig. 9.

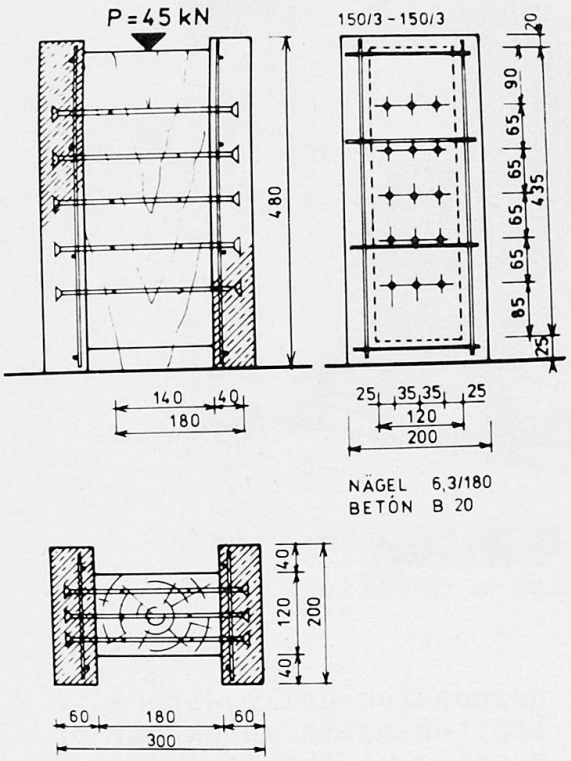


Fig. 7. The testing element for the coupling test

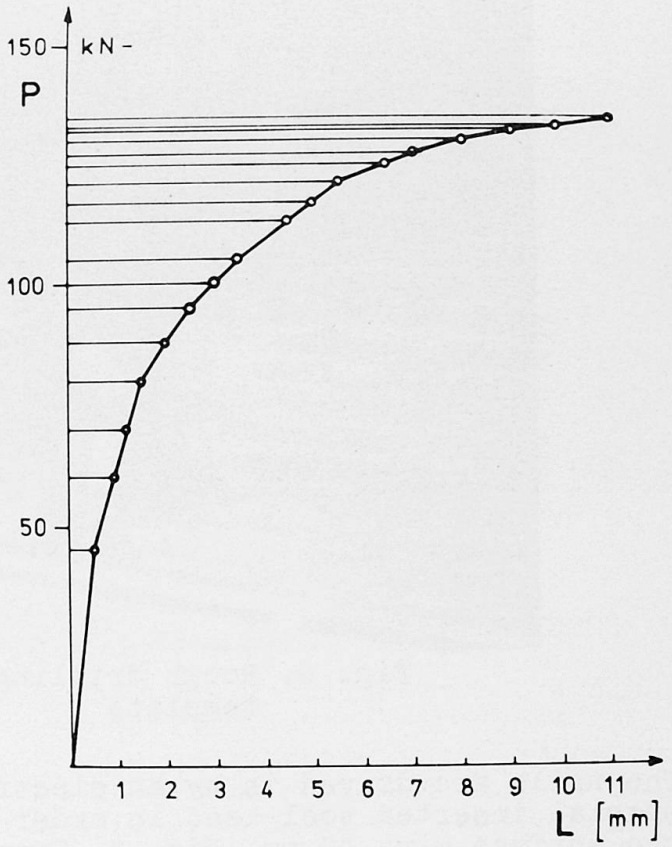


Fig. 8. The results of the coupling test

The calculated testing load of the testing element ... P = 45 kN
The deformation between wood and concrete slab ..... $\Delta L = 0{,}5$ mm
The collapse load ..... $P_{col} = 140$ kN
The safety against collapse : n = 140 : 45 ≐ 3,12 x

8. SAVING OF DAMAGED WOODEN BEAM PARTS

The possibility to help to damaged beam headings is shown in figure 10.

The rotten beams or the rotten parts of them are to be changed by new wood, after chemical conservation of the healthy parts of the wooden ceiling.

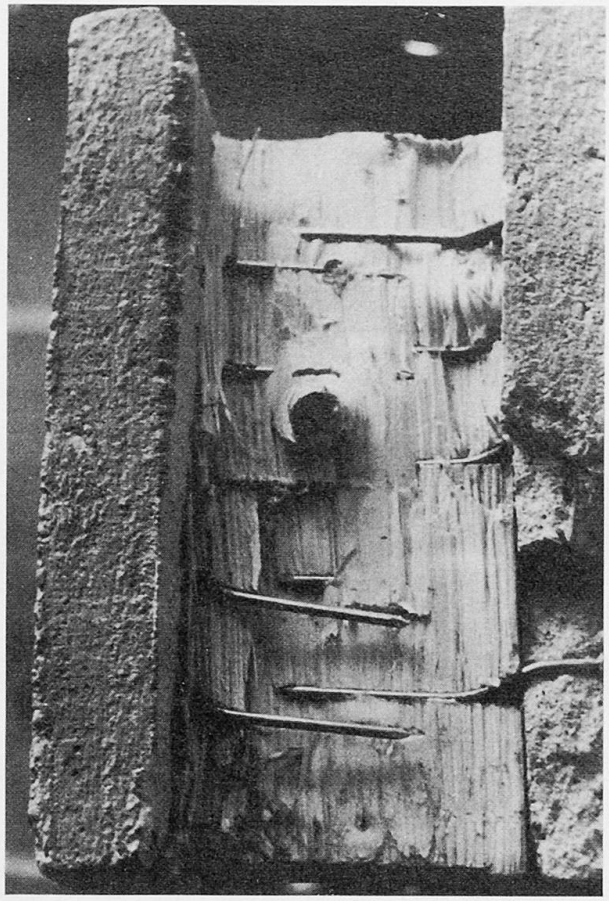


Fig. 9. Deformation of the nails after collapse

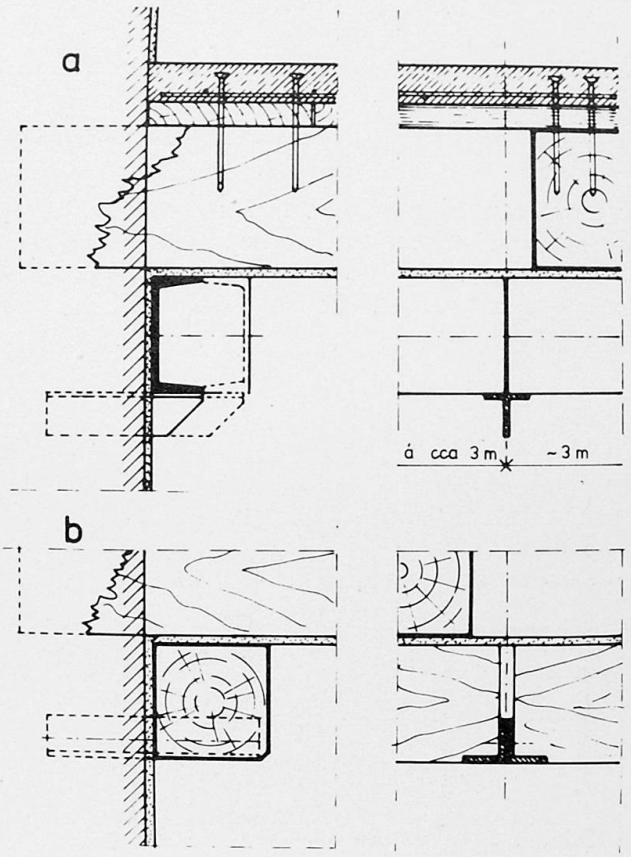


Fig. 10. Saving of damaged beam headings

9. CONCLUSION

There are many historical buildings in the world with wooden ceilings which are to be strengthened. The 23 years experience with our method of strengthening of such ceilings shows that it is a very usefull and economical method. The costs of the strengthening are less than the half of the costs of new ceiling constructions.

This method can be used for new wooden ceilings too.

We can offer our know-how to all people interested in such strengthening.

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Strengthening of Masonry Buildings in the United States

Renforcement des bâtiments en maçonnerie aux Etats-Unis

Verstärkung von Backsteinbauwerken in den Vereinigten Staaten

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Born in 1915. John B. Scalzi received his M.S. and Ph.D. in Civil Engineering from the Massachusetts Institute of Technology. Dr. Scalzi is Program Director in the Earthquake Engineering Program at the National Science Foundation. He has authored many papers and technical books on structural engineering.

SUMMARY

Research in the general field of Civil Engineering is supported principally by the National Science Foundation (NSF), an agency of the Federal Government. The broad fields of earthquake engineering research are described in general announcements of the NSF programs and budget descriptions. High among the priority topics listed in the Earthquake Engineering Program is the entire field of risk analysis, repair, strengthening, and evaluation of masonry structures with special emphasis on the seismic behaviour of unreinforced masonry buildings of clay brick and/or concrete block.

RESUME

La recherche dans le domaine général du génie civil est financée essentiellement par la «National Science Foundation» (NSF), un organisme du gouvernement fédéral. Les projets de recherche dans le génie sismique sont décrits de façon générale dans les programmes NSF et dans les budgets correspondants. Parmi les sujets prioritaires mentionnés dans le programme du génie sismique, il y a l'analyse de risque, la réparation, le renforcement et l'inspection des structures en maçonnerie; un accent particulier est mis sur le comportement aux séismes de bâtiments non renforcés en maçonnerie de briques en terre cuite ou de blocs en béton.

ZUSAMMENFASSUNG

Die allgemeine Forschung auf dem Gebiet des Bauingenieurwesens wird hauptsächlich von der «National Science Foundation» (NSF) unterstützt, einer Amtsstelle der Bundesbehörden. Die weiten Zweige der von Forschern betriebenen Erdbebenforschung werden in Mitteilungen der NSF-Programme und -Budgetbeschreibungen dargelegt. An vorderer Stelle unter den aufgeführten Themen des Programms für Erdbebenwesen steht das umfassende Gebiet der Risikoanalyse, Reparatur, Verstärkung und Schätzung von Backsteintragwerken mit besonderer Betonung auf das Erdbebenverhalten von unverstärkten Backsteinbauwerken aus Tonziegeln und/oder Betonblöcken.

Research in the general field of Civil Engineering is supported principally by the National Science Foundation (NSF), an agency of the Federal Government. The recipients of the funding support are the university professors and researchers, although we do support some non-profit organizations, such as: the National Academies of Science and Engineering; the Earthquake Engineering Research Institute-the professional society for the earthquake community; the Applied Technology Council-the research organization of the Structural Engineers Association of California; and private professional organizations, such as consulting engineering firms.

The procedure by which the research is supported is the peer review process. This process is based on unsolicited research proposals received from investigators noted above in response to the general objectives of the programs announced by the National Science Foundation.

The proposal is reviewed by experts in the same field as the proposal, and a decision to award or decline is made by the NSF Program Director based mostly on the nature of the review comments. Thus, it is a random process for the selection of specific projects to be supported in any NSF designated discipline, such as the Earthquake Hazard Mitigation Program, in the Division of Civil and Environmental Engineering.

The broad objectives of the Earthquake Engineering Program are to support research projects directed at developing the basic technical knowledge required to mitigate the losses from damaging earthquakes, and to disseminate the acquired information to decision-makers; such as consulting engineers, architects, planners, local, state, and Federal regulatory bodies and agencies, and other related financial organizations and business institutions. The broad fields of earthquake engineering research to be pursued by the researchers are described in general announcements of the NSF programs and budget descriptions.

High among the priority topics listed in the Earthquake Engineering Program is the entire field of risk analysis, repair, strengthening, and evaluation of masonry structures with special emphasis on the seismic behavior of unreinforced masonry buildings of clay brick and/or concrete block.

The greatest risk for loss of lives and property is in the large number of unreinforced masonry buildings which exist in the United States since colonial times. These buildings constitute the largest inventory of hazardous buildings in all regions of the country including the moderate to high risk earthquake zones. As a result, there are many different types of buildings with masonry construction which means that the number of research projects which are required are endless. Because of the unique nature of masonry construction, very little research on their seismic performance had been conducted until the San Fernando earthquake of 1971 brought it to the attention of the earthquake community and the

Federal Government because of the collapse of many masonry buildings. Practically all of the existing buildings were constructed before seismic regulations were instituted and/or enforced. Therefore, in order to strengthen these buildings, it is necessary to develop the appropriate research information which will evaluate the inherent strength of the structure and then to verify various methods for the repair and strengthening of the several structural components such as walls, roofs, floors, and connections of all assemblies. These topics become the basis for the research projects supported by NSF.

One of the important features of the NSF earthquake program is the dissemination of the research results. An effective method has been the support of a North American Masonry Conference periodically (four year intervals) by which researchers present the results of their work to a broad audience and exchange data and information on their projects. The First Conference (1) was held at the University of Colorado, Boulder, Colorado, in 1978, anad the Second one (2) was held at the campus of the University of Maryland at College Park, Maryland.

These Conferences produce a Proceedings of the papers presented and those accepted for publication in order to contain in one volume all the research data which was developed for the time period between Conferences.

By this method, a researcher or designer can easily refer to the Conference Proceedings to learn what is being done and what has been accomplished in the total research field of masonry analysis and construction.

A brief description of the current research projects is presented below with a statement of the objectives and the status of the project at this time.

3. METHODOLOGY FOR MITIGATION OF SEISMIC HAZARDS IN EXISTING UNREINFORCED MASONRY BUILDINGS

This project will develop a methodology for the mitigation of seismic hazards in existing unreinforced masonry buildings to be applied throughout the entire United States because of the various degrees of vulnerability to earthquake damage. The methodology will provide analysis techniques and procedures to determine: degree of seismic hazard, mechanical properties of unreinforced masonry materials and components, requirements for hazard mitigation, and cost-effective methods for repair and strengthening these types of buildings.

The research consists of a combined analytical and experimental investigation which includes: dynamic testing of full scale walls subjected to out-of-plane motions typical of those which would occur in a building at the top and bottom of the wall; and dynamic and static testing of full scale horizontal diaphragms subjected to in-plane loadings. The walls varied from 10 to 16 feet (3.0 to 4.9 m) with height-to-thickness ratios between 14 and 25. The walls were of 3 wythes of common clay brick, grouted and ungrouted concrete block, and grouted clay block. The diaphragms were 20 x 16 feet (6.1 by 19.3 m) and included wood, metal deck, and concrete filled metal deck systems.

The objective of these tests is to correlate the diaphragm response to the resistance against collapse of unreinforced masonry walls subjected to

out-of-plane seismic motions. Simplified analytical methods will be developed from this research. As a matter of interest, the results from this investigation contributed significantly to the development of a city ordinance to reduce the earthquake hazard of existing buildings in the city of Los Angeles, California in 1981.

4. BASIC PROPERTIES OF CLAY UNIT MASONRY

The objective of this research is to investigate, in-depth, the factors which control the strength of brick masonry in compression under static and dynamic loading environments. As of April 1, 1983, the following tasks have been accomplished:

1) Triaxial tests of several mortar mixes to develop mortar strength, modulus of elasticity and poisson's ratio for use in a masonry failure analytical model.

2) An apparatus for tensile testing of whole bricks and for use in combined tensile-compressive tests of bricks has been developed. This data is also required for the failure model.

3) A method to reduce the lateral confining friction force exerted on compression specimens has been developed. This is used in compression tests of brick and brick prisms to obtain "unconfined" compressive strengths for use in the failure model. Compression tests of strain-gauged bricks have demonstrated the effectiveness of the interface friction reduction method.

Future work includes experiments of prisms made with different brick-mortar combinations to enable comparisons between theoretical predictions based on the failure model and actual performance, and also includes cyclic compression tests of prisms at rates simulating seismic loadings on the compression of full-scale masonry.

5. AN INVESTIGATION INTO NONDESTRUCTIVE EVALUATION OF MASONRY STRUCTURES

Six nondestructive evaluation (NDE) methods were investigated to assess their potential use for strength and condition evaluation of masonry using unmodified commercially available equipment. The methods were: vibration, rebound hammer, penetration, ultrasonic pulse velocity, mechanical pulse velocity, and acoustic-mechanical pulse. The candidate methods were applied to two-wythe cantilever wall specimens. Companion small-scale specimens, specimens removed from the walls subsequent to the NDE tests, and in-the-wall specimens were tested to destruction to provide compression, shear and flexural strength data for correlation studies. Flaws, in the form of delaminated bed joints were created in the wall specimens to determine the ability of each NDE method to detect defects in masonry.

Measurements from each method except acoustic-mechanical pulse were compared to strength properties as established by destructive tests using bivariate linear regression analyses. Results of the analyses indicate that strength properties of the masonry tested could be generally estimated by some of the NDE methods considered with the highest correlation coefficients in the 0.8 to 0.9 range. Investigation of the

acoustic-mechanical pulse method was very limited, but indicated that consistent measurements could be obained and that flaws could be detected.

It was concluded that NDE methods offer a means of relative quality assessment and flaw detection, and with present procedures and equipment, a limited quantitive evaluation of the type of masonry tested. It was also concluded that with refinements of procedure and some modifications to equipment, the efficacy of most of the methods evaluated would be enhanced. Further investigation of the acoustic-mechanical pulse method appears warranted based upon the very limited experience gained and that further studies are needed to assess the utility of NDE for other types of masonry.

6. CYCLIC RESPONSE OF MASONRY ANCHOR BOLTS

This experimental research project is nearing completion in which the strength of "J" bolts embedded in brick and concrete block masonry is determined. The bolts investigated varied in diameter from 3/8" to 1-1/4" and were tested in shear, axial, and combined shear and axial directions. The tests were repeated with monotonic and cyclic loading.

Results indicate that strengths increase with bolt diameter up to the 3/4" size, with little additional benefit from larger sizes. Current code allowables appear to have a safety factor of approximately ten. The results of this research are extremely important to the designers, engineers, and code bodies because very little reliable data was available when most of the currently used values for bolt strengths were established.

7. SEISMIC RESPONSE OF COMPOSITE MASONRY IN NEW AND EXISTING STRUCTURES

This research project assesses the behavior of new and existing composite masonry buildings subjected to static and cyclic loadings using an analytical and experimental approach.

The analytical phase includes the development of a finite element model in the form of a "Superelement" which may be used in the analysis of large and complex composite walls. The "Superelement" includes both wythes and the collar joint, and considers shear failure at the collar joint as well as in-plane failure of each wythe. The emphasis is on the development of criteria for delamination of the interface due to shear stresses caused by the superimposed loads, thermal effects, expansion and shrinkage due to moisture, and creep.

The finite element model for the interface shear strength will be verified by means of static and cyclic tests. The values of constants assumed in the development of the finite element model concerning shear failure will be modified, if necessary, to achieve a better correlation between the analytical and experimental results. The results of this research project will develop criteria for the design of composite masonry walls subjected to in-plane loads.

During the first year of this project, a composite element that is capable of transferring shear under linear elastic conditions was developed. Currently, thermal effects, moisture expansion and contraction, and creep phenomenon are being incorporated into the model. Sixty composite wall specimens, each 16 inches square, constructed with 3/8" thick slushed

mortar collar joint have been tested under static and cyclic loads. Variables in these tests were mortar, reinforcement, type of loading, and absorption in brick and block. Specimens with 2 in. collar joints are currently under preparation for continuation of the experimental phase.

8. STRENGTHENING OF BRICK MASONRY WALLS WITH SHOTCRETE

The objectives of this experimental research project on strengthening of existing unreinforced brick walls are four-fold:

(1) Investigate the bond strength between shotcrete and old, molded clay brick and the effect of wetting the brick or coating it with epoxy prior to application of shotcrete; (2) determine the interface shear capacity of the brick-shotcrete composite; (3) determine the need for and influence of steel dowels which anchor two wythes of brick to a surface layer of shotcrete; (4) investigate the possible use of a thick, lightly reinforced layer of shotcrete on one or both sides of a brick wall for seismic strengthening in regions of moderate earthquake risk.

One or two wythe brick panels measuring 1m x 1m and 1.3m x 1.3m were reinforced with a 9cm layer of shotcrete on one side and were tested to determine the overall behavior of the brick-shotcrete system and to determine whether the bond between the brick and shotcrete is sufficient to develop full composite behavior for in-plane loads. The shotcrete (f' = 47.9 MPa) was reinforced with a welded wire fabric, with a reinforcement ratio of 0.0018 in each direction. Some panels were dry prior to shotcreting, others were thoroughly wetted, while some were coated with epoxy. Wall panels were tested under a reversed cycle, in-plane load across the diagonals. Results indicate that similar panels with different surface conditions developed the same strengths but that the wetted and epoxy coated brick panels exhibited greater inelastic deflection capacity and, thus, greater seismic resistance. The shotcreted panels averaged more than thirty times stronger than unstrengthened brick panels. Unreinforced wall panels could not sustain reversed loading.

Reinforcing bar dowels of size number three (10mm diameter) were epoxy bonded into two wythe panels prior to shotcreting. Panels without such dowels demonstrated the same strength as those with dowels but at large reversed cycle deflections. Panels with dowels remained intact while those without dowels delaminated. Some panels were strengthened with a 3.8cm layer of shotcrete reinforced with an expanded metal lath (0.0025 reinforcement ratio). Strengthening of the brick panels was shown to be proportional to the thickness of the shotcrete. Future tests will examine the out-of-plane behavior of 0.7m x 2m panels in order to determine better values for the interface shear resistance.

9. SEISMIC RESISTANCE OF MASONRY PIERS

This research project has the objective of improving the seismic resistance of masonry piers. The investigation consists of experimental and analytical studies. By improving behavior, we mean increase the ability of the pier to sustain deformation and absorb energy before it fails.

The experimental arrangement can be explained briefly. The pier is first subjected to a specified vertical loading using vertical actuators. Then horizontal displacements are imposed cyclically at the top of the pier by

means of horizontal actuators. The amplitude is increased monotonically until failure. Combinations of displacements and horizonal forces necessary to realize these displacements are recorded and result in hysteresis loops. The envelope of these loops acts as a measure of the pier's capacity to absorb energy. A recent improvement in the test procedure which prevents rotation at the top of a pier has been achieved by removing restraining rods and coupling the vertical actuator loads to the horizontal displacements.

The method used for increasing seismic resistance of piers depends on whether the pier will demonstrate a flexural or shear failure. Flexural failures will occur when the vertical load is relatively light, that is, at the upper stories of a building. This failure can be improved by reducing the vertical reinforcement and using toe plates to reduce toe crushing. Shear failure (typical x-cracking) occurs when the vertical load is large. Shear failure can be improved by the addition of horizontal reinforcement. The most important aspect of this failure mode is the anchorage of the horizontal bars. Anchor plates at the ends of the bars have dramatically increased the ductile behavior of the walls.

10. BASIC PROPERTIES AND STRENGTH OF CONNECTIONS FOR CONCRETE MASONRY BUILDINGS

The objectives of this project are the determination of the basic properties of concrete masonry units, prisms, and walls, using various mortars, and the strength of connections between walls and floors.

The basic properties of concrete masonry have been determined and published in several journals, such as the First North American Masonry Conference. The investigation of the strength of the conections is in its final stages and publication of the results will be available later in the year.

11. REPAIR OF MASONRY WALLS WITH EPOXIES

This project was recently awarded, therefore, the researcher is just starting his planning for the series of tests which will be performed subsequently. The objective of the investigation is to determine the feasibility of drilling a hole in a one or two story masonry wall, then inserting a reinforcing bar of pre-determined size, and grouting around the bar with standard grout, concrete mixture, and structural epoxy. A cost-benefit study will be made to determine the best method for the repair and/or strengthening of the wall to increase the seismic resistance. There are many masonry buildings of one and two stories in height in the United States which could be preserved by this technique if it is found to be effective, technically and economically.

12. SLENDERNESS RATIOS FOR MASONRY WALLS

The objective of this project, which is completed, was to determine the response of walls with different ratios of height-to-thickness (slenderness ratio) when subjected to eccentric axial forces acting in combination with lateral forces which simulated the condition of gravity loads on the walls with wind or seismic forces in the lateral direction.

A total of thirty full size panels were tested consisting of tilt-up concrete, concrete block, clay brick, and clay blocks. Slenderness ratios

of 30 to 60 were investigated by using a special test fixture constructed specially for this project.

A result for one of the walls is the following data. Concrete masonry wall 24 ft., 8 inches high, four feet wide, reinforced with five number four bars of grade 60 steel, block strength of 2600 psi, slenderness ratio of 38, subjected to a vertical load on the wall of 860 pounds per lineal foot and lateral load of 75 pounds per square foot at yield of steel, deflected 6.5 inches and attained a maximum deflection of 11.2 inches.

Similar data for the other tests have been evaluated and the results are being used to formulate building code requirements for consideration by code writing bodies.

It is interesting to note that this project was independently supported by the Structural Engineers Association of Southern California and the Southern California Chapter of the American Concrete Institute.

The National Bureau of Standards (NBS) (13) of the U.S. Department of Commerce has conducted several projects related to the strength, stability, and structural performance of masonry walls and has reported the results in the Structural Journal of the American Society of Civil Engineers and by several NBS publications of the Building Science Series.

Most of the projects noted above have been supported by the National Science Foundation as a part of its Earthquake Hazard Mitigation Program.

1. Proceedings of the First North American Masonry Conference, August 1978. Editor, J.L. Noland and J.E. Amrhein, University of Colorado, Boulder, CO 80309.

2. Proceedings of the Second North American Masonry Conference, August 1982. Editor, Donald W. Vannoy and James Colville, University of Maryland, College Park, MD 20742.

3. Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings. ABK-A Joint Venture, M.S. Agbabian, S.B. Barnes, John Kariotis, Project Director Robert Ewing, Agbabian Associates, 250 North Nash Street, El Segundo, CA 90245.

4. Basic Properties of Clay Unit Masonry, Atkinson-Noland and Associates, and University of Colorado, James L. Noland and Daniel P. Abrams, Address: Atkinson-Noland, 2619 Spruce Street, Boulder, CO 80302.

5. An Investigation into Non Destructive Evaluation of Masonry Structures. James L. Noland. Address: see (4).

6. Cyclic Response of Masonry Anchor Bolts, Russell H. Brown, Department of Civil Engineering, Clemson University, Clemson, South Carolina 29631.

7. Seismic Response of Composite Masonry in New and Existing Structures. Russell H. Brown and Subhash Anand, Clemson University - see (6) for address.

8. Strengthening of Brick Masonry Walls with Shotcrete. Lawrence F. Kahn, School of Engineering, Georgia Institute of Technology, Atlanta, GA 30332.

9. Seismic Resisance of Masonry Piers. Hugh D. McNiven, Director-Earthquake Engineering Research Center, 47th and Hoffman Boulevard, Richmond, CA 94804.

10. Basic Properties and Strength of Connections for Concrete Masonry Buildings. Gilbert A. Hegemier, University of California, San Diego, CA 92307.

11. Repair of Masonry Walls with Epoxies. Joseph M. Plecnik, Department of Civil Engineering, North Carolina State University, Raleigh, North Carolina 27607.

12. Slenderness Ratios for Masonry Walls-Structural Engineers Association of Southern California and ACI-Southern California Chapter. Contact: James E. Amrhein, Director of Engineering, Masonry Institute of America, 2550 Beverly Boulevard, Los Angeles, CA 90057.

13. National Bureau of Standards-Contact: Felix Y. Yokel, National Bureau of Standards, U.S. Department of Commerce, Washington, D.C. 20234.

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